



**Geotechnical Investigation,  
Proposed 7- to 8-Story Residential Development,  
24422 Avenida De La Carlota,  
Laguna Hills, California**

**Prepared for**

**BUCHANAN STREET PARTNERS, LP.**

June 29, 2023

GMU Project No. 23-008-00



June 29, 2023

Mr. Matthew Haugen  
**Buchanan Street Partners, LP.**  
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Newport Beach, CA 92660

PROJECT: 23-008-00

SUBJECT: Geotechnical Investigation, Proposed 7- to 8-Story Residential  
Development, 24422 Avenida De La Carlota, Laguna Hills, California.

Dear Mr. Haugen:

GMU is pleased to present this geotechnical report for the subject project, which summarizes our subsurface exploration, accumulated data, conclusions, and recommendations.

Please note that this report has not been prepared for the use by other parties or projects other than those named or described herein. This report may not contain sufficient information for other parties or other purposes.

We appreciate the opportunity to work on this project. Please do not hesitate to contact the undersigned if you have any questions regarding any aspect of this report.

Respectfully submitted,

A handwritten signature in green ink, appearing to read 'D. Hansen', is written over a light blue horizontal line.

David Hansen, M.Sc., PE, GE 3056  
Associate Geotechnical Engineer

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Addressee: (electronic copy)

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## **INTRODUCTION**

### **PURPOSE**

This report presents the results of our geotechnical investigation for the 7- to 8-story residential building proposed to the north of the intersection of Avenida De La Carlota and Los Alisos Boulevard at 24422 Avenida De La Carlota within the City of Laguna Hills, California. The purpose of this report is to provide a review of the preliminary architectural plans and grading plans, summarize the results of our geotechnical investigation, and then provide geotechnical recommendations pertaining to site grading and remedial grading and for the design and construction of the proposed residential structure and other associated site improvements. The scope of our services to prepare this report is outlined in the following section.

### **SCOPE**

1. Reviewed the reference (1) architectural plans and reference (2) conceptual grading plan for the subject site.
2. Reviewed background information pertaining to the site including published and unpublished regional geologic maps and literature and seismic hazard maps and reports.
3. Performed a site visit to plot and mark the locations of five exploratory drill holes. Coordinated with Underground Services Alert (USA) to determine the location of existing utilities and provide advance notification of subsurface drilling locations. Coordinated with client and subconsultants to set up the site exploration.
4. Conducted a subsurface exploration program that consisted of the advancement of five (5) exploratory drill holes using a hollow-stem auger drill rig to depths of 31.5 to 51 feet below the existing ground surfaces to determine site-specific subsurface geologic and groundwater or seepage conditions. The drill holes were logged by a geologist and soil and bedrock samples were collected for laboratory testing.
5. Performed laboratory testing on bulk and undisturbed samples of soil and bedrock that were collected during our subsurface exploration. Laboratory testing included in-place moisture content and dry density, maximum density and optimum moisture content, particle-size analysis (soil gradation), Atterberg Limits, expansion index, determination of consolidation and shear strength characteristics, and chemical analysis to determine corrosion potential.

6. Interpreted and evaluated field conditions and laboratory data.
7. Performed geotechnical engineering analyses using the field and laboratory data in conjunction with the conceptual grading plans. The analysis addressed site seismicity, foundation design and anticipated settlement, and exterior flatwork and pavement design.
8. Prepared this report which summarizes the results of our research, subsurface exploration, field and laboratory testing, analyses, and conclusions, and provides recommendations regarding:
  - Site preparation, remedial excavation, and precise grading requirements.
  - Acceptability of the site soils for use as fill and backfill.
  - Site seismicity and seismic design parameters.
  - Foundation design, including types, depths, and bearing values.
  - Anticipated settlement of the structure.
  - Lateral earth pressures for retaining/basement wall design.
  - Temporary slope requirements.
  - Design parameters for exterior improvements.
  - Corrosion potential of onsite soil and bedrock materials.
  - Exterior concrete pavement design.
  - Installation of underground utilities.

## **SITE LOCATION AND DESCRIPTION**

The subject site currently consists of a paved parking lot that is located north of the intersection of Avenida De La Carlota and Los Alisos Boulevard within the City of Laguna Hills, California. The general location of the subject site with respect to nearby roadways is shown on Plate 1 – Location Map.

The site has been previously graded into a relatively flat parking lot that slopes towards the west at a sheet flow gradient of approximately 2 percent. Elevations within the parking lot range from approximately 372 to 380 feet above mean sea level. The parking lot is bordered on the north by an existing office building, on the southwest by an approximately 5- to 8-foot-high slope that descends to Avenida De La Carlota at a slope ratio of 2:1, horizontal to vertical, and on the southeast by an approximately 2- to 12-foot-high slope that both descends and ascends to Los Alisos Boulevard at a slope ratio of 2:1, horizontal to vertical.

The parking lot has occasional medians or planter areas that contain groundcover, shrubs and trees. The adjacent slopes to the west and southeast are also covered by groundcover, shrubs and trees and do not show any signs of erosion or slumping.

Other improvements within the site include concrete walkways and patios along the front of the existing office building, concrete curbs around the perimeter of the parking lot and around the medians and planter areas, and concrete swales or ribbon gutters throughout the parking lot. It is also expected that underground utility lines and irrigation lines run through the parking lot.

Just outside of the property lines along the southwest and southeast sides of the site, there are concrete sidewalks adjacent to Avenida De La Carlota and Los Alisos Boulevard along with underground utility line vault boxes and stub-outs.

## **PROPOSED IMPROVEMENTS**

Based on the reference (1) architectural plans and reference (2) conceptual grading plans, it is proposed to construct a 7- to 8-story residential building that will consist of four levels of residential units (Levels 4 through 7) underlain by 3 levels of above-ground parking (Parking Levels P1 through P3) within the subject site. The southwest side of the building will be underlain by a subterranean level of parking (Parking Level B1). The parking levels will also have maintenance and storage rooms, a leasing office, and trash rooms. Level 4 will include a podium with a pool and courtyard. Levels 5 through 7 will have an interior area that is open to the pool and courtyard below.

Retaining walls on the order of 10 feet in height will be required around the perimeter of the underground parking level. It is expected that the structure will be of combined concrete, steel, and wood construction. Structural loads are not available at this time but are expected to be on the order of 300 to 400 kips for column loads and 6 to 15 kips/ft. for wall loads. Once the actual building loads are known, we should be provided with this information so that we may provide revised or alternative recommendations, if necessary.

Exterior improvements will be limited to re-aligning the existing paved driveways and parking stalls along the north and northeast sides of the new building. In addition, new hardscape features (concrete walkways and patios) will be constructed around the perimeter of the new building along with new landscape and planter areas.

According to the conceptual grading plans, the subterranean parking level will have a finish floor elevation of 364.5 feet above mean sea level while parking level P1 will have a finish floor elevation of 374.5 feet above mean sea level. These two parking levels will underly the entire structure. Based on these proposed grades, cuts of approximately 8 to 12 feet will be required to reach proposed grades within the subterranean parking area while fills of only 1 to 2 feet and cuts of up to 5 feet will be required within the parking level P1 area and the adjacent landscape and hardscape areas. Existing ground elevations and proposed grades are shown on the enclosed Geologic Map - Plate 2, which uses the conceptual grading plan as a base map, and on the enclosed Geologic Sections 1-1' through 3-3' - Plates 3.0 through 3.2.

The proposed cuts within the site can be made by laying back the sidewalls of the excavation in accordance with the recommendations of this report; therefore, shoring will not be required.

## **SUBSURFACE EXPLORATION**

Our subsurface exploration consisted of the advancement of five exploratory drill holes (DH-1 through DH-5) to depths of 31.5 to 51 feet below the existing ground surfaces using a hollow-stem auger drill rig to confirm existing subsurface geologic and groundwater or seepage conditions. The drill holes were logged by a geologist and soil and bedrock samples were collected for laboratory testing. The approximate locations of the drill holes are shown on the attached Geotechnical Map, Plate 2, which uses the conceptual grading plan as a base map. The logs of the drill holes are presented in Appendix A.

## **LABORATORY TESTING**

Laboratory testing for the subject investigation was performed to determine soil and bedrock engineering classifications and properties. Laboratory testing included the following: in-place moisture and dry density, maximum dry density and optimum moisture content, particle size analysis, compaction testing, expansion index, direct shear, consolidation, R-value, and chemical analysis of corrosion potential. Laboratory procedures and test results are presented in our Appendix B – GMU Geotechnical Laboratory Procedures and Test Results. Pertinent laboratory test data is also shown in our drill hole logs.

## **GEOLOGIC FINDINGS**

### **REGIONAL GEOLOGIC SETTING**

The site is located at the southeastern edge of the Tustin Plain section of the Peninsular Range Province where the plain meets the rolling hills of the San Joaquin Hills. Geologic units within the area of the subject property are composed primarily of Holocene and Pleistocene-age slopewash/colluvial materials, non-marine terrace deposits, and bedrock materials of the Niguel Formation.

### **LOCAL GEOLOGY AND SUBSURFACE SOIL CONDITIONS**

Based on our subsurface exploration, the subject site is underlain by approximately 4 to 12 feet of compacted fill that overlies both native slopewash/colluvial materials and bedrock materials of the Niguel Formation. The non-marine terrace deposits were not encountered. The fill materials

within the northwesterly and northeasterly portions of the site are underlain directly by the Niguel Formation bedrock while the fill materials within the southern portion of the site are underlain by approximately 6.5 to 10 feet of in-place native slopewash/colluvial materials and then the bedrock materials. The existing fill depths and geologic units within the site are shown in plan view on the attached Geotechnical Map, Plate 2 while the subsurface geologic conditions are shown in profile on Plates 3.0 through 3.2 – Sections 1-1' through 3-3'. The map and sections were compiled using the geologic information from our recent subsurface exploration.

Detailed descriptions of the soil and bedrock materials within the site are described in the following sections.

### **Artificial Fill (Qaf)**

Artificial fill materials were encountered to depths of 4 to 12 feet below the existing ground surfaces within our drill holes DH-1 through DH-5. These fill materials were likely placed during the previous grading to create the existing parking lot and the building pad of the adjacent office building. These fill materials were likely derived from the both the native slopewash/colluvial materials and from the bedrock materials and consist of interlayered sandy clays (CL), sandy to clayey silts (ML), and clayey sands (SC) that were observed to be olive to olive gray to brown, damp to moist, and firm to stiff or medium dense with occasional minor gravel.

### **Slopewash/Colluvial Materials (Qsw/Qcol)**

Slopewash/Colluvial materials were encountered below the artificial fill materials within drill holes DH-2, DH-4 and DH-5 at depths of 5 to 12 feet below the existing ground surfaces. These materials consist of fine grained, sandy silts (ML) and sandy to silty clays (CL) that were observed to be damp to moist and firm but with some organics and porosity.

### **Niguel Formation Bedrock Materials (Tn)**

Bedrock of the Niguel Formation lies below the fill materials within the northwest and northeast portions of the site and below the colluvial materials within the south portion of the site. The Niguel Formation bedrock was observed to consist of a damp to moist to very moist, moderately hard to hard, fine to medium grained sandy siltstones and clayey siltstones.

### **Local Geologic Structure**

The bedrock materials of the Niguel Formation are thickly bedded to massive and without any discernible bedding or planes of weakness.

## **GROUNDWATER AND SEEPAGE**

No static groundwater was encountered within our drill holes at least to the maximum depth explored (51 feet); however, some seepage was encountered within drill holes DH-3, DH-4 and DH-5 at depths of 30 to 31 feet below the existing ground surface.

Note that the above groundwater and seepage observations reflect the conditions present during our recent subsurface exploration but are not intended to be a representation or guarantee of groundwater conditions which may exist or occur in the future. Future groundwater conditions are dependent on numerous other factors such as rainfall, irrigation, drainage, utilities, owner improvements, maintenance, etc.

Groundwater or seepage may occur within the site in the future in two general forms:

1. Perched groundwater or seepage along the contact between the fill and slopewash/colluvial materials or along the contact between the slopewash/colluvial materials and the underlying bedrock.
2. Seepage within isolated zones of bedrock, typically near stratigraphic contacts (i.e., sandy siltstone vs. clayey siltstone) or at structural discontinuities (i.e., faults or shears).

## **FAULTING AND SEISMICITY**

The site is not located within an official Alquist-Priolo Earthquake Fault Zone, and no known active faults are shown on the reviewed geologic maps crossing the site. The nearest known active fault is the San Joaquin Hills Blind Thrust (SJHBT) fault, which is located approximately 0.5 miles from the site and capable of generating a maximum earthquake magnitude ( $M_w$ ) of 7.1. The site is also located within approximately 9 miles of the Newport Inglewood (N-I) fault, which is capable of generating a maximum earthquake magnitude ( $M_w$ ) of 7.5. Given the proximity of the site to these and numerous other active and potentially active faults, the site will likely be subject to earthquake ground motions in the future.

## **SEISMIC HAZARD ZONES**

Through the Seismic Hazards Mapping Act, the California Geological Survey (formerly the California Division of Mines and Geology) has established Seismic Hazard Zones for most of the densely populated areas of southern and northern California. According to the Seismic Hazard Zone map for the Dana Point 7.5-minute quadrangle (CDMG, 2001), the site is not situated within an area that has been designated as being susceptible to either earthquake-induced liquefaction or landsliding.

The site is underlain by approximately 4 to 12 feet of cohesive artificial fill materials and then by firm cohesive slopewash/colluvial materials and moderately hard to hard bedrock materials. In addition, no static groundwater was encountered below the site during our recent subsurface

exploration. Based on these existing conditions, we concur with the Seismic Hazard Zones map that the subject site is not susceptible to either earthquake-induced liquefaction or landsliding. In addition, hazard from other secondary seismic effects, including tsunamis, are considered to be nil based on the distance of the site from the oceans.

## **GEOTECHNICAL ENGINEERING FINDINGS**

### **SOIL AND BEDROCK EXPANSION**

Based on the results of our laboratory testing, the existing onsite fill materials and native slopewash/colluvial materials consist predominantly of sandy silts and sandy clays that have a low to moderately expansion potential while the underlying bedrock materials consist of non-expansive to slightly expansive sandy siltstones and clayey siltstones.

### **BEARING MATERIALS AND POTENTIAL DIFFERENTIAL MOVEMENT**

Proposed design grades will result in the northwestern foundations and slabs of the structure bearing directly on moderately hard to hard bedrock materials; however, the remainder of the structure will be underlain by the existing fill and native slopewash/colluvial materials. As a result of this transition, there is the potential for differential settlement of the underlying differing materials under the heavy loads of the structure. Therefore, those portions of the hotel structure underlain directly by bedrock may be designed with conventional footings founded into bedrock while those portions of the hotel structure underlain by the existing fill and native slopewash/colluvial materials should be supported on caissons that extend through the fill and native slopewash/colluvial materials and into the underlying bedrock. To provide proper support of the lowermost slabs that will be constructed on-grade, remedial grading will be required as described in the “Site Preparation and Grading” section of this report.

### **SOIL CORROSION**

To evaluate the corrosion potential of the on-site fill, native slopewash/colluvial materials and bedrock materials to both ferrous metals and concrete, representative samples were tested for pH, minimum resistivity, soluble chlorides, and soluble sulfates. The results of the chemical testing are listed in Table B-1 of Appendix B and indicate that the on-site soil and bedrock materials should be considered to have:

- low to very low minimum resistivity (corrosive to severely corrosive to ferrous metals).
- A negligible sulfate exposure to concrete per the ACI 318 Table 4.3.1 (non-corrosive to concrete).
- Elevated chloride contents (corrosive to ferrous metals).

## **SOIL INFILTRATION**

Based on the potential for any future subsurface water to become perched atop the underlying moderately hard to hard and relatively impermeable bedrock materials, infiltration of storm water into the onsite soils is not recommended from a geotechnical point of view.

## **EXCAVATION CHARACTERISTICS**

### **Rippability**

It is expected that the soil and bedrock materials underlying the site can be excavated with conventional grading equipment such as dozers, loaders, excavators, and backhoes.

### **Trenching**

We expect that excavation of new utility trenches within new fill materials or existing soil can be accomplished utilizing conventional trenching machines and backhoes. Excavation of utility trenches in bedrock materials may require excavators. Trench support requirements will be limited to those required by safety laws or other locations where trench slopes will need to be flattened or supported by shoring designed to suit the specific conditions exposed.

### **Volume Change**

To aid planning for the anticipated grading, we estimate that the change in volume of the on-site fill and native slopewash/colluvial materials that are excavated and replaced as new compacted fill at an average relative compaction of 92% will result in an average of about 2% to 5% loss in volume. The onsite bedrock materials are expected to undergo about 2% gain in volume. It should be noted that the aforementioned values are approximate and are for rough planning purposes only.

## **CONCLUSIONS AND RECOMMENDATIONS**

Based on the geologic and geotechnical findings, it is our opinion that the proposed grading and construction is feasible and practical from a geotechnical standpoint if accomplished in accordance with the City of Laguna Hills grading and building requirements and the recommendations presented herein. It is also the opinion of GMU Geotechnical that proposed grading and construction will not adversely affect the geologic stability of adjoining properties provided grading and construction are performed in accordance with the recommendations provided in this report. A summary of conclusions is as follows:

1. The site should be considered developable and is not expected to adversely impact adjacent properties from a geotechnical perspective utilizing standard grading techniques.
2. In general, the onsite earth materials consist of 4 to 12 feet of existing artificial fill materials overlying 6.5 to 10 feet of soft to firm slopewash/colluvial materials and then moderately hard to hard bedrock materials of the Niguel Formation.
3. Static groundwater was not encountered to the maximum depth explored (51 feet below the existing ground surfaces); however, some seepage was encountered at a depth of 30 to 31 feet below the existing ground surface within drill holes DH-3 through DH-5.
4. The potential for liquefaction is considered non-existent.
5. There are no known active faults within the subject site. The site seismicity is typical for the Laguna Hills area. Structure design should be in accordance with the current CBC.
6. Based on visual observations and laboratory testing of the on-site soil and bedrock materials, minor corrective grading will be required below the proposed structure.
7. Corrective grading consisting of the removal and re-compaction of the existing unsuitable surficial soils will be required for any exterior improvements proposed on-grade and beyond the removals performed for the parking/ residential structure.
8. The parking/ residential structure will need to be supported on a combination of conventional footings and caissons founded into bedrock materials.
9. The onsite fill materials and native slopewash/colluvial materials consist of predominantly of sandy silts and sandy clays that are slightly to moderately expansive while the underlying bedrock materials consist of non-expansive to slightly expansive sandy and clayey siltstones. Slabs constructed on-grade will need to be designed for expansive soil conditions.
10. The testing of onsite soils indicates negligible levels of sulfates; however, they have elevated levels of chlorides. Therefore, Type II/V cement with a water/cement ratio of 0.50 and minimum compressive strength of 4,000 psi is recommended for proposed building foundations and floor slabs and other onsite structures from a geotechnical perspective.
11. Corrosion testing indicates that the on-site soils are severely corrosive to ferrous metals and concrete reinforcement. Consequently, any metal exposed to the soil will need protection, and concrete reinforcement should have adequate (3 to 4 inches) concrete cover wherever concrete is in contact with soil.
12. Infiltration of storm water into the onsite soil is not recommended.

## **SITE PREPARATION AND GRADING**

### **General**

The subject site should be precise graded in accordance with the requirements of the City of Laguna Hills grading manual (and all other applicable codes and ordinances) and the recommendations as outlined in the following sections of this report. The geotechnical aspects of future grading plans and improvement plans should be reviewed by GMU Geotechnical prior to grading and construction. Particular care should be taken to confirm that all project plans conform to the recommendations provided in this report. All planned and corrective grading should also be monitored by GMU Geotechnical to verify general compliance with the recommendations outlined in this report.

### **Demolition and Clearing**

All significant organic materials such as groundcover, shrubs, roots, construction debris, or other decomposable materials should be removed from areas to be graded.

The on-site soil materials are suitable for use as compacted fill from a geotechnical perspective if care is taken to remove all significant organic materials and other decomposable debris and any oversize inert material.

Cavities and excavations created upon removal of subsurface obstructions, such as existing buried utilities, should be cleared of loose soil, shaped to provide access for backfilling and compaction equipment, and then backfilled with properly compacted fill.

GMU Geotechnical (GMU) should provide periodic observation and testing services during demolition operations to document compliance with the above recommendations. In addition, should unusual or adverse soil conditions or buried structures be encountered during grading that are not described herein, these conditions should be brought to the immediate attention of the project geotechnical consultant for corrective recommendations.

### **Corrective Grading – Residential/ Parking Structure Garage Slabs**

Proposed design grades will result in the northwestern foundations and garage slabs of the structure being underlain directly by moderately hard to hard and relatively non-expansive bedrock materials, while the remainder of the structure and its garage slabs will be underlain by the existing fill and native slopewash/colluvial materials that are expansive and subject to compression. Therefore, to provide more uniform bearing and expansion conditions for the proposed lowermost parking garage slabs, the building pad should be over-excavated to a depth of at least 3 feet below proposed pad finish grades and the excavated materials then thoroughly blended, moisture conditioned, and replaced as properly compacted fill.

Those portions of the proposed residential/ parking structure that will be underlain by existing fill or in-place native slopewash/colluvial materials will be supported on caissons as recommended in a subsequent section of this report.

### **Corrective Grading – Exterior Hardscape Structure and Pavement Areas**

The existing surficial artificial fill materials up to depths of 18 to 24 inches have become desiccated and porous due to weathering and root growth or will be disturbed during proposed clearing and demolition of the site. Therefore, the exterior hardscape structure and pavement areas around the new building should be over-excavated to a minimum depth of at least 24 inches below proposed subgrades or bottoms of footings.

Before replacing excavated materials as properly compacted fill, the exposed bottom surfaces should be:

- Scarified to a depth of 6 inches.
- Moisture conditioned (as necessary) to at least 2 percentage points above the optimum moisture content (i.e., if the optimum moisture content is 12%, the compacted fill's moisture content shall be at least 14%).
- Recompact in-place to at least 90% relative compaction per ASTM Test Method D 1557.

Within shallow cut areas where some of the unsuitable surficial soils will already be removed to reach proposed grades, the depths of over-excavation may be reduced accordingly. Within deeper cut areas where all the unsuitable surficial soils are removed, only scarification, moisture conditioning and re-compaction will be required as described above.

To provide proper support of all exterior improvements, the recommended over-excavation and re-compaction should extend at least 2 feet beyond the perimeter edges of new improvements; however, consideration should be given to the protection of existing structures or improvements to be protected in-place. Along the sides of the site, the tops of the over-excavation sidewalls should be kept at least 12 inches away from any existing property line structures to be protected in-place and should extend down into the bottom of the excavation at a 1:1 slope ratio.

## **FILL MATERIAL AND PLACEMENT**

### **Suitability**

All on-site soil and bedrock materials are considered suitable for use as general compacted fill from a geotechnical perspective if care is taken to remove all significant organic and other

decomposable debris, and to remove and stockpile any hard rock fragments or cobbles larger than 6 inches in maximum diameter.

### **Compaction Standard and Methodology**

All soil material used as compacted fill, processed in-place, or used to backfill walls and trenches, should be:

- Moistened, dried, or blended as necessary to a minimum of 2 percentage points over the optimum moisture content.
- Compacted to at least 90% relative compaction as determined by ASTM Test Method D 1557.

### **Material Blending**

The existing soil and bedrock materials are expected to have variable moisture contents and expansion potentials. Therefore, the soil and bedrock materials to be handled during grading will require thorough blending to become a more homogeneous fill material and either drying out or addition of water to meet acceptable moisture ranges for sufficient compaction (i.e., minimum 2 percentage points over the optimum moisture content).

### **Use of Oversize Rock**

Although not expected, any oversize rock materials or cobbles generated during grading should be collected and hauled off-site unless they can be broken down to 6 inches in diameter or less.

## **TEMPORARY EXCAVATION STABILITY**

During site grading, temporary excavations up to approximately 10 feet in height are expected to be created during construction of the proposed underground parking structure and its subterranean walls. Trench excavations will also be required for underground utility lines.

As shown on Plate 2 – Geotechnical Map and Plates 3.0 through 3.2 – Geotechnical Sections 1-1' through 3-3', the excavation sidewalls are typically expected to expose firm artificial fill and native slopewash/colluvial materials and also thickly bedded to massive and moderately hard siltstone bedrock materials without any discernible bedding or planes of weakness.

Based on the anticipated engineering characteristics of the onsite soil and bedrock materials, OSHA Type B soil characteristics may be assumed.

From a geotechnical standpoint, temporary excavations within the site may be cut vertical to a height of 4 feet and then those portions of the excavation sidewalls above a height of 4 feet should be cut back at a maximum slope ratio of 1:1, horizontal to vertical.

These temporary excavation recommendations are provided only as general guidelines and all work associated with temporary excavations should also meet the minimal requirements as set forth by CAL-OSHA for Type B soils. Temporary slope and trench excavation construction, maintenance, and safety are the responsibility of the contractor. Other factors that should be considered with respect to the stability of temporary slopes include construction traffic and storage of materials on or near the tops of the slopes, construction scheduling, presence of nearby walls or structures, and weather conditions at the time of construction.

Based on the reference (1) conceptual grading plan, there is room within the site to lay back the temporary excavation sidewalls at the above recommended slope configuration without either encroaching into the adjacent properties or undermining existing adjacent structures.

## **UTILITY TRENCH BACKFILL CONSIDERATIONS**

### **General**

New utility line pipeline trenches (greater than 2-feet deep) should be backfilled with both select bedding materials beneath and around the pipes (pipe zone) and compacted soil above the pipe bedding. Recommendations for the material types to be used and their proper placement are provided in the following sections.

### **Pipe Zone (Bedding and Shading)**

The pipe bedding materials should extend to at least 12 inches above the crown of pipes that are 8 inches in diameter or greater. Pipes less than 8 inches in diameter should be covered with at least 6 inches of pipe bedding materials. Pipe bedding should consist of either clean sand with a sand equivalent (SE) of at least 20 or crushed rock. If crushed rock is used, it should consist of ¾-inch crushed rock that conforms to current “Greenbook” standards. Additionally, if crushed rock is used a separation fabric such as Mirafi 140N, or equivalent, should be placed over the top of the crushed rock zone prior to placing fill soils. Pipe bedding should also meet the minimum requirements of the City of Laguna Hills. If the requirements of the City are more stringent, they should take precedence over the geotechnical recommendations. Sufficient laboratory testing should be performed to verify the bedding meets the minimum requirements of the Greenbook and County grading codes.

Based on our subsurface exploration and knowledge of the onsite materials, the soils that will be excavated from the pipeline trenches will not meet the recommendations for pipe bedding materials; therefore, imported materials will be required for pipe bedding.

Granular pipe bedding material having a sand equivalent of 20 or greater should be properly placed in thicknesses not exceeding 3 feet or one-foot over the top of pipe, whichever is less, and then sufficiently jetted in place. The top of the jetted sand should be densified with hand operated compaction equipment to the satisfaction of the geotechnical consultants representative prior to placing the trench backfill. With proper techniques, jetting is not expected to have an adverse impact on existing site soils.

### **Trench Backfill**

All existing soil material within the limits of the pipeline alignment are considered suitable for use as trench backfill above the pipe bedding zone if care is taken to remove all significant organic and other decomposable debris, moisture condition the soil materials as necessary, and separate and selectively place and/or stockpile any inert materials larger than 6 inches in maximum diameter.

Imported soils are not anticipated for backfill since the on-site soils are suitable. However, if imported soils are used, the soils should consist of clean, granular materials with physical and chemical characteristics similar to those described herein for on-site soils. Any imported soils to be used as backfill should be evaluated and approved by GMU prior to placement.

Soils to be used as trench backfill should be moistened, dried, or blended as necessary to achieve a minimum of 2 percentage points over optimum moisture content, placed in loose lifts no greater than 8 inches thick, and mechanically compacted/densified to at least 90% relative compaction as determined by ASTM Test Method D 1557. Jetting is not permitted in the trench zone.

### **SURFACE DRAINAGE**

Design of surface drainage is outside GMU's purview and should be designed and confirmed by the project civil engineer. Surface drainage should be carefully controlled to prevent runoff over graded slope surfaces and ponding of water on flat pad areas. Positive drainage away from graded slopes and pad areas is essential to reduce the potential for erosion or saturation. Maintaining positive drainage of all landscaping areas along with avoiding over-irrigation will help minimize the possibility of "perched" groundwater accumulating slightly below the graded surfaces. Surface drainage should be designed in accordance with Section 1804.4 of the 2022 CBC and Section R401.3 of the 2022 CRC.

## **SOIL MOISTURE CONTROL, IRRIGATION, AND LANDSCAPING**

The on-site soils are subject to volume change (both expansion and contraction) in response to changes in moisture. Future planting, irrigation, landscaping, and maintenance should therefore strive to maintain a uniform soil moisture content that is similar to the moisture content at which the fills were placed. Over-irrigation should be avoided; furthermore, the fills should not be allowed to become excessively dry or saturated.

Planter areas placed adjacent to building foundations are not recommended. If planter areas are proposed up against building foundations, irrigation should be carefully controlled. A watering program that maintains a uniform, near optimum moisture condition in the soils should be implemented for the landscape areas.

Overwatering and subsequent saturation of the soils will cause excessive soil expansion and heave and, therefore, should be avoided. On the other hand, allowing the soils to dry out will cause excessive soil shrinkage.

As an alternative to a conventional irrigation system, drip irrigation that maintains constant moisture conditions is strongly recommended for all planter areas. The owner is advised that all drainage devices should be properly maintained throughout the lifetime of the development.

Plants known to have excessive root systems should also be avoided near structural improvements as they can cause heave conditions. Conversely, the root systems can also dry out the soils and cause excessive soil shrinkage below adjacent footings or slabs. Drought-resistant and maintenance-free plant species are recommended.

## **FOUNDATION DESIGN RECOMMENDATIONS**

### **Structure Seismic Design**

The average shear wave velocity for the upper 100 feet of subsurface soils ( $V_{s30}$ ) within the subject site is estimated to be between 1200 and 2500 feet per second which corresponds to a “very dense soil and soft rock” soil profile. Therefore, based on this soil profile, the site should be designated as Site Class C. The seismic design coefficients based on ASCE 7-16 and 2022 CBC are listed in the following table.

**2022 CBC and ASCE 7-16 Seismic Design Parameters**  
**(To be utilized as per the requirements of Section 11.4.8 of ASCE 7-16)**

<b>Seismic Item</b>	<b>Design Value</b>	<b>ASCE 7-16 or 2022 CBC Reference</b>
Site Class based on soil profile (ASCE 7-16 Table 20.3-1)	D <sup>(a)</sup>	ASCE 7-16 Table 20.3-1
Short Period Spectral Acceleration $S_s$	1.203 <sup>(a)</sup>	CBC Figures 1613.2.1 (1-10)
1-sec. Period Spectral Acceleration $S_1$	0.434 <sup>(a)</sup>	CBC Figures 1613.2.1 (1-10)
Site Coefficient $F_a$ (2019 CBC Table 1613.2.3(1))	1.200 <sup>(a)</sup>	CBC Table 1613.2.3 (1)
Site Coefficient $F_v$ (2019 CBC Table 1613.2.3(2))	1.500 <sup>(a)</sup>	CBC Table 1613.2.3 (2)
Short Period MCE* Spectral Acceleration $S_{MS}$ $S_{MS} = F_a S_s$	1.443 <sup>(b)</sup>	CBC Equation 16-20
1-sec. Period MCE Spectral Acceleration $S_{M1}$ $S_{M1} = F_v S_1$	0.652 <sup>(b)</sup>	CBC Equation 16-21
Short Period Design Spectral Acceleration $S_{DS}$ $S_{DS} = 2/3 S_{MS}$	0.962 <sup>(b)</sup>	CBC Equation 16-22
1-sec. Period Design Spectral Acceleration $S_{D1}$ $S_{D1} = 2/3 S_{M1}$	0.434 <sup>(b)</sup>	CBC Equation 16-23
Short Period Transition Period $T_S$ (sec) $T_S = S_{D1}/S_{DS}$	0.451 <sup>(b)</sup>	ASCE 7-16 Section 11.4.6
Long Period Transition Period $T_L$ (sec)	8 <sup>(b)</sup>	ASCE 7-16 Figures 22-14 to 22-17
MCE <sup>(c)</sup> Peak Ground Acceleration (PGA)	0.505 <sup>(a)</sup>	ASCE 7-16 Figures 22-9 to 22-13
Site Coefficient $F_{PGA}$ (ASCE 7-16 Table 11.8-1)	1.200 <sup>(a)</sup>	ASCE 7-16 Table 11.8-1
Modified MCE <sup>(c)</sup> Peak Ground Acceleration ( $PGA_M$ )	0.605 <sup>(a)</sup>	ASCE 7-16 Equation 11.8-1
Seismic Design Category	D <sup>(b)</sup>	ASCE 7-16 Tables 11.6.1 and 11.6.2

(a) Design Values Obtained from USGS Earthquake Hazards Program website that are based on the ASCE-7-16 and 2022 CBC and site coordinates of N33.60733° and W117.69912°.

(b) Design Values Determined per CBC Equations 16-20 through 16-23 and ASCE Tables 11.4-2 and 11.6.1-2.

(c) MCE: Maximum Considered Earthquake.

Since the Site Class is designated as D and the  $S_1$  value is greater than or equal to 0.2, the 2022 CBC requires either a site-specific ground motion hazard analysis per Section 21.2 of ASCE 7-16 or the application of Exception 2 of Section 11.4.8 of ASCE 7-16. Exception 2 states that a site specific ground motion hazard analysis is not required provided that the value of the seismic response coefficient,  $C_s$ , is conservatively calculated by the project structural engineer using Eqn. 12.8-2 of ASCE 7-16 for values of  $T \leq 1.5T_s$  and taken as equal to 1.5 times the value computed in accordance with either Eqn. 12.8-3 for  $T_L \geq T > 1.5T_s$  or Eqn. 12.8-4 for  $T > T_L$ .

Per the 2022 CBC and ASCE 7-16, the Design Earthquake peak ground acceleration ( $PGA_D$ ) may be assumed to be equivalent to  $S_{DS}/2.5$ ; therefore, for the subject site, a  $PGA_D$  value of 0.38g (0.962g/2.5) should be used.

It should be recognized that much of southern California is subject to some level of damaging ground shaking due to movement along the major active (and potentially active) fault zones that characterize this region. Design utilizing the 2022 CBC is not meant to completely protect against damage or loss of function. Therefore, the preceding parameters should be considered as minimum design criteria.

## **Foundation Design Considerations**

As described previously, proposed design grades will result in the northwestern foundations and slabs of the structure bearing directly on moderately hard to hard bedrock materials; however, the remainder of the structure will be underlain by the existing fill and native slopewash/colluvial materials with a combined depth of up to approximately 15 feet. To mitigate the potential for adverse total and differential settlement, those portions of the building structure underlain directly by bedrock may be designed with conventional shallow footings founded into bedrock while those portions of the structure underlain by fill and/or in-place slopewash/colluvial materials should be supported on caissons that extend through the fill and in-place slopewash/colluvial materials and into the underlying bedrock.

The methods used in the design and construction of the slab foundation system should conform to all applicable and current codes, ordinances, and standards. The allowable limits selected for foundation deflection due to any differential soil movements should be coordinated with the architect and structural engineer responsible for the design of the structure framing and roof systems. They should confirm that such deflection will not cause excessive distress to those systems or to interior and exterior walls and ceilings of the planned structures.

As previous mentioned actual structural loads are not available at this time. The foundation design parameters outlined below are based on expected structural loads on the order of 300 to 400 kips for column loads and 6 to 15 kips/ft. for wall loads. Once actual building loads are forwarded to GMU, these parameters may need to be revised or alternative recommendations may need to be provided.

## **Bedrock Parameters**

Bearing Material:	Bedrock
Bearing Value:	3000 psf, based on a minimum 12 inch deep by 12 inch wide footing: (see subsequent sections for actual minimum recommended footing embedment and dimensions.) <ul style="list-style-type: none"><li>• May be increased 10% for each additional foot of footing width and by 20% for each additional foot of footing depth into bedrock to a maximum of 6000 psf).</li><li>• One-third increase for wind or seismic loading.</li></ul>
Coefficient of Friction:	0.35 <ul style="list-style-type: none"><li>• One-third increase for wind or seismic loading.</li></ul>

Passive Resistance: 375 psf/ft of depth to a maximum value of 3,500 psf.

- Disregard upper 6 inches
- One-third increase for wind or seismic loading.

Modulus of Subgrade Reaction: 125 pci

### **Minimum Conventional Foundation Design Recommendations**

The following design parameters are considered applicable for design and construction of conventional shallow foundations.

Footings Depths: Perimeter Footings Constructed On-Grade: Minimum 24-inch embedment below lowest adjacent final grade or at least 2 feet into competent bedrock materials, whichever is greater.

Interior Footings: Minimum 18 inches below bottoms of adjacent grade or at least 2 feet into bedrock, whichever is greater.

Footings Reinforcement: Minimum four No. 4 bars, two top and two bottom, but final reinforcement to be determined by structural engineer.

### **Caisson Recommendations**

Caissons may be used if designed in accordance with the following design parameters:

Foundation Material: Bedrock

Skin Friction: 300 psf for a minimum embedment of 5 feet into bedrock.

- One-third increase for wind or seismic loading.

End Bearing: 6000 psf for a minimum 5 feet embedment into bedrock.

- May combined with skin friction without reduction.
- One-third increase for wind or seismic loading.

Passive Resistance 450 psf/per foot of caisson depth.

- Can be applied over 2 pile diameters provided piles are spaced at least 3 pile diameters, center to center (e.g. 900 psf/ft of pile diameter per foot of depth).
- One-third increase for wind or seismic loading.

Minimum Caisson Diameter:	24 inches.
Minimum Caisson Depth:	To a minimum depth of at least 5 feet into competent bedrock.
Minimum Grade Beam Depth:	18 inches
Minimum Beam Reinforcement:	Two #4's both top and bottom.

### **Parking Garage Slabs**

The garage slabs to be constructed on-grade should be designed by the structural engineer using the following minimum design recommendations.

Slab Thickness:	Minimum 6-inch-thick slabs.
Slab Reinforcement:	Minimum No. 4 bars at 18 inches on center, both ways.
Slab Subsection:	4 inches of ¾-inch graded rock to act as a capillary break.
Slab Subgrade Moisture Content:	2 percentage points over optimum to minimum depth of 18 inches.

### **Foundation Settlement**

Provided that all building foundations are extended through the existing fill and native slopewash/colluvial materials and into competent bedrock as recommended previously, total settlements of the foundations can be expected to range from ½ of an inch to an inch with a maximum differential settlement of approximately ½ of an inch over a span of 40 feet. Once the actual building loads are known, we will re-analyze our settlement estimates and provide revised total and differential design settlement amounts, as necessary.

### **Vapor Retarder/Barrier**

- Stego 15 Mil Class A or equivalent
  - Constructed below all moisture-sensitive floor areas of the foundation system.
  - Installed per manufacture's specifications as well as with all applicable recognized installation procedures such as ASTM E 1643-98.
  - Joints between the sheets and the openings for utility piping should be lapped and taped. If the retarder/barrier is not continuously placed across footings/ribs, the retarder/barrier should, as a minimum, be lapped into the sides of the footing/rib trenches down to the bottom of the trench.

- Punctures in the vapor retarder/barrier should be repaired prior to concrete placement.
- The moisture vapor retarder/barrier may be placed directly on the subgrade soil. Prior to placing the retarder/barrier, the subgrade should be smooth and free of any protrusions that may damage the retarder.
- From a geotechnical standpoint, sand is not required above the moisture vapor retarder/barrier system. If sand above the retarder system is selected by the architect or structural engineer, then it should be placed in a dry condition.

Note: The architect may choose to omit the vapor retarder if a fully enclosed waterproofing system is utilized below the concrete slabs.

### **Water Vapor Transmission**

As discussed above, placement of a moisture vapor retarder/barrier below all slab areas is recommended where moisture sensitive flooring will be placed. This moisture vapor retarder/barrier recommendation is intended only to reduce moisture vapor transmissions from the soil beneath the concrete and is consistent with the current standard of the industry for residential construction in Southern California. It is not intended to provide a “waterproof” or “vapor proof” barrier or reduce vapor transmission from sources above the retarder. Sources above the retarder include any sand placed on top of the retarder (i.e., to be determined by the project structural designer) and from the concrete itself (i.e., vapor emitted during the curing process). The evaluation of water vapor from any source and its effect on any aspect of the proposed living space above the slab (i.e., floor covering applicability, mold growth, etc.) is outside our purview and the scope of this report.

### **Floor Coverings**

Prior to the placement of flooring, the floor slabs should be properly cured and tested to verify that the water vapor transmission rate (WVTR) is compatible with the flooring requirements.

### **CONCRETE**

Based on the results of our laboratory testing, the onsite soil and bedrock materials have negligible sulfate contents; however, they have elevated levels of chlorides per Section 1904.3 of the 2022 CBC. Therefore, we recommend using the following:

#### Structural Elements (i.e., foundations, walls, etc.)

- Cement Type: II/V
- Maximum Water Cement Ratio: 0.50
- Minimum Strength: 4,000 psi (geotechnical perspective only)

- Reinforcement steel should be covered by at least 3 inches of concrete placed against earth.

Consideration should also be given to including a corrosion inhibiting additive within the concrete mix. These recommendations will serve to minimize the potential of water and/or vapor transmission through the concrete, minimize the potential for physical attack to concrete from non-sulfate based salts, and add additional protection to embedded steel reinforcement. In addition, wet curing of the concrete as described in ACI Publication 308 should be considered.

#### Non-structural Elements (i.e., flatwork, pavement, etc.)

Non-structural onsite concrete (i.e. walkways, patios, driveways, etc.) may be designed with concrete strengths that are determined by the engineer or designer responsible for that particular site improvement. The concrete design should account for the severe sulfate content of the onsite soils. Specific flatwork concrete requirements are provided in Appendix D.

The aforementioned recommendations for concrete are made from a soils perspective only. Final concrete mix design as well as any concrete testing is outside our purview. All applicable codes, ordinances, regulations, and guidelines should be followed in regard to designing a durable concrete with respect to the potential for detrimental exposure from the on-site soils and/or changes in the environment.

### **CORROSION PROTECTION OF METAL STRUCTURES**

The results of the laboratory chemical tests performed on soil samples collected within and adjacent to the subject area indicate that the on-site soils are severely corrosive to ferrous metals. Consequently, metal structures which will be in direct contact with the soil (i.e., underground metal conduits, pipelines, metal sign posts, metal door frames, etc.) and/or in close proximity to the soil (wrought iron fencing, etc.) may be subject to corrosion. The use of special coatings or cathodic protection around buried metal structures has been shown to be beneficial in reducing corrosion potential. The potential for corrosion of ferrous metal reinforcing elements embedded in structural concrete will be reduced by the use of the recommended maximum water/cement ratio for concrete and additional concrete cover.

The laboratory testing program performed for this project does not address the potential for corrosion to copper piping. In this regard, a corrosion engineer should be consulted to perform more detailed testing and develop appropriate mitigation measures (if necessary). Otherwise, the on-site soils should be considered corrosive to copper.

The above discussion is provided for general guidance regarding the corrosiveness of the on-site soils to typical metal structures used for construction. Detailed corrosion testing and recommendations for protecting buried ferrous metal and/or copper elements is beyond our purview.

## **SITE WALL AND RETAINING WALL DESIGN CRITERIA**

### **General**

Retaining walls will be required for the perimeter subterranean parking garage walls. In addition, miscellaneous exterior walls ranging from free-standing screen walls, planter walls, and low-height retaining walls to full-height retaining walls are planned at the site.

The bedrock parameters and minimum footing design recommendations provided previously for the building foundations should be used for the design of the parking structure retaining walls.

The criteria contained in the following sections may be used for the design and construction of the exterior retaining walls and site walls within the site that will be founded on compacted fill rather than bedrock.

### **Soil Parameters**

Bearing Material:	Compacted Fill
Allowable Bearing Value:	2000 psf, based on a 12 inch deep by 12 inch wide footing: (see subsequent section for actual minimum recommended footing dimensions.) <ul style="list-style-type: none"><li>• May be increased 10% for each additional foot of width and by 20% for each additional foot of depth to a maximum of 3000 psf).</li><li>• One-third increase for wind or seismic loading.</li></ul>
Coefficient of Friction:	0.30 <ul style="list-style-type: none"><li>• One-third increase for wind or seismic loading.</li></ul>
Passive Resistance:	300 psf/ft of depth to a maximum value of 3,000 psf. 175 psf/ft of depth (sloping ground) <ul style="list-style-type: none"><li>• Disregard upper 6 inches (level ground)</li><li>• Disregard upper 12 inches (sloping ground)</li><li>• One-third increase for wind or seismic loading.</li></ul>

### **Minimum Footing Design Recommendations**

Minimum Footing Width:	24 inches
Minimum Footing Depth:	Depth below lowest adjacent grade to bottom of footing: 18 inches

Minimum Reinforcement: Four #4 bars (two at top and two at bottom).

### **Construction Joints**

Construction joints within free-standing walls should be implemented and designed by a structural engineer. As a minimum, construction joints should be provided at a maximum interval of 20 feet and at all angle points and other locations where differential movement is likely to occur. Joints to consist of a clear vertical break of all masonry materials.

### **Retaining Wall Lateral Earth Pressures**

- Lateral Earth Pressures: 40 pcf (Active – Level Backfill).  
60 pcf (Active – 2:1 Backfill).  
55 pcf (At-Rest – Level Backfill).  
70 pcf (At-Rest – 2:1 Backfill).

The unrestrained (active) values are applicable only when the walls are designed and constructed as cantilevered walls allowing sufficient wall movement to mobilize the active pressure conditions. A wall movement of at least 0.01 H (where H = the total height of the wall) should be assumed for the unrestrained (active) values to be applicable. Where the wall movement must be held to less than 0.01H, the restrained or at-rest pressures should be used.

Per the 2022 CBC, the following seismic lateral earth coefficients and lateral earth pressures should be utilized for walls with a retaining height in excess of 6 feet. These values are based on a “design level ground” acceleration (PGA) equivalent to  $S_{DS}/2.5$  ( $0.962/2.5 = 0.38g$ ).

- Seismic Lateral Earth Coefficient:  $K_H = (0.5)PGA = (0.5)0.38g = 0.19g$
- Seismic Earthquake Pressure (EFP): 17.5 pcf (normal triangular pressure distribution added to either active or at-rest earth pressures)
- Unit Weight of Backfill: 125 pcf

### **Waterproofing**

The back side of all retaining walls should be waterproofed down to and onto the top of the foundation prior to placing subdrains or backfill. The design and selection of the waterproofing system is outside the scope of our report and is outside our purview.

## **Wall Backfill and Drainage**

The retaining walls should be provided with subdrain systems and backfilled per the Retaining Wall Construction Detail diagram (Plate C-1) contained within Appendix C.

## **POLE FOUNDATION DESIGN PARAMETERS**

The following geotechnical design parameters may be used for the design of pole foundations for proposed pedestrian light poles and similar structures:

Bearing Materials:	Engineered fill
Minimum Pole Foundation Depth:	Per Structural Engineer (3 feet minimum)
Minimum Pole Foundation Diameter:	18 inches
Allowable End Bearing:	3000 psf (for minimum pole depth of 3 feet)
Allowable Skin Friction:	250 psf
Allowable Passive Resistance:	320 psf/per foot of pole foundation depth. <ul style="list-style-type: none"><li>○ Disregard the upper foot due to soil disturbance unless placed in concrete hardscape or asphalt areas</li><li>○ Can be applied over 2 pole foundation diameters provided poles are spaced at least 3 pole diameters, center to center (e.g., 640 psf/ft of pole diameter per foot of depth).</li></ul>

## **MISCELLANEOUS FOUNDATIONS**

Foundations for miscellaneous structures to be constructed at the site (i.e., marketing or monument signs, sculptures, and similar structures) should be embedded a minimum of 18 inches deep below the lowest adjacent final grade. Minimum reinforcement for continuous footings should consist of two #4 bars placed in the top and bottom of the footings (four bars total). For pad footings, a minimum of two #4 bars, both ways, should be placed near the bottoms of the footings.

## **CONCRETE FLATWORK**

### **Flatwork Design**

Concrete flatwork should be designed in minimum accordance with the recommendations contained in Appendix D - Table 1. It should be noted that the recommendations contained in this table are largely to improve “post-cure” performance relative to expansive soils. All other aspects of concrete design (i.e., concrete mix design, curing, type, and location of joints, etc.) as well as concrete inspection of any kind is outside our purview. It is recommended that the final flatwork design be reviewed by our office prior to bidding.

Even with extensive crack control and expansive soil mitigation, all concrete flatwork will crack and move (i.e., lift) to some degree due to a variety of mechanisms. Consequently, concrete cracking and movement and hence concrete repair/replacement should be anticipated.

### **Subgrade Preparation**

Due to the length of time that is expected to elapse between initial site grading and eventual placement of exterior concrete flatwork along with the likely disturbance of the exterior subgrade soils due to significant construction activity, it is expected that the exterior subgrade soils will become disturbed. Therefore, just prior to concrete placement, any disturbed surficial subgrade soils below concrete flatwork areas should be scarified, watered to achieve a moisture content that is at least 2% over optimum, and then re-compacted in-place to a minimum relative compaction of 90 percent. This moisture content should extend to a depth of approximately 18 inches into the subgrade soils and be maintained in the subgrade during concrete placement to promote uniform curing of the concrete and minimize the development of unsightly shrinkage cracks. Flooding or ponding of the subgrade is not considered feasible to achieve the above moisture conditions since this method would likely require construction of numerous earth berms to contain the water. Therefore, moisture conditioning should be achieved with sprinklers or a light spray applied to the subgrade over a period of several days just prior to pouring concrete. Soil density and presoaking should be observed, tested, and accepted by GMU prior to pouring the concrete.

All concrete has the tendency to crack, and cracks in concrete can be caused by many different factors. When constructing concrete decks, patios, walkways, etc., it is important that the ground on which these improvements are to rest be properly prepared, including moisture conditioning. Slab thickness, location of joints, reinforcement, and concrete mixture must also be appropriate for the intended use. Proper placement, finishing, and curing of concrete are also very important factors in minimizing cracking.

## PAVEMENT DESIGN

The following asphalt pavement structural sections are considered applicable for the design of new drive aisles and parking stalls within the site. The structural sections are based on an R-value of 17 and traffic indices of 5.5 for the drive aisles and 4.5 for the parking stalls.

<b>Location</b>	<b>Traffic Index</b>	<b>Asphalt Concrete (in.)/ Aggregate Base (in.)</b>	<b>Full Depth Asphalt Concrete (in.)</b>
Drive Aisles	5.5	4/8.0	7.5
Parking Stalls	4.5	4/4.5	6

Aggregate base may consist of either CAB or CMB as per current Greenbook standards. The base materials (CAB or CMB) and asphalt concrete materials (AC) should be of a type meeting the minimum County of Orange and Greenbook standards. Asphalt pavement base course shall be 3/4" (II-B3-PG 64-10) while the surface course shall be 1/2" (III-C2-PG 64-10). The subgrade soils should be moisture conditioned to at about 2% above the optimum moisture content, compacted to at least 90% relative compaction and be unyielding. For full depth asphalt sections, the subgrade soils should be compacted to at least 95% relative compaction to a depth of 12 inches and be unyielding. The AB and AC materials should be compacted to at least 95% relative compaction.

## FUTURE PLAN REVIEW AND RESPONSES

GMU Geotechnical, Inc. (GMU) should review future project plans to check for conformance to the recommendations provided herein, provide geotechnical response letters to support the permit process, and provide additional geotechnical recommendations as needed. Specifically, GMU should review the following:

- Finalized Grading Plans
- Building Foundation Plans along with Calculations and Actual Building Loads
- Site Wall/Retaining Wall Plans and Calculations, if any.
- Landscape Plans

## GEOTECHNICAL OBSERVATIONS/TESTING DURING CONSTRUCTION

Consistent with the standard of practice in the geotechnical industry, these observation and testing services will need to be provided by GMU during construction as a condition of our geotechnical design report.

### **Rough Grading**

- Attend a pre-grade meeting with contractor and design team.
- Observe processing of areas to receive fill or areas of shallow cut.
- Field density testing relative to fill placement.
- Grading release letter to obtain building permit from City of Dana Point.

### **Foundation and Slab Construction**

- Observe foundation excavations, including caissons.
- Observe backfill and compaction of interior utility trenches.
- Testing of subgrade preparation for building and garage floor slabs (i.e., pre-saturation).
- Observation of moisture vapor retarder placement.
- Observation of placement of subdrains behind subterranean garage walls and basement walls.
- Observation and testing of placement and compaction of backfill behind subterranean garage walls and basement walls.

### **On-Site Improvements**

- Observe footing excavations for exterior site walls, retaining walls, and similar structures.
- Observation of placement of subdrains behind exterior retaining walls.
- Observation and testing of placement and compaction of backfill behind exterior retaining walls.
- Testing for gas, sewer, water, storm drain, electric, telephone, cable and all other utility trench backfills.
- Testing of subgrade preparation for concrete pavement and flatwork (i.e., pre-saturation).

### **Laboratory Testing**

- Laboratory tests (compaction, sand equivalent, and subgrade and pre-saturation moisture tests).

A final report will be prepared to submit to the City of Laguna Hills documenting the results of our observation and testing as required for occupancy.

## LIMITATIONS

All parties reviewing or utilizing this report should recognize that the findings, conclusions, and recommendations presented represent the results of our professional geological and geotechnical engineering efforts and judgments. Due to the inexact nature of the state of the art of these professions and the possible occurrence of undetected variables in subsurface conditions, we cannot guarantee that the conditions actually encountered during grading and site construction will be identical to those observed, sampled, and interpreted during our study, or that there are no unknown subsurface conditions which could have an adverse effect on the use of the property. We have exercised a degree of care comparable to the standard of practice presently maintained by other professionals in the fields of geotechnical engineering and engineering geology, and believe that our findings present a reasonably representative description of geotechnical conditions and their probable influence on the grading and use of the property.

Our conclusions and recommendations are based on the assumption that our firm will act as the geotechnical engineer of record during construction and grading of the project to observe the actual conditions exposed, to verify our design concepts and the grading contractor's general compliance with the project geotechnical specifications, and to provide our revised conclusions and recommendations should subsurface conditions differ significantly from those used as the basis for our conclusions and recommendations presented in this report. Since our conclusions and recommendations are based on a limited amount of current and previous geotechnical exploration and analysis, all parties should recognize the need for possible revisions to our conclusions and recommendations during grading of the project.

It should be further noted that the recommendations presented herein are intended solely to minimize the effects of post-construction soil movements. Consequently, minor cracking and/or distortion of all on-site improvements should be anticipated.

This report has not been prepared for the use by other parties or projects other than those named or described herein. This report may not contain sufficient information for other parties or other purposes.

Mr. Matthew Haugen, **Buchanan Street Partners, LP.**  
*Geotechnical Investigation, Proposed 7- to 8-Story Residential Development, 24422 Avenida De La Carlota,  
Laguna Hills, California*

## CLOSURE

We are pleased to present the results of our geotechnical investigation for this project. The Plates and Appendices that complete this report are listed in the Table of Contents.

If you have any questions concerning our findings, please call and we will be glad to discuss them with you.



Respectfully submitted,

David Hansen, M.Sc., PE, GE 3056  
Associate Geotechnical Engineer



Alan B. Mutchnick, PG, CEG 1789  
Associate Engineering Geologist

dwh/23-008-00R (06-29-23)

## REFERENCES

### SITE-SPECIFIC REFERENCES

- (1) Buchanan Street Partners Architectural Plans, *24422 Avenida De La Carlota, Laguna Hills, CA*; prepared by Architects Orange, dated April 19, 2023.
- (2) *Conceptual Grading Plan, 24422 Avenida De La Carlota, Laguna Hills, California*; prepared by Fuscoe Engineering, dated March 2023.

### REGIONAL GEOLOGY REFERENCES

California Division of Mines and Geology, 2001, *Seismic Hazard Zone Report for the San Juan Capistrano 7.5-Minute Quadrangles, Orange County, California*, Seismic Hazard Zone Report 053.

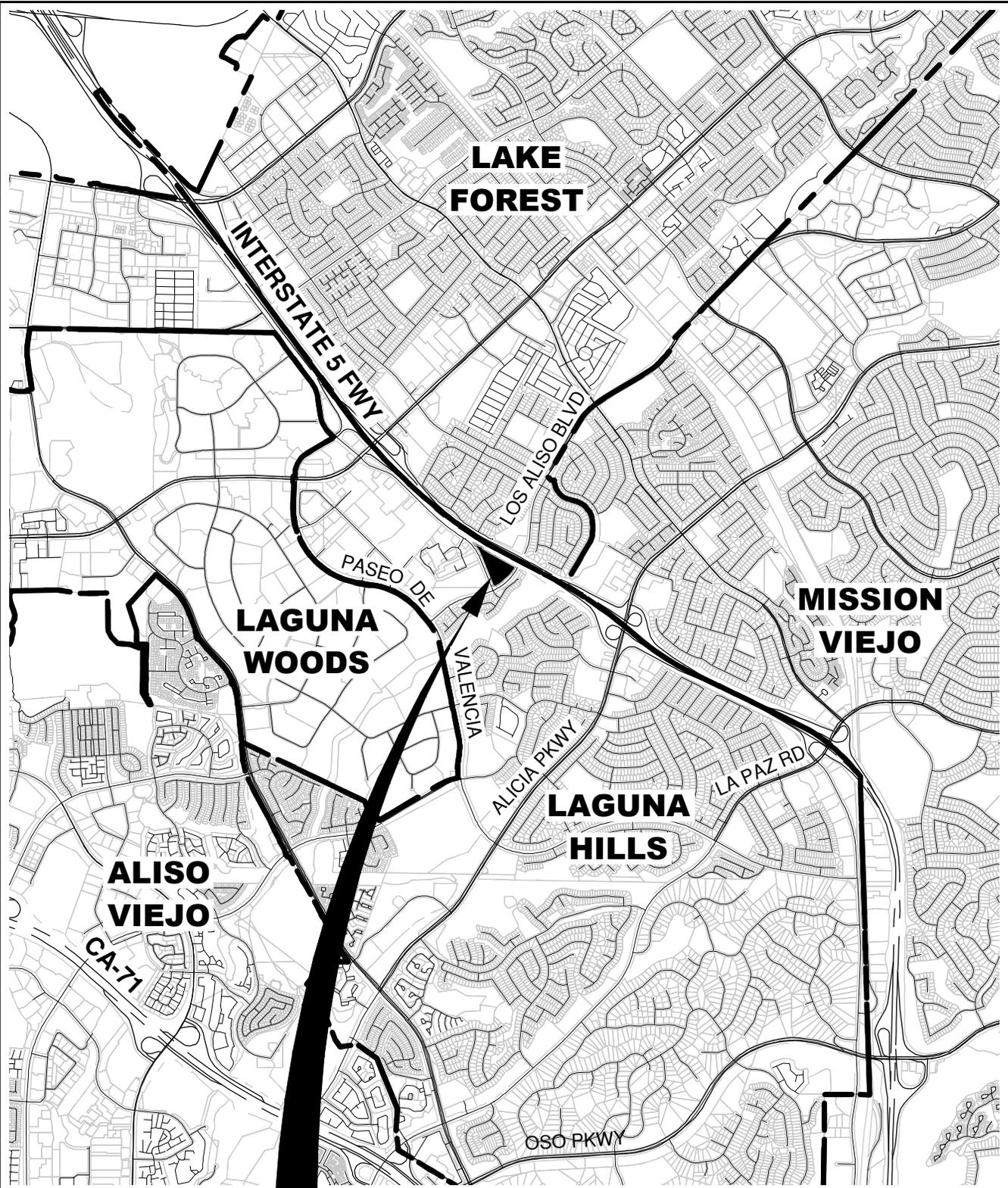
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Morton, P.K., Miller, R.V., and Evans, J.R., 1976, *Environmental Geology of Orange County, California: California Division of Mines and Geology, Open File Report 79-8 LA*.

Morton, P.K., and Miller, R.V., 1981, *Geologic Map of Orange County, Showing Mines and Mineral Deposits*: California Division of Mines and Geology, Scale: 1" = 4000'.

DRAWING: c:\2023\23-008-00.dwg\300800\_plate 1\_location map.dwg PLOTTED: 7/5/2023 11:09 AM BY: Jesus Meza



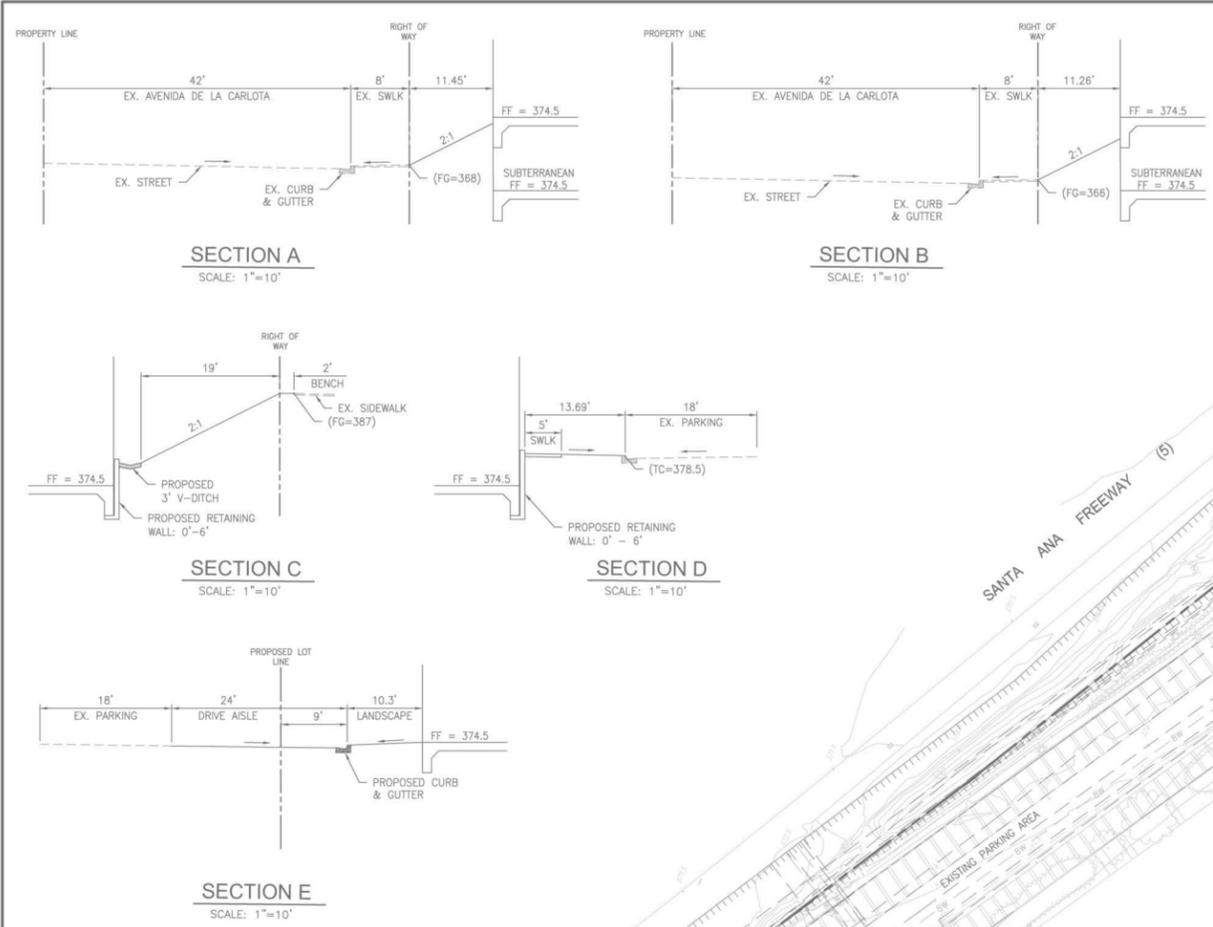
**PROJECT LOCATION**  
 24422 AVENIDA DE LA CARLOTA  
 LAGUNA HILLS, CALIFORNIA



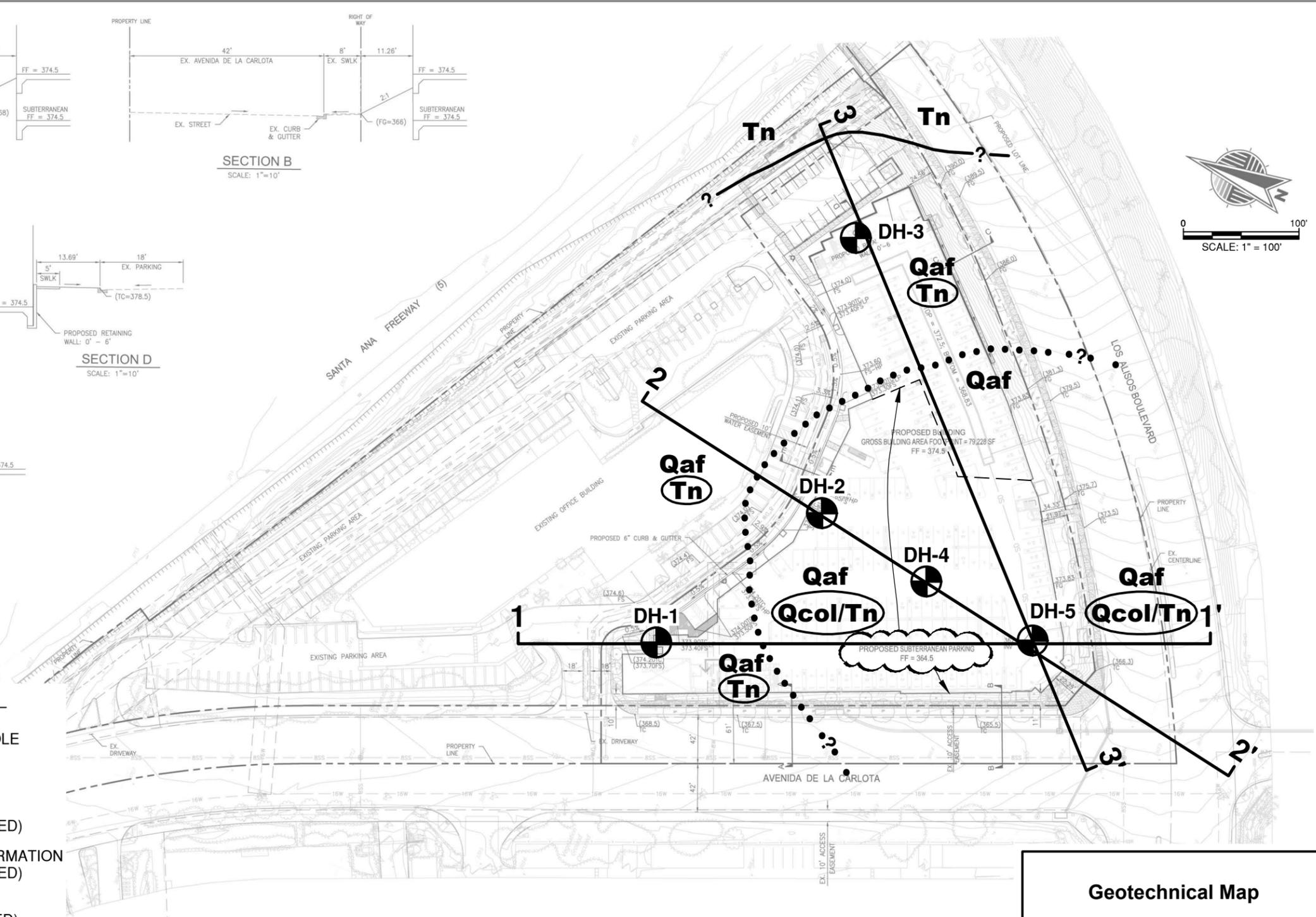
**Location Map**

	Date: June 29, 2023	Plate 1
	Project No.: 23-008-00	

DRAWING: q:\2023\23-008-00\dwg\2300800\_plate 2\_geo map.dwg PLOTTED: 7/5/2023 11:55 AM BY: Jesus Meza



- GEOTECHNICAL LEGEND**
- DH-5** LOCATION OF DRILL HOLE
  - Qaf** ARTIFICIAL FILL
  - Qcol** COLLUVIUM (CIRCLED WHERE BURIED)
  - Tn** BEDROCK - NIGUEL FORMATION (CIRCLED WHERE BURIED)
  - .....** GEOLOGIC CONTACT (DOTTED WHERE BURIED)
  - 3' / 3'** GEOLOGIC SECTION



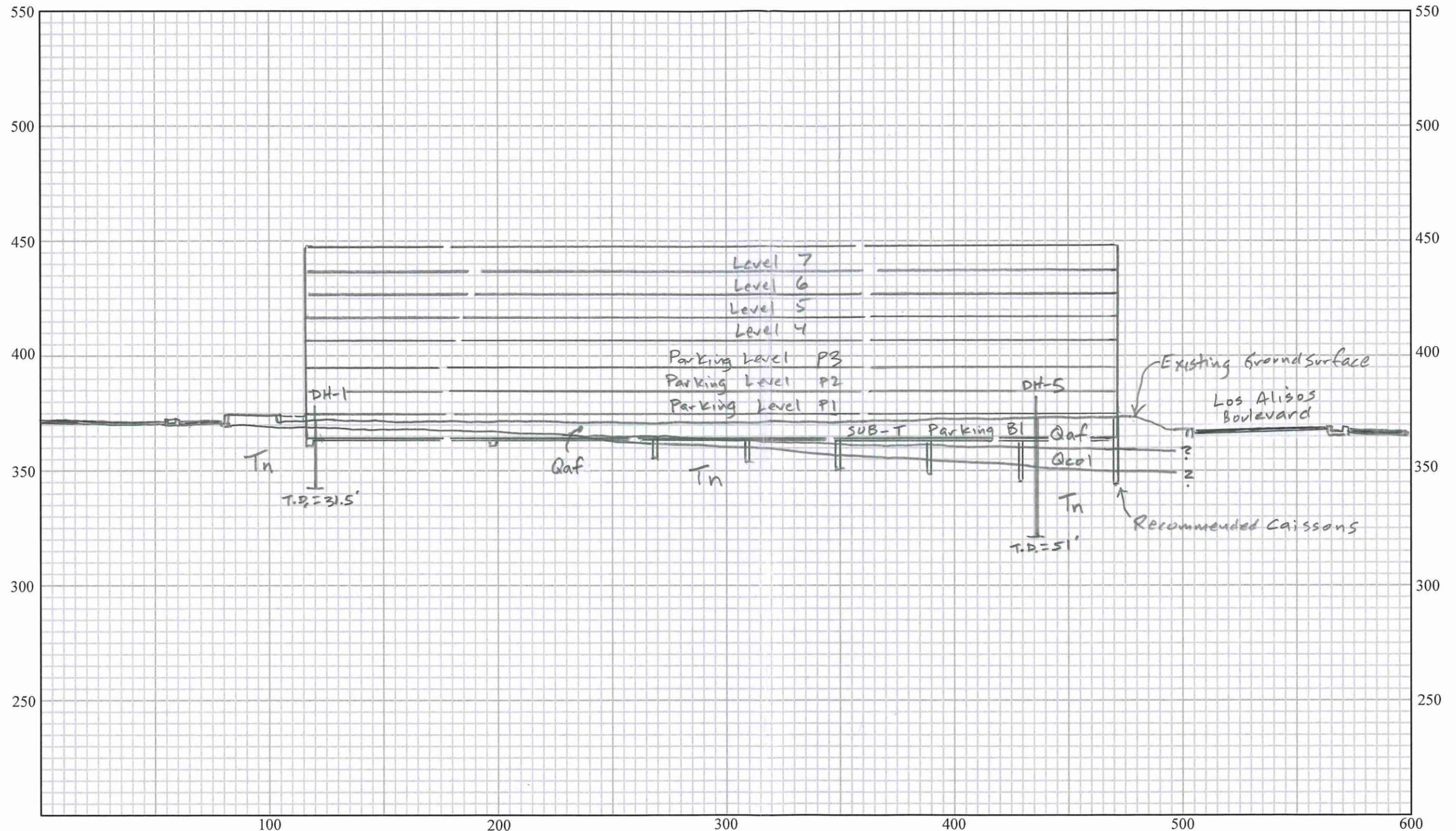
**Geotechnical Map**

<b>GMU</b> <small>ENGINEERS &amp; GEOLOGISTS</small>	Date: June 29, 2023	Plate 2
	Project No.: 23-008-00	

MARCH 2023		PREPARED UNDER THE SUPERVISION OF:				<b>CONCEPTUAL GRADING PLAN</b>	SHEET C-4 OF C-5
NO.	DATE	REVISIONS	APPROVED				

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1 1'



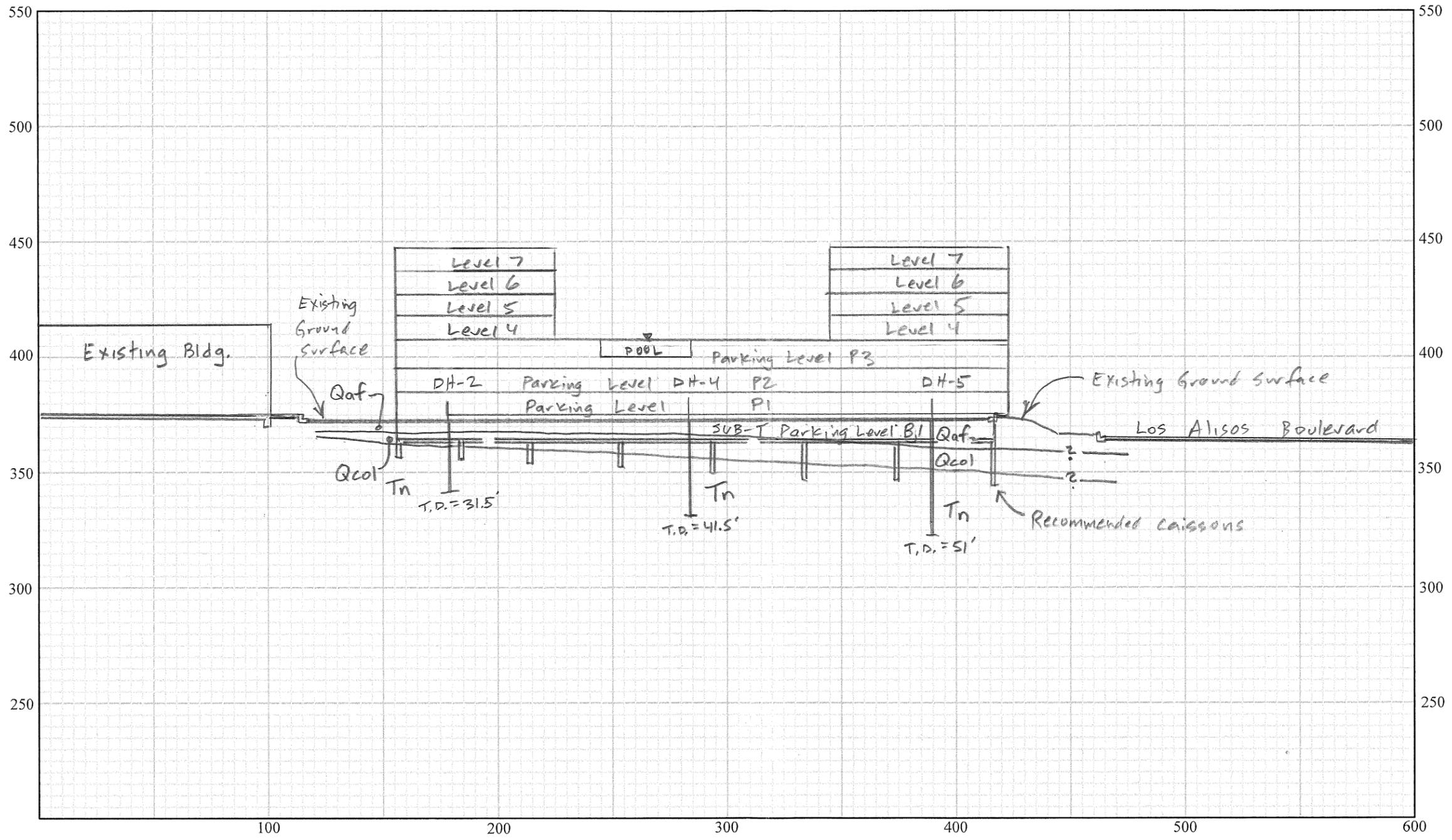
SCALE 1" = 50'

### Geologic Section 1-1'

<b>GMU</b>	Date: June 29, 2023	Plate 3.0
	Project No.: 23-008-00	

2

2'



SCALE 1" = 50'

### Geologic Section 2-2'

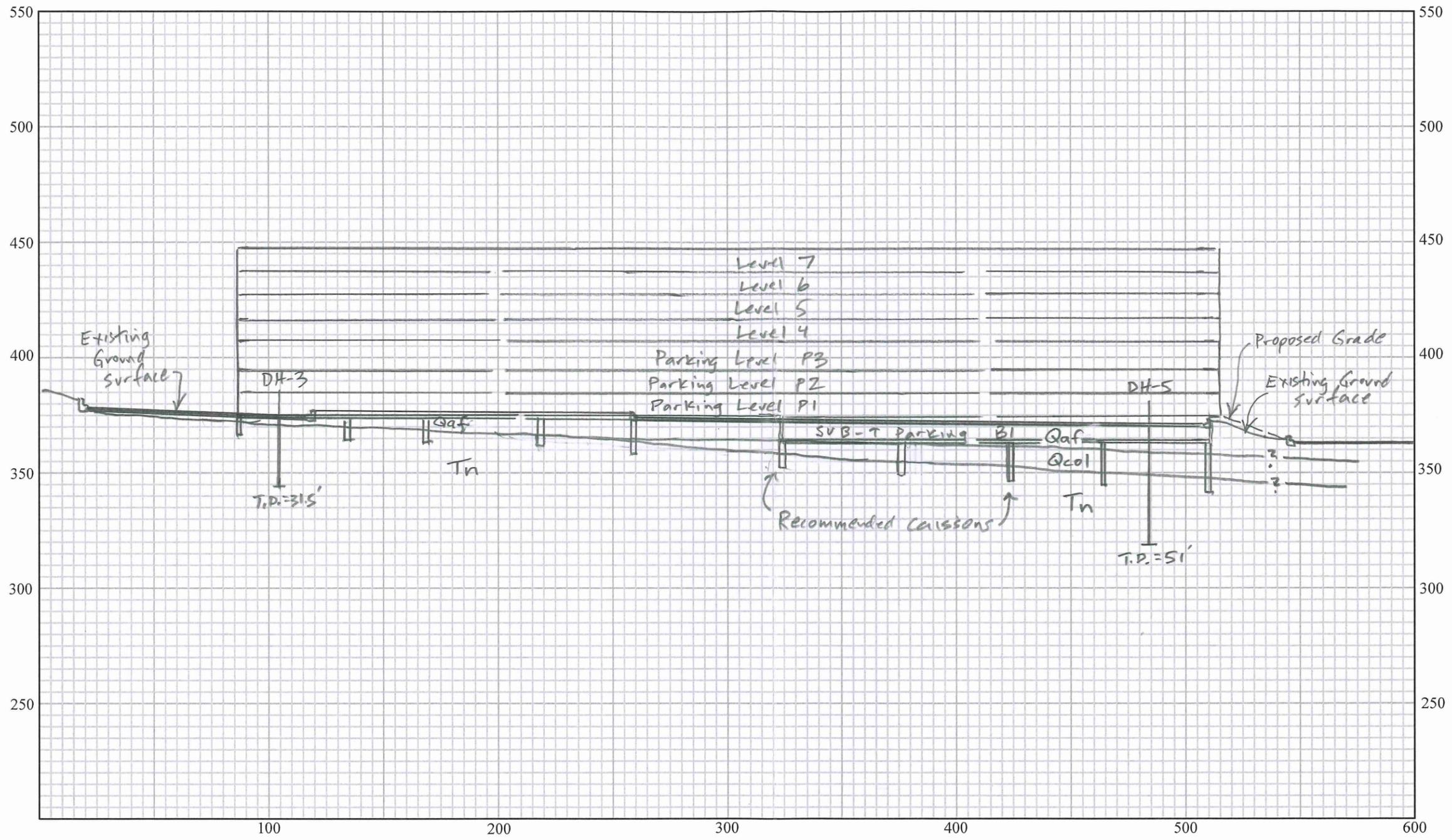


Date: June 29, 2023	Plate 3.1
Project No.: 23-008-00	

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3

3'



SCALE 1" = 50'

### Geologic Section 3-3'



Date: June 29, 2023	Plate 3.2
Project No.: 23-008-00	

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# APPENDIX A

## Geotechnical Exploration Procedures and Drill Hole Logs

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## **APPENDIX A**

### **GMU GEOTECHNICAL EXPLORATION PROCEDURES AND DRILL HOLE LOGS**

Our exploration at the subject site consisted of the advancement of five (five) exploratory drill holes to depths of 31.5 to 51 feet below the existing ground surfaces using a hollow-stem auger drilling rig. Our drill holes were logged by a geologist and drive and bulk samples of the excavated soil and bedrock materials were collected for laboratory testing. Blow counts recorded during sampling from the California Modified Sampler (Cal Mod) are shown on the drill hole logs. The logs of each drill hole are contained in this Appendix A, and the Legend to Logs is presented as Plates A-1 and A-2. The approximate locations of the drill holes are shown on Plate 2 – Geotechnical Map.

“Undisturbed” Cal Mod samples were taken using a 3.0-inch thin walled outside-diameter drive sampler which contains a 2.416-inch-diameter brass sample sleeve 6 inches in length. Bulk samples of the soil materials were also collected from various depths of the site soils.

The geologic and engineering field descriptions and classifications that appear on these logs are prepared according to Corps of Engineers and Bureau of Reclamation standards. Major soil classifications are prepared according to the Unified Soil Classification System as modified by ASTM Standard No. 2487. Since the descriptions and classifications that appear on the Log of Drill Hole are intended to be that which most accurately describe a given interval of a drill hole (frequently an interval of several feet), discrepancies do occur in the Unified Soil Classification System nomenclature between that interval and a particular sample in that interval. For example, an 8-foot-thick interval in a log may be identified as silty sand (SM) while one sample taken within the interval may have individually been identified as sandy silt (ML). This discrepancy is frequently allowed to remain to emphasize the occurrence of local textural variations in the interval.

MAJOR DIVISIONS		Group Letter	Symbol	TYPICAL NAMES
<b>COARSE-GRAINED SOILS</b> More Than 50% Retained On No.200 Sieve  Based on The Material Passing The 3-Inch (75mm) Sieve.  Reference: ASTM Standard D2487	<b>GRAVELS</b> 50% or More of Coarse Fraction Retained on No.4 Sieve	Clean Gravels	GW	Well Graded Gravels and Gravel-Sand Mixtures, Little or No Fines.
			GP	Poorly Graded Gravels and Gravel-Sand Mixtures Little or No Fines.
		Gravels With Fines	GM	Silty Gravels, Gravel-Sand-Silt Mixtures.
			GC	Clayey Gravels, Gravel-Sand-Clay Mixtures.
	<b>SANDS</b> More Than 50% of Coarse Fraction Passes No.4 Sieve	Clean Sands	SW	Well Graded Sands and Gravelly Sands, Little or No Fines.
			SP	Poorly Graded Sands and Gravelly Sands, Little or No Fines.
		Sands With Fines	SM	Silty Sands, Sand-Silt Mixtures.
			SC	Clayey Sands, Sand-Clay Mixtures.
<b>FINE-GRAINED SOILS</b> 50% or More Passes The No.200 Sieve  Based on The Material Passing The 3-Inch (75mm) Sieve.  Reference: ASTM Standard D2487	<b>SILTS AND CLAYS</b> Liquid Limit Less Than 50%	ML	Inorganic Silts, Very Fine Sands, Rock Flour, Silty or Clayey Fine Sands or Clayey Silts With Slight Plasticity.	
		CL	Inorganic Clays of Low To Medium Plasticity, Gravelly Clays, Sandy Clays, Silty Clays, Lean Clays.	
		OL	Organic Silts and Organic Silty Clays of Low Plasticity	
	<b>SILTS AND CLAYS</b> Liquid Limit 50% or Greater	MH	Inorganic Silts, Micaceous or Diatomaceous Fine Sandy or Silty Soils, Elastic Silts.	
		CH	Inorganic Clays of High Plasticity, Fat Clays.	
		OH	Organic Clays of Medium To High Plasticity, Organic Silts.	
<b>HIGHLY ORGANIC SOILS</b>		PT	Peat and Other Highly Organic Soils.	

The descriptive terminology of the logs is modified from current ASTM Standards to suit the purposes of this study

#### ADDITIONAL TESTS

DS = Direct Shear  
 HY = Hydrometer Test  
 TC = Triaxial Compression Test  
 UC = Unconfined Compression  
 CN = Consolidation Test  
 (T) = Time Rate  
 EX = Expansion Test  
 CP = Compaction Test  
 PS = Particle Size Distribution  
 EI = Expansion Index  
 SE = Sand Equivalent Test  
 AL = Atterberg Limits  
 FC = Chemical Tests  
 RV = Resistance Value  
 SG = Specific Gravity  
 SU = Sulfates  
 CH = Chlorides  
 MR = Minimum Resistivity  
 pH  
 (N) = Natural Undisturbed Sample  
 (R) = Remolded Sample  
 CS = Collapse Test/Swell-Settlement

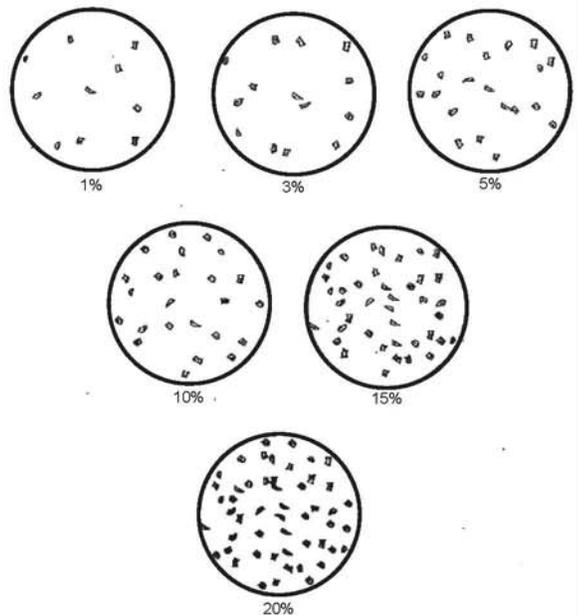
#### GEOLOGIC NOMENCLATURE

B = Bedding C = Contact J = Joint  
 F = Fracture Flt = Fault S = Shear  
 RS = Rupture Surface  = Seepage  
 = Groundwater

#### SAMPLE SYMBOLS

 Undisturbed Sample (California Sample)  
 Undisturbed Sample (Shelby Tube)  
 Bulk Sample  
 Unsuccessful Sampling Attempt  
 SPT Sample

5  
 10  
 15 Blows per 6-Inches Penetration  
 10: 10 Blows for 12-Inches Penetration  
 6/4: 6 Blows for 4-Inches Penetration  
 P: Push  
 (13): Uncorrected Blow Counts ("N" Values) for 12-Inches Penetration- Standard Penetration Test (SPT)



**LEGEND TO LOGS**  
 ASTM Designation: D 2487  
 (Based on Unified Soil Classification System)

Plate  
**A-1**

SOIL DENSITY/CONSISTENCY			
FINE GRAINED			
Consistency	Field Test	SPT (#blows/foot)	Mod (#blows/foot)
Very Soft	Easily penetrated by thumb, exudes between fingers	<2	<3
Soft	Easily penetrated one inch by thumb, molded by fingers	2-4	3-6
Firm	Penetrated over 1/2 inch by thumb with moderate effort	4-8	6-12
Stiff	Penetrated about 1/2 inch by thumb with great effort	8-15	12-25
Very Stiff	Readily indented by thumbnail	15-30	25-50
Hard	Indented with difficulty by thumbnail	>30	>50
COARSE GRAINED			
Density	Field Test	SPT (#blows/foot)	Mod (#blows/foot)
Very Loose	Easily penetrated with 0.5" rod pushed by hand	<4	<5
Loose	Easily penetrated with 0.5" rod pushed by hand	4-10	5-12
Medium Dense	Easily penetrated 1' with 0.5" rod driven by 5lb hammer	10-30	12-35
Dense	Difficult to penetrate 1' with 0.5" rod driven by 5lb hammer	31-50	35-60
Very Dense	Penetrated few inches with 0.5" rod driven by 5lb hammer	>50	>60

BEDROCK HARDNESS		
Density	Field Test	SPT (#blows/foot)
Soft	Can be crushed by hand, soil like and structureless	1-30
Moderately Hard	Can be grooved with fingernails, crumbles with hammer	30-50
Hard	Can't break by hand, can be grooved with knife	50-100
Very Hard	Scratches with knife, chips with hammer blows	>100

MODIFIERS	
Trace	1%
Few	1-5%
Some	5-12%
Numerous	12-20%
Abundant	>20%

GRAIN SIZE			
Description	Sieve Size	Grain Size	Approximate Size
Boulders	>12"	>12"	Larger than a basketball
Cobbles	3-12"	3-12"	Fist-sized to basketball-sized
Gravel	Coarse	3/4-3"	Thumb-sized to fist-sized
	Fine	#4-3/4"	Pea-sized to thumb-sized
Sand	Coarse	#10-#4	Rock-salt-sized to pea-sized
	Medium	#40-#10	Sugar-sized to rock salt-sized
	Fine	#200-#40	Flour-sized to sugar-sized
Fines	passing #200	<0.0029"	Flour-sized and smaller

MOISTURE CONTENT
Dry- Very little or no moisture
Damp- Some moisture but less than optimum
Moist- Near optimum
Very Moist- Above optimum
Wet/Saturated- Contains free moisture



**LEGEND TO LOGS**  
 ASTM Designation: D 2487  
 (Based on Unified Soil Classification System)

Plate  
**A-2**

**Project: Buchanan Street Partners**  
**Project Location: 24422 Avenida De La Carlota, Laguna Hills**  
**Project Number: 23-008-00**

# Log of Drill Hole DH-1

Sheet 1 of 2

Date(s) Drilled <b>1/20/23</b>	Logged By <b>RA</b>	Checked By <b>DW</b>
Drilling Method <b>Hollow Stem Auger</b>	Drilling Contractor <b>2R Drilling</b>	Total Depth of Drill Hole <b>31.5 feet</b>
Drill Rig Type <b>CME 75</b>	Diameter(s) of Hole, inches <b>8</b>	Approx. Surface Elevation, ft MSL <b>372.0</b>
Groundwater Depth [Elevation], feet <b>NA</b>	Sampling Method(s) <b>Open drive sampler with 6-inch sleeve, Bulk</b>	Drill Hole Backfill <b>Native</b>
Remarks		Driving Method and Drop <b>Auto Hammer</b>

ELEVATION, feet	DEPTH, feet	GRAPHIC LOG	GEOLOGICAL CLASSIFICATION AND DESCRIPTION	ORIENTATION DATA	ENGINEERING CLASSIFICATION AND DESCRIPTION	SAMPLE DATA			TEST DATA		
						SAMPLE	NUMBER OF BLOWS / 6"	DRIVING WEIGHT, lbs	MOISTURE CONTENT, %	DRY UNIT WEIGHT, pcf	ADDITIONAL TESTS
370			<b>ARTIFICIAL FILL (Qaf)</b> Few roots up to 1/4" in diameter, few subangular gravel up to 1/2" in diameter		ASHPALT CONCRETE - 4-inches AGGREGATE BASE - 6-inches SANDY CLAY (CL); olive gray, damp, firm, fine- to medium-grained sand						
5			<b>BEDROCK - NIGUEL FORMATION (Tn)</b> Slightly oxidized sand grains		SANDY SILTSTONE; pale brown to light gray, damp, moderately hard to hard, fine- to medium-grained sand		12 25 36	140	26		
365			Becomes unoxidized		SANDY CLAYEY SILTSTONE; pale brown, damp, moderately hard to hard, fine-grained sand		10 19 21	140	22	110	
10			Pinhole sized charcoal fragments		SANDY SILTSTONE; pale brown, damp, moderately hard to hard, fine-grained sand		16 26 26	140	25		
360											
15											
355											

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**Drill Hole DH-1**

Project: Buchanan Street Partners  
 Project Location: 24422 Avenida De La Carlota, Laguna Hills  
 Project Number: 23-008-00

# Log of Drill Hole DH-1

Sheet 2 of 2

ELEVATION, feet	DEPTH, feet	GRAPHIC LOG	GEOLOGICAL CLASSIFICATION AND DESCRIPTION	ORIENTATION DATA	ENGINEERING CLASSIFICATION AND DESCRIPTION	SAMPLE DATA			TEST DATA		
						SAMPLE NUMBER	NUMBER OF BLOWS / 6"	DRIVING WEIGHT, lbs	MOISTURE CONTENT, %	DRY UNIT WEIGHT, pcf	ADDITIONAL TESTS
350			<b>BEDROCK - NIGUEL FORMATION (Tn)</b> Sand grains become slight to moderately oxidized, yellow fine-grained sand blips, increase in mica abundance		SANDY SILTSTONE; pale brown, damp, moderately hard, fine-grained sand	12 15 25		140	26	108	
	25		Decrease in oxidized sand grains, minor rig chatter			12 18 27		140	27		
345											
	30		Increase in oxidized sand grains, yellow fine-grained sand blips		Becomes pale brown to light gray, few fine-grained sand	10 18 27		140	26	109	
					Total Depth = 31.5' No Groundwater						

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Drill Hole DH-1

**Project:** Buchanan Street Partners  
**Project Location:** 24422 Avenida De La Carlota, Laguna Hills  
**Project Number:** 23-008-00

# Log of Drill Hole DH-2

Sheet 1 of 2

Date(s) Drilled	1/20/23	Logged By	DW/RA	Checked By		
Drilling Method	Hollow Stem Auger	Drilling Contractor	2R Drilling	Total Depth of Drill Hole	31.5 feet	
Drill Rig Type	CME 75	Diameter(s) of Hole, inches	8	Approx. Surface Elevation, ft MSL	372.0	
Groundwater Depth [Elevation], feet	NA □	Sampling Method(s)	Open drive sampler with 6-inch sleeve, Bulk	Drill Hole Backfill	Native	
Remarks					Driving Method and Drop	Auto Hammer

ELEVATION, feet	DEPTH, feet	GRAPHIC LOG	GEOLOGICAL CLASSIFICATION AND DESCRIPTION	ORIENTATION DATA	ENGINEERING CLASSIFICATION AND DESCRIPTION	SAMPLE DATA			TEST DATA			
						SAMPLE	NUMBER OF BLOWS / 6"	DRIVING WEIGHT, lbs	MOISTURE CONTENT, %	DRY UNIT WEIGHT, pcf	ADDITIONAL TESTS	
370			<u>ARTIFICIAL FILL (Qaf)</u>		ASPHALT CONCRETE - 3.5-inches AGGREGATE BASE - 6.0-inches							
					SANDY CLAY (CL); olive with gray, firm, damp, fine- to medium-grained sand, trace gravel				20			PS ATT CP EI pH SU CH MR R-val
5			<u>SLOPEWASH/COLLUVIUM (Qsw/Qcol)</u>		SANDY SILT (SL); grayish yellow, damp, firm, fine-grained sand		4 6 15	140	16	96		
365			Moderate rig chatter									
10			Some rootlets, caliche stringers		SILTY CLAY (CL); gray and brown, damp to moist, very stiff, some fine-grained sand		6 10 18	140	20	98		
360			<u>BEDROCK - NIGUEL FORMATION (Tn)</u> Moderately weathered		CLAYEY SILTSTONE; light gray and light olive, damp, moderately hard, some very fine-grained sand							
15			Oxidized blips				13 22 30	140	24	108	DS	
355												

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**Drill Hole DH-2**







**Project:** Buchanan Street Partners  
**Project Location:** 24422 Avenida De La Carlota, Laguna Hills  
**Project Number:** 23-008-00

# Log of Drill Hole DH-4

Sheet 1 of 2

Date(s) Drilled <b>1/20/23</b>	Logged By <b>DW</b>	Checked By
Drilling Method <b>Hollow Stem Auger</b>	Drilling Contractor <b>2R Drilling</b>	Total Depth of Drill Hole <b>41.5 feet</b>
Drill Rig Type <b>CME 75</b>	Diameter(s) of Hole, inches <b>8</b>	Approx. Surface Elevation, ft MSL <b>373.0</b>
Groundwater Depth [Elevation], feet <b>NA</b>	Sampling Method(s) <b>Open drive sampler with 6-inch sleeve, Bulk</b>	Drill Hole Backfill <b>Native</b>
Remarks		Driving Method and Drop <b>Auto Hammer</b>

ELEVATION, feet	DEPTH, feet	GRAPHIC LOG	GEOLOGICAL CLASSIFICATION AND DESCRIPTION	ORIENTATION DATA	ENGINEERING CLASSIFICATION AND DESCRIPTION	SAMPLE DATA			TEST DATA				
						SAMPLE	NUMBER OF BLOWS / 6"	DRIVING WEIGHT, lbs	MOISTURE CONTENT, %	DRY UNIT WEIGHT, pcf	ADDITIONAL TESTS		
370	5		<b>ARTIFICIAL FILL (Qaf)</b>		ASPHALT CONCRETE - 3.5-inches AGGREGATE BASE - 6.0-inches								
					SANDY SILT (ML); brown with gray, moist, firm, fine- to medium-grained sand, trace gravel								
					CLAYEY SILT (ML); light gray with orange staining, moist, firm to stiff, some very fine-grained sand		6 10 19	140	24	97			
					<b>SLOPEWASH (Qsw)</b> Moderate rig chatter, some rootlets		SANDY CLAY (CL); dark brown, damp, firm, fine- to medium-grained sand						
365	10		Rootlets continue		Becomes moist to very moist		4 4 10	140	14	93	CN		
					SANDY SILT (ML); light brown, moist, firm to stiff, fine- to medium-grained sand, some gravel		6 12 14	140	11				
360	15												
355			<b>BEDROCK - NIGUEL FORMATION (Tn)</b> Moderately weathered		CLAYEY SILTSTONE; yellow with gray, moist, moderately hard, trace very fine-grained sand								

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**Drill Hole DH-4**

Project: Buchanan Street Partners  
 Project Location: 24422 Avenida De La Carlota, Laguna Hills  
 Project Number: 23-008-00

# Log of Drill Hole DH-4

Sheet 2 of 2

ELEVATION, feet	DEPTH, feet	GRAPHIC LOG	GEOLOGICAL CLASSIFICATION AND DESCRIPTION	ORIENTATION DATA	ENGINEERING CLASSIFICATION AND DESCRIPTION	SAMPLE DATA			TEST DATA		
						SAMPLE NUMBER	NUMBER OF BLOWS / 6"	DRIVING WEIGHT, lbs	MOISTURE CONTENT, %	DRY UNIT WEIGHT, pcf	ADDITIONAL TESTS
350	25		<b>BEDROCK - NIGUEL FORMATION (Tn)</b> Manganese staining blips, rootlets within fractures		CLAYEY SILTSTONE; light yellowish brown with gray and orange staining, damp to moist, moderately hard, some very fine-grained sand	12 19 23	140	25	108		
345					SANDY SILTSTONE; light olive, moist, moderately hard, very fine-grained sand	10 13 17	140	27			
340	30		Slight Seepage		CLAYEY SILTSTONE; light olive and yellow, moist to very moist, moderately hard, some very fine-grained sand	10 13 16	140	26	106		
335	35		Starting to become unoxidized		SANDY SILTSTONE; gray with orange staining, damp to moist, moderately hard	15 16 20	140	28			
330	40		Unoxidized, faint subhorizontal wavy laminations		Becomes dark gray and dark brownish gray, damp, hard	10 28 38	140	25	107		
Total Depth = 41.5' Seepage at 30'											

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**Drill Hole DH-4**

**Project:** Buchanan Street Partners  
**Project Location:** 24422 Avenida De La Carlota, Laguna Hills  
**Project Number:** 23-008-00

# Log of Drill Hole DH-5

Sheet 1 of 3

Date(s) Drilled	1/20/23	Logged By	DW	Checked By		
Drilling Method	Hollow Stem Auger	Drilling Contractor	2R Drilling	Total Depth of Drill Hole	51.0 feet	
Drill Rig Type	CME 75	Diameter(s) of Hole, inches	8	Approx. Surface Elevation, ft MSL	374.0	
Groundwater Depth [Elevation], feet	NA	Sampling Method(s)	Open drive sampler with 6-inch sleeve, Bulk	Drill Hole Backfill	Native	
Remarks					Driving Method and Drop	Auto Hammer

ELEVATION, feet	DEPTH, feet	GRAPHIC LOG	GEOLOGICAL CLASSIFICATION AND DESCRIPTION	ORIENTATION DATA	ENGINEERING CLASSIFICATION AND DESCRIPTION	SAMPLE DATA			TEST DATA		
						SAMPLE	NUMBER OF BLOWS / 6"	DRIVING WEIGHT, lbs	MOISTURE CONTENT, %	DRY UNIT WEIGHT, pcf	ADDITIONAL TESTS
370	5		<u>ARTIFICIAL FILL (Qaf)</u>  Lifts of brown silty sand, pinhole porosity		ASPHALT CONCRETE - 3.5-inches AGGREGATE BASE - 6.0-inches SANDY SILT (ML); olive, damp, medium dense, fine- to medium-grained sand  Becomes yellow with olive and brown, trace gravel	X	10 13 13	140	12	110	
365	10		Moderate rig chatter, fine-grained silty sandstone fragments		CLAYEY SAND (SC); yellow and olive, moist, dense, fine- to coarse-grained sand, some gravel		27 40 40	140	7	123	
360	15		<u>SLOPEWASH/COLLUVIUM (Qsw/Qcol)</u>  Grass pieces		SANDY SILT (ML); brownish gray, moist, very stiff, fine- to medium-grained sand, trace gravel		5 9 10	140	20	92	CN
355											

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**Drill Hole DH-5**

Project: Buchanan Street Partners  
 Project Location: 24422 Avenida De La Carlota, Laguna Hills  
 Project Number: 23-008-00

# Log of Drill Hole DH-5

Sheet 2 of 3

ELEVATION, feet	DEPTH, feet	GRAPHIC LOG	GEOLOGICAL CLASSIFICATION AND DESCRIPTION	ORIENTATION DATA	ENGINEERING CLASSIFICATION AND DESCRIPTION	SAMPLE DATA			TEST DATA		
						SAMPLE NUMBER	NUMBER OF BLOWS / 6"	DRIVING WEIGHT, lbs	MOISTURE CONTENT, %	DRY UNIT WEIGHT, pcf	ADDITIONAL TESTS
			Ped surfaces around gravel		SANDY CLAY (CL); brownish gray, moist, firm, fine- to medium-grained sand, trace gravel		10 7 9	140	17 14	93	PS ATT EI pH SU CH MR
			Moderate rig chatter Moderately weathered <b>BEDROCK - NIGUEL FORMATION (Tn)</b>		SANDY SILTSTONE; yellowish brown, damp, moderately hard, fine-grained sand						
			No rig chatter								
350	25		Manganese stained blips - pinhole sized				17 20 48	140	11		
345	30		Some seepage		CLAYEY SILTSTONE; light brown, soft to moderately hard, wet, fine- to medium-grained sand		15 17 32	140	17	114	
340	35		Moderately to well cemented		SANDY SILTSTONE; gray with orange staining, damp, moderately hard, some very fine-grained sand		12 19 23	140	28		
335	40		Becomes unoxidized, faint subhorizontal wavy laminations		CLAYEY SILTSTONE; dark gray, damp, moderately hard		16 24 31	140	26	107	
330											

DH\_REV3 23-008-00.GPJ GMULAB.GPJ 6/20/23



Drill Hole DH-5



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# APPENDIX B

## Geotechnical Laboratory Procedures and Test Results

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## **APPENDIX B**

### **GMU GEOTECHNICAL LABORATORY PROCEDURES AND TEST RESULTS**

#### **MOISTURE AND DENSITY**

Field moisture content and in-place density were determined for selected 6-inch sample sleeve of undisturbed soil material obtained from the drill holes. The field moisture content was determined in general accordance with ASTM Test Method D 2216 by obtaining one-half the moisture sample from each end of the 6-inch sleeve. The in-place dry density of the sample was determined by using the wet weight of the entire sample.

At the same time the field moisture content and in-place density were determined, the soil material at each end of the sleeve was classified according to the Unified Soil Classification System. The results of the field moisture content and in-place density determinations are presented on the right-hand column of the Log of Drill Hole and are summarized on Table B-1. The results of the visual classifications were used for general reference.

#### **PARTICLE SIZE DISTRIBUTION**

As part of the engineering classification of the materials underlying the site, samples of the onsite soils were tested to determine their distribution of particle sizes. The distribution was determined in general accordance with ASTM Test Method D 422 using U.S. Standard Sieve Openings 3", 1.5", 3/4, 3/8, and U.S. Standard Sieve Nos. 4, 10, 20, 40, 60, 100, and 200. In addition, on some samples a standard hydrometer test was performed to determine the distribution of particle sizes passing the No. 200 sieve (i.e., silt and clay-size particles). The results of the tests are contained in this Appendix B. Key distribution categories (% gravel; % sand, etc.) are contained on Table B-1.

#### **ATTERBERG LIMITS**

As part of their engineering classification, samples of the on-site soil and bedrock materials were tested to determine their relative plasticity. This relative plasticity is based on the Atterberg limits determined in general accordance with ASTM Test Method D 4318. The results of these tests are contained in this Appendix B and also Table B-1.

## **EXPANSION TESTS**

To provide a standard definition of one-dimensional expansion, tests were performed on typical on-site soil and bedrock materials in general accordance with ASTM Test Method D 4829. The result from each test is reported as the “expansion index”. The results of these tests are contained in Appendix B and also Table B-1.

## **COMPACTION TEST**

A bulk sample representative of the on-site soils was tested to determine its maximum dry density and optimum moisture content. These compactive characteristics were determined in general accordance with ASTM Test Method D 1557. The results of this test are contained in this Appendix B and also Table B-1.

## **CHEMICAL TESTS**

The corrosion potential of typical on-site soil and bedrock materials under long-term contact with both metal and concrete was determined by chemical and electrical resistance tests. The soluble sulfate test for potential concrete corrosion was performed in general accordance with California Test Method 417, the minimum resistivity test for potential metal corrosion was performed in general accordance with California Test Method 643, and the concentration of soluble chlorides was determined in general accordance with California Test Method 422. The results of these tests are contained in Table B-1.

## **CONSOLIDATION TESTS**

The one-dimensional consolidation properties of “undisturbed” samples of the onsite slopewash/colluvial materials were evaluated in general accordance with the provisions of ASTM Test Method D 2435. Sample diameter was 2.416 inches and sample height was 1.00 inch. Water was added during the test at various normal loads to evaluate the potential for hydro-collapse and to produce saturation during the remainder of the testing. Consolidation readings were taken regularly during each load increment until the change in sample height was less than approximately 0.0001 inch over a two-hour period. The graphic presentation of consolidation data is a representation of volume change in change in axial load. The results of these tests are contained in this Appendix B.

## **DIRECT SHEAR STRENGTH TESTS**

Direct shear tests were performed on typical on-site bedrock materials. The general philosophy and procedure of the tests were in accord with ASTM Test Method D 3080 - "Direct Shear Tests for Soils Under Consolidated Drained Conditions".

The tests were single shear tests and were performed using a sample diameter of 2.416 inches and a height of 1.00 inch. The normal load was applied by a vertical dead load system. A constant rate of strain was applied to the upper one-half of the sample until failure occurred. Shear stress was monitored by a strain gauge-type precision load cell and deflection was measured with a digital dial indicator. This data was transferred electronically to data acquisition software which plotted shear strength vs. deflection. The shear strength plots were then interpreted to determine peak and ultimate shear strengths. Strain rates compatible with the grain size distribution of the soils was utilized. The interpreted results of these tests are shown in this Appendix B.

## **R-VALUE TEST**

A bulk sample representative of the onsite soils was tested to measure the response of a compacted sample to a vertically applied pressure under specific conditions. The R-value of a material is determined when the material is in a state of saturation such that water will be exuded from the compacted test specimen when a 16.8 kN load (2.07 MPa) is applied. The results from this test procedure are reported in this Appendix B-1.

**TABLE B-1  
SUMMARY OF SOIL LABORATORY DATA**

Sample Information			Geologic Unit	USCS Group Symbol	In Situ Water Content, %	In Situ Dry Unit Weight, pcf	In Situ Saturation, %	Sieve/Hydrometer				Atterberg Limits			Compaction		Expansion Index	R-Value	Chemical Test Results			
Boring Number	Depth, feet	Elevation, feet						Gravel, %	Sand, %	<#200, %	<2µ, %	LL	PL	PI	Maximum Dry Unit Weight, pcf	Optimum Water Content, %			pH	Sulfate (ppm)	Chloride (ppm)	Min. Resistivity (ohm/cm)
DH-1	5	367.0	Tn	Siltstone	15.9																	
DH-1	10	362.0	Tn	Siltstone	11.7	110	62															
DH-1	15	357.0	Tn	Siltstone	14.6																	
DH-1	20	352.0	Tn	Siltstone	16.1	108	80															
DH-1	25	347.0	Tn	Siltstone	17.0																	
DH-1	30	342.0	Tn	Siltstone	15.7	109	80															
DH-2	1	371.0	Qafc	CL	20.1			5	21	74	21	43	23	20	114.0	14.5	35	17	8.7	161	492	1459
DH-2	5	367.0	Qsw/Qcol	ML	16.1	96	59															
DH-2	10	362.0	Qsw/Qcol	CL	20.3	98	78															
DH-2	15	357.0	Tn	Siltstone	14.5	108	72															
DH-2	20	352.0	Tn	Siltstone	16.8																	
DH-2	25	347.0	Tn	Siltstone	13.2	109	68															
DH-2	30	342.0	Tn	Siltstone	15.8																	
DH-3	5	371.0	Tn	Siltstone	12.3	113	70															
DH-3	10	366.0	Tn	Siltstone	15.6																	
DH-3	15	361.0	Tn	Siltstone	12.0	108	60															
DH-3	20	356.0	Tn	Siltstone	13.4																	
DH-3	25	351.0	Tn	Siltstone	14.6	107	71															
DH-3	30	346.0	Tn	Siltstone	15.9																	
DH-4	5	368.0	Qafc	ML	15.6	97	59															
DH-4	10	363.0	Qsw/Qcol	CL	14.1	93	48															
DH-4	15	358.0	Qsw/Qcol	ML	11.1																	
DH-4	20	353.0	Tn	Siltstone	15.3	108	76															
DH-4	25	348.0	Tn	Siltstone	17.4																	
DH-4	30	343.0	Tn	Siltstone	16.1	106	76															

GMU\_TABLE\_SOIL\_LAB\_DATA\_23-008-00.GPJ\_FNC\_AB\_GWGN01.GDT\_7/2/23

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Project No. 23-008-00



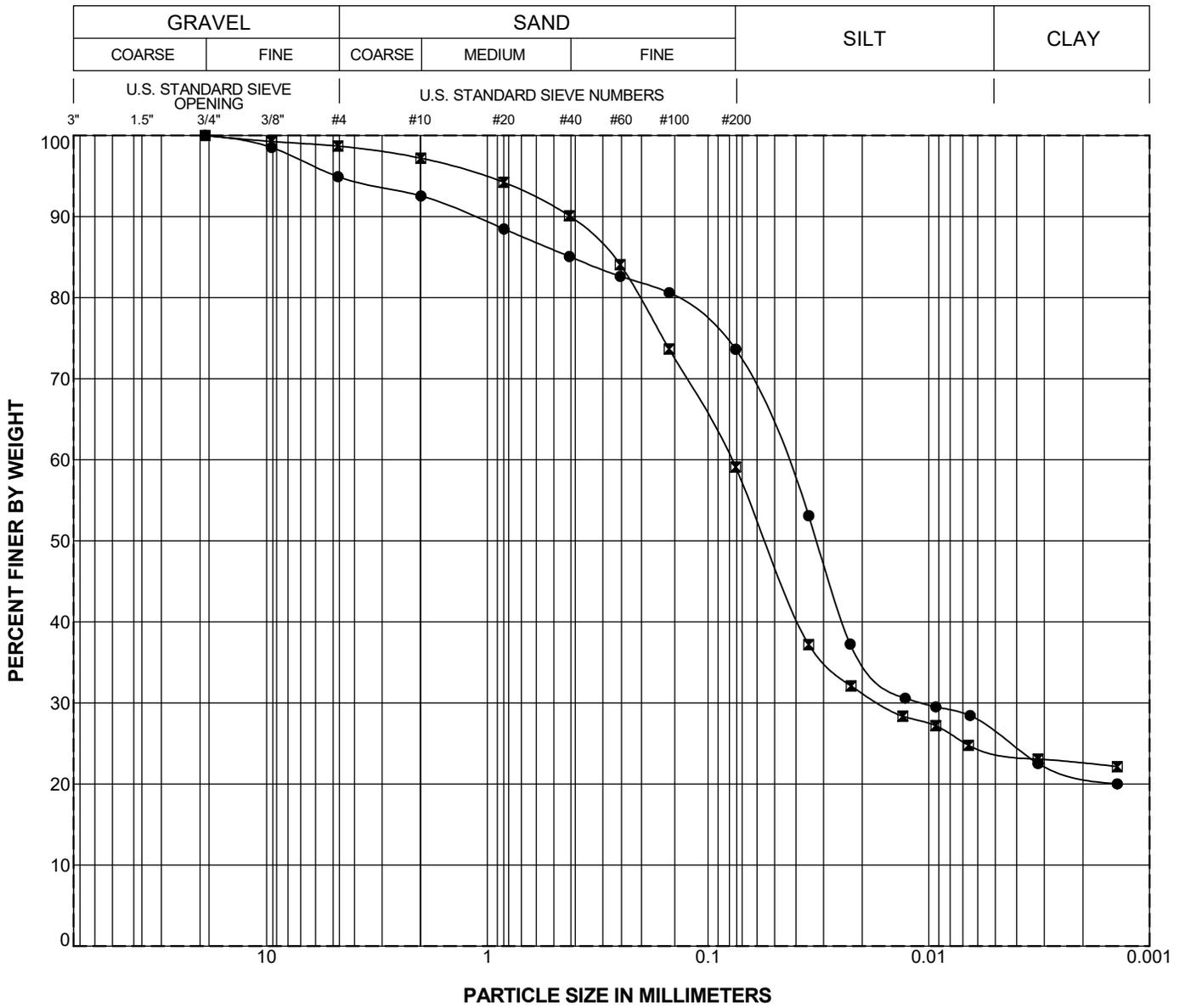
**TABLE B-1  
SUMMARY OF SOIL LABORATORY DATA**

Sample Information			Geologic Unit	USCS Group Symbol	In Situ Water Content, %	In Situ Dry Unit Weight, pcf	In Situ Saturation, %	Sieve/Hydrometer				Atterberg Limits			Compaction		Expansion Index	R-Value	Chemical Test Results			
Boring Number	Depth, feet	Elevation, feet						Gravel, %	Sand, %	<#200, %	<2µ, %	LL	PL	PI	Maximum Dry Unit Weight, pcf	Optimum Water Content, %			pH	Sulfate (ppm)	Chloride (ppm)	Min. Resistivity (ohm/cm)
DH-4	35	338.0	Tn	Siltstone	18.4																	
DH-4	40	333.0	Tn	Siltstone	14.8	107	72															
DH-5	5	369.0	Qafc	CL	12.1	110	64															
DH-5	10	364.0	Qafc	SC	7.5	123	58															
DH-5	15	359.0	Qsw/Qcol	SM-ML	19.8	92	66															
DH-5	20	354.0	Qsw/Qcol	SM/SC	17.2	93	59															
DH-5	20.2	353.8	Qsw/Qcol	CL	13.7			1	40	59	23	32	18	14		56	7.8	131	468	716		
DH-5	25	349.0	Tn	Siltstone	11.4																	
DH-5	30	344.0	Tn	Siltstone	13.1	114	77															
DH-5	35	339.0	Tn	Siltstone	18.4																	
DH-5	40	334.0	Tn	Siltstone	15.7	107	76															
DH-5	45	329.0	Tn	Siltstone	16.4																	
DH-5	50	324.0	Tn	Siltstone	15.2	109	78															

GMU\_TABLE\_SOIL\_LAB\_DATA\_23-008-00.GPJ\_FNC\_AB\_GWGN01.GDT\_7/2/23

Project: Buchanan Street Partners  
Project No. 23-008-00





Boring Number	Depth (feet)	Geologic Unit	Symbol	LL	PI	Classification
DH-2	1.0	Qafc	●	43	20	LEAN CLAY with SAND (CL)
DH-5	20.2	Qsw/Qcol	⊠	32	14	SANDY LEAN CLAY (CL)

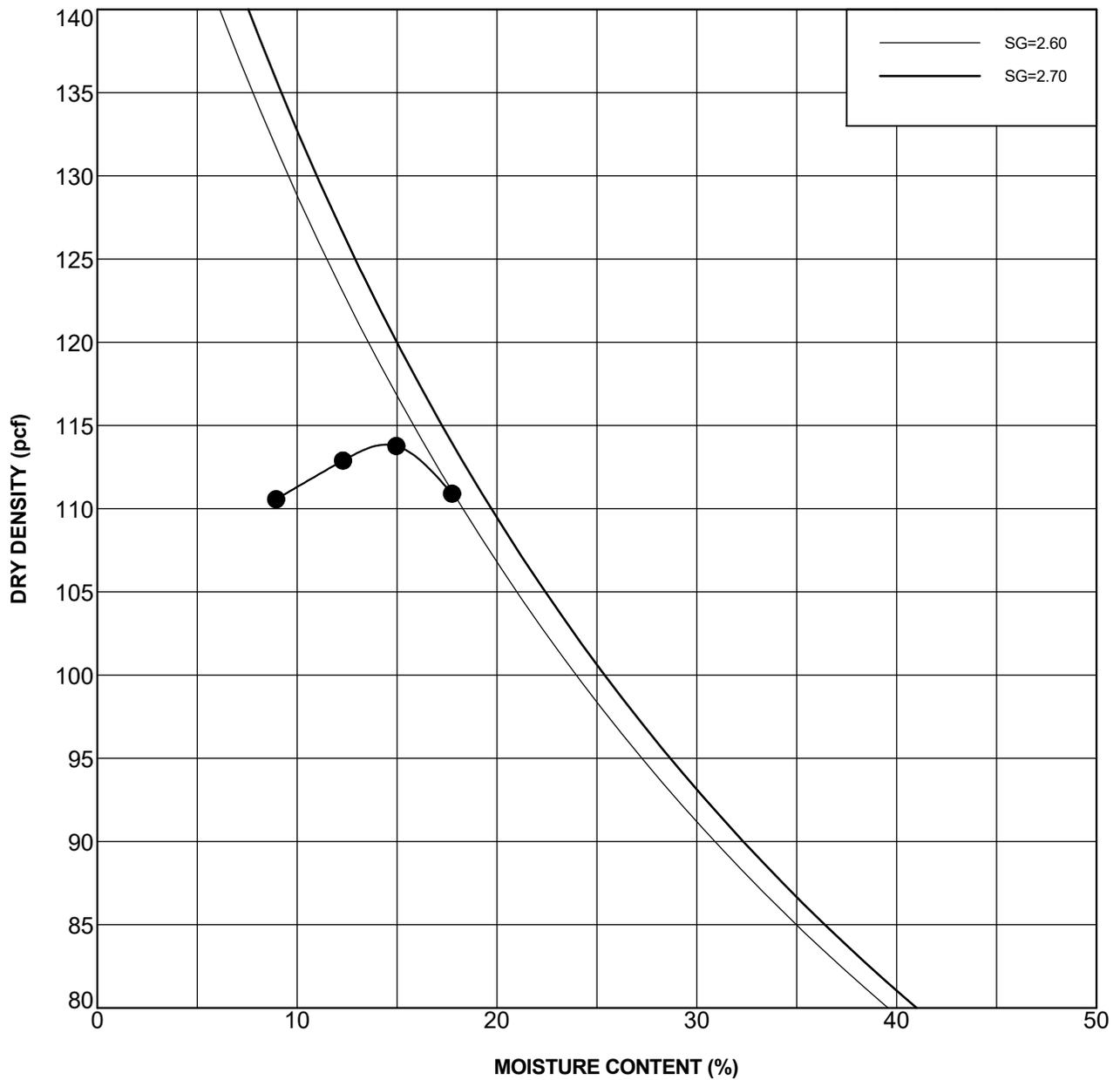
GMU\_GRAIN\_SIZE\_23-008-00.GPJ 6/20/23

## PARTICLE SIZE DISTRIBUTION

Project: Buchanan Street Partners  
Project No. 23-008-00





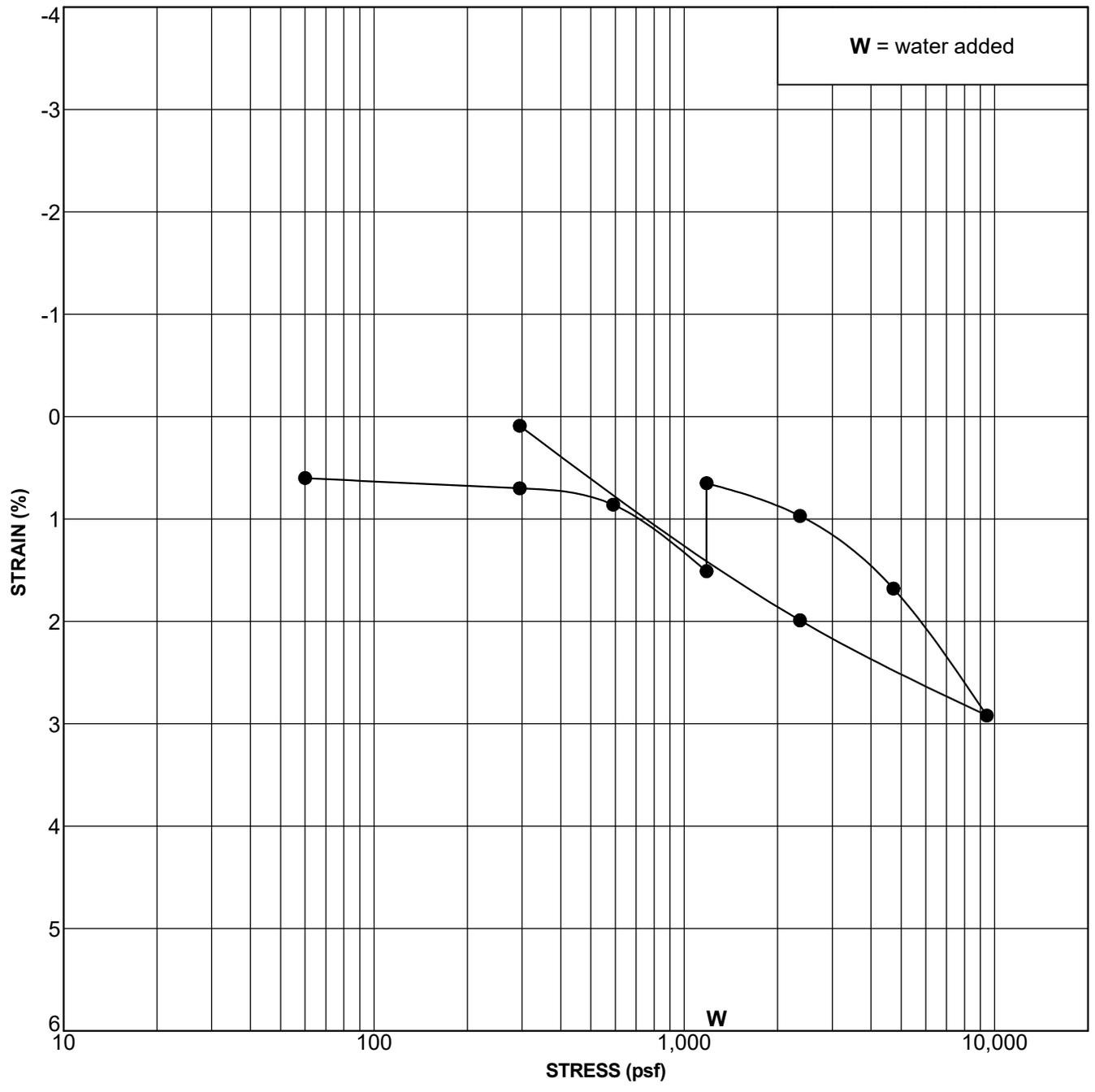


Boring Number	Depth (feet)	Geologic Unit	Symbol	Maximum Dry Density, pcf	Optimum Moisture Content, %	Classification
DH-2	1.0	Qafc	●	114	14.5	LEAN CLAY with SAND (CL)

## COMPACTION TEST DATA

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 Project No. 23-008-00





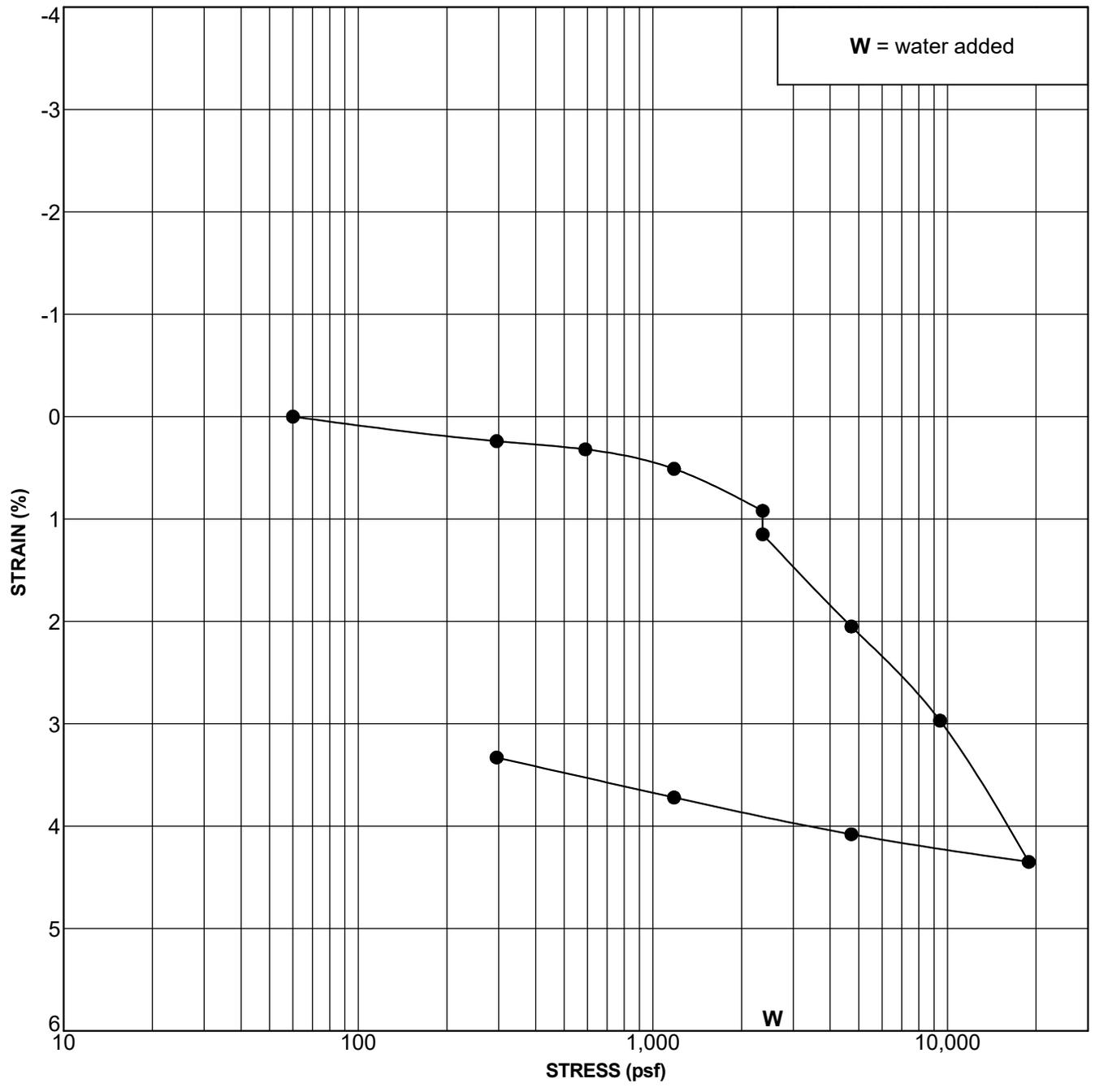
GMU\_CONSOL 23-008-00.GPJ GM&U.GDT 6/20/23

Boring Number	Depth (feet)	Geologic Unit	Symbol	In Situ or Remolded Sample	% Hydro-Collapse	Classification
DH-4	10.0	Qsw/Qcol	●	In Situ	63.49	SANDY CLAY (CL)

## CONSOLIDATION TEST DATA

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 Project No. 23-008-00





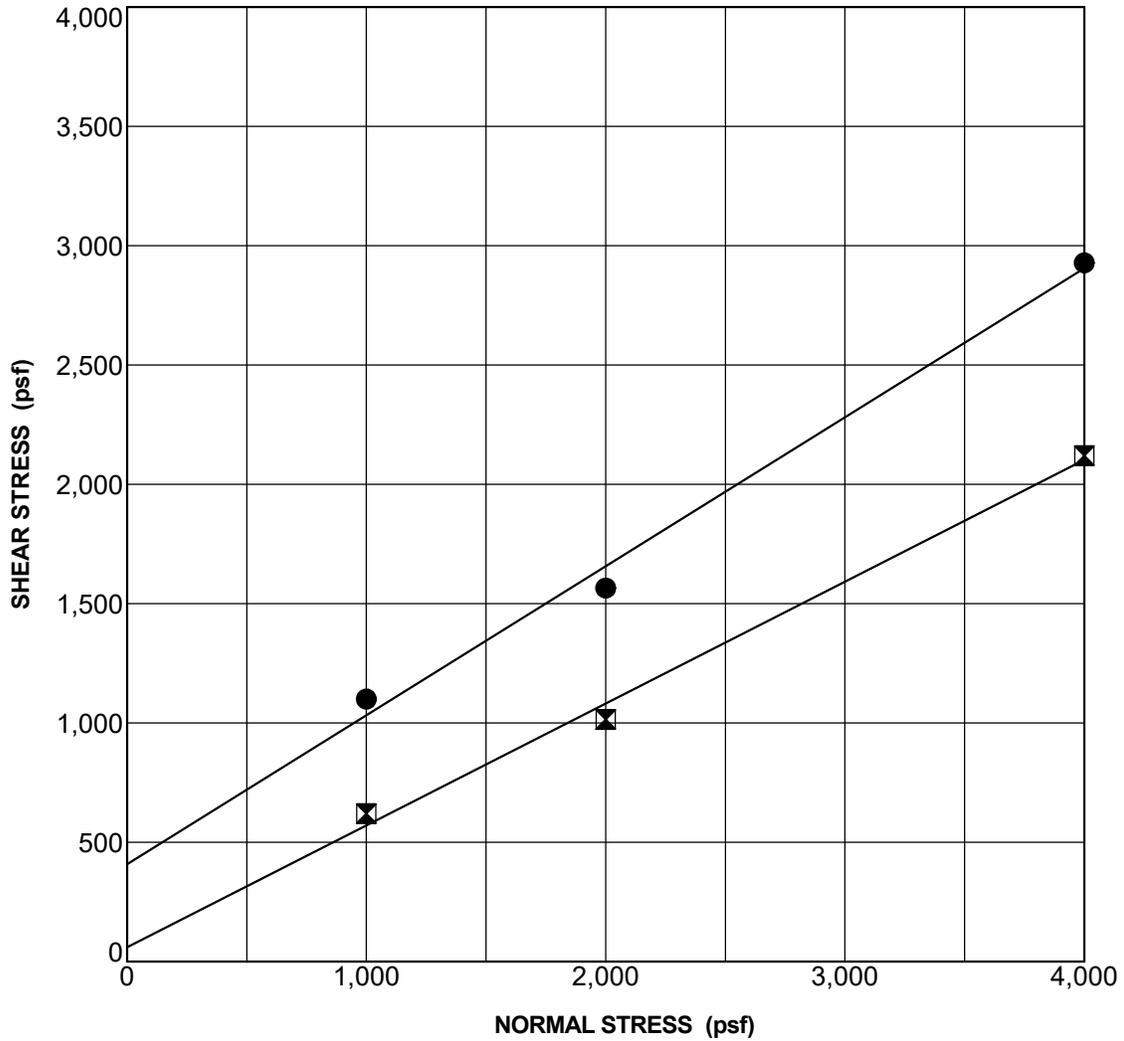
GMU\_CONSOL 23-008-00.GPJ GM&U.GDT 6/20/23

Boring Number	Depth (feet)	Geologic Unit	Symbol	In Situ or Remolded Sample	% Hydro-Collapse	Classification
DH-5	15.0	Qsw/Qcol	●	In Situ	0.23	SILTY SAND (SM-ML)

## CONSOLIDATION TEST DATA

Project: Buchanan Street Partners  
Project No. 23-008-00





**SAMPLE AND TEST DESCRIPTION**

**Sample Location:** DH-2 @ 15.0 ft    **Geologic Unit:** Tn    **Classification:** SANDY SILTSTONE  
**Strain Rate (in/min):** 0.001    **Sample Preparation:** Undisturbed  
**Notes:** Sample saturated prior and during shearing

**STRENGTH PARAMETERS**

STRENGTH TYPE	COHESION (psf)	FRICTION ANGLE (degrees)
● Peak Strength	420	32.0
✕ Ultimate Strength	210	29.0

**SHEAR TEST DATA**

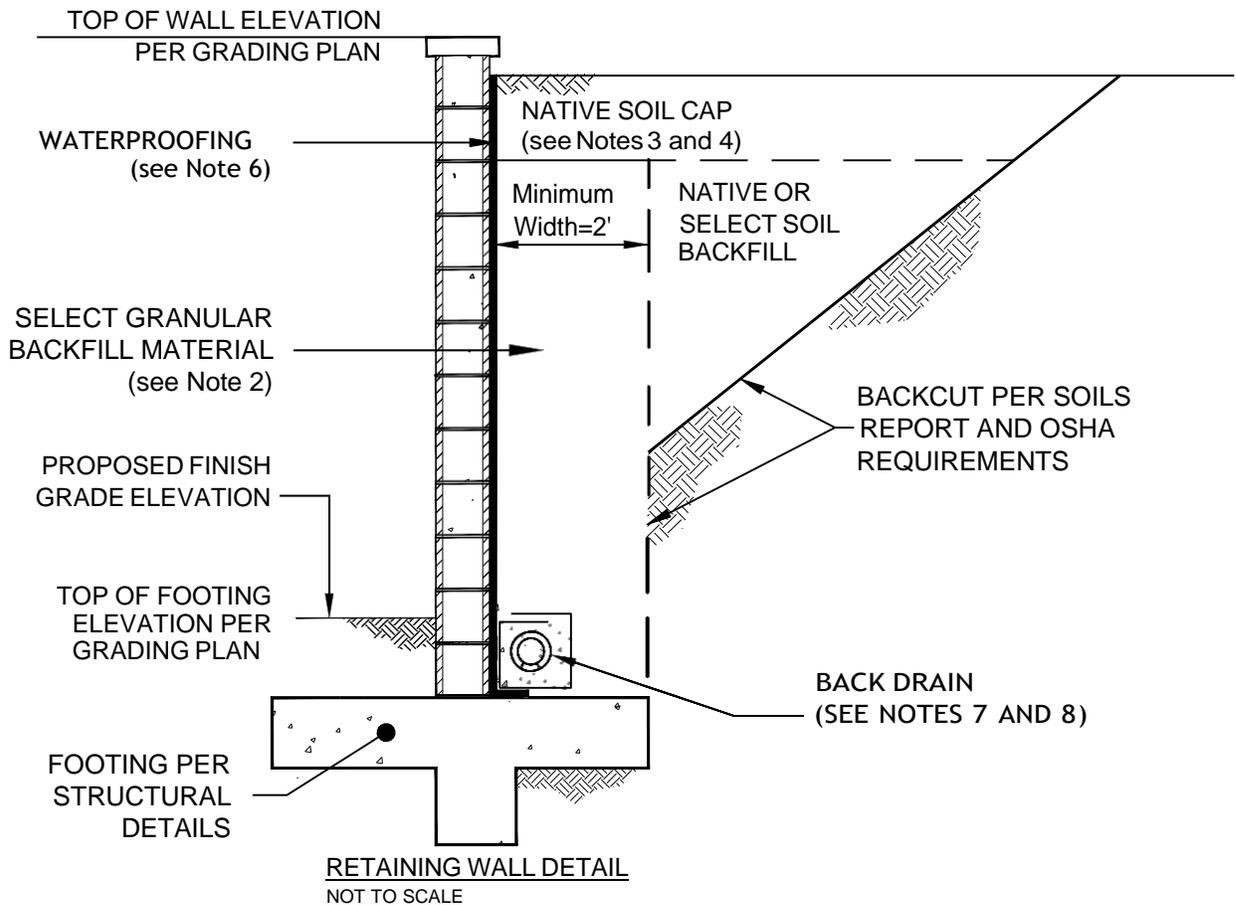
Project: Buchanan Street Partners  
Project No. 23-008-00

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# APPENDIX C

## Retaining Wall Construction Details

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1. FINAL DETERMINATION OF THE MATERIAL TO BE USED FOR BACKFILL SHALL BE MADE BY GMU.
2. ALL SELECT BACKFILL TO WITHIN 1 TO 2 FEET OF FINAL GRADE SHOULD CONSIST OF FREE-DRAINING GRANULAR MATERIAL (I.E. SE 30 SAND, PEA GRAVEL, OR CRUSHED ROCK). CRUSHED ROCK, IF USED, SHOULD BE WRAPPED IN FILTER FABRIC (MIRAFI 140N OR EQUIVALENT) TO MINIMIZE THE POTENTIAL FOR MIGRATION OF FINES INTO THE ROCK. THE SELECT BACKFILL SHOULD BE MOISTURE CONDITIONED TO ACHIEVE OVER OPTIMUM MOISTURE CONTENT PER THE SOILS REPORT AND COMPACTED TO AT LEAST 90% RELATIVE COMPACTION AS DETERMINED BY ASTM TEST METHOD D 1557.
3. FINE-GRAINED NATIVE SOILS SHOULD BE USED TO CAP THE SELECT BACKFILL ZONE.
4. ALL NATIVE OR SELECT SOIL WALL BACKFILL SHOULD BE MOISTURE CONDITIONED AS NECESSARY TO OVER OPTIMUM MOISTURE CONTENT PER THE SOILS REPORT AND COMPACTED TO AT LEAST 90% RELATIVE COMPACTION AS DETERMINED BY ASTM TEST METHOD D 1557.
5. THE BACKSIDE OF THE WALLS SHOULD BE WATERPROOFED DOWN TO AND ACROSS THE TOP OF THE FOOTING. THE DESIGN AND SELECTION OF THE WATERPROOFING SYSTEM IS OUTSIDE OF THE PURVIEW OF GMU.
6. THE WATERPROOFING SYSTEM AND ANY DRAIN BOARDS SHOULD BE PROTECTED FROM DAMAGE BY CONSTRUCTION ACTIVITIES. THE TOP EDGE OF THE WATERPROOFING AND ANY DRAIN BOARDS SHOULD BE PROPERLY ADHERED TO THE WALL AND SEALED TO PREVENT THE POSSIBLE ACCUMULATION OF DEBRIS BETWEEN THE DRAINAGE/WATERPROOFING SYSTEM AND THE WALL.
7. THE BACKDRAIN SYSTEM SHOULD CONSIST OF 4" PERFORATED PIPE SURROUNDED BY AT LEAST ONE CUBIC FOOT OF 3/4"-1.5" OPEN GRADED GRAVEL WRAPPED IN MIRAFI 140N FILTER FABRIC (OR EQUIVALENT). THE PERFORATED PIPE SHOULD CONSIST OF SDR-35 OR SCHEDULE 40 PVC PIPE (OR APPROVED EQUIVALENT) LAID ON AT LEAST 2" OF CRUSHED ROCK WITH THE PERFORATIONS LAID DOWN. THE BACKDRAIN GRADIENT SHOULD NOT BE LESS THAN 1% WHEN POSSIBLE. THE PERFORATED PIPE SHOULD OUTLET INTO AREA DRAINS OR OTHER SUITABLE OUTLET POINTS AT RUNS OF 200 FEET OR LESS, IF PRACTICAL. IF THE BACKDRAINS CANNOT BE OUTLETED BY GRAVITY FLOW, A SUMP PUMP SYSTEM WILL NEED TO BE DESIGNED AND CONSTRUCTED. REDUNDANT BACK-UP PUMPS OR COMPONENTS ARE RECOMMENDED. DESIGN OF THIS SYSTEM IS OUTSIDE OF THE PURVIEW OF GMU.
8. THE TIE-IN LOCATIONS FOR BACKDRAIN OUTLETS SHOULD BE SHOWN ON THE PRECISE GRADING, SITE WALL, AND/OR LANDSCAPE PLANS.

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# APPENDIX D

## Concrete Flatwork Recommendations

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**TABLE 1  
FLATWORK RECOMMENDATIONS, RESIDENTIAL SITE,  
24422 AVENIDA DE LA CARLOTA, LAGUNA HILLS, CA**

Description	Subgrade Preparation	Minimum Concrete Thickness (Full)	Edge Thickness	Reinforcement <sup>(2)</sup>	Joint Spacing (Maximum)	Cement Type	Sulfate Resistance
Isolated Concrete Sidewalks and Walkways (≤6 feet in width) <sup>(4)</sup>	1) 2% over optimum to 18" <sup>(1)</sup> , 2) 2" of sand or well graded rock (i.e., Class II base or equiv.) above moisture conditioned subgrade.	4 inches	Not Required	1) No. 3 bars at 18" o.c. <sup>(2)</sup> , 2) where adjacent to curbs or structures and at cold joints/ expansion joints use dowels: No. 3 bars at 18" o.c. <sup>(5)</sup>	5 feet	II/V	(3)
Concrete Walkways, Patios, Entryways and Courtyards (> 6 feet in width) <sup>(4)</sup>	1) 2% over optimum to 18" <sup>(1)</sup> , 2) 2" of sand or well graded rock (i.e., Class II base or equiv.) above moisture conditioned subgrade.	5 inches	Where adjacent to landscape areas – 12" from adjacent finish grade. Min. 8" width	1) No. 3 bars at 18" o.c. <sup>(2)</sup> extend into thickened edge, 2) Thickened Edge: one No. 3 bar placed in long direction, 3) dowel into adjacent curbs or structures and across cold joints/ expansion joints <sup>(5)</sup>	8 feet	II/V	(3)
Concrete Driveways, Trash Enclosures and Fire Access Lanes <sup>(4)</sup>	1) 2% over optimum to 18" <sup>(1)</sup> , 2) 4" of well graded rock (i.e., Class II base or equiv.) above moisture conditioned subgrade.	8 inches	Where adjacent to landscape areas - 12" from adjacent finish grade. Min. 8" width	1) No. 3 bars at 18" o.c. <sup>(2)</sup> extend into thickened edge, 2) Thickened Edge: one No. 3 bar placed in long direction, 3) dowel into adjacent curbs or structures and across cold joints/ expansion joints <sup>(5)</sup>	8 feet	II/V	(3)
Concrete Interlocking Pavers (non-vehicular) <sup>(4,6)</sup>	1) 2% over optimum to 18" <sup>(1)</sup> , 2) 4 inches of CAB or CMB compacted to a minimum of 95% relative compaction or concrete sub slab may be used in lieu of base section (see adjacent columns).	4 inch concrete sub slab if base section not used	Where adjacent to landscape areas - 12" from adjacent finish grade. Min. 8" width	1) No. 3 bars at 18" o.c. <sup>(2)</sup> extend into thickened edge, 2) Thickened Edge: one No. 3 bar placed in long direction, 3) dowel into adjacent curbs or structures and across cold joints/ expansion joints <sup>(5)</sup>	8 feet	II/V	(3)
Concrete Interlocking Pavers (vehicular) <sup>(4, 6)</sup>	Subgrade: 2% over optimum to 18" <sup>(1)</sup> <u>Non-fire access:</u> 8 inches of CAB or CMB compacted to 95% relative compaction over Mirafi 600x or equivalent geotextile, or concrete sub slab may be used in lieu of base/geotextile section (see adjacent column) <u>Fire Access:</u> 10 inches of CAB or CMB compacted to 95% over 600X or equivalent geotextile, or concrete sub slab may be utilized in lieu of base/geotextile section (see adjacent column)	<u>Non-Fire Access:</u> 5-inch concrete sub slab if base section not used <u>Fire Access:</u> 6-inch concrete sub slab if base section not used	Where adjacent to landscape areas - 12" from adjacent finish grade. Min. 8" width	1) No. 3 bars at 18" o.c. <sup>(2)</sup> extend into thickened edge, 2) Thickened Edge: one No. 3 bar placed in long direction, 3) dowel into adjacent curbs or structures and across cold joints/ expansion joints <sup>(5)</sup>	8 feet	II/V	(3)

- (1) The moisture content of the subgrade must be verified by the geotechnical consultant prior to sand/rock placement.
- (2) Reinforcement to be placed at or above the mid-point of the slab (i.e., a minimum of 2.0 to 2.5 inches above the prepared subgrade).
- (3) Soils having severe levels of sulfates as defined by CBC are expected. Concrete mix design shall be selected by the concrete designer. Concrete mix design is outside the geotechnical engineer's purview.
- (4) Where concrete/ flatwork is adjacent a stucco surface, a ¼" to ½" foam separation/expansion joint should be used.
- (5) If dowels are placed in cored holes, the core holes shall be placed at alternating in-plane angles (i.e., not cored straight into slab).
- (6) Pavers to be installed per minimum manufacturers recommendations.

General Note: Minor deviations to the above recommendations may be required at the discretion of the soils engineer or his representative.