

Appendix D:
Geology and Soils Supporting Information

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D-1: Preliminary Geotechnical Subsurface Evaluation

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July 30, 2025

Project No. 25046-01

Ms. Diane Scioli
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710 South Parker Street, Suite 5200
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Subject: Preliminary Geotechnical Subsurface Evaluation for Proposed Residential Development, Arena Site, Southeast of Santiago Canyon Road and Nicky Way, City of Orange, California

In accordance with your request, LGC Geotechnical, Inc. has performed a preliminary geotechnical subsurface evaluation for the proposed residential development, Arena Site, located southeast of Santiago Canyon Road and Nicky Way in the City of Orange, California. The purpose of our study was to evaluate the site geotechnical conditions in the context of the updated site plans and to provide appropriate preliminary geotechnical design parameters and recommendations for the proposed development.

Should you have any questions regarding this report, please do not hesitate to contact our office. We appreciate this opportunity to be of service.

Sincerely,

LGC Geotechnical, Inc.

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1.0 INTRODUCTION

LGC Geotechnical has performed a preliminary geotechnical subsurface evaluation for the proposed residential development Arena Site, located southeast of Santiago Canyon Road and Nicky Way in the City of Orange, California (Figure 1). This report summarizes our findings, conclusions, and preliminary geotechnical design recommendations relative to the proposed development.

It should be noted that LGC Geotechnical does not provide environmental consulting services and did not address the environmental conditions (if any) of the subject site.

1.1 Project Description

The approximately 7.4-acre site is bound to the north by Santiago Canyon Road, to the east by a school, to the south by residences, and to the west by North Nicky Way. The most current land use is a horse arena with associated infrastructure and includes a fruit stand and associated parking lot.

Based on the conceptual site plan (Hunsaker, 2025), proposed development is anticipated to include the construction of 30 lots for residential construction, interior streets and associated improvements, and perimeter slopes and retaining walls.

Proposed grades are expected to include design cuts and fills up to approximately 10 feet. Proposed cut and fill slopes are on the order of up to approximately 22 and 24 feet tall, respectively, with natural existing slopes up to approximately 30 feet tall. Proposed walls in design cut conditions are on the order of up to approximately 10 feet tall and proposed walls in design fill conditions are on the order of up to approximately 7 feet tall. The proposed building structures are anticipated to be light-weight at-grade structures with maximum column and wall loads of approximately 20 kips and 2 kips per linear foot, respectively. Please note no structural loads were provided to us at the time of this report.

The preliminary recommendations given in this report are based upon the provided preliminary grading information and estimated structural loading and layout information provided above. We understand that the project plans are currently being developed at this time; LGC Geotechnical should be provided with updated project plans and the actual structural loads when they become available, in order to either confirm or modify the recommendations provided herein.

1.2 Background

Review of historic aerial photographs suggests the following (Historic Aerials, 2025):

1946 Aerial Photo: As early as 1946, it appears the site consisted of a citrus orchard.

1963 Aerial Photo: By 1963, Santiago Canyon Road had been constructed north of the subject site.

1992 Aerial Photo: By 1992, the citrus orchard appears to no longer be operational but the site remains undeveloped.

1994 Aerial Photos: The fruit stand has been constructed in the northwestern portion of the site and it appears 2 horse training areas had been constructed in the eastern portion of the site.

2014 Aerial Photos: The horse training areas have been reconfigured to allow for 4 separate areas. Additionally, viewing structures and a building were added. The site has essentially remained unchanged since.

Ginter & Associates, Inc. performed a geotechnical investigation of the site in 2022 (Ginter, 2022). The field evaluation consisted the excavation of five (5) hollow-stem auger infiltration borings to a depth of approximately 5 feet below existing grade and six (6) exploratory test pits ranging in depth from approximately 3.5 to 8.5 feet below existing grade. Groundwater was not encountered to the maximum explored depth. Laboratory testing was not available for our review. The test pit logs by others are presented in Appendix B.

1.3 Subsurface Evaluation

LGC Geotechnical's limited subsurface evaluation consisted of the excavation, sampling, and logging of hollow-stem auger borings and exploratory test pits.

Three exploratory hollow-stem auger infiltration borings (I-1 through I-3) were excavated using a truck-mounted drill rig equipped with 8-inch-diameter hollow-stem augers to depths ranging from approximately 5 feet to 8 feet below existing grade. An LGC Geotechnical representative observed the drilling operations, logged the borings, and collected soil samples for laboratory testing. Driven soil samples were collected by means of the Modified California Drive (MCD) sampler. The MCD sampler (2.4-inch ID, 3.0-inch OD) was driven using a 140-pound automatic hammer falling 30 inches to advance the sampler a total depth of 18 inches or until refusal. Bulk samples were also collected and logged for laboratory testing at select depths. The raw blow counts for each 6-inch increment of penetration were recorded on the boring logs. The borings were backfilled with cuttings. At the completion of infiltration testing, the perforated pipe was removed, and the resulting void backfilled with the previously excavated soils.

Nine exploratory geotechnical test pits (TP-1 through TP-9) were excavated utilizing a standard backhoe in order to estimate remedial grading depths and obtain samples for laboratory testing. A geologist observed the operation, logged the geotechnical test pits and collected soil samples. The exploratory test pits were subsequently backfilled with tamped native soils.

The approximate locations of our borings and test pits are provided on the Geotechnical Map (Sheet 1). Boring and geotechnical test pit logs are provided in Appendix B.

1.4 Laboratory Testing

Representative bulk and driven (relatively undisturbed) samples were obtained for laboratory testing during our field evaluation. Laboratory testing included in-situ moisture content and in-situ dry density, fines content, Atterberg limits, expansion index, direct shear, laboratory compaction and corrosion (sulfate, chloride, pH and minimum resistivity).

The following is a summary of the laboratory test results:

- Dry density of the samples collected ranged from approximately 97 pounds per cubic foot (pcf) to 106 pcf, with an average of 101 pcf. Field moisture contents ranged from approximately 4 to 10 percent, with an average of 8 percent.
- Three samples were tested for fines content indicating a fines content (passing No. 200 sieve) ranging from 54 to 62 percent. According to the Unified Soils Classification System (USCS), the tested samples are classified as “fine grained” soil.
- One Atterberg Limit (liquid limit and plastic limit) test was performed. Results indicated a Plasticity Index (PI) value of 7. The plot is provided in Appendix C.
- Expansion potential testing indicated an expansion index value ranging from 5 to 32, corresponding to “Very Low to Low” expansion potential.
- Two direct shear tests were performed. The shear stress versus normal stress plots are provided in Appendix C.
- One laboratory compaction test of near surface samples indicated a maximum dry density of 140.5 pcf with an optimum moisture content of 7.0 percent.
- Corrosion testing indicated a soluble sulfate content of approximately 0.005 percent, a chloride content of 26 parts per million (ppm), pH of 8.2, and a minimum resistivity of 2,546 ohm-centimeters.

A summary of the laboratory test results is presented in Appendix C. The moisture and dry density results are presented on the boring logs in Appendix B.

1.5 Field Infiltration Testing

Three falling head field infiltration tests (I-1 through I-3) were performed to an approximate depth ranging from 5 to 8 feet below existing grade. Estimation of infiltration rates for the site was accomplished in general accordance with the guidelines set forth by Orange County (2013). A 3-inch diameter perforated PVC pipe with filter sock was placed in the borehole, and the annulus was backfilled with gravel, including placement of approximately 2 inches of gravel at the bottom of the borehole. The infiltration wells were pre-soaked the day prior to testing. During the pre-test, if the water level drops more than 6 inches in 25 minutes for two consecutive readings, the test procedure for coarse-grained soils was followed. If the water level did not drop 6 inches in one or both pre-test readings, the procedure for fine-grained soils was followed. The procedure for coarse-grained soils requires performing the test for one hour and taking one reading every 10 minutes from a fixed reference point. The procedure for fine-grained soils requires performing the test for six hours and taking one reading every 30 minutes from a fixed reference point.

The pre-tests indicated the procedure for the fine-grained soils should be followed for two of the tests and coarse-grained for one of the tests. The calculated infiltration is normalized relative to the three-dimensional flow that occurs within the field test to a one-dimensional flow out of the bottom of the boring only (i.e., "Porchet Method"). The measured infiltration rates (for feasibility purposes only) are provided in Table 1 below. Please note that infiltration is discussed in Section 4.11 and field data is provided in Appendix D.

TABLE 1

Summary of Infiltration Testing

Infiltration Test Location	Infiltration Test Depth Below Existing Grade (ft)	Observed Infiltration Rate* (Inch/Hr.)
I-1	7	1.5
I-2	8	0.3
I-3	5	1.5

*Normalized to One-Dimensional Flow, does not include any Factor of Safety

It should be emphasized that infiltration test results are only representative of the location and depth where they are performed. Varying subsurface conditions may exist outside of the test locations which could alter the calculated infiltration rates indicated above. Infiltration tests are performed using relatively clean water free of particulates, silt, etc. Refer to Section 4.11 for subsurface water infiltration recommendations.



FIGURE 1
Site Location Map

PROJECT NAME	Milan Capital - Arena Site, Orange
PROJECT NO.	25046-01
ENG. / GEOL.	BPP / BPG
SCALE	Not to Scale
DATE	July 2025

2.0 GEOTECHNICAL CONDITIONS

2.1 Regional Geology

The subject site is generally located within the Peninsular Ranges Geomorphic Province of California, more specifically within the broad valley that is part of the Santiago Creek watershed coming out of the Peralta Hills located northeast of the site. The Peralta Hills are the western foothills of the Santa Ana Mountain range, bounded by the local El Modeno Fault. Review of published regional geologic maps indicated that a strand of the El Modeno Fault passes at depth below the subject site (USGS, 2025), however, the fault is known to be an ancient, or “pre-Holocene” fault (i.e., not a potential surface rupture hazard).

The site is located about a thousand feet south of the existing Santiago Creek; the creek then joins the Santa Ana River several miles downstream. The overall Santiago Creek floodplain is generally flat to gently sloped to the southwest, and consists of very old, non-marine sediments dissected by younger drainages and overlain by young fan deposits (CDMG, 2001).

2.2 Site-Specific Geology

Based on review of available geologic maps, the primary geologic unit underlying the site is middle-Pleistocene age, older alluvial deposits with isolated areas of younger alluvial deposits (not differentiated). The material is generally derived from Santiago Creek floodplain deposits and is alternately described as old terrace deposits (USGS, 2004). The material as encountered is brown to reddish brown, sandy clay to sandy silt to silty sands with gravels, cobbles and scattered boulders, slightly moist to moist, dense/stiff to very dense/hard. Difficult drilling conditions and auger refusal were encountered during drilling due to high rock content, and practical refusal was encountered during several test pit excavations using a standard backhoe. Minor amounts of artificial fill mantle the older alluvial deposits across the majority of the site, that range in depth from approximately 1 foot to 5.5 feet below existing grade. The undocumented artificial fill was observed to be silty to clayey sand with gravels. Refer to boring and test pit logs, Appendix B.

It should be noted that borings and test pits are only representative of the location and time where/when they are performed and varying subsurface conditions may exist outside of the performed location. In addition, subsurface conditions can change over time. The soil descriptions provided above should not be construed to mean that the subsurface profile is uniform, and that soil is homogeneous within the project area. For details on the stratigraphy at the exploration locations, refer to Appendix B.

2.3 Landslides

No landslides were observed within the site and a review of regional geologic maps of the area does not indicate the presence of known or suspected landslides in the direct vicinity of the site. Obvious indications of deep-seated landslides were not observed during our site evaluation.

2.4 Groundwater

Groundwater was not encountered to the maximum explored depth of approximately 9 feet. Groundwater is not expected to be a major constraint during construction activities.

Groundwater and/or groundwater seepage conditions may occur in the future due to changes in land use and/or following periods of heavy rain. Seasonal fluctuations of groundwater elevations should be expected over time. In general, groundwater levels fluctuate with the seasons and local zones of perched groundwater may be present within the near-surface materials due to local landscape and agricultural irrigation or precipitation.

2.5 Seismicity and Faulting

Prompted by damaging earthquakes in Northern and Southern California, State legislation and policies concerning the classification and land-use criteria associated with faults have been developed. The Alquist-Priolo Earthquake Fault Zoning Act was implemented in 1972 to prevent the construction of urban developments across the trace of active faults. California Geologic Survey Special Publication 42 was created to provide guidance for following and implementing the law requirements, revised in 2018 (CGS, 2018a). According to the State Geologist, a “Holocene-active” fault is defined as one which has had surface displacement within Holocene time (roughly the last 11,700 years). Regulatory Earthquake Fault Zones have been delineated to encompass traces of known Holocene-active faults to address hazards associated with surface fault rupture within California. Where developments for human occupation are proposed within these zones, the state requires detailed fault evaluations be performed so that engineering-geologists can identify the locations of active faults and recommend setbacks from locations of possible surface fault rupture.

No indication of Holocene-active faulting was observed during our evaluation. The site is not located within a mapped State of California Earthquake Fault-Rupture Hazard Zone per the CGS (2025). The El Modeno Fault is an ancient, “pre-Holocene” fault that is inferred to be present below the site, precise location not known as it is buried by older alluvium. The possibility of damage due to ground rupture is considered low since no Holocene-active faults are known to cross the site.

Secondary effects of seismic shaking resulting from large earthquakes on the major faults in the Southern California region, which may affect the site, include ground lurching, and shallow ground rupture, soil liquefaction, and dynamic settlement. These secondary effects of seismic shaking are a possibility throughout the Southern California region and are dependent on the distance between the site and causative fault and the onsite geology. Nearby faults that could produce these secondary effects include the Whittier-Elsinore, San Andreas, Puente Hills, Chino, and Newport-Inglewood Fault Zones, among others. A discussion of these secondary effects and their potential impact on the site is provided in the following sections.

2.5.1 Liquefaction and Dynamic Settlement

Liquefaction is a seismic phenomenon in which loose, saturated, granular soils behave similarly to a fluid when subject to high-intensity ground shaking. Liquefaction occurs when three general conditions coexist: 1) shallow groundwater; 2) low density non-cohesive (granular) soils; and 3) high-intensity ground motion. Studies indicate that saturated, loose near-surface cohesionless soils exhibit the highest liquefaction potential, while dry, dense, cohesionless soils and cohesive soils exhibit low to negligible liquefaction potential. Effects of liquefaction on level ground include settlement, sand boils, and bearing capacity failures below structures. Dynamic settlement of dry loose sands can occur as the sand particles tend to settle and densify as a result of a seismic event.

Based on our review of the State of California Seismic Hazard Zone for liquefaction potential (CGS, 2025), the subject site is not located within a liquefaction hazard zone. Based on the lack of shallow groundwater and the site consisting of compacted fill overlying dense/very stiff native soils following grading, the potential for liquefaction induced settlement is considered very low.

2.5.2 Lateral Spreading

Lateral spreading is a type of liquefaction-induced ground failure associated with the lateral displacement of surficial blocks of sediment resulting from liquefaction in a subsurface layer. Once liquefaction transforms the subsurface layer into a fluid mass, gravity plus the earthquake inertial forces may cause the mass to move downslope towards a free face (such as a river channel or an embankment). Lateral spreading may cause large horizontal displacements and such movement typically damages pipelines, utilities, bridges, and structures.

Due to very low potential for liquefaction the potential for lateral spreading is also considered very low.

2.6 Seismic Design Parameters

The site seismic characteristics were evaluated per the guidelines set forth in Chapter 16, Section 1613 of the 2022 California Building Code (CBC) and applicable portions of ASCE 7-16 which has been adopted by the CBC. Please note that the following seismic parameters are only applicable for code-based acceleration response spectra and are not applicable for where site-specific ground motion procedures are required by ASCE 7-16. Representative site coordinates of latitude 33.813302 degrees north and longitude -117.785262 degrees west were utilized in our analyses. The maximum considered earthquake (MCE) spectral response accelerations (S_{MS} and S_{M1}) and adjusted design spectral response acceleration parameters (S_{DS} and S_{D1}) for Site Class C are provided in Table 2 on the following page. The structural designer should contact the geotechnical consultant if structural conditions (e.g., number of stories, seismically isolated structures, etc.) require site-specific ground motions.

TABLE 2
Seismic Design Parameters

Selected Parameters from 2022 CBC, Section 1613 - Earthquake Loads	Seismic Design Values	Notes/Exceptions
Distance to applicable faults classifies the site as a "Near-Fault" site.		Section 11.4.1 of ASCE 7
Site Class	C	Chapter 20 of ASCE 7
S _s (Risk-Targeted Spectral Acceleration for Short Periods)	1.501g	From SEAOC, 2025
S ₁ (Risk-Targeted Spectral Accelerations for 1-Second Periods)	0.533g	From SEAOC, 2025
F _a (per Table 1613.2.3(1))	1.2	For Simplified Design Procedure of Section 12.14 of ASCE 7, F _a shall be taken as 1.4 (Section 12.14.8.1)
F _v (per Table 1613.2.3(2))	1.467	-
S _{MS} for Site Class C [Note: S _{MS} = F _a S _s]	1.801g	-
S _{M1} for Site Class C [Note: S _{M1} = F _v S ₁]	0.782g	-
S _{DS} for Site Class C [Note: S _{DS} = (2/3) S _{MS}]	1.201g	-
S _{D1} for Site Class C [Note: S _{D1} = (2/3) S _{M1}]	0.521g	-
C _{RS} (Mapped Risk Coefficient at 0.2 sec)	0.926	ASCE 7 Chapter 22
C _{R1} (Mapped Risk Coefficient at 1 sec)	0.922	ASCE 7 Chapter 22

A deaggregation of the PGA based on a 2,475-year average return period (MCE) indicates that an earthquake magnitude of 6.8 at a distance of approximately 11.4 km from the site would contribute the most to this ground motion. A deaggregation of the PGA based on a 475-year average return period (Design Earthquake) indicates that an earthquake magnitude of 6.6 at a distance of approximately 16 km from the site would contribute the most to this ground motion (USGS, 2014).

Section 1803.5.12 of the 2022 CBC (per Section 11.8.3 of ASCE 7) states that the maximum considered earthquake geometric mean (MCE_G) Peak Ground Acceleration (PGA) should be used for liquefaction potential. The PGA_M for the site is equal to 0.749g (SEAOC, 2025). The design PGA is equal to 0.499g (2/3 of PGA_M).

2.7 Preliminary Soil Shear Strength Parameters

The soil shear strength parameters presented in Table 3 on the following page were utilized in

our preliminary slope stability analysis and are based on our recent laboratory testing and published shear strength information for the on-site soils. Laboratory test results are provided in Appendix C.

Table 3

Summary of Shear Strength Parameters for Slope Stability Analysis

Description	ϕ (Degrees)	Cohesion (psf)
Artificial Fill - Compacted	32	100
Older Alluvial Deposits	30	150

2.8 Preliminary Global Slope Stability Analysis

Global slope stability analysis was performed on Cross-Sections A through D. See Sheet 1 for cross-section locations. Preliminary slope stability analysis was performed using the computer program GSTABL7 with GEOSTASE, version 4.30.31 (Gregory Geotechnical Software, 2019). Potential rotational failure modes were analyzed using Bishop's Modified Method. Slope stability analysis was performed for static and pseudo-static (seismic) loading conditions. A minimum factor of safety of 1.5 and 1.1 is typically required for static and seismic loading conditions.

Based on the conceptual site plan (Hunsaker, 2025), slope stability analysis indicates global factors of safety greater than 1.5 and 1.1 for static and seismic loading conditions, respectively. Slope stability analysis is provided in Appendix E.

2.9 Oversized Material

Oversized material (material larger than 8 inches in maximum dimension) are very likely to be encountered during site grading (and utility construction) based on our subsurface evaluation. Recommendations are provided for appropriate handling of oversized materials in Appendix F. If feasible, crushing oversized materials onsite or exporting oversized materials may be considered. Incorporating oversized materials into "rock fills" (windrows, rock blankets or individual rock burial) is likely not feasible due to the limited depth of grading. Alternatively, it may be feasible to overexcavate the central "Open Space" area for oversize disposal. Special handling recommendations should be provided on a case-by-case basis, if necessary.

2.10 Expansion Potential

Based on the results of our laboratory testing, fills at the completion of grading are anticipated to have a "Very Low" to "Low" expansion potential. Final expansion potential of site soils should be determined at the completion of grading. Results of expansion testing at finish grades will be utilized to confirm final foundation design.

3.0 CONCLUSIONS

Based on the results of our subsurface evaluation and geotechnical review of the proposed plan, it is our opinion that the proposed grading of the site is considered feasible from a geotechnical standpoint, provided that the recommendations provided here and in future reports are incorporated during site grading and development. A summary of our geotechnical conclusions are as follows:

- The primary geologic unit underlying the site is middle Pleistocene age, older alluvial deposits with isolated areas of younger alluvial deposits (not differentiated). The material as encountered is brown to reddish brown, sandy clay to sandy silt to silty sands with gravels, cobbles and scattered boulders, slightly moist to moist, dense/stiff to very dense/hard. Minor amounts of artificial fill overlay the older alluvial deposits across the site that range in depth from approximately 1 foot to 5.5 feet below the majority of site existing grades.
- Groundwater was not encountered to the maximum explored depth of approximately 9 feet. Groundwater is not expected to be a major constraint during construction activities.
- The subject site is not located within a State of California Earthquake Fault Rupture Zone (aka, "AP Zone"). The main seismic hazard that may affect the site is ground shaking from one of the active regional faults. The subject site will likely experience strong seismic ground shaking during its design life.
- Based on the lack of groundwater and the site consisting of compacted fill overlying dense/very stiff native soils following grading, the potential for liquefaction induced settlement is considered very low.
- In general, excavation for grading and removals should be achievable with heavy duty construction equipment in good working order.
- Oversized materials (greater than 8 inches in maximum dimension) are very likely to be encountered during site grading. Recommendations are provided for appropriate handling of oversized materials in Appendix F.
- Based on the results of our preliminary laboratory testing, site soils are anticipated to have "Very Low" to "Low" expansion potential. Mitigation measures will be required for any planned foundations and site improvements such as concrete flatwork to minimize the impacts of expansive soils.
- Pre-soaking of the subgrade for building slabs and site flatwork is recommended due to site expansive soils. The duration of this process varies greatly based on the chosen method and is also dependent on factors such as soil type and weather conditions. Time duration for presoaking should be accounted for in the construction schedule. Typically, approximately one to two weeks is necessary. Additional time at the completion of presoaking may be necessary for the surface soils of the pad to dry back sufficiently to be capable of supporting trenching equipment.
- From a geotechnical perspective, the existing onsite soils are considered suitable material for use as general fill, provided that they are relatively free from oversized material (larger than 8 inches in maximum dimension), construction debris, and significant organic material. Moisture conditioning will be required to obtain the required compaction.
- Portions of the onsite soils are suitable for backfill of site retaining walls, but some soils may not be suitable based on their fines content, expansion potential and oversized nature (material greater

than 3 inches in maximum dimension). Therefore, stockpiling of on-site sandy material or import of sandy soils meeting project recommendations will be required.

4.0 RECOMMENDATIONS

The following recommendations are to be considered preliminary and should be confirmed upon completion of grading and earthwork operations. In addition, they should be considered minimal from a geotechnical viewpoint, as there may be more restrictive requirements from the architect, structural engineer, building codes, governing agencies, or the owner.

It should be noted that the following geotechnical recommendations are intended to provide sufficient information to develop the site in general accordance with the 2022 CBC requirements. With regard to the potential occurrence of potentially catastrophic geotechnical hazards such as fault rupture, earthquake-induced landslides, liquefaction, etc. the following geotechnical recommendations should provide adequate protection for the proposed development to the extent required to reduce seismic risk to an “acceptable level.” The “acceptable level” of risk is defined by the California Code of Regulations as “that level that provides reasonable protection of the public safety, though it does not necessarily ensure continued structural integrity and functionality of the project” [Section 3721(a)]. Therefore, repair and remedial work of the proposed improvements may be required after a significant seismic event. With regards to the potential for less significant geologic hazards to the proposed development, the recommendations contained herein are intended as a reasonable protection against the potential damaging effects of geotechnical phenomena such as expansive soils, fill settlement, groundwater seepage, etc. It should be understood, however, that although our recommendations are intended to maintain the structural integrity of the proposed development and structures given the site geotechnical conditions, they cannot preclude the potential for some cosmetic distress or nuisance issues to develop as a result of the site geotechnical conditions.

The geotechnical recommendations contained herein must be confirmed to be suitable or modified based on the actual as-graded condition.

4.1 Site Earthwork

We anticipate that earthwork at the site will consist of site preparation, removal of existing vegetation and manure, excavation of slope stabilization fills (keyways), remedial grading, retaining wall construction, and fill placement.

We recommend that earthwork onsite be performed in accordance with the following recommendations, future grading plan review report(s), the 2022 CBC, City of Orange grading requirements, and the General Earthwork and Grading Specifications for Rough Grading included in Appendix F. In case of conflict, the following recommendations shall supersede all previous recommendations and those included as part of Appendix F. The following recommendations should be considered preliminary and may be revised based upon review of the project plans and/or based on the actual conditions encountered during site grading and construction.

4.1.1 Site Preparation

Prior to grading of areas to receive structural fill or engineered improvements, the areas should be cleared of surface obstructions, debris, vegetation, and manure. The surface

obstructions, debris, and vegetation should be removed and properly disposed of off-site. Holes resulting from the removal of buried obstructions, which extend below proposed finish grades, should be replaced with suitable compacted fill material. Any abandoned sewer or storm drain lines should be completely removed and replaced with properly placed compacted fill. We recommend the trenches associated with demolition which extend below the remedial grading depth be backfilled and properly compacted prior to the demolition contractor leaving the site.

If cesspools or septic systems are encountered, they should be removed in their entirety. The resulting excavation should be backfilled with properly compacted fill soils. As an alternative, cesspools can be backfilled with lean sand-cement slurry. Any encountered wells should be properly abandoned in accordance with regulatory requirements. At the conclusion of the clearing operations, a representative of LGC Geotechnical should observe and accept the site prior to further grading.

4.1.2 Remedial Grading

In order to provide a relatively uniform bearing condition for the planned residential building structures and improvements, we recommend the site soils be temporarily removed and recompacted. Compressible near surface soils shall be removed to suitable, competent native materials prior to re-placement as compacted fill to design grades. Subsurface site soils should be temporarily removed and recompacted according to the criteria outlined below. Existing undocumented fill encountered during rough grading operations should be temporarily removed and recompacted for use as compacted fill

Buildings: Soils within the proposed residential pads should be temporarily removed and recompacted to a minimum depth of 5 to 7 feet below existing grade or 3 feet beneath the base of the foundations, whichever is deeper. The estimated minimum temporary removal depth below existing grade locations can be found in the Geotechnical Map (Sheet 1), along with the lateral limits.

Minor Site Structures: For minor site structures, such as free-standing, minor retaining walls, etc., temporary removal and recompaction should extend at least 3 feet beneath existing grade or 2 feet beneath the base of foundations, whichever is deeper. In general, the envelope for remedial grading should extend laterally a minimum distance of 3 feet beyond the edges of the proposed improvements mentioned above.

Non-Building Areas: Within non-building areas such as pavement, hardscape areas, and central "Open Space" the soils should be temporarily removed and replaced as properly compacted fill to a minimum depth of 2 feet below existing grade or 1-foot below the proposed finished subgrade, whichever is deeper. In general, the envelope for remedial grading should extend laterally a minimum distance of 2 feet beyond the edges of the proposed improvements mentioned above.

Local conditions may be encountered which could require additional remedial grading beyond the above-noted minimum to obtain an acceptable subgrade. The actual depths and lateral extents of remedial grading should be determined by the geotechnical

consultant based on the subsurface conditions encountered during grading. Remedial grading areas should be accurately staked/recorded in the field by the Project Surveyor.

4.1.3 Keyways

It is recommended that design slopes be constructed with a minimum 15-foot wide stabilization fill keyway. The keyways should extend a depth of approximately 5 feet at the toe of the proposed slope and be tilted back into the slope to produce a minimum 1-foot fall across the width of the keyway. Preliminary keyways dimensions are presented on the Geotechnical Map (Sheet 1). Installation of subdrains along backcuts of recommended stability fill keyways should also be anticipated for site grading. Refer to Appendix F for keyway grading details.

4.1.4 Geologic Mapping

Remedial grading bottoms, backcuts, keyway excavations and cut slopes must be geologically mapped by the geotechnical consultant during earthwork construction to confirm the anticipated conditions. If unanticipated adverse geologic features are exposed, additional analysis and/or remediation measures may be required. The grading contractor must trim the backcuts with a slope board to remove loose material to allow for confirmation geologic mapping. Updated and/or revised geotechnical recommendations may be required based on observed conditions.

4.1.5 Subgrade Preparation

Removal bottoms, over-excavation bottoms and areas to receive fill should be observed and accepted by LGC Geotechnical prior to subsequent fill placement. Areas approved for fill placement should be scarified to a minimum depth of 6 to 8 inches, brought to a near-optimum moisture condition, and re-compacted in place per the project requirements.

4.1.6 Temporary Excavations

Temporary excavations should be performed in accordance with project plans, specifications, and all Occupational Safety and Health Administration (OSHA) requirements. Excavations should be laid back or shored in accordance with OSHA requirements before personnel or equipment are allowed to enter. Based on our field investigation, the majority of site soils are anticipated to be OSHA Type "B" soils (refer to the attached boring logs). Sandy soils are present and should be considered susceptible to caving and raveling. Soil conditions should be regularly evaluated during construction to verify conditions are as anticipated. The contractor shall be responsible for providing the "competent person" required by OSHA standards to evaluate soil conditions. Close coordination with the geotechnical consultant should be maintained to facilitate construction while providing safe excavations. Excavation safety is the sole responsibility of the contractor.

Where proposed improvements will be adjacent to property lines, the potential for impacting existing offsite improvements may be reduced by performing “ABC” slot cuts while performing earthwork removal and recompaction. “ABC” slot cuts are defined as excavations perpendicular to sensitive property boundaries that are divided into multiple “slots” of equal width. If slots are labeled A, B, C, A, B, C, etc., then all “A” slots can be excavated at the same time but must be backfilled before all “B” slots can be excavated, etc. Any given slot should be backfilled immediately with properly compacted fill to finish grade prior to excavation of the adjacent two slots. Please note some sands susceptible to caving are present at the site. Recommendations for slot cut dimensions should be evaluated during grading. Protection of the existing offsite improvements during grading is the responsibility of the contractor.

Vehicular traffic, stockpiles, and equipment storage should be set back from the perimeter of excavations a minimum distance equivalent to a 1:1 (horizontal to vertical) projection from the bottom of the excavation or 5 feet, whichever is greater, unless the cut is shored and designed for applicable surcharge load. Once an excavation has been initiated, it should be backfilled as soon as practical. Prolonged exposure of temporary excavations may result in localized instability. Excavations should be planned so that they are not initiated without sufficient time to shore/fill them prior to weekends, holidays, or forecasted rain.

It should be noted that any excavation that extends below a 1:1 (horizontal to vertical) projection of an existing foundation will remove existing support of the structure foundation. If requested, temporary shoring parameters will be provided.

4.1.7 Material for Fill

From a geotechnical perspective, onsite soils are generally considered suitable for use as general compacted fill, provided they are relatively free of organic materials and construction debris. Any encountered oversized material (material larger than 8 inches in maximum dimension) must be appropriately handled as outlined in Appendix F. Refer to Section 4.1.8, Select Material for MSE Wall Backfill, for required fill materials with the Backfill Zone.

From a geotechnical viewpoint, any required import soils for general fill (i.e., non-retaining wall backfill) should consist of clean, relatively granular soils of “Very Low” to “Low” expansion potential (expansion index 50 or less based on American Society for Testing and Materials [ASTM] D 4829), and relatively free of organic materials, construction debris and any material greater than 3 inches in maximum dimension. Import for any required retaining wall backfill should meet the criteria outlined in the following paragraph. Source samples should be provided to the geotechnical consultant for laboratory testing a minimum of 4 working days prior to any planned importation.

Conventional (masonry) retaining wall backfill should consist of sandy soils with a maximum of 35 percent fines (passing the No. 200 sieve) per American Society for Testing and Materials (ASTM) Test Method D1140 (or ASTM D6913/D422) and a “Very Low” expansion potential (EI of 20 or less per ASTM D4829). Soils should also be

screened of organic materials, construction debris, and any material greater than 3 inches in maximum dimension. Portions of the onsite soils are suitable for backfill of site retaining walls, but some soils may not be suitable based on their fines content, expansion potential and oversized nature (material greater than 3 inches in maximum dimension). Therefore, stockpiling of on-site sandy material or import of sandy soils meeting project recommendations will be required. These preliminary findings regarding suitability for retaining walls will be confirmed during grading.

Aggregate base (crushed aggregate base or crushed miscellaneous base) should conform to the latest requirements of Section 200-2 of the Standard Specifications for Public Works Construction ("Greenbook") for untreated base materials (except processed miscellaneous base) or Caltrans Class 2 aggregate base.

4.1.8 Select Material for MSE Wall Backfill

Mechanically Stabilized Earth (MSE) walls will require select backfill materials to be placed within the geogrid "reinforced" zone per local stability design in accordance with the specifications by the MSE wall designer. Generally, select backfill for the reinforced zone should consist of sandy materials with a maximum of 35 percent passing the No. 200 sieve, Plasticity Index (PI) not exceeding 20 and materials less than 3 inches in maximum dimension. The MSE wall designer may have more stringent reinforced backfill specifications. Portions of the onsite soils are suitable for backfill of site MSE retaining walls, but some soils may not be suitable based on their fines content, expansion potential and oversized nature (material greater than 3 inches in maximum dimension). Therefore, stockpiling of on-site sandy material or import of sandy soils meeting project recommendations should be expected. Samples of MSE retaining wall backfill should be sampled prior to construction to confirm the findings of the evaluation.

A representative from LGC Geotechnical should observe, probe, and test the backfill to verify compliance with the project recommendations.

4.1.9 General Fill Placement and Compaction (Materials 8 inches or less)

Material to be placed as fill should be brought to above-optimum moisture content (generally near optimum to about 2 percent above optimum moisture content) and recompacted to at least 90 percent relative compaction (per American Society for Testing and Materials [ASTM] Test Method D1557). Moisture conditioning of site soils will be required in order to achieve adequate compaction. Drying and/or mixing the very moist soils will be required prior to reusing the materials in compacted fills. Soils are also present that will require additional moisture in order to achieve the required compaction.

The optimum lift thickness to produce a uniformly compacted fill will depend on the type and size of compaction equipment used. In general, fill should be placed in uniform lifts not exceeding 8 inches in compacted thickness. Each lift should be thoroughly compacted and accepted prior to subsequent lifts. Generally, placement and compaction of fill should be performed in accordance with local grading ordinances and under the observation and

testing performed by the geotechnical consultant. Any encountered oversized material as previously defined must be appropriately handled as recommended in Appendix F. Oversized materials greater than 8 inches in diameter should not be placed within 10 feet of finished grade or within 15 feet of any slope face.

Fill placed on any slopes greater than 5:1 (horizontal to vertical) should be properly keyed and benched into firm and competent soils as it is placed in lifts. During backfill of excavations, the fill should be properly benched into firm and competent soils of temporary backcut slopes as it is placed in lifts.

Fill slope faces should also be compacted to project requirements. This may require overbuilding of the slope face and trimming back to design grades. Placement of sand or gravel lacking cohesive soil for binder on the outer slope face should be avoided in order to reduce potential for surficial instability such as erosion rills. To improve surficial stability, vegetation specified by the landscape architect should be established on the slope face as soon as it is practical.

Aggregate base material (crushed aggregate base and crushed miscellaneous base) should be compacted to a minimum of 95 percent relative compaction at or slightly above optimum moisture content per ASTM D1557. Subgrade below aggregate base should be compacted to a minimum of 90 percent relative compaction at near-optimum moisture content (generally within optimum and 2 percent above optimum moisture content) per ASTM D1557.

If gap-graded $\frac{3}{4}$ -inch rock is used for backfill (around storm drain storage chambers, retaining wall backfill, etc.) it will require compaction. Rock shall be placed in thin lifts (typically not exceeding 6 inches) and mechanically compacted with observation by geotechnical consultant. Backfill rock shall meet the requirements of ASTM D2321. Gap-graded rock is required to be entirely wrapped in filter fabric (Mirafi 140N or approved alternative) to prevent the migration of fines into the rock backfill.

For MSE wall backfill, it is imperative that adequate compaction meeting project recommendations be obtained in the zone immediately behind the block units where compaction is typically achieved using hand equipment (e.g., whacker, etc.) in lieu of rubber-tired construction equipment.

4.1.10 Trench and Conventional Retaining Wall Backfill and Compaction

Bedding material used within the pipe zone should conform to the requirements of the current Greenbook and the pipe manufacturer. Where applicable, sand having a sand equivalent (SE) of 20 or greater (per Caltrans Test Method [CTM] 217) may be used to bed and shade the pipes within the bedding zone. Sand backfill should be densified by jetting or flooding and then tamped to ensure adequate compaction. Sand grains should be from a natural source with rounded shape. Manufactured sand from crushed rock or recycled material is not suitable for jetting/flooding as the grains are typically angular in shape and do not densify well enough with these methods. Manufactured sand can be used as shading material when mechanical compaction efforts are used. The onsite soils

may generally be considered suitable as trench backfill (zone defined as 12 inches above the pipe to subgrade), provided the soils are screened of rocks greater than 6 inches in maximum dimension, construction debris and organic material. Trench backfill should be compacted in uniform lifts (as outlined above in Section “Material for Fill”) by mechanical means to at least 90 percent relative compaction (per ASTM D1557). If gap-graded rock is used for trench backfill, refer to the above Section 4.1.9.

Conventional retaining wall backfill should consist of select sandy soils as outlined in preceding Section 4.1.7. The limits of select sandy backfill should extend at minimum ½ the height of the retaining wall or the width of the heel (if applicable), whichever is greater (Figure 2). Retaining wall backfill soils should be compacted in relatively uniform thin lifts to at least 90 percent relative compaction (per ASTM D1557). Jetting or flooding of retaining wall backfill materials should not be permitted.

In backfill areas where mechanical compaction of soil backfill is impractical due to space constraints, typically sand-cement slurry may be substituted for compacted backfill. The slurry should contain about one sack of cement per cubic yard. When set, such a mix typically has the consistency of compacted soil. Sand cement slurry placed near the surface within landscape areas should be evaluated for potential impacts on planned improvements.

A representative from LGC Geotechnical should observe, probe, and test the backfill to verify compliance with the project recommendations.

4.1.11 Shrinkage and Bulking

Volumetric changes in earth quantities will occur when excavated onsite earth materials are replaced as properly compacted fill. The following is an estimate of shrinkage and bulking factors for the various geologic units found onsite, refer to Table 4 below. These estimates are based on in-place densities of the various materials and on the estimated average degree of relative compaction achieved during grading.

TABLE 4

Estimated Shrink/Bulk

Soil Type	Allowance	Estimated Range
Topsoil/Undocumented Artificial Fill (upper 2 feet)	Shrinkage	10% to 20%
Older Alluvial Deposits	Shrinkage	0% to 5%
Export of Oversize Rock (greater than 8" diameter)	Shrinkage	100%

It should be stressed that these values are only estimates and that an actual shrinkage factor is extremely difficult to predetermine. The effective shrinkage of onsite soils will depend primarily on the type of compaction equipment and method of compaction used

onsite by the contractor. The above shrinkage and bulking estimates are intended as an aid for project engineers in determining preliminary earthwork quantities. However, these estimates should be used with some caution since they are not absolute values. Contingencies such as a balance pad should be made for balancing earthwork quantities based on actual shrinkage and subsidence that occurs during grading. Shrinkage and bulking are also expected to vary with variations in survey accuracy during rough grading.

4.2 Slope Stability

Preliminary global slope stability for the site has been evaluated utilizing four cross sections throughout the site, refer to Appendix E. Design slopes will require keyways at the toe of slope. Preliminary recommendations for construction of slopes are presented below.

4.2.1 Fill Slopes

Design fill slopes at the site are anticipated to be both grossly and surficially stable as designed provided they are constructed in accordance with the General Earthwork and Grading Specifications (Appendix F) and proper irrigation, landscaping and maintenance is implemented (refer to Section 4.2.4). Fill slopes should be constructed with a maximum slope ratio of 2:1 (horizontal to vertical). Slope faces should also be compacted to minimum project recommendations. This may require overbuilding of the slope face and trimming back to design grades. To improve surficial stability, vegetation specified by the landscape architect should be established on the slope face as soon as it is practical, refer to Section 4.2.4. Keyways are recommended at the base of the proposed fill slopes. Keyways should be constructed in accordance with the General Earthwork and Grading Specification (Appendix F).

4.2.2 Cut Slopes

Cut slopes should be constructed with a maximum slope ratio of 2:1 (horizontal to vertical). All cut slopes should be observed by the engineering geologist during grading. Should unfavorable conditions (e.g., seepage, out-of-slope jointing/foliations) be observed during grading, the cut slopes in question may require stabilization by means of compacted fill buttress/stabilization fills and the installation of a subdrain system.

Fill over cut slope conditions are anticipated, and should be constructed in accordance with the General Earthwork and Grading Specifications included in Appendix F. Preliminary recommendations for keyway design and construction are provided in Appendix F.

4.2.3 Existing Slopes

Existing natural slopes will remain after grading operations around the southeastern perimeter of the site. These natural slopes will be subject to natural phenomena such as

erosion, sloughing and surficial instabilities. It is impossible to predict where or when this may happen. Should erosion or slippage occur, it should be promptly repaired. Paramount in reducing the potential for either erosion or slippage is to properly maintain these slopes (refer to Section 4.2.4).

4.2.4 Slope Maintenance Guidelines

It is recommended that site slopes be planted with ground cover vegetation as soon as practical to protect against erosion by reducing runoff velocity. Deep-rooted vegetation that requires little water and is able to survive local climate conditions should also be established to protect against surficial slumping. Under no circumstances should slopes be allowed to be bare of vegetation. Landscape vegetation should not be “trimmed” to root structures leaving no protection of the slopes. Irrigation levels should be kept to the minimum level necessary to establish healthy plant growth. Slopes must not be overwatered. If automatic sprinklers are used, they should be adjusted during periods of rainfall. A landscape professional should be consulted for landscape recommendations.

A program for the elimination of burrowing animals in both native and graded slope areas should be established to protect slope stability by reducing the potential for surface water to penetrate into the slope. Continuous erosion control, rodent control, and maintenance are essential to the long-term stability of all slopes. Trenches excavated on a slope face for utility or irrigation lines and/or for any purpose must be properly backfilled and compacted to project recommendations (refer to Section 4.1.10) to the slope face. Observation/testing and acceptance by the geotechnical consultant during trench backfill are recommended. V-ditches should be inspected and cleared of loose soil and/or debris on a routine basis, especially prior to and during the rainy season.

4.3 Provisional Foundation Recommendations

Given that the expansion index and plasticity index values correspond to “Very Low” to “Low” level of expansion (EI of 50 or less per ASTM D4829), the foundation system shall be designed for effects of expansive soil. Provided that the remedial grading recommendations provided herein are implemented, the site may be considered suitable for the support of the proposed structures using a conventional or post-tension foundation system. Preliminary foundation recommendations are provided in the following sections. Recommended soil bearing and estimated settlement due to structural loads are provided in Section 4.4.

Please note that the following foundation recommendations are preliminary and must be confirmed by LGC Geotechnical at the completion of project plans (i.e., foundation, grading, and site layout plans) as well as completion of grading.

4.3.1 Provisional Conventional Foundation Design Parameters

Conventional foundations may be designed in accordance with Wire Reinforcement Institute (WRI) procedure for slab-on-ground foundations per Section 1808 of the 2022 CBC to resist expansive soils. The following preliminary soil parameters may be used:

- Effective Plasticity Index: 15
- Climatic Rating: $C_w = 15$
- Reinforcement: Per structural designer.
- Moisture condition subgrade soils to 100 % of optimum moisture content to a depth of 12 inches prior to trenching for footings.

4.3.2 Provisional Post-Tensioned Foundation Design Parameters

These parameters have been determined in general accordance with Post-Tensioning Institute (PTI) Standard Requirements for Design of Shallow Post-Tensioned Concrete Foundations on Expansive Soils, referenced in Chapter 18 of the 2022 CBC. In utilizing these parameters, the foundation engineer should design the foundation system in accordance with the allowable deflection criteria of applicable codes and the requirements of the structural designer/architect. Other types of stiff slabs may be used in place of the CBC post-tensioned slab design provided that, in the opinion of the foundation structural designer, the alternative type of slab is at least as stiff and strong as that designed by the CBC/PTI method.

Our design parameters are based on our experience with similar projects, preliminary laboratory testing results, and the anticipated nature of the soil (with respect to expansion potential). Please note that implementation of our recommendations will not eliminate foundation movement (and related distress) should the moisture content of the subgrade fluctuate. It is the intent of these recommendations to help maintain the integrity of the proposed structures and reduce (not eliminate) movement, based upon the anticipated site soil conditions.

TABLE 5**Geotechnical Parameters for Post-Tensioned Foundation Slab Design**

Parameter	PT Slab with Perimeter Footing	PT Mat with Thickened Edge
Thorntwaite Moisture Index	-20	-20
Constant Soil Suction	PF 3.9	PF 3.9
Center Lift		
Edge moisture variation distance, e_m	9.0 feet	9.0 feet
Center lift, y_m	0.25 inch	0.3 inch
Edge Lift		
Edge moisture variation distance, e_m	5.5 feet	5.5 feet
Edge lift, y_m	0.55 inch	0.66 inch
Modulus of Subgrade Reaction, k (assuming presoaking as indicated below)	200 pci	200 pci
Minimum perimeter footing/thickened edge embedment below finish grade	12 inches	6 inches
Perimeter foundation reinforcement	N/A ¹	N/A ¹
Minimum slab thickness	5 inches ¹	8 inches ¹
Presoak (moisture conditioning)	100% of Opt. 12 inches	100% of Opt. 12 inches
1. Recommendations for foundation reinforcement and slab thickness are ultimately the purview of the foundation engineer/structural engineer based upon geotechnical criteria and structural engineering considerations. 2. Recommendations for sand below slabs have traditionally been included with geotechnical foundation recommendations, although they are not the purview of the geotechnical consultant. The sand layer requirements are the purview of the foundation engineer/structural engineer and should be provided in accordance with ACI Publication 302 "Guide for Concrete Floor and Slab Construction". 3. Recommendations for vapor retarders below slabs are also the purview of the foundation engineer/structural engineer and should be provided in accordance with applicable code requirements.		

4.3.3 Foundation Subgrade Preparation and Maintenance

Moisture conditioning (presoaking) of the subgrade soils is recommended prior to trenching the foundation. The duration of this process varies greatly based on the chosen method and is also dependent on factors such as soil type and weather conditions. Time duration for presoaking from completion of rough grading to trenching of foundations should be accounted for in the construction schedule (typically 1 to 2 weeks). The recommendations specific to the anticipated site soil conditions, including recommended presoak, are presented in Section 4.3.2. The subgrade moisture condition of the building pad soils should be maintained at near-optimum moisture content up to the time of concrete placement.

The geotechnical parameters provided herein assume that if the areas adjacent to the foundation are planted and irrigated, these areas will be designed with proper drainage and adequately maintained so that ponding, which causes significant moisture changes below the foundation, does not occur. Our recommendations do not account for excessive irrigation and/or incorrect landscape design. Plants should only be provided with sufficient irrigation for life and not overwatered to saturate subgrade soils. Sunken planters placed adjacent to the foundation should either be designed with an efficient drainage system or liners to prevent moisture infiltration below the foundation. Some lifting of the perimeter foundation beam should be expected even with properly constructed planters.

In addition to the factors mentioned above, the owner should be made aware of the potential negative influences of trees and/or other large vegetation. Roots that extend near the vicinity of foundations can cause distress to foundations. The owner (and the owner's landscape architect) should not plant trees/large shrubs closer to the foundations than a distance equal to half the mature height of the tree or 20 feet, whichever is more conservative unless specifically provided with root barriers to prevent root growth below the foundation.

It is the owner's responsibility to perform periodic maintenance during hot and dry periods to ensure that adequate watering has been provided to keep soils from separating or pulling back from the foundation.

4.3.4 Slab Underlayment Guidelines

The following is for informational purposes only since slab underlayment (e.g., moisture retarder, sand or gravel layers for concrete curing and/or capillary break) is unrelated to the geotechnical performance of the foundation and thereby not the purview of the geotechnical consultant. Post-construction moisture migration should be expected below the foundation. The foundation engineer/architect should determine whether the use of a capillary break (sand or gravel layer), in conjunction with the vapor retarder, is necessary or required by code. Sand layer thickness and location (above and/or below vapor retarder) should also be determined by the foundation engineer/architect.

4.4 Soil Bearing and Lateral Resistance

Provided our earthwork recommendations are implemented, an allowable soil bearing pressure of 2,000 pounds per square foot (psf) may be used for the design of footings having a minimum width of 12 inches and minimum embedment of 12 inches below lowest adjacent ground surface. This value may be increased by 300 psf for each additional foot of embedment or 150 psf for each additional foot of foundation width to a maximum value of 3,000 psf. An allowable soil bearing pressure of 1,200 psf may be used for a mat slab a minimum of 6 inches below lowest adjacent grade. These allowable bearing pressures are applicable for level (ground slope equal to or flatter than 5H:1V) conditions only. Bearing values indicated above are for total dead loads and live loads. The above vertical bearing may be increased by one-third for short durations of loading which will include the effect of wind or seismic forces.

In utilizing the above-mentioned allowable bearing capacity and provided our earthwork recommendations are implemented, foundation settlement due to structural loads is anticipated to be 1-inch or less. Differential settlement may be taken as half of the total settlement (i.e., ½-inch over a horizontal span of 40 feet).

Resistance to lateral loads can be provided by friction acting at the base of foundations and by passive earth pressure. For concrete/soil frictional resistance, an allowable coefficient of friction of 0.3 may be assumed with dead-load forces. An allowable passive lateral earth pressure of 250 psf per foot of depth (or pcf) to a maximum of 2,500 psf may be used for lateral resistance. Allowable passive pressure may be increased to 340 pcf to a maximum of 3,400 psf for short duration seismic loading. These passive pressures are applicable for level (ground slope equal to or flatter than 5H:1V) conditions only. For a 2:1 (horizontal to vertical) downward sloping condition, a reduced passive lateral earth pressure of 100 pcf to a maximum of 1,000 psf may be used. This allowable passive pressure may be increased to 135 pcf to a maximum of 1,350 psf for short duration seismic loading. We recommend that the upper foot of passive resistance be neglected if finished grade will not be covered with concrete or asphalt. Frictional resistance and passive pressure may be used in combination without reduction. The provided allowable passive pressures are based on a factor of safety of 1.5 and 1.1 for static and seismic loading conditions, respectively. The structural designer should incorporate appropriate factors of safety and/or load factors in their design.

4.5 Foundation Setback from Top-of-Slope and Bottom-of-Slope

Foundations should have adequate setback from top and bottom of slopes in accordance with the City of Orange or the 2022 CBC, whichever is more conservative. Per the 2022 CBC, the minimum top-of-slope setback is $H/3$, with a maximum required setback of 40 feet, where H is the total height of the slope. As an alternative to moving the building footprint, setback requirements may be accomplished by deepened footings or deep foundations. The minimum bottom-of-slope setback is $H/2$, with a maximum required setback of 15 feet. When determining the height of the slope, any wall should be included in the total slope height. As an alternative to moving the building footprint, top-of-slope setback requirements may be accomplished by deep footings or deep foundations. Refer to Chapter 18 of the 2022 CBC for more information. It is the purview of the project civil engineer to implement the appropriate foundation setbacks.

4.6 Lateral Earth Pressures for Conventional Retaining Wall Design

The following lateral earth pressures are presented in Table 6 on the following page for approved granular soils a maximum of 35 percent fines (passing the No. 200 sieve per ASTM D1140) and Very Low expansion potential (EI of 20 or less per ASTM D4829). Retaining wall backfill should also be limited to fill material not exceeding 3 inches in greatest dimension. Portions of the onsite soils are suitable for backfill of site retaining walls, but some soils may not be suitable based on their fines content, expansion potential and oversized nature (material greater than 3 inches in maximum dimension). Therefore, stockpiling of on-site sandy material or import of sandy soils meeting project recommendations will be required. The retaining wall designer should clearly indicate on the retaining wall plans the required sandy backfill.

Lateral earth pressures are provided as equivalent fluid unit weights, in psf/ft of depth or pcf. These values do not contain an appreciable factor of safety, so the retaining wall designer should apply the applicable factors of safety and/or load factors during design. A soil unit weight of 120 pcf may be assumed for calculating the actual weight of soil over the wall footing.

TABLE 6

Lateral Earth Pressures – Select Onsite Sandy Soils or Import

Conditions	Equivalent Fluid Unit Weight (pcf)	
	Level Backfill	2:1 Backfill Sloping Upwards
	Approved Onsite Sandy Soils or Import	Approved Onsite Sandy Soils or Import
Active	35	55
At-Rest	55	70

If the wall can yield enough to mobilize the full shear strength of the soil, it can be designed for “active” pressure. If the wall cannot yield under the applied load, the earth pressure will be higher. The equivalent fluid pressure values assume free-draining conditions. Retaining wall structures should be provided with appropriate drainage and appropriately waterproofed (Refer to Figure 2). Please note that waterproofing and outlet systems are not the purview of the geotechnical consultant. If conditions other than those assumed above are anticipated, the equivalent fluid pressure values should be provided on an individual-case basis by the geotechnical consultant.

Retaining wall structures should be provided with appropriate drainage and appropriately waterproofed. To reduce, but not eliminate, saturation of near surface (upper approximate 1-foot) soils in front of the retaining walls, the perforated subdrain pipe should be located as low as possible behind the retaining wall. The outlet pipe should be sloped to drain to a suitable outlet.

Surcharge loading effects from any adjacent structures should be evaluated by the retaining wall designer. In general, structural loads within a 1:1 (horizontal to vertical) upward projection from the bottom of the proposed basement/retaining wall footing will surcharge the proposed retaining structure. In addition to the recommended earth pressure, retaining walls adjacent to streets should be designed to resist vehicular traffic if applicable. Uniform surcharges may be estimated using the applicable coefficient of lateral earth pressure using a rectangular distribution. A factor of 0.35 and 0.5 may be used for the active and at-rest conditions, respectively. The vertical traffic surcharge may be determined by the structural designer. The retaining/basement wall designer should contact the geotechnical engineer for any required geotechnical input in estimating any applicable surcharge loads.

If required, the retaining wall designer may use a seismic lateral earth pressure increment of 10 pcf for a level backfill condition and 25 pcf for a sloping condition (up to an inclination of a 2:1). This increment should be applied in addition to the provided static lateral earth pressure using a triangular distribution with the resultant acting at H/3 in relation to the base of the retaining structure (where H is the retained height). For the restrained, at-rest condition, the seismic increment may be added to the applicable active lateral earth pressure (in lieu of the at-rest lateral earth pressure) when analyzing short duration seismic loading. Per Section 1803.5.12 of the 2022 CBC, the seismic lateral earth pressure is applicable to structures assigned to Seismic Design Category D through F for retaining wall structures supporting more than 6 feet of backfill height. The provided seismic lateral earth pressure should not be used for retaining walls exceeding 10 feet in height. If a retaining wall greater than 10 feet in height is proposed, the retaining wall designer should contact the geotechnical consultant for specific seismic lateral earth pressure increments based on the configuration of the planned retaining wall structures.

Soil bearing and lateral resistance (friction coefficient and passive resistance) are provided in Section 4.4. Earthwork considerations (temporary backcuts, backfill, compaction, etc.) for retaining walls are provided in Section 4.1 (Site Earthwork) and the subsequent earthwork related sub-sections.

4.7 Preliminary MSE Walls Design Criteria

The following soil shear strength parameters may be used for local stability design of site MSE walls. A soil unit weight of 120 pcf may be used for reinforced, retained and foundation soils.

TABLE 7

Soil Shear Strength Parameters for MSE Wall Design

Soil Zone	ϕ (Degrees)	Cohesion (psf)
Foundation Soils	32	100
Reinforced Select Sandy Soils	30	0
Retained Soils	30	0

The above design soil shear strength parameters are contingent on the proper backfill material being used in the construction of the MSE walls, refer to Section 4.1.8. The MSE wall designer should also incorporate any applicable surcharge loading into the local stability design.

Heel drains are recommended for MSE walls constructed where the backcut adjacent to the end of the geogrid is native materials.

The geogrid areas behind the walls should be considered a restricted use area where no future excavations should be made into the geogrid zone, in order to protect the integrity of the wall. Damaging the geogrid reinforcement may result in negative consequences with respect to MSE wall stability and long-term durability. Project designers, construction contractors and future homeowners should be made aware of this recommended restricted use area.

LGC Geotechnical should provide global stability analysis for planned MSE walls once plans are

developed. Prior to construction, LGC Geotechnical should review proposed MSE wall plans prior to construction to verify that our geotechnical recommendations are implemented.

4.8 Soil Corrosivity to Concrete and Metal

Although not corrosion engineers (LGC Geotechnical is not a corrosion consultant), several governing agencies in Southern California require the geotechnical consultant to determine the corrosion potential of soils on buried concrete and metal facilities. We therefore present the results of our testing with regard to corrosion for the use of the client and other consultants, as they determine necessary.

Preliminary corrosion testing of a near-surface bulk sample indicated a soluble sulfate content less than approximately 0.005 percent, a chloride content of 26 parts per million (ppm), pH of 8.2, and a minimum resistivity of 2,546 ohm-centimeters. Based on Caltrans Corrosion Guidelines (Caltrans, 2021), soils are considered corrosive to structural elements if the pH is 5.5 or less, or the chloride concentration is 500 ppm or greater, or the sulfate concentration is 1,500 ppm (0.15 percent) or greater. Based on the preliminary test results, soils are not considered corrosive using Caltrans criteria. However, based on minimum resistivity the soils are considered moderately corrosive to metallic improvements. If improvements that may be susceptible to corrosion are proposed, it is recommended that further evaluation by a corrosion engineer be performed.

Based on preliminary laboratory sulfate test results, the near surface soils are designated to a class "S0" per ACI 318, Table 19.3.1.1 with respect to sulfates. Concrete in direct contact with the onsite soils can be designed according to ACI 318, Table 19.3.2.1 using the "S0" sulfate classification.

Laboratory testing may need to be performed at the completion of grading by the project corrosion engineer to further evaluate the as-graded soil corrosivity characteristics, including sulfate, chloride, pH, minimum resistivity, etc. and provide design level recommendations. Accordingly, revision of the corrosion potential may be needed, should future test results differ substantially from the conditions reported herein. The client and/or other members of the development team should consider this during the design and planning phase of the project and formulate an appropriate course of action.

4.9 Control of Surface Water and Drainage Control

From a geotechnical perspective, positive drainage of surface water away from structures is very important. Water should not be allowed to pond adjacent to buildings or to flow freely down a graded slope. Per section 1804.4 of the 2022 CBC, positive drainage may be accomplished by providing drainage away from buildings at a gradient of at least 5 percent for earthen surfaces for a distance of at least 10 feet away from the face of wall. Eave gutters are recommended and should reduce water infiltration into the subgrade soils if the downspouts are properly connected to appropriate outlets. Ultimately surface drainage and code compliance are the purview of the project civil engineer.

Planters with open bottoms adjacent to buildings should be avoided. Planters should not be designed adjacent to buildings unless provisions for drainage, such as catch basins, liners, and/or area drains, are made. Overwatering must be avoided.

Planters adjacent to a building or structure should be avoided wherever possible or be properly designed (e.g., lined with a membrane), to reduce the penetration of water into the adjacent footing subgrades and thereby reduce moisture related damage to the foundation. Planting areas at grade should be provided with appropriate positive drainage. Wherever possible, exposed soil areas should be above adjacent paved grades to facilitate drainage. Planters should not be depressed below adjacent paved grades unless provisions for drainage, such as multiple depressed area drains, are constructed. Adequate drainage gradients, devices, and curbing should be provided to prevent runoff from adjacent pavement or walks into the planting areas. Irrigation methods should promote uniformity of moisture in planters and beneath adjacent concrete flatwork. Overwatering and underwatering of landscape areas must be avoided.

Area drain inlets should be maintained and kept clear of debris in order to properly function. Homeowners should also be made aware that excessive irrigation of neighboring properties can cause seepage and moisture conditions on adjacent lots. Homeowners should be furnished with these recommendations communicating the importance of maintaining positive drainage away from structures, towards streets, when they design their improvements.

The impact of heavy irrigation or inadequate runoff gradients can create perched water conditions. This may result in seepage or shallow groundwater conditions where previously none existed. Maintaining adequate surface drainage and controlled irrigation will significantly reduce the potential for nuisance-type moisture problems. To reduce differential earth movements such as heaving and shrinkage due to the change in moisture content of foundation soils, which may cause distress to a residential structure and associated improvements, moisture content of the soils surrounding the structure should be kept as relatively constant as possible.

Subdrains and v-ditches must be properly maintained, and their outlets kept free draining and clear of any potential obstructions. Routine maintenance should be performed, especially prior to and during the rainy season. Failure to properly maintain these elements may result in slope failures, slumps, excessive erosion, localized saturated zones, nuisance type water issues, etc. Any future trenches excavated on a slope face for utility or irrigation lines and/or for any purpose should be properly backfilled and compacted to the slope face. Observation/testing and acceptance by the geotechnical consultant during trench backfill are recommended.

A program for the elimination of burrowing animals in both native and graded slope areas must be established and properly maintained to protect slope stability by reducing the potential for surface water to penetrate into the soil. Continuous erosion control, rodent control, and maintenance are essential to the long-term stability of all slopes.

4.10 Subsurface Water Infiltration

Recent regulatory changes have occurred that mandate that storm water be infiltrated below

grade into subsurface soils rather than collected in a conventional storm drain system. Typically, a combination of methods are implemented to reduce surface water runoff and increase infiltration including; permeable pavements/pavers for roadways and walkways, directing surface water runoff to grass-lined swales, retention areas, and/or drywells, etc.

It should be noted that collecting and concentrating surface water for the purpose of intentionally infiltrating below grade, conflicts with the geotechnical engineering objective of directing surface water away from slopes, structures and other improvements. The geotechnical stability and integrity of a site is reliant upon appropriately handling surface water. In general, the vast majority of geotechnical distress issues are directly related to improper drainage. In general, distress in the form of movement of improvements could occur as a result of soil saturation and loss of soil support, expansion, internal soil erosion, collapse and/or settlement.

The results of our field infiltration testing indicate the observed 1-D infiltration rate for I-1 through I-3 (not including required factors of safety for design) ranged from 0.3 to 1.5 inches per hour. The design infiltration rate is thereby equal to the Observed Infiltration Rate divided by the design factor of safety. The design factor of safety must be a minimum of 2.0 but may be increased at the discretion of the design engineer. Infiltration of stormwater is not required when the factored infiltration rate (measured infiltration rate with safety factors applied) is less than 0.3 inches per hour in the vicinity of the BMPs.

Based on the results of field infiltration testing indicating relatively low and variable infiltration rates, the very stiff/dense and fine-grained nature of the on-site soils, and hillside nature of the site, we recommend against the intentional infiltration of stormwater into the subsurface soils.

4.11 Preliminary Asphalt Concrete Pavement Sections

The following preliminary minimum asphalt concrete (AC) street sections are provided in Table 8 for Traffic Indices (TI) of 5.0, 5.5 and 6.5. These sections are based on an estimated R-value of 20. These recommendations must be confirmed with R-value testing of representative near-surface soils at the completion of grading and after underground utilities have been installed and backfilled. We are not responsible for selecting a design Traffic Index. Final pavement sections should be confirmed by the project civil engineer based upon the final design Traffic Index. We are not responsible for selecting a design Traffic Index. If requested, LGC Geotechnical will provide sections for alternate TI values. Should the City of Orange have more stringent requirements, updated pavement recommendations can be provided.

TABLE 8

Preliminary Asphalt Concrete Pavement Section Options

Assumed Traffic Index	5.0	5.5	6.5
R -Value Subgrade	20	20	20
AC Thickness	4.0 inches	4.0 inches	5.0 inches
Aggregate Base Thickness	5.5 inches	7.5 inches	9.0 inches

Due to anticipated heavy construction traffic prior to the completion of the project, we recommend that the total thickness (base course and capping course) of AC be placed at essentially the same time. Construction traffic loading on only the base course of the AC will increase the potential for pavement distress. It should be noted that construction traffic such as concrete trucks will likely exceed traffic loading after completion of construction. Additionally, earth moving equipment should not be allowed to drive on paved surfaces. The weights of these vehicles far exceed the limits of the pavement and will cause distress, likely requiring the pavement sections to be removed and replaced.

Increasing the thickness of asphalt or adding additional base material will reduce the likelihood of the pavement experiencing distress during its service life. The above recommendations are based on the assumption that proper maintenance and irrigation of the areas adjacent to the roadway will occur through the design life of the pavement. Failure to maintain a proper maintenance and/or irrigation program may jeopardize the integrity of the pavement.

Earthwork recommendations regarding aggregate base and subgrade are provided in the previous Section 4.1 (Site Earthwork) and the related sub-sections of this report.

Please note that as the geotechnical consultant of record we will only be performing temperature checks of the asphalt and testing the wet density for calculating relative compaction. We recommend obtaining the services of others if material testing and evaluation of the quality of workmanship by the contractor is desired or required by the regulatory agency.

4.12 Vehicular Concrete Pavers

A design Traffic Index (TI) of 5.0 has been assumed. R-Value testing will be performed on finished grade soils at the completion of subsurface utility construction, however, for these preliminary geotechnical recommendations an R-Value of 20 has been assumed.

Concrete pavers should be a minimum of 3 and 1/8 inches (80 mm) thick, rated for vehicular traffic and placed in a herringbone pattern. The manufacturer's specific recommendations regarding the pavers (required bedding and jointing sand, etc.) should be implemented during construction. It should be noted that pavers are typically underlain by 1 to 1½ inches of bedding sand followed by compacted aggregate base over compacted subgrade soils. The concrete pavers and sand should be underlain by 9.5 inches of aggregate base for a design TI of 5.0. Final paver sections should be confirmed by the project civil engineer or City Engineer based upon the final design Traffic Index. We are not responsible for selecting a design Traffic Index. If requested, LGC Geotechnical will provide sections for alternate TI values. Should the city of Orange have more stringent requirements, updated pavement recommendations can be provided.

Concrete bands around the perimeter of the pavers are recommended. The concrete bands should be at least 6 inches thick, with two No. 4 rebars placed longitudinally at approximately mid-height. The concrete should be underlain by a minimum of 4 inches of crushed base material.

The thicknesses above are minimum thicknesses. Increasing the thickness of any or all of the above layers will reduce the likelihood of the pavement experiencing distress during its service life. The above recommendations are based on the assumption that proper maintenance and irrigation of the areas adjacent to the roadway will occur through the design life of the pavement. Failure to maintain a proper maintenance and/or irrigation program may jeopardize the integrity of the pavement and pavers.

Earthwork recommendations regarding aggregate base and subgrade are provided in the previous Section 4.1 (Site Earthwork) and the related sub-sections of this report.

4.13 Nonstructural Concrete Flatwork

Nonstructural concrete flatwork (such as walkways, private drives, patio slabs, etc.) has a potential for cracking due to changes in soil volume related to soil-moisture fluctuations. To reduce the potential for excessive cracking and lifting, concrete may be designed in accordance with the minimum guidelines outlined in Table 9 below. These guidelines will reduce the potential for irregular cracking and promote cracking along construction joints, but will not eliminate all cracking or lifting. Thickening the concrete and/or adding additional reinforcement will further reduce cosmetic distress.

TABLE 9

**Minimal Guidelines for Nonstructural Concrete Flatwork for
Low Expansion Potential**

	Community Sidewalks (≤6 feet wide)	Patios/Entryways / Walkways (adjacent to homes or flatwork >6 feet wide)	Vehicular Driveways
Minimum Thickness (in.)	4 (nominal)	4 (full)	4 (full)
Presoak	Wet down prior to placing	Wet down prior to placing	Wet down prior to placing
Reinforcement	—	No. 3 at 24 inches on centers	No. 3 at 24 inches on centers
Thickened Edge (in.)	—	—	8 x 8
Crack Control Joints	Saw cut or deep open tool joint to a minimum of 1/3 the concrete thickness	Saw cut or deep open tool joint to a minimum of 1/3 the concrete thickness	Saw cut or deep open tool joint to a minimum of 1/3 the concrete thickness

Maximum Joint Spacing	5 feet	6 feet	10 feet or quarter cut whichever is closer
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To reduce the potential for driveways to separate from the garage slab, the builder may elect to install dowels to tie these two elements together. Similarly, future homeowners should consider the use of dowels to connect flatwork to the foundation.

4.14 Geotechnical Plan Review

Grading plans, retaining wall plans, foundation plans and final project drawings should be reviewed by this office prior to grading to verify that our geotechnical recommendations, provided herein, have been appropriately incorporated.

4.15 Geotechnical Observation and Testing During Construction

The recommendations provided in this report are based on limited subsurface observations and geotechnical analysis. The interpolated subsurface conditions should be checked in the field during grading by a representative of LGC Geotechnical. Geotechnical observation and testing is required per Section 1705 of the 2022 California Building Code (CBC).

Geotechnical observation and/or testing should be performed by LGC Geotechnical at the following stages:

- During grading (key excavations, removal bottoms, remedial grading, fill placement, etc.);
- During retaining wall backfill and compaction;
- During utility trench backfill and compaction;
- After presoaking building pads and other concrete-flatwork subgrades, and prior to placement of aggregate base or concrete;
- After building and wall footing excavation and prior to placing steel reinforcement and/or concrete;
- Preparation of pavement subgrade and placement of aggregate base; and
- When any unusual soil conditions are encountered during any construction operation subsequent to issuance of this report.

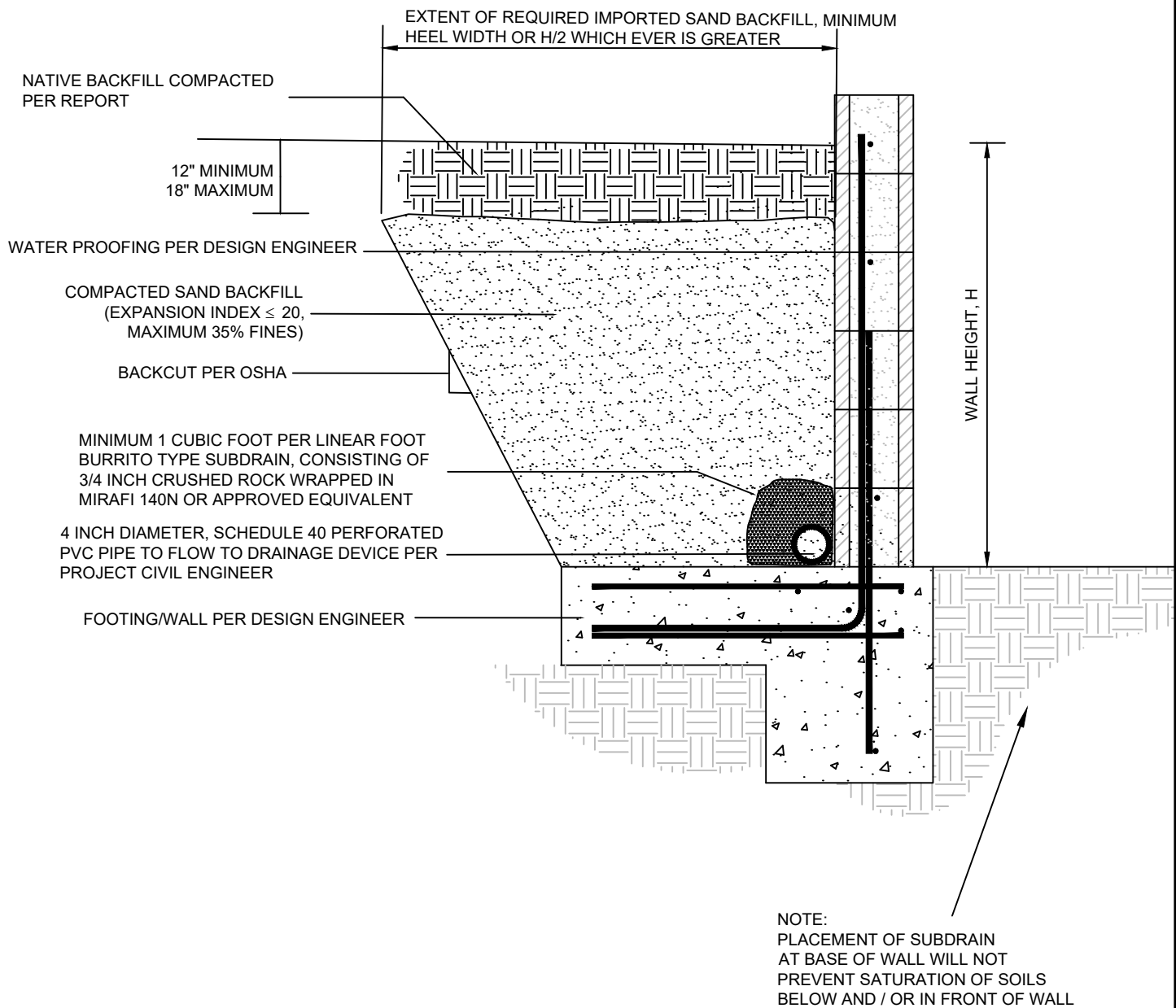
5.0 LIMITATIONS

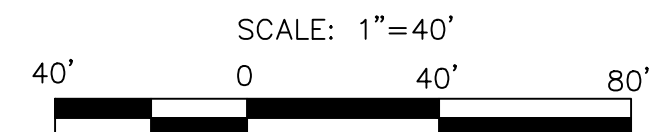
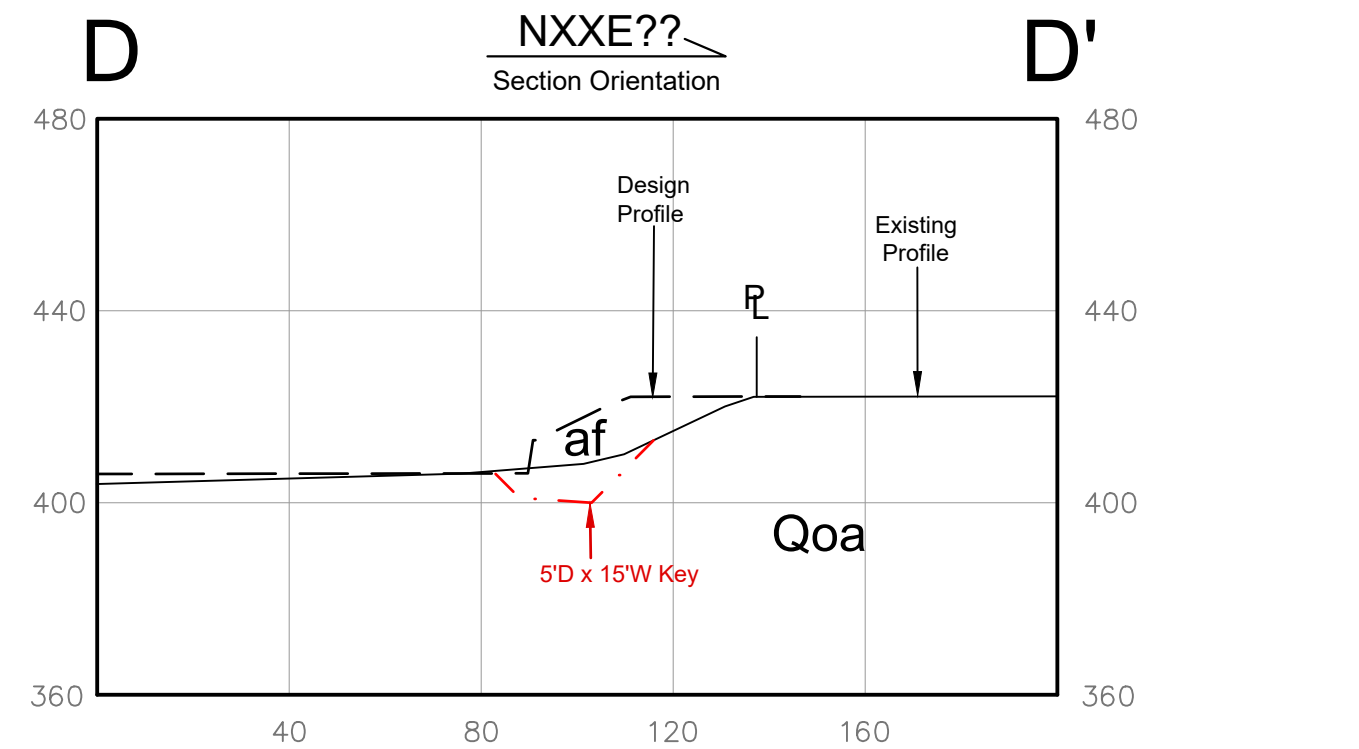
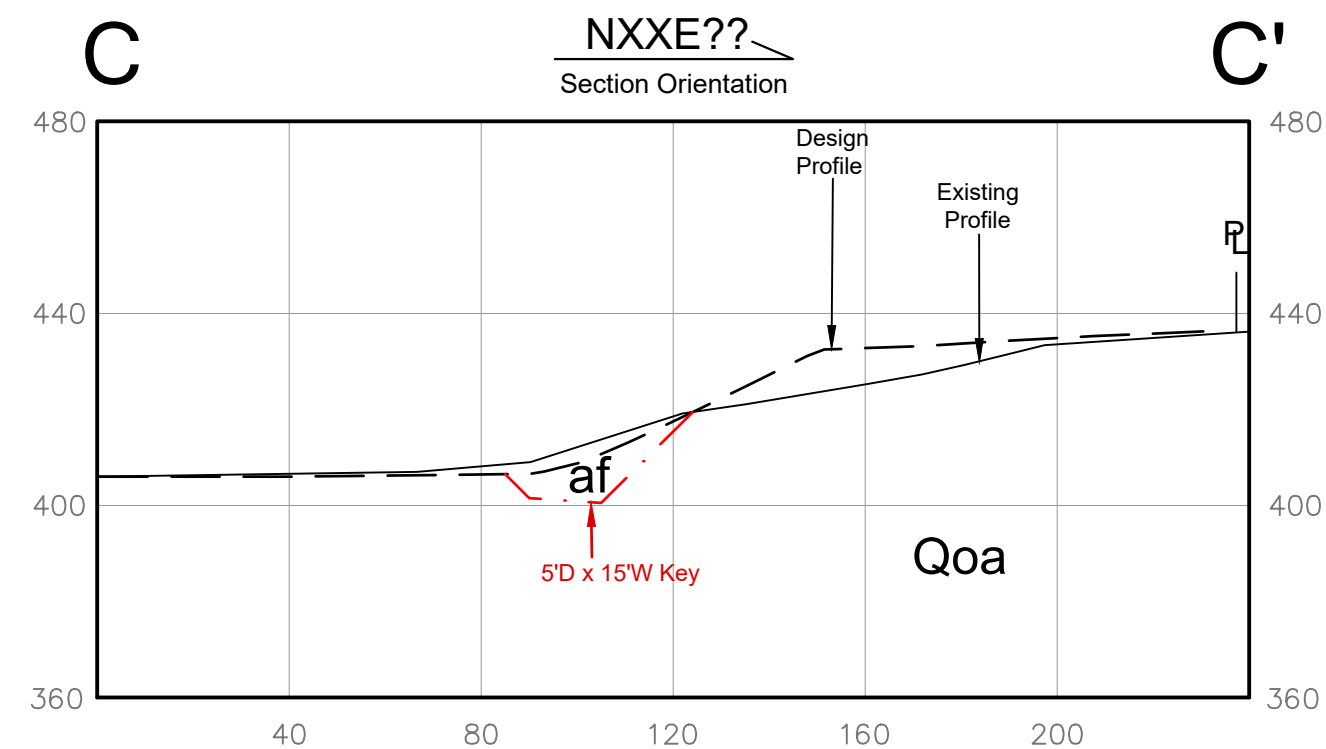
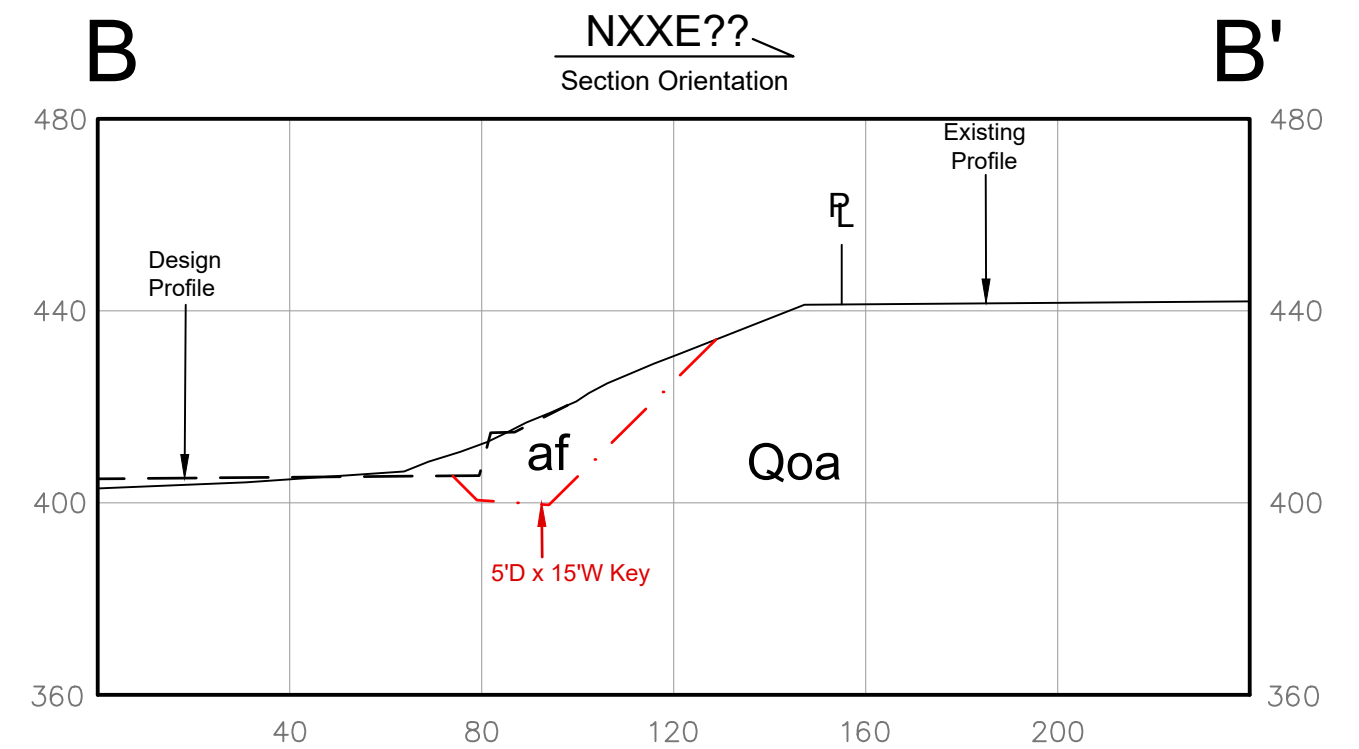
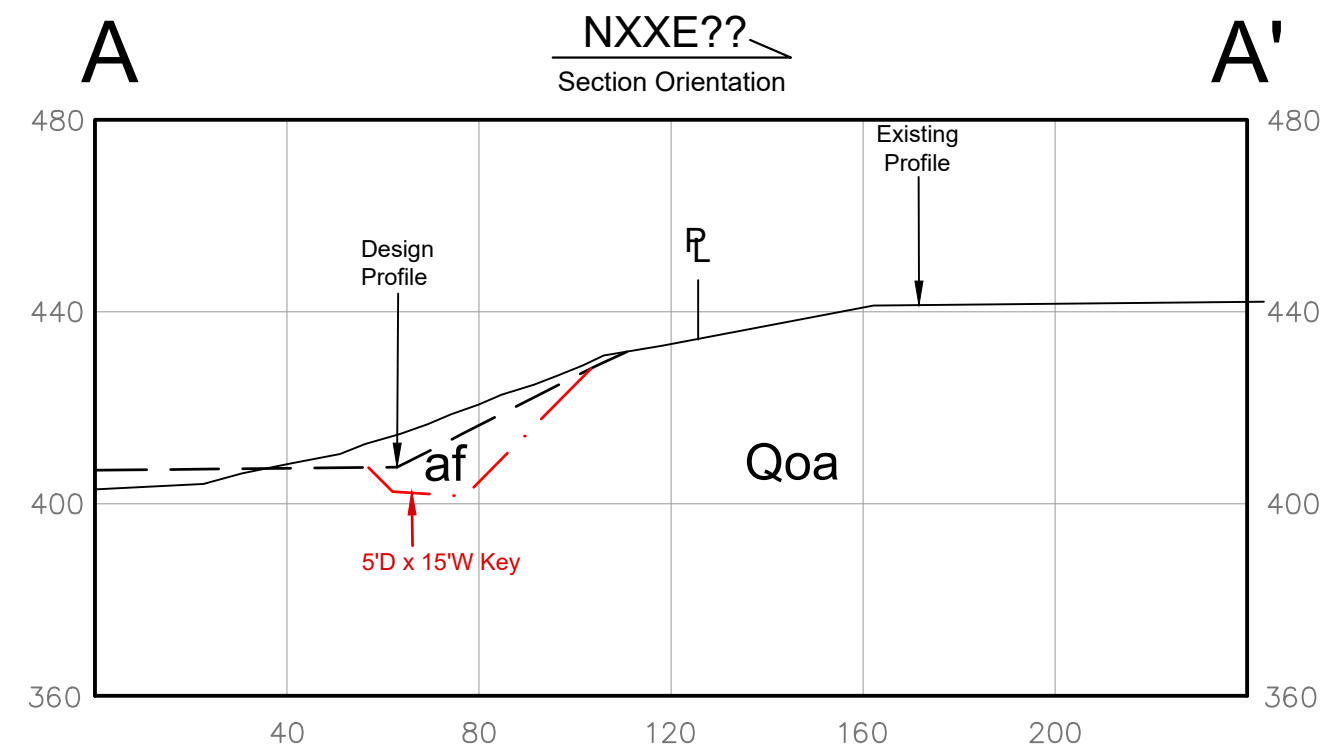
Our services were performed using the degree of care and skill ordinarily exercised, under similar circumstances, by reputable engineers and geologists practicing in this or similar localities. No other warranty, expressed or implied, is made as to the conclusions and professional advice included in this report. The samples taken and submitted for laboratory testing, the observations made, and the in-situ field testing performed are believed representative of the entire project; however, soil and geologic conditions revealed by excavation may be different than our preliminary findings. If this occurs, the changed conditions must be evaluated by the project soils engineer and geologist and design(s) adjusted as required or alternate design(s) recommended.

This report is issued with the understanding that it is the responsibility of the owner, or of his/her representative, to ensure that the information and recommendations contained herein are brought to the attention of the architect and/or project engineer and incorporated into the plans, and the necessary steps are taken to see that the contractor and/or subcontractor properly implements the recommendations in the field. The contractor and/or subcontractor should notify the owner if they consider any of the recommendations presented herein to be unsafe.

The findings of this report are valid as of the present date. However, changes in the conditions of a property can and do occur with the passage of time, whether they be due to natural processes or the works of man on this or adjacent properties. Therefore, the findings, conclusions, and recommendations presented in this report can be relied upon only if LGC Geotechnical has the opportunity to observe the subsurface conditions during grading and construction of the project, in order to confirm that our preliminary findings are representative for the site.

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 modification, and should not be relied upon after a period of 3 years.





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Figure 3
Geotechnical Cross-Sections (A-D)

PROJECT NAME	Milan Capital - Arena Site, Orange
PROJECT NO.	25046-01
ENG. / GEOL.	DJB / KTM
SCALE	1" = 40'
DATE	July 2025

Appendix A

References

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APPENDIX A (Cont'd)

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Appendix B

Exploration Logs

Geotechnical Boring Log Borehole I-1

Date: 6/26/2025	Drilling Company: 2R Drilling
Project Name: Milan - Arena Site, Orange	Type of Rig: CME 85
Project Number: 25046-01	Drop: 30" Hole Diameter: 8"
Elevation of Top of Hole: ~-400' MSL	Drive Weight: 140 pounds
Hole Location: See Geotechnical Map	Page 1 of 1

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	Logged By OJL Sampled By OJL Checked By BPP DESCRIPTION	Type of Test
395	0							@ 0' - 2" of Asphalt over 9" of Base	
								Quaternary Old Alluvial Fan Deposits (Qof)	
								@5' - Sandy CLAY: brown, moist, very stiff	-#200
395	5		R-1	14 16 13	97.1	9.9	CL		
390	10							Total Depth = 7' Groundwater Not Encountered 3" Perforated Pipe with Filter Sock Installed, Surrounded by Gravel, and Presoaked on 6/26/25 Backfilled with Cuttings on 6/27/25	
385	15								
380	20								
375	25								
370	30								



THIS SUMMARY APPLIES ONLY AT THE LOCATION OF THIS BORING AND AT THE TIME OF DRILLING. SUBSURFACE CONDITIONS MAY DIFFER AT OTHER LOCATIONS AND MAY CHANGE AT THIS LOCATION WITH THE PASSAGE OF TIME. THE DATA PRESENTED IS A SIMPLIFICATION OF THE ACTUAL CONDITIONS ENCOUNTERED. THE DESCRIPTIONS PROVIDED ARE QUALITATIVE FIELD DESCRIPTIONS AND ARE NOT BASED ON QUANTITATIVE ENGINEERING ANALYSIS.

SAMPLE TYPES:
 B BULK SAMPLE
 R RING SAMPLE (CA Modified Sampler)
 G GRAB SAMPLE
 SPT STANDARD PENETRATION TEST SAMPLE



TEST TYPES:
 DS DIRECT SHEAR
 MD MAXIMUM DENSITY
 SA SIEVE ANALYSIS
 S&H SIEVE AND HYDROMETER
 EI EXPANSION INDEX
 CN CONSOLIDATION
 CR CORROSION
 AL ATTERBERG LIMITS
 CO COLLAPSE/SWELL
 RV R-VALUE
 -#200 % PASSING # 200 SIEVE

Geotechnical Boring Log Borehole I-2

Date: 6/26/2025	Drilling Company: 2R Drilling
Project Name: Milan - Arena Site, Orange	Type of Rig: CME 85
Project Number: 25046-01	Drop: 30" Hole Diameter: 8"
Elevation of Top of Hole: ~400' MSL	Drive Weight: 140 pounds
Hole Location: See Geotechnical Map	Page 1 of 1

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	Logged By OJL Sampled By OJL Checked By BPP DESCRIPTION	Type of Test
	0							@ 0' - Dried vegetation with gravel and some cobbles	
395	5		R-1	22 8 9	105.7	9.3	CL/ML to CL	Quaternary Old Alluvial Fan Deposits (Qof) @ 5' - Sandy SILT/CLAY to CLAY: brown, slightly moist, stiff	#200 DS
390	10							Total Depth = 8' Groundwater Not Encountered 3" Perforated Pipe with Filter Sock Installed, Surrounded by Gravel, and Presoaked on 6/26/25 Backfilled with Cuttings on 6/27/2025	
385	15								
380	20								
375	25								
370	30								



THIS SUMMARY APPLIES ONLY AT THE LOCATION OF THIS BORING AND AT THE TIME OF DRILLING. SUBSURFACE CONDITIONS MAY DIFFER AT OTHER LOCATIONS AND MAY CHANGE AT THIS LOCATION WITH THE PASSAGE OF TIME. THE DATA PRESENTED IS A SIMPLIFICATION OF THE ACTUAL CONDITIONS ENCOUNTERED. THE DESCRIPTIONS PROVIDED ARE QUALITATIVE FIELD DESCRIPTIONS AND ARE NOT BASED ON QUANTITATIVE ENGINEERING ANALYSIS.

SAMPLE TYPES:
 B BULK SAMPLE
 R RING SAMPLE (CA Modified Sampler)
 G GRAB SAMPLE
 SPT STANDARD PENETRATION TEST SAMPLE



TEST TYPES:
 DS DIRECT SHEAR
 MD MAXIMUM DENSITY
 SA SIEVE ANALYSIS
 S&H SIEVE AND HYDROMETER
 EI EXPANSION INDEX
 CN CONSOLIDATION
 CR CORROSION
 AL ATTERBERG LIMITS
 CO COLLAPSE/SWELL
 RV R-VALUE
 #200 % PASSING # 200 SIEVE

Geotechnical Boring Log Borehole I-3

Date: 6/26/2025	Drilling Company: 2R Drilling
Project Name: Milan - Arena Site, Orange	Type of Rig: CME 85
Project Number: 25046-01	Drop: 30" Hole Diameter: 8"
Elevation of Top of Hole: ~402' MSL	Drive Weight: 140 pounds
Hole Location: See Geotechnical Map	Page 1 of 1

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	Logged By OJL Sampled By OJL Checked By BPP DESCRIPTION	Type of Test
400	0							@ 0' - Gravel with dead vegetation	
395	5		R-1	8 28 31	105.7	3.8	ML	Quaternary Old Alluvial Fan Deposits (Qof) @ 5' - Gravelly SILT with SAND: orangish brown, slightly moist, hard	#200
390	10							Total Depth = 5' Groundwater Not Encountered 3" Perforated Pipe with Filter Sock Installed, Surrounded by Gravel, and Presoaked on 6/26/25 Backfilled with Cuttings on 6/27/2025	
385	15								
380	20								
375	25								
	30								



THIS SUMMARY APPLIES ONLY AT THE LOCATION OF THIS BORING AND AT THE TIME OF DRILLING. SUBSURFACE CONDITIONS MAY DIFFER AT OTHER LOCATIONS AND MAY CHANGE AT THIS LOCATION WITH THE PASSAGE OF TIME. THE DATA PRESENTED IS A SIMPLIFICATION OF THE ACTUAL CONDITIONS ENCOUNTERED. THE DESCRIPTIONS PROVIDED ARE QUALITATIVE FIELD DESCRIPTIONS AND ARE NOT BASED ON QUANTITATIVE ENGINEERING ANALYSIS.

SAMPLE TYPES:
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 R RING SAMPLE (CA Modified Sampler)
 G GRAB SAMPLE
 SPT STANDARD PENETRATION TEST SAMPLE

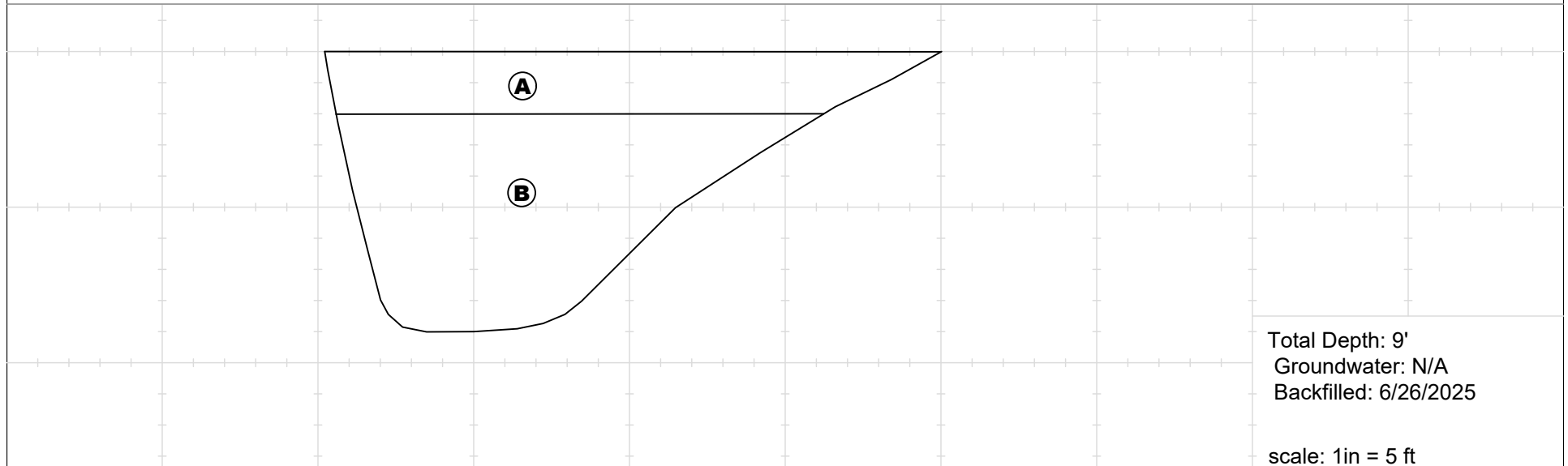
GROUNDWATER TABLE

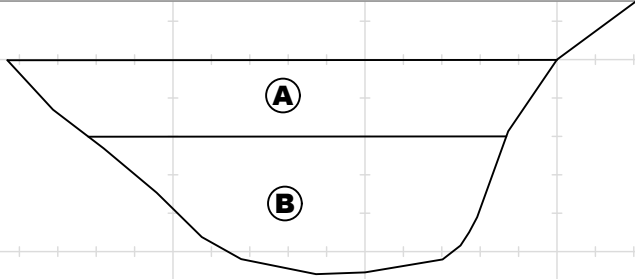
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 RV R-VALUE
 #200 % PASSING # 200 SIEVE

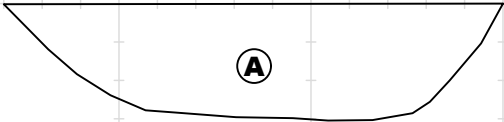
Project Name: Milan Capital - Arena Site	Logged By: KMC	Trench No.: TP-1	
Project Number: 25046-01	Date: 6/26/2025	Engineering Properties:	
Equipment: Backhoe	Location: See Geotechnical Map		

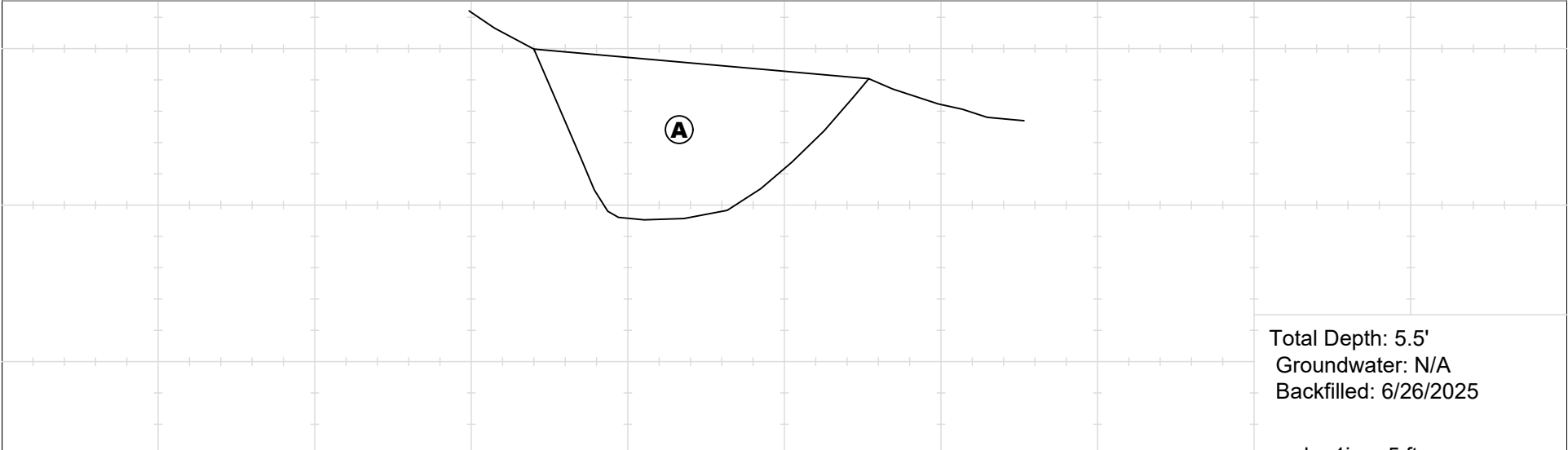
Geologic Attitudes	Unit	SOIL DESCRIPTION:	GEOLOGIC UNIT	USCS	SAMPLE No	MOISTURE (%)	DRY DENSITY (PCF)
	A	Artificial Fill, Undocumented (Afu) @ 0' to 2' - Clayey SAND with Gravel: light brown, dry, dense, gravel up 2" in diameter	afu	SC	B-1 @0'-9'		
	B	Quaternary Young Alluvial Fan Deposits (Qya) @ 2' to 9' - Clayey GRAVEL (2-3') to Well-Graded Gravels (3-9'): light brown with tint of red, dry, medium dense, fine gravel to cobbles up to 24" in diameter, clast dominant (0 - 8" in diameter ~60%, 8" - 24" in diameter ~20%), subangular to prodominately subrounded gravel to boulders, sand is fine- to medium-grained, no fines (binder) present, unconsolidated, friable / side walls collapsing, rootlets to 5'	Qya	GC to GW			

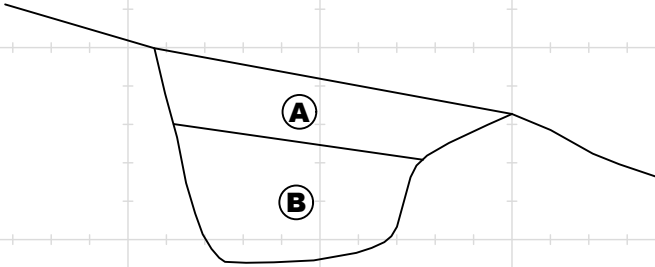
GRAPHICAL REPRESENTATION BELOW: **Elevation: 405' MSL** **Surface Slope: 0 deg.** **Trend: N60W**



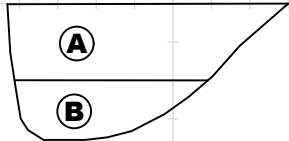
Project Name: Milan Capital - Arena Site		Logged By: KMC		Trench No.: TP-2			
Project Number: 25046-01		Date: 6/26/2025		Engineering Properties:			
Equipment: Backhoe		Location: See Geotechnical Map					
Geologic Attitudes	Unit	SOIL DESCRIPTION:	GEOLOGIC UNIT	USCS	SAMPLE No	MOISTURE (%)	DRY DENSITY (PCF)
	A	Artificial Fill, Undocumented (Afu) @ 0' to 2' - Silty SAND with Gravel: light brown, dry, medium dense, fine- to medium-grained, blocky, subangular to subrounded gravel, rootlets	afu	SM	B-1 @0'-5.5'		
	B	Quaternary Old Alluvial Fan Deposits (Qof) @ 2' to 5.5' - Clayey SAND with Gravel: light brown to light reddish brown, dry to slightly moist, dense, fine-grained, subrounded to subangular gravel, subrounded cobbles / boulders, 0 - 8" in diameter ~10%, 8" - 12" in diameter ~3%	Qof	SC			
GRAPHICAL REPRESENTATION BELOW:			Elevation: 408' MSL	Surface Slope: 0 deg.	Trend: N10W		
							
Total Depth: 5.5' Groundwater: N/A Backfilled: 6/26/2025 scale: 1in = 5 ft							

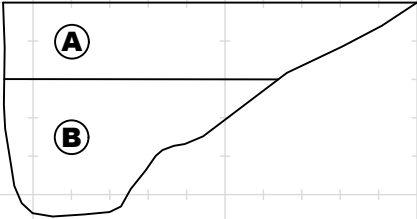
Project Name: Milan Capital - Arena Site		Logged By: KMC		Trench No.: TP-3			
Project Number: 25046-01		Date: 6/26/2025		Engineering Properties:			
Equipment: Backhoe		Location: See Geotechnical Map					
Geologic Attitudes	Unit	SOIL DESCRIPTION:	GEOLOGIC UNIT	USCS	SAMPLE No	MOISTURE (%)	DRY DENSITY (PCF)
	A	Quaternary Young Alluvial Fan Deposits (Qof) @ 0' to 3' - Silty, Sandy GRAVEL: brown, dry to slightly moist, dense, fine- to medium-grained sand, clast dominant, subangular to subrounded fine gravel to boulders up to 24" in diameter, 0 - 8" in diameter ~40%, 8" - 24" in diameter ~15%, roots up to $\frac{1}{4}$" in diameter	Qof	GM			
GRAPHICAL REPRESENTATION BELOW: Elevation: 421' MSL Surface Slope: 13 deg. Trend: N15E  Refusal: 3' Groundwater: N/A Backfilled: 6/26/2025 scale: 1in = 5 ft							

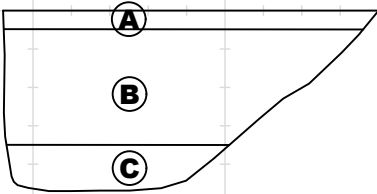
Project Name: Milan Capital - Arena Site		Logged By: KMC		Trench No.: TP-4			
Project Number: 25046-01		Date: 6/26/2025		Engineering Properties:			
Equipment: Backhoe		Location: See Geotechnical Map					
Geologic Attitudes	Unit	SOIL DESCRIPTION:	GEOLOGIC UNIT	USCS	SAMPLE No	MOISTURE (%)	DRY DENSITY (PCF)
@4' B: N50E 11SE	A	Quaternary Old Alluvial Fan Deposits (Qof) @ 0' to 5.5' - Clayey SAND with Gravel: brown, dry to slightly moist, medium dense, fine-grained, 1-2" gravel <~5%, trace cobbles / boulders up to 15" <~2%, abundant roots up to 1/2" diameter to 3' below ground surface	Qof	SC	B-1 @0'-5.5'		
GRAPHICAL REPRESENTATION BELOW: Elevation: 416' MSL Surface Slope: 15 deg. Trend: N15E							
 Total Depth: 5.5' Groundwater: N/A Backfilled: 6/26/2025 scale: 1in = 5 ft							

Project Name: Milan Capital - Arena Site		Logged By: KMC		Trench No.: TP-5			
Project Number: 25046-01		Date: 6/26/2025		Engineering Properties:			
Equipment: Backhoe		Location: See Geotechnical Map					
Geologic Attitudes	Unit	SOIL DESCRIPTION:	GEOLOGIC UNIT	USCS	SAMPLE No	MOISTURE (%)	DRY DENSITY (PCF)
	A	Slopewash (Qw) @ 0' to 2' - Silty CLAY: light grayish brown, dry, medium stiff, rootlets / roots 1/8th in diameter	Qw	CL	B-1 @0'-5.5'		
	B	Quaternary Old Alluvial Fan Deposits (Qof) @ 2' to 5.5' - Clayey SAND with Gravel: brown, dry, dense, fine-grained, subangular to subrounded cobbles predominately 2" - 6", 8" to 12" ~3%, gravel / cobbles predominately @ 4' to 5.5' below ground surface	Qof	SC			
GRAPHICAL REPRESENTATION BELOW:			Elevation: 416' MSL	Surface Slope: 14 deg.	Trend: N10W		
							
Practical Refusal: 5.5' Groundwater: N/A Backfilled: 6/26/2025 scale: 1in = 5 ft							

Project Name: Milan Capital - Arena Site		Logged By: KMC		Trench No.: TP-6			
Project Number: 25046-01		Date: 6/26/2025		Engineering Properties:			
Equipment: Backhoe		Location: See Geotechnical Map					
Geologic Attitudes	Unit	SOIL DESCRIPTION:	GEOLOGIC UNIT	USCS	SAMPLE No	MOISTURE (%)	DRY DENSITY (PCF)
	A	Quaternary Old Alluvial Fan Deposits (Qof) @ 0' to 3.5' - Silty SAND with Gravel and Clay: brown, dry to slightly moist, dense, subangular to subrounded gravel / cobbles 0 - 8" in diameter ~10%, 8" - 12" in diameter ~5%	Qof	SM		B-1 @ 0 - 3.5'	
GRAPHICAL REPRESENTATION BELOW:							
Elevation: 401' MSL Surface Slope: 0 deg. Trend: N75E  Total Depth: 3.5' Groundwater: N/A Backfilled: 6/26/2025 scale: 1in = 5 ft							

Project Name: Milan Capital - Arena Site		Logged By: KMC		Trench No.: TP-7			
Project Number: 25046-01		Date: 6/26/2025		Engineering Properties:			
Equipment: Backhoe		Location: See Geotechnical Map					
Geologic Attitudes	Unit	SOIL DESCRIPTION:	GEOLOGIC UNIT	USCS	SAMPLE No	MOISTURE (%)	DRY DENSITY (PCF)
	A	Artificial Fill, Undocumented (Afu) @ 0' to 2' - Clayey SILT with Gravel: light gray, dry, dense, 0.5" to 1" angular gravel, debris	afu	ML	B-1 @0'-5.5'		
	B	Quaternary Old Alluvial Fan Deposits (Qof) @ 2' to 5.5' - Clayey GRAVEL with Sand: brown with orange tint, dry to slightly moist, dense, clast dominant, subrounded gravel / cobbles 0 - 8" in diameter ~60%, 8" - 12" in diameter ~5%, fine-grained sand, friable, sidewalls eroded out	Qof	GC			
GRAPHICAL REPRESENTATION BELOW:			Elevation: 402' MSL	Surface Slope: 0 deg.	Trend: E-W		
							
Practical Refusal: 3.5' Groundwater: N/A Backfilled: 6/26/2025 scale: 1in = 5 ft							

Project Name: Milan Capital - Arena Site		Logged By: KMC		Trench No.: TP-8			
Project Number: 25046-01		Date: 6/26/2025		Engineering Properties:			
Equipment: Backhoe		Location: See Geotechnical Map					
Geologic Attitudes	Unit	SOIL DESCRIPTION:	GEOLOGIC UNIT	USCS	SAMPLE No	MOISTURE (%)	DRY DENSITY (PCF)
	A	Artificial Fill, Undocumented (Afu) @ 0' to 2' - Sandy CLAY with Gravel: brown, dry, medium stiff, low plasticity, blocky, rootlets	afu	CL	B-1 @0'-5.5'		
	B	Quaternary Old Alluvial Fan Deposits (Qof) @ 2' to 5.5' - Silty CLAY with trace fine gravel: dark brown, dry, hard, 1" to 2" subrounded to subangular gravel	Qof				
GRAPHICAL REPRESENTATION BELOW: Elevation: 399' MSL Surface Slope: 0 deg. Trend: N - S							
 Total Depth: 5.5' Groundwater: N/A Backfilled: 6/26/2025 scale: 1in = 5 ft							

Project Name: Milan Capital - Arena Site		Logged By: KMC		Trench No.: TP-9			
Project Number: 25046-01		Date: 6/26/2025		Engineering Properties:			
Equipment: Backhoe		Location: See Geotechnical Map					
Geologic Attitudes	Unit	SOIL DESCRIPTION:	GEOLOGIC UNIT	USCS	SAMPLE No	MOISTURE (%)	DRY DENSITY (PCF)
	A	Artificial Fill, Undocumented (Afu) @ 0' to 0.5' - Silty CLAY: brown, dry, medium stiff, low plasticity, blocky, rootlets / roots	afu	CL	B-1 @0'-4.5'		
	B	Quaternary Old Alluvial Fan Deposits (Qof) @ 0.5' to 3.5' - Silty SAND: brown, dry, dense, fine-grained, 1" roots to 3'	Qof	SM			
	C	@ 3.5' to 4.5' - Well Graded Gravels: brown, dry, dense, clast dominant, subrounded gravel / cobbles / boulders 0 - 8" in diameter ~60%, 8" - 24" in diameter ~5%, friable		GW			
GRAPHICAL REPRESENTATION BELOW:			Elevation: 399' MSL	Surface Slope: 0 deg.	Trend: N-S		
							
Practical Refusal: 4.5' Groundwater: N/A Backfilled: 6/26/2025 scale: 1in = 5 ft							



GINTER AND ASSOCIATES, INC

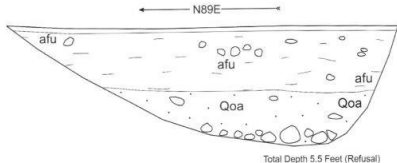
LOG OF EXPLORATORY TRENCH

PROJECT #: 172-01
DATE STARTED: 02/09/15
DATE FINISHED: 02/09/15
NOTES:

PROJECT NAME: Rio Santiago
GROUND ELEV: 408

TRENCH DESIG: T-1
LOGGED BY: BW

Trench T-1 (Profile of south wall)



SCALE: 1 inch = 2.5 Feet

Lithologic Description

0- 2.75 Feet- Artificial Fill (afu):

Sandy Clay and Clayey Sand, dark brown to dark reddish brown, moist, loose to medium dense, larger rocks absent (tilled/farmed?), minor dark brown Clayey Sand soil from surface to approximately 0.15 feet. Frequent pebble and cobble size clasts, subangular to subround, up to 6", chiefly mafic volcanics, with lesser granitics and metamorphics.

2.75- 5.5 Feet- Older Alluvium (Qoa):

Clayey Sand and lesser Gravelly Sand, reddish brown to dark reddish brown, slightly moist, hard to very hard, frequent pebble to boulder size clasts (up to 1 foot), chiefly mafic volcanics, with lesser granitics and metamorphics.

Refusal at 5.5. feet.



GINTER AND ASSOCIATES, INC LOG OF EXPLORATORY TRENCH

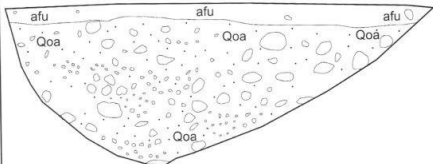
PROJECT #: 172-01
DATE STARTED: 02/09/15
DATE FINISHED: 02/09/15
NOTES:

PROJECT NAME: Rio Santiago
GROUND ELEV: 402

TRENCH DESIG: T-2
LOGGED BY: BW

Trench T-2 (Profile of east wall)

← N12E →



Total Depth 7 Feet (Refusal)

SCALE: 1 inch = 2.5 Feet

Lithologic Description

0- 1.25 Feet- Artificial Fill (afu):

Sandy Clay and Clayey Sand, dark brown to dark reddish brown, moist, loose to medium dense, larger rocks absent (tilled/farmed?), minor dark brown Clayey Sand soil from surface to approximately 0.15 feet. Frequent pebble and cobble size clasts, subangular to subround, up to 6", chiefly mafic volcanics, with lesser granitics and metamorphics.

2.75- 5.5 Feet- Older Alluvium (Qoa):

Gravelly Sand, reddish brown to dark reddish brown, slightly moist, hard to very hard, frequent pebble to boulder size clasts (up to 1.25 foot), chiefly mafic volcanics, with lesser granitics and metamorphics.

Refusal at 7 feet.



GINTER AND ASSOCIATES, INC LOG OF EXPLORATORY TRENCH

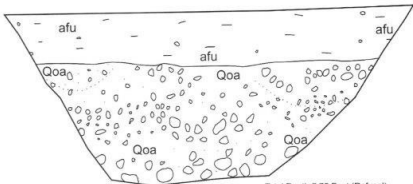
PROJECT #: 172-01
DATE STARTED: 02/09/15
DATE FINISHED: 02/09/15
NOTES:

PROJECT NAME: Rio Santiago
GROUND ELEV: 403

TRENCH DESIG: T-3
LOGGED BY: BW

Trench T-3 (Profile of south wall)

← N82E →



Total Depth 7.75 Feet (Refusal)

SCALE: 1 inch = 2.5 Feet

Lithologic Description

0- 2.5 Feet- Artificial Fill (afu):

Sandy Clay and Clayey Sand, dark brown to dark reddish brown, moist, loose to medium dense, larger rocks absent (tiled/farmed?), minor dark brown Clayey Sand soil from surface to approximately 0.10 feet. Frequent pebble and cobble size clasts, subangular to subround, up to 6", chiefly mafic volcanics, with lesser granitics and metamorphics.

2.5- 7.75 Feet- Older Alluvium (Qoa):

Gravelly Sand, reddish brown to dark reddish brown, slightly moist, hard to very hard, frequent pebble to boulder size clasts (up to 1.25 foot), chiefly mafic volcanics, with lesser granitics and metamorphics.

Refusal at 7.75 feet.



GINTER AND ASSOCIATES, INC

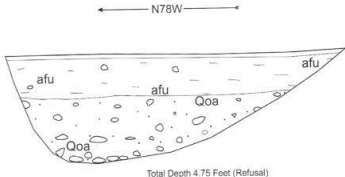
LOG OF EXPLORATORY TRENCH

PROJECT #: 172-01
DATE STARTED: 02/09/15
DATE FINISHED: 02/09/15
NOTES:

PROJECT NAME: Rio Santiago
GROUND ELEV: 417

TRENCH DESIG: T-4
LOGGED BY: 8W

Trench T-4 (Profile of south wall)



SCALE: 1 inch = 2.5 Feet

Lithologic Description

0- 2.75 Feet- Artificial Fill (afu):

Sandy Clay and Clayey Sand, dark brown to dark reddish brown, moist, loose to medium dense, larger rocks absent (tilled/farmed?), Clean tan fill sand surface to approximately 0.2 feet. Infrequent pebble and cobble size clasts, subangular to subround, up to 3", chiefly mafic volcanics, with lesser granitics and metamorphics.

2.75- 4.75 Feet- Older Alluvium (Qoa):

Clayey Sand and lesser Gravelly Sand, reddish brown to dark reddish brown, slightly moist, hard to very hard, frequent pebble to boulder size clasts (up to 1 foot), chiefly mafic volcanics, with lesser granitics and metamorphics.

Refusal at 4.75 feet.



GINTER AND ASSOCIATES, INC

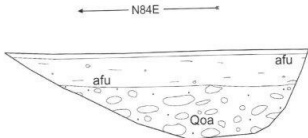
LOG OF EXPLORATORY TRENCH

PROJECT #: 172-01
DATE STARTED: 02/09/15
DATE FINISHED: 02/09/15
NOTES:

PROJECT NAME: Rio Santiago
GROUND ELEV: 410

TRENCH DESIG: T-5
LOGGED BY: BW

Trench T-5 (Profile of south wall)



Total Depth 3.75 Feet (Refusal)

SCALE: 1 inch = 2.5 Feet

Lithologic Description

0- 1.5 Feet- Artificial Fill (afu):

Sandy Clay and Clayey Sand, dark brown to dark reddish brown, moist, loose to medium dense, larger rocks absent (filled/farmed?). Clean tan fill sand surface to approximately 0.15 feet. Infrequent pebble and cobble size clasts, subangular to subround, up to 3", chiefly mafic volcanics, with lesser granitics and metamorphics.

1.5- 3.75 Feet- Older Alluvium (Qoa):

Clayey Sand and lesser Gravelly Sand, reddish brown to dark reddish brown, slightly moist, hard to very hard, frequent pebble to boulder size clasts (up to 1 foot), chiefly mafic volcanics, with lesser granitics and metamorphics.

Refusal at 3.75 feet.



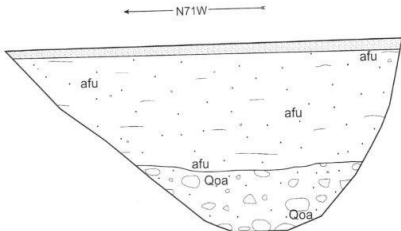
GINTER AND ASSOCIATES, INC LOG OF EXPLORATORY TRENCH

PROJECT #: 172-01
DATE STARTED: 02/09/15
DATE FINISHED: 02/09/15
NOTES:

PROJECT NAME: Rio Santiago
GROUND ELEV: 412

TRENCH DESIG: T-6
LOGGED BY: BW

Trench T-6 (Profile of north wall)



Total Depth 8.5 Feet (Refusal)

SCALE: 1 inch = 2.5 Feet

Lithologic Description

0- 5.5 Feet- Artificial Fill (afu):

Sandy Clay and Clayey Sand, dark brown to brown, moist, dense, occasional pebbles, upper 4 inches: clean tan fill Sand, slightly moist, loose.

5.5- 8.5 Feet- Older Alluvium (Qoa):

Gravelly Sand, reddish brown to dark reddish brown, slightly moist, hard to very hard, frequent pebble to boulder size clasts (up to 2 feet), chiefly mafic volcanics, with lesser granitics and metamorphics.

Refusal at 8.5 feet.

Appendix C
Laboratory Test Results

APPENDIX C

Laboratory Testing Procedures and Test Results

The laboratory testing program was formulated towards providing data relating to the relevant engineering properties of the soils with respect to residential construction. Samples considered representative of site conditions were tested in general accordance with American Society for Testing and Materials (ASTM) procedure and/or California Test Methods (CTM), where applicable. The following summary is a brief outline of the test type and a table summarizing the test results.

Moisture and Density Determination Tests: Moisture content (ASTM D2216) and dry density determinations (ASTM D2937) were performed on relatively undisturbed samples obtained from the test borings. The results of these tests are presented in the boring logs. Where applicable, only moisture content was determined from undisturbed or disturbed samples.

Grain Size Distribution/Fines Content: Representative samples were dried, weighed and soaked in water until individual soil particles were separated (per ASTM D421) and then washed on a No. 200 sieve (ASTM D1140). Where applicable, the portion retained on the No. 200 sieve and dried and then sieved on a U.S. Standard brass sieve set in accordance with ASTM D6913 (sieve).

Sample Location	Description	% Passing # 200 Sieve
I-1 @ 5 feet	Clay with Gravel	54
I-2 @ 5 feet	Sandy Silt/Clay	62
I-3 @ 5 feet	Gravelly Silt with Sand	57

Expansion Index: The expansion potential of selected samples was evaluated by the Expansion Index Test, Standard ASTM D4829. Specimens are molded under a given compactive energy to approximately the optimum moisture content and approximately 50 percent saturation or approximately 90 percent relative compaction. The prepared 1-inch-thick by 4-inch-diameter specimens are loaded to an equivalent 144 psf surcharge and are inundated with tap water until volumetric equilibrium is reached. The results of these tests are presented in the table below.

Sample Location	Expansion Index	Expansion Potential*
TP-6 @ 0-3.5 feet	5	Very Low
TP-8 @ 0-5.5 feet	32	Low

* ASTM D4829

Atterberg Limits: The liquid and plastic limits ("Atterberg Limits") were determined per ASTM D4318 for engineering classification of fine-grained material and presented in the table below. The USCS soil classification indicated in the table below is based on the portion of sample

APPENDIX C (Cont'd)

Laboratory Testing Procedures and Test Results

passing the No. 40 sieve and may not necessarily be representative of the entire sample. The plots are provided in this Appendix.

Sample Location	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	USCS Soil Classification
TP-6 @ 0-3.5 feet	22	15	7	CL-ML

Maximum Density Tests: The maximum dry density and optimum moisture content of typical materials were determined in accordance with ASTM D1557. The results of these tests are presented in the table below:

Sample Location	Sample Description	Maximum Dry Density (pcf)	Optimum Moisture Content (%)
TP-1 @ 0-9 feet	Light Brown Silty Gravel with Sand	140.5	7.0

Direct Shear: One direct shear test was performed on remolded samples and one on relatively undisturbed samples, which were soaked for a minimum of 24 hours prior to testing. The samples were tested under various normal loads using a motor-drive, strain-controlled, direct shear testing apparatus (ASTM D3080). The plots are provided in this Appendix.

Chloride Content: Chloride content was tested in accordance with Caltrans Test Method (CTM) 422. The results are presented below.

Sample Location	Chloride Content, ppm
TP-8 @ 0-5.5 feet	26

Soluble Sulfates: The soluble sulfate contents of selected samples were determined by standard geochemical methods (CTM 417). The soluble sulfate content is used to determine the appropriate cement type and maximum water-cement ratios. The test results are presented in the table below.

Sample Location	Sulfate Content (ppm)	Sulfate Exposure Class *
TP-8 @ 0-5.5 feet	50	S0

*Based on ACI 318R-14, Table 19.3.1.1

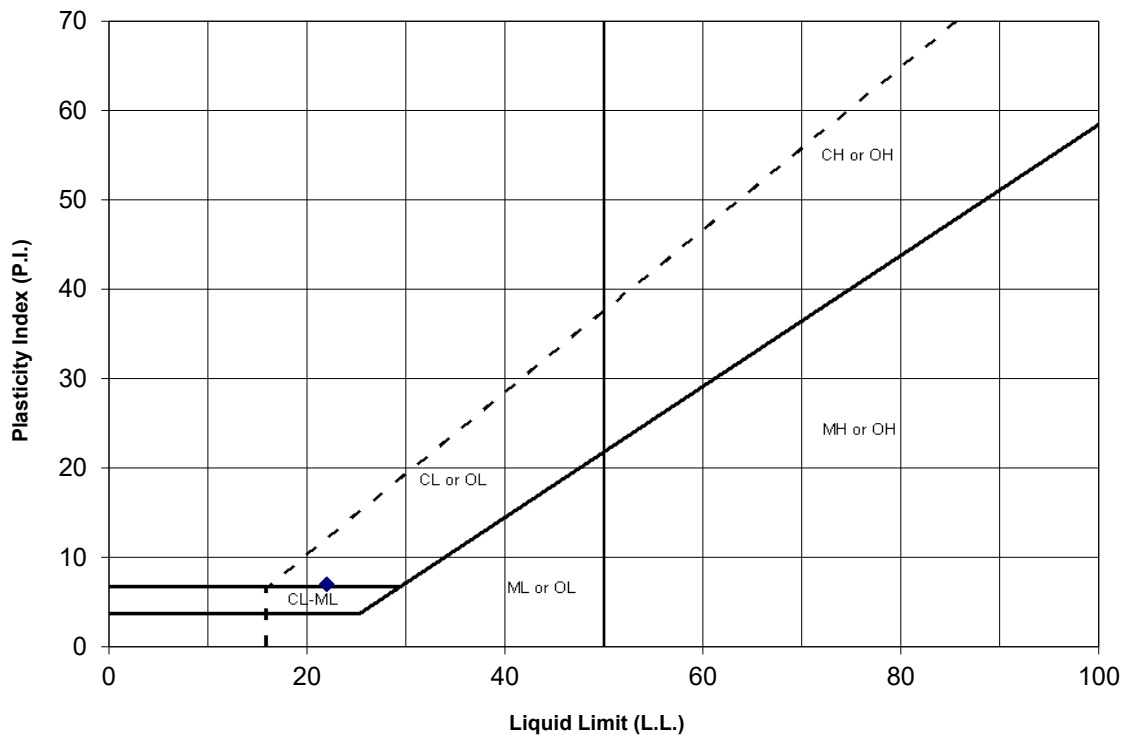
APPENDIX C (Cont'd)

Laboratory Testing Procedures and Test Results

Minimum Resistivity and pH Tests: Minimum resistivity and pH tests were performed in general accordance with CTM 643 and standard geochemical methods. The results are presented in the table below.

Sample Location	pH	Minimum Resistivity (ohms-cm)
TP-8 @ 0-5.5 feet	8.2	2,546

PLASTICITY CHART - CLASSIFICATION OF FINE-GRAINED SOILS



Symbol	Location.:	Sample No.:	Depth (ft)	Passing No. 200 Sieve (%)	Liquid Limit (%) LL	Plastic Limit (%) PL	Plasticity Index (%) PI	USCS
◆	TP-6	B-1	0-3.5'	-	22	15	7	CL-ML



ATTERBERG LIMITS
(ASTM D 4318)

Project Number: 25046-01
Date: Jul-25

Arena Site, Orange

DIRECT SHEAR TEST
Consolidated Drained - ASTM D 3080

Project Name: [Milan - Arena Site, Orange](#)

Project No.: [25046-01](#)

Boring No.: [I-2](#)

Sample No.: [R-1](#)

Soil Identification: [Brown lean clay \(CL\)](#)

Tested By: [G. Bathala](#)

Checked By: [J. Ward](#)

Sample Type: [Ring](#)

Depth (ft.): [5.0](#)

Date: [07/02/25](#)

Date: [07/16/25](#)

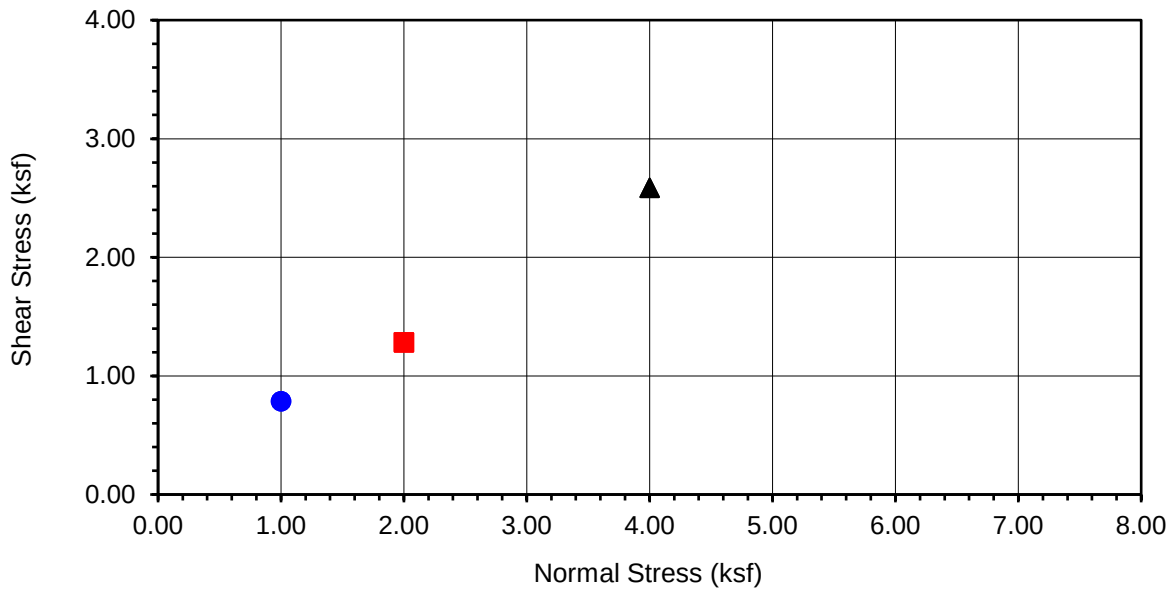
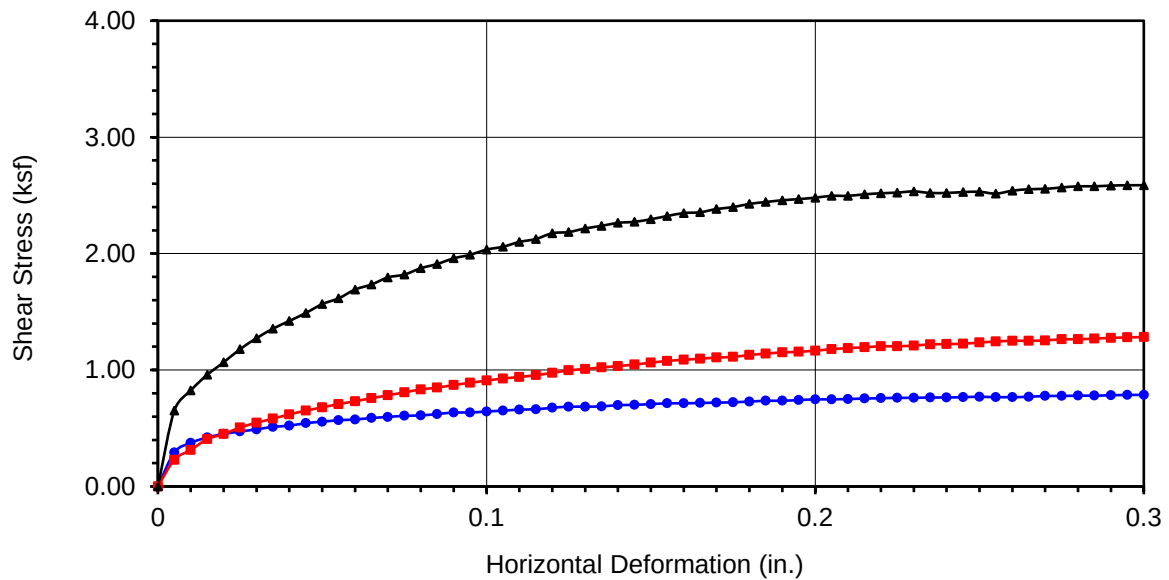
Sample Diameter(in):	2.415	2.415	2.415
Sample Thickness(in.):	1.000	1.000	1.000
Weight of Sample + ring(gm):	183.28	187.13	188.21
Weight of Ring(gm):	43.35	45.04	45.08

Before Shearing

Weight of Wet Sample+Cont.(gm):	98.30	98.30	98.30
Weight of Dry Sample+Cont.(gm):	94.60	94.60	94.60
Weight of Container(gm):	49.36	49.36	49.36
Vertical Rdg.(in): Initial	0.0000	0.2406	0.2485
Vertical Rdg.(in): Final	-0.0154	0.2812	0.3119

After Shearing

Weight of Wet Sample+Cont.(gm):	212.45	183.45	183.04
Weight of Dry Sample+Cont.(gm):	192.14	163.92	165.05
Weight of Container(gm):	67.10	36.53	37.13
Specific Gravity (Assumed):	2.70	2.70	2.70
Water Density(pcf):	62.43	62.43	62.43



Boring No.	I-2
Sample No.	R-1
Depth (ft)	5
<u>Sample Type:</u>	
Ring	
<u>Soil Identification:</u>	
Brown lean clay (CL)	

Normal Stress (kip/ft ²)	1.000	2.000	4.000
Peak Shear Stress (kip/ft ²)	● 0.786	■ 1.283	▲ 2.587
Shear Stress @ End of Test (ksf)	○ 0.786	□ 1.283	△ 2.587
Deformation Rate (in./min.)	0.0017	0.0017	0.0017
Initial Sample Height (in.)	1.000	1.000	1.000
Diameter (in.)	2.415	2.415	2.415
Initial Moisture Content (%)	8.18	8.18	8.18
Dry Density (pcf)	107.6	109.2	110.0
Saturation (%)	39.0	40.7	41.5
Soil Height Before Shearing (in.)	0.9846	0.9594	0.9366
Final Moisture Content (%)	16.2	15.3	14.1

DIRECT SHEAR TEST RESULTS
Consolidated Drained - ASTM D 3080

Project No.: 25046-01

Milan - Arena Site, Orange

07-25

DIRECT SHEAR TEST

Consolidated Drained - ASTM D 3080

Project Name: [Milan - Arena Site, Orange](#)

Project No.: [25046-01](#)

Boring No.: [TP-1](#)

Sample No.: [B-1](#)

Soil Identification: [Light brown silty gravel with sand \(GM\)s](#)

Tested By: [G. Bathala](#)

Checked By: [J. Ward](#)

Sample Type: [90% Remold](#)

Depth (ft.): [0-9](#)

Date: [07/10/25](#)

Date: [07/16/25](#)

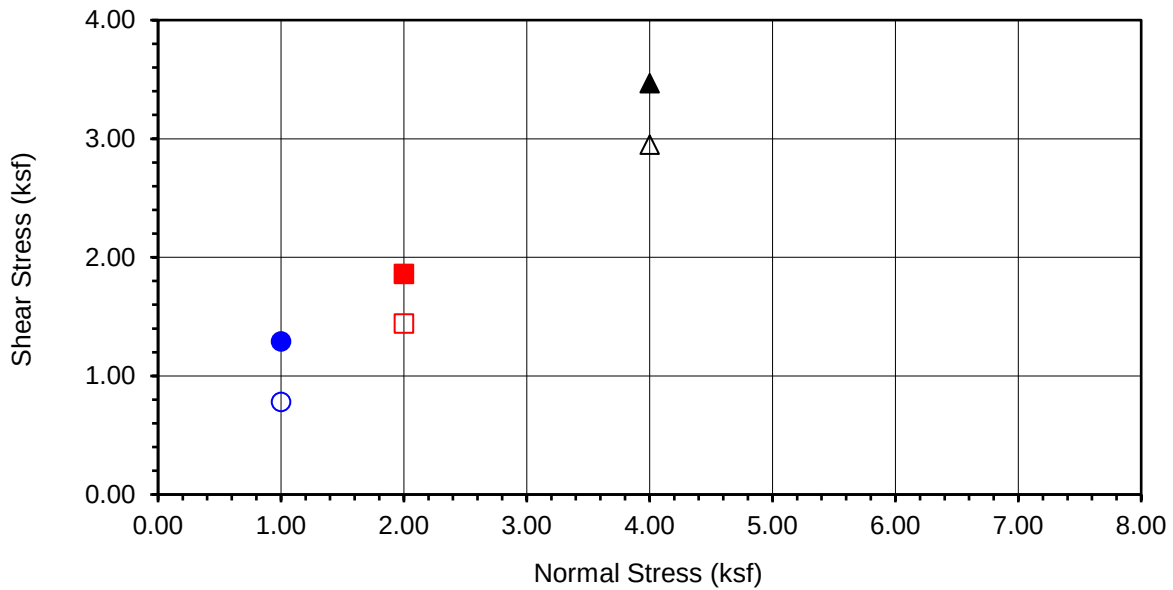
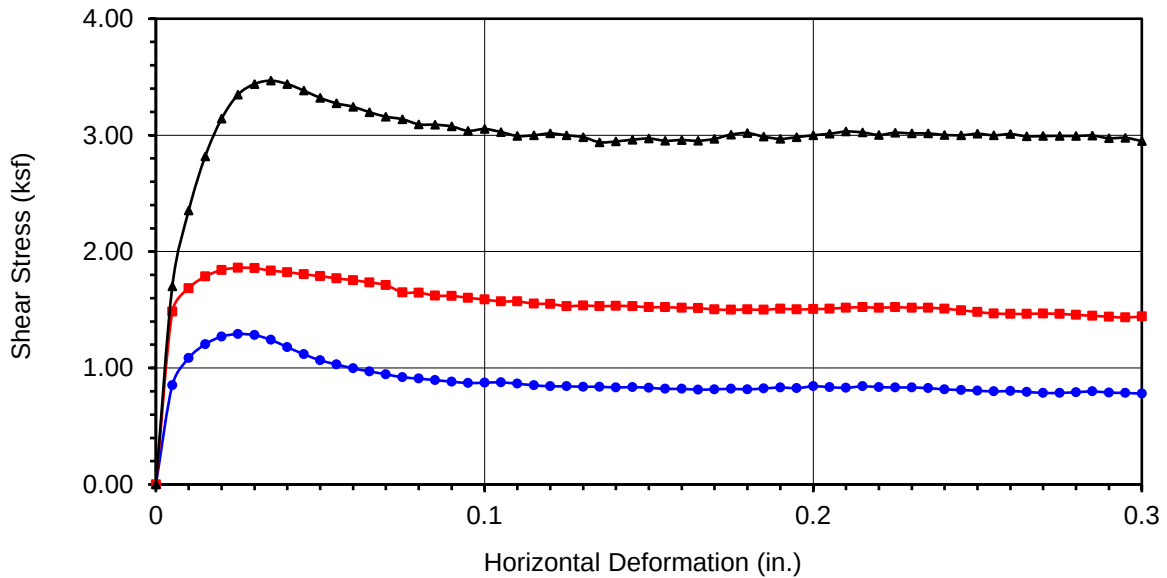
Sample Diameter(in):	2.415	2.415	2.415
Sample Thickness(in.):	1.000	1.000	1.000
Weight of Sample + ring(gm):	205.03	208.17	208.98
Weight of Ring(gm):	41.89	44.92	45.34

Before Shearing

Weight of Wet Sample+Cont.(gm):	190.78	190.78	190.78
Weight of Dry Sample+Cont.(gm):	181.65	181.65	181.65
Weight of Container(gm):	49.39	49.39	49.39
Vertical Rdg.(in): Initial	0.0000	0.2466	0.2587
Vertical Rdg.(in): Final	-0.0084	0.2631	0.2764

After Shearing

Weight of Wet Sample+Cont.(gm):	232.54	217.34	243.94
Weight of Dry Sample+Cont.(gm):	215.50	201.15	228.21
Weight of Container(gm):	64.98	51.17	76.50
Specific Gravity (Assumed):	2.70	2.70	2.70
Water Density(pcf):	62.43	62.43	62.43



Boring No.	TP-1
Sample No.	B-1
Depth (ft)	0-9
<u>Sample Type:</u>	
90% Remold	
<u>Soil Identification:</u>	
Light brown silty gravel with sand (GM)s	

Normal Stress (kip/ft ²)	1.000	2.000	4.000
Peak Shear Stress (kip/ft ²)	● 1.292	■ 1.861	▲ 3.468
Shear Stress @ End of Test (ksf)	○ 0.780	□ 1.443	△ 2.949
Deformation Rate (in./min.)	0.0033	0.0033	0.0033
Initial Sample Height (in.)	1.000	1.000	1.000
Diameter (in.)	2.415	2.415	2.415
Initial Moisture Content (%)	6.90	6.90	6.90
Dry Density (pcf)	126.9	127.0	127.3
Saturation (%)	56.8	57.0	57.5
Soil Height Before Shearing (in.)	0.9916	0.9835	0.9823
Final Moisture Content (%)	11.3	10.8	10.4

DIRECT SHEAR TEST RESULTS
Consolidated Drained - ASTM D 3080

Project No.: 25046-01

Milan - Arena Site, Orange

07-25

Appendix D
Infiltration Test Results

Infiltration Test Data Sheet

LGC Geotechnical, Inc

131 Calle Iglesia Suite 200, San Clemente, CA 92672 tel. (949) 369-6141

Project Name: Milan - Arena Site, Orange
Project Number: 25046-01
Date: 6/27/2025
Boring Number: I-1

Test hole dimensions (if circular)

Boring Depth (feet)*: 7
Boring Diameter (inches): 8
Pipe Diameter (inches): 3

*measured at time of test

Minimum test Head (D_o):

(What the sounder tape should read)

Boring Depth - (5 x Boring Radius) 5.4 ft

(Shallow) The value on the sounder tape should be close to this value during testing for **DEEP** testing fill to 4 feet below top of hole

Pre-Test (Sandy Soil Criteria)*

Trial No.	Start Time (24:HR)	Stop Time (24:HR)	Time Interval (min)	Initial Depth to Water (feet)	Final Depth to Water (feet)	Total Change in Water Level (feet)	Greater Than or Equal to 0.5 feet (yes/no)
1	8:44	9:14	30.0	4.65	5.80	1.15	Yes
2	9:14	9:44	30.0	4.70	5.57	0.87	Yes

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

Main Test Data

Trial No.	Start Time (24:HR)	Stop Time (24:HR)	Time Interval, Dt (min)	Initial Depth to Water, D_o (feet)	Final Depth to Water, D_f (feet)	Change in Water Level, DD (feet)	Measured Infiltration Rate(in/hr)
1	9:47	9:57	10.0	4.90	5.18	0.28	1.58
2	9:57	10:07	10.0	4.88	5.09	0.21	1.16
3	10:07	10:17	10.0	4.84	5.08	0.24	1.31
4	10:17	10:27	10.0	4.95	5.21	0.26	1.50
5	10:30	10:44	14.0	4.90	5.24	0.34	1.39
6	10:45	10:55	10.0	5.00	5.28	0.28	1.66

Measured Infiltration Rate (No Factor of Safety)

1.5

Feasibility Factor of Safety

See Report

Measured Infiltration Rate (With Factor of Safety)

See Report

Sketch:

Notes:



Infiltration Test Data Sheet

LGC Geotechnical, Inc

131 Calle Iglesia Suite 200, San Clemente, CA 92672 tel. (949) 369-6141

Project Name: Milan - Arena Site, Orange
Project Number: 25046-01
Date: 6/27/2025
Boring Number: I-2

Test hole dimensions (if circular)

Boring Depth (feet)*: 8
Boring Diameter (inches): 8
Pipe Diameter (inches): 3

*measured at time of test

Test pit dimensions (if rectangular)

Pit Depth (feet):
Pit Length (feet):
Pit Breadth (feet):

Minimum test Head (D_o):

(What the sounder tape should read)

Boring Depth - (5 x Boring Radius) 6.4 ft

(Shallow) The value on the sounder tape should be close to this value during testing for **DEEP** testing fill to 4 feet below top of hole

Pre-Test (Sandy Soil Criteria)*

Trial No.	Start Time (24:HR)	Stop Time (24:HR)	Time Interval (min)	Initial Depth to Water (feet)	Final Depth to Water (feet)	Total Change in Water Level (feet)	Greater Than or Equal to 0.5 feet (yes/no)
1	8:37	9:07	30.0	4.81	4.97	0.16	No
2	9:07	9:37	30.0	4.70	4.91	0.21	No

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

Main Test Data

Trial No.	Start Time (24:HR)	Stop Time (24:HR)	Time Interval, Dt (min)	Initial Depth to Water, D_o (feet)	Final Depth to Water, D_f (feet)	Change in Water Level, DD (feet)	Measured Infiltration Rate(in/hr)
1	9:37	10:07	30.0	4.55	4.78	0.23	0.3
2	10:08	10:38	30.0	4.36	4.56	0.20	0.2
3	10:38	11:16	38.0	4.56	4.82	0.26	0.2
4	11:16	11:47	31.0	4.65	4.86	0.21	0.2
5	11:48	12:18	30.0	4.78	5.000	0.22	0.3
6	12:18	12:48	30.0	4.82	5.050	0.23	0.3
7	12:49	13:19	30.0	4.79	5.000	0.21	0.3
8	13:20	13:50	30.0	4.81	5.03	0.22	0.3
9	13:50	14:20	30.0	4.84	5.04	0.20	0.2
10	14:20	14:50	30.0	4.83	5.04	0.21	0.3
11	14:50	15:20	30.0	4.83	5.04	0.21	0.3
12	15:20	15:50	30.0	4.75	4.97	0.22	0.3

Measured Infiltration Rate (No Factor of Safety) 0.3

Feasibility Factor of Safety See Report

Measured Infiltration Rate (With Factor of Safety) See Report

Sketch:

Notes:



Infiltration Test Data Sheet

LGC Geotechnical, Inc

131 Calle Iglesia Suite 200, San Clemente, CA 92672 tel. (949) 369-6141

Project Name: Milan - Arena Site, Orange
Project Number: 25046-01
Date: 6/27/2025
Boring Number: I-3

Test hole dimensions (if circular)

Boring Depth (feet)*: 5
Boring Diameter (inches): 8
Pipe Diameter (inches): 3

*measured at time of test

Test pit dimensions (if rectangular)

Pit Depth (feet):
Pit Length (feet):
Pit Breadth (feet):

Minimum test Head (D_o):

(What the sounder tape should read)

Boring Depth - (5 x Boring Radius) 3.4 ft

(Shallow) The value on the sounder tape should be close to this value during testing for **DEEP** testing fill to 4 feet below top of hole

Pre-Test (Sandy Soil Criteria)*

Trial No.	Start Time (24:HR)	Stop Time (24:HR)	Time Interval (min)	Initial Depth to Water (feet)	Final Depth to Water (feet)	Total Change in Water Level (feet)	Greater Than or Equal to 0.5 feet (yes/no)
1	8:40	9:10	30.0	3.94	4.41	0.47	No
2	9:10	9:40	30.0	3.91	4.41	0.50	Yes

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

Main Test Data

Trial No.	Start Time (24:HR)	Stop Time (24:HR)	Time Interval, Dt (min)	Initial Depth to Water, D_o (feet)	Final Depth to Water, D_f (feet)	Change in Water Level, DD (feet)	Measured Infiltration Rate(in/hr)
1	9:41	10:11	30.0	3.71	4.20	0.49	1.6
2	10:12	10:46	34.0	3.80	4.35	0.55	1.8
3	10:47	11:17	30.0	3.81	4.29	0.48	1.7
4	11:21	11:52	31.0	3.75	4.18	0.43	1.4
5	11:52	12:23	31.0	3.70	4.15	0.45	1.4
6	12:24	12:54	30.0	3.74	4.17	0.43	1.4
7	12:55	13:25	30.0	3.70	4.16	0.46	1.5
8	13:26	13:56	30.0	3.72	4.15	0.43	1.4
9	13:56	14:26	30.0	3.74	4.18	0.44	1.5
10	14:26	14:56	30.0	3.77	4.21	0.44	1.5
11	14:56	15:26	30.0	3.72	4.15	0.43	1.4
12	15:26	15:56	30.0	3.80	4.23	0.43	1.5

Measured Infiltration Rate (No Factor of Safety)

1.5

Feasibility Factor of Safety

See Report

Measured Infiltration Rate (With Factor of Safety)

See Report

Sketch:

Notes:



Appendix E
Slope Stability Analysis

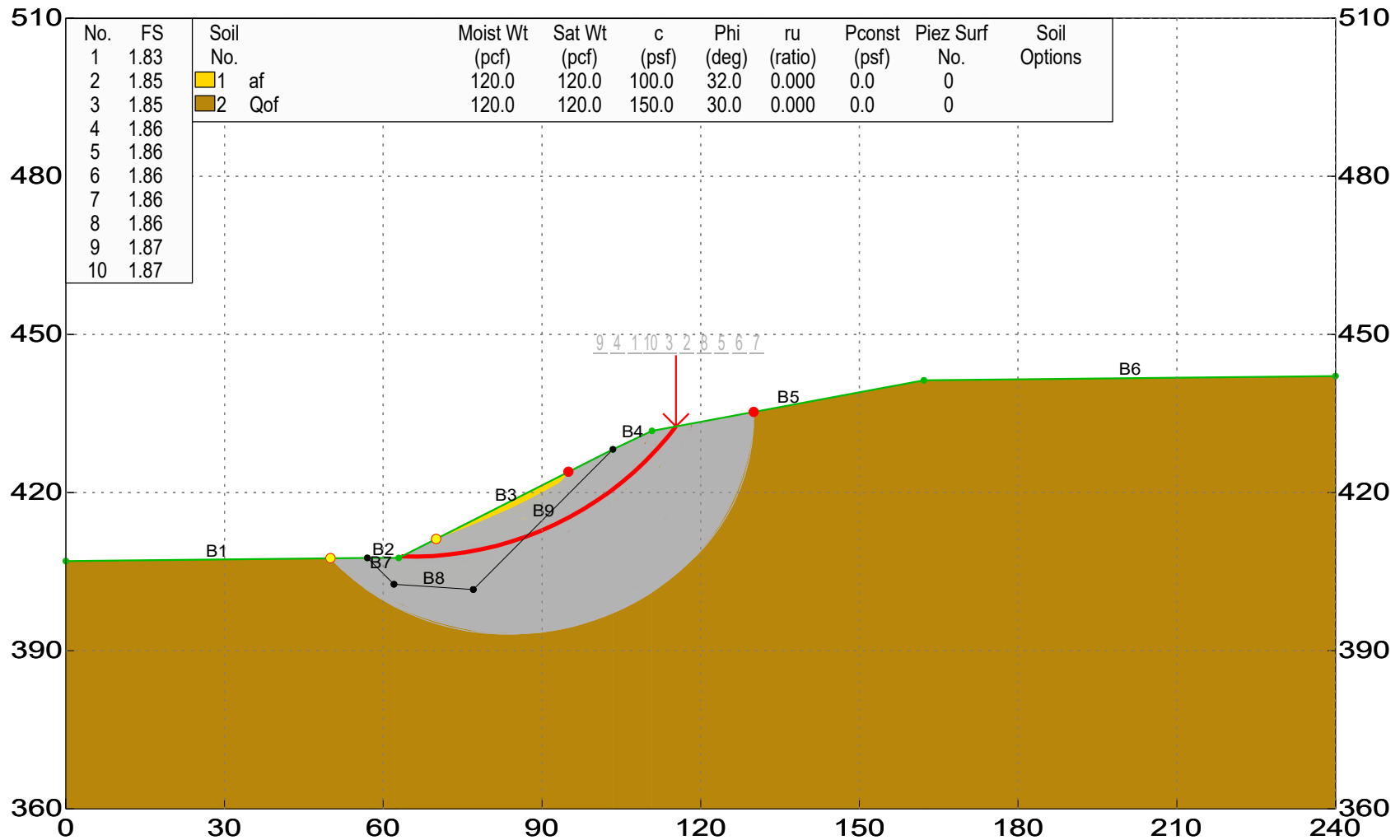
Summary of Slope Stability Analysis

Cross-Section	Factor of Safety	Description
A-A'	1.83	Static (Rotational)
	1.34	Seismic (Rotational)
B-B'	1.68	Static (Rotational)
	1.24	Seismic (Rotational)
C-C'	1.98	Static (Rotational)
	1.42	Seismic (Rotational)
D-D'	1.53	Static (Rotational)
	1.15	Seismic (Rotational)

Milan - Arena Site, Orange Section A-A' (Static)

LGC Geotechnical / BPP

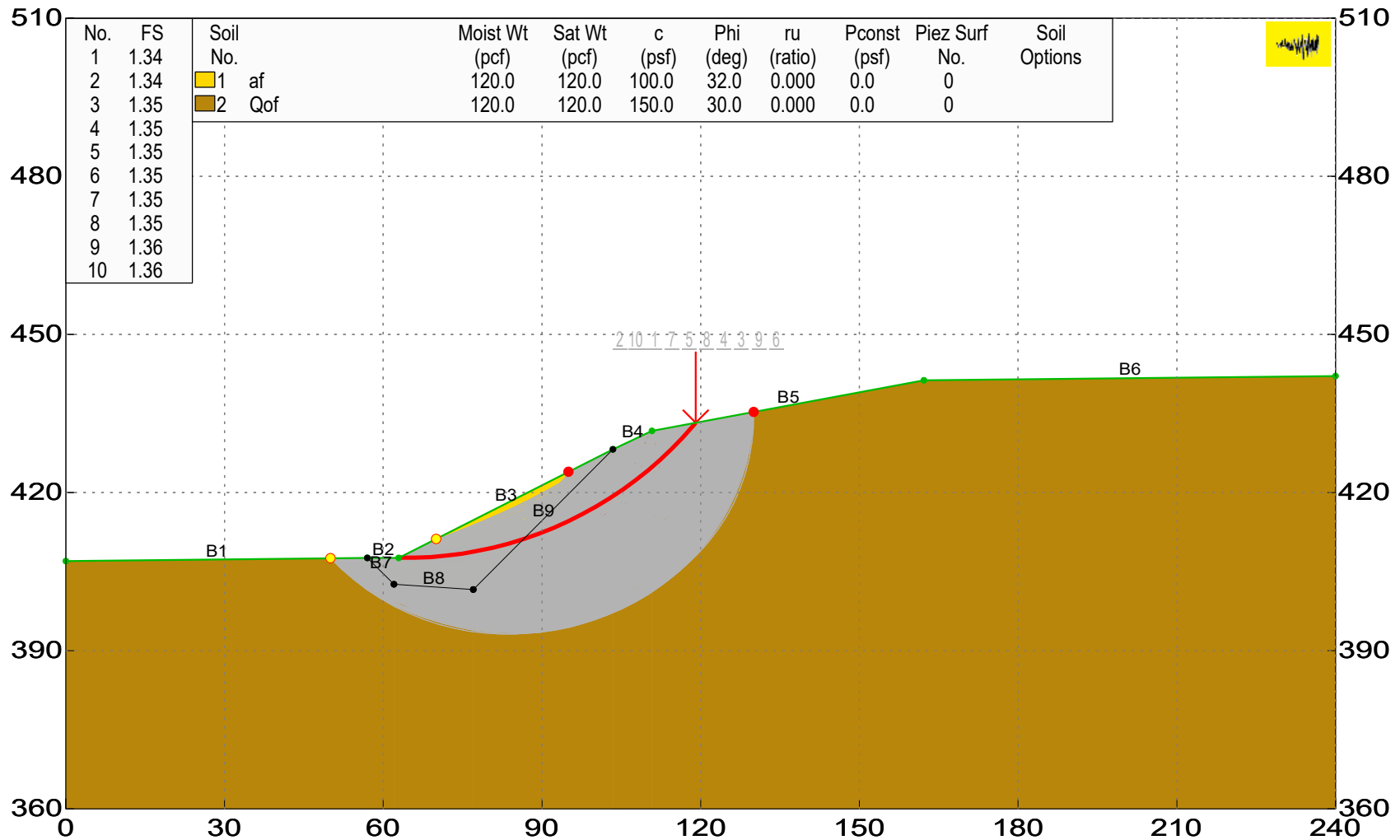
\Section A - Static.gsd



Milan - Arena Site, Orange Section A-A' (Seismic)

LGC Geotechnical / BPP

\Section A - Seismic.gsd



GEOSTASE FS = 1.34

Simplified Bishop Method

kh = 0.15000

GEOSTASE FS = 1.68
Simplified Bishop Method

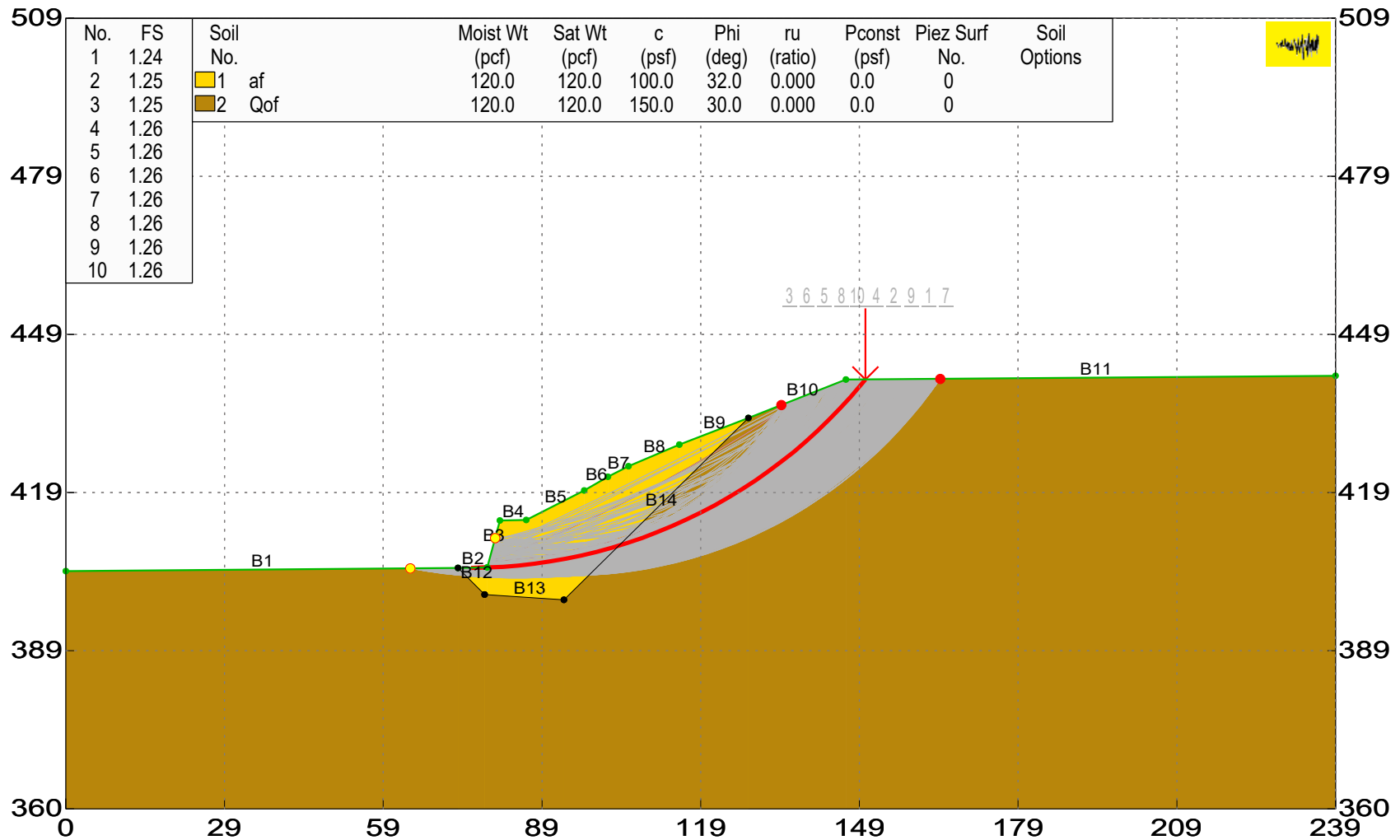
\Section B - Static.gsd



Milan - Arena Site, Orange Section B-B' (Seismic)

LGC Geotechnical / BPP

\Section B - Seismic.gsd



GEOSTASE FS = 1.24

Simplified Bishop Method

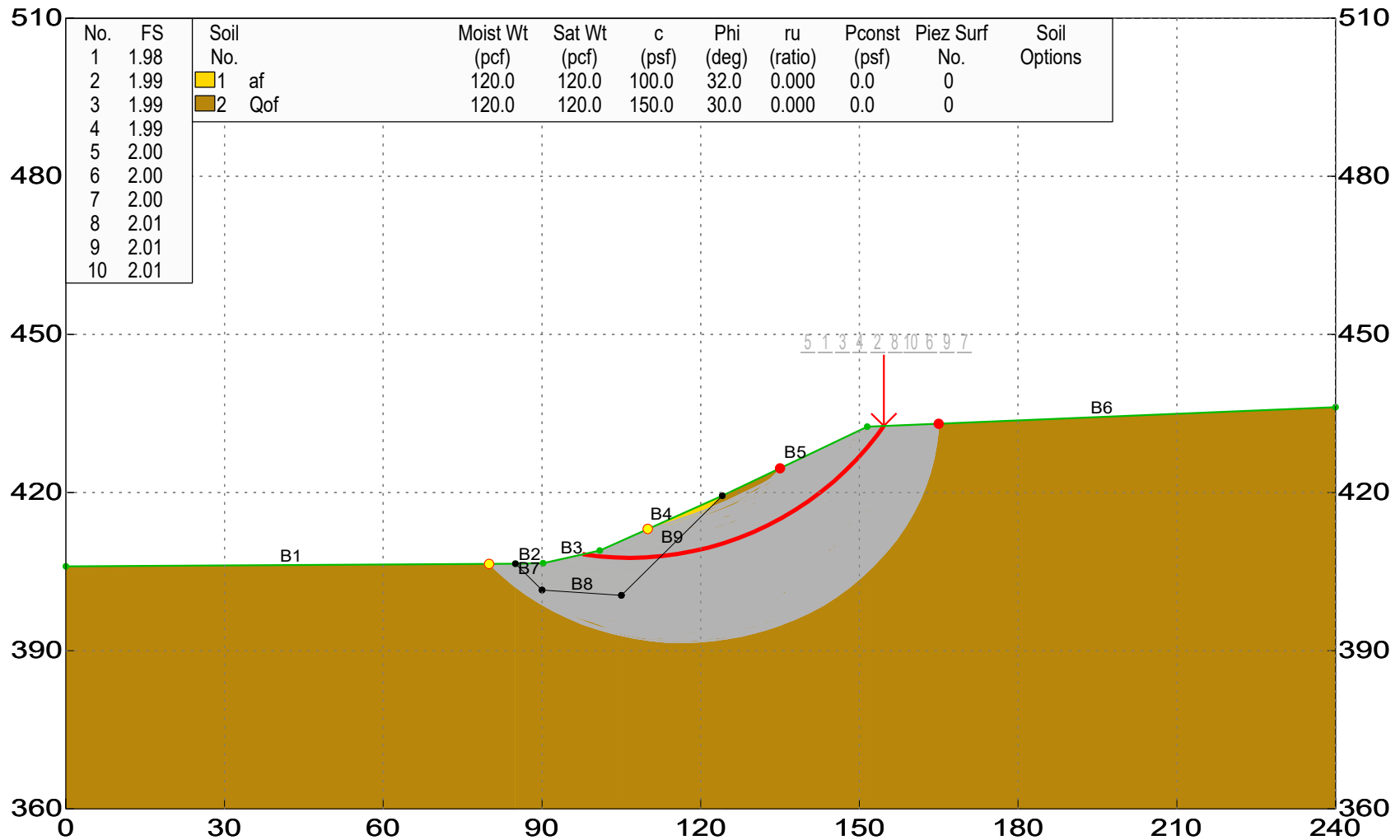
kh = 0.15000

Milan - Arena Site, Orange

Section C-C' (Static)

LGC Geotechnical / BPP

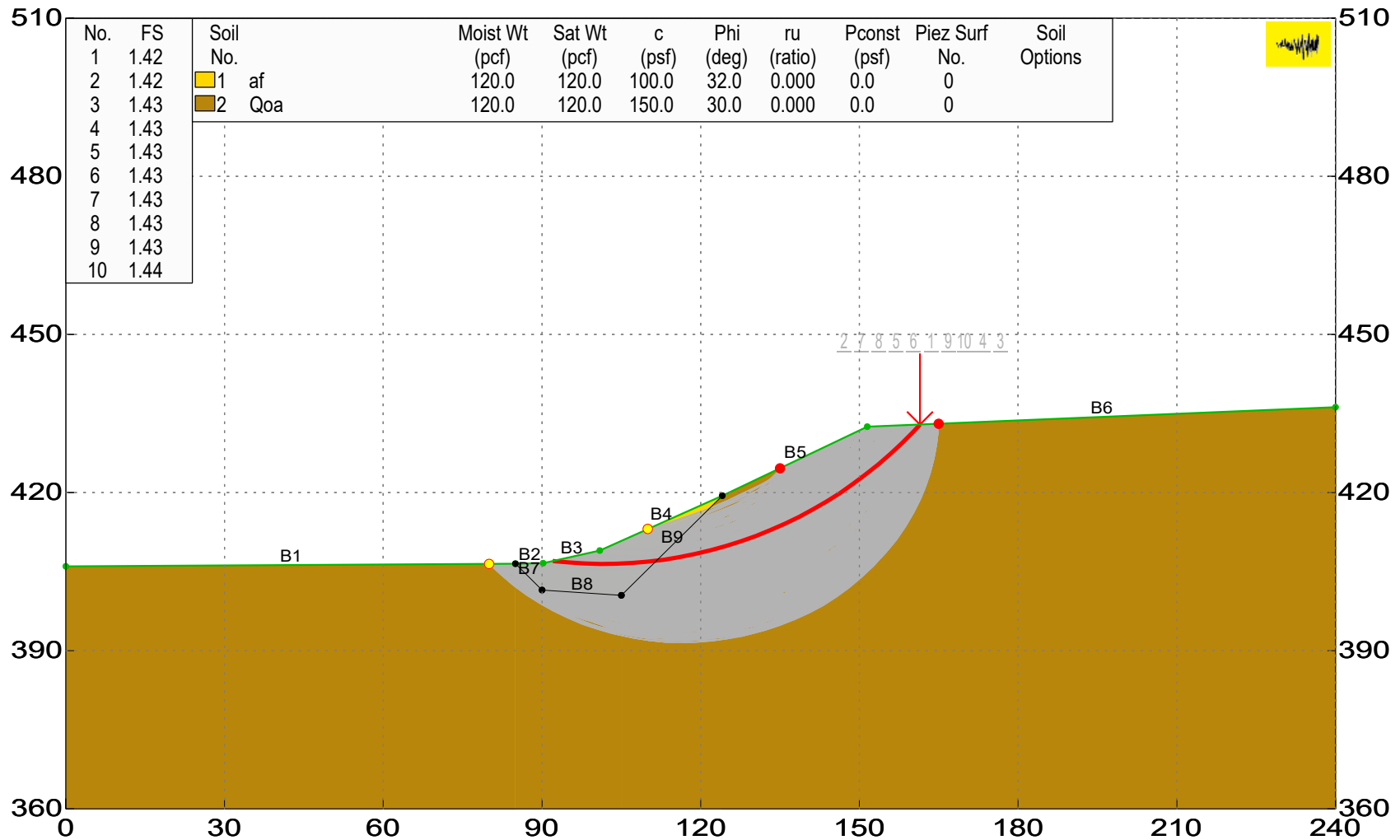
\Section C - Static.gsd



Milan - Arena Site, Orange Section C-C' (Seismic)

LGC Geotechnical / BPP

\Section C - Seismic.gsd



GEOSTASE FS = 1.42

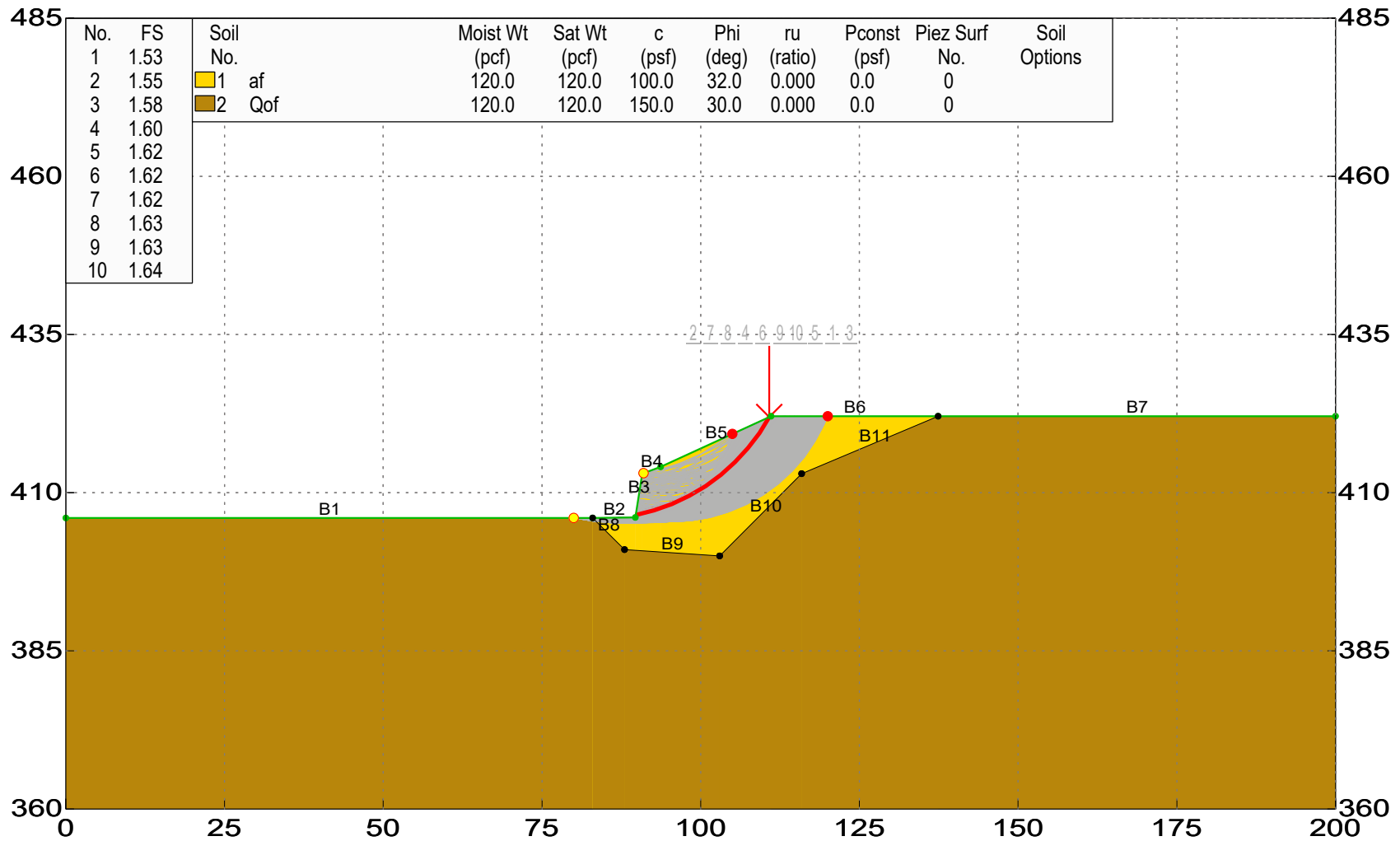
Simplified Bishop Method

kh = 0.15000

Milan - Arena Site, Orange Section D-D' (Static)

LGC Geotechnical / BPP

\Section D - Static.gsd



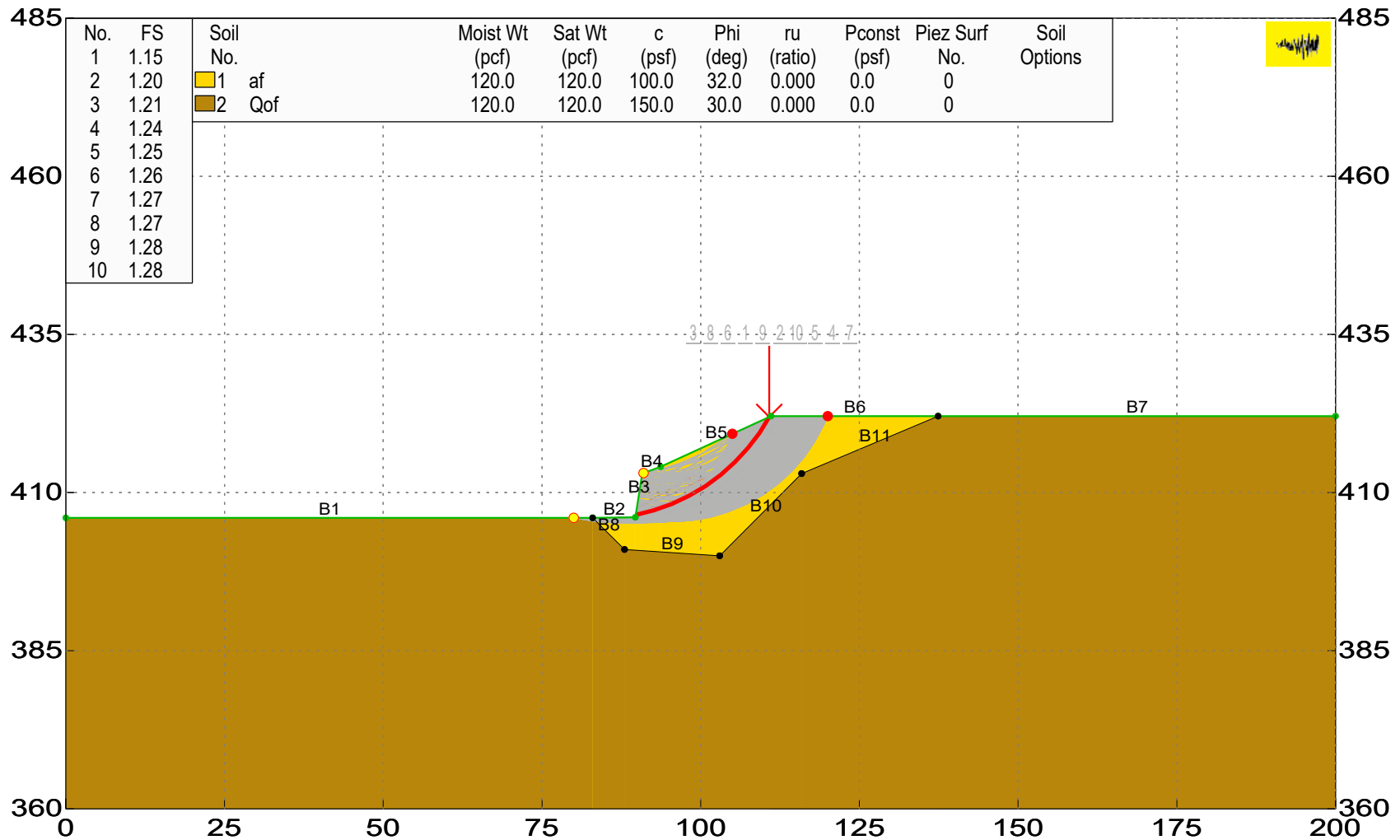
GEOSTASE FS = 1.53

Simplified Bishop Method

Milan - Arena Site, Orange Section D-D' (Seismic)

LGC Geotechnical / BPP

\Section D - Seismic.gsd



GEOSTASE FS = 1.15

Simplified Bishop Method

kh = 0.15000

Appendix F
General Earthwork and
Grading Specifications for Rough Grading

General Earthwork and Grading Specifications for Rough Grading

1.0 General

1.1 Intent

These General Earthwork and Grading Specifications are for the grading and earthwork shown on the approved grading plan(s) and/or indicated in the geotechnical report(s). These Specifications are a part of the recommendations contained in the geotechnical report(s). In case of conflict, the specific recommendations in the geotechnical report shall supersede these more general Specifications. Observations of the earthwork by the project Geotechnical Consultant during the course of grading may result in new or revised recommendations that could supersede these specifications or the recommendations in the geotechnical report(s).

1.2 The Geotechnical Consultant of Record

Prior to commencement of work, the owner shall employ a qualified Geotechnical Consultant of Record (Geotechnical Consultant). The Geotechnical Consultant shall be responsible for reviewing the approved geotechnical report(s) and accepting the adequacy of the preliminary geotechnical findings, conclusions, and recommendations prior to the commencement of the grading.

Prior to commencement of grading, the Geotechnical Consultant shall review the "work plan" prepared by the Earthwork Contractor (Contractor) and schedule sufficient personnel to perform the appropriate level of observation, mapping, and compaction testing.

During the grading and earthwork operations, the Geotechnical Consultant shall observe, map, and document the subsurface exposures to verify the geotechnical design assumptions. If the observed conditions are found to be significantly different than the interpreted assumptions during the design phase, the Geotechnical Consultant shall inform the owner, recommend appropriate changes in design to accommodate the observed conditions, and notify the review agency where required.

The Geotechnical Consultant shall observe the moisture-conditioning and processing of the subgrade and fill materials and perform relative compaction testing of fill to confirm that the attained level of compaction is being accomplished as specified. The Geotechnical Consultant shall provide the test results to the owner and the Contractor on a routine and frequent basis.

1.3 The Earthwork Contractor

The Earthwork Contractor (Contractor) shall be qualified, experienced, and knowledgeable in earthwork logistics, preparation and processing of ground to receive fill, moisture-conditioning and processing of fill, and compacting fill. The Contractor shall review and accept the plans, geotechnical report(s), and these Specifications prior to commencement of grading. The Contractor shall be solely responsible for performing the grading in accordance with the project plans and specifications. The Contractor shall prepare and submit to the owner and the Geotechnical Consultant a work plan that indicates the sequence of earthwork grading, the number of "equipment" of work and the estimated quantities of daily earthwork

contemplated for the site prior to commencement of grading. The Contractor shall inform the owner and the Geotechnical Consultant of changes in work schedules and updates to the work plan at least 24 hours in advance of such changes so that appropriate personnel will be available for observation and testing. The Contractor shall not assume that the Geotechnical Consultant is aware of all grading operations.

The Contractor shall have the sole responsibility to provide adequate equipment and methods to accomplish the earthwork in accordance with the applicable grading codes and agency ordinances, these Specifications, and the recommendations in the approved geotechnical report(s) and grading plan(s). If, in the opinion of the Geotechnical Consultant, unsatisfactory conditions, such as unsuitable soil, improper moisture condition, inadequate compaction, insufficient buttress key size, adverse weather, etc., are resulting in a quality of work less than required in these specifications, the Geotechnical Consultant shall reject the work and may recommend to the owner that construction be stopped until the conditions are rectified. It is the contractor's sole responsibility to provide proper fill compaction.

2.0 Preparation of Areas to be Filled

2.1 Clearing and Grubbing

Vegetation, such as brush, grass, roots, and other deleterious material shall be sufficiently removed and properly disposed of in a method acceptable to the owner, governing agencies, and the Geotechnical Consultant.

The Geotechnical Consultant shall evaluate the extent of these removals depending on specific site conditions. Earth fill material shall not contain more than 1 percent of organic materials (by volume). Nesting of the organic materials shall not be allowed.

If potentially hazardous materials are encountered, the Contractor shall stop work in the affected area, and a hazardous material specialist shall be informed immediately for proper evaluation and handling of these materials prior to continuing to work in that area.

As presently defined by the State of California, most refined petroleum products (gasoline, diesel fuel, motor oil, grease, coolant, etc.) have chemical constituents that are considered to be hazardous waste. As such, the indiscriminate dumping or spillage of these fluids onto the ground may constitute a misdemeanor, punishable by fines and/or imprisonment, and shall not be allowed. The contractor is responsible for all hazardous waste relating to his work. The Geotechnical Consultant does not have expertise in this area. If hazardous waste is a concern, then the Client should acquire the services of a qualified environmental assessor.

2.2 Processing

Existing ground that has been declared satisfactory for support of fill by the Geotechnical Consultant shall be scarified to a minimum depth of 6 inches. Existing ground that is not satisfactory shall be over-excavated as specified in the following section. Scarification shall continue until soils are broken down and free of oversize material and the working surface is reasonably uniform, flat, and free of uneven features that would inhibit uniform compaction.

2.3 Over-excavation

In addition to removals and over-excavations recommended in the approved geotechnical report(s) and the grading plan, soft, loose, dry, saturated, spongy, organic-rich, highly fractured or otherwise unsuitable ground shall be over-excavated to competent ground as evaluated by the Geotechnical Consultant during grading.

2.4 Benching

Where fills are to be placed on ground with slopes steeper than 5:1 (horizontal to vertical units), the ground shall be stepped or benched. Please see the Standard Details for a graphic illustration. The lowest bench or key shall be a minimum of 15 feet wide and at least 2 feet deep, into competent material as evaluated by the Geotechnical Consultant. Other benches shall be excavated a minimum height of 4 feet into competent material or as otherwise recommended by the Geotechnical Consultant. Fill placed on ground sloping flatter than 5:1 shall also be benched or otherwise over-excavated to provide a flat subgrade for the fill.

2.5 Evaluation/Acceptance of Fill Areas

All areas to receive fill, including removal and processed areas, key bottoms, and benches, shall be observed, mapped, elevations recorded, and/or tested prior to being accepted by the Geotechnical Consultant as suitable to receive fill. The Contractor shall obtain a written acceptance from the Geotechnical Consultant prior to fill placement. A licensed surveyor shall provide the survey control for determining elevations of processed areas, keys, and benches.

3.0 Fill Material

3.1 General

Material to be used as fill shall be essentially free of organic matter and other deleterious substances evaluated and accepted by the Geotechnical Consultant prior to placement. Soils of poor quality, such as those with unacceptable gradation, high expansion potential, or low strength shall be placed in areas acceptable to the Geotechnical Consultant or mixed with other soils to achieve satisfactory fill material.

3.2 Oversize

Oversize material defined as rock, or other irreducible material with a maximum dimension greater than 8 inches, shall not be buried or placed in fill unless location, materials, and placement methods are specifically accepted by the Geotechnical Consultant. Placement operations shall be such that nesting of oversized material does not occur and such that oversize material is completely surrounded by compacted or densified fill. Oversize material shall not be placed within 10 vertical feet of finish grade or within 2 feet of future utilities or underground construction.

3.3 Import

If importing of fill material is required for grading, proposed import material shall meet the requirements of the geotechnical consultant. The potential import source shall be given to the Geotechnical Consultant at least 48 hours (2 working days) before importing begins so that its suitability can be determined and appropriate tests performed.

4.0 Fill Placement and Compaction

4.1 Fill Layers

Approved fill material shall be placed in areas prepared to receive fill (per Section 3.0) in near-horizontal layers not exceeding 8 inches in loose thickness. The Geotechnical Consultant may accept thicker layers if testing indicates the grading procedures can adequately compact the thicker layers. Each layer shall be spread evenly and mixed thoroughly to attain relative uniformity of material and moisture throughout.

4.2 Fill Moisture Conditioning

Fill soils shall be watered, dried back, blended, and/or mixed, as necessary to attain a relatively uniform moisture content at or slightly over optimum. Maximum density and optimum soil moisture content tests shall be performed in accordance with the American Society of Testing and Materials (ASTM Test Method D1557).

4.3 Compaction of Fill

After each layer has been moisture-conditioned, mixed, and evenly spread, it shall be uniformly compacted to not less than 90 percent of maximum dry density (ASTM Test Method D1557). Compaction equipment shall be adequately sized and be either specifically designed for soil compaction or of proven reliability to efficiently achieve the specified level of compaction with uniformity.

4.4 Compaction of Fill Slopes

In addition to normal compaction procedures specified above, compaction of slopes shall be accomplished by backrolling of slopes with sheepfoot rollers at increments of 3 to 4 feet in fill elevation, or by other methods producing satisfactory results acceptable to the Geotechnical Consultant. Upon completion of grading, relative compaction of the fill, out to the slope face, shall be at least 90 percent of maximum density per ASTM Test Method D1557.

4.5 Compaction Testing

Field tests for moisture content and relative compaction of the fill soils shall be performed by the Geotechnical Consultant. Location and frequency of tests shall be at the Consultant's discretion based on field conditions encountered. Compaction test locations will not necessarily be selected on a random basis. Test locations shall be selected to verify adequacy of compaction levels in areas that are judged to be prone to inadequate compaction (such as close to slope faces and at the fill/bedrock benches).

4.6 Frequency of Compaction Testing

Tests shall be taken at intervals not exceeding 2 feet in vertical rise and/or 1,000 cubic yards of compacted fill soils embankment. In addition, as a guideline, at least one test shall be taken on slope faces for each 5,000 square feet of slope face and/or each 10 feet of vertical height of slope. The Contractor shall assure that fill construction is such that the testing schedule can be accomplished by the Geotechnical Consultant. The Contractor shall stop or slow down the earthwork construction if these minimum standards are not met.

4.7 Compaction Test Locations

The Geotechnical Consultant shall document the approximate elevation and horizontal coordinates of each test location. The Contractor shall coordinate with the project surveyor to assure that sufficient grade stakes are established so that the Geotechnical Consultant can determine the test locations with sufficient accuracy. At a minimum, two grade stakes within a horizontal distance of 100 feet and vertically less than 5 feet apart from potential test locations shall be provided.

5.0 Subdrain Installation

Subdrain systems shall be installed in accordance with the approved geotechnical report(s), the grading plan, and the Standard Details. The Geotechnical Consultant may recommend additional subdrains and/or changes in subdrain extent, location, grade, or material depending on conditions encountered during grading. All subdrains shall be surveyed by a land surveyor/civil engineer for line and grade after installation and prior to burial. Sufficient time should be allowed by the Contractor for these surveys.

6.0 Excavation

Excavations, as well as over-excavation for remedial purposes, shall be evaluated by the Geotechnical Consultant during grading. Remedial removal depths shown on geotechnical plans are estimates only. The actual extent of removal shall be determined by the Geotechnical Consultant based on the field evaluation of exposed conditions during grading. Where fill-over-cut slopes are to be graded, the cut portion of the slope shall be made, evaluated, and accepted by the Geotechnical Consultant prior to placement of materials for construction of the fill portion of the slope, unless otherwise recommended by the Geotechnical Consultant.

7.0 Trench Backfills

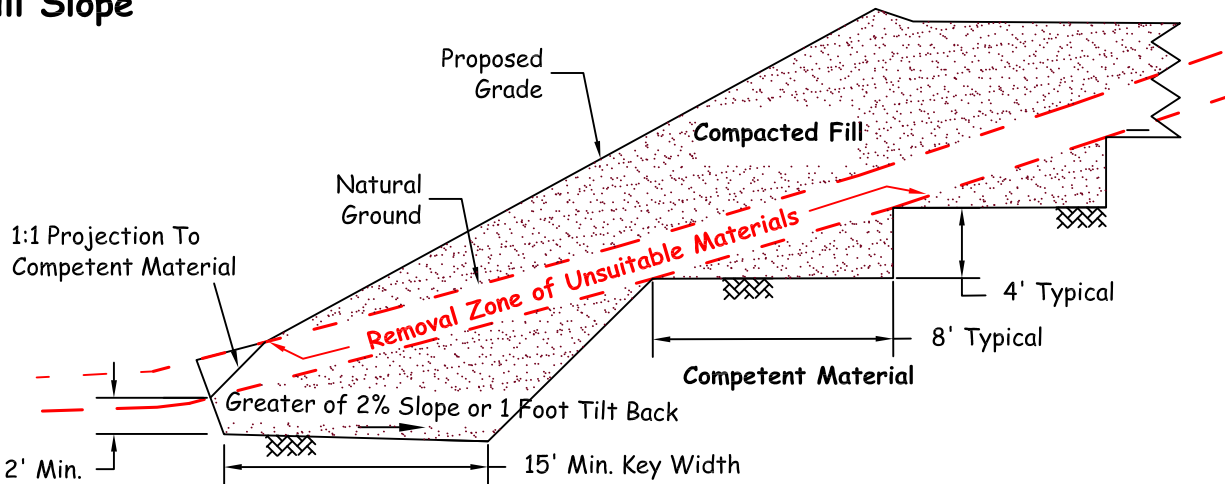
7.1 The Contractor shall follow all OHSA and Cal/OSHA requirements for safety of trench excavations.

7.2 All bedding and backfill of utility trenches shall be done in accordance with the applicable provisions of Standard Specifications of Public Works Construction. Bedding material shall have a Sand Equivalent greater than 30 (SE>30). The bedding shall be placed to 1 foot over

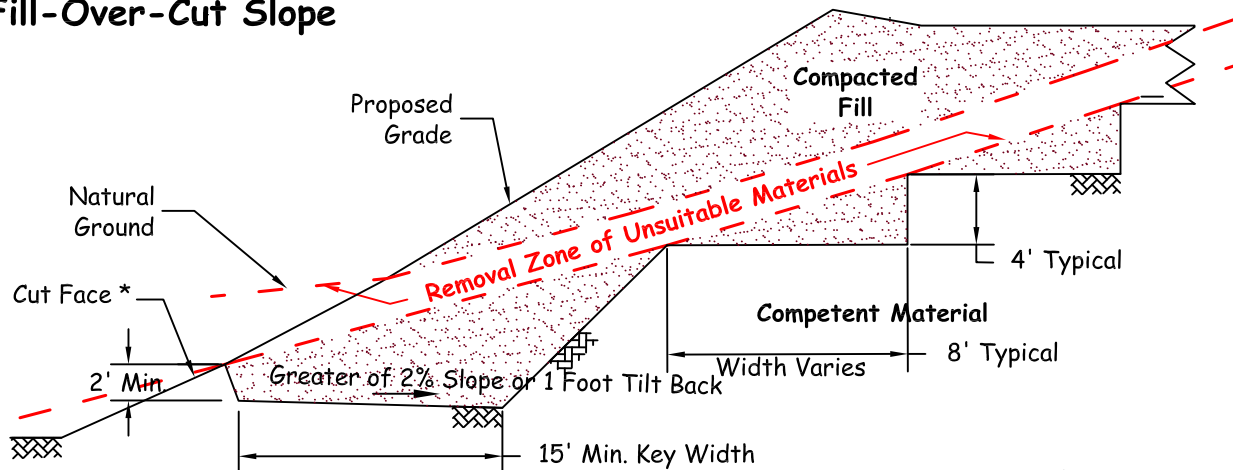
the top of the conduit and densified by jetting. Backfill shall be placed and densified to a minimum of 90 percent of maximum from 1 foot above the top of the conduit to the surface.

- 7.3** The jetting of the bedding around the conduits shall be observed by the Geotechnical Consultant.
- 7.4** The Geotechnical Consultant shall test the trench backfill for relative compaction. At least one test should be made for every 300 feet of trench and 2 feet of fill.
- 7.5** Lift thickness of trench backfill shall not exceed those allowed in the Standard Specifications of Public Works Construction unless the Contractor can demonstrate to the Geotechnical Consultant that the fill lift can be compacted to the minimum relative compaction by his alternative equipment and method.

Fill Slope

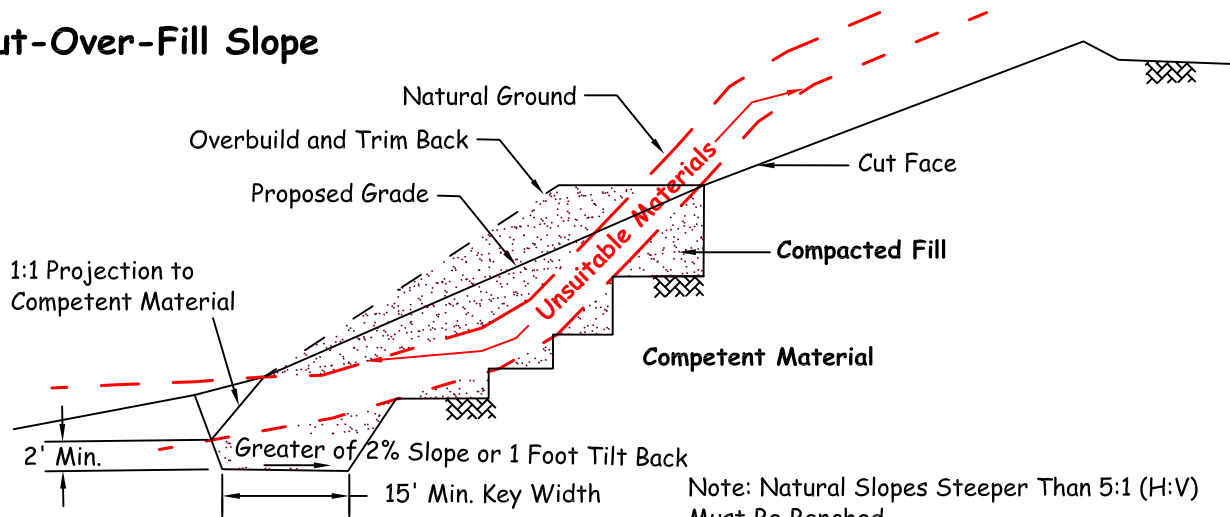


Fill-Over-Cut Slope

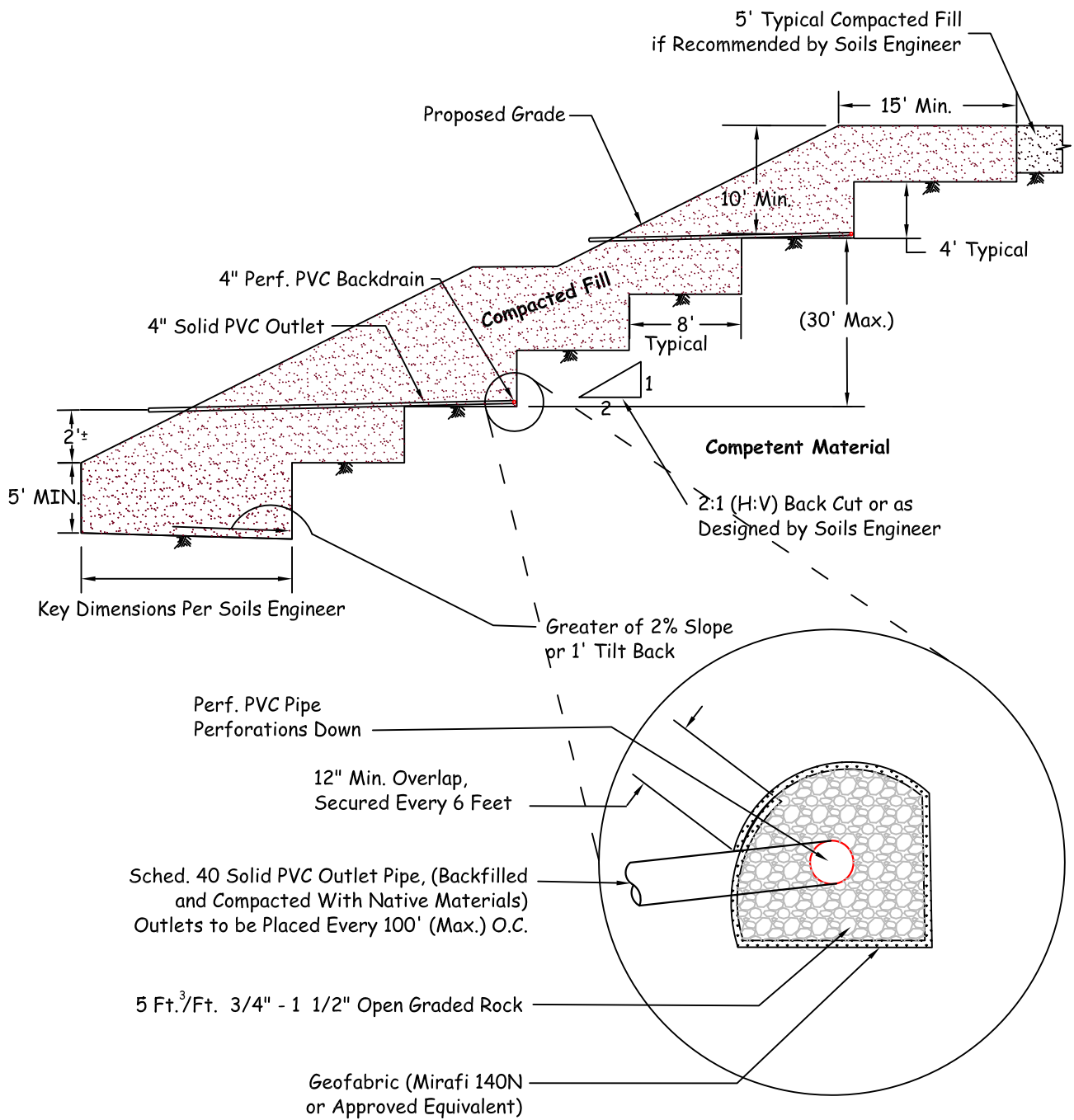


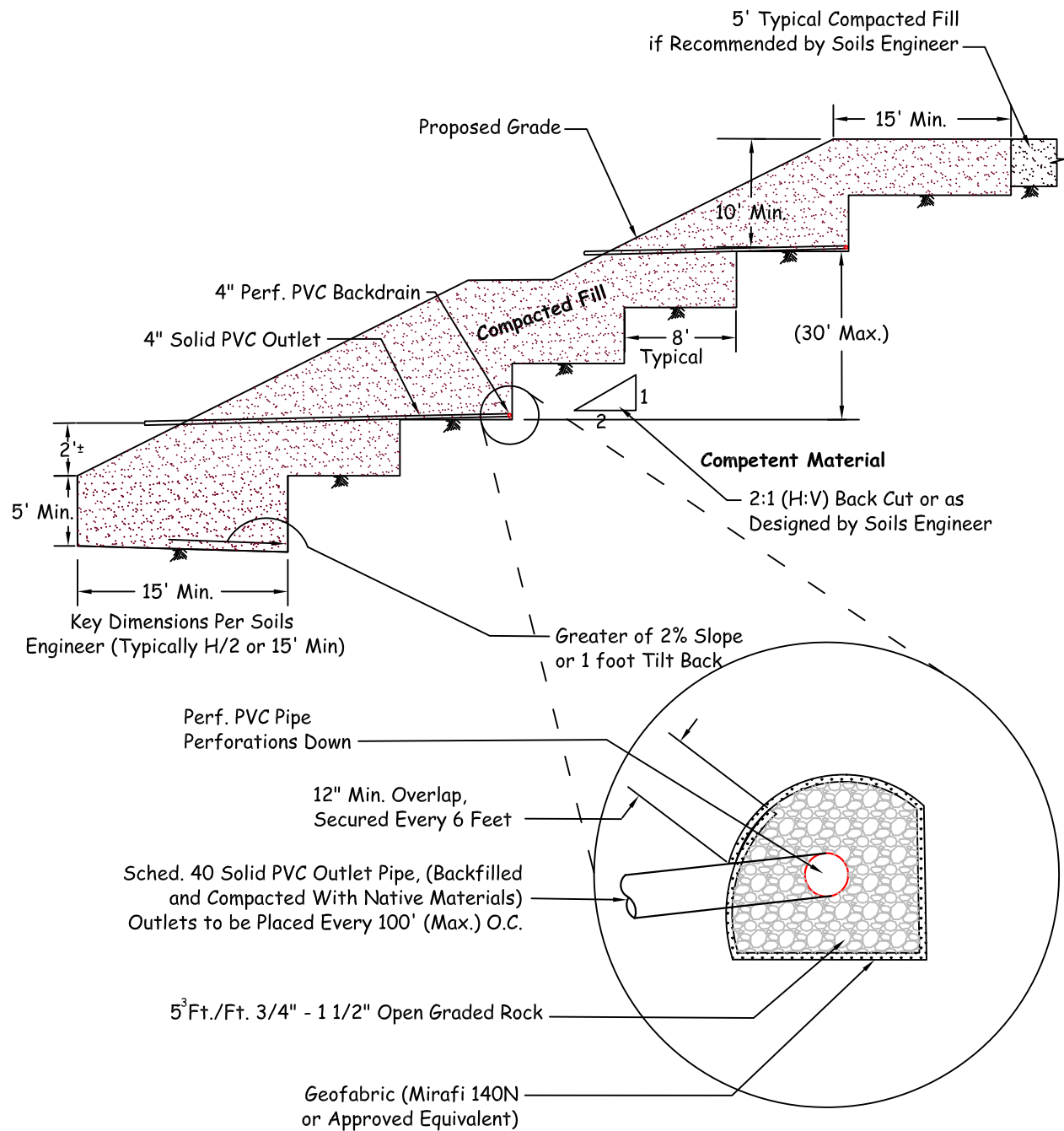
* Construct Cut Slope First

Cut-Over-Fill Slope

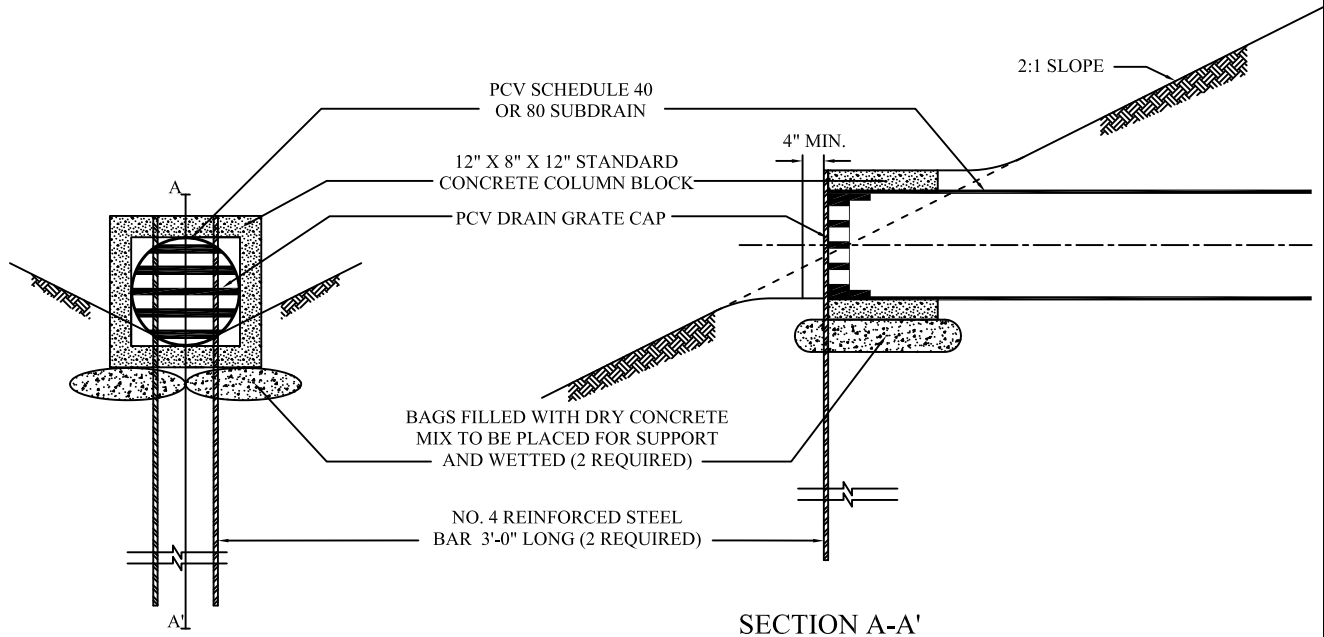


Note: Natural Slopes Steeper Than 5:1 (H:V) Must Be Benched.

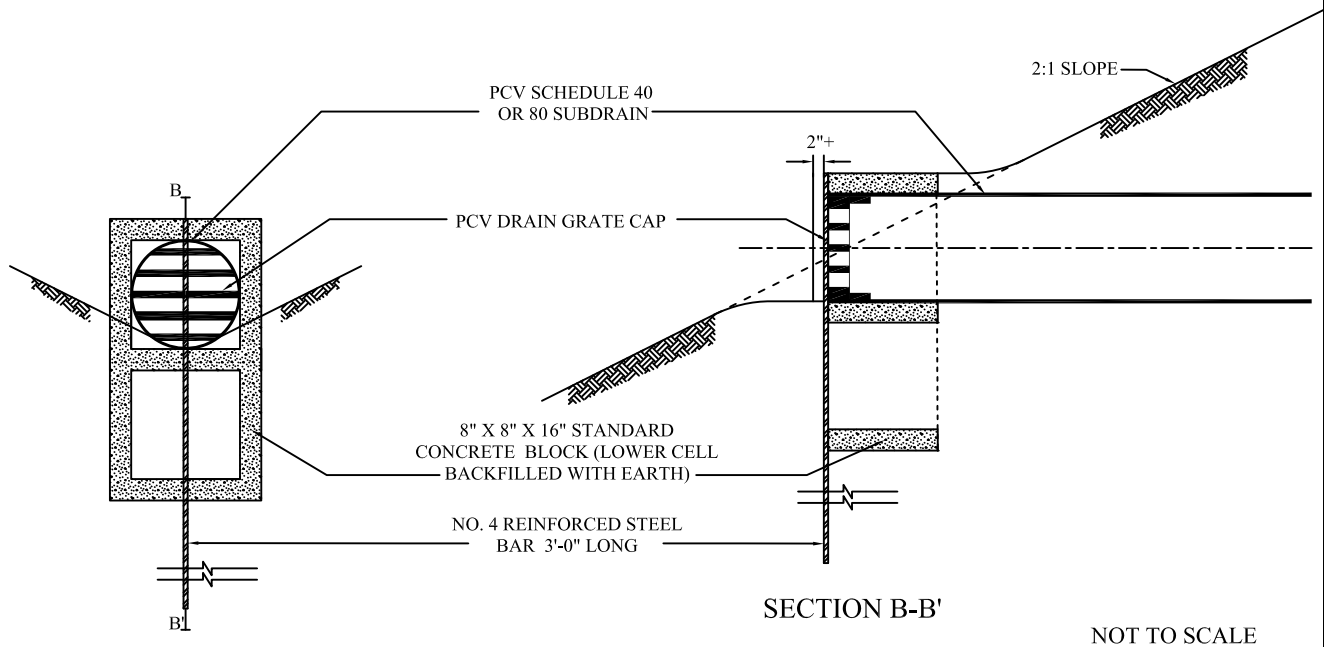




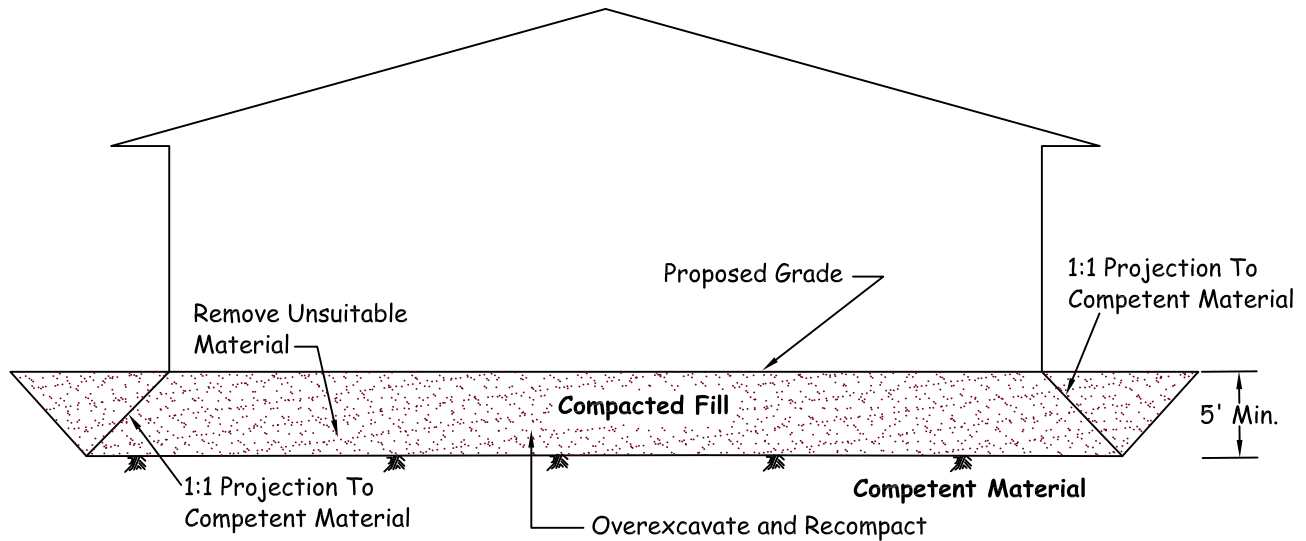
SUBDRAIN OUTLET MARKER -6" & 8" PIPE



SUBDRAIN OUTLET MARKER -4" PIPE



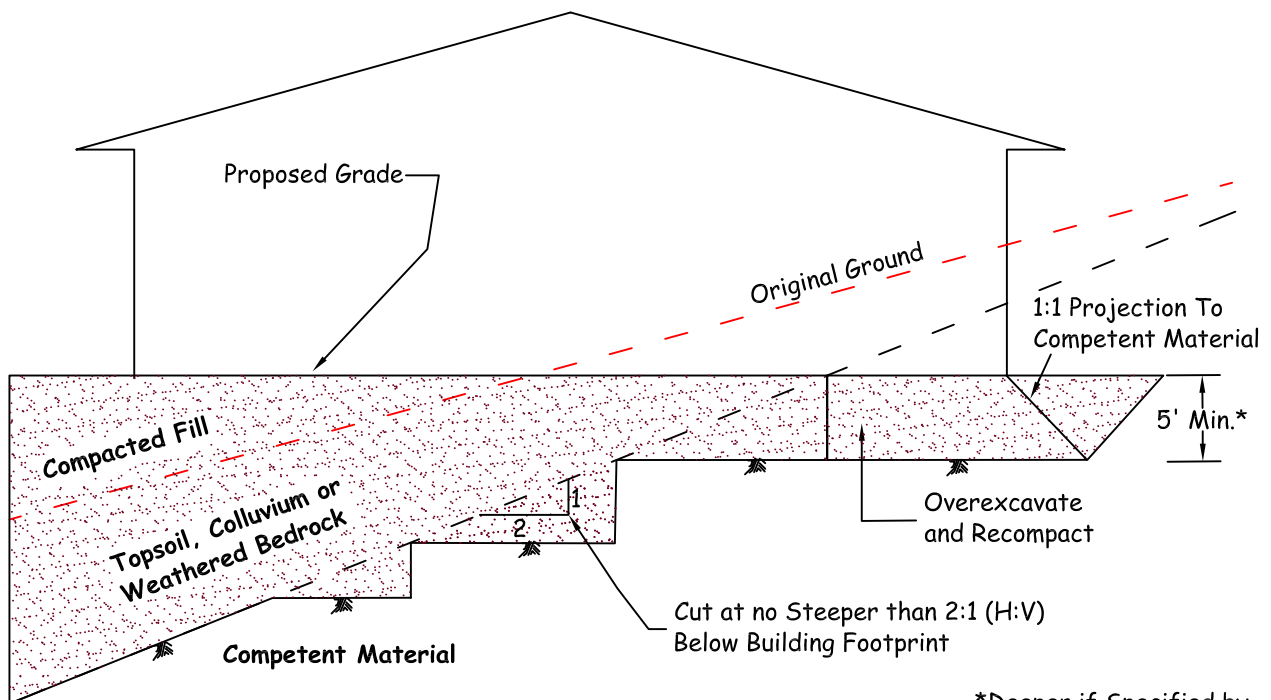
Cut Lot (Exposing Unsuitable Soils at Design Grade)



Note 1: Removal Bottom Should be Graded With Minimum 2% Fall Towards Street or Other Suitable Area (as Determined by Soils Engineer) to Avoid Ponding Below Building

Note 2: Where Design Cut Lots are Excavated Entirely Into Competent Material, Overexcavation May Still be Required for Hard-Rock Conditions or for Materials With Variable Expansion Characteristics.

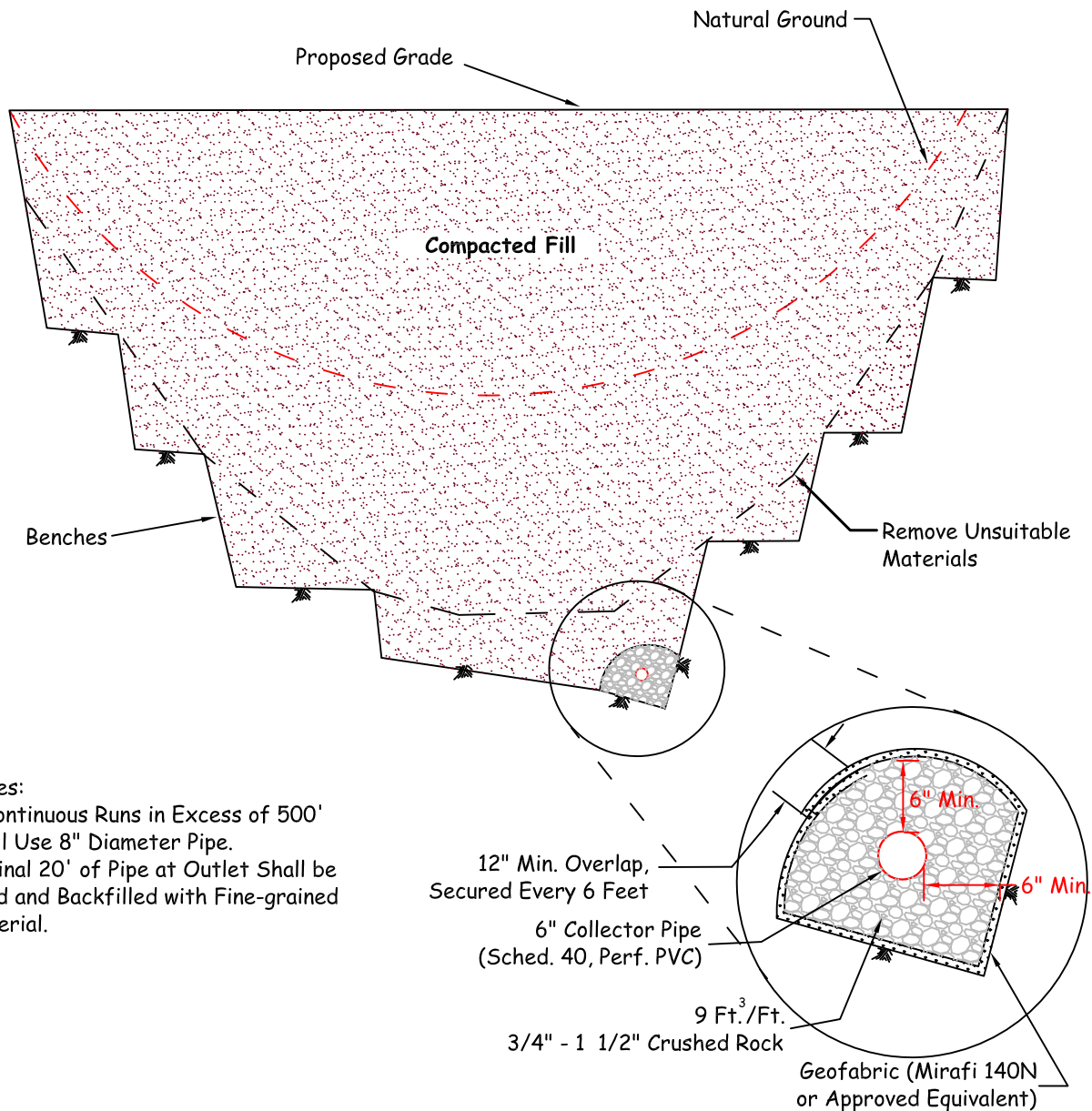
Cut/Fill Transition Lot



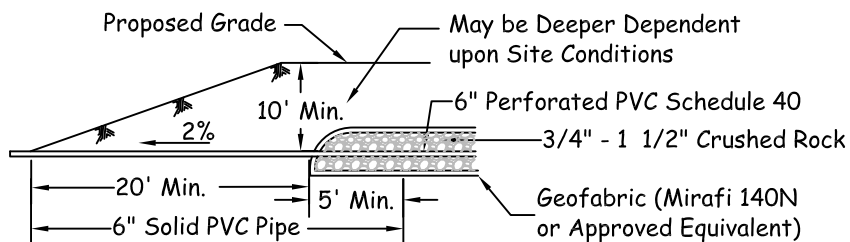
*Deeper if Specified by Soils Engineer

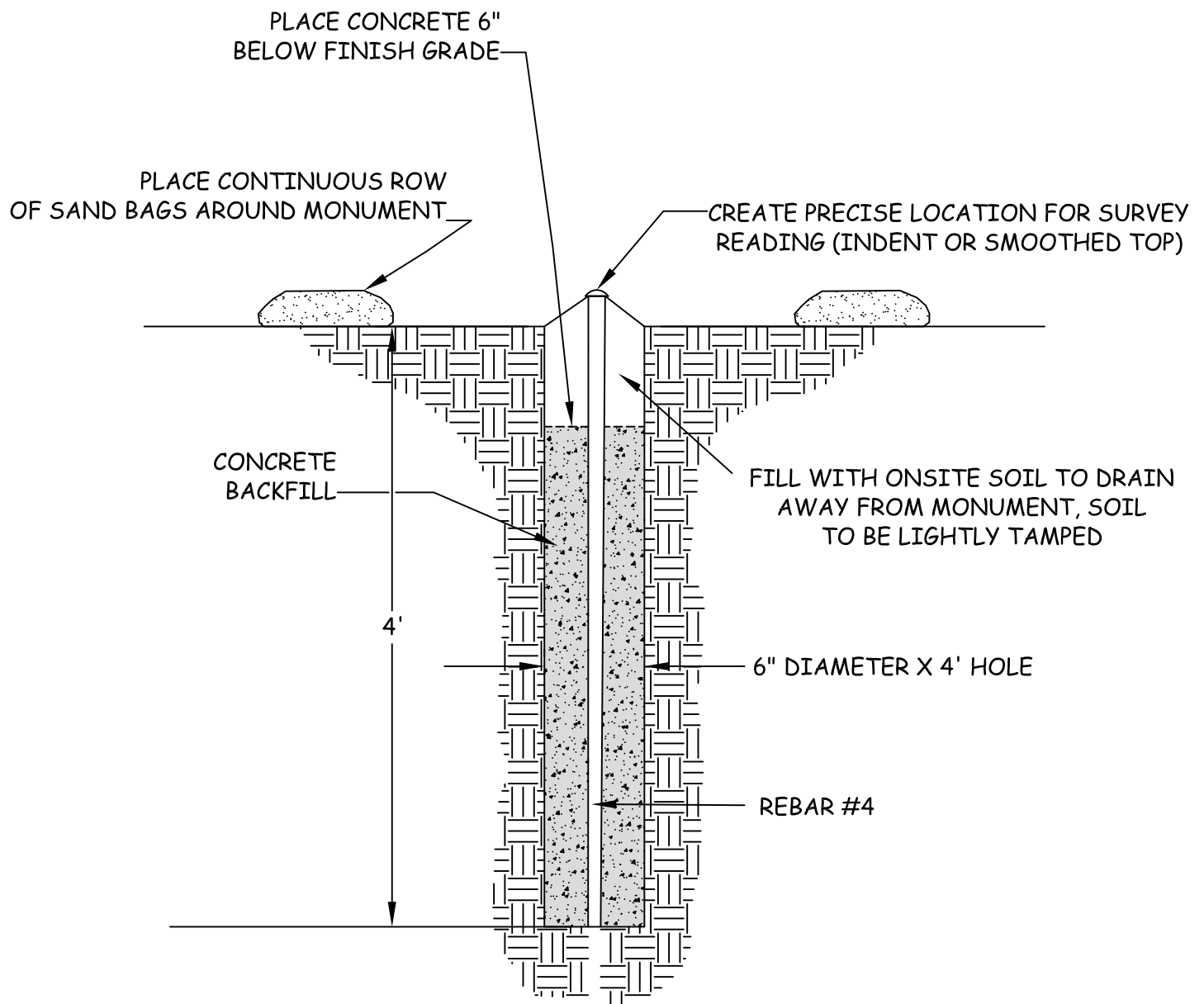


CUT AND TRANSITION LOT OVEREXCAVATION DETAIL



Proposed Outlet Detail

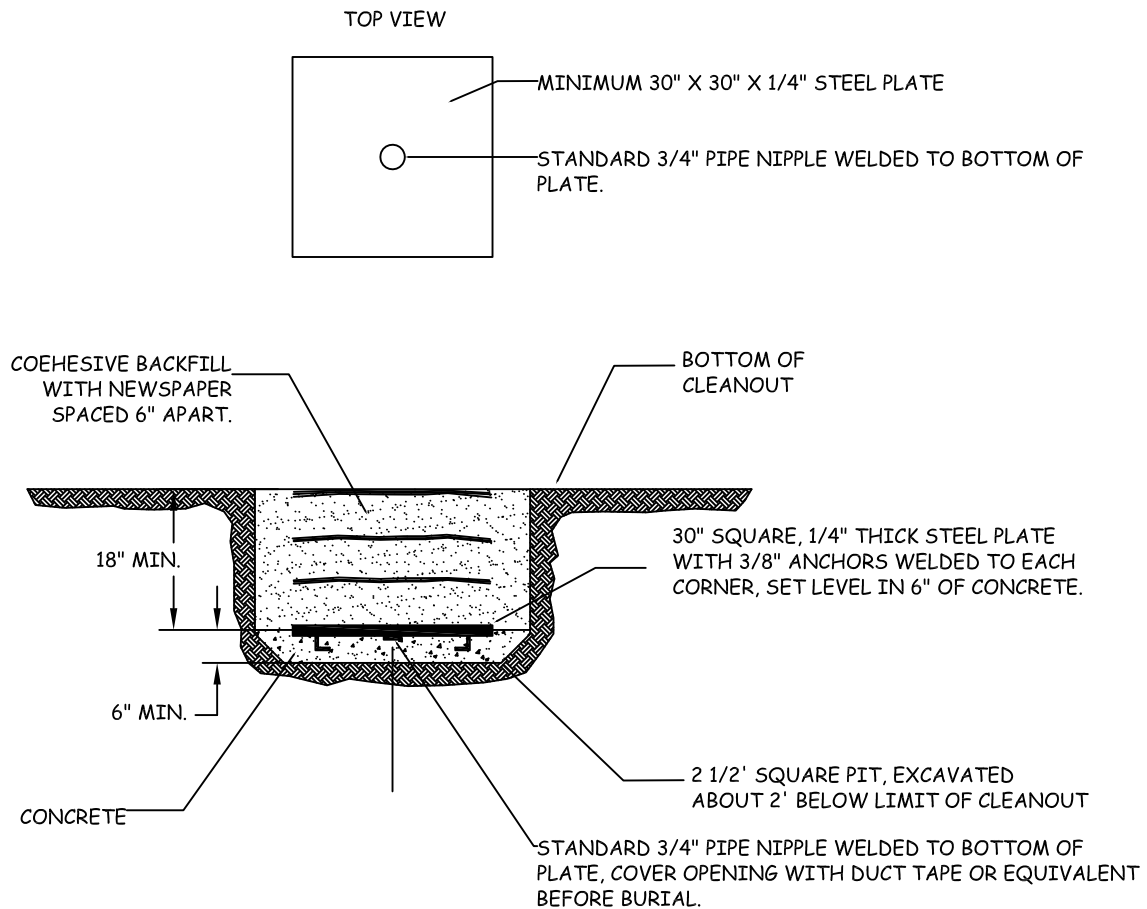




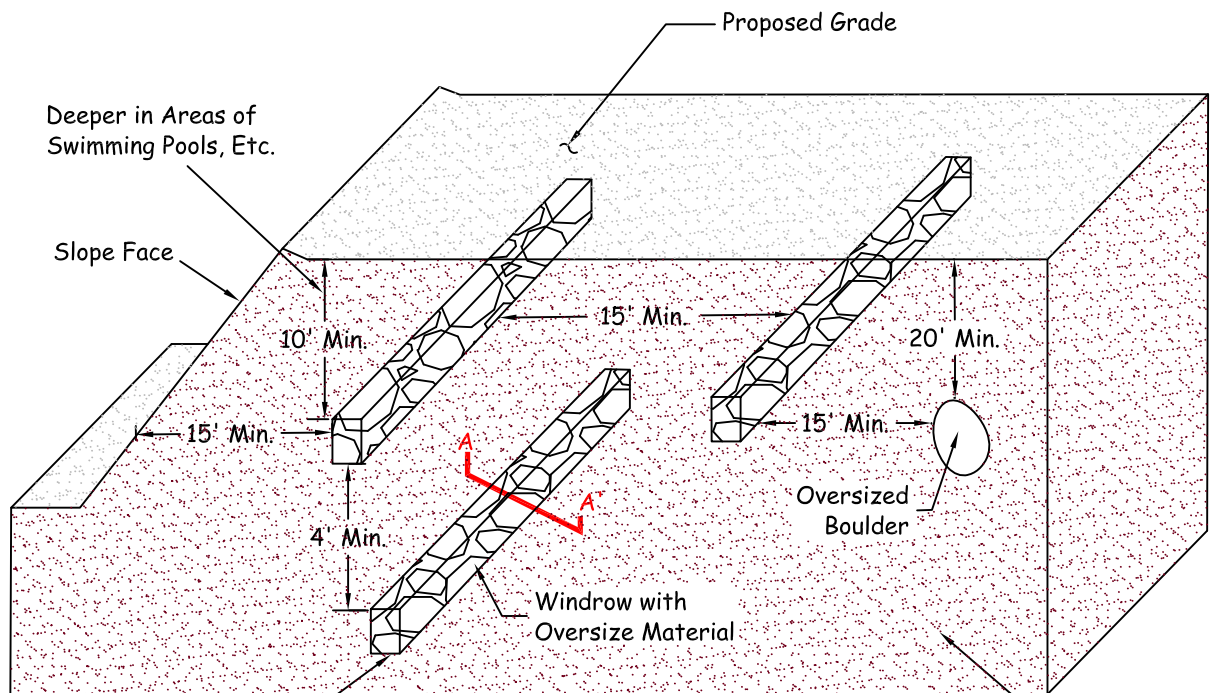
NO CONSTRUCTION EQUIPMENT WITHIN 25 FEET
OF ANY INSTALLED SETTLEMENT MONUMENTS



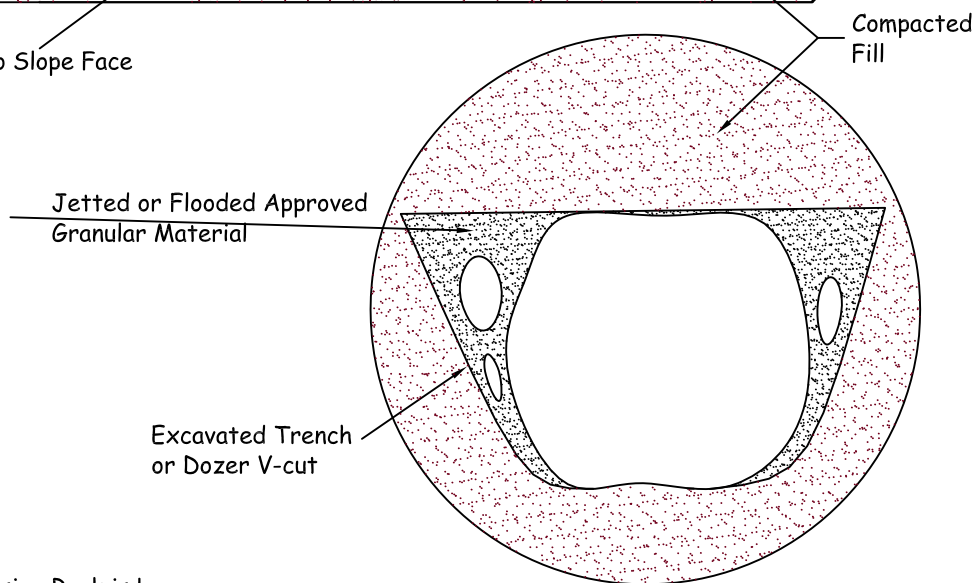
TYPICAL SURFACE SETTLEMENT MONUMENT



1. SURVEY FOR HORIZONTAL AND VERTICAL LOCATION TO NEAREST .01 INCH PRIOR TO BACKFILL USING KNOW LOCATIONS THAT WILL REMAIN INTACT DURING THE DURATION OF THE MONITORING PROGRAM. KNOW POINTS EXPLICITLY NOT ALLOWED ARE THOSE LOCATED ON FILL OR THAT WILL BE DESTROYED DURING GRADING.
2. IN THE EVENT OF DAMAGE TO SETTLEMENT PLATE DURING GRADING, CONTRACTOR SHALL IMMEDIATELY NOTIFY THE GEOTECHNICAL ENGINEER AND SHALL BE RESPONSIBLE FOR RESTORING THE SETTLEMENT PLATES TO WORKING ORDER.
3. DRILL TO RECOVER AND ATTACH RISER PIPE.

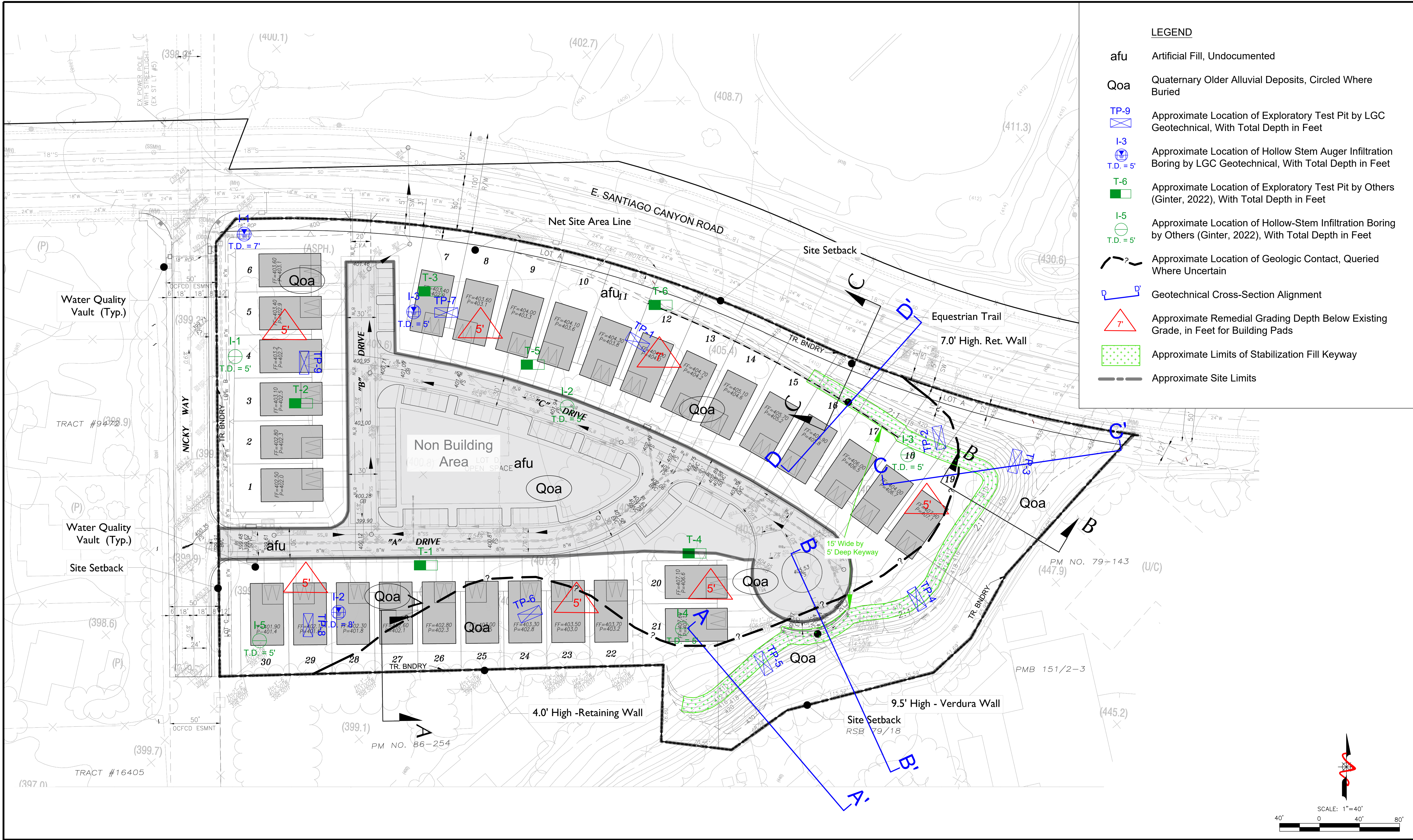


Windrow Parallel to Slope Face



Note: Oversize Rock is Larger than 8" in Maximum Dimension.

Section A-A'



LEGEND

- afu Artificial Fill, Undocumented
- Qoa Quaternary Older Alluvial Deposits, Circled Where Buried
- TP-9 Approximate Location of Exploratory Test Pit by LGC Geotechnical, With Total Depth in Feet
- I-3 Approximate Location of Hollow Stem Auger Infiltration Boring by LGC Geotechnical, With Total Depth in Feet
- T-6 Approximate Location of Exploratory Test Pit by Others (Ginter, 2022), With Total Depth in Feet
- I-5 Approximate Location of Hollow-Stem Infiltration Boring by Others (Ginter, 2022), With Total Depth in Feet
- Approximate Location of Geologic Contact, Queried Where Uncertain
- Geotechnical Cross-Section Alignment
- Approximate Remedial Grading Depth Below Existing Grade, in Feet for Building Pads
- Approximate Limits of Stabilization Fill Keyway
- Approximate Site Limits



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Geotechnical Map

PROJECT NAME	Milan Capital - Arena Site, Orange		
PROJECT NO.	25046-01		
ENG. / GEOL.	DJB / KTM		SHEET 1 of 1
SCALE	1" = 40'		
DATE	July 2025		

D-2: Supplemental Infiltration Recommendations

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March 4, 2026

Project No. 25046-01

Ms. Diane Scioli
Milan Capital REI X, LLC
710 South Parker Street, Suite 5200
Orange, CA 92868

Subject: Supplemental Infiltration Recommendations for the Proposed Residential Development, Arena Site, Southeast of Santiago Canyon Road and Nicky Way, City of Orange, California

References: LGC Geotechnical, Inc., 2025, Preliminary Geotechnical Subsurface Evaluation for the Proposed Residential Development, Southeast of Santiago Canyon Road and Nicky Way, City of Orange, California, Project No. 25046-01, dated July 30, 2025.

County of Orange, 2013, Technical Guidance Document (TGD) for the Preparation of Conceptual and/Preliminary and/or Project Water Quality Management Plans (WQMPs).

Introduction

In accordance with your request, LGC Geotechnical, Inc. (LGC Geotechnical) has provided supplemental infiltration recommendations for the proposed residential development, Arena Site, located southeast of Santiago Canyon Road and Nicky Way in the City of Orange, California (Figure 1 – Site Location Map). We previously performed infiltration testing (LGC Geotechnical, 2025) that included 3 infiltration tests.

Subsurface Geotechnical Evaluation

LGC Geotechnical previously performed a subsurface geotechnical evaluation of the site consisting of the excavation of 3 hollow-stem auger infiltration borings.

Three exploratory hollow-stem auger infiltration borings (I-1 through I-3) were excavated using a truck-mounted drill rig equipped with 8-inch-diameter hollow-stem augers to depths ranging from approximately 5 feet to 8 feet below existing grade. An LGC Geotechnical representative observed the drilling operations, logged the borings, and collected soil samples for laboratory testing. Driven soil samples were collected by means of the Modified California Drive (MCD) sampler. The MCD sampler (2.4-inch ID, 3.0-inch OD) was driven using a 140-pound automatic hammer falling 30 inches to advance the sampler a total depth of 18 inches or until refusal. Bulk samples were also collected and logged for laboratory testing at select depths. The raw blow counts for each 6-inch increment of penetration were recorded on the boring logs. The borings were backfilled with cuttings. At the

completion of infiltration testing, the perforated pipe was removed, and the resulting void backfilled with the previously excavated soils.

The approximate locations of the infiltration borings are shown on the Geotechnical Map (Sheet 1). Infiltration boring logs are presented in Appendix A.

Field Percolation Testing

Three falling head field infiltration tests (I-1 through I-3) were performed to an approximate depth ranging from 5 to 8 feet below existing grade. Estimation of infiltration rates for the site was accomplished in general accordance with the guidelines set forth by Orange County (2013). A 3-inch diameter perforated PVC pipe with filter sock was placed in the borehole, and the annulus was backfilled with gravel, including placement of approximately 2 inches of gravel at the bottom of the borehole. The infiltration wells were pre-soaked the day prior to testing. During the pre-test, if the water level drops more than 6 inches in 25 minutes for two consecutive readings, the test procedure for coarse-grained soils was followed. If the water level did not drop 6 inches in one or both pre-test readings, the procedure for fine-grained soils was followed. The procedure for coarse-grained soils requires performing the test for one hour and taking one reading every 10 minutes from a fixed reference point. The procedure for fine-grained soils requires performing the test for six hours and taking one reading every 30 minutes from a fixed reference point.

The pre-tests indicated the procedure for the fine-grained soils should be followed for two of the tests and coarse-grained for one of the tests. The calculated infiltration is normalized relative to the three-dimensional flow that occurs within the field test to a one-dimensional flow out of the bottom of the boring only (i.e., "Porchet Method"). The measured infiltration rates (for feasibility purposes only) are provided in Table 1 below. Please note that infiltration is discussed below and field data is provided in Appendix C.

TABLE 1

Summary of Infiltration Testing

Infiltration Test Location	Infiltration Test Depth Below Existing Grade (ft)	Observed Infiltration Rate* (Inch/Hr.)
I-1	7	1.5
I-2	8	0.3
I-3	5	1.5

*Normalized to One-Dimensional Flow, does not include any Factor of Safety

It should be emphasized that infiltration test results are only representative of the location and depth where they are performed. Varying subsurface conditions may exist outside of the test locations which could alter the calculated infiltration rates indicated above. Infiltration tests are performed using relatively clean water free of particulates, silt, etc. Refer to the section below for subsurface water infiltration recommendations.

Laboratory Testing

Laboratory testing was performed on representative soil samples obtained from our subsurface evaluation. Laboratory testing included in-situ density and moisture tests and fines content.

The following is a summary of the laboratory test results.

- Dry density of the samples collected ranged from approximately 97 pounds per cubic foot (pcf) to 106 pcf, with an average of 101 pcf. Field moisture contents ranged from approximately 4 to 10 percent, with an average of 8 percent.
- Three samples were tested for fines content indicating a fines content (passing No. 200 sieve) ranging from 54 to 62 percent. According to the Unified Soils Classification System (USCS), the tested samples are classified as “fine grained” soil.

A summary of the results is presented in Appendix B. The moisture and dry density test results are presented on the boring logs in Appendix A.

Subsurface Infiltration Conclusion

Recent regulatory changes have occurred that mandate that storm water be infiltrated below grade into subsurface soils rather than collected in a conventional storm drain system. Typically, a combination of methods are implemented to reduce surface water runoff and increase infiltration including; permeable pavements/pavers for roadways and walkways, directing surface water runoff to grass-lined swales, retention areas, and/or drywells, etc.

It should be noted that collecting and concentrating surface water for the purpose of intentionally infiltrating below grade, conflicts with the geotechnical engineering objective of directing surface water away from slopes, structures and other improvements. The geotechnical stability and integrity of a site is reliant upon appropriately handling surface water. In general, the vast majority of geotechnical distress issues are directly related to improper drainage. In general, distress in the form of movement of improvements could occur as a result of soil saturation and loss of soil support, expansion, internal soil erosion, collapse and/or settlement.

The results of our field infiltration testing indicate the observed 1-D infiltration rate for I-1 through I-3 (not including required factors of safety for design) ranged from 0.3 to 1.5 inches per hour. The design infiltration rate is thereby equal to the Observed Infiltration Rate divided by the design factor of safety. The design factor of safety must be a minimum of 2.0 but may be increased at the discretion of the design engineer. Infiltration of stormwater is not required when the factored infiltration rate (measured infiltration rate with safety factors applied) is less than 0.3 inches per hour in the vicinity of the BMPs. The design infiltration rate is determined by dividing the observed infiltration by a series of safety factors for site suitability and design considerations that are the purview of both the geotechnical consultant and designer of the infiltration system (County of Orange, 2013). The recommended geotechnical factors of safety that are to be used to determine the design infiltration rate are provided in Table 2 on the following page.

TABLE 2

Geotechnical Factors of Safety for Design Infiltration Rate

A: Site Suitability Considerations (From Table VII.3)	
Consideration	F.S.
Soil Assessment Methods	2
Texture Class	3
Site Soil Variability	3
Depth to Groundwater/Impervious Layer	2
Calculated Suitability Assessment F.S.	2.5
B: Design Related Considerations (From Table VII.4)	
Consideration	F.S.
Tributary Size Area	Per Infiltration Designer
Level of Pretreatment	Per Infiltration Designer
Redundancy of Treatment	Per Infiltration Designer
Compaction during Construction	Per Infiltration Designer
Calculated Design F.S.	Per Infiltration Designer
Combined F.S.= Suitability F.S x Design F.S.	TBD

The results of our field infiltration testing indicate the observed 1-D infiltration rate for I-1 through I-3 (not including required factors of safety for design) ranged from 0.3 to 1.5 inches per hour. Based on our experience and fine-grained nature of the on-site soils, we recommend an observed infiltration rate of 0.3 in/hr. be utilized when determining the design infiltration rate. The factor of safety used to determine the design infiltration rate is determined by multiplying the calculated suitability assessment factor of safety of 2.5 by the design factor of safety which is to be determined by the infiltration system designer. The combined adjusted factor of safety must be a minimum of 2.0 but need not exceed 9.0

Based on the results of field infiltration testing indicating relatively low and variable infiltration rates, the very stiff/dense and fine-grained nature of the on-site soils, and hillside nature of the site, we recommend against the intentional infiltration of stormwater into the subsurface soils.

Sincerely,

LGC Geotechnical, Inc.



Dennis Boratynec, GE 2770
President



Branden Petersen, EIT
Project Engineer

DJB/BPP/amm

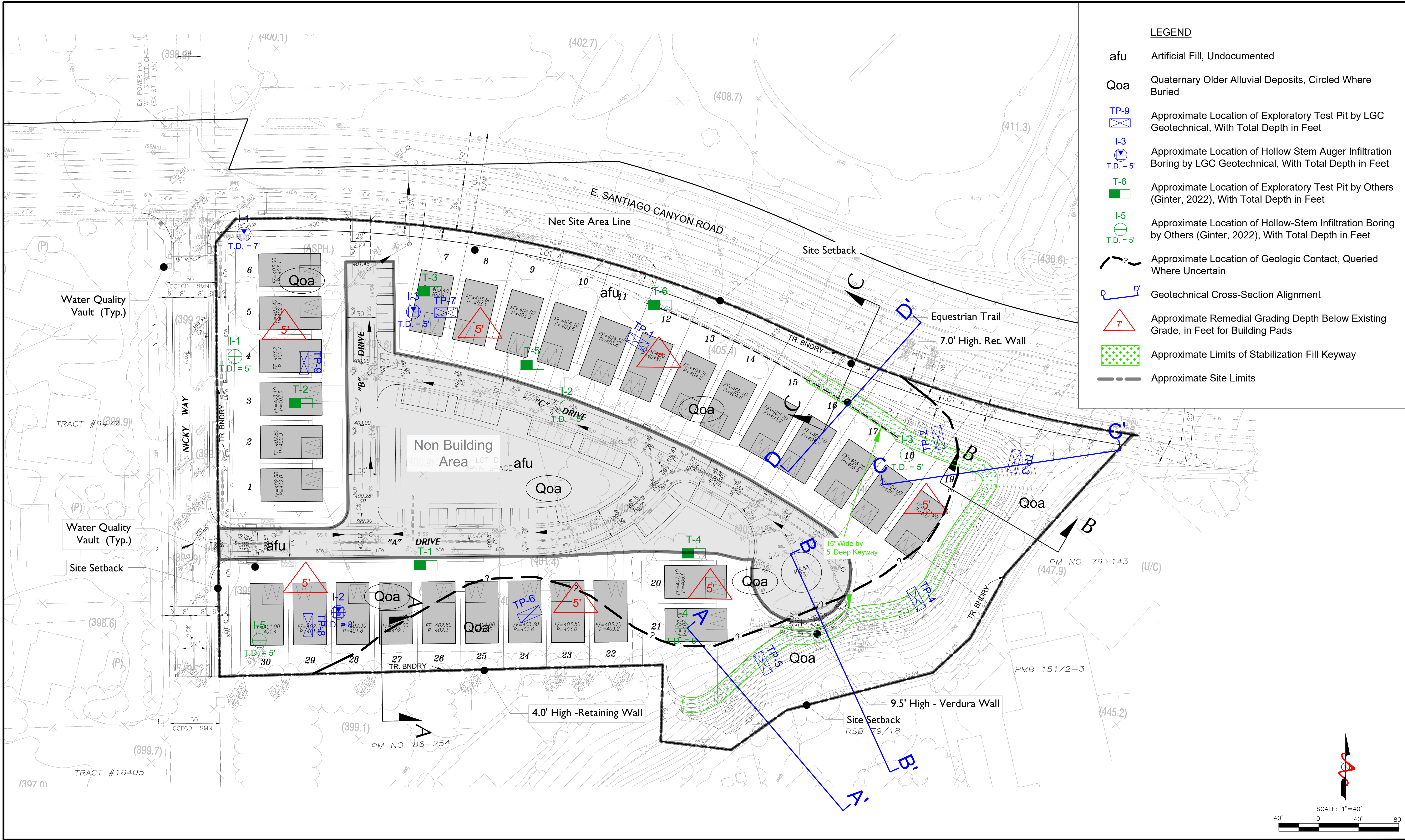
Attachment: Figure 1 – Site Location Map
Sheet 1 – Geotechnical Map
Appendix A – Infiltration Boring Logs
Appendix B – Lab Data
Appendix C – Infiltration Test Results

Distribution: (1) Addressee (via email)



FIGURE 1
Site Location Map

PROJECT NAME	Milan Capital - Arena Site, Orange
PROJECT NO.	25046-01
ENG. / GEOL.	BPP / BPG
SCALE	Not to Scale
DATE	March 2026



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Geotechnical Map

PROJECT NAME	Milan Capital - Arena Site, Orange		
PROJECT NO.	25046-01		
ENG. / GEOL.	DJB / KTM		SHEET 1 of 1
SCALE	1" = 40'		
DATE	March 2026		

Appendix A
Infiltration Boring Logs

Geotechnical Boring Log Borehole I-1

Date: 6/26/2025	Drilling Company: 2R Drilling
Project Name: Milan - Arena Site, Orange	Type of Rig: Truck Mounted Rig (CME 85)
Project Number: 25046-01	Drop: 30" Hole Diameter: 8"
Elevation of Top of Hole: ~400' MSL	Drive Weight: 140 pounds
Hole Location: See Geotechnical Map	Page 1 of 1

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	Logged By OJL Sampled By OJL Checked By BPP DESCRIPTION	Type of Test
395	0							@ 0' - 2" of Asphalt over 9" of Base	
			B-1	14	97.1	9.9	CL	<u>Quaternary Older Alluvial Deposits (Qoa)</u>	
			R-1	16				@5' - Sandy CLAY: brown, moist, very stiff	-#200
				13					
390	5								
385	10							Total Depth = 7'	
380	15							Groundwater Not Encountered	
375	20							3" Perforated Pipe with Filter Sock Installed, Surrounded by Gravel, and Presoaked on 6/26/25	
370	25							Backfilled with Cuttings on 6/27/25	
	30								



THIS SUMMARY APPLIES ONLY AT THE LOCATION OF THIS BORING AND AT THE TIME OF DRILLING. SUBSURFACE CONDITIONS MAY DIFFER AT OTHER LOCATIONS AND MAY CHANGE AT THIS LOCATION WITH THE PASSAGE OF TIME. THE DATA PRESENTED IS A SIMPLIFICATION OF THE ACTUAL CONDITIONS ENCOUNTERED. THE DESCRIPTIONS PROVIDED ARE QUALITATIVE FIELD DESCRIPTIONS AND ARE NOT BASED ON QUANTITATIVE ENGINEERING ANALYSIS.

SAMPLE TYPES:
 B BULK SAMPLE
 R RING SAMPLE (CA Modified Sampler)
 G GRAB SAMPLE
 SPT STANDARD PENETRATION TEST SAMPLE



TEST TYPES:
 DS DIRECT SHEAR
 MD MAXIMUM DENSITY
 SA SIEVE ANALYSIS
 S&H SIEVE AND HYDROMETER
 EI EXPANSION INDEX
 CN CONSOLIDATION
 CR CORROSION
 AL ATTERBERG LIMITS
 CO COLLAPSE/SWELL
 RV R-VALUE
 -#200 % PASSING # 200 SIEVE

Geotechnical Boring Log Borehole I-2

Date: 6/26/2025	Drilling Company: 2R Drilling
Project Name: Milan - Arena Site, Orange	Type of Rig: Truck Mounted Rig (CME 85)
Project Number: 25046-01	Drop: 30" Hole Diameter: 8"
Elevation of Top of Hole: ~400' MSL	Drive Weight: 140 pounds
Hole Location: See Geotechnical Map	Page 1 of 1

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	Logged By OJL Sampled By OJL Checked By BPP DESCRIPTION	Type of Test
	0							@ 0' - Dried vegetation with gravel and some cobbles	
395	5		R-1	22 8 9	105.7	9.3	CL/ML to CL	Quaternary Older Alluvial Deposits (Qoa) @ 5' - Sandy SILT/CLAY to CLAY: brown, slightly moist, stiff	#200 DS
390	10							Total Depth = 8' Groundwater Not Encountered 3" Perforated Pipe with Filter Sock Installed, Surrounded by Gravel, and Presoaked on 6/26/25 Backfilled with Cuttings on 6/27/2025	
385	15								
380	20								
375	25								
370	30								



THIS SUMMARY APPLIES ONLY AT THE LOCATION OF THIS BORING AND AT THE TIME OF DRILLING. SUBSURFACE CONDITIONS MAY DIFFER AT OTHER LOCATIONS AND MAY CHANGE AT THIS LOCATION WITH THE PASSAGE OF TIME. THE DATA PRESENTED IS A SIMPLIFICATION OF THE ACTUAL CONDITIONS ENCOUNTERED. THE DESCRIPTIONS PROVIDED ARE QUALITATIVE FIELD DESCRIPTIONS AND ARE NOT BASED ON QUANTITATIVE ENGINEERING ANALYSIS.

SAMPLE TYPES:
 B BULK SAMPLE
 R RING SAMPLE (CA Modified Sampler)
 G GRAB SAMPLE
 SPT STANDARD PENETRATION TEST SAMPLE



TEST TYPES:
 DS DIRECT SHEAR
 MD MAXIMUM DENSITY
 SA SIEVE ANALYSIS
 S&H SIEVE AND HYDROMETER
 EI EXPANSION INDEX
 CN CONSOLIDATION
 CR CORROSION
 AL ATTERBERG LIMITS
 CO COLLAPSE/SWELL
 RV R-VALUE
 #200 % PASSING # 200 SIEVE

Geotechnical Boring Log Borehole I-3

Date: 6/26/2025	Drilling Company: 2R Drilling
Project Name: Milan - Arena Site, Orange	Type of Rig: Truck Mounted Rig (CME 85)
Project Number: 25046-01	Drop: 30" Hole Diameter: 8"
Elevation of Top of Hole: ~402' MSL	Drive Weight: 140 pounds
Hole Location: See Geotechnical Map	Page 1 of 1

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	Logged By OJL Sampled By OJL Checked By BPP DESCRIPTION	Type of Test
400	0							@ 0' - Gravel with dead vegetation	
395	5		R-1	8 28 31	105.7	3.8	ML	Quaternary Older Alluvial Deposits (Qoa) @ 5' - Gravelly SILT with Sand: orangish brown, slightly moist, hard	#200
390	10							Total Depth = 5' Groundwater Not Encountered 3" Perforated Pipe with Filter Sock Installed, Surrounded by Gravel, and Presoaked on 6/26/25 Backfilled with Cuttings on 6/27/2025	
385	15								
380	20								
375	25								
	30								



THIS SUMMARY APPLIES ONLY AT THE LOCATION OF THIS BORING AND AT THE TIME OF DRILLING. SUBSURFACE CONDITIONS MAY DIFFER AT OTHER LOCATIONS AND MAY CHANGE AT THIS LOCATION WITH THE PASSAGE OF TIME. THE DATA PRESENTED IS A SIMPLIFICATION OF THE ACTUAL CONDITIONS ENCOUNTERED. THE DESCRIPTIONS PROVIDED ARE QUALITATIVE FIELD DESCRIPTIONS AND ARE NOT BASED ON QUANTITATIVE ENGINEERING ANALYSIS.

SAMPLE TYPES:
 B BULK SAMPLE
 R RING SAMPLE (CA Modified Sampler)
 G GRAB SAMPLE
 SPT STANDARD PENETRATION TEST SAMPLE



TEST TYPES:
 DS DIRECT SHEAR
 MD MAXIMUM DENSITY
 SA SIEVE ANALYSIS
 S&H SIEVE AND HYDROMETER
 EI EXPANSION INDEX
 CN CONSOLIDATION
 CR CORROSION
 AL ATTERBERG LIMITS
 CO COLLAPSE/SWELL
 RV R-VALUE
 #200 % PASSING # 200 SIEVE

Appendix B
Laboratory Data

APPENDIX B

Laboratory Testing Procedures and Test Results

The laboratory testing program was formulated towards providing data relating to the relevant engineering properties of the soils with respect to residential construction. Samples considered representative of site conditions were tested in general accordance with American Society for Testing and Materials (ASTM) procedure and/or California Test Methods (CTM), where applicable. The following summary is a brief outline of the test type and a table summarizing the test results.

Moisture and Density Determination Tests: Moisture content (ASTM D2216) and dry density determinations (ASTM D2937) were performed on relatively undisturbed samples obtained from the test borings. The results of these tests are presented in the boring logs. Where applicable, only moisture content was determined from undisturbed or disturbed samples.

Grain Size Distribution/Fines Content: Representative samples were dried, weighed and soaked in water until individual soil particles were separated (per ASTM D421) and then washed on a No. 200 sieve (ASTM D1140). Where applicable, the portion retained on the No. 200 sieve and dried and then sieved on a U.S. Standard brass sieve set in accordance with ASTM D6913 (sieve).

Sample Location	Description	% Passing # 200 Sieve
I-1 @ 5 feet	Clay with Gravel	54
I-2 @ 5 feet	Sandy Silt/Clay	62
I-3 @ 5 feet	Gravelly Silt with Sand	57

Appendix C
Infiltration Test Results

Infiltration Test Data Sheet

LGC Geotechnical, Inc

131 Calle Iglesia Suite 200, San Clemente, CA 92672 tel. (949) 369-6141

Project Name: Milan - Arena Site, Orange
Project Number: 25046-01
Date: 6/27/2025
Boring Number: I-1

Test hole dimensions (if circular)

Boring Depth (feet)*: 7
Boring Diameter (inches): 8
Pipe Diameter (inches): 3

*measured at time of test

Minimum test Head (D_o):

(What the sounder tape should read)

Boring Depth - (5 x Boring Radius) 5.4 ft

(Shallow) The value on the sounder tape should be close to this value during testing for **DEEP** testing fill to 4 feet below top of hole

Pre-Test (Sandy Soil Criteria)*

Trial No.	Start Time (24:HR)	Stop Time (24:HR)	Time Interval (min)	Initial Depth to Water (feet)	Final Depth to Water (feet)	Total Change in Water Level (feet)	Greater Than or Equal to 0.5 feet (yes/no)
1	8:44	9:14	30.0	4.65	5.80	1.15	Yes
2	9:14	9:44	30.0	4.70	5.57	0.87	Yes

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

Main Test Data

Trial No.	Start Time (24:HR)	Stop Time (24:HR)	Time Interval, Dt (min)	Initial Depth to Water, D_o (feet)	Final Depth to Water, D_f (feet)	Change in Water Level, DD (feet)	Measured Infiltration Rate(in/hr)
1	9:47	9:57	10.0	4.90	5.18	0.28	1.58
2	9:57	10:07	10.0	4.88	5.09	0.21	1.16
3	10:07	10:17	10.0	4.84	5.08	0.24	1.31
4	10:17	10:27	10.0	4.95	5.21	0.26	1.50
5	10:30	10:44	14.0	4.90	5.24	0.34	1.39
6	10:45	10:55	10.0	5.00	5.28	0.28	1.66

Measured Infiltration Rate (No Factor of Safety)

1.5

Feasibility Factor of Safety

See Report

Measured Infiltration Rate (With Factor of Safety)

See Report

Sketch:

Notes:



Infiltration Test Data Sheet

LGC Geotechnical, Inc

131 Calle Iglesia Suite 200, San Clemente, CA 92672 tel. (949) 369-6141

Project Name: Milan - Arena Site, Orange
Project Number: 25046-01
Date: 6/27/2025
Boring Number: I-2

Test hole dimensions (if circular)

Boring Depth (feet)*: 8
Boring Diameter (inches): 8
Pipe Diameter (inches): 3

*measured at time of test

Test pit dimensions (if rectangular)

Pit Depth (feet):
Pit Length (feet):
Pit Breadth (feet):

Minimum test Head (D_o):

(What the sounder tape should read)

Boring Depth - (5 x Boring Radius) 6.4 ft

(Shallow) The value on the sounder tape should be close to this value during testing for **DEEP** testing fill to 4 feet below top of hole

Pre-Test (Sandy Soil Criteria)*

Trial No.	Start Time (24:HR)	Stop Time (24:HR)	Time Interval (min)	Initial Depth to Water (feet)	Final Depth to Water (feet)	Total Change in Water Level (feet)	Greater Than or Equal to 0.5 feet (yes/no)
1	8:37	9:07	30.0	4.81	4.97	0.16	No
2	9:07	9:37	30.0	4.70	4.91	0.21	No

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

Main Test Data

Trial No.	Start Time (24:HR)	Stop Time (24:HR)	Time Interval, Dt (min)	Initial Depth to Water, D_o (feet)	Final Depth to Water, D_f (feet)	Change in Water Level, DD (feet)	Measured Infiltration Rate(in/hr)
1	9:37	10:07	30.0	4.55	4.78	0.23	0.3
2	10:08	10:38	30.0	4.36	4.56	0.20	0.2
3	10:38	11:16	38.0	4.56	4.82	0.26	0.2
4	11:16	11:47	31.0	4.65	4.86	0.21	0.2
5	11:48	12:18	30.0	4.78	5.000	0.22	0.3
6	12:18	12:48	30.0	4.82	5.050	0.23	0.3
7	12:49	13:19	30.0	4.79	5.000	0.21	0.3
8	13:20	13:50	30.0	4.81	5.03	0.22	0.3
9	13:50	14:20	30.0	4.84	5.04	0.20	0.2
10	14:20	14:50	30.0	4.83	5.04	0.21	0.3
11	14:50	15:20	30.0	4.83	5.04	0.21	0.3
12	15:20	15:50	30.0	4.75	4.97	0.22	0.3

Measured Infiltration Rate (No Factor of Safety) 0.3

Feasibility Factor of Safety See Report

Measured Infiltration Rate (With Factor of Safety) See Report

Sketch:

Notes:



Infiltration Test Data Sheet

LGC Geotechnical, Inc

131 Calle Iglesia Suite 200, San Clemente, CA 92672 tel. (949) 369-6141

Project Name: Milan - Arena Site, Orange
Project Number: 25046-01
Date: 6/27/2025
Boring Number: I-3

Test hole dimensions (if circular)

Boring Depth (feet)*: 5
Boring Diameter (inches): 8
Pipe Diameter (inches): 3

*measured at time of test

Test pit dimensions (if rectangular)

Pit Depth (feet):
Pit Length (feet):
Pit Breadth (feet):

Minimum test Head (D_o):

(What the sounder tape should read)

Boring Depth - (5 x Boring Radius) 3.4 ft

(Shallow) The value on the sounder tape should be close to this value during testing for **DEEP** testing fill to 4 feet below top of hole

Pre-Test (Sandy Soil Criteria)*

Trial No.	Start Time (24:HR)	Stop Time (24:HR)	Time Interval (min)	Initial Depth to Water (feet)	Final Depth to Water (feet)	Total Change in Water Level (feet)	Greater Than or Equal to 0.5 feet (yes/no)
1	8:40	9:10	30.0	3.94	4.41	0.47	No
2	9:10	9:40	30.0	3.91	4.41	0.50	Yes

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

Main Test Data

Trial No.	Start Time (24:HR)	Stop Time (24:HR)	Time Interval, Dt (min)	Initial Depth to Water, D_o (feet)	Final Depth to Water, D_f (feet)	Change in Water Level, DD (feet)	Measured Infiltration Rate(in/hr)
1	9:41	10:11	30.0	3.71	4.20	0.49	1.6
2	10:12	10:46	34.0	3.80	4.35	0.55	1.8
3	10:47	11:17	30.0	3.81	4.29	0.48	1.7
4	11:21	11:52	31.0	3.75	4.18	0.43	1.4
5	11:52	12:23	31.0	3.70	4.15	0.45	1.4
6	12:24	12:54	30.0	3.74	4.17	0.43	1.4
7	12:55	13:25	30.0	3.70	4.16	0.46	1.5
8	13:26	13:56	30.0	3.72	4.15	0.43	1.4
9	13:56	14:26	30.0	3.74	4.18	0.44	1.5
10	14:26	14:56	30.0	3.77	4.21	0.44	1.5
11	14:56	15:26	30.0	3.72	4.15	0.43	1.4
12	15:26	15:56	30.0	3.80	4.23	0.43	1.5

Measured Infiltration Rate (No Factor of Safety) 1.5

Feasibility Factor of Safety See Report

Measured Infiltration Rate (With Factor of Safety) See Report

Sketch:

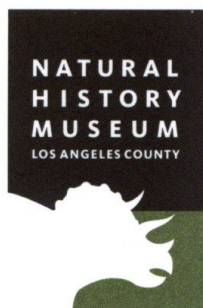
Notes:



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D-3: NHMLA Record Search Paleontology

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Natural History Museum
of Los Angeles County
900 Exposition Boulevard
Los Angeles, CA 90007

tel 213.763.DINO
www.nhm.org

Research & Collections

e-mail: paleorecords@nhm.org

September 1, 2025

First Carbon Solutions
Attn: Matthew Dionisio

re: Paleontological resources records search for the 6224.0003 Orange Arena Residential Project

Dear Matthew:

I have conducted a search of our paleontology collection records for the proposed development at the 6224.0003 Orange Arena Residential Project area as outlined on the portion of the Orange USGS topographic quadrangle map that you sent to me via e-mail on August 21, 2025. We do not have fossil localities that have been recorded or georeferenced directly within the proposed project area, but we do have fossil localities nearby from similar sedimentary deposits that may occur in the proposed project area, either at the surface or at depth.

The following table shows the closest known localities in the collection of the Natural History Museum of Los Angeles County (NHMLA).

Locality Number	Location	Formation	Taxa	Depth
LACM VP 6927 - 6930	Peralta Hills, NE of Serrano Ave and Cannon St.	Sespe / Vacqueros Formation (sandy claystone and clay)	Horse relative (<i>Parahippus</i>), unidentified artiodactyl (<i>Artiodactyla</i>), pig-like mammal (<i>Tayassuidae</i>), camel family (<i>Camelidae</i>)	Unknown (recovered during grading)
LACM IP 23649 - 23650	Sec. 14; T4S; R9W Roadcut on north side of Santiago Canyon Road. Just NW of entrance to Santiago Canyon landfill	Topanga Formation (pebble conglomerate bed)	Invertebrates (uncatalogued)	Exposed in gullies
LACM VP 6292	Richfield, CA (south of Yorba Linda)	Topanga Formation (silty with lots of gypsum)	Shark (<i>Mustelus</i>)	Unknown depth in roadcut
LACM VP 3292	Panorama Heights, along ridge NW of Newport Ave	Terrace deposits (Pleistocene)	Elephant clade (<i>Proboscidae</i>)	Unknown
LACM IP 771, 1425, 30182		Vaqueros Formation	Invertebrates, including <i>Turritella ocoyana</i>	Unknown

VP, Vertebrate Paleontology; IP, Invertebrate Paleontology; bgs, below ground surface

This records search is limited to the records of the NHMLA. It is not intended as a paleontological assessment of the project for the purposes of California Environmental Quality

Act (CEQA) or National Environmental Policy Act (NEPA). Potentially fossil-bearing units are present in the project area, either at the surface or in the subsurface. As such, NHMLA recommends that a paleontological assessment be conducted by a paleontologist meeting Federal (43 Code of Federal Regulations Part 49.110) or Society of Vertebrate Paleontology standards for compliance with applicable regulations, such as CEQA or NEPA.

Sincerely,

A handwritten signature in black ink that reads "Alyssa Bell". The signature is written in a cursive, flowing style. The first name "Alyssa" is written in a larger, more prominent script, and the last name "Bell" is written in a slightly smaller, more compact script. The signature is set against a light beige rectangular background.

Alyssa Bell, Ph.D.
Natural History Museum of Los Angeles County