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GEOTECHNICAL
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MATERIALS

REVISED PRELIMINARY GEOTECHNICAL INVESTIGATION

BONITA SELF STORAGE
5780 QUARRY ROAD
BONITA, CALIFORNIA

PREPARED FOR

INSITE PROPERTY GROUP
REDONDO BEACH, CALIFORNIA

OCTOBER 8, 2021
PROJECT NO. G2674-42-01

Project No. G2674-42-01
October 8, 2021

InSite Property Group
811 N. Catalina Avenue, Suite 1360
Redondo Beach, California 90227

Attention: Ms. Annie Baek

Subject: REVISED PRELIMINARY GEOTECHNICAL INVESTIGATION
BONITA SELF STORAGE
5780 QUARRY ROAD
BONITA, CALIFORNIA

Dear Ms. Baek:

In accordance with your request, we have prepared a revised preliminary geotechnical investigation for the subject project. This report includes the findings of our investigation and our conclusions and recommendations pertaining to geotechnical aspects of developing the property as newly proposed.

It is our opinion that the site can be developed as currently proposed, provided the recommendations of this report are followed.

Should you have questions regarding this report, or if we may be of further service, please contact the undersigned at your convenience.

Very truly yours,

GEOCON INCORPORATED

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REVISED PRELIMINARY GEOTECHNICAL INVESTIGATION

1. PURPOSE AND SCOPE

This report includes the results of our preliminary geotechnical investigation for the proposed self-storage building and RV parking lot planned at 5780 Quarry Road, Bonita, California (*see Vicinity Map*, Figure 1). The purpose of this study is to evaluate surface and subsurface soil conditions, general site geology, and to identify geotechnical constraints that may impact development of the property.

To prepare this geotechnical report, we reviewed the following report and plans:

1. *Preliminary Grading Plan, Bonita, California*, prepared by Kimley-Horn, print date August 11, 2021.

2. *ALTA/NSPS Land Title Survey, 5780 – 5790 Quarry Road, Bonita, California,* prepared by BWE, dated January 27, 2021 (Project No. 6927s).

The scope of our field investigation included a site reconnaissance, excavation and logging of twelve exploratory test pits, drilling five, small-diameter borings, performing 4 infiltration tests in areas of proposed storm water basins or other storm water management devices, and reviewing published and unpublished geologic literature and reports (see List of References). Appendix A presents a discussion of our field investigation. We performed laboratory tests on soil samples obtained from the exploratory test pits and borings to evaluate pertinent physical properties for engineering analyses. The results of laboratory tests are presented in Appendix B.

The locations of exploratory trenches are shown the Geologic Map (*see* Figure 2). The base map used to generate Figure 2 was from a CAD file provided by Kimley Horn. Appendix A presents the exploratory test pit logs and details of the field investigation. Appendix B presents details of the laboratory tests and a summary of test results. Appendix C provides storm water management recommendations. Appendix D presents recommended grading specifications.

The conclusions and recommendations presented herein are based on our analysis of the data obtained from the exploratory field investigation, laboratory test results, and our experience with similar soil and geologic conditions on the subject property and adjacent properties.

2. SITE AND PROJECT DESCRIPTION

The site consists of an approximately 10.7-acre trapezoidal-shaped vacant lot located at 5780 Quarry Road in Bonita, California (*see* Figure 1). The site is bounded by Quarry Road to the west, existing residential homes to the south, and County of San Diego open space to the north and east. Site topography is undulatory and gently sloping, ranging in elevation from approximately 135 feet Mean Sea Level (MSL) at the southeast corner to approximately 195 feet MSL to northeast corner. A storm drain line and sewer main are present along the western property boundary. Several public and private utility easements cross or boarder the site.

Based on our review of historical aerial photographs, several phases of development have occurred on the property. Until circa 1986, the property was used as farm/agricultural lands. Several farm buildings are visible in historical photographs but were demolished between 1986 and 1990. Foundation remnants from the farm buildings may be buried on the site.

Based on the referenced preliminary grading plan and discussions with you, new proposed development includes constructing an RV parking lot and a three-story storage facility building, with associated utilities, drive aisles, customer parking, hardscape and landscape. The lower two floors of the storage building will be subterranean with a lower floor elevation of 147.3 feet Mean Sea Level (MSL) and a basement finish floor elevation of 135 feet MSL. A review of the preliminary grading plan indicates maximum cuts and fills on the order of approximately 20 feet and 10 feet, respectively.

The descriptions above are based on a review of the referenced plans. If development plans differ significantly from those described herein, Geocon Incorporated should be contacted for review and possible revisions to this report.

3. SOIL AND GEOLOGIC CONDITIONS

Based on observations during our subsurface investigation, the site is underlain by surficial deposits consisting of undocumented fill and topsoil overlying metavolcanic rock. The soil and geologic units are described below. The lateral extent of soil and geologic units is shown on the Geologic Map, Figure 2 and on the Geologic Cross-Sections, Figure 3.

3.1 Undocumented Fill (Qudf)

We encountered existing fills during our field investigation in all of the exploratory trenches, except T-1, T-3, and T-6. The fill thicknesses range from approximately 1 to 3.5 feet. Deeper areas of fill may be present within the areas of utility easements, or in unexplored areas of the site. Fill is also present in the slope along the western property boundary. We could not find any compaction reports or documentation for fill present on the site; therefore, the fill is considered undocumented from a geotechnical standpoint.

The undocumented fill encountered in our exploratory trenches varied from loose to medium dense, moist to wet, silty to clayey sand and soft, sandy silt. Some debris such as plastic, glass, wood, and metal is present within the fill. Several end-dumped stockpiles of concrete rumble, asphalt, soil, and construction materials are also present in the northern half of the site.

Laboratory expansion index testing indicates the fill possess a “medium” expansion potential (EI of less than 90). The undocumented fill is unsuitable for support of settlement sensitive structures and improvements and will require complete removal, screening (for trash), and placement as compacted fill during site grading. End-dumped stockpiles of trash and miscellaneous construction materials are unsuitable as fill and should be exported from the site.

3.2 Topsoil (Unmapped)

Topsoil underlies the undocumented fill or mantles the site, ranging in thickness from 0.5 to 5.5 feet. The topsoil consists of damp to wet, silty to sandy clay. Laboratory expansion index testing indicates the undocumented fill possess a “medium” expansion potential (EI of less than 90). Remedial grading including complete removal and recompaction of the topsoil will be required in areas receiving improvements.

3.3 Metavolcanic Rock (Mzu)

Jurassic- to Cretaceous-age (Mesozoic) Metavolcanic Rock is present below the surficial soils, varying from weak to strong and completely to moderately weathered rock. Rock outcrops are exposed in the northwest portion of the site. The completely weathered (Saprolite) portions of the rock are generally rippable and can be excavated using conventional grading and trenching equipment. Deeper, less weathered portions of the metavolcanic rock may be marginally rippable or non-rippable. Very heavy ripping, rock breaking, or blasting may be required if planned excavations extend below the weathered portions of the metavolcanic rock, as discussed herein. The Metavolcanic Rock is suitable for support of proposed fill and structural loads.

We performed borings B-1 through B-5 to evaluate potential rippable and non-rippable characteristics of the subterranean portion of the storage facility building. The borings were drilled to depths ranging from 13 feet to 26 feet using an M-10 drill rig equipped with 8-inch diameter hollow stem augers. Based on the preliminary grading plan, we expect excavations for the building foundations will achieve depths of up to approximately 20 feet below existing grade. At depths where hard rock or practical refusal was encountered using the drill rig, the drill rig switched to percussion drilling equipped with a 4-inch hammer bit. Percussion drilling was required in borings B-1 and B-2 due to hard rock conditions. Borings B-3 through B-5 were drilled to the estimated building foundation depth without the use of percussion drilling. Figures 2 and 3 depict the approximate areas where drill rig refusal was encountered and marginally rippable to non-rippable rock should be expected. However, hard rock requiring rock breaking techniques could be encountered in other areas of the building pad as rock rippability is a function of natural weathering processes that can vary both vertically and horizontally over short distances depending on jointing, fracturing, and/or mineralogic discontinuities within the bedrock. Perspective contractors should evaluate the geotechnical data and use their own judgment to identify the boundary between rippable and non-rippable rock.

4. GROUNDWATER

We did not encounter groundwater or seepage during our site investigation. However, it is not uncommon for shallow seepage conditions to develop where none previously existed when sites are irrigated or infiltration is implemented. Seepage is dependent on seasonal precipitation, irrigation, land use, among other factors, and varies as a result. Proper surface drainage will be important to future performance of the project. We do not expect groundwater to be encountered during construction of the proposed development.

5. GEOLOGIC HAZARDS

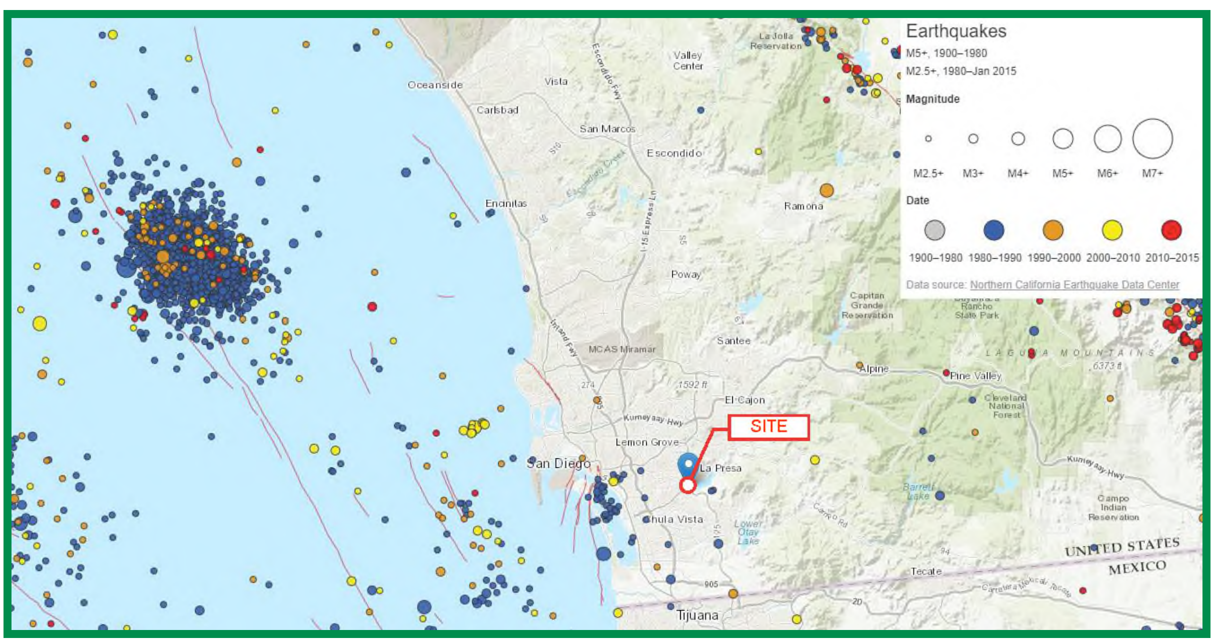
5.1 Faulting and Seismicity

Our review of referenced geologic materials and our knowledge of the area surrounding the site indicates the site is not underlain by active, potentially active, or inactive faults. An active fault is defined by the California Geological Survey (CGS) as a fault showing evidence for activity within the last 11,700 years. The site is not located within a State of California Earthquake Fault Zone.

The USGS has developed a program to evaluate the approximate location of faulting in the area of properties. The following figure shows the location of the existing faulting in the San Diego County and Southern California region. The fault traces are shown as solid, dashed and dotted that represent well-constrained, moderately constrained and inferred, respectively. The fault line colors represent fault with ages less than 150 years (red), 15,000 years (orange), 130,000 years (green), 750,000 years (blue) and 1.6 million years (black).

Faults in Southern California

The San Diego County and Southern California region is seismically active. The following figure presents the occurrence of earthquakes with a magnitude greater than 2.5 from the period of 1900 through 2015 according to the Bay Area Earthquake Alliance website.



Earthquakes in Southern California

Considerations important in seismic design include frequency and duration of motion and soil conditions underlying the site. Seismic design of structures should be evaluated in accordance with the California Building Code (CBC) guidelines currently adopted by the local agency.

5.2 Ground Rupture

Ground surface rupture occurs when movement along a fault is sufficient to cause a gap or rupture where the upper edge of the fault zone intersects the ground surface. The potential for ground rupture is considered to be very low due to the absence of active faults at the subject site.

5.3 Liquefaction

Liquefaction typically occurs when a site is located in a zone with seismic activity, onsite soils are cohesionless or silt/clay with low plasticity, groundwater is encountered within 50 feet of the surface and soil densities are less than about 70 percent of the maximum dry densities. If the four previous criteria are met, a seismic event could result in a rapid pore water pressure increase from the earthquake-generated ground accelerations. Due to the lack of a permanent, near-surface groundwater table and the dense nature of the onsite soils, liquefaction potential for the site is considered very low.

5.4 Landslides

We did not observe evidence of previous or incipient slope instability at the site during our study. Published geologic mapping indicates landslides are not present on or adjacent to the site. Therefore, in our professional opinion, the potential for a landslide is not a significant concern for this project.

5.5 Flooding

The Federal Emergency Management Agency (FEMA 2012) locates the site within a Flood Zone X area, indicating a minimal risk to inundation by 100-year and 500-year floods. However, the site is located near Flood Zones A and AE, which represent possible areas of flooding related to the Sweetwater Reservoir which is located approximately 0.5 miles east of the site.

6. CONCLUSIONS AND RECOMMENDATIONS

6.1 General

6.1.1 No soil or geologic conditions were encountered during our investigation that would preclude the development of the property as presently planned provided the recommendations of this report are followed. From a geotechnical engineering standpoint, it is our opinion that the site is suitable for the proposed development.

6.1.2 The site is underlain by undocumented fill and topsoil overlying metavolcanic rock. Remedial grading including complete removal and recompaction of undocumented fill and topsoil in the area of planned improvements will be necessary. The metavolcanic rock is suitable for support of additional fill and structural loads. Upper portions of the metavolcanic rock may require removal if highly expansive soils (Saprolite) are encountered near finish grade.

6.1.3 Very heavy ripping or blasting may be required if planned excavations extend below the weathered portions of the metavolcanic rock. We expect portions of the excavation to reach the subterranean basement of the proposed storage facility will encounter hard rock and very heavy ripping and/or the use of a rock breaker, may be required.

6.1.4 Several end-dumped stockpiles of concrete rumble, asphalt, soil, and miscellaneous materials were observed in the northern half of the site. Trash, debris, and asphalt cannot be mixed with the fill and must be exported from the site. Concrete rubble and soil may be suitable to incorporate into deep fill areas, provided the oversize hold-down criteria discussed in the grading section of this report are followed.

6.1.5 Clay portions of the topsoil and completely weathered (Saprolite) portions of the metavolanic rock may possess “high” expansive characteristics. Selective grading will be required to provide a cap of at least three feet of soil with “very low” to “medium” expansion potential (EI of 90 or less). If encountered, highly expansive soils should not be placed within 3 feet of finish grade.

6.1.6 Groundwater and/or seepage-related problems are not anticipated, provided that surface drainage is directed into properly designed drainage structures and away from pavement edges, buildings and other moisture-sensitive developments.

6.1.7 We did not encounter groundwater during our subsurface exploration, and we do not expect it to be a constraint during project development. However, seepage within surficial soils materials may be encountered during the grading operations during the rainy seasons.

6.1.8 The risk associated with geologic hazards due to fault-related ground rupture, liquefaction, landslides, and flooding is low.

6.1.9 Based on our review of the project plans, we opine the planned development can be constructed in accordance with our recommendations provided herein. We do not expect the planned development will destabilize or result in settlement of adjacent properties if properly constructed.

6.1.10 Subdrains will not be required on this project with the exception of subdrains for subterranean building levels or retaining walls.

6.1.11 The proposed storage facility building can be supported on a conventional foundation system founded in properly compacted fill or formational bedrock.

6.1.12 Subsurface conditions observed may be extrapolated to reflect general soil/geologic conditions; however, some variations in subsurface conditions between boring locations should be anticipated.

6.1.13 Geocon Incorporated should review the grading and foundation plans prior to finalizing. If plans differ significantly from those described herein, Geocon should be contacted to check if additional analyses will be required.

6.2 Excavation and Soil Characteristics

6.2.1 Excavation of the undocumented fill, topsoil, and completely weathered portions (Saprolite) of the metavolcanic rock should be possible with moderate to heavy effort using conventional heavy-duty equipment. Excavation of the less weathered portions of the metavolcanic rock may require very heavy effort or rock breaking technics (rock breaker or blasting) and may generate oversized material.

6.2.2 The soil encountered in the field investigation is considered to be “expansive” (expansion index [EI] of greater than 20) as defined by 2019 California Building Code (CBC) Section 1803.5.3. Table 6.2 presents soil classifications based on the expansion index. We expect a majority of the soil encountered possess a “low” to “medium” expansion potential (EI of 90 or less).

TABLE 6.2
EXPANSION CLASSIFICATION BASED ON EXPANSION INDEX

Expansion Index (EI)	ASTM D 4829 Expansion Classification	2019 CBC Expansion Classification
0 – 20	Very Low	Non-Expansive
21 – 50	Low	Expansive
51 – 90	Medium	
91 – 130	High	
Greater Than 130	Very High	

6.2.3 We performed laboratory tests on samples of the site materials to evaluate the percentage of water-soluble sulfate content. Appendix B presents results of the laboratory water-soluble sulfate content tests. The test results indicate the on-site materials at the locations tested possess a "S0" sulfate exposure to concrete structures as defined by 2019 CBC Section 1904 and ACI 318-08 Sections 4.2 and 4.3. The presence of water-soluble sulfates is not a visually discernible characteristic; therefore, other soil samples from the site could yield different concentrations. Additionally, over time landscaping activities (i.e., addition of fertilizers and other soil nutrients) may affect the concentration.

6.2.4 Geocon Incorporated does not practice in the field of corrosion engineering. Therefore, further evaluation by a corrosion engineer may be needed if improvements susceptible to corrosion are planned.

6.3 Subdrains

6.3.1 With the exception of subdrains for subterranean building levels or retaining walls, no other subdrains are anticipated.

6.4 Temporary Excavation Slopes, Shoring and Tiebacks

6.4.1 The recommendations included herein are provided for stable excavations. It is the responsibility of the contractor and their competent person to ensure all excavations, temporary slopes and trenches are properly constructed and maintained in accordance with applicable OSHA guidelines in order to maintain safety and the stability of the excavations and adjacent improvements. These excavations should not be allowed to become saturated or to dry out. Surcharge loads should not be permitted to a distance equal to the height of the excavation from the top of the excavation. The top of the excavation should be a minimum of 15 feet from the edge of existing improvements. Excavations steeper than those recommended or closer than 15 feet from an existing surface improvement should be shored in accordance with applicable OSHA codes and regulations.

6.4.2 The stability of the excavations is dependent on the design and construction of the shoring system and site conditions. Therefore, Geocon Incorporated cannot be responsible for site safety and the stability of the proposed excavations.

6.4.3 The design of shoring is governed by soil and groundwater conditions, and by the depth and width of the excavated area. Continuous support of the excavation face can be provided by a system of soldier piles and wood lagging or soil nail walls. Excavations exceeding 15 feet may require soil nails, tieback anchors or internal bracing to provide additional wall restraint. Geocon Incorporated should be contacted if shoring recommendations other than soldier piles and wood lagging or sheet piles will be required.

6.4.4 The condition of existing buildings, streets, sidewalks, and other structures/improvements around the perimeter of the planned excavation should be documented prior to the start of shoring and excavation work. Special attention should be given to documenting existing cracks or other indications of differential settlement within these adjacent structures, pavements, and other improvements. Underground utilities sensitive to settlement should be videotaped prior to construction to check the integrity of pipes. In addition, monitoring points should be established indicating location and elevation around the excavation and upon existing buildings. These points should be monitored on a weekly basis during excavation work, then monthly thereafter.

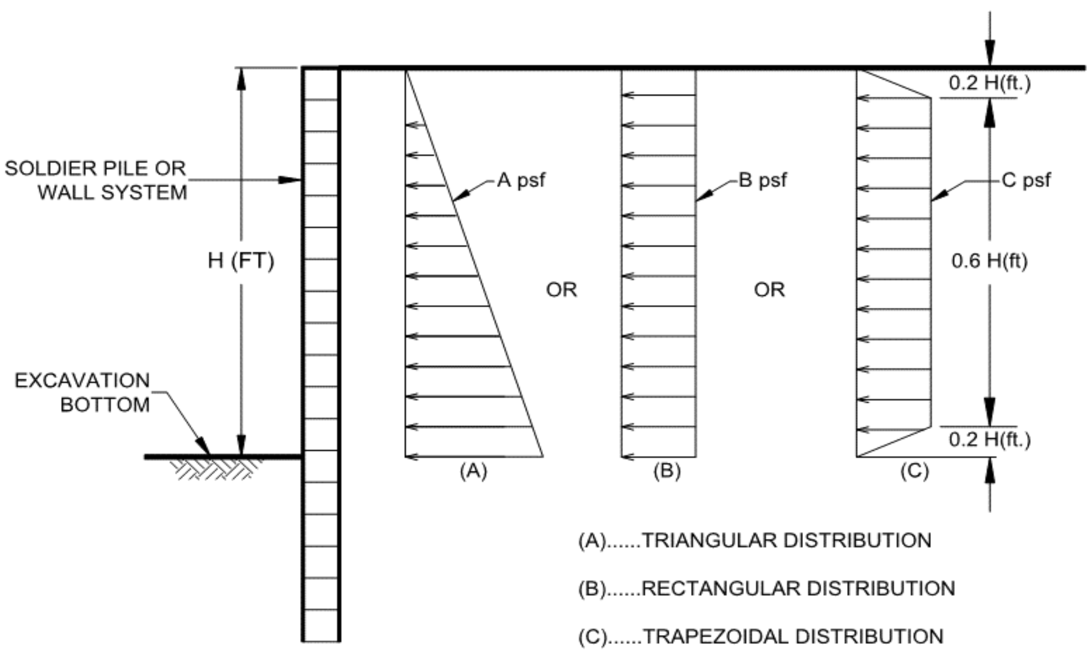
6.4.5 In general, ground conditions are moderately suited for soldier pile and tieback anchor wall construction techniques. However, gravel, cobble, and oversized material may be encountered in the existing materials that could be difficult to drill. Additionally, if cohesionless sands are encountered, some raveling may result along the unsupported portions of excavations.

6.4.6 Shoring with a level backfill should be designed using a lateral pressure envelope acting on the back of the shoring as presented in Table 6.4.1 assuming a level backfill. The distributions are shown on the Active Pressures for shoring. Triangular distribution should be used for cantilevered shoring and, the trapezoidal and rectangular distribution should be used for multi-braced systems such as tieback anchors and rakers. The project shoring engineer should determine the applicable soil distribution for the design of the shoring system. Additional lateral earth pressure due to the surcharging effects from construction equipment, sloping backfill, planned stockpiles, adjacent structures and/or traffic loads should be considered, where appropriate, during design of the shoring system.

TABLE 6.4.1
SUMMARY OF SHORING WALL RECOMMENDATIONS

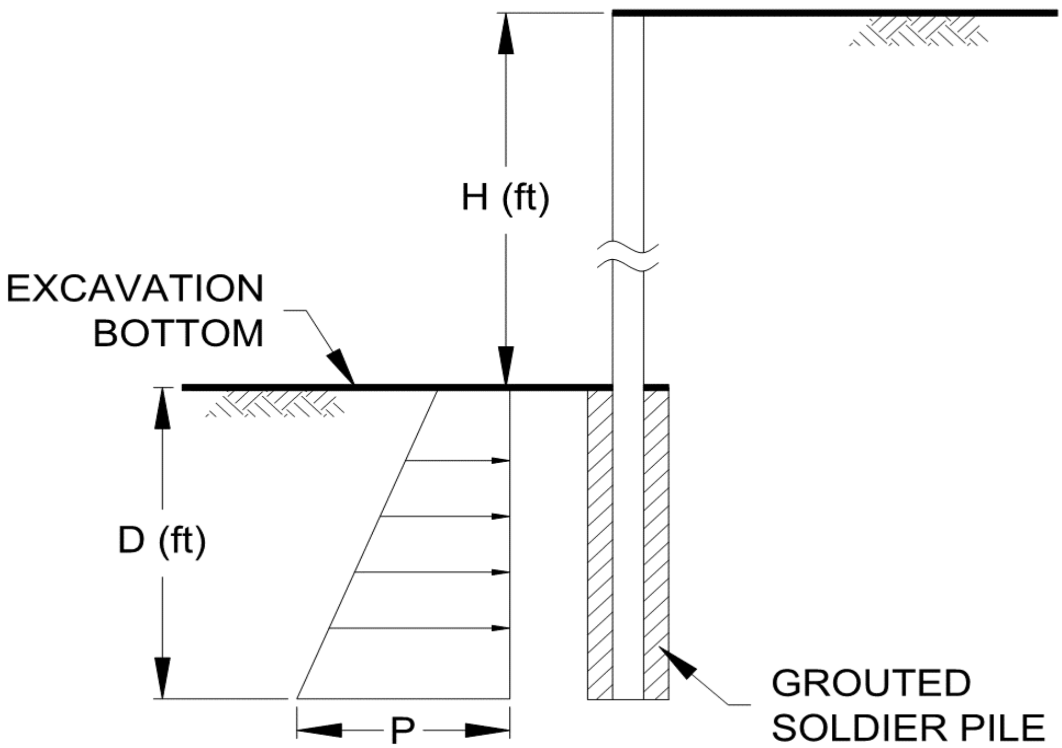
Parameter	Value
Triangular Distribution, A	23H psf
Rectangular Distribution, B	15H psf
Trapezoidal Distribution, C	19H psf
Passive Pressure, P	425D + 500 psf
Effective Zone Angle, E	29 degrees
Maximum Design Lateral Movement	1 Inch
Maximum Design Vertical Movement	½ Inch
Maximum Design Retained Height, H	20 Feet

H equals the height of the retaining portion of the wall in feet
D equals the embedment depth of the retaining wall in feet



Active Pressures on Shoring

6.4.7 The passive resistance can be assumed to act over a width of three pile diameters. Typically, soldier piles are embedded a minimum of 0.5 times the maximum height of the excavation (this depth is to include footing excavations) if tieback anchors are not employed. The project structural engineer should determine the actual embedment depth.



Passive Pressures on Shoring

6.4.8 We should observe the drilled shafts for the soldier piles prior to the placement of steel reinforcement to check that the exposed soil conditions are similar to those expected and that footing excavations have been extended to the appropriate bearing strata and design depths. If unexpected soil conditions are encountered, foundation modifications may be required.

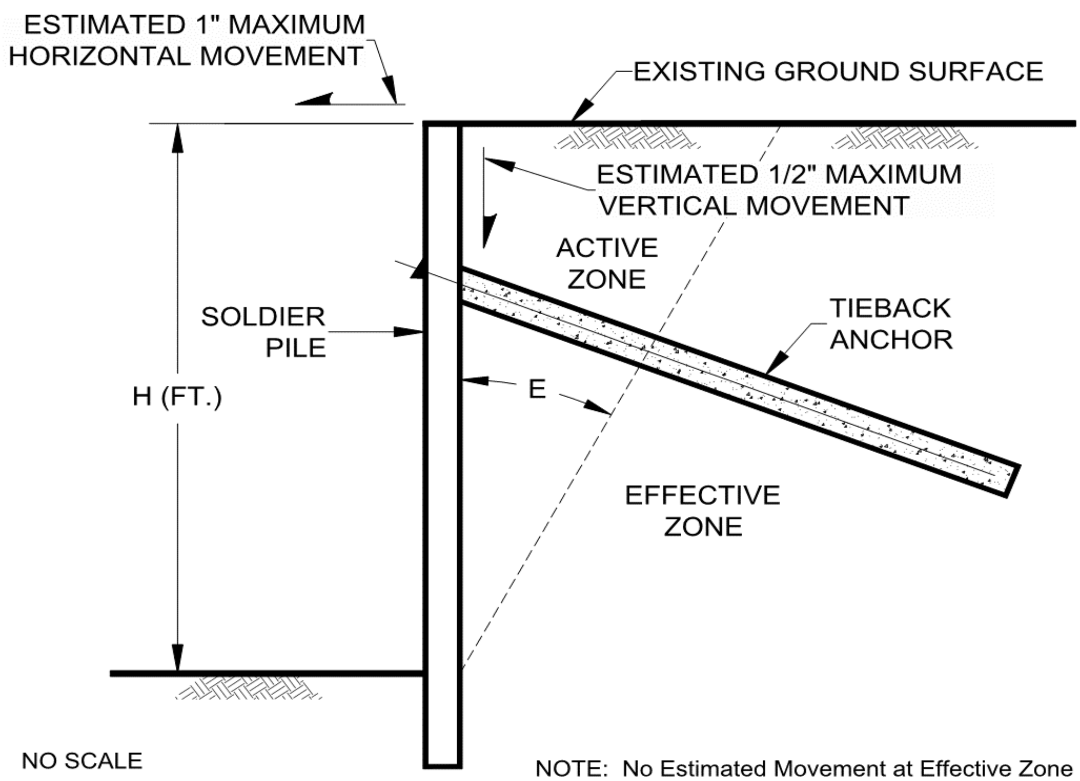
6.4.9 Lateral movement of shoring is associated with vertical ground settlement outside of the excavation. Therefore, it is essential that the soldier pile system allow very limited amounts of lateral displacement. Earth pressures acting on a lagging wall can cause movement of the shoring toward the excavation and result in ground subsidence outside of the excavation. Consequently, horizontal movements of the shoring wall should be accurately monitored and recorded during excavation and construction.

6.4.10 Survey points should be established at the top of the pile on at least 20 percent of the soldier piles. An additional point located at an intermediate point between the top of the pile and the base of the excavation should be monitored on at least 20 percent of the piles if tieback anchors will be used. These points should be monitored on a weekly basis during excavation work and monthly, thereafter, until the permanent support system is constructed.

6.4.11 The project civil engineer should provide the approximate location, depth, and pipe type of the underground utilities to the shoring engineer to help select the shoring type and shoring design. The shoring system should be designed to limit horizontal soldier pile movement to a maximum of 1 inch. The amount of horizontal deflection can be assumed to be essentially zero along the Active Zone and Effective Zone boundary. The magnitude of movement for intermediate depths and distances from the shoring wall can be linearly interpolated.

6.4.12 Tieback anchors employed in shoring should be designed in conformance with FHWA or PTI standards.

6.4.13 The anchors fully penetrate the Active Zone behind the shoring. The Active Zone can be considered the wedge of soil from the face of the shoring to a plane extending upward from the base of the excavation as shown on the Active Zone Detail. Normally, tieback anchors are contractor-designed and installed, and there are numerous anchor construction methods available. Non-shrinkage grout should be used for the construction of the tieback anchors.



Active Zone Detail

6.4.14 A pressure grouting tube should be installed during the construction of the tieback. Post grouting should be performed if adequate capacity cannot be obtained by other construction methods.

6.4.15 Anchor capacity is a function of construction method, depth of anchor, batter, diameter of the bonded section and the length of the bonded section. Anchor capacity should be evaluated using the strength parameters shown in Table 6.4.2.

TABLE 6.4.2
SOIL STRENGTH PARAMETERS FOR TEMPORARY SHORING

Description	Cohesion (psf)	Friction Angle (Degrees)
Compacted Fill	300	29
Weathered Metavolcanic Rock	400	32

6.4.16 Tie-back anchor tests should be performed in conformance with either FHWA or PTI standards. Tieback anchors that fail to meet project specified test criteria should be replaced or additional anchors should be constructed.

6.4.17 Lagging should keep pace with excavation. The excavation should not be advanced deeper than three feet below the bottom of lagging at any time. These unlagged gaps of up to three feet should only be allowed to stand for short periods of time in order to decrease the probability of soil instability and should never be unsupported overnight. Backfilling should be conducted when necessary between the back of lagging and excavation sidewalls to reduce sloughing in this zone and all voids should be filled by the end of each day. Further, the excavation should not be advanced further than four feet below a row of tiebacks prior to those tiebacks being proof tested and locked off unless otherwise specific by the shoring engineer.

6.4.18 An accurate survey of existing utilities and other underground structures adjacent to the shoring wall should be conducted. The survey should include both locations and depths of existing utilities. Locations of anchors should be adjusted as necessary during the design and construction process to accommodate the existing and proposed utilities.

6.5 Soil Nail Wall

6.5.1 As an alternative to a soldier beam and tieback shoring wall, a soil nail wall can be used. The wall should be designed by an engineer familiar with the design of soil nail walls.

6.5.2 Soil nail walls should not be considered a permanent design to support the seismic lateral loads and soil pressures on a building wall. Therefore, the proposed building should be designed to support the expected lateral loads.

6.5.3 In general, ground conditions are moderately suited to soil nail wall construction techniques.

6.5.4 Testing of the soil nails should be performed in accordance with the guidelines of the FHWA or PTI standards. Geocon Incorporated should observe the nail installation and perform the nail testing.

6.5.5 The soil strength parameters listed in Table 6.5 can be used in design of the soil nails. The bond strength is dependent on drilling method, diameter, and construction method. Therefore, the designer should evaluate the bond strength based on the existing soil conditions and the construction method.

TABLE 6.5
SOIL STRENGTH PARAMETERS FOR SOIL NAIL WALLS

Description	Cohesion (psf)	Friction Angle (degrees)	Estimated Ultimate Bond Stress (psi)*
Compacted Fill	300	29	10
Weathered Metavolcanic Rock	400	32	20

*Assuming gravity fed, open hole drilling techniques.

6.6 Preliminary Grading Recommendations

6.6.1 The grading recommendations provided herein are preliminary and should be updated once a grading plan is prepared, building locations are determined, and cut and fill depths are known.

6.6.2 Grading should be performed in accordance with the *Recommended Grading Specifications* in Appendix D. Where the recommendations of this report conflict with Appendix D, the recommendations of this section take precedence.

6.6.3 A pre-construction conference with the owner, contractor, civil engineer, and soil engineer in attendance should be held at the site prior to beginning grading. Special soil handling requirements can be discussed at that time.

6.6.4 Site preparation should begin with the removal of deleterious debris and vegetation. The depth of removal should be such that materials to be used in fill are generally free of deleterious debris or organic matter. Material generated during stripping operations and/or site demolition should be exported from the site. Existing underground improvements within the site should be removed and the resulting depressions properly backfilled in accordance with the procedures described herein.

6.6.5 Earthwork should be observed, and compacted fill observed and tested by a representative of Geocon Incorporated.

6.6.6 Within the area of the development and grading, undocumented fill, topsoil, and upper 12 inches of the weathered metavolcanic rock should be removed and replaced with properly compacted fill. We expect the depth of the removals will range from 2 feet to 7 feet below existing grade in general throughout the site. Deeper removals may be required in areas where loose or saturated materials are encountered. The estimated depths of the removals are presented on the Figures 2 and 3. Deeper removals may be required if saturated or loose fill soil is encountered. A representative of Geocon should be on-site during removals to evaluate the limits of the remedial grading.

6.6.7 Excavations up to 15 feet to 20 feet will be necessary to reach foundation elevation for the subterranean portion of the proposed storage facility. Basement level excavations will expose very dense Metavolcanic Rock that may require very heavy ripping using a Caterpillar D-9 (or larger) fitted with a single shank, rock breaking using hydraulic hammer, or other excavation method. Hard rock conditions are anticipated at the west end of the building foundation excavation, as shown on the Geologic Map and Geologic Cross-Section (Figures 2 and 3). However, hard rock requiring rock breaking techniques could be encountered in other areas of the building pad. Perspective contractors should evaluate the geotechnical data and use their own judgment to identify the boundary between rippable and non-rippable rock and the appropriate technique for excavating to basement foundation levels.

6.6.8 Excavations in hard rock may result in over excavation of building foundations. Where this occurs, loose rock should be removed from footing excavations and the resulting overexcavation backfilled with concrete or 2-sack cement slurry so the entire foundation will be bearing on Metavolcanic Rock.

6.6.9 As discussed, excavations within portions of the building pad will encounter hard rock that may require rock breaking techniques. As an alternative to constructing footings in hard rock and to facilitate foundation excavation, the building pad can be undercut to a depth of at least 1 foot below the bottom of building footings (including elevator pits) and replaced with properly compacted fill. Undercutting may generate oversize material that could require exporting.

6.6.10 Excavation of subterranean building levels and building foundations may generate oversize material. Oversize material may be used in structural fills provided it is placed in accordance with the spacing and hold-down requirements provided in Appendix D. Alternatively, oversized material should be exported from the site.

6.6.11 Underground utilities outside of the building pad located in areas of hard rock should be undercut to a depth of at least 2 feet below the bottom of the utility to facilitate ease of excavations, if necessary.

6.6.12 Table 6.6.1 provides a summary of the grading recommendations.

TABLE 6.6.1
SUMMARY OF GRADING RECOMMENDATIONS

Area	Removal Requirements
Remedial Grading	• Remove Undocumented Fill, Topsoil and Upper 1 Foot of Formation • At Least Five Feet Beyond Planned Structural Improvements
Storage Facility Building	Remedial grading not required. Heavy ripping may be necessary.
Utilities in Cut areas	Where hard rock is exposed, undercut utilities a depth of 2 feet below the utility
Exposed Bottoms of Remedial Grading	Scarify Upper 12 Inches

6.6.13 The bottom of the excavations should be sloped 1 percent to the adjacent street or deepest fill to promote drainage along the contact of the compacted fill and native formational bedrock.

6.6.14 Prior to fill being placed, the existing ground surface should be scarified, moisture conditioned as necessary, and compacted to a depth of at least 12 inches. The site should then be brought to final subgrade elevations with fill compacted in layers. In general, soil native to the site is suitable for use from a geotechnical engineering standpoint as fill if relatively free from vegetation, debris and other deleterious material. Layers of fill should be about 6 to 8 inches in loose thickness and no thicker than will allow for adequate bonding and compaction. Fill, including backfill and scarified ground surfaces, should be compacted to a dry density of at least 90 percent of the laboratory maximum dry density near to slightly above optimum moisture content in accordance with ASTM Test Procedure D 1557. Fill materials placed below optimum moisture content may require additional moisture conditioning prior to placing additional fill. Overly wet materials will require drying and/or mixing with drier soils to facilitate proper compaction.

6.6.15 Imported fill (if necessary) should consist of the characteristics presented in Table 6.6.2. Geocon Incorporated should be notified of the import soil source and should perform laboratory testing of import soil prior to its arrival at the site to determine its suitability as fill material.

TABLE 6.6.2
SUMMARY OF IMPORT FILL RECOMMENDATIONS

Soil Characteristic	Values
Expansion Potential	"Very Low" to "Medium" (Expansion Index of 90 or less)
Particle Size	Maximum Dimension Less Than 3 Inches
	Generally Free of Debris

6.7 Seismic Design Criteria – 2019 California Building Code

6.7.1 Table 6.7.1 summarizes site-specific design criteria obtained from the 2019 California Building Code (CBC; Based on the 2018 International Building Code [IBC] and ASCE 7-16), Chapter 16 Structural Design, Section 1613 Earthquake Loads. We used the computer program *U.S. Seismic Design Maps*, provided by the Structural Engineers Association (SEA) to calculate the seismic design parameters. The short spectral response uses a period of 0.2 second. We evaluated the Site Class based on the discussion in Section 1613.2.2 of the 2019 CBC and Table 20.3-1 of ASCE 7-16. The values presented herein are for the risk-targeted maximum considered earthquake (MCE_R). Sites designated as Site Class D, E and F may require additional analyses if requested by the project structural engineer and client.

TABLE 6.7.1
2019 CBC SEISMIC DESIGN PARAMETERS

Parameter	Value	2019 CBC Reference
Site Class	C	Section 1613.2.2
MCE_R Ground Motion Spectral Response Acceleration – Class B (short), S_S	0.835g	Figure 1613.2.1(1)
MCE_R Ground Motion Spectral Response Acceleration – Class B (1 sec), S_1	0.298g	Figure 1613.2.1(2)
Site Coefficient, F_A	1.200	Table 1613.2.3(1)
Site Coefficient, F_V	1.500	Table 1613.2.3(2)
Site Class Modified MCE_R Spectral Response Acceleration (short), S_{MS}	1.002g	Section 1613.2.3 (Eqn 16-36)
Site Class Modified MCE_R Spectral Response Acceleration – (1 sec), S_{M1}	0.447g	Section 1613.2.3 (Eqn 16-37)
5% Damped Design Spectral Response Acceleration (short), S_{DS}	0.668g	Section 1613.2.4 (Eqn 16-38)
5% Damped Design Spectral Response Acceleration (1 sec), S_{D1}	0.298g	Section 1613.2.4 (Eqn 16-39)

6.7.2 Table 6.7.2 presents the mapped maximum considered geometric mean (MCE_G) seismic design parameters for projects located in Seismic Design Categories of D through F in accordance with ASCE 7-16.

TABLE 6.7.2
ASCE 7-16 PEAK GROUND ACCELERATION

Parameter	Value	ASCE 7-16 Reference
Mapped MCE_G Peak Ground Acceleration, PGA	0.363g	Figure 22-7
Site Coefficient, F_{PGA}	1.200	Table 11.8-1
Site Class Modified MCE_G Peak Ground Acceleration, PGA_M	0.436g	Section 11.8.3 (Eqn 11.8-1)

6.7.3 Conformance to the criteria in Tables 6.7.1 and 6.7.2 for seismic design does not constitute any kind of guarantee or assurance that significant structural damage or ground failure will not occur in the event of a large earthquake. The primary goal of seismic design is to protect life, not to avoid all damage, since such design may be economically prohibitive.

6.7.4 The project structural engineer and architect should evaluate the appropriate Risk Category and Seismic Design Category for the planned structures. The values presented herein assume a Risk Category of II and resulting in a Seismic Design Category D. Table 6.7.3 presents a summary of the risk categories in accordance with ASCE 7-16.

TABLE 6.7.3
ASCE 7-16 RISK CATEGORIES

Risk Category	Building Use	Examples
I	Low risk to Human Life at Failure	Barn, Storage Shelter
II	Nominal Risk to Human Life at Failure (Buildings Not Designated as I, III or IV)	Residential, Commercial and Industrial Buildings
III	Substantial Risk to Human Life at Failure	Theaters, Lecture Halls, Dining Halls, Schools, Prisons, Small Healthcare Facilities, Infrastructure Plants, Storage for Explosives/Toxins
IV	Essential Facilities	Hazardous Material Facilities, Hospitals, Fire and Rescue, Emergency Shelters, Police Stations, Power Stations, Aviation Control Facilities, National Defense, Water Storage

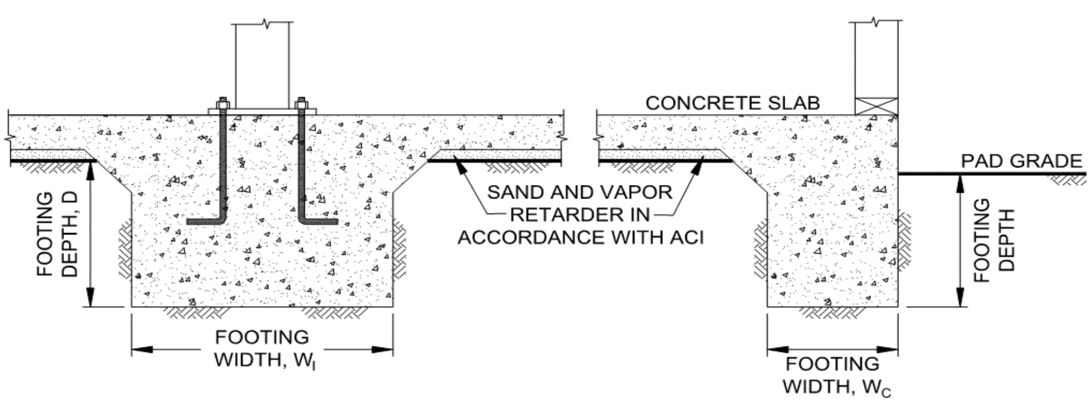
6.8 Preliminary Foundation Recommendations

6.8.1 The proposed structures can be supported on a shallow foundation system. Foundations for the structure should consist of continuous strip footings and/or isolated spread footings. Tables 6.8 provides a summary of the foundation design recommendations.

TABLE 6.8
SUMMARY OF FOUNDATION RECOMMENDATIONS

Parameter	Value
Minimum Continuous Foundation Width, W_C	12 inches
Minimum Isolated Foundation Width, W_I	24 inches
Minimum Foundation Depth, D	24 Inches Below Lowest Adjacent Grade
Minimum Concrete Reinforcement	4 No. 5 Bars, 2 at the Top and 2 at the Bottom
For Footings Founded in Compacted Fill	
Allowable Bearing Capacity	2,000 psf
Bearing Capacity Increase	500 psf per Foot of Depth
	300 psf per Foot of Width
Maximum Allowable Bearing Capacity	4,000 psf
For Footings Founded in Metavolcanic Rock	
Allowable Bearing Capacity	6,000 psf
Bearing Capacity Increase	500 psf per Foot of Depth
	300 psf per Foot of Width
Maximum Allowable Bearing Capacity	8,000 psf
Estimated Total Settlement	1 inch
Estimated Differential Settlement	½ inch in 40 Feet
Footing Size Used for Settlement	9-foot Square
Design Expansion Index	50 or less

6.8.2 The foundations should be embedded in accordance with the recommendations herein and the Wall/Column Footing Dimension Detail. The embedment depths should be measured from the lowest adjacent pad grade for both interior and exterior footings. Footings should be deepened such that the bottom outside edge of the footing is at least 7 feet horizontally from the face of the slope (unless designed with a post-tensioned foundation system as discussed herein).



Wall/Column Footing Dimension Detail

6.8.3 The bearing capacity values presented herein are for dead plus live loads and may be increased by one-third when considering transient loads due to wind or seismic forces.

6.8.4 We should observe the foundation excavations prior to the placement of reinforcing steel and concrete to check that the exposed soil conditions are similar to those expected and that they have been extended to the appropriate bearing strata. Foundation modifications may be required if unexpected soil conditions are encountered.

6.8.5 Geocon Incorporated should be consulted to provide additional design parameters as required by the structural engineer.

6.9 Concrete Slabs-On-Grade

6.9.1 Concrete slabs-on-grade for the structures should be constructed in accordance with Table 6.9.

TABLE 6.9
MINIMUM CONCRETE SLAB-ON-GRADE RECOMMENDATIONS

Parameter	Value
Minimum Concrete Slab Thickness	5 inches
Minimum Concrete Reinforcement	No. 3 Bars, 18 Inches on Center, Both Directions
Typical Slab Underlayment	3 to 4 Inches of Sand
Design Expansion Index	90 or less

6.9.2 Slabs that may receive moisture-sensitive floor coverings or may be used to store moisture-sensitive materials should be underlain by a vapor retarder. The vapor retarder design should be consistent with the guidelines presented in the American Concrete Institute's (ACI) *Guide for Concrete Slabs that Receive Moisture-Sensitive Flooring Materials* (ACI 302.2R-06). In addition, the membrane should be installed in accordance with manufacturer's recommendations and ASTM requirements and installed in a manner that prevents puncture. The vapor retarder used should be specified by the project architect or developer based on the type of floor covering that will be installed and if the structure will possess a humidity-controlled environment.

6.9.3 The bedding sand thickness should be determined by the project foundation engineer, architect, and/or developer. It is common to have 3 to 4 inches of sand for 5-inch and 4-inch thick slabs, respectively, in the southern California region. However, we should be contacted to provide recommendations if the bedding sand is thicker than 6 inches. The foundation design engineer should provide appropriate concrete mix design criteria and curing measures to assure proper curing of the slab by reducing the potential for rapid moisture loss and subsequent cracking and/or slab curl. We suggest that the foundation design engineer present the concrete mix design and proper curing methods on the foundation plans. It is critical that the foundation contractor understands and follows the recommendations presented on the foundation plans.

6.9.4 Concrete slabs should be provided with adequate crack-control joints, construction joints and/or expansion joints to reduce unsightly shrinkage cracking. The design of joints should consider criteria of the American Concrete Institute (ACI) when establishing crack-control spacing. Crack-control joints should be spaced at intervals no greater than 12 feet. Additional steel reinforcing, concrete admixtures and/or closer crack control joint spacing should be considered where concrete-exposed finished floors are planned.

6.9.5 Special subgrade presaturation is not deemed necessary prior to placing concrete; however, the exposed foundation and slab subgrade soil should be moisturized to maintain a moist condition as would be expected in any such concrete placement.

6.9.6 The concrete slab-on-grade recommendations are based on soil support characteristics only. The project structural engineer should evaluate the structural requirements of the concrete slabs for supporting expected loads.

6.9.7 The recommendations of this report are intended to reduce the potential for cracking of slabs due to expansive soil (if present), differential settlement of existing soil or soil with varying thicknesses. However, even with the incorporation of the recommendations presented herein, foundations, stucco walls, and slabs-on-grade placed on such conditions may still exhibit some cracking due to soil movement and/or shrinkage. The occurrence of concrete shrinkage cracks is independent of the supporting soil characteristics. Their occurrence may be reduced and/or controlled by limiting the slump of the concrete, proper concrete placement and curing, and by the placement of crack control joints at periodic intervals, in particular, where re-entrant slab corners occur.

6.10 Exterior Concrete Flatwork

6.10.1 Exterior concrete flatwork not subject to vehicular traffic should be constructed in accordance with the recommendations presented in Table 6.10. The recommended steel reinforcement would help reduce the potential for cracking.

TABLE 6.10
MINIMUM CONCRETE FLATWORK RECOMMENDATIONS

Expansion Index, EI	Minimum Concrete Reinforcement* Options	Minimum Thickness
EI $\leq$ 90	6x6-W2.9/W2.9 (6x6-6/6) welded wire mesh	4 Inches
	No. 3 Bars 18 inches on center, Both Directions	

*In excess of 8 feet square.

6.10.2 The subgrade soil should be properly moisturized and compacted prior to the placement of steel and concrete. The subgrade soil should be compacted to a dry density of at least 90 percent of the laboratory maximum dry density near to slightly above optimum moisture content in accordance with ASTM D 1557.

6.10.3 Even with the incorporation of the recommendations of this report, exterior concrete flatwork has a potential to experience some uplift due to expansive soil beneath grade. Steel reinforcement should overlap continuously in flatwork to reduce the potential for vertical offsets. Additionally, flatwork should be structurally connected to the curbs, where possible, to reduce the potential for offsets between the curbs and the flatwork.

6.10.4 Concrete flatwork should be provided with crack control joints to reduce and/or control shrinkage cracking. Crack control spacing should be determined by the project structural engineer based upon the slab thickness and intended usage. Criteria of the American Concrete Institute (ACI) should be taken into consideration when establishing crack control spacing. Subgrade soil for exterior slabs not subjected to vehicle loads should be compacted in accordance with criteria presented in the grading section prior to concrete placement. Subgrade soil should be properly compacted and the moisture content of subgrade soil should be verified prior to placing concrete. Base materials will not be required below concrete improvements.

6.10.5 Where exterior flatwork abuts the structure at entrant or exit points, the exterior slab should be dowelled into the structure’s foundation stemwall. This recommendation is intended to reduce the potential for differential elevations that could result from differential settlement or minor heave of the flatwork. Dowelling details should be designed by the project structural engineer.

6.10.6 The recommendations presented herein are intended to reduce the potential for cracking of exterior slabs as a result of differential movement. However, even with the incorporation of the recommendations presented herein, slabs-on-grade will still crack. The occurrence of concrete shrinkage cracks is independent of the soil supporting characteristics. Their occurrence may be reduced and/or controlled by limiting the slump of the concrete, the use of crack control joints and proper concrete placement and curing. Crack control joints should be spaced at intervals no greater than 12 feet. Literature provided by the Portland Concrete Association (PCA) and American Concrete Institute (ACI) present recommendations for proper concrete mix, construction, and curing practices, and should be incorporated into project construction.

6.11 Retaining Walls

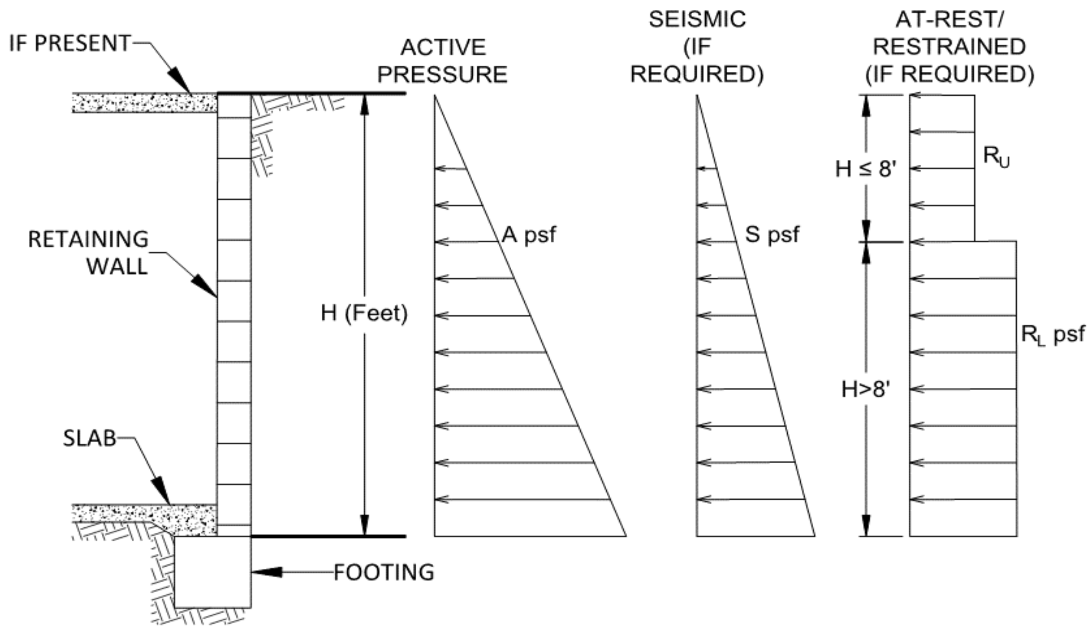
6.11.1 Retaining walls should be designed using the values presented in Table 6.11.1. Soil with an expansion index (EI) of greater than 50 should not be used as backfill material behind retaining walls. Onsite soils generally possess a *medium* expansion potential (EI of 90 or less); therefore; import of *low* expansive soil will be required for retaining wall backfill.

TABLE 6.11.1
RETAINING WALL DESIGN RECOMMENDATIONS

Parameter	Value
Active Soil Pressure, A (Fluid Density, Level Backfill)	35 pcf
Active Soil Pressure, A (Fluid Density, 2:1 Sloping Backfill)	50 pcf
Seismic Pressure, S	15H psf
At-Rest/Restrained Walls Additional Uniform Pressure (0 to 8 Feet High)	7H psf
Expected Expansion Index for the Subject Property	EI≤50

H equals the height of the retaining portion of the wall.

6.11.2 Retaining walls should be designed as shown in the Retaining Wall Loading Diagram below.



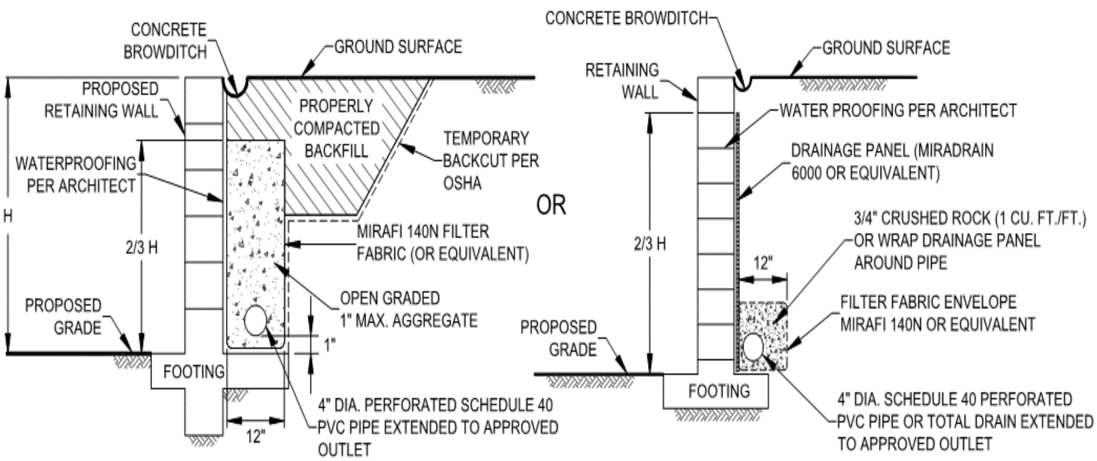
Retaining Wall Loading Diagram

6.11.3 Unrestrained walls are those that are allowed to rotate more than 0.001H (where H equals the height of the retaining portion of the wall) at the top of the wall. Where walls are restrained from movement at the top (at-rest condition), an additional uniform pressure should be applied to the wall. For retaining walls subject to vehicular loads within a horizontal distance equal to two-thirds the wall height, a surcharge equivalent to 2 feet of fill soil should be added.

6.11.4 The structural engineer should determine the Seismic Design Category for the project in accordance with Section 1613.3.5 of the 2019 CBC or Section 11.6 of ASCE 7-10. For structures assigned to Seismic Design Category of D, E, or F, retaining walls that support more than 6 feet of backfill should be designed with seismic lateral pressure in accordance with Section 1803.5.12 of the 2019 CBC. The seismic load is dependent on the retained height where H is the height of the wall, in feet, and the calculated loads result in pounds per square foot (psf) exerted at the base of the wall and zero at the top of the wall.

6.11.5 Retaining walls should be designed to ensure stability against overturning sliding, and excessive foundation pressure. Where a keyway is extended below the wall base with the intent to engage passive pressure and enhance sliding stability, it is not necessary to consider active pressure on the keyway.

6.11.6 Drainage openings through the base of the wall (weep holes) should not be used where the seepage could be a nuisance or otherwise adversely affect the property adjacent to the base of the wall. The recommendations herein assume a properly compacted granular (EI of 50 or less) free-draining backfill material with no hydrostatic forces or imposed surcharge load. The retaining wall should be properly drained as shown in the Typical Retaining Wall Drainage Detail. If conditions different than those described are expected, or if specific drainage details are desired, Geocon Incorporated should be contacted for additional recommendations.



Typical Retaining Wall Drainage Detail

6.11.7 The retaining walls may be designed using either the active and restrained (at-rest) loading condition or the active and seismic loading condition as suggested by the structural engineer. Typically, it appears the design of the restrained condition for retaining wall loading may be adequate for the seismic design of the retaining walls. However, the active earth pressure combined with the seismic design load should be reviewed and considered in the design of the retaining walls.

6.11.8 Table 6.11.2 provides a summary of retaining wall foundation recommendations.

TABLE 6.11.2
SUMMARY OF RETAINING WALL FOUNDATION RECOMMENDATIONS

Parameter	Value
Minimum Retaining Wall Foundation Width	12 inches
Minimum Retaining Wall Foundation Depth	12 Inches
Minimum Steel Reinforcement	Per Structural Engineer
For Footings Founded in Compacted Fill	
Allowable Bearing Capacity	2,000 psf
Bearing Capacity Increase	500 psf per Foot of Depth
	300 psf per Foot of Width
Maximum Allowable Bearing Capacity	4,000 psf
For Footings Founded in Metavolcanic Rock	
Allowable Bearing Capacity	6,000 psf
Bearing Capacity Increase	500 psf per Foot of Depth
	300 psf per Foot of Width
Maximum Allowable Bearing Capacity	8,000 psf
Estimated Total Settlement	1 inch
Estimated Differential Settlement	½ inch in 40 Feet

6.11.9 The recommendations presented herein are generally applicable to the design of rigid concrete or masonry retaining walls. In the event that other types of walls (such as mechanically stabilized earth [MSE] walls) are planned, Geocon Incorporated should be consulted for additional recommendations.

6.11.10 Soil contemplated for use as retaining wall backfill, including import materials, should be identified in the field prior to backfill. At that time, Geocon Incorporated should obtain samples for laboratory testing to evaluate its suitability. Modified lateral earth pressures may be necessary if the backfill soil does not meet the required expansion index or shear strength. City or regional standard wall designs, if used, are based on a specific active lateral earth pressure and/or soil friction angle. In this regard, on-site soil to be used as backfill may or may not meet the values for standard wall designs. Geocon Incorporated should be consulted to assess the suitability of the on-site soil for use as wall backfill if standard wall designs will be used.

6.12 Lateral Loading

6.12.1 Table 6.12 should be used to help design the proposed structures and improvements to resist lateral loads for the design of footings or shear keys. The allowable passive pressure assumes a horizontal surface extending at least 5 feet, or three times the surface generating the passive pressure, whichever is greater. The upper 12 inches of material in areas not protected by floor slabs or pavement should not be included in design for passive resistance.

TABLE 6.12
SUMMARY OF LATERAL LOAD DESIGN RECOMMENDATIONS

Parameter	Value
Passive Pressure Fluid Density	350 pcf
Coefficient of Friction (Concrete and Soil)	0.35
Coefficient of Friction (Along Vapor Barrier)	0.2 to 0.25*

*Per manufacturer's recommendations.

6.12.2 The passive and frictional resistant loads can be combined for design purposes. The lateral passive pressures may be increased by one-third when considering transient loads due to wind or seismic forces.

6.13 Preliminary Pavement Recommendations

6.13.1 We calculated the flexible pavement sections in general conformance with the *Caltrans Method of Flexible Pavement Design* (Highway Design Manual, Section 608.4) using an estimated Traffic Index (TI) of 5.0, 5.5, 6.0, and 7.0 for parking stalls, driveways, medium truck traffic areas, and heavy truck traffic areas, respectively. The project civil engineer and owner should review the pavement designations to determine appropriate locations for pavement thickness. The final pavement sections for the parking lot should be based on the R-Value of the subgrade soil encountered at final subgrade elevation. We have used an R-Value of 6 (based laboratory test results) and 78 for the subgrade soil and base materials, respectively, for the purposes of this preliminary analysis. Table 6.13.1 presents the preliminary flexible pavement sections.

TABLE 6.13.1
PRELIMINARY FLEXIBLE PAVEMENT SECTION

Location	Assumed Traffic Index	Subgrade R-Value	Asphalt Concrete (inches)	Class 2 Aggregate Base (inches)
Parking stalls for automobiles and light-duty vehicles	5.0	6	3	9.5
Driveways for automobiles and light-duty vehicles	5.5	6	3	11.5
Medium truck traffic areas	6.0	6	4	11.5
Driveways for heavy truck traffic	7.0	6	4	15.5

6.13.2 Prior to placing base materials, the upper 12 inches of the subgrade soil should be scarified, moisture conditioned as necessary, and recompacted to a dry density of at least 95 percent of the laboratory maximum dry density near to slightly above optimum moisture content as determined by ASTM D 1557. Similarly, the base material should be compacted to a dry density of at least 95 percent of the laboratory maximum dry density near to slightly above optimum moisture content. Asphalt concrete should be compacted to a density of at least 95 percent of the laboratory Hveem density in accordance with ASTM D 2726.

6.13.3 A rigid Portland cement concrete (PCC) pavement section should be placed in roadway aprons and cross gutters. We calculated the rigid pavement section in general conformance with the procedure recommended by the American Concrete Institute report ACI 330R-08 Guide for Design and Construction of Concrete Parking Lots using the parameters presented in Table 6.13.2.

TABLE 6.13.2
RIGID PAVEMENT DESIGN PARAMETERS

Design Parameter	Design Value
Modulus of subgrade reaction, k	50 psi
Modulus of rupture for concrete, M_R	500 psi
Concrete Compressive Strength	3,000 psi
Traffic Category, TC	A and C
Average daily truck traffic, ADTT	10 and 100

6.13.4 Based on the criteria presented herein, the PCC pavement sections should have a minimum thickness as presented in Table 6.13.3.

TABLE 6.13.3
RIGID VEHICULAR PAVEMENT RECOMMENDATIONS

Location	Portland Cement Concrete (inches)
Automobile Parking Stalls (TC=A)	6
Driveways (TC=C)	7.5

6.13.5 The PCC vehicular pavement should be placed over subgrade soil that is compacted to a dry density of at least 95 percent of the laboratory maximum dry density near to slightly above optimum moisture content.

6.13.6 The rigid pavement should also be designed and constructed incorporating the parameters presented in Table 6.13.4.

TABLE 6.13.4
ADDITIONAL RIGID PAVEMENT RECOMMENDATIONS

<table>
<tr><th>Subject</th><th>Value</th></tr>
<tr><td rowspan="3">Thickened Edge</td><td>1.2 Times Slab Thickness</td></tr>
<tr><td>Minimum Increase of 2 Inches</td></tr>
<tr><td>4 Feet Wide</td></tr>
<tr><td rowspan="3">Crack Control Joint Spacing</td><td>30 Times Slab Thickness</td></tr>
<tr><td>Max. Spacing of 12 feet for 5.5-Inch-Thick</td></tr>
<tr><td>Max. Spacing of 15 Feet for Slabs 6 Inches and Thicker</td></tr>
<tr><td rowspan="2">Crack Control Joint Depth</td><td>Per ACI 330R-08</td></tr>
<tr><td>1 Inch Using Early-Entry Saws on Slabs Less than 9 Inches Thick</td></tr>
<tr><td rowspan="3">Crack Control Joint Width</td><td>¼-Inch for Sealed Joints</td></tr>
<tr><td>⅜-Inch is Common for Sealed Joints</td></tr>
<tr><td>$^{1}/_{10}$- to $^{1}/_{8}$-Inch is Common for Unsealed Joints</td></tr>
</table>

6.13.7 Reinforcing steel will not be necessary within the concrete for geotechnical purposes with the possible exception of dowels at construction joints as discussed herein.

6.13.8 To control the location and spread of concrete shrinkage cracks, crack-control joints (weakened plane joints) should be included in the design of the concrete pavement slab. Crack-control joints should be sealed with an appropriate sealant to prevent the migration of water through the control joint to the subgrade materials. The depth of the crack-control joints should be determined by the referenced ACI report.

6.13.9 To provide load transfer between adjacent pavement slab sections, a butt-type construction joint should be constructed. The butt-type joint should be thickened by at least 20 percent at the edge and taper back at least 4 feet from the face of the slab. As an alternative to the butt-type construction joint, dowelling can be used between construction joints for pavements of 7 inches or thicker. As discussed in the referenced ACI guide, dowels should consist of smooth, 1-inch-diameter reinforcing steel 14 inches long embedded a minimum of 6 inches into the slab on either side of the construction joint. Dowels should be located at the midpoint of the slab, spaced at 12 inches on center and lubricated to allow joint movement while still transferring loads. In addition, tie bars should be installed as recommended in Section 3.8.3 of the referenced ACI guide. The structural engineer should provide other alternative recommendations for load transfer.

6.13.10 Concrete curb/gutter should be placed on soil subgrade compacted to a dry density of at least 90 percent of the laboratory maximum dry density near to slightly above optimum moisture content. Cross-gutters that receives vehicular should be placed on subgrade soil compacted to a dry density of at least 95 percent of the laboratory maximum dry density near to slightly above optimum moisture content. Base materials should not be placed below the curb/gutter, or cross-gutters so water is not able to migrate from the adjacent parkways to the pavement sections. Where flatwork is located directly adjacent to the curb/gutter, the concrete flatwork should be structurally connected to the curbs to help reduce the potential for offsets between the curbs and the flatwork.

6.14 Storm Water Management

6.14.1 If storm water management devices are not properly designed and constructed, there is a risk for distress to improvements and property located hydrologically down gradient or adjacent to these devices. Factors such as the amount of water being detained, its residence time, and soil permeability have an important effect on seepage transmission and the potential adverse impacts that may occur if the storm water management features are not properly designed and constructed. We have not performed a hydrogeological study at the site. If infiltration of storm water runoff into the subsurface occurs, downstream improvements may be subjected to seeps, springs, slope instability, raised groundwater, movement of foundations and slabs, or other undesirable impacts as a result of water infiltration.

6.14.2 We performed an infiltration study on the property. A summary of our study and storm water management recommendations are provided in Appendix C. Based on the results of our study, full and partial infiltration is considered infeasible due to slow infiltration characteristics of the on-site soil. Basins should utilize a liner to prevent infiltration from causing adverse settlement and heave, and water migration into utility trench backfill, and under foundations.

6.15 Site Drainage and Moisture Protection

6.15.1 Adequate site drainage is critical to reduce the potential for differential soil movement, erosion and subsurface seepage. Under no circumstances should water be allowed to pond adjacent to footings. The site should be graded and maintained such that surface drainage is directed away from structures in accordance with 2019 CBC 1804.4 or other applicable standards. In addition, surface drainage should be directed away from the top of slopes into swales or other controlled drainage devices. Roof and pavement drainage should be directed into conduits that carry runoff away from the proposed structure.

6.15.2 In the case of basement walls or building walls retaining landscaping areas, a water-proofing system should be used on the wall and joints, and a Miradrain drainage panel (or similar) should be placed over the waterproofing. The project architect or civil engineer should provide detailed specifications on the plans for all waterproofing and drainage.

6.15.3 Underground utilities should be leak free. Utility and irrigation lines should be checked periodically for leaks. Detected leaks should be repaired promptly. Detrimental soil movement could occur if water is allowed to infiltrate the soil for prolonged periods of time.

6.15.4 Landscaping planters adjacent to paved areas are not recommended due to the potential for surface or irrigation water to infiltrate the pavement's subgrade and base course. Area drains to collect excess irrigation water and transmit it to drainage structures or impervious above-grade planter boxes can be used. In addition, where landscaping is planned adjacent to the pavement, construction of a cutoff wall along the edge of the pavement that extends at least 6 inches below the bottom of the base material should be considered.

6.15.5 We should prepare a storm water infiltration feasibility report of storm water management devices are planned.

6.16 Grading and Foundation Plan Review

6.16.1 Geocon Incorporated should review the grading and building foundation plans for the project prior to final design submittal to evaluate if additional analyses and/or recommendations are required.

6.17 Testing and Observation Services During Construction

6.17.1 Geocon Incorporated should provide geotechnical testing and observation services during the grading operations, foundation construction, utility installation, retaining wall backfill and pavement installation. Table 6.17 presents the typical geotechnical observations we would expect for the proposed improvements.

TABLE 6.17
EXPECTED GEOTECHNICAL TESTING AND OBSERVATION SERVICES

Construction Phase	Observations	Expected Time Frame
Ground Modification	Ground Modification Installation	Full Time
	Confirmation Testing	Part Time to Full Time
Grading	Base of Removal	Part Time During Removals
	Geologic Logging	Part Time to Full Time
	Fill Placement and Soil Compaction Operations	Full Time
Soldier Piles	Solder Pile Drilling Depth	Part Time
Tieback Anchors	Tieback Drilling and Installation	Full Time
	Tieback Testing	Full Time
Soil Nail Walls	Soil Nail Drilling and Installation	Full Time
	Soil Nail Testing	Full Time
Foundations	Drilling Operations for Piles	Full Time
	Foundation Excavation Observations	Part Time
Utility Backfill	Fill Placement and Soil Compaction Operations	Part Time to Full Time
Retaining Wall Backfill	Fill Placement and Soil Compaction Operations	Part Time to Full Time
Subgrade for Sidewalks, Curb/Gutter and Pavement	Soil Compaction Operations	Part Time
Pavement Construction	Base Placement and Compaction	Part Time
	Asphalt Concrete Placement and Compaction	Full Time

LIMITATIONS AND UNIFORMITY OF CONDITIONS

1. The firm that performed the geotechnical investigation for the project should be retained to provide testing and observation services during construction to provide continuity of geotechnical interpretation and to check that the recommendations presented for geotechnical aspects of site development are incorporated during site grading, construction of improvements, and excavation of foundations. If another geotechnical firm is selected to perform the testing and observation services during construction operations, that firm should prepare a letter indicating their intent to assume the responsibilities of project geotechnical engineer of record. A copy of the letter should be provided to the regulatory agency for their records. In addition, that firm should provide revised recommendations concerning the geotechnical aspects of the proposed development, or a written acknowledgement of their concurrence with the recommendations presented in our report. They should also perform additional analyses deemed necessary to assume the role of Geotechnical Engineer of Record.

2. The recommendations of this report pertain only to the site investigated and are based upon the assumption that the soil conditions do not deviate from those disclosed in the investigation. If any variations or undesirable conditions are encountered during construction, or if the proposed construction will differ from that anticipated herein, Geocon Incorporated should be notified so that supplemental recommendations can be given. The evaluation or identification of the potential presence of hazardous or corrosive materials was not part of the scope of services provided by Geocon Incorporated.

3. This report is issued with the understanding that it is the responsibility of the owner or his representative to ensure that the information and recommendations contained herein are brought to the attention of the architect and engineer for the project and incorporated into the plans, and the necessary steps are taken to see that the contractor and subcontractors carry out such recommendations in the field.

4. The findings of this report are valid as of the present date. However, changes in the conditions of a property can occur with the passage of time, whether they be due to natural processes or the works of man on this or adjacent properties. In addition, changes in applicable or appropriate standards may occur, whether they result from legislation or the broadening of knowledge. Accordingly, the findings of this report may be invalidated wholly or partially by changes outside our control. Therefore, this report is subject to review and should not be relied upon after a period of three years.

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N

NO SCALE

VICINITY MAP

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5780 QUARRY ROAD
BONITA, CALIFORNIA

NB / RA		DSK/GTYPD	DATE 10 - 11 - 2021	PROJECT NO. G2674 - 42 - 01	FIG. 1

Plotted:10/11/2021 7:57AM | By:ALVIN LADRILLONO | File Location:Y:\PROJECTS\G2674-42-01 5780 Quarry Rd\DETAILS\G2674-42-01 VicinityMap.dwg

GEOCON LEGEND

- **Qudf** ........UNDOCUMENTED FILL
- **Mzu** ........METAVOLCANIC ROCK (Dotted Where Buried)
- B-2 ....APPROX. LOCATION OF GEOTECHNICAL BORING
- (7') ....APPROX. DEPTH OF REMOVALS BELOW EXISTING GRADE
- T-12 ....APPROX. LOCATION OF TRENCH
- (10') ....APPROX. DEPTH TO POTENTIALLY NON-RIPPABLE ROCK
- ?........APPROX. LOCATION OF GEOLOGIC CONTACT (Queried Where Uncertain)
- B B' ....APPROX. LOCATION OF GEOLOGIC CROSS-SECTION
- ....APPROX. LOCATION OF INFILTRATION TEST
- ....APPROX. LIMITS OF HARD ROCK WITHIN BUILDING ENVELOPE. VERY HEAVY RIPPING OR ROCK BREAKER MAY BE REQUIRED. NOTE: ZONES OF HARD ROCK MAY EXIST BETWEEN BORINGS OR IN UNEXPLORED AREAS OF THE SITE

GEOLOGIC MAP

5780 QUARRY ROAD
BONITA, CALIFORNIA

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SCALE 1" = 40'	DATE 10 - 11 - 2021	
PROJECT NO. G2674 - 42 - 01		FIGURE 2
SHEET 1 OF 1		

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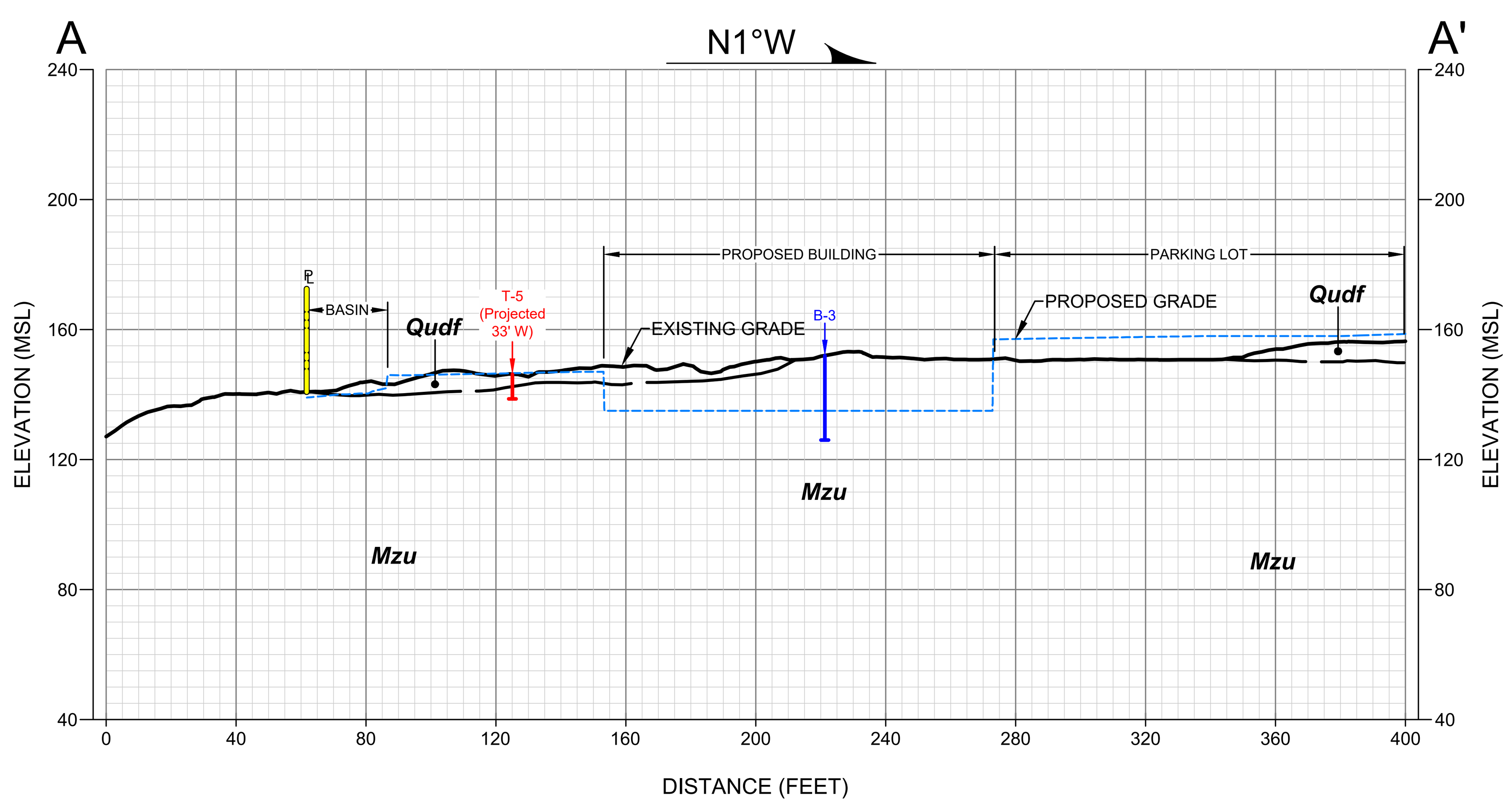
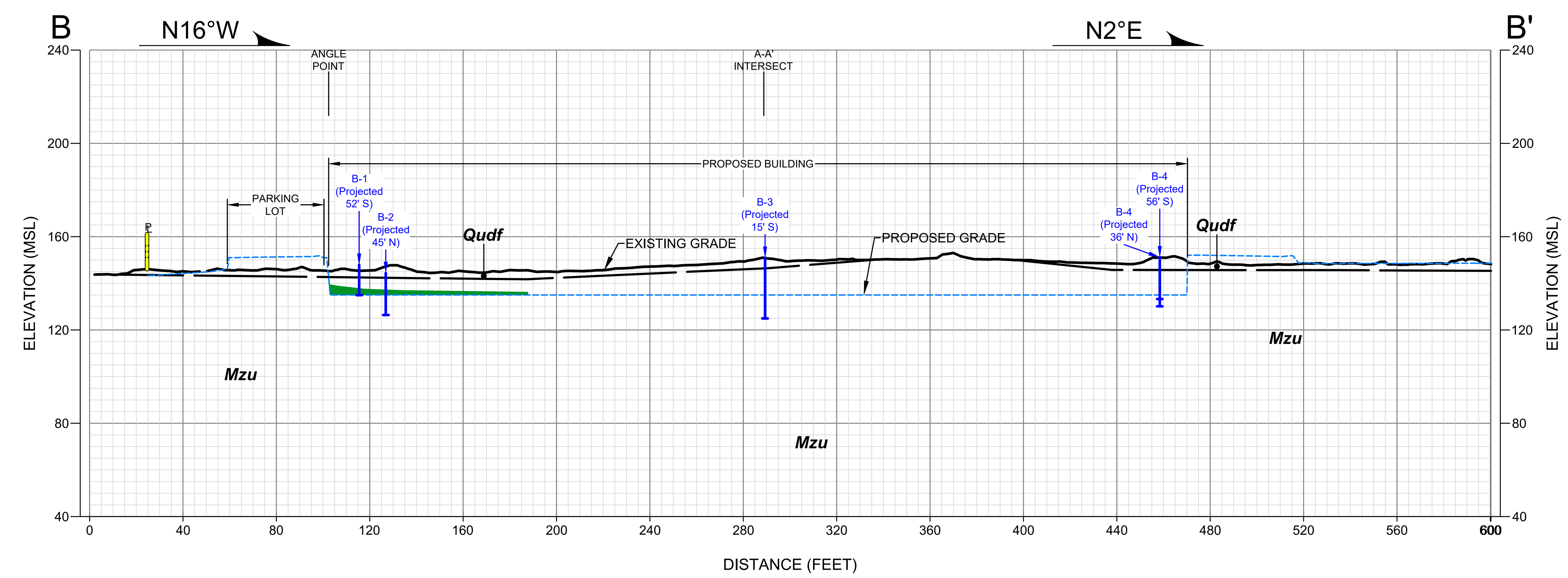


GEOLOGIC CROSS-SECTION A-A'

SCALE: 1" = 40' (Vert. = Horiz.)



GEOLOGIC CROSS-SECTION B-B'

SCALE: 1" = 40' (Vert. = Horiz.)

GEOCON LEGEND

- **Qudf** ........UNDOCUMENTED FILL
- **Mzu** ........METAVOLCANIC ROCK (Dotted Where Buried)
- B-5 ....APPROX. LOCATION OF GEOTECHNICAL BORING
- B-5 ....APPROX. LOCATION OF TRENCH
- ....APPROX. LOCATION OF GEOLOGIC CONTACT (Queried Where Uncertain)
- ....APPROX. LIMITS OF HARD ROCK; BREAKER MAY BE REQUIRED

GEOLOGIC CROSS - SECTION

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BONITA, CALIFORNIA

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SCALE	1" = 40'
DATE	10 - 11 - 2021
PROJECT NO.	G2674 - 42 - 01
FIGURE	3
SHEET	1 OF 1

APPENDIX A

APPENDIX A

FIELD INVESTIGATION

We performed our field investigation on February 17, 2021, which consisted of excavating twelve exploratory trenches to depths ranging from approximately 3 feet to 17 feet below existing grade. We performed a supplemental field investigation on September 13, 2021, which consisted of drilling five exploratory borings and four, infiltration tests using an M10 drill rig equipped with 8-inch hollow stem augers. If hard drilling or refusal was encountered, the drill rig switched to percussion drill using a 4-inch hammer bit. Borings were drilled to depths ranging from 13 feet to 26 feet and the infiltration test holes were drilled to depths between 4 feet to 5 feet. The approximate locations of the trenches, borings, and infiltration tests are shown on Figure 2.

We obtained samples during drilling using a California-modified, split-spoon sampler. We visually examined, classified, and logged the soil conditions encountered in general accordance with American Society for Testing and Materials (ASTM) practice for Description and Identification of Soils (Visual-Manual Procedure D 2488). Exploratory trench and boring logs are presented on Figures A-1 through A-17. The logs depict the various soil types encountered and indicate the depths at which samples were obtained.

BORING B 1

ELEV. (MSL.) **148'** DATE COMPLETED **09-13-2021**

EQUIPMENT **M10 DRILL RIG W/ 8" AUGER & 4" DTH** BY: **N. BORJA**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**TOPSOIL** Medium dense, damp, reddish brown to brown, Silty, fine to medium SAND; trace grave and cobble			
2								
4					**METAVOLCANIC ROCK** Weak, completely weathered, reddish brown, METAVOLCANIC ROCK			
6								
8								
10					-Becomes strong; potentially rippable; very hard digging			
12								
					BORING TERMINATED AT 13 FEET			

Figure A-13,
Log of Boring B 1, Page 1 of 1

G2674-42-01.GPJ

SAMPLE SYMBOLS			
	... SAMPLING UNSUCCESSFUL	... STANDARD PENETRATION TEST	... DRIVE SAMPLE (UNDISTURBED)
	... DISTURBED OR BAG SAMPLE	... CHUNK SAMPLE	... WATER TABLE OR ... SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

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PROJECT NO. G2674-42-01

BORING B 2

ELEV. (MSL.) **145'** DATE COMPLETED **09-13-2021**

EQUIPMENT **M10 DRILL RIG W/ 8" AUGER & 4" DTH** BY: **N. BORJA**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0 – 3					**TOPSOIL** Medium dense, damp, little reddish brown, Silty, fine to medium SAND; trace gravel and cobble			
3 – 18					**METAVOLCANIC ROCK** Strong, slightly weathered, blush gray, METAVOLCANIC ROCK; practical refusal at 6 feet; switched to percussion with 4 inch hammer hit			
5	B2-1					50/5"		
10					-Hand drilling below 10 feet			
18					BORING TERMINATED AT 18 FEET			

Figure A-14,
Log of Boring B 2, Page 1 of 1

G2674-42-01.GPJ

SAMPLE SYMBOLS ... SAMPLING UNSUCCESSFUL ... STANDARD PENETRATION TEST ... DRIVE SAMPLE (UNDISTURBED) ... DISTURBED OR BAG SAMPLE ... CHUNK SAMPLE ... WATER TABLE OR ... SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

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BORING B 3

ELEV. (MSL.) **151'** DATE COMPLETED **09-10-2021**

EQUIPMENT **YETI W/ 6" AUGER** BY: **N. BORJA**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**TOPSOIL** Loose, dry, light brown, Silty, fine to medium SAND; trace gravel			
4					**METAVOLCANIC ROCK** Weak, completely weathered, moist, dark reddish brown, METAVOLCANIC ROCK (saprolite); highly plastic			
	B3-1					22		
	B3-2							
10	B3-3					18		
12					Medium dense, moist, reddish brown, Silty, fine to coarse SAND; few gravel and cobble			
	B3-4					39		
18					Weak, weathered, damp, brown and bluish gray, METAVOLCANIC ROCK; fractured; rippable; poor recovery at 20 feet due to rock			
20	B3-5					71		
	B3-6					65/11"		
26					BORING TERMINATED AT 26 FEET			

Figure A-15,
Log of Boring B 3, Page 1 of 1

G2674-42-01.GPJ

SAMPLE SYMBOLS			
	... SAMPLING UNSUCCESSFUL	... STANDARD PENETRATION TEST	... DRIVE SAMPLE (UNDISTURBED)
	... DISTURBED OR BAG SAMPLE	... CHUNK SAMPLE	... WATER TABLE OR ... SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

PROJECT NO. G2674-42-01

BORING B 4

ELEV. (MSL.) **151'** DATE COMPLETED **09-10-2021**

EQUIPMENT **YETI W/ 6" AUGER** BY: **N. BORJA**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**TOPSOIL** Loose, dry, dark brown, Silty, fine to coarse SAND			
2					**METAVOLCANIC ROCK** Weak, completely weathered, reddish brown, METAVOLCANIC ROCK (saprolite)			
4	B4-1					30		
6	B4-2							
8					-Becomes weak, highly weathered; hard drilling at 8 feet			
10	B4-3					65/10.5"		
12					-Becomes moderately weak, weathered; very hard drilling at 11.5 feet			
14								
	B4-4				-No recovery due to rock	50/4"		
16	B4-5				Very dense, damp, reddish brown, Silty, fine to coarse SAND			
18								
20	B4-6				-No recovery	86/10"		

Figure A-16,
Log of Boring B 4, Page 1 of 1

G2674-42-01.GPJ

SAMPLE SYMBOLS
... SAMPLING UNSUCCESSFUL
... STANDARD PENETRATION TEST
... DRIVE SAMPLE (UNDISTURBED)
... DISTURBED OR BAG SAMPLE
... CHUNK SAMPLE
... WATER TABLE OR ... SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

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BORING B 5

ELEV. (MSL.) **149'** DATE COMPLETED **09-10-2021**

EQUIPMENT **YETI W/ 6" AUGER** BY: **N. BORJA**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**TOPSOIL** Loose to medium dense, dry, light brown, Silty, fine to medium SAND			
2								
4					**METAVOLCANIC ROCK** Weak, completely weathered, damp, dark reddish brown, METAVOLCANIC ROCK			
	B5-1					70/10"		
6					-Hard drilling at 6 feet			
8					-Becomes moderately weathered			
10	B5-2				-No recovery due to rock	76/8.5"		
12								
14								
	B5-3				-No recovery due to rock	96/9"		
16					BORING TERMINATED AT 16 FEET			

Figure A-17,
Log of Boring B 5, Page 1 of 1

G2674-42-01.GPJ

SAMPLE SYMBOLS	... SAMPLING UNSUCCESSFUL	... STANDARD PENETRATION TEST	... DRIVE SAMPLE (UNDISTURBED)
	... DISTURBED OR BAG SAMPLE	... CHUNK SAMPLE	... WATER TABLE OR ... SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

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APPENDIX B

APPENDIX B

LABORATORY TESTING

We performed laboratory tests in accordance with generally accepted test methods of the American Society for Testing and Materials (ASTM) or other suggested procedures. We tested selected samples for maximum dry density and optimum moisture content, expansion index, water-soluble sulfate exposure, chloride content, and resistance value (R-value). The results of our laboratory tests are presented on the following tables, summary sheets, and graphs. The exploratory boring logs are presented in Appendix A.

SUMMARY OF LABORATORY MAXIMUM DRY DENSITY AND OPTIMUM MOISTURE CONTENT TEST RESULTS ASTM D 1557

Sample No.	Description	Maximum Dry Density (pcf)	Optimum Moisture Content (% dry wt.)
T3-1	Reddish brown, Sandy CLAY	119.1	13.0

SUMMARY OF LABORATORY EXPANSION INDEX TEST RESULTS ASTM D 4829

Sample No.	Moisture Content (%)		Dry Density (pcf)	Expansion Index	Expansion Classification
	Before Test	After Test			
T1-1	10.8	21.9	107.7	59	Medium
T6-1	12.6	29.2	100.5	83	Medium
T7-1	13.2	28.6	99.5	53	Medium

SUMMARY OF LABORATORY WATER-SOLUBLE SULFATE TEST RESULTS CALIFORNIA TEST NO. 417

Sample No.	Water-Soluble Sulfate (%)	Classification
T1-1	0.008	S0
T6-1	0.003	S0
T7-1	0.005	S0

SUMMARY OF LABORATORY CHLORIDE ION TEST RESULTS
AASHTO T 291

Sample No.	Chloride Ion Content (ppm)	Chloride Ion Content (%)
T1-1	148	0.015
T6-1	88	0.009
T7-1	171	0.017

SUMMARY OF LABORATORY RESISTANCE VALUE (R-VALUE) TEST RESULTS
ASTM D 2844

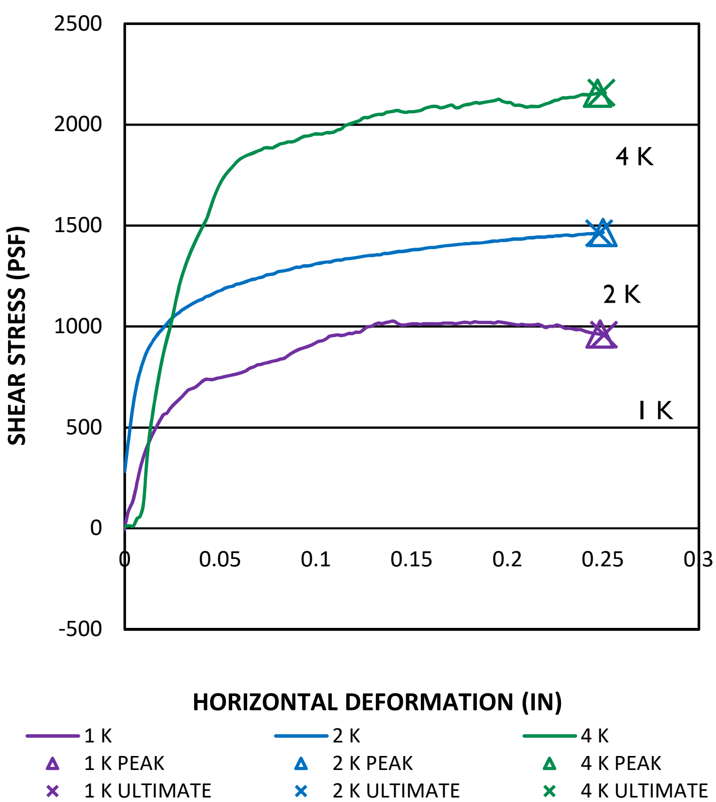
Sample No.	R-Value
B5-1	6

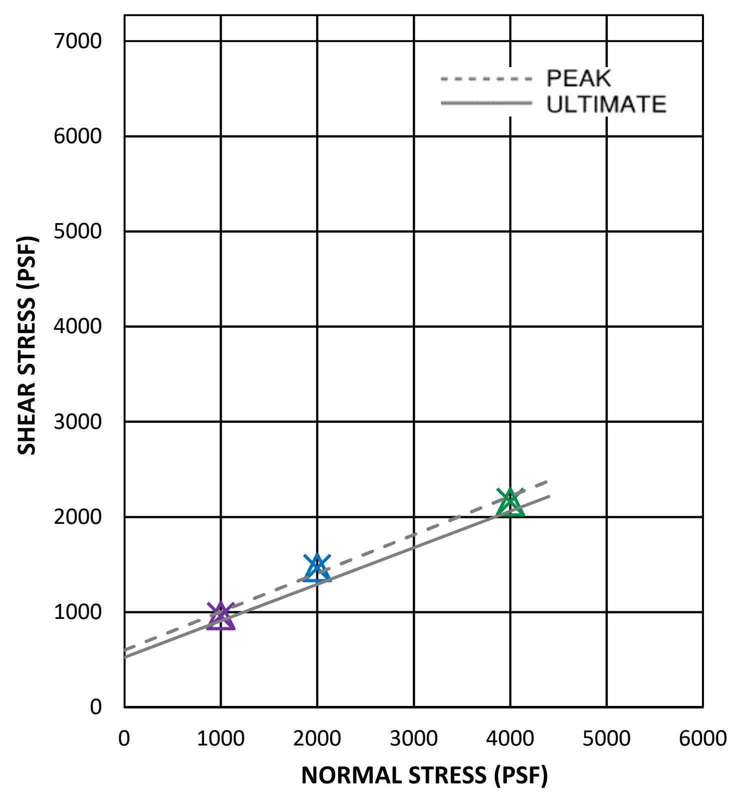
SAMPLE NO.:	T3-1	GEOLOGIC UNIT:	Topsoil
SAMPLE DEPTH (FT):	2.5	NATURAL/REMOLDED:	R

INITIAL CONDITIONS				
NORMAL STRESS TEST LOAD	1 K	2 K	4 K	AVERAGE
ACTUAL NORMAL STRESS (PSF):	1000	2000	4000	--
WATER CONTENT (%):	13.8	13.5	13.2	13.5
DRY DENSITY (PCF):	106.6	106.7	107.1	106.8

AFTER TEST CONDITIONS				
NORMAL STRESS TEST LOAD	1 K	2 K	4 K	AVERAGE
WATER CONTENT (%):	22.7	22.7	22.7	22.7
PEAK SHEAR STRESS (PSF):	962	1466	2155	--
ULT.-E.O.T. SHEAR STRESS (PSF):	959	1463	2158	--

RESULTS		
PEAK	COHESION, C (PSF)	600
	FRICTION ANGLE (DEGREES)	22
ULTIMATE	COHESION, C (PSF)	525
	FRICTION ANGLE (DEGREES)	21





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DIRECT SHEAR - ASTM D 3080

5780 QUARRY ROAD

PROJECT NO.: G2674-42-01

- Based on published geologic maps, the site is underlain by the Ardath Shale. A landslide is mapped north starting two houses north of the property. Based on our review of published geologic maps and the geotechnical investigation prepared by GEI, it does not appear the property is located on a landslide feature.

- Grading resulted in the building pad being underlain by a cut to fill transition. The eastern approximately two-thirds of the building is overlies native Ardath Shale. The west side of the lot and the southwest corner of the building pad is underlain by compacted fill.

- Caissons that extend to native Ardath Shale bedrock were used to support the portion of the structure and the pool in the southwest corner of the building pad. The foundation for the remainder of the structure is on shallow footings bearing on the native Ardath Shale.

- The inspection logs in the as-graded report do not document the bottom elevation of the caissons, however, the grading report indicates that all caissons were extended to a minimum depth of 10 feet into native formational Ardath Shale.

- The compacted fill placed during grading and behind building and retaining walls was compacted to at least 90 percent relative compaction based on our review of the grading report.

- Wall drains were documented by the geotechnical engineer.

- The report indicates that low expansive backfill soils were used as wall backfill, however, no laboratory testing was provided in the report. We requested the laboratory testing data from the geotechnical engineer, but have not received anything as of today.

- We did not perform slope stability analysis to check GEI's analysis; however, there was no laboratory testing provided in their geotechnical investigation report that documents the strength parameters used in their analysis.

- As indicated above, we were not able to obtain laboratory test data from the geotechnical engineer to confirm that the soils utilized as wall backfill meet the requirements listed on the wall plans.

<u>Summary of Findings:</u>

1. The building foundation is bearing on native formational Ardath Shale. This should provide suitable support for the foundation.

2. Fills were compacted to a minimum 90 percent relative compaction, which is the Standard of Care in Southern California.

3. No laboratory test data was provided so we cannot assess the following:

    a. If proper soil strengths were used in the stability analysis. There is no laboratory test data that supports the values utilized;
    b. The expansion characteristics and sulfate concentration of finish grade soils;
    c. If the walls were backfilled with soil that meets the requirements shown on the wall plans (i.e, shear strength, gradation, plasticity index, expansion) and if the soil in the foundation and retained zones of the walls meets the shear strength listed on the plans.

APPENDIX C

APPENDIX C

STORM WATER MANAGEMENT

We understand storm water management devices are being proposed in accordance with the current Storm Water Standards (SWS). If not properly constructed, there is a potential for distress to improvements and properties located hydrologically down gradient or adjacent to these devices. Factors such as the amount of water to be detained, its residence time, and soil permeability have an important effect on seepage transmission and the potential adverse impacts that may occur if the storm water management features are not properly designed and constructed. We have not performed a hydrogeological study at the site. If infiltration of storm water runoff occurs, downstream properties and improvements may be subjected to seeps, springs, slope instability, raised groundwater, movement of foundations and slabs, or other undesirable impacts as a result of water infiltration.

Hydrologic Soil Group

The United States Department of Agriculture (USDA), Natural Resources Conservation Services, possesses general information regarding the existing soil conditions for areas within the United States. The USDA website also provides the Hydrologic Soil Group. Table C-1 presents the descriptions of the hydrologic soil groups. In addition, the USDA website also provides an estimated saturated hydraulic conductivity for the existing soil.

TABLE C-1
HYDROLOGIC SOIL GROUP DEFINITIONS

Soil Group	Soil Group Definition
A	Soils having a high infiltration rate (low runoff potential) when thoroughly wet. These consist mainly of deep, well drained to excessively drained sands or gravelly sands. These soils have a high rate of water transmission.
B	Soils having a moderate infiltration rate when thoroughly wet. These consist chiefly of moderately deep or deep, moderately well drained or well drained soils that have moderately fine texture to moderately coarse texture. These soils have a moderate rate of water transmission.
C	Soils having a slow infiltration rate when thoroughly wet. These consist chiefly of soils having a layer that impedes the downward movement of water or soils of moderately fine texture or fine texture. These soils have a slow rate of water transmission.
D	Soils having a very slow infiltration rate (high runoff potential) when thoroughly wet. These consist chiefly of clays that have a high shrink-swell potential, soils that have a high water table, soils that have a claypan or clay layer at or near the surface, and soils that are shallow over nearly impervious material. These soils have a very slow rate of water transmission.

The property is underlain by undocumented fill, topsoil, and Metavolcanic Rock. Table C-2 presents the information from the USDA website for the subject property.

TABLE C-2
USDA WEB SOIL SURVEY – HYDROLOGIC SOIL GROUP

Map Unit Name	Map Unit Symbol	Approximate Percentage of Property	Hydrologic Soil Group
Auld clay, 5 to 9 percent slopes	AwC	100	C

Infiltration Testing

We performed five infiltration tests at the locations shown on Figure 2. The tests were performed in 8-inch-diameter boreholes excavated by an M10 drill rig. Table C-3 presents the results of the testing. The calculation sheets are also attached.

We used the guidelines presented in the Riverside County Low Impact Development BMP Design Handbook. Based on this widely accepted guideline, the saturated hydraulic conductivity (Ksat) is equivalent to the infiltration rate. Therefore, the Ksat value determined from our testing is assumed to be the unfactored infiltration rate.

TABLE C-3
UNFACTORED, FIELD-SATURATED, INFILTRATION TEST RESULTS

Test No.	Depth (inches)	Geologic Unit	Field Infiltration Rate, I (in/hr)	Factored* Field Infiltration Rate, I (in/hr)
A-1	61	Topsoil	0.002	0.001
A-2	57	Topsoil	0.002	0.001
A-3	48	Topsoil	0.420	0.210
A-4	59	Topsoil	0.036	0.018

* Factor of Safety of 2.0 for feasibility determination.

STORM WATER MANAGEMENT CONCLUSIONS

Soil Types

Undocumented Fill (Qudf) – We encountered existing fills during our field investigation in all of the exploratory trenches, except T-1, T-3, and T-6. The fill thicknesses range from approximately 1 to 3.5 feet. Deeper areas of fill may be present within the areas of utility easements, or in unexplored areas of the site. Fill is also present in the slope along the western property boundary. Water that is allowed to migrate into the undocumented fill will cause settlement. Therefore, full and partial infiltration should be considered infeasible within undocumented fill.

Topsoil (Unmapped) – We encountered topsoil varying between 0.5 to 5.5 feet thick across the site. Topsoil within structural improvement areas will be removed and replaced with compacted fill. Water that is allowed to migrate into the topsoil may cause soil movement (heave or settlement). Therefore, full and partial infiltration should be considered infeasible within topsoil.

Metavolcanic Rock (Mzu) – Metavolcanic Rock underlies the undocumented fill and topsoil. varying from weak to strong and completely to moderately weathered rock. Rock outcrops are exposed in the northwest portion of the site. Infiltration into rock is not feasible due to low infiltration characteristics.

Groundwater Elevation

We did not encounter groundwater during our subsurface investigation to depths of up to 26 feet below the existing ground surface. Infiltration should not impact groundwater.

Existing Utilities

No known utilities cross the site. Infiltration due to utility concerns would be feasible.

Soil or Groundwater Contamination

We are unaware of contaminated soil or groundwater on the property. Therefore, full and partial infiltration associated with this risk is considered feasible.

Slopes

There are no existing slopes that would be impacted by infiltration.

Infiltration Rates

Our test results indicated infiltration rates between 0.002 and 0.420 in/hr. Infiltration rates in the area of A-1, A-2, and A-4 range from 0.002 in/hr to 0.036 in/hr. The average infiltration rate at the location of these tests are 0.013 in/hr (factored infiltration rate 0.007 in/hr) which are not high enough to support full or partial infiltration. However, in the area of A-3, the infiltration rate of 0.420 in/hr (factored infiltration rate 0.210 in/hr) is high enough to support partial infiltration.

Storm Water Management Devices

Liners and subdrains should be incorporated into the design and construction of the planned storm water devices. The liners should be impermeable (e.g. High-density polyethylene, HDPE, with a thickness of about 30 mil or equivalent Polyvinyl Chloride, PVC) to prevent water migration. The subdrains should be perforated within the liner area, installed at the base and above the liner, be at least 3 inches in diameter and consist of Schedule 40 PVC pipe. The subdrains outside of the liner should consist of solid pipe. The penetration of the liners at the subdrains should be properly waterproofed. The subdrains should be connected to a proper outlet. The devices should also be installed in accordance with the manufacturer's recommendations. Liners should be installed on the side walls of the proposed basins in accordance with a partial infiltration design.

Storm Water Standard Worksheets

The SWS requests the geotechnical engineer complete the *Categorization of Infiltration Feasibility Condition* (Worksheet C.4-1) worksheet information to help evaluate the potential for infiltration on the property. The attached Worksheet C.4-1 presents the completed information for the submittal process.

The regional storm water standards also have a worksheet (Worksheet Form D.5-1) that helps the project civil engineer estimate the factor of safety based on several factors. Table C-4 describes the suitability assessment input parameters related to the geotechnical engineering aspects for the factor of safety determination.

TABLE C-4
SUITABILITY ASSESSMENT RELATED CONSIDERATIONS FOR INFILTRATION FACILITY SAFETY FACTORS

Consideration	High Concern – 3 Points	Medium Concern – 2 Points	Low Concern – 1 Point
Assessment Methods	Use of soil survey maps or simple texture analysis to estimate short-term infiltration rates. Use of well permeameter or borehole methods without accompanying continuous boring log. Relatively sparse testing with direct infiltration methods	Use of well permeameter or borehole methods with accompanying continuous boring log. Direct measurement of infiltration area with localized infiltration measurement methods (e.g., Infiltrometer). Moderate spatial resolution	Direct measurement with localized (i.e. small-scale) infiltration testing methods at relatively high resolution or use of extensive test pit infiltration measurement methods.
Predominant Soil Texture	Silty and clayey soils with significant fines	Loamy soils	Granular to slightly loamy soils
Site Soil Variability	Highly variable soils indicated from site assessment or unknown variability	Soil boring/test pits indicate moderately homogenous soils	Soil boring/test pits indicate relatively homogenous soils
Depth to Groundwater/ Impervious Layer	<5 feet below facility bottom	5-15 feet below facility bottom	>15 feet below facility bottom

Table C-5 presents the estimated factor values for the evaluation of the factor of safety. This table only presents the suitability assessment safety factor (Part A) of the worksheet. The project civil engineer should evaluate the safety factor for design (Part B) and use the combined safety factor for the design infiltration rate.

TABLE C-5
FACTOR OF SAFETY WORKSHEET D.5-1 DESIGN VALUES[1]

Suitability Assessment Factor Category	Assigned Weight (w)	Factor Value (v)	Product (p = w x v)
Assessment Methods	0.25	2	0.50
Predominant Soil Texture	0.25	3	0.75
Site Soil Variability	0.25	2	0.50
Depth to Groundwater/Impervious Layer	0.25	1	0.25
Suitability Assessment Safety Factor, $S_A = \Sigma p$			2.0

[1] The project civil engineer should complete Worksheet D.5-1 using the data on this table. Additional information is required to evaluate the design factor of safety.

CONCLUSIONS

Our results indicate the site has relatively slow infiltration characteristics within the topsoil. Because of the site conditions, it is our opinion that there is a potential for lateral water migration. Undocumented fill and topsoil also exists on the property and has a high potential for adverse settlement or expansion when wetted. It is our opinion that full or partial infiltration is infeasible on this site. Our evaluation included the soil and geologic conditions, estimated settlement and volume change of the underlying soil, slope stability, utility considerations, groundwater mounding, retaining walls, foundations and existing groundwater elevations.

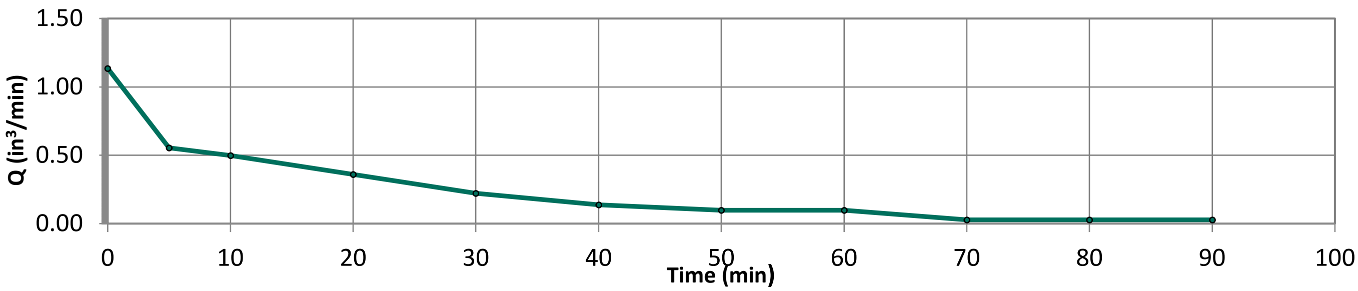
TEST NO.: A1

GEOLOGIC UNIT: Topsoil

EXCAVATION ELEVATION (MSL, FT): 153

TEST INFORMATION	
BOREHOLE DIAMETER (IN):	8
BOREHOLE DEPTH (FT):	5.1
TEST/BOTTOM ELEVATION (MSL, FT):	148
MEASURED HEAD HEIGHT (IN):	4.5
CALCULATED HEAD HEIGHT (IN):	6.3
FACTOR OF SAFETY:	2.0

TEST RESULTS	
STEADY FLOW RATE (IN^3/MIN):	**0.028**
FIELD-SATURATED INFILTRATION RATE (IN/HR):	**0.002**
FACTORED INFILTRATION RATE (IN/HR):	**0.001**



TEST DATA

Reading	Time Elapsed (min)	Water Weight Consumed (lbs)	Water Volume Consumed (in^3)	Q (in^3/min)
1	0.00	0.000	0.00	0.00
2	5.00	0.205	5.68	1.135
3	5.00	0.100	2.77	0.554
4	10.00	0.180	4.98	0.498
5	10.00	0.130	3.60	0.360
6	10.00	0.080	2.22	0.222
7	10.00	0.050	1.38	0.138
8	10.00	0.035	0.97	0.097
9	10.00	0.035	0.97	0.097
10	10.00	0.010	0.28	0.028
11	10.00	0.010	0.28	0.028
12	10.00	0.010	0.28	0.028

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AARDVARK PERMEAMETER TEST RESULTS

Bonita Self Storage

PROJECT NO.: G2674-42-01

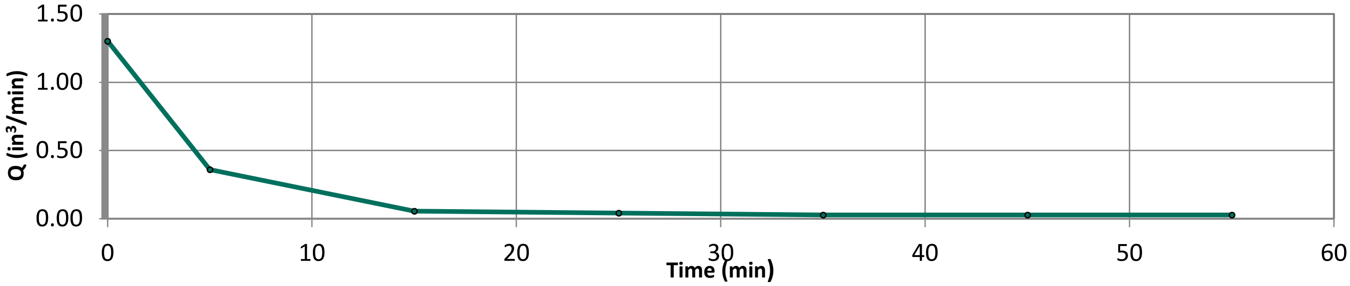
TEST NO.: **A2**

GEOLOGIC UNIT: **Topsoil**

EXCAVATION ELEVATION (MSL, FT): **152**

TEST INFORMATION	
BOREHOLE DIAMETER (IN):	8
BOREHOLE DEPTH (FT):	4.8
TEST/BOTTOM ELEVATION (MSL, FT):	147
MEASURED HEAD HEIGHT (IN):	4.5
CALCULATED HEAD HEIGHT (IN):	6.3
FACTOR OF SAFETY:	2.0

TEST RESULTS	
STEADY FLOW RATE (IN^3/MIN):	**0.028**
FIELD-SATURATED INFILTRATION RATE (IN/HR):	**0.002**
FACTORED INFILTRATION RATE (IN/HR):	**0.001**



TEST DATA

Reading	Time Elapsed (min)	Water Weight Consumed (lbs)	Water Volume Consumed (in^3)	Q (in^3/min)
1	0.00	0.000	0.00	0.00
2	5.00	0.235	6.51	1.302
3	10.00	0.130	3.60	0.360
4	10.00	0.020	0.55	0.055
5	10.00	0.015	0.42	0.042
6	10.00	0.010	0.28	0.028
7	10.00	0.010	0.28	0.028
8	10.00	0.010	0.28	0.028

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AARDVARK PERMEAMETER TEST RESULTS

Bonita Self Storage

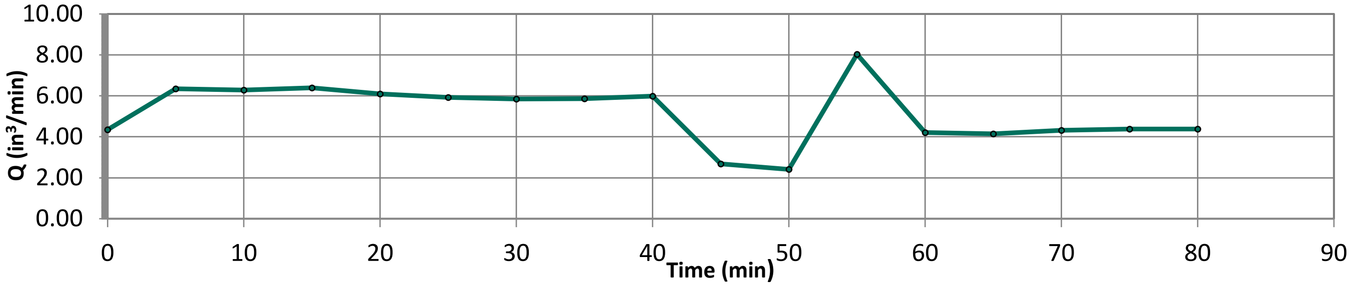
PROJECT NO.: **G2674-42-01**

TEST NO.: **A3** GEOLOGIC UNIT: **Topsoil**

EXCAVATION ELEVATION (MSL, FT): **142**

TEST INFORMATION	
BOREHOLE DIAMETER (IN):	8
BOREHOLE DEPTH (FT):	4.0
TEST/BOTTOM ELEVATION (MSL, FT):	138
MEASURED HEAD HEIGHT (IN):	4.0
CALCULATED HEAD HEIGHT (IN):	6.1
FACTOR OF SAFETY:	2.0

TEST RESULTS	
STEADY FLOW RATE (IN^3/MIN):	**4.357**
FIELD-SATURATED INFILTRATION RATE (IN/HR):	**0.420**
FACTORED INFILTRATION RATE (IN/HR):	**0.210**



TEST DATA

Reading	Time Elapsed (min)	Water Weight Consumed (lbs)	Water Volume Consumed (in^3)	Q (in^3/min)
1	0.00	0.000	0.00	0.00
2	5.00	0.785	21.74	4.348
3	5.00	1.145	31.71	6.342
4	5.00	1.135	31.43	6.286
5	5.00	1.155	31.98	6.397
6	5.00	1.100	30.46	6.092
7	5.00	1.070	29.63	5.926
8	5.00	1.055	29.22	5.843
9	5.00	1.060	29.35	5.871
10	5.00	1.080	29.91	5.982
11	5.00	0.485	13.43	2.686
12	5.00	0.435	12.05	2.409
13	5.00	1.450	40.15	8.031
14	5.00	0.760	21.05	4.209
15	5.00	0.750	20.77	4.154
16	5.00	0.780	21.60	4.320
17	5.00	0.790	21.88	4.375
18	5.00	0.790	21.88	4.375

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AARDVARK PERMEAMETER TEST RESULTS

Bonita Self Storage

PROJECT NO.: **G2674-42-01**

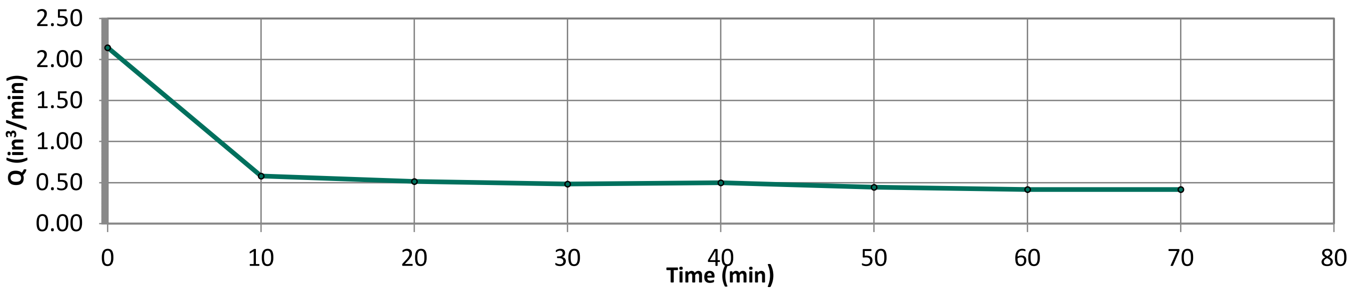
TEST NO.: **A4**

GEOLOGIC UNIT: **Topsoil**

EXCAVATION ELEVATION (MSL, FT): **146**

TEST INFORMATION	
BOREHOLE DIAMETER (IN):	8
BOREHOLE DEPTH (FT):	4.9
TEST/BOTTOM ELEVATION (MSL, FT):	141
MEASURED HEAD HEIGHT (IN):	5.0
CALCULATED HEAD HEIGHT (IN):	6.2
FACTOR OF SAFETY:	2.0

TEST RESULTS	
STEADY FLOW RATE (IN^3/MIN):	**0.425**
FIELD-SATURATED INFILTRATION RATE (IN/HR):	**0.036**
FACTORED INFILTRATION RATE (IN/HR):	**0.018**



TEST DATA

Reading	Time Elapsed (min)	Water Weight Consumed (lbs)	Water Volume Consumed (in^3)	Q (in^3/min)
1	0.00	0.000	0.00	0.00
2	10.00	0.775	21.46	2.146
3	10.00	0.210	5.82	0.582
4	10.00	0.185	5.12	0.512
5	10.00	0.175	4.85	0.485
6	10.00	0.180	4.98	0.498
7	10.00	0.160	4.43	0.443
8	10.00	0.150	4.15	0.415
9	10.00	0.150	4.15	0.415

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AARDVARK PERMEAMETER TEST RESULTS

Bonita Self Storage

PROJECT NO.: **G2674-42-01**

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

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

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

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

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

Summarize infiltration testing methods, testing locations, replicates, and results and summarize estimates of reliable infiltration rates according to procedures outlined in D.5. Documentation should be included in project geotechnical report.

We performed four infiltration tests in an 8-inch-diameter borehole excavated by an M10 drill rig. Our test results indicated infiltration rates between 0.002 and 0.420 in/hr. Infiltration rates in the area of A-1, A-2, and A-4 range from 0.002 in/hr to 0.036 in/hr. The average infiltration rate at the location of these tests are 0.013 in/hr (factored infiltration rate 0.007 in/hr) which are not high enough to support full or partial infiltration. However, in the area of A-3, the infiltration rate of 0.420 in/hr (factored infiltration rate 0.210 in/hr) is high enough to support partial infiltration.

Our results indicate the site has relatively slow infiltration characteristics within the topsoil. Because of the site conditions, it is our opinion that there is a potential for lateral water migration. Undocumented fill and topsoil also exists on the property and has a high potential for adverse settlement or expansion when wetted. It is our opinion that full or partial infiltration is infeasible on this site.

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

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

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

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

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

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

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions		Worksheet C.4-1: Form I-8A[10]
Part 2 – Partial vs. No Infiltration Feasibility Screening Criteria		
DMA(s) Being Analyzed:		Project Phase:
PROPOSED MODULAR WETLAND TREATMENT BIOFIL[T]		DESIGN
Criteria 3 : Infiltration Rate Screening		
3A	**NRCS Type C, D, or "urban/unclassified":** Is the mapped hydrologic soil group according to the NRCS Web Soil Survey or UC Davis Soil Web Mapper is Type C, D, or "urban/unclassified" and corroborated by available site soil data? ☐ Yes; the site is mapped as C soils and a reliable infiltration rate of 0.15 in/hr. is used to size partial infiltration BMPS. Answer "Yes" to Criteria 3 Result. ☐ Yes; the site is mapped as D soils or "urban/unclassified" and a reliable infiltration rate of 0.05 in/hr. is used to size partial infiltration BMPS. Answer "Yes" to Criteria 3 Result. ☑ No; infiltration testing is conducted (refer to Table D.3-1), continue to Step 3B.	
3B	**Infiltration Testing Result:** Is the reliable infiltration rate (i.e. average measured infiltration rate/2) greater than 0.05 in/hr. and less than or equal to 0.5 in/hr? ☐ Yes; the site may support partial infiltration. Answer "Yes" to Criteria 3 Result. ☑ No; the reliable infiltration rate (i.e. average measured rate/2) is less than 0.05 in/hr., partial infiltration is not required. Answer "No" to Criteria 3 Result.	
Criteria 3 Result	Is the estimated reliable infiltration rate (i.e., average measured infiltration rate/2) greater than or equal to 0.05 inches/hour and less than or equal to 0.5 inches/hour at any location within each DMA where runoff can reasonably be routed to a BMP? ☑ Yes; Continue to Criteria 4. ☐ No: Skip to Part 2 Result.	
Summarize infiltration testing and/or mapping results (i.e. soil maps and series description used for infiltration rate). We performed four infiltration tests in an 8-inch-diameter borehole excavated by an M10 drill rig. Our test results indicated infiltration rates between 0.002 and 0.420 in/hr. Infiltration rates in the area of A-1, A-2, and A-4 range from 0.002 in/hr to 0.036 in/hr. The average infiltration rate at the location of these tests are 0.013 in/hr (factored infiltration rate 0.007 in/hr) which are not high enough to support full or partial infiltration. However, in the area of A-3, the infiltration rate of 0.420 in/hr (factored infiltration rate 0.210 in/hr) is high enough to support partial infiltration.		

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

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

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

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

We performed four infiltration tests in an 8-inch-diameter borehole excavated by an M10 drill rig. Our test results indicated infiltration rates between 0.002 and 0.420 in/hr. Infiltration rates in the area of A-1, A-2, and A-4 range from 0.002 in/hr to 0.036 in/hr. The average infiltration rate at the location of these tests are 0.013 in/hr (factored infiltration rate 0.007 in/hr) which are not high enough to support full or partial infiltration. However, in the area of A-3, the infiltration rate of 0.420 in/hr (factored infiltration rate 0.210 in/hr) is high enough to support partial infiltration.

Our results indicate the site has relatively slow infiltration characteristics within the topsoil. Because of the site conditions, it is our opinion that there is a potential for lateral water migration. Undocumented fill and topsoil also exists on the property and has a high potential for adverse settlement or expansion when wetted. It is our opinion that full or partial infiltration is infeasible on this site.

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

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

APPENDIX D

APPENDIX D

RECOMMENDED GRADING SPECIFICATIONS

FOR

5780 QUARRY ROAD
BONITA, CALIFORNIA

PROJECT NO. G2674-42-01

RECOMMENDED GRADING SPECIFICATIONS

1. GENERAL

1.1 These Recommended Grading Specifications shall be used in conjunction with the Geotechnical Report for the project prepared by Geocon. The recommendations contained in the text of the Geotechnical Report are a part of the earthwork and grading specifications and shall supersede the provisions contained hereinafter in the case of conflict.

1.2 Prior to the commencement of grading, a geotechnical consultant (Consultant) shall be employed for the purpose of observing earthwork procedures and testing the fills for substantial conformance with the recommendations of the Geotechnical Report and these specifications. The Consultant should provide adequate testing and observation services so that they may assess whether, in their opinion, the work was performed in substantial conformance with these specifications. It shall be the responsibility of the Contractor to assist the Consultant and keep them apprised of work schedules and changes so that personnel may be scheduled accordingly.

1.3 It shall be the sole responsibility of the Contractor to provide adequate equipment and methods to accomplish the work in accordance with applicable grading codes or agency ordinances, these specifications and the approved grading plans. If, in the opinion of the Consultant, unsatisfactory conditions such as questionable soil materials, poor moisture condition, inadequate compaction, and/or adverse weather result in a quality of work not in conformance with these specifications, the Consultant will be empowered to reject the work and recommend to the Owner that grading be stopped until the unacceptable conditions are corrected.

2. DEFINITIONS

2.1 **Owner** shall refer to the owner of the property or the entity on whose behalf the grading work is being performed and who has contracted with the Contractor to have grading performed.

2.2 **Contractor** shall refer to the Contractor performing the site grading work.

2.3 **Civil Engineer** or **Engineer of Work** shall refer to the California licensed Civil Engineer or consulting firm responsible for preparation of the grading plans, surveying and verifying as-graded topography.

2.4 **Consultant** shall refer to the soil engineering and engineering geology consulting firm retained to provide geotechnical services for the project.

2.5 **Soil Engineer** shall refer to a California licensed Civil Engineer retained by the Owner, who is experienced in the practice of geotechnical engineering. The Soil Engineer shall be responsible for having qualified representatives on-site to observe and test the Contractor's work for conformance with these specifications.

2.6 **Engineering Geologist** shall refer to a California licensed Engineering Geologist retained by the Owner to provide geologic observations and recommendations during the site grading.

2.7 **Geotechnical Report** shall refer to a soil report (including all addenda) which may include a geologic reconnaissance or geologic investigation that was prepared specifically for the development of the project for which these Recommended Grading Specifications are intended to apply.

3. MATERIALS

3.1 Materials for compacted fill shall consist of any soil excavated from the cut areas or imported to the site that, in the opinion of the Consultant, is suitable for use in construction of fills. In general, fill materials can be classified as *soil* fills, *soil-rock* fills or *rock* fills, as defined below.

3.1.1 **Soil fills** are defined as fills containing no rocks or hard lumps greater than 12 inches in maximum dimension and containing at least 40 percent by weight of material smaller than ¾ inch in size.

3.1.2 **Soil-rock fills** are defined as fills containing no rocks or hard lumps larger than 4 feet in maximum dimension and containing a sufficient matrix of soil fill to allow for proper compaction of soil fill around the rock fragments or hard lumps as specified in Paragraph 6.2. **Oversize rock** is defined as material greater than 12 inches.

3.1.3 **Rock fills** are defined as fills containing no rocks or hard lumps larger than 3 feet in maximum dimension and containing little or no fines. Fines are defined as material smaller than ¾ inch in maximum dimension. The quantity of fines shall be less than approximately 20 percent of the rock fill quantity.

3.2 Material of a perishable, spongy, or otherwise unsuitable nature as determined by the Consultant shall not be used in fills.

3.3 Materials used for fill, either imported or on-site, shall not contain hazardous materials as defined by the California Code of Regulations, Title 22, Division 4, Chapter 30, Articles 9 and 10; 40CFR; and any other applicable local, state or federal laws. The Consultant shall not be responsible for the identification or analysis of the potential presence of hazardous materials. However, if observations, odors or soil discoloration cause Consultant to suspect the presence of hazardous materials, the Consultant may request from the Owner the termination of grading operations within the affected area. Prior to resuming grading operations, the Owner shall provide a written report to the Consultant indicating that the suspected materials are not hazardous as defined by applicable laws and regulations.

3.4 The outer 15 feet of *soil-rock* fill slopes, measured horizontally, should be composed of properly compacted *soil* fill materials approved by the Consultant. *Rock* fill may extend to the slope face, provided that the slope is not steeper than 2:1 (horizontal:vertical) and a soil layer no thicker than 12 inches is track-walked onto the face for landscaping purposes. This procedure may be utilized provided it is acceptable to the governing agency, Owner and Consultant.

3.5 Samples of soil materials to be used for fill should be tested in the laboratory by the Consultant to determine the maximum density, optimum moisture content, and, where appropriate, shear strength, expansion, and gradation characteristics of the soil.

3.6 During grading, soil or groundwater conditions other than those identified in the Geotechnical Report may be encountered by the Contractor. The Consultant shall be notified immediately to evaluate the significance of the unanticipated condition.

4. CLEARING AND PREPARING AREAS TO BE FILLED

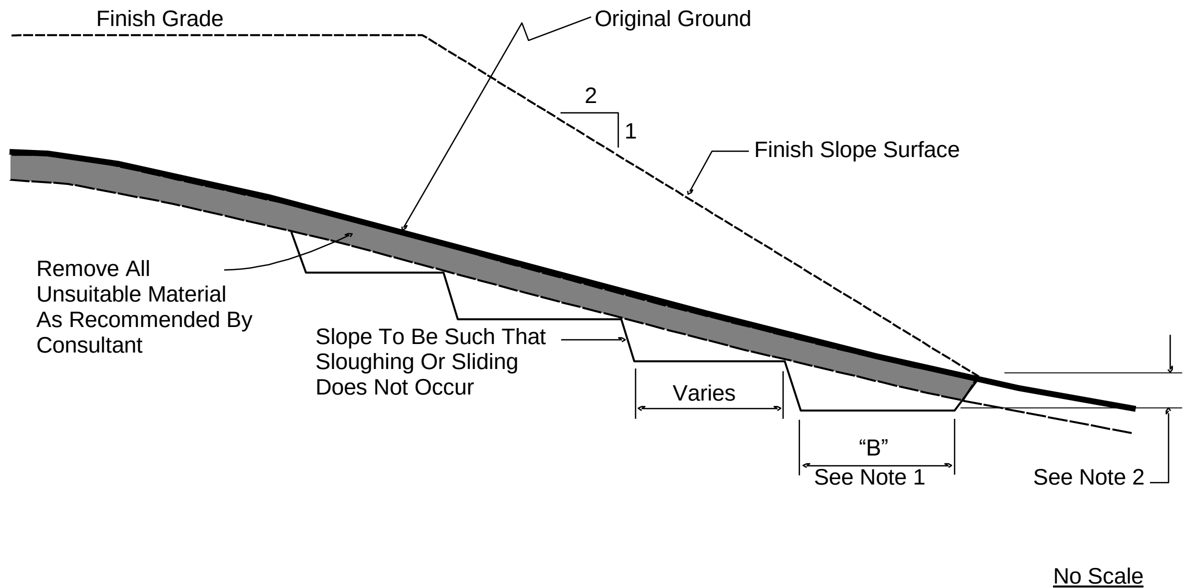
4.1 Areas to be excavated and filled shall be cleared and grubbed. Clearing shall consist of complete removal above the ground surface of trees, stumps, brush, vegetation, man-made structures, and similar debris. Grubbing shall consist of removal of stumps, roots, buried logs and other unsuitable material and shall be performed in areas to be graded. Roots and other projections exceeding 1½ inches in diameter shall be removed to a depth of 3 feet below the surface of the ground. Borrow areas shall be grubbed to the extent necessary to provide suitable fill materials.

4.2 Asphalt pavement material removed during clearing operations should be properly disposed at an approved off-site facility or in an acceptable area of the project evaluated by Geocon and the property owner. Concrete fragments that are free of reinforcing steel may be placed in fills, provided they are placed in accordance with Section 6.2 or 6.3 of this document.

4.3 After clearing and grubbing of organic matter and other unsuitable material, loose or porous soils shall be removed to the depth recommended in the Geotechnical Report. The depth of removal and compaction should be observed and approved by a representative of the Consultant. The exposed surface shall then be plowed or scarified to a minimum depth of 6 inches and until the surface is free from uneven features that would tend to prevent uniform compaction by the equipment to be used.

4.4 Where the slope ratio of the original ground is steeper than 5:1 (horizontal:vertical), or where recommended by the Consultant, the original ground should be benched in accordance with the following illustration.

TYPICAL BENCHING DETAIL



DETAIL NOTES: (1) Key width "B" should be a minimum of 10 feet, or sufficiently wide to permit complete coverage with the compaction equipment used. The base of the key should be graded horizontal, or inclined slightly into the natural slope.

(2) The outside of the key should be below the topsoil or unsuitable surficial material and at least 2 feet into dense formational material. Where hard rock is exposed in the bottom of the key, the depth and configuration of the key may be modified as approved by the Consultant.

4.5 After areas to receive fill have been cleared and scarified, the surface should be moisture conditioned to achieve the proper moisture content, and compacted as recommended in Section 6 of these specifications.

5. COMPACTION EQUIPMENT

5.1 Compaction of *soil* or *soil-rock* fill shall be accomplished by sheepsfoot or segmented-steel wheeled rollers, vibratory rollers, multiple-wheel pneumatic-tired rollers, or other types of acceptable compaction equipment. Equipment shall be of such a design that it will be capable of compacting the *soil* or *soil-rock* fill to the specified relative compaction at the specified moisture content.

5.2 Compaction of *rock* fills shall be performed in accordance with Section 6.3.

6. PLACING, SPREADING AND COMPACTION OF FILL MATERIAL

6.1 *Soil* fill, as defined in Paragraph 3.1.1, shall be placed by the Contractor in accordance with the following recommendations:

6.1.1 *Soil* fill shall be placed by the Contractor in layers that, when compacted, should generally not exceed 8 inches. Each layer shall be spread evenly and shall be thoroughly mixed during spreading to obtain uniformity of material and moisture in each layer. The entire fill shall be constructed as a unit in nearly level lifts. Rock materials greater than 12 inches in maximum dimension shall be placed in accordance with Section 6.2 or 6.3 of these specifications.

6.1.2 In general, the *soil* fill shall be compacted at a moisture content at or above the optimum moisture content as determined by ASTM D 1557.

6.1.3 When the moisture content of *soil* fill is below that specified by the Consultant, water shall be added by the Contractor until the moisture content is in the range specified.

6.1.4 When the moisture content of the *soil* fill is above the range specified by the Consultant or too wet to achieve proper compaction, the *soil* fill shall be aerated by the Contractor by blading/mixing, or other satisfactory methods until the moisture content is within the range specified.

6.1.5 After each layer has been placed, mixed, and spread evenly, it shall be thoroughly compacted by the Contractor to a relative compaction of at least 90 percent. Relative compaction is defined as the ratio (expressed in percent) of the in-place dry density of the compacted fill to the maximum laboratory dry density as determined in accordance with ASTM D 1557. Compaction shall be continuous over the entire area, and compaction equipment shall make sufficient passes so that the specified minimum relative compaction has been achieved throughout the entire fill.

6.1.6 Where practical, soils having an Expansion Index greater than 50 should be placed at least 3 feet below finish pad grade and should be compacted at a moisture content generally 2 to 4 percent greater than the optimum moisture content for the material.

6.1.7 Properly compacted *soil* fill shall extend to the design surface of fill slopes. To achieve proper compaction, it is recommended that fill slopes be over-built by at least 3 feet and then cut to the design grade. This procedure is considered preferable to track-walking of slopes, as described in the following paragraph.

6.1.8 As an alternative to over-building of slopes, slope faces may be back-rolled with a heavy-duty loaded sheepsfoot or vibratory roller at maximum 4-foot fill height intervals. Upon completion, slopes should then be track-walked with a D-8 dozer or similar equipment, such that a dozer track covers all slope surfaces at least twice.

6.2 *Soil-rock* fill, as defined in Paragraph 3.1.2, shall be placed by the Contractor in accordance with the following recommendations:

6.2.1 Rocks larger than 12 inches but less than 4 feet in maximum dimension may be incorporated into the compacted *soil* fill, but shall be limited to the area measured 15 feet minimum horizontally from the slope face and 5 feet below finish grade or 3 feet below the deepest utility, whichever is deeper.

6.2.2 Rocks or rock fragments up to 4 feet in maximum dimension may either be individually placed or placed in windrows. Under certain conditions, rocks or rock fragments up to 10 feet in maximum dimension may be placed using similar methods. The acceptability of placing rock materials greater than 4 feet in maximum dimension shall be evaluated during grading as specific cases arise and shall be approved by the Consultant prior to placement.

6.2.3 For individual placement, sufficient space shall be provided between rocks to allow for passage of compaction equipment.

6.2.4 For windrow placement, the rocks should be placed in trenches excavated in properly compacted *soil* fill. Trenches should be approximately 5 feet wide and 4 feet deep in maximum dimension. The voids around and beneath rocks should be filled with approved granular soil having a Sand Equivalent of 30 or greater and should be compacted by flooding. Windrows may also be placed utilizing an "open-face" method in lieu of the trench procedure, however, this method should first be approved by the Consultant.

6.2.5 Windrows should generally be parallel to each other and may be placed either parallel to or perpendicular to the face of the slope depending on the site geometry. The minimum horizontal spacing for windrows shall be 12 feet center-to-center with a 5-foot stagger or offset from lower courses to next overlying course. The minimum vertical spacing between windrow courses shall be 2 feet from the top of a lower windrow to the bottom of the next higher windrow.

6.2.6 Rock placement, fill placement and flooding of approved granular soil in the windrows should be continuously observed by the Consultant.

6.3 *Rock* fills, as defined in Section 3.1.3, shall be placed by the Contractor in accordance with the following recommendations:

6.3.1 The base of the *rock* fill shall be placed on a sloping surface (minimum slope of 2 percent). The surface shall slope toward suitable subdrainage outlet facilities. The *rock* fills shall be provided with subdrains during construction so that a hydrostatic pressure buildup does not develop. The subdrains shall be permanently connected to controlled drainage facilities to control post-construction infiltration of water.

6.3.2 *Rock* fills shall be placed in lifts not exceeding 3 feet. Placement shall be by rock trucks traversing previously placed lifts and dumping at the edge of the currently placed lift. Spreading of the *rock* fill shall be by dozer to facilitate *seating* of the rock. The *rock* fill shall be watered heavily during placement. Watering shall consist of water trucks traversing in front of the current rock lift face and spraying water continuously during rock placement. Compaction equipment with compactive energy comparable to or greater than that of a 20-ton steel vibratory roller or other compaction equipment providing suitable energy to achieve the required compaction or deflection as recommended in Paragraph 6.3.3 shall be utilized. The number of passes to be made should be determined as described in Paragraph 6.3.3. Once a *rock* fill lift has been covered with *soil* fill, no additional *rock* fill lifts will be permitted over the *soil* fill.

6.3.3 Plate bearing tests, in accordance with ASTM D 1196, may be performed in both the compacted *soil* fill and in the *rock* fill to aid in determining the required minimum number of passes of the compaction equipment. If performed, a minimum of three plate bearing tests should be performed in the properly compacted *soil* fill (minimum relative compaction of 90 percent). Plate bearing tests shall then be performed on areas of *rock* fill having two passes, four passes and six passes of the compaction equipment, respectively. The number of passes required for the *rock* fill shall be determined by comparing the results of the plate bearing tests for the *soil* fill and the *rock* fill and by evaluating the deflection variation with number of passes. The required number of passes of the compaction equipment will be performed as necessary until the plate bearing deflections are equal to or less than that determined for the properly compacted *soil* fill. In no case will the required number of passes be less than two.

6.3.4 A representative of the Consultant should be present during *rock* fill operations to observe that the minimum number of “passes” have been obtained, that water is being properly applied and that specified procedures are being followed. The actual number of plate bearing tests will be determined by the Consultant during grading.

6.3.5 Test pits shall be excavated by the Contractor so that the Consultant can state that, in their opinion, sufficient water is present and that voids between large rocks are properly filled with smaller rock material. In-place density testing will not be required in the *rock* fills.

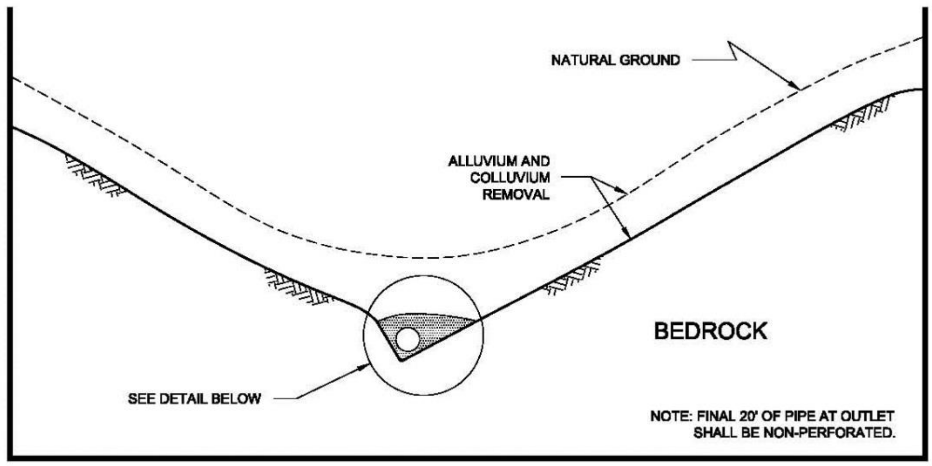
6.3.6 To reduce the potential for “piping” of fines into the *rock* fill from overlying *soil* fill material, a 2-foot layer of graded filter material shall be placed above the uppermost lift of *rock* fill. The need to place graded filter material below the *rock* should be determined by the Consultant prior to commencing grading. The gradation of the graded filter material will be determined at the time the *rock* fill is being excavated. Materials typical of the *rock* fill should be submitted to the Consultant in a timely manner, to allow design of the graded filter prior to the commencement of *rock* fill placement.

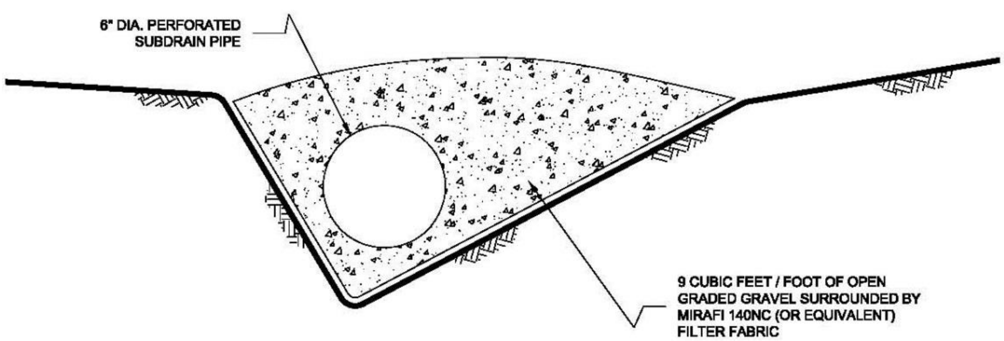
6.3.7 *Rock* fill placement should be continuously observed during placement by the Consultant.

7. SUBDRAINS

7.1 The geologic units on the site may have permeability characteristics and/or fracture systems that could be susceptible under certain conditions to seepage. The use of canyon subdrains may be necessary to mitigate the potential for adverse impacts associated with seepage conditions. Canyon subdrains with lengths in excess of 500 feet or extensions of existing offsite subdrains should use 8-inch-diameter pipes. Canyon subdrains less than 500 feet in length should use 6-inch-diameter pipes.

TYPICAL CANYON DRAIN DETAIL





NOTES:

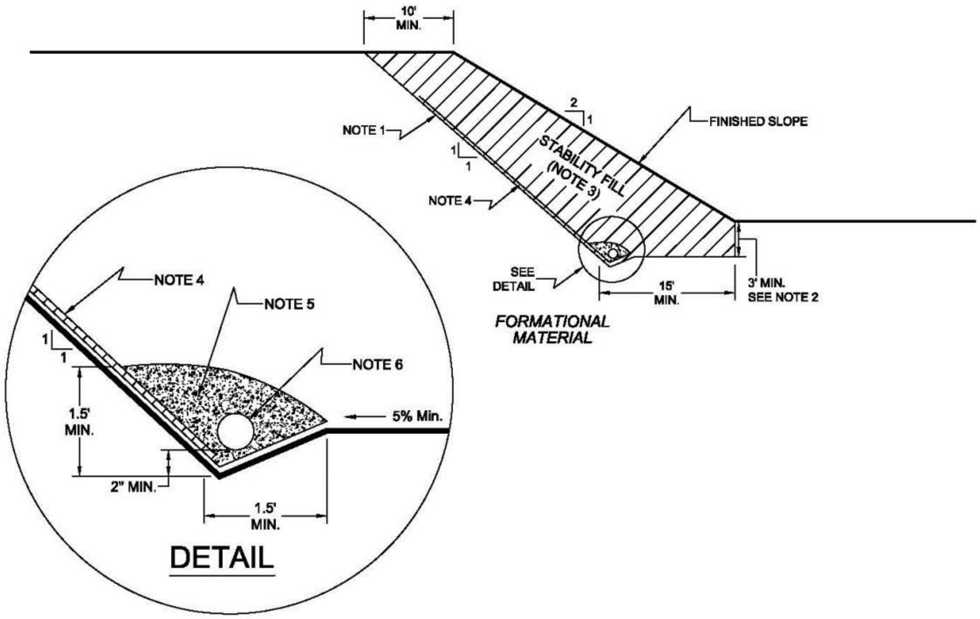
1......6-INCH DIAMETER, SCHEDULE 80 PVC PERFORATED PIPE FOR FILLS IN EXCESS OF 100-FEET IN DEPTH OR A PIPE LENGTH OF LONGER THAN 500 FEET.

2......6-INCH DIAMETER, SCHEDULE 40 PVC PERFORATED PIPE FOR FILLS LESS THAN 100-FEET IN DEPTH OR A PIPE LENGTH SHORTER THAN 500 FEET.

NO SCALE

7.2 Slope drains within stability fill keyways should use 4-inch-diameter (or lager) pipes.

TYPICAL STABILITY FILL DETAIL



NOTES:

1.....EXCAVATE BACKCUT AT 1:1 INCLINATION (UNLESS OTHERWISE NOTED).

2.....BASE OF STABILITY FILL TO BE 3 FEET INTO FORMATIONAL MATERIAL, SLOPING A MINIMUM 5% INTO SLOPE.

3.....STABILITY FILL TO BE COMPOSED OF PROPERLY COMPACTED GRANULAR SOIL.

4.....CHIMNEY DRAINS TO BE APPROVED PREFABRICATED CHIMNEY DRAIN PANELS (MIRADRAIN G200N OR EQUIVALENT) SPACED APPROXIMATELY 20 FEET CENTER TO CENTER AND 4 FEET WIDE. CLOSER SPACING MAY BE REQUIRED IF SEEPAGE IS ENCOUNTERED.

5.....FILTER MATERIAL TO BE 3/4-INCH, OPEN-GRADED CRUSHED ROCK ENCLOSED IN APPROVED FILTER FABRIC (MIRAFI 140NC).

6.....COLLECTOR PIPE TO BE 4-INCH MINIMUM DIAMETER, PERFORATED, THICK-WALLED PVC SCHEDULE 40 OR EQUIVALENT, AND SLOPED TO DRAIN AT 1 PERCENT MINIMUM TO APPROVED OUTLET.

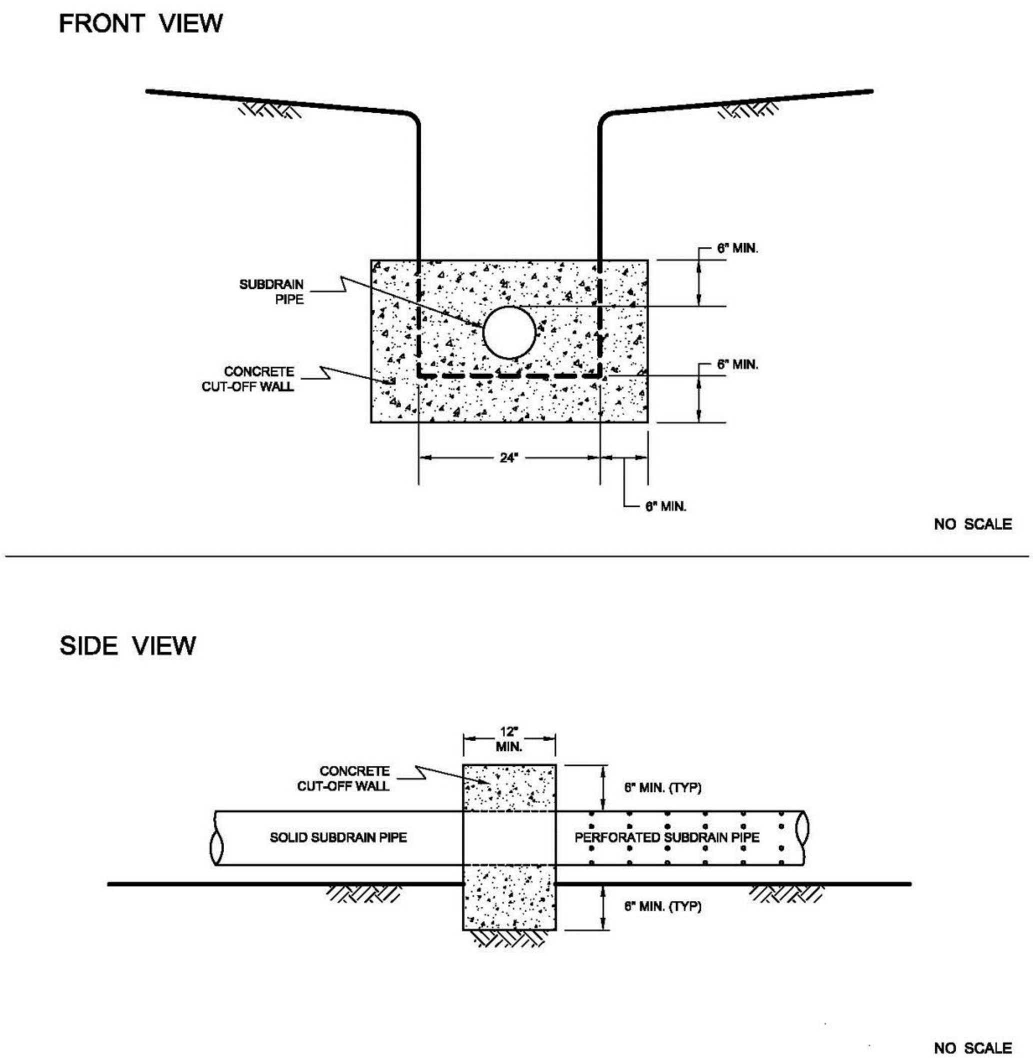
NO SCALE

7.3 The actual subdrain locations will be evaluated in the field during the remedial grading operations. Additional drains may be necessary depending on the conditions observed and the requirements of the local regulatory agencies. Appropriate subdrain outlets should be evaluated prior to finalizing 40-scale grading plans.

7.4 *Rock* fill or *soil-rock* fill areas may require subdrains along their down-slope perimeters to mitigate the potential for buildup of water from construction or landscape irrigation. The subdrains should be at least 6-inch-diameter pipes encapsulated in gravel and filter fabric. *Rock* fill drains should be constructed using the same requirements as canyon subdrains.

7.5 Prior to outletting, the final 20-foot segment of a subdrain that will not be extended during future development should consist of non-perforated drainpipe. At the non-perforated/ perforated interface, a seepage cutoff wall should be constructed on the downslope side of the pipe.

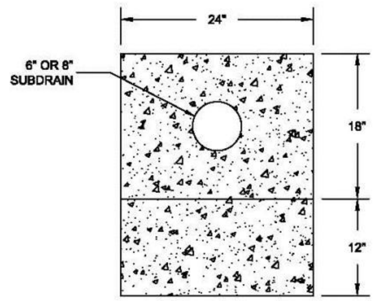
TYPICAL CUT OFF WALL DETAIL



7.6 Subdrains that discharge into a natural drainage course or open space area should be provided with a permanent headwall structure.

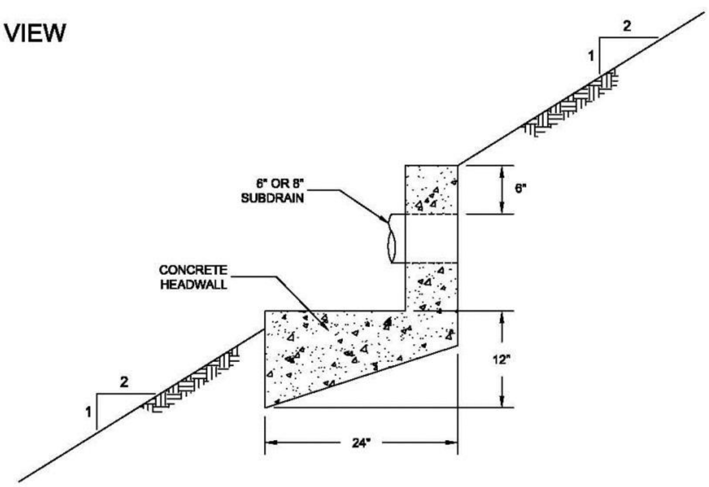
TYPICAL HEADWALL DETAIL

FRONT VIEW



NO SCALE

SIDE VIEW



NOTE: HEADWALL SHOULD OUTLET AT TOE OF FILL SLOPE
OR INTO CONTROLLED SURFACE DRAINAGE

NO SCALE

7.7 The final grading plans should show the location of the proposed subdrains. After completion of remedial excavations and subdrain installation, the project civil engineer should survey the drain locations and prepare an “as-built” map showing the drain locations. The final outlet and connection locations should be determined during grading operations. Subdrains that will be extended on adjacent projects after grading can be placed on formational material and a vertical riser should be placed at the end of the subdrain. The grading contractor should consider videoing the subdrains shortly after burial to check proper installation and functionality. The contractor is responsible for the performance of the drains.

8. OBSERVATION AND TESTING

8.1 The Consultant shall be the Owner's representative to observe and perform tests during clearing, grubbing, filling, and compaction operations. In general, no more than 2 feet in vertical elevation of *soil* or *soil-rock* fill should be placed without at least one field density test being performed within that interval. In addition, a minimum of one field density test should be performed for every 2,000 cubic yards of *soil* or *soil-rock* fill placed and compacted.

8.2 The Consultant should perform a sufficient distribution of field density tests of the compacted *soil* or *soil-rock* fill to provide a basis for expressing an opinion whether the fill material is compacted as specified. Density tests shall be performed in the compacted materials below any disturbed surface. When these tests indicate that the density of any layer of fill or portion thereof is below that specified, the particular layer or areas represented by the test shall be reworked until the specified density has been achieved.

8.3 During placement of *rock* fill, the Consultant should observe that the minimum number of passes have been obtained per the criteria discussed in Section 6.3.3. The Consultant should request the excavation of observation pits and may perform plate bearing tests on the placed *rock* fills. The observation pits will be excavated to provide a basis for expressing an opinion as to whether the *rock* fill is properly seated and sufficient moisture has been applied to the material. When observations indicate that a layer of *rock* fill or any portion thereof is below that specified, the affected layer or area shall be reworked until the *rock* fill has been adequately seated and sufficient moisture applied.

8.4 A settlement monitoring program designed by the Consultant may be conducted in areas of *rock* fill placement. The specific design of the monitoring program shall be as recommended in the Conclusions and Recommendations section of the project Geotechnical Report or in the final report of testing and observation services performed during grading.

8.5 We should observe the placement of subdrains, to check that the drainage devices have been placed and constructed in substantial conformance with project specifications.

8.6 Testing procedures shall conform to the following Standards as appropriate:

8.6.1 Soil and Soil-Rock Fills:

8.6.1.1 Field Density Test, ASTM D 1556, *Density of Soil In-Place By the Sand-Cone Method.*

8.6.1.2 Field Density Test, Nuclear Method, ASTM D 6938, *Density of Soil and Soil-Aggregate In-Place by Nuclear Methods (Shallow Depth).*

8.6.1.3 Laboratory Compaction Test, ASTM D 1557, *Moisture-Density Relations of Soils and Soil-Aggregate Mixtures Using 10-Pound Hammer and 18-Inch Drop.*

8.6.1.4. Expansion Index Test, ASTM D 4829, *Expansion Index Test*.

9. PROTECTION OF WORK

9.1 During construction, the Contractor shall properly grade all excavated surfaces to provide positive drainage and prevent ponding of water. Drainage of surface water shall be controlled to avoid damage to adjoining properties or to finished work on the site. The Contractor shall take remedial measures to prevent erosion of freshly graded areas until such time as permanent drainage and erosion control features have been installed. Areas subjected to erosion or sedimentation shall be properly prepared in accordance with the Specifications prior to placing additional fill or structures.

9.2 After completion of grading as observed and tested by the Consultant, no further excavation or filling shall be conducted except in conjunction with the services of the Consultant.

10. CERTIFICATIONS AND FINAL REPORTS

10.1 Upon completion of the work, Contractor shall furnish Owner a certification by the Civil Engineer stating that the lots and/or building pads are graded to within 0.1 foot vertically of elevations shown on the grading plan and that all tops and toes of slopes are within 0.5 foot horizontally of the positions shown on the grading plans. After installation of a section of subdrain, the project Civil Engineer should survey its location and prepare an *as-built* plan of the subdrain location. The project Civil Engineer should verify the proper outlet for the subdrains and the Contractor should ensure that the drain system is free of obstructions.

10.2 The Owner is responsible for furnishing a final as-graded soil and geologic report satisfactory to the appropriate governing or accepting agencies. The as-graded report should be prepared and signed by a California licensed Civil Engineer experienced in geotechnical engineering and by a California Certified Engineering Geologist, indicating that the geotechnical aspects of the grading were performed in substantial conformance with the Specifications or approved changes to the Specifications.

LIST OF REFERENCES

1. County of San Diego (2010), *Multi-Jurisdictional Hazard Mitigation Plan, San Diego County, California*, dated August 2010;

2. FEMA (2012), *Flood Map Service Center*, FEMA website, https://msc.fema.gov/portal/home, flood map number 06073C2200G, effective May 16, 2012;

3. Jennings, C. W., 1994, California Division of Mines and Geology, *Fault Activity Map of California and Adjacent Areas*, California Geologic Data Map Series Map No. 6.

4. SEAOC (2019), *OSHPD Seismic Design Maps:* Structural Engineers Association of California website, http://seismicmaps.org/;

5. USGS (2019), *Quaternary Fault and Fold Database of the United States*: U.S. Geological Survey website, https://www.usgs.gov/natural-hazards/earthquake-hazards/faults;

6. Unpublished reports and maps on file with Geocon Incorporated.