GEOTECHNICAL AND GEOLOGIC HAZARD INVESTIGATION

May 5, 2021

Prepared For:

Mr. Dennis Astl
Palomar Community College
1140 W. Mission Road
San Marcos, California 92069

Proposed Softball & Football Fields and Fieldhouse
1140 W. Mission Road
San Marcos, CA 92069

Project No.: 113821-0001310.00

NV5 West, Inc.
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Palomar Community College
1140 W. Mission Road
San Marcos, CA 92069

May 5, 2021
Project No.: 113821-1310.00

Attention: Mr. Dennis Astl, Manager, Construction & Facilities Planning

Subject: Preliminary Geotechnical Investigation Report

Project: Proposed Softball & Football Fields and Fieldhouses
Palomar Community College
1140 W. Mission Road
San Marcos, California

Reference: Geotechnical Investigation, "*Proposed Temporary Parking Lot, Palomar Community College, San Marcos, California,*" prepared by NV5, dated September 3, 2015, project number 766.

Dear Mr. Astl:

This report presents the results of NV5 West, Inc.'s (NV5) geotechnical investigation for the proposed Softball & Football Fields and Fieldhouses, located at the Palomar College – San Marcos Campus, California. Based on the information obtained during this investigation, it is NV5's opinion that the site is suitable for the proposed development, provided that the pertinent recommendations and design parameters contained in this report are incorporated into the design and construction of the project.

NV5 appreciates the opportunity to provide this geotechnical engineering service for this project and looks forward to continuing its role as your geotechnical engineering consultant.

Respectfully submitted,

NV5 West, Inc.

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1.0 INTRODUCTION

This report presents the results of the geotechnical engineering investigation for the proposed Softball & Football Fields and Fieldhouse project, located at the Palomar College – San Marcos Campus, California. The approximate location of the project site is presented on *Figure 1, Site Location Map* and *Figure 2, Vicinity Map*.

The purpose of this study was to evaluate the subsurface soil conditions at the subject site and to provide recommendations for project earthwork, seismic design criteria, and foundation design parameters for the proposed project development. This report summarizes the data collected and presents NV5's findings, conclusions, and recommendations.

This report has been prepared for the exclusive use of the client and their consultants in the design of the proposed new structure and appurtenant improvements. In particular, it should be noted that this report has not been prepared from the perspective of a construction bid preparation instrument and should be considered by prospective construction bidders only as a source of general information subject to interpretation and refinement by their own expertise and experience, particularly with regard to construction feasibility. Contract requirements as set forth by the project plans and specifications will supersede any general observations and specific recommendations presented in this report.

2.0 SCOPE OF SERVICES

NV5's scope of services for this project included the following tasks:

- Review of preliminary project sketches, geologic maps and geotechnical literature pertaining to the site vicinity.
- Review of NV5's previous geotechnical investigation performed within the project development area on September 3, 2015 (Refer to *Appendix A*).
- Performance of a site reconnaissance to observe the general surficial site conditions and to mark out proposed boring locations.
- Procurement of a County of San Diego Department of Environmental Health boring permit for the applicable exploratory borings as required by law.
- Preparation of a Health and Safety Plan (HASP) addressing the field exploration work.
- Coordination with entities having an interest in the field exploration activities including the Palomar College staff, the exploration subcontractor (Baja Exploration), private utility locating subcontractor (GPRS, LLC), and Underground Service Alert of California.
- Performance of a subsurface investigation, which included the drilling, logging, and sampling of ten (10) exploratory borings and two (2) percolation test borings located within the proposed project area to explored depths ranging from 5 to 26.5 feet below ground surface (bgs).
- Performance of appropriate laboratory testing on selected representative bulk-disturbed and relatively undisturbed drive-samples of the materials obtained during the field exploration program to aid in the classification and to evaluate their pertinent geotechnical engineering properties.

- Performing an assessment of general seismic conditions and geologic hazards affecting the site area and their possible impact on the subject project.
- Engineering evaluation and analysis of the geotechnical data collected to develop geotechnical recommendations for the design and construction of the proposed development. Specifically the following items were addressed:
    - Evaluation of general subsurface conditions and description of types, distribution, and engineering characteristics of subsurface materials.
    - Addressing any potential geologic/geotechnical hazards.
    - General recommendations for earthwork, including site preparation, excavation, site drainage, and the placement of compacted fill.
    - Recommendations for temporary slopes/cuts and shoring.
    - Evaluation of project feasibility and suitability of on-site soils for foundation support.
    - Recommendations for design of suitable foundation systems including allowable bearing capacity, lateral resistance, settlement estimates, and slab-on-grade construction.
    - Design parameters for retaining walls (active and at-rest pressures) and waterproofing recommendations.
    - Determination of seismic design criteria.
    - Recommendations for subgrade preparation within proposed exterior flatwork and pavement areas including flexible and rigid pavement sections.
- Preparation of this report, including reference maps and graphics, summarizing the data collected and presenting NV5's findings, conclusions, and geotechnical recommendations for the design and construction of the proposed development.

3.0 PROJECT AND SITE DESCRIPTION

It is understood that the proposed development will include a new football field, relocated softball field, stadium lights, new storage buildings and two fieldhouses. The new football field, bleachers and press box will be constructed over the existing temporary parking lot and the softball field will be relocated slightly to the east. A storage building for each field is planned along with two new fieldhouses at the south end of the site. In addition, the site improvements will include grading for foundation construction, and ground preparation for associated flatwork, pavements, and utilities. The approximate location of the proposed new sports fields and appurtenant structures, the approximate limits of the proposed development, and the existing site conditions are presented in *Figure 3, Geotechnical Exploration Map*.

The area of the proposed development is currently developed consisting of an existing base-surface parking lot and a softball field, and with their associated improvements. The proposed development area is relatively level with at an elevation of approximately 575 feet above Mean Sea Level (MSL). Besides the lawn-grass softball field, vegetation across the proposed development area is relatively sparse, consisting of minor areas of decorative shrubs, some ornamental trees, and a few young to mature trees.

4.0 FIELD EXPLORATION PROGRAM

Before starting the field exploration program, a field reconnaissance was conducted to observe site conditions and mark out the locations for the planned subsurface explorations. A County of San Diego Department of Environmental Health (CoSD-DEH) boring permit was obtained for NV5's "deep" exploratory borings, and for borings where shallow groundwater was anticipated and/or encountered. As required by law, Underground Service Alert was notified of the locations of the exploratory borings, in addition to utilizing a private utility locating subcontractor, prior to drilling. NV5 also coordinated the field work schedule with Palomar College staff.

4.1 EXPLORATORY DRILLING

The subsurface conditions at the subject site were explored on March 17 and 18, 2021 by drilling ten (10) exploratory borings and two (2) percolation test borings within the proposed development area using a truck-mounted drilling rig equipped with 8-inch outer-diameter hollow stem augers. The approximate locations of the exploratory borings and percolation test borings are shown in *Figure 3, Geotechnical Exploration Map*.

The materials encountered in the exploratory borings were continuously observed, classified, and logged by an NV5 geologist in general accordance with the Unified Soil Classification System (USCS/ASTM D2487) and ASTM D2488. The logs of the exploratory borings are presented in *Appendix B, Exploratory Boring Logs*. Subsequent to logging and sampling, the exploratory borings were backfilled with the excavated materials with the exception of the "deep" borings and borings where shallow groundwater was encountered. In accordance with the NV5 acquired CoSD-DEH boring permit, these borings were backfilled up to 5 feet bgs with methodically hydrated bentonite chips, and capped with compacted soil cuttings above. Boring locations B-9, B-10, and P-1, located within the lawn-grass softball field, were patched with grass plugs, and boring B-6 was patched with approximately 4 inches of rapid set concrete.

Representative bulk-disturbed and relatively undisturbed drive-samples were retrieved during exploratory drilling at selected depths appropriate to the investigation. The samples were labeled in the field and transported to NV5's laboratory for observation, evaluation, and testing. The drive samples were obtained using the California Modified Split Spoon (CAL) and Standard Penetration Test (SPT) samplers. The Sampling Methods and Logging Methods used to collect and describe the earth materials encountered during the field investigation are presented in *Appendix B*.

4.2 FIELD PERCOLATION TESTING

On March 17, 2021, two (2) percolation tests were performed within the design teams specified areas to evaluate the infiltration characteristics of the on-site soils as it relates to the feasibility of storm water runoff infiltration. The percolation tests were conducted in the 8-inch diameter borings (P1 and P2) at depths of approximately five feet bgs. The percolation tests were performed in general accordance with the San Diego County Department of Environmental Health (CoSD-DEH) Percolation Test Procedures. The approximate locations of the percolation tests are presented in *Figure 3, Geotechnical Exploration Map*.

Prior to percolation testing and in accordance with the CoSD-DEH percolation test procedures, the percolation test borings were presoaked on March 17, 2021. On March 18, 2021, NV5's field representative noted that the presoak water level had dropped less than 1/8 of an inch for a 24-hour period. The water level was periodically monitored during the second day of drilling activities and did not change over the span of seven hours onsite.

The percolation test data at locations P-1 and P-2 suggest infiltration rates that are considered unfavorable (i.e. very low transmissivity of groundwater). In addition, these locations are underlain at shallow to moderate depth by colluvial and sedimentary formational materials consisting of Sandy Lean Clay (CL) and Fat Clay (CH). Based on the previous factors discussed and the natural variability in stratigraphy of the formational materials underlying the percolation test area, the project site may lead to the build-up and subsequent lateral migration of infiltration water. In addition, NV5 laboratory expansion test results yielded medium expansion potential soils. If these expansive soils are provided a water source, deep and extensive saturation of expansive clayey soils may occur and could result in damage to the proposed structures, flatwork, and associated improvements. For these reasons, it is NV5's opinion that the subject site is considered **not** suitable for infiltration of storm water runoff in any amount. If desired, discharge basins at or near these locations may be designed with an impermeable liner and piped to a suitable discharge outlet.

In general, infiltration of storm water runoff should not be permitted in close proximity (50 feet) to fill and/or cut slopes. In addition, infiltration basins should have a minimum setback distance of 20 feet from any existing and/or proposed new structures, and be provided with a cutoff barrier to prevent the lateral migration and accumulation of infiltration water.

The in-situ infiltration characteristics of the subsurface materials are primarily a function of the amount of fines (i.e., silt and clay size), the relative density, and other anomalies associated with the placement of fill or natural depositional/weathering processes (e.g., compaction/lamination, smearing, cementation).

5.0 LABORATORY TESTING

Laboratory testing was performed on selected representative bulk-disturbed and relatively undisturbed drive samples of the materials obtained from the exploratory borings to aid in the material classifications and to evaluate their geotechnical engineering properties. The following tests were performed:

- In-situ Density and Moisture Content (ASTM D2937 and ASTM D2216)
- Determination of Percentage of Particles Passing No. 200 Sieve (ASTM D1140)
- Expansion Index (ASTM D4829)
- Atterberg Limits (ASTM D4318)
- Corrosivity test series, including sulfate content, chloride content, pH-value, and resistivity (CTM 417, 422, and 532/643)

Testing was performed in general accordance with applicable ASTM standards and California Test Methods (CTM). A summary of the laboratory testing program and the laboratory test results are presented in *Appendix C, Laboratory Test Results*.

6.0 GEOLOGIC AND SOIL INFORMATION

6.1 REGIONAL GEOLOGIC SETTING

The project site is located in south San Diego County and within the coastal section of the Peninsular Ranges Geomorphic Province. This province is characterized by northwest-trending mountain ranges bordered by relatively straight-sided, sediment-floored valleys. The northwest trend is also reflected in the direction of the dominant geologic structural features, which consist of northwest-trending faults and fault zones. Two major, predominantly northwest-trending, fault zones traverse the San Diego metropolitan and the inland county areas: the Rose Canyon-Newport Inglewood (connected) fault zones roughly 12 miles to the west and the Elsinore fault zone roughly 17 miles to the northeast.

Typical geologic stratigraphy of the bedrock units in the area of the subject site include Mesozoic-age igneous intrusive rocks and metamorphic rocks, Cenozoic-age sedimentary rocks, and Quaternary-age sedimentary deposits.

6.2 SITE-SPECIFIC GEOLOGIC AND SOIL INFORMATION

During the subsurface investigation at the project site claystone and sandstone sedimentary bedrock deposits of the Tertiary-age Santiago Formation (Kennedy-Tan, 2007; Map symbol: Tsa) were encountered in exploratory borings B-1, B-2, B-3, B7, and B-9 underlying existing fill and alluvium at encountered depths ranging from 9 to 14 feet bgs, and extending to the maximum depth explored (approximately 26.5 feet bgs). The general geologic conditions in the project vicinity are displayed in *Figure 4, Regional Geologic Map*.

Geotechnical information compiled during this investigation is presented in *Appendix B, Exploratory Boring Logs*. The lateral distribution of the earth materials encountered during the field investigation in light of the existing conditions and proposed development is presented in *Figures 5a & 5b, Geologic Cross Section A-A' and Geologic Cross Section B-B'*. Descriptions and details of the geologic units encountered are discussed in the follow paragraphs.

- Fill: Existing fill soils were encountered near-surface at all of the exploratory borings locations, with thickness ranging from 2 to 9 feet bgs. The encountered fill materials consist of red-brown, dark brown, gray-brown, and dark gray brown, moist, soft to very stiff, fine- to medium-grained and fine- to coarse-grained, sandy lean clay (CL) and dark brown, moist, loose to dense fine- to coarse clayey sand (SC).

- Alluvium (Qa): Alluvial materials were encountered underlying the fill materials at all exploratory borings locations at encountered depths ranging from 2 to 9 feet bgs with an encountered vertical thickness ranging from 4 to 11.5 feet. The encountered alluvium consists of gray, light gray, gray-brown, yellow-brown, dark brown, and red-brown, moist, stiff to hard, sandy lean clay (CL) and sandy fat clay (CH).

- Santiago Formation (Tsa): Interbedded and massive claystone and sandstone sedimentary bedrock deposits of the Santiago Formation was encountered at exploratory boring locations B-1, B-2, B-3, B-7, and B-9 underlying the fill and alluvium at encountered depths ranging from 9 to 14 feet bgs, and extending to the maximum depth explored (approximately 26.5 feet bgs). The encountered claystone unit of the formational bedrock consists of gray, gray-brown, and red-brown, moist, very stiff to hard, sandy fat clay (CH). The encountered sandstone unit of the formational bedrock consists of yellow-gray and red-brown, moist, dense to very dense, fine-grained clayey sand (SC).

6.3 GROUNDWATER

Groundwater was encountered at exploratory boring locations B-1 and B-9 at a depth of 7 and 22 feet bgs, respectively. In addition, wet material was encountered at exploratory boring locations B-3 at depths 8 to 10 feet bgs. As such, groundwater should be considered a factor depending on depths and extent of proposed project excavations. Additional discussion regarding groundwater constraints are provided in Section 8.5 of this report.

6.4 FAULTS

The numerous faults in southern California include active, potentially active, and inactive faults. As used in this report, the definitions of fault terms are based on those developed for the Alquist-Priolo Special Studies Zones Act of 1972 and published by the California Division of Mines and Geology (Hart and Bryant, 1997).

Active faults are defined as those that have experienced surface displacement within Holocene time (approximately the last 11,000 years) and/or have been included within any of the state-designated *Earthquake Fault Zones* (previously known as Alquist-Priolo Special Studies Zones). Faults are considered potentially active if they exhibit evidence of surface displacement since the beginning of Quaternary time (approximately two million years ago) but not since the beginning of Holocene time. Inactive faults are those that have not had surface movement since the beginning of Quaternary time.

The site is not mapped within a State-designated *Earthquake Fault Zone*, nor have active faults been mapped on the subject site. Furthermore, evidence of active faulting at the site was not observed during the investigation.

The closest known active fault to the site is the Newport-Inglewood/Rose Canyon (connected) fault zone located approximately 12 miles west of the site. Other important active faults that could affect the San Diego area and their distance to the site are included in the following Table 1. In addition, *Figure 6, Regional Fault Map* depicts the site location in relation to known active faults in the region.

Table 1 - Distance From the Site to Major Active Faults

Fault Name	Distance From the Site
Newport-Inglewood/Rose Canyon (connected)	12 miles
Elsinore	17 miles
Palos Verdes-Coronado Bank (connected)	27 miles
Earthquake Valley	34 miles
San Jacinto	42 miles
Chino	52 miles
San Andreas	59 miles
Superstition Hills	70 miles
Coyote Hills	65 miles
Laguna Salada	80 miles

7.0 SEISMIC AND GEOTECHNICAL HAZARDS

The findings of NV5's seismic and geotechnical hazards evaluation for the proposed project are summarized in the following sections.

7.1 FAULT RUPTURE

The site is not located within an Earthquake Fault Zone delineated by the State of California for the hazard of fault surface rupture. The surface traces of any active or potentially active faults are not known to pass directly through, or to project toward the site. Therefore, the potential for surface rupture due to faulting occurring beneath the site during the design life of the proposed structures is considered very low.

7.2 SEISMIC SHAKING

The project site is located in an area of California considered a seismically active area, and as such, the seismic hazard most likely to impact the site is ground shaking resulting from an earthquake along one of the known active faults in the region.

Preliminary seismic parameters were developed for the project site based on the 2019 California Building Code (CBC) and ASCE 7-16 guidance document. Using the Structural Engineers Association of California's U.S. Seismic Design Maps Online Calculator (https://seismicmaps.org/), based on the following site coordinates: Latitude = 33.147463 degrees, and Longitude = -117.182669 degrees. The earthquake hazard level of the Maximum Considered Earthquake (MCE) is defined in ASCE 7-16 as the ground motion having a probability of exceedance of 2 percent in 50 years. The preliminary seismic design parameters for the project site are presented in Table 2 below. NV5 should be contacted to provide revisions to these parameters if other codes are specified. Based on discussions with the project structural engineer, all of the buildings design will be governed by Exception #2, of Section 11.4.8 of ASCE 7-16 and a site specific ground motion hazard analysis is not required.

Table 2 - Recommended 2019 CBC Seismic Design Parameters

Design Parameter	Recommended Value	Reference
Site Class	D (Stiff Soil)	CBC Section 1613.3.2
Mapped Spectral Accelerations for short periods, S_S	0.898g	CBC Section 1613.2.1
Mapped Spectral Accelerations for 1-sec period, S_1	0.331g	CBC Section 1613.2.1
Short-Period Site Coefficient, F_a	1.141	CBC Table 1613.2.3
Long-Period Site Coefficient, F_v	1.969	CBC Table 1613.2.3
(1) MCE_R (5% damped) spectral response acceleration for short periods adjusted for site class, S_{MS}	1.025g	CBC Section 1613.2.3
(1) MCE_R (5% damped) spectral response acceleration at 1-second period adjusted for site class, S_{M1}	0.652g	CBC Section 1613.2.3
Design spectral response acceleration (5% damped) at short periods, S_{DS}	0.683g	CBC Section 1613.2.4
Design spectral response acceleration (5% damped) at 1-second period, S_{D1}	0.435g	CBC Section 1613.2.4
Seismic Design Category	D	CBC Section 1613.2.5
(2) MCE_G Peak Ground Acceleration adjusted for site class effects, PGA_M	0.470g	ASCE 7-16 Section 11.8.3

(1) MCE_R = Risk-adjusted Maximum Considered Earthquake
(2) MCE_G = Geometric-mean Maximum Considered Earthquake

7.3 LIQUEFACTION AND SEISMICALLY-INDUCED SETTLEMENT

Liquefaction of soils can be caused by ground shaking during earthquakes. Research and historical data indicate that loose, relatively clean granular soils are susceptible to liquefaction and dynamic settlement, whereas the stability of the majority of clayey silts, silty clays and clays is not adversely affected by ground shaking. Liquefaction is generally known to occur in saturated cohesionless soils at depths shallower than approximately 50 feet. Dynamic settlement due to earthquake shaking can occur in both dry and saturated sands.

The site is underlain predominantly by indurated clay-rich and well-consolidated, Tertiary-age Santiago formation (Tsa) sedimentary bedrock deposits which are not considered to be susceptible to liquefaction. The upper soils above the formational material are medium dense to dense granular material or clay rich cohesive soils. Therefore, the potential for liquefaction and associated ground deformation occurring beneath the structural site areas during a seismic event is considered low.

Seismic settlement is often caused when loose to medium-dense granular soils are densified during ground shaking. As shown in the San Marcos Safety Element, the site is not mapped within seismic induced settlement or liquefaction zone.

7.4 LANDSLIDES AND SLOPE INSTABILITY

The building pad area is located on and surrounded by developed and/or previously graded, relatively level ground. There are no known landslides on or near the project site, and the site is not located in the path of any known landslides.

It is NV5's opinion that the potential for damage to the proposed structures due to landsliding or slope instability is considered very low.

7.5 SUBSIDENCE

The site is not located in an area of known ground subsidence due to the withdrawal of subsurface fluids. Accordingly, the potential for subsidence occurring at the site due to the withdrawal of oil, gas, or water is considered to be very low.

7.6 TSUNAMIS, INUNDATION SEICHE, AND FLOODING

The site is located at or above an elevation of approximately 573 feet above MSL. Its lowest point is located approximately 7.6 miles from an exposed lagoon and 8.4 miles from the Pacific Ocean. Also, the site is not located downslope of any large body of water that could affect the site in the event of an earthquake-induced failure or seiche (oscillation in a body of water due to earthquake shaking). Therefore, the potential for damaging tsunamis (seismic sea waves) or seiche is considered very low.

Based on a review of Federal Emergency Management Agency (FEMA) flood insurance rate map (FIRM), the site is not located within a 500-year floodplain and is denoted as an "area of minimal flood hazard, Zone X." Based on the map review, the potential for significant flooding of the site is considered to be very low. Site drainage should be addressed by the project civil engineer.

7.7 EXPANSIVE SOILS

NV5's investigation revealed that fill and colluvial soils within the proposed project area consists of soils characterized as Clayey Sand (SC), Sandy Lean CLAY (CL) and Fat Clay (CH). Laboratory expansion index test results indicate these soils possess a **medium expansion potential**. Without adequate accommodation for these soils in the foundation design and/or treatment of these soils, damage from hydro-expansion/contraction of the clayey fill soils may occur to building foundations, slabs, pavements, and associated site flatwork. Detailed recommendations for structural design and/or treatment of these expansive soils are provided in the grading and earthwork recommendations section.

8.0 CONCLUSIONS AND DESIGN RECOMMENDATIONS

8.1 GENERAL

Based on the results of the field exploration, laboratory testing, and geotechnical engineering evaluation and analyses of the accumulated data, the proposed development is considered geotechnically feasible, provided the recommendations contained herein are incorporated into the project plans and specifications and implemented during construction.

Significant geotechnical concerns for the project include the presence of low density fill soils that are potentially compressible, and the medium to high expansion potential of site soils. The following potential mitigative recommendation alternatives (removal, moisture conditioning, and recompaction of the near-surface fill soils) are provided herein for consideration. Detailed recommendations for site preparation and earthwork, foundations, exterior and interior slabs, and corrosion potential are provided in the following report sections.

8.2 GRADING AND EARTHWORK

Site grading should be performed in accordance with the following recommendations and the *Typical Earthwork Guidelines* provided in *Appendix D*. In the event of conflict, the recommendations presented herein supersede those of *Appendix D*.

- <u>Clearing and Grubbing</u> - Prior to grading, the project area should be cleared of significant surface vegetation, demolition rubble, trash, pavement, debris, etc. Any buried organic debris or other unsuitable contaminated material encountered during subsequent excavation and grading work should also be removed. Removed material and debris should be properly disposed of offsite. Holes resulting from removal of buried obstructions which extend below finished site grades should be filled with properly compacted soils.

- <u>Excavatability</u> – Based on the subsurface explorations, it is anticipated that the on-site soils can be excavated by modern conventional heavy-duty excavating equipment in good operating conditions, however, the grading contractor should make their own assessment. Excavations near or below groundwater will encounter wet and loose or soft ground conditions. Caving should also be anticipated in excavations including drilled holes. Wet soils are anticipated to be subject to pumping under heavy equipment loads.

- <u>Site Remedial Grading and Treatment of Potentially Expansive Fill and Colluvial Soils</u> – Areas to receive surface improvements or fill soils should be treated as follows:

    - <u>Building Pad Excavation/Fill Removal and Subgrade Preparation</u> – Prior to fill placement, underlying potentially compressible existing fill and alluvial soils should be removed to a depth of 5 feet below existing site grade under proposed building pads. The areal extent of the excavation should extend laterally a distance of at least 5 feet outside perimeter building footings. The extent and depths of removals and overexcavations should be evaluated by NV5 representative in the field.

      The materials exposed in the bottom of the excavation should be scarified to a minimum depth of 8 inches, moisture conditioned to approximately 2 percent above the optimum moisture, and uniformly recompacted prior to placement of fill soils.

    - <u>Fill Removal Under Non-building Areas</u> – Prior to fill placement, fill soils should be removed to a depth of 2 feet and 2 feet laterally beyond the proposed improvements, exposed subgrade scarified a minimum depth of 8 inches, moisture conditioned to approximately 2 percent over optimum moisture and uniformly recompacted prior to placement of fill soils in areas to receive improvements (turf football field, flatwork, stadium seats, and other ancillary structures, etc) outside of building pad limits.

- Structural Fill Placement – Fill placed should be adequately moisture conditioned and uniformly mixed prior to placement. These fills should be compacted to at least 90 percent of the soils laboratory determined maximum dry density (based on ASTM D1557) and at a moisture content approximately 2 percent above the optimum moisture, unless otherwise directed by the geotechnical engineer.

- Fill Placement Thickness and Testing – The optimum lift thickness to produce a uniformly compacted fill will depend on the size and type of construction equipment used. In general, fill should be placed in uniform lifts not exceeding 8 inches in loose thickness. Fill should be moisture conditioned to approximately 2 percent over optimum moisture content and uniformly compacted to at least 90 percent of the soils laboratory determined maximum dry density (based on ASTM D1557), unless otherwise directed by the geotechnical consultant of record. Placement, moisture conditioning, and compaction of fill should be continuously observed and tested by the geotechnical consultant of record. Fill should be tested for relative compaction and moisture content at a minimum of every 2-foot in vertical height.

- Material for Fills – Fill materials should be free of deleterious or oversized materials. Any rocks with a maximum dimension greater than 6 inches should be screened and removed, and rocks with a maximum dimension greater than 3 inches should not be placed in the upper 3 feet of the building pad or in utility trenches.

  As noted, laboratory testing of onsite soils indicated medium expansion potential. Expansive materials are not considered suitable for reuse as compacted fill in the building pads. These materials are not considered suitable for reuse as compacted fills in utility trenches, retaining wall backfill and below foundations, or below any settlement or heave sensitive structures and improvements. Since site grading may redistribute the on-site soils, potential expansive soil properties should be verified at the completion of rough grading.

  Import soils are anticipated to be needed for the proposed improvements. Potential import sources should be sampled and tested for suitability by NV5 prior to delivery to the site. Imported fill materials should consist of granular soils free from vegetation, debris, or rocks larger than 3 inches in maximum dimension, and the tested Expansion Index value should not exceed 20 (i.e., very low expansion potential). Additionally, import materials should not be considered corrosive as defined by Caltrans (2018) corrosion guidelines and ACI 318. To reduce the potential of importing contaminated materials to the site, prior to delivery, soil materials obtained from off-site sources should be sampled and tested in accordance with standard practice (Department of Toxic Substances Control [DTSC], 2001). Soils that exhibit a known risk to human health, the environment, or both, should not be imported to the site..

- Graded Slopes – Graded slopes should be constructed at a gradient of 2:1 (H:V) or flatter. Fill placed on sloping ground should be keyed and benched in accordance with the recommendations in *Appendix D, Typical Earthwork Guidelines*. To reduce the potential for surface runoff over slope faces, cut slopes should be provided with brow ditches and berms should be constructed at the top of fill slopes.

- Import Soils - Import soils should be sampled and tested for suitability by NV5 prior to delivery to the site. Imported fill materials should consist of clean granular soils free from vegetation, debris, or rocks larger than 3 inches in maximum dimension. The Expansion Index value should not exceed 20 (i.e., very low expansion potential).

8.3 TEMPORARY EXCAVATIONS

Temporary, shallow excavations with vertical side slopes less than 4 feet high will generally be stable, although there is a potential for sloughing. For temporary excavations greater than 4 feet in height, and where working space permits, excavations should be sloped at an inclination ratio flatter than 1.5:1 (horizontal distance: vertical distance) in fill and alluvium and 1:1 in Santiago Formation.

In these soil types, vertical excavations greater than 4 feet high should not be attempted without proper shoring to prevent instabilities. Shoring may be accomplished with hydraulic shores and trench plates, trench boxes, and/or soldier piles and lagging. The actual method of a shoring system should be provided and by a contractor experienced in installing temporary shoring under similar soil conditions and designed by an experienced licensed professional.

All trench excavations and access pits should be shored in accordance with Cal-OSHA regulations. For planning purposes, fill and alluvium may be considered as Type C and Santiago Formation may be considered as Type B, as defined in the current Cal-OSHA soil classification.

The excavation support system should be designed to resist lateral earth pressures of the soil and hydrostatic pressures. It is common practice for an experienced contractor to design and install shoring structure. The preliminary shoring design parameters are provided as follows for reference. The final design of the temporary shoring should be reviewed by the project geotechnical engineer.

For the design of a cantilever soldier piles and lagging shoring system the structure should be designed to resist the lateral earth, water, and surcharge loadings. For the subsurface conditions at this site, the unfactored earth pressure distribution (p in psf) can be calculated as follows:

$$\boldsymbol{P = K.\gamma.H + Surcharge\ 1}$$

Where:

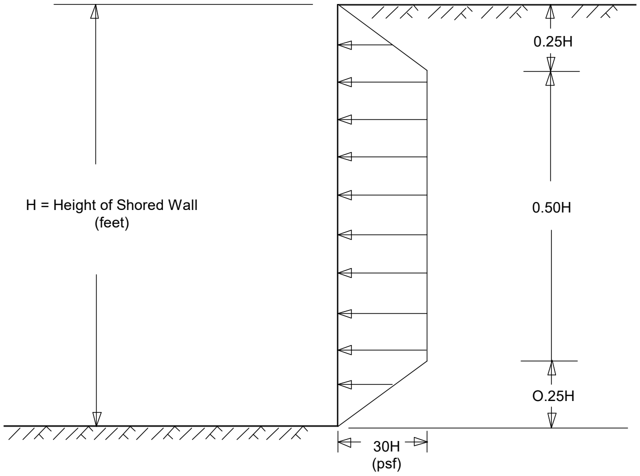
- H= height of the excavation
- ϒ = soil unit weight, where for above water ground is 120 pcf, and for below water level is γ'=57.6 pcf
- K_0=0.55 at-rest earth pressures should be assumed for the geotechnical design, where the wall support does not allow lateral displacement
- K_a=0.38 active earth pressure should be assumed for the geotechnical design, where the wall support allow for lateral yielding
- Surcharge 1: The surcharge for typical construction activities, a minimum of 2 feet equivalent soil surcharge is recommended
- Hydrostatic pressures acting below the groundwater table should be considered in shoring designs.

Stockpiled (excavated) materials should be placed no closer to the edge of a trench excavation than a distance defined by a line drawn upward from the bottom of the trench at an inclination of 1(H):1(V), but no closer than 4 feet. All trench excavations should be made in accordance with Cal-OSHA requirements.

8.4 TEMPORARY SHORING

The following preliminary recommendations are provided for the design of temporary shoring systems. The actual shoring design should be provided by a registered civil engineer in the State of California experienced in the design and construction of shoring under similar conditions. Once the final excavation and shoring plans are complete, the plans and the design should be reviewed by NV5 for conformance with the design intent and geotechnical recommendations. The shoring system should further satisfy requirements of Cal-OSHA.

For design of cantilevered temporary shoring, a triangular distribution of lateral earth pressure may be used. It may be assumed that the subgrade soils, with a level surface behind the cantilevered shoring, will exert an equivalent fluid pressure of 46 pcf. Tied-back or braced shoring should be designed to resist a trapezoidal distribution of lateral earth pressure. The recommended pressure distribution, for the case where the grade is level behind the shoring, is illustrated in the following diagram with the maximum pressure equal to 30H in psf, where H is the height of the shored wall in feet. Hydrostatic pressure should be included, where applicable. To reduce the potential for distress to adjacent structures, we recommend that the shoring system be designed to limit the ground settlement behind the shoring to ½ inch or less.



Any surcharge (live, including traffic, or dead load) located within a 1H:1V plane drawn upward from the base of the shored excavation should be added to the lateral earth pressures. The vertical loads imposed by existing structures, if any, should be determined by the structural engineer. The lateral load contribution of a uniform surcharge load located across the 1H:1V zone behind the excavation may be calculated in accordance with *Figure 7, Lateral Surcharge Loads*. Lateral load contributions of surcharges located at a distance behind the shored wall should be provided by NV5 once the load configurations and layouts are known. As a minimum, a 2-ft equivalent soil surcharge is recommended to account for nominal construction traffic loads.

8.5 DEWATERING

Groundwater was encountered at exploratory boring locations B-1 and B-9 at a depth of 7 and 22 feet, respectively. Contractors should anticipate wet drilling conditions for drilled pole foundations or other excavations that extend near or below the water table. Any cases of seepage or heavy precipitation should be monitored during construction. The actual means and methods of any dewatering scheme should be established by a contractor with local experience. It is important to note that temporary dewatering, if necessary, will require a permit and plan that complies with RWQCB regulations. Based on the subsurface exploration the onsite near-surface soils may be considered to be relatively impermeable.

8.6 EXCAVATION BOTTOM STABILITY

The bottom of excavations near or below groundwater are anticipated to be unstable. In general, unstable bottom conditions may be mitigated by overexcavating the excavation bottom to suitable depths and replacing with gravel wrapped in suitable filter fabric. Recommendations for stabilizing excavation bottoms should be based on evaluation in the field by the geotechnical consultant at the time of construction. However, as a general guideline, overexcavation of approximately 2 to 3 feet may be appropriate to develop a stable excavation bottom.

8.7 FOUNDATIONS

Recommendations for the design and construction of foundation system alternatives are presented below. Footings should be designed and reinforced in accordance with the recommendations of the structural engineer and should conform to the latest edition of the California Building Code.

8.7.1 Design Parameters for Conventional Spread Footings –

In general, conventional footings for proposed building structures should be founded entirely on at least 3 feet of granular compacted fill prepared in accordance with Section 8.2. Foundations should be designed using the geotechnical design parameters presented in Table 5.

Table 5
Geotechnical Design Parameters for Shallow Building Foundations –

Foundation Dimensions	Continuous or spread footing foundations should be 24 inches in width and embedded 24 inches below the lowest adjacent grade. Footing bottoms should bear on at least 3 feet of granular engineered fill.

Net Allowable Bearing Capacity (dead-plus-live load)	2,000 pounds per square foot (psf), which may be increased by 250 psf for each additional foot of depth and by 100 psf for each additional foot of width to a maximum of 3,000 psf, assuming footings are supported on granular compacted fill. A one-third (1/3) increase is allowed for wind or seismic loads.
Reinforcement	Reinforce in accordance with requirements as provided by the project Structural Engineer.
Allowable Coefficient of Friction	0.35 0.10 in the event a vapor barrier is used.
Allowable Lateral Passive Resistance (Equivalent Fluid Pressure)	350 pounds per cubic foot (pcf) per foot of depth. Upper 1 foot should be ignored if not protected by a pavement or slab. A one-third (1/3) increase in passive resistance value may be used for wind and seismic loads. The total allowable lateral resistance may be taken as the sum of the frictional resistance and the passive resistance, provided that the passive bearing resistance does not exceed one-half (1/2) of the total allowable lateral passive resistance.

Note: The above parameters assume level ground or sloping no steeper than 5H:1V.

For smaller, non-building structures such as trash enclosures, freestanding CMU screen walls, the following foundation parameters may be used. Footing should bear entirely on at least 1 foot of granular compacted fill.

Table 6
Geotechnical Design Parameters for Shallow non-Building Foundations –

Foundation Dimensions	Continuous or spread footing foundations should be 24 inches in width and embedded 12 inches below the lowest adjacent grade. Footing bottoms should bear on at least 1 foot of granular engineered fill.

Net Allowable Bearing Capacity (dead-plus-live load)	1,000 pounds per square foot (psf), which may be increased by 250 psf for each additional foot of depth and by 100 psf for each additional foot of width to a maximum of 2,000 psf, assuming footings are supported on granular compacted fill. A one-third (1/3) increase is allowed for wind or seismic loads.
Reinforcement	Reinforce in accordance with requirements as provided by the project Structural Engineer.
Allowable Coefficient of Friction	0.35 0.10 in the event a vapor barrier is used.
Allowable Lateral Passive Resistance (Equivalent Fluid Pressure)	350 pounds per cubic foot (pcf) per foot of depth. Upper 1 foot should be ignored if not protected by a pavement or slab. A one-third (1/3) increase in passive resistance value may be used for wind and seismic loads. The total allowable lateral resistance may be taken as the sum of the frictional resistance and the passive resistance, provided that the passive bearing resistance does not exceed one-half (1/2) of the total allowable lateral passive resistance.

Note: The above parameters assume level ground or sloping no steeper than 5H:1V.

For uplift calculations, the following parameters may be used: soil unit weight of 120 pcf, friction angle of 30 degrees, and zero cohesion. Where footings are located behind retaining walls or near and parallel to major underground utilities, the footings should be extended below a plane projected at a slope of 1H: 1V upward from the bottom of the retaining wall or the underground utility to avoid surcharging the retaining wall or underground utility with building loads.

8.7.2 Settlement

Estimated settlements will depend on the foundation size and depth, supporting materials (mitigation alternatives) and the loads imposed and the allowable bearing values used for design. For preliminary design purposes, based on anticipated column footing dimension of 5 feet width and length or less, the total settlement (static plus seismic) for building foundations loaded in accordance with the net allowable bearing capacities recommended above is estimated to be 1 inch. Differential settlements are anticipated to be 1/2 inch over a distance of 40 feet.

8.7.3 Pole Footing Design (Constrained and Non-Constrained at Grade)

It is assumed that lightpole, flagpole, scoreboard and other pier foundations may be excavated beyond the limits of building pad grading from preliminary grading plans. Due to variable levels of fill at the site, the fill can remain in place and can serve as a foundation support, however the pier should be embedded through the fill into native soils as recommended below.

Table 7
Geotechnical Design Parameters for Pier Foundations –

Foundation Dimensions	Minimum pier diameter should be 18 inches and be embedded a minimum of 5 feet into native soils (either Alluvium or Santiago Formation) below fill soils.
Net Allowable End Bearing Capacity (dead-plus-live load)	Assuming a minimum 8 feet embedment below surface grades, end bearing capacity may be used as 3,000 pounds per square foot (psf). A one-third (1/3) increase is allowed for wind or seismic loads.
Allowable Skin Friction (unit)	250 psf per foot of depth
Allowable Uplift (unit)	150 psf per foot of depth (neglecting pier self-weight)
Reinforcement	Reinforce in accordance with requirements as provided by the project Structural Engineer.
Allowable Lateral Passive Resistance (Equivalent Fluid Pressure)	250 pounds per cubic foot (pcf) per foot of depth. A one-third (1/3) increase in passive resistance value may be used for wind and seismic loads. The upper 2 feet of soil should be neglected unless it is removed and replaced with engineered soil or constrained by concrete at the surface.

For axial loading, we recommend that either skin friction or end bearing be utilized but not both. Total settlement of pole footings as recommended above is estimated to be 1 inch. Differential settlements are anticipated to be 1/2 inch over a horizontal distance of 40 feet.

8.7.4 Mat Foundations

Per correspondence with the design team, it is understood that a reinforced concrete mat foundation is being considered to support the press box building. The thick rigid mat foundation would allow for the entire footprint of the structure to carry building loads. The mat should be designed for a bearing capacity of 1,500 psf, and supported on at least 3 feet of granular engineered fill materials.

A modulus of subgrade reaction (k) of 100 pounds per cubic inch (pci) may be assumed for design of the mat. The k-value is estimated based on theory derived from a 1-foot square bearing plate assumption. Depending on how the subgrade modulus is used in the design, the k-value may need to be modified to reflect differences in foundation size and shape.

Based on the information provided above, it is expected that the total settlements are estimated to be up to 1- inch and differential settlements of ½ inch or less over a distance of 40 feet.

8.7.5 Foundation Observation

Footing excavations should be observed to be clean of loosened soil and debris before placing steel or concrete and “probed” for satisfactory materials at design elevations. If loose/soft soils or unsatisfactory materials are encountered, these materials should be removed and may be replaced with a two-sack, sand-cement slurry or structural concrete. Footing excavations should be deepened as necessary to extend into satisfactory bearing materials; however, NV5 should be notified to review the proposed change.

Drilled pile/pier excavations should be observed by NV5’s geotechnical representative during excavation to check that they extend to the recommended depths and the materials encountered are consistent with the design assumptions. Unlike driven or screwed piles, drilled concrete pile installation will require disposal of cuttings and may need to use steel casing due to potential for caving. No voids should surround the casing. Due to the lateral and skin friction demands on the piles, construction methods should be chosen that ensure the piles and casings are installed tightly within the native undisturbed material. In the event that permanent steel casing is utilized, no frictional resistance for the cased pile zones should be specified. The estimated capacity of the piles relies on a concrete bond between the walls of the drilled shaft and the surrounding soil/rock. It is imperative that the borehole walls not be contaminated with drill cuttings or loose materials. The use of rotator or oscillator methods during drilling will likely allow for cuttings to be trapped between the borehole walls and the drilling rod, which might result in a reduction of the pile capacity. Rotator or oscillator drilling methods should not to be used during the pile construction.

8.7.6 Turf Fields

New turf field construction can be supported on 2 feet of engineered fill as provided in Section 8.2 above, provided the bottom of the removal is scarified to a depth of 8 inches, moisture conditioned to approximately 2 percent over optimum moisture content and recompacted prior to placement of additional fill.

As noted, infiltration at the site is not feasible. Drainage from the fields should be routed to closed pipe drains and/or acceptable retention basins.

8.7.7 Interior Concrete Slabs-on-Grade

Interior concrete slabs-on-grade may be supported at grade on new properly compacted fill in accordance with the recommendations herein, unless otherwise directed by the geotechnical engineer. For design of these concrete slabs, a modulus of subgrade reaction (k) of 100 pci may be used. Floor slabs should be designed and reinforced in accordance with the structural engineer's recommendations. NV5 recommends that interior floor slabs be at least 5 inches thick. In areas where slabs will be covered with moisture-sensitive flooring, it is common practice to place a capillary break consisting of approximately 4 inches of free draining crushed gravel on the finished subgrade soil that, in turn, is overlain by a flexible sheet membrane, such as Stego Wrap™ or an equivalent meeting the requirements of ASTM E1745-09, that serves as a water and/or moisture vapor retarder. The crushed gravel should be graded so that 100 percent passes the 1-inch sieve and less than 5 percent passes the No. 4 sieve. Care should be taken to properly place, lap, and seal the membrane in accordance with the manufacturer's recommendations to provide a vapor tight barrier. Tears and punctures in the membrane should be completely repaired prior to placement of concrete. Subgrade soil located below the vapor retarder should be moisture-conditioned and compacted in accordance with recommendations for Building Pad presented in *Section 8.2 Grading and Earthwork* of this report.

At a minimum, slabs should be reinforced with No. 4 reinforcing bars spaced at 18 inches on-center, each way, placed in the middle one-third of the section, to help control shrinkage cracking of concrete. Reinforcement should be properly placed and supported on "chairs". Welded wire mesh is not recommended. The concrete reinforcement and joint spacing should conform to the minimum requirements of the American Concrete Institute (ACI) section 302.1R and established by the project structural engineer.

8.7.8 Exterior Concrete Slabs-on-Grade

Exterior concrete flatwork should have a minimum concrete thickness of 4 inches. Concrete slabs should be supported on at least 4 inches of Class 2 aggregate base compacted to at least 95 percent of the maximum dry density. The upper 12 inches of subgrade soil located below the aggregate base should be moisture-conditioned and properly compacted in accordance with the recommendations for *Structural Fill Placement (Outside of Building Pad),* presented in *Section 8.2 - Grading and Earthwork* of this report.

Driveway slab areas and vehicular portions of connecting sidewalks should have a minimum concrete thickness of 6 inches. Driveway concrete slabs should be underlain by at least 6 inches of Class 2 aggregate base compacted to at least 95 percent of the maximum dry density. The upper 12 inches of subgrade soil located below the aggregate base should be reconditioned to achieve a moisture content of approximately 2 percent above the optimum moisture content, and recompacted to a minimum of 95 percent relative compaction (ASTM D1557), unless otherwise directed by the geotechnical engineer.

In some cases, isolated "edge" cracking or heaving forms along the outside portions of exterior flatwork because of seasonal or man-made wetting and drying of the subgrade soil. This potential can be reduced by placing lateral cutoffs, i.e., inverted curbs, heavy plastic membranes, or manufactured composite drains, along the outside edges of the flatwork. The lateral cutoffs typically extend vertically 12 to 18 inches into the subgrade soils.

For exterior concrete flatwork, it is recommended that narrow strip concrete slabs, such as sidewalks, be reinforced with at least No. 3 reinforcing bars placed longitudinally at 18 inches on-center. Wide exterior slabs should be reinforced with at least No. 3 reinforcing bars placed 18 inches on-center, each way. The reinforcement should be extended through the control joints to reduce the potential for differential movement. Control joints should be constructed in accordance with recommendations from the structural engineer or architect.

8.8 UTILITY TRENCH BACKFILL

All subsurface utility trench backfill, including water, gas, storm drain, sewer, irrigation, telecommunication, and electrical lines should be properly compacted. Trenches should be placed outside the line of influence from any nearby footing, such that the bottom of the trench is outside a 1:1 (H:V) plane projected from the bottom corner of the footing. Water jetting should not be used for compaction. The material within the pipe zone (i.e. 6 inches below to 12 inches above the pipe) should consist of free-draining sand or small gravel with a minimum sand equivalent of 30. There should be sufficient clearance along the side of the utility pipe or line to allow for compaction equipment. The pipe bedding shall be compacted under the haunches and alongside the pipe. For the design of thrust blocks, please refer to *Figure 8, Thrust Block Detail.*

8.9 CONTROLLED LOW-STRENGTH MATERIAL

If necessary, a Controlled Low-Strength Material (CLSM) may used for backfill under footing once the fill has been removed per Section 8.2. CSLM should have an ultimate compressive strength of 100 psi and should be tested by ASTM D4832 performed at 1 test per 50 cubic yards or fraction thereof.

8.10 RETAINING WALLS

Retaining walls should be designed in accordance with the following recommendations and design parameters:

- Bearing Capacity - The proposed wall may be supported on continuous footings bearing on 3 feet of properly compacted granular fill soils at a minimum depth of 24 inches beneath the lowest adjacent grade. At this depth, footings may be designed for an allowable soil-bearing value of 2,000 psf. This value may be increased by one-third for loads of short duration, such as wind or seismic forces.

- Lateral Earth Pressures - Based on laboratory test results and encountered soil conditions, the recommended lateral earth pressures for preliminary design of flexible retaining walls supported on shallow foundations are summarized in the following Table 8:

Table 8 - Recommended Lateral Earth Pressures

Parameter	Recommended Values				
	Level Backfill	5H:1V Slope	4H:1V Slope	3H:1V Slope	2H:1V Slope
Static Active Earth Pressure (P_a)	37H	43H	45H	49H	62H
Static At-Rest Earth Pressure (P_o)	60H	72H	75H	79H	87H
Seismic Earth Pressure (P_e)	23H	26H	27H	30H	38H
Coefficient of Friction (μ) for Lateral Resistance of Footing	0.35	N/A	N/A	N/A	N/A
Passive Earth Pressure (P_p) for Lateral Resistance of Footing	250H	N/A	N/A	N/A	N/A

Table Notes:
1. All values of height (H) are in feet (ft) and pressure (P) in pounds per square feet (psf).
2. Seismic earth pressure (P_e) is in addition to the static active pressure, P_a and P_o which should be distributed as an regular triangle along the wall height.
3. The above pressure values do not include hydrostatic pressures that might be caused by groundwater or water trapped behind the structure.
4. The pressures listed in the table were based on the assumption that backfill soils will be granular and compacted to 90 percent of maximum dry density (per ASTM D1557).
5. The coefficient of friction (μ) should be applied to dead normal (buoyant) loads when evaluating the sliding frictional resistance.
6. A resistance factor of 0.5 has been applied to the passive earth pressure and may be combined with the sliding frictional resistance using a resistance factor of 0.80. Neglect the upper 6 inches for passive pressure unless the surface is contained by a pavement or a slab. The passive earth pressure should not exceed a maximum value of 3,000 psf.
7. In addition to the above-mentioned pressures, retaining walls must be designed to resist horizontal pressures that may be generated by surcharge loads applied at the ground surface such as from uniform loads or vehicle loads. *Figure 7* may be used to evaluate these surcharge loads.

- Drainage and Waterproofing - Retaining walls should be properly drained, and if desired, appropriately waterproofed. Adequate backfill drainage is essential to provide a free-drained backfill condition and to reduce the potential for the development of hydrostatic pressure buildup behind walls. Drainage behind the retaining walls may be provided with geosynthetic drainage composite such as TerraDrain, MiraDrain, or equivalent, placed continuously along the back of the wall and connected to a 4-inch diameter perforated pipe. The pipe should be sloped at least 2 percent and surrounded by 3 cubic feet per foot of ¾-inch crushed rock wrapped in suitable non-woven filter fabric (Mirafi 140N or equivalent) or Caltrans Class 2 permeable granular filter materials. The crushed rock should meet the requirements defined in Section 200-1.2 of the latest edition of the Standard Specification for Public Works Construction (Greenbook). These drains should be connected to an adequate discharge system. Retaining wall drainage details are included in *Appendix D.*

- Retaining Wall Backfill Compaction - Retaining wall backfill material should be non-expansive (E.I. of 20 or less) and free draining. Backfill should be brought to near-optimum moisture conditions and compacted by mechanical means to at least 90 percent relative compaction

(ASTM D1557). Care should be taken when selecting/using compaction equipment in close proximity to retaining walls so that the walls are not damaged by excessive loading.

8.11 PAVEMENTS

Design of asphalt concrete pavement sections depends primarily on support characteristics (strength) of soil beneath the pavement section and on cumulative traffic loads within the service life of the pavement. Strength of the pavement subgrade is represented by R-value test data. An R-value test was performed from the previous geotechnical report on a representative sample of the near-surface soil with a result of 22. A summary of laboratory test results are included in *Appendix C, Laboratory Test Results*.

Traffic loads within service life of a pavement are represented by a Traffic Index (TI), which is calculated based on anticipated traffic loads and on the projected number of load repetitions during the design life of the pavement. The design TI value should be verified by the project Civil/Traffic Engineer.

Preliminary pavement section recommendations were developed using a design R-value of 15 and Traffic Index (TI) values assumed for light auto/maintenance vehicles and drive lanes. Based on these design parameters, analysis in accordance with California Department of Transportation (Caltrans) Highway Design Manual, and assuming compliance with site preparation recommendations, NV5 recommends the flexible and rigid structural pavement sections presented in the following Table 9.

Table 9 - Recommended Pavement Sections (Design R-value = 15)

Location	Flexible Pavement (inches)		Rigid Pavement (inches)	
	Hot-Mix Asphalt (HMA)	Aggregate Base (AB)	Jointed Plain Portland Cement Concrete (JPCP)	Aggregate Base (AB)
Light Auto Parking and Drive Lanes (TI=5-6)	3.5	9.0	6.0	6.0
Fire Lanes (TI=7-8)	4.5	14.0	6.0	12.0

Assuming that the near-surface on-site soils will be thoroughly mixed and compacted during grading operations, it is recommended that R-value testing be performed on representative soil samples after rough grading operations on the upper 2 feet to confirm applicability of the above pavement sections. If the paved areas are to be used during construction, or if the type and frequency of traffic is greater than assumed in the design, the pavement section should be re-evaluated for the anticipated traffic.

The upper 12 inches of subgrade soils should be moisture conditioned to approximately 2 percent above the optimum moisture content (unless otherwise directed by the geotechnical engineer), and compacted to a minimum dry density of 95 percent of the materials maximum density as determined by the ASTM D1557 test procedure. The aggregate base should conform to Class II aggregate base in accordance with Section 400.2.3 of the latest Regional Supplement to Greenbook Standard Specifications for Public Works Construction. The base course should also be compacted to a minimum dry density of 95 percent. Field and lab testing should be used to check compaction, aggregate gradation, and compacted thickness.

The asphalt pavement should be compacted to 95 percent of the unit weight as tested in accordance with the Hveem procedure (ASTM D1560). The maximum lift thickness should be 4.0 inches. The asphalt material shall conform to Type III, Class B2 or B3 of the Standard Specifications for Public Works Construction and the supplement. An approved mix design should be submitted 30 days prior to placement. The mix design should include proportions of materials, maximum density and required lay-down temperature range. Field and lab testing should be used to verify oil content, aggregate gradation, compaction, compacted thickness, and lay-down temperature.

Control joints are required for the Portland cement concrete pavement (rigid) at a maximum of 15 feet spacing each way and should be constructed immediately after concrete finishing.

The performance of pavements is highly dependent upon providing positive surface drainage away from the edge of the pavement. The ponding of water on or adjacent to pavement areas will likely cause failure of the subgrade and resultant pavement distress. Where planters are proposed, the perimeter curb should extend at least 6 inches below the subgrade elevation of the adjacent pavement. In addition, experience indicates that even with these provisions, a saturated subgrade condition can develop as a result of increased irrigation, landscaping and surface runoff. A subdrainage system should be considered along the perimeter of pavement subgrade areas to reduce the potential of this condition developing. The subdrain system should be designed to intercept irrigation water and surface runoff prior to entry into the pavement subgrade and carry the water to a suitable outlet.

8.12 CORROSION POTENTIAL

The California Department of Transportation (Caltrans) Corrosion Guidelines (Version 3.0, dated March 2018) considers a site to be corrosive to structural elements "if one or more of the following conditions exist for the representative soil and/or water samples taken at the site: Chloride concentration is 500 ppm or greater, sulfate concentration of 1,500 ppm or greater, or the pH of 5.5 or less". Minimum resistivity in soil or water is considered an indicator parameter and is not used to define a corrosive soil environment. Caltrans' Guidelines state that a "minimum resistivity value for soil and/or water less than 1,100 Ohm-cm indicates the presence of high quantities of soluble salts and a higher propensity for corrosion".

Representative samples of the site soils obtained from the borings were tested to evaluate the corrosion potential. The tests include pH, electrical resistivity, and soluble chloride and sulfate concentrations. Results of the corrosivity tests performed are summarized in the Table 10 below and presented in *Appendix C, Laboratory Test Results.*

Table 10 - Corrosivity Test Results

Boring Location and Sample Depth (ft.)	Soil Type	pH	Minimum Resistivity (Ohm-cm)	Water Soluble Sulfate Content (ppm)	Water Soluble Chloride Content (ppm)
B-2 @ 1 – 5	Sandy Lean CLAY (CL)	8.3	490	170	440

Based on experience and Caltrans Corrosion Guidelines, the site soils are considered corrosive. Based on the ACI 318 criteria, the potential for sulfate attack is considered negligible for water-soluble sulfate contents in soil ranging from 0 to 0.10 percent by weight (0 to 1,000 ppm), indicating that soils underlying the site may be considered to have a negligible potential for sulfate attack. However, due to the potential for variability of on-site soils, we recommend that Type II/V cement be used for concrete in contact with soil with a water-cement ratio no higher than 0.45 by weight for the project. Exposure category for concrete in contact with soil per ACI-318 may be taken as F0, S2, W1, and C1.

As indicated in the 2006 edition (second edition) of "Corrosion Basics - An Introduction", a general guideline for soil resistivity and corrosion-severity ratings is presented in the following Table 11:

Table 11 - Soil Resistivity Versus Corrosion Severity

Soil Resistivity	Corrosivity
<1,000 ohm-cm	Extremely Corrosive
1,000 to 3,000 ohm-cm	Highly Corrosive
3,000 to 5,000 ohm-cm	Corrosive
5,000 to 10,000 ohm-cm	Moderately Corrosive
10,000 to 20,000 ohm-cm	Mildly Corrosive
>20,000 ohm-cm	Essentially Noncorrosive

Soil resistivity is not the only parameter affecting the risk of corrosion damage; and a high soil resistivity will not guarantee the absence of serious corrosion. For example, the American Water Works Association (AWWA) has developed a numerical soil-corrosivity scale, applicable to cast-iron alloys. The test results do suggest the potential for soils to be extremely corrosive to ferrous metal pipes.

Any imported soils should be evaluated for corrosion characteristics if they will be in contact with buried or at-grade structures and appropriate mitigation measures should be included in the structure design. It is recommended that a corrosion specialist be contacted to determine if mitigation measures are necessary.

8.13 DRAINAGE CONTROL

Although not all of the recommendations may be applicable to this project, the intent of this section is to provide general information regarding the control of surface water. The control of surface water is essential to the satisfactory performance of the building and site improvements. Surface water should be controlled so that conditions of uniform moisture are maintained beneath the structure, even during periods of heavy rainfall. The following recommendations are considered minimal.

- Berms, drainage swales, catch basins, and storm water drainage pipe should be installed along all existing top-of-slope areas within the project limits, as a minimum erosion control measure.

- Ponding and areas of low flow gradients should be avoided.
- If bare soil within 5 feet of the structure is not avoidable, then a gradient of 5 percent or more should be provided sloping away from the improvement. Corresponding paved surfaces should be provided with a gradient of at least 1 percent.
- The remainder of the unpaved areas should be provided with a drainage gradient of at least 2 percent.
- Positive drainage devices, such as graded swales, paved ditches, and/or catch basins should be employed to accumulate and to convey water to appropriate discharge points.
- Concrete walks and flatwork should not obstruct the free flow of surface water.
- Brick flatwork should be sealed by mortar or be placed over an impermeable membrane.
- Area drains should be recessed below grade to allow free flow of water into the basin.
- Enclosed raised planters should be sealed at the bottom and provided with an ample flow gradient to a drainage device. Recessed planters and landscaped areas should be provided with area inlet and subsurface drain pipes.
- Planters should not be located adjacent to the structure wherever possible. If planters are to be located adjacent to the structure, the planters should be positively sealed, should incorporate a subdrain, and should be provided with free discharge capacity to a drainage device.
- Planting areas at grade should be provided with positive drainage. Wherever possible, the grade of exposed soil areas should be established above adjacent paved grades. Drainage devices and curbing should be provided to prevent runoff from adjacent pavement or walks into planted areas.
- Gutter and downspout systems should be provided to capture discharge from roof areas. The accumulated roof water should be conveyed to off-site disposal areas by a pipe or concrete swale system.
- Landscape watering should be performed judiciously to preclude either soaking or desiccation of soils. The watering should be such that it just sustains plant growth without excessive watering. Sprinkler systems should be checked periodically to detect leakage and they should be turned off during the rainy season.

9.0 DESIGN REVIEW AND CONSTRUCTION MONITORING

Geotechnical review of plans and specifications is of paramount importance in engineering practice. The poor performance of many structures has been attributed to inadequate geotechnical review of construction documents. Additionally, observation and testing of the subgrade will be important to the performance of the proposed improvements. The following sections present recommendations relative to the review of construction documents and the monitoring of construction activities.

9.1 PLANS AND SPECIFICATIONS

The design plans and specifications should be reviewed by NV5 prior to bidding and construction, as the geotechnical recommendations may need to be reevaluated in consideration of the actual design configuration. This review is necessary to evaluate whether the recommendations contained in this report and future reports have been properly incorporated into the project plans and specifications.

9.2 CONSTRUCTION MONITORING

Site preparation, removal of unsuitable soils, assessment of imported fill materials, fill placement, and other earthwork operations should be observed and tested. The substrata exposed during the construction may differ from that encountered in the test borings. Continuous observation by a representative of NV5 during construction allows for evaluation of the soil/rock conditions as they are encountered, and allows the opportunity to recommend appropriate revisions where necessary.

10.0 LIMITATIONS

The recommendations and opinions expressed in this report are based on NV5's review of background documents and on information obtained from field explorations. It should be noted that this study did not evaluate the possible presence of hazardous materials on any portion of the site.

Due to the limited nature of the field explorations, conditions not observed and described in this report may be present on the site. Uncertainties relative to subsurface conditions can be reduced through additional subsurface exploration. Additional subsurface evaluation and laboratory testing can be performed upon request. It should be understood that conditions different from those anticipated in this report may be encountered during grading operations, e.g., the extent of removal of unsuitable soil, and that additional effort may be required to mitigate them.

Site conditions, including ground-water level, can change with time as a result of natural processes or the activities of man at the subject site or at nearby sites. Changes to the applicable laws, regulations, codes, and standards of practice may occur as a result of government action or the broadening of knowledge. The findings of this report may, therefore, be invalidated over time, in part or in whole, by changes over which NV5 has no control.

NV5's recommendations for this site are, to a high degree, dependent upon appropriate quality control of subgrade preparation, fill placement, and foundation construction. Accordingly, the recommendations are made contingent upon the opportunity for NV5 to observe grading operations and foundation excavations for the proposed construction. If parties other than NV5 are engaged to provide such services, such parties must be notified that they will be required to assume complete responsibility as the geotechnical engineer of record for the geotechnical phase of the project by concurring with the recommendations in this report and/or by providing alternative recommendations.

This document is intended to be used only in its entirety. No portion of the document, by itself, is designed to completely represent any aspect of the project described herein. NV5 should be contacted if the reader requires additional information or has questions regarding the content, interpretations presented, or completeness of this document.

NV5 has endeavored to perform its evaluation using the degree of care and skill ordinarily exercised under similar circumstances by reputable geotechnical professionals with experience in this area in similar soil conditions. No other warranty, either expressed or implied, is made as to the conclusions and recommendations contained in this report.

11.0 SELECTED REFERENCES

American Concrete Institute (ACI), 2014, ACI 318 Building Code Requirements for Structural Concrete and Commentary.

American Society of Civil Engineers / Structural Engineering Institute (ASCE/SEI), 2016; Minimum Design Loads for Buildings and Other Structures, ASCE/SEI 7-16.

ASTM, 2001; Soil and Rock: American Society for Testing and Materials: vol. 4.08 for ASTM test methods D-420 to D-4914; and vol. 4.09 for ASTM test methods D-4943 to highest number.

Boulanger, R.W. and Idriss, I.M., 2014; "CPT and SPT Based Liquefaction Triggering Procedures," University of California Davis, Center for Geotechnical Modeling Report No. UCD/CGM-14/01, pp. 1-134

Bowles, J.E.; 1997; Foundation Analysis and Design, 5th Edition.

Building News, 2018, “Greenbook,” Standard Specifications for Public Works Construction: BNI Publications.

CBC, 2019, California Building Code.

California Department of Transportation (Caltrans), 2018, Corrosion Guidelines (Version 3.0), Division of Engineering and Testing Services, Corrosion Technology Branch: dated March.

California Department of Conservation, Division of Mines and Geology, 1997; Guidelines for Evaluation and Mitigation of Seismic Hazards in California: Special Publication 117, 74 pp.

California Department of Conservation, Division of Mines and Geology, 1998; Maps of Known Active Fault Near-Source Zones in California and Adjacent Portions of Nevada: International Conference of Building Officials, dated February, Scale 1” = 4 km.

California Environmental Protection Agency, 2020, (waterboards.ca.gov) GeoTracker

California Geological Survey, 2008a; Special Publication 117A, Guidelines for Evaluating and Mitigating Seismic Hazards in California

California Geological Survey, 2008ba, “Appendix A: California Fault Parameters for the National Seismic Hazard Maps and Working Group on California Earthquake Probabilities 2007,” Special Report 203A.

California Geological Survey, 2010; Fault Activity Map of California.

Department of Toxic Substances Control, 2001, Information Advisory – Clean Imported Fill Material Fact Sheet, https://dtsc.ca.gov/information-advisory-clean-imported-fill-material-fact-sheet/: dated October.

FEMA, Flood Map Service Center, msc.fema.gov, panel number 06073C0793G.

Kennedy, M.P. & Tan, S.S., 2007, Geologic Map of the Oceanside 30’ x 60’ Quadrangle, California.

Hart, E.W., and Bryant, W.A., 1997; Fault-Rupture Hazard Zones in California, Alquist-Priolo Earthquake Fault Zoning Act with Index to Earthquake Fault Zone Maps: California Department of Conservation, Division of Mines and Geology Special Publication 42, 38 pp.

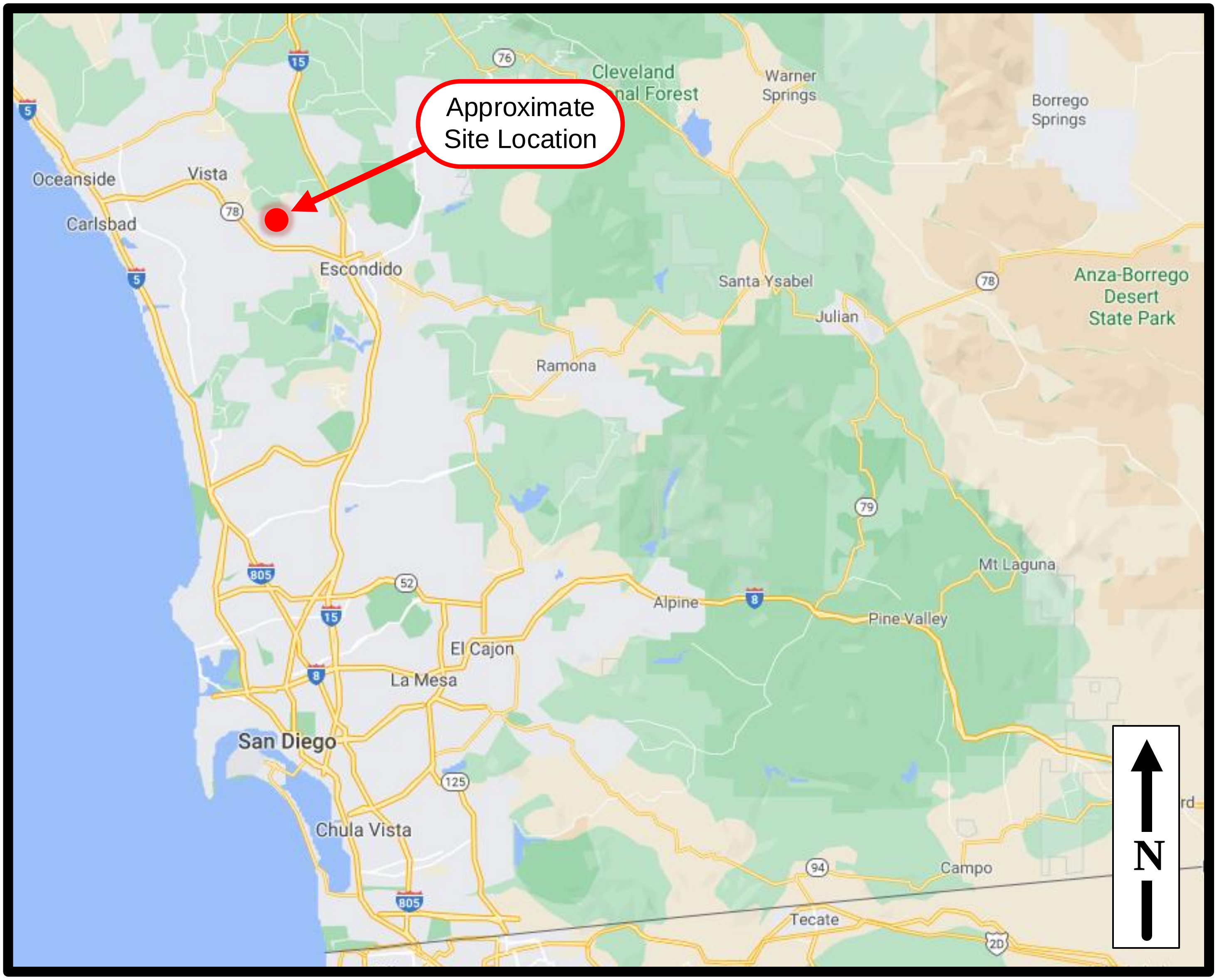
San Marcos General Plan, Part 6 Safety Element, updated 2012.

Southern California Earthquake Center, 2002; Recommended Procedures for Implementation of DMG Special Publication 117 Guidelines for Analyzing and Mitigating Landslide Hazards in California: dated March, 127 pp.

USGS Seismic Ground Motion Design Parameter Tool, https://seismicmaps.org/

USGS Probabilistic Seismic Hazards Deaggregation Tool, http://geohazards.usgs.gov/deaggint/2008/.

FIGURES



NOTE: Map Not to Scale.

Reference: Google Maps 2021

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Project No:	**113821-0001310.00**
Drafted By:	**A. Hespeler**
Date:	**April 2021**

SITE LOCATION MAP
Proposed Softball & Football Fields and
Field House – Palomar Community College
1140 W. Mission Road
San Marcos, CA

FIGURE
1

Approximate Scale
1 inch = 800 feet

(When printed on 8.5" by 11'' paper.)

Reference: Nearmap.com Imagery 2021

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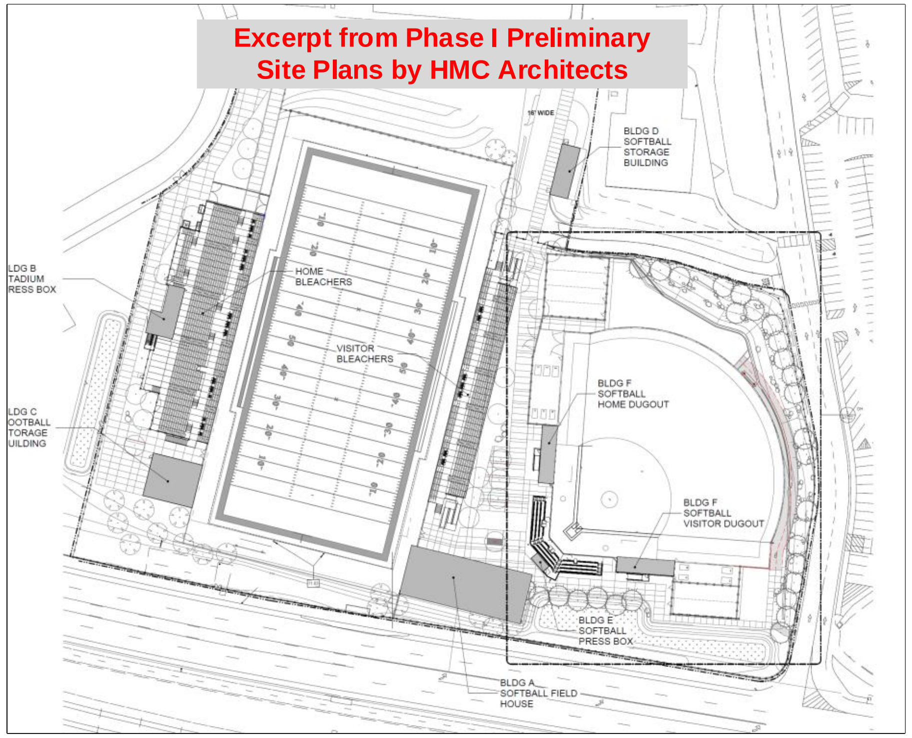
Date:	**April 2021**

VICINITY MAP
Proposed Softball & Football Fields and
Field House – Palomar Community College
1140 W. Mission Road
San Marcos, CA

FIGURE
2

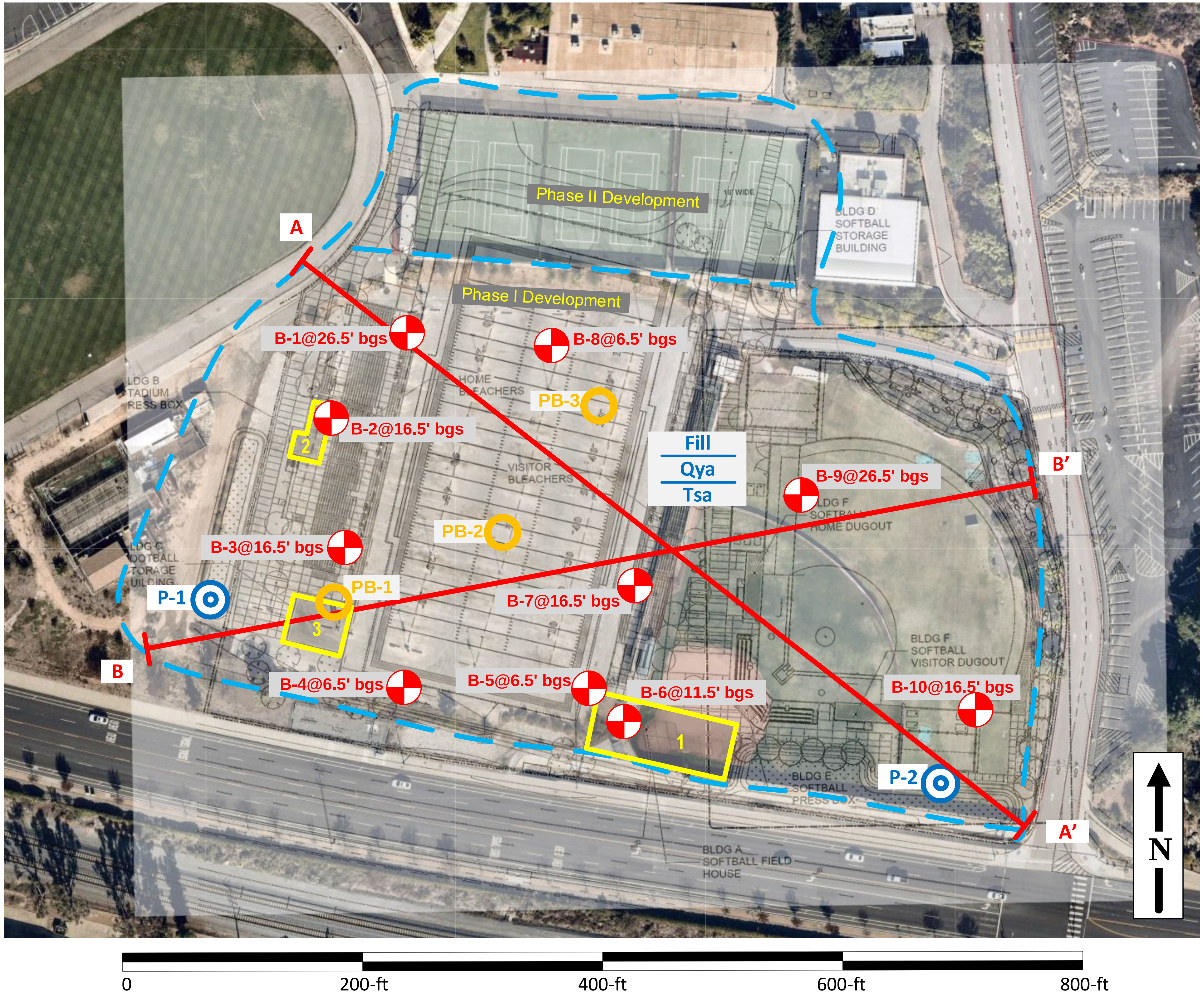
LEGEND

B-10 ◐	Approximate location of exploratory boring.
P-2 ◎	Approximate location of percolation test.
PB-3 ○	Approximate location of previous exploratory boring drilled in September 2015.
B B'	Approximate location of geologic cross-section.
▬ ▬	Approximate limits of area under the purview of this investigation.
▬	Approximate location and footprint of proposed new athletics structures (denoted as follows): 1. "Bldg. A – Softball Field House" 2. "Bldg. B – Stadium Press Box" 3. "Bldg. C – Football Storage Building"
Fill Qya Tsa	Indicates geologic units encountered (stacking indicates vertical position).



Excerpt from Phase I Preliminary Site Plans by HMC Architects

NOTE: This map is intended for geotechnical information only. All locations and dimensions are approximate. Actual dimensions and locations of proposed structures should be obtained from the approved project plans.



Approximate Scale:
1" = 100'
(When printed on 11" by 17" paper.)

Reference: Nearmap.com Imagery 2021

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GEOTECHNICAL EXPLORATION MAP
Proposed Softball & Football Fields and
Field House – Palomar Community College
1140 W. Mission Road
San Marcos, CA

FIGURE
3

Description of Map Units

Qya	Young alluvial flood-plain deposits (Holocene and late Pleistocene)
Tsa	Santiago Formation (middle Eocene)
Kt	Tonalite, undivided (mid-Cretaceous)

EXCERPT FROM THE *"GEOLOGIC MAP OF THE OCEANSIDE 30' X 60' QUADRANGLE, CALIFORNIA"*
By: Kennedy, M.P. & Tan, S.S., 2007

NOTE: Map Not to Scale.

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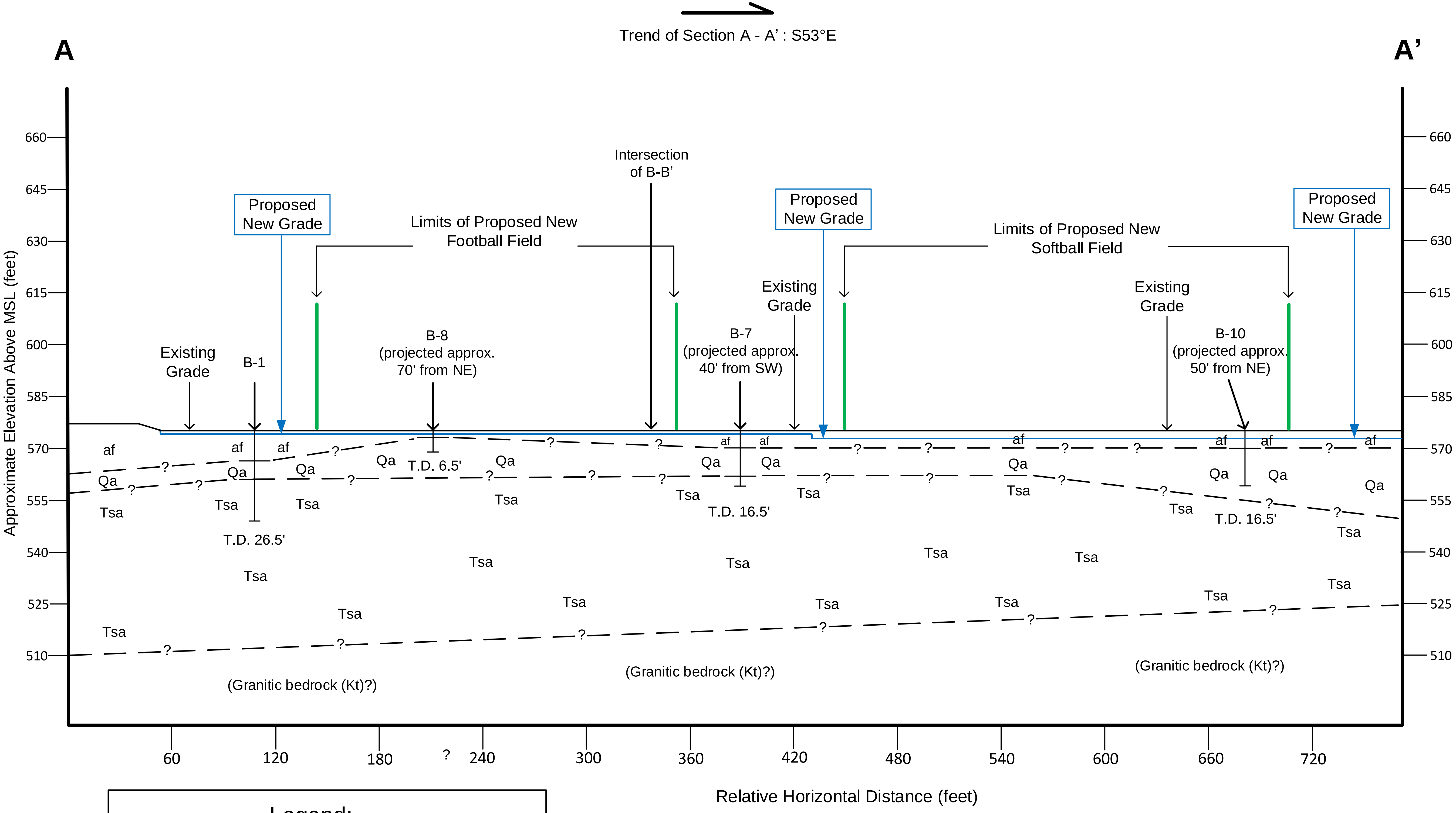
Date: **April 2021**

REGIONAL GEOLOGIC MAP
Proposed Softball & Football Fields and
Field House – Palomar Community College
1140 W. Mission Road
San Marcos, CA

FIGURE
4



Legend:

B-1	Approximate Location of Exploratory Boring
—?—	Approximate Geologic Contact
T.D.	Termination Depth of Boring
af	Fill
Qa	Alluvium
Tsa	Santiago Formation

Horizontal Scale: 1" = 60'
Vertical Scale: 1" = 30'
Vertical Exaggeration = x2
(When printed on 11" by 17" paper.)

NOTE: This Cross-Section is intended for geotechnical information only. All locations and dimensions are approximate. Actual dimensions and locations of proposed structures and proposed grades should be obtained from the approved project plans.

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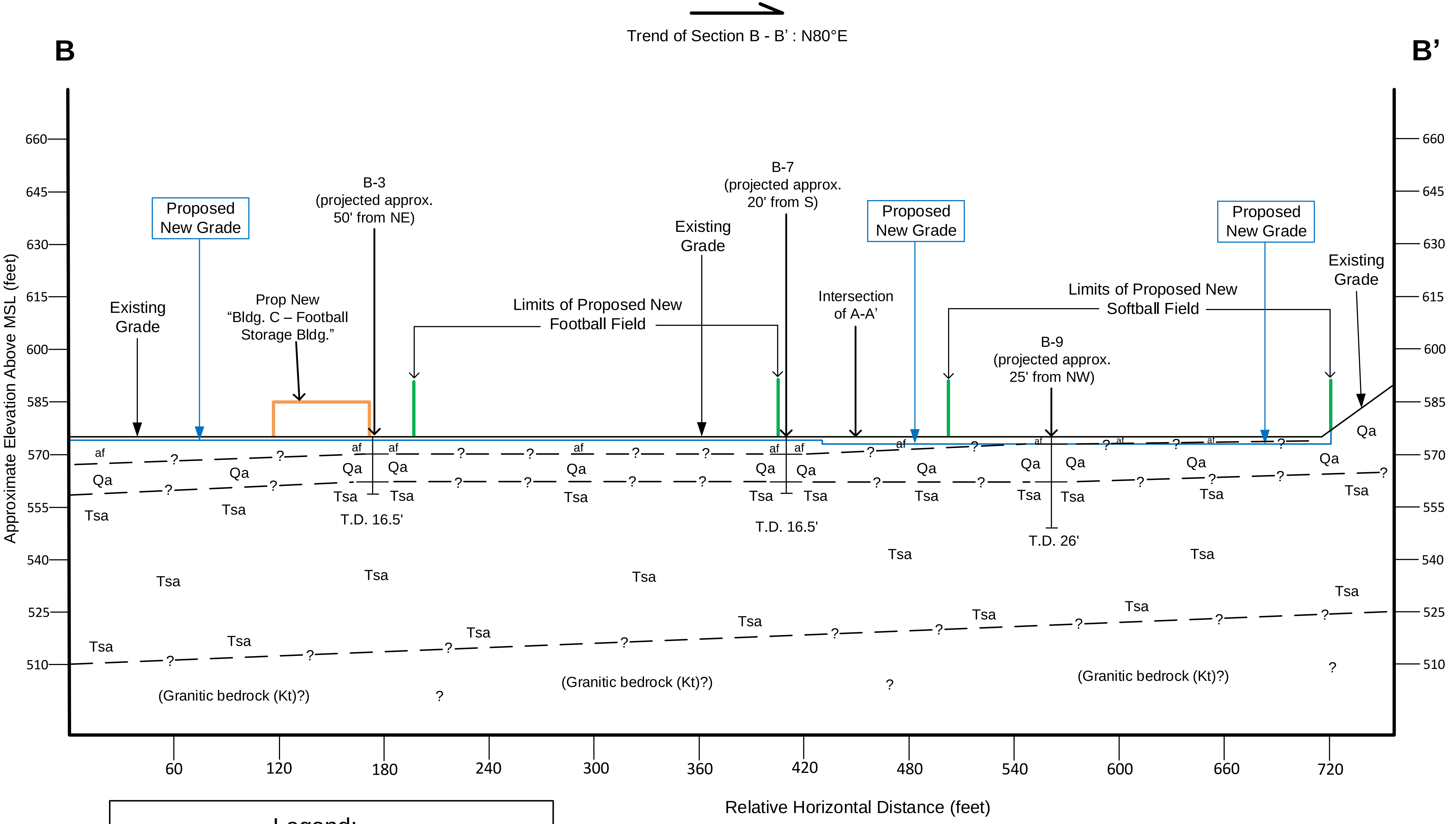
GEOLOGIC CROSS SECTION A-A'
Proposed Softball & Football Fields and
Field House – Palomar Community College
1140 W. Mission Road
San Marcos, CA

FIGURE
5a



Legend:

B-1	Approximate Location of Exploratory Boring
—?—	Approximate Geologic Contact
T.D.	Termination Depth of Boring
af	Fill
Qa	Alluvium
Tsa	Santiago Formation

Horizontal Scale: 1" = 60'
Vertical Scale: 1" = 30'
Vertical Exaggeration = x2
(When printed on 11" by 17" paper.)

NOTE: This Cross-Section is intended for geotechnical information only. All locations and dimensions are approximate. Actual dimensions and locations of proposed structures and proposed grades should be obtained from the approved project plans.

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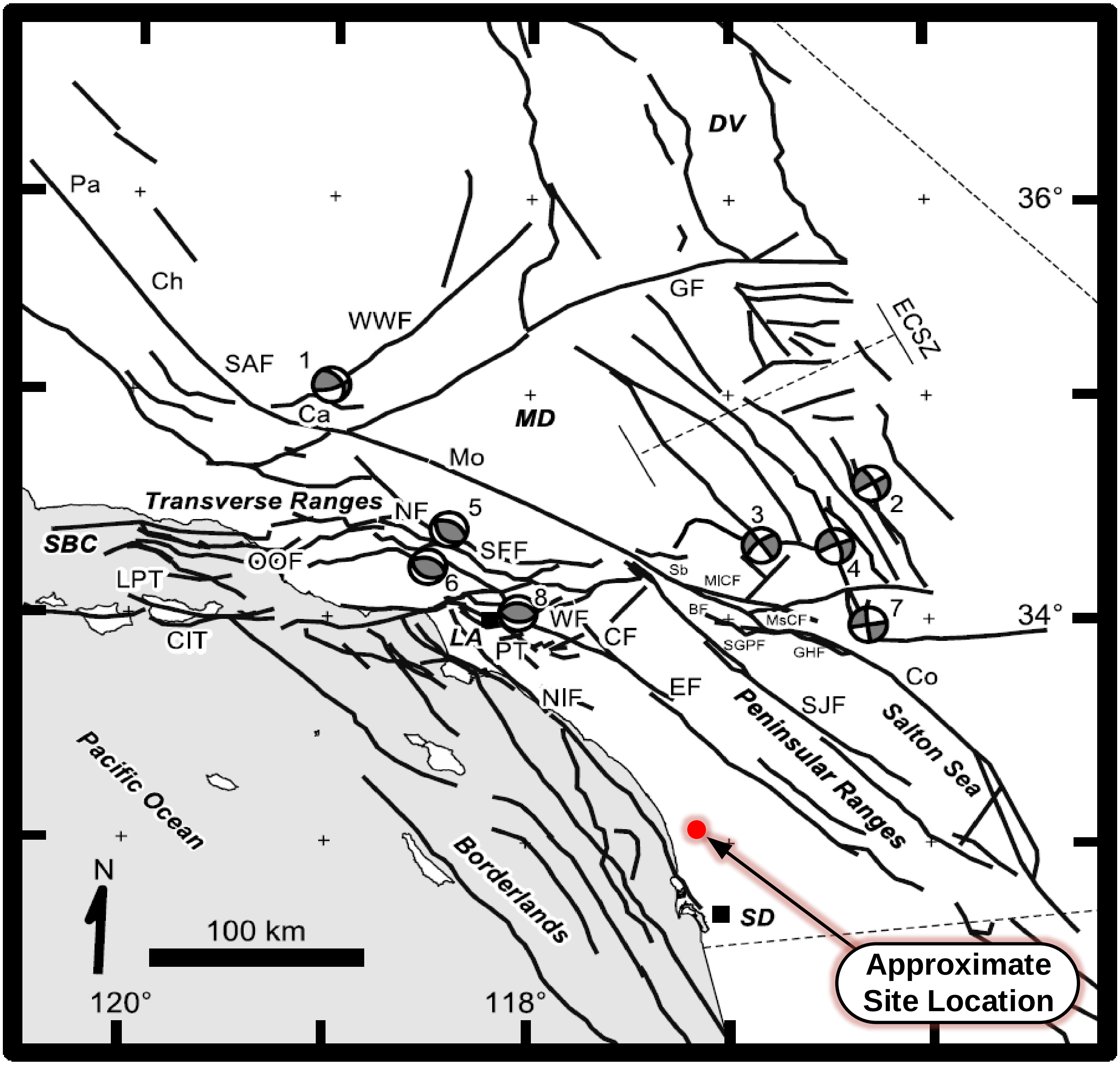
GEOLOGIC CROSS SECTION B-B'
Proposed Softball & Football Fields and
Field House – Palomar Community College
1140 W. Mission Road
San Marcos, CA

FIGURE
5b



Map of southern California showing the geographic regions, faults and focal mechanisms of the more significant earthquakes. **Regions:** Death Valley, DV; Mojave Desert MD; Los Angeles, LA; Santa Barbara Channel, SBC; and San Diego, SD. **Indicated Faults:** Banning fault, BF; Channel Island thrust, CIT; Chino fault, CF; Eastern California Shear Zone, ECSZ; Elsinore fault, EF; Garlock fault, GF; Garnet Hill fault, GHF; Lower Pitas Point thrust, LPT; Mill Creek fault, MICF; Mission Creek fault, MsCF; Northridge fault, NF; Newport Inglewood fault, NIF; offshore Oak Ridge fault, OOF; Puente Hills thrust, PT; San Andreas fault (sections: Parkfield, Pa; Cholame, Ch; Carrizo; Ca; Mojave, Mo; San Bernardino, Sb; and Coachella, Co); San Fernando fault, SFF; San Gorgonio Pass fault, SGPF; San Jacinto fault, SJF; Whittier fault, WF; and White Wolf fault, WWF. **Earthquake Focal Mechanisms:** 1952 Kern County, 1; 1999 Hector Mine, 2; 1992 Big Bear, 3; 1992 Landers, 4; 1971 San Fernando, 5; 1994 Northridge, 6; 1992 Joshua Tree, 7; and 1987 Whittier Narrows, 8.

Reference: Plesch, Anndreas et. al., 2007, Community Fault Model (CFM) for Southern California; in the *Bulletin of the Seismological Society of America*, Vol. 97, No. 6. pp. 1793-1802, dated December.

NOTE: For Schematic Use Only-Not a Construction Drawing

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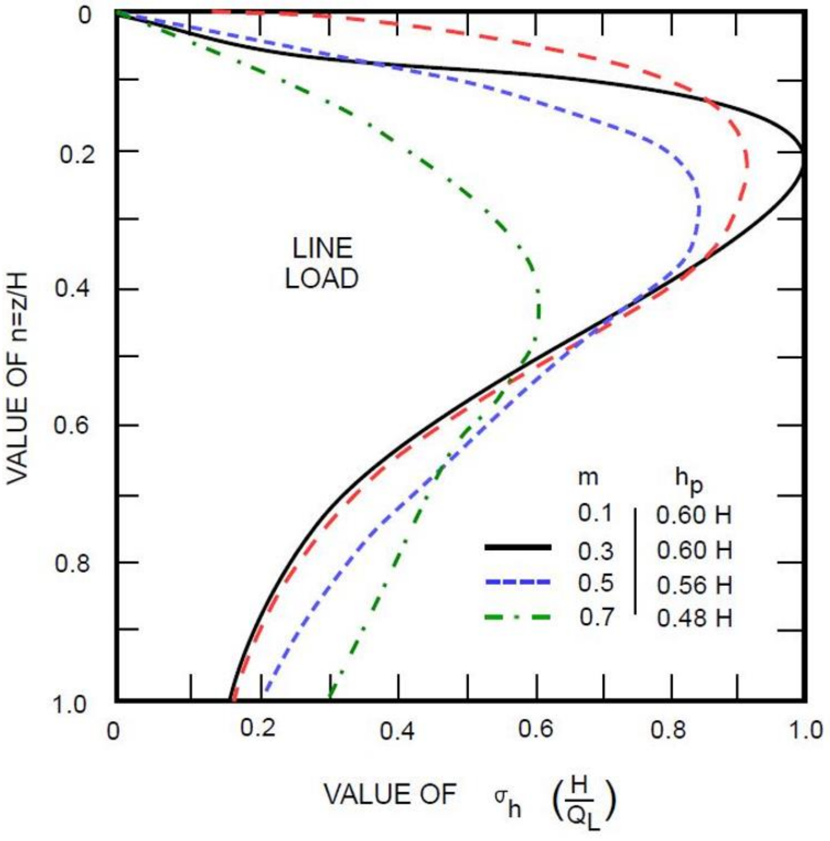
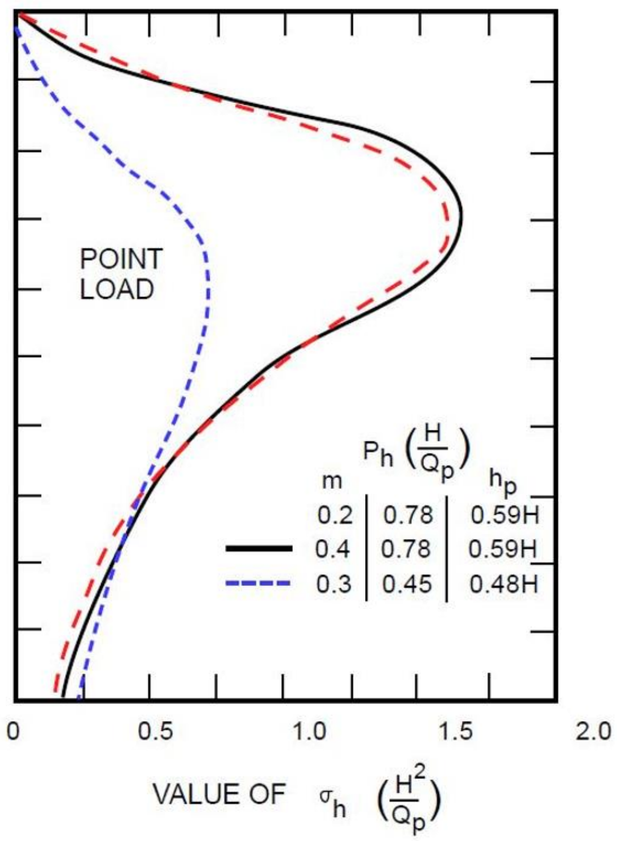
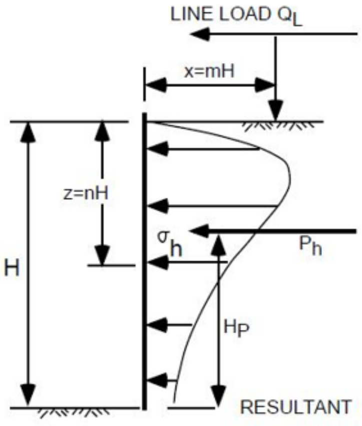
Date: **April 2021**

REGIONAL FAULT MAP
Proposed Softball & Football Fields and
Field House – Palomar Community College
1140 W. Mission Road
San Marcos, CA

FIGURE
6







FOR m $\leqq$ 0.4 :

$$\sigma_h \left(\frac{H}{Q_L}\right) = \frac{0.20\, n}{(0.16 + n^2)^2}$$

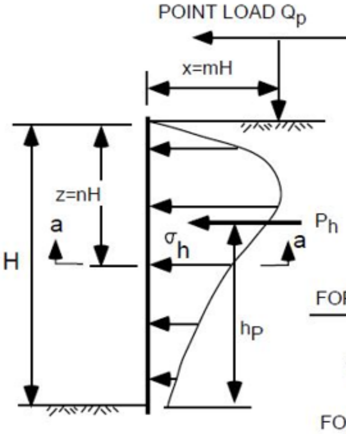
$P_H = 0.55\, Q_L$

FOR m > 0.4 :

$$\sigma_h \left(\frac{H}{Q_L}\right) = \frac{1.28\, m^2 n}{(m^2 + n^2)^2}$$

$$\text{RESULTANT } P_H = \frac{0.64\, Q_L}{(m^2 + 1)}$$

PRESSURE FROM LINE LOAD Q_L
(BOUSSINESQ EQUATION MODIFIED BY EXPERIMENT)



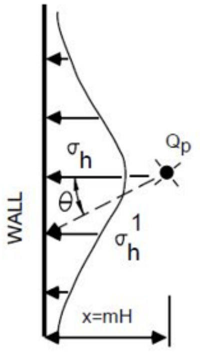
FOR m $\leqq$ 0.4 :

$$\sigma_h \left(\frac{H^2}{Q_p}\right) = \frac{0.28\, n^2}{(0.16 + n^2)^3}$$

FOR m > 0.4 :

$$\sigma_h \left(\frac{H^2}{Q_p}\right) = \frac{1.77\, m^2 n^2}{(m^2 + n^2)^3}$$

$$\sigma_h^1 = \sigma_h \cos 2\,(1.1\theta)$$



SECTION a ~a
PRESSURE FROM POINT LOAD Q_p
(BOUSSINESQ EQUATION MODIFIED BY EXPERIMENT)

N|V|5

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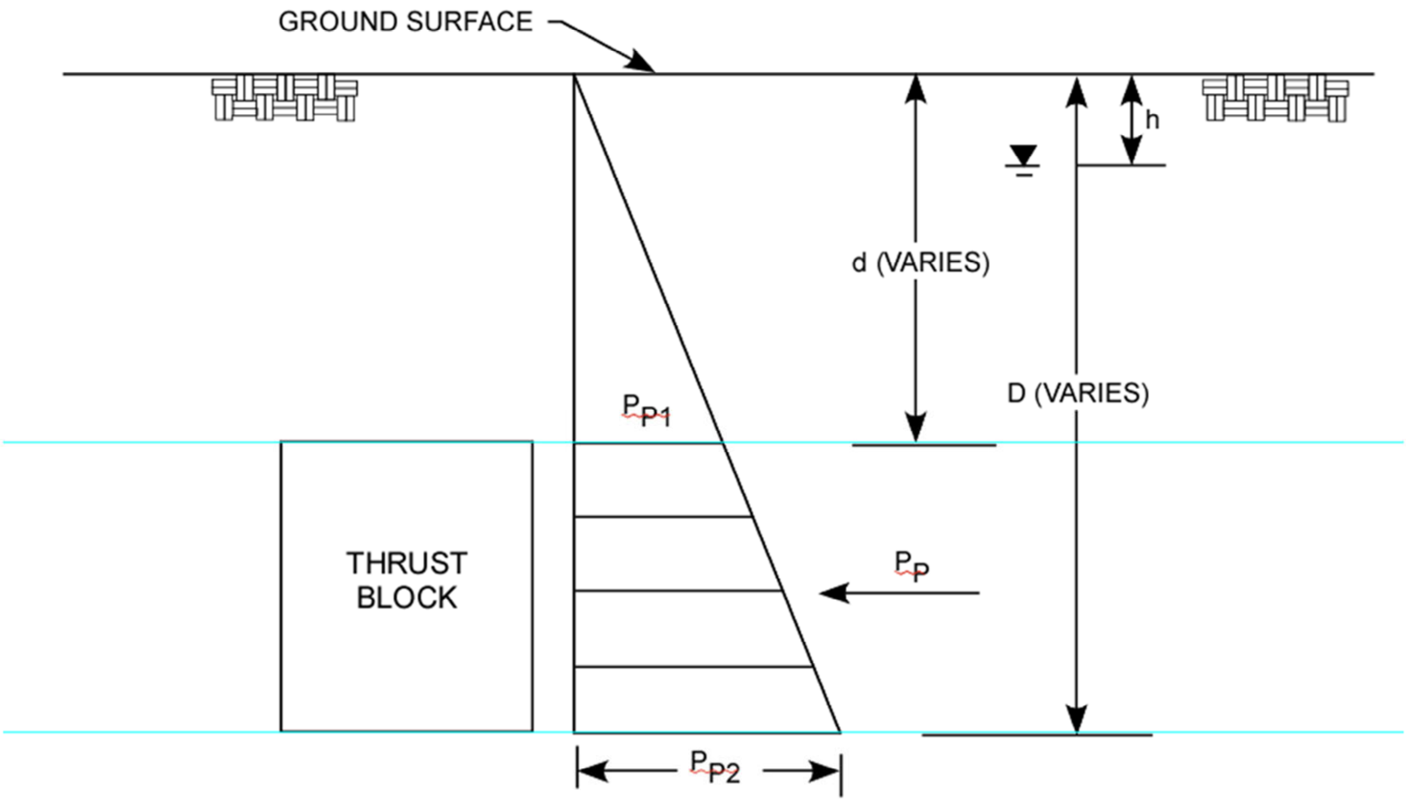
Drafted By: **A. Hespeler**
Date: **April 2021**

LATERAL SURCHARGE LOADS
Proposed Softball & Football Fields and
Field House – Palomar Community College
1140 W. Mission Road
San Marcos, CA

FIGURE 7



Notes:

1. Groundwater Below block:
   $Pp = 175 (D^2-d^2)$ lb/ft

2. Groundwater Above block:
   $Pp = 1.4 (D-d)[115h + 52(D-d)]$ lb/ft

3. Assumes Backfill is Granular Material

4. Assumes Thrust Block is adjacent to competent material

5. D, d, and h are in feet

6. $\underline{\blacktriangledown}$ Groundwater Table

N|V|5

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Date: **April 2021**

THRUST BLOCK DETAIL
Proposed Softball & Football Fields and
Field House – Palomar Community College
1140 W. Mission Road
San Marcos, CA

FIGURE
8

APPENDIX A

Previous NV5 Geotechnical Investigation

(Performed in September 2015)

GEOTECHNICAL INVESTIGATION
PALOMAR COMMUNITY COLLEGE DISTRICT
PROPOSED TEMPORARY PARKING LOT
SAN MARCOS, CALIFORNIA

ENVIRONMENTAL
GEOTECHNICAL / CONSTRUCTION MATERIALS TESTING
INFRASTRUCTURE ENGINEERING
MUNICIPAL SERVICES
ASSET MANAGEMENT / CERTIFICATION

September 3, 2015

NV5 West, Inc.
15092 Avenue of Science, Suite 200
San Diego, California 92128
(858) 715-5800
www.NV5.com

A NV5 Company – Offices Nationwide

Palomar Community College District
1140 West Mission Road
San Marcos, California 92069

September 3, 2015
Contract No.: 766

Attention: Mr. Dennis Astl, Manager, Construction and Facilities Planning

Subject: Geotechnical Investigation

Project: Proposed Temporary Parking Lot
Palomar Community College
San Marcos, California

Dear Mr. Astl:

As requested, NV5 West, Inc. (NV5) is pleased to submit the results of the geotechnical investigation for the subject project. The purpose of this investigation was to evaluate the subsurface conditions at the proposed Temporary Parking Lot project site. We understand that the proposed construction includes temporary parking lot and associated improvements including light poles, pavement and fences. The results of the geotechnical field explorations, laboratory tests, and geotechnical engineering recommendations and conclusions are presented herewith.

Based on the subsurface exploration, subsequent testing of the subsurface soils, and engineering analyses it was concluded that the construction of the proposed project is geotechnically feasible provided the recommendations contained herein are appropriately incorporated into the design and implemented during construction.

It is recommended that the forthcoming project specifications, in particular the earthwork/compaction sections, be reviewed by NV5 for consistency with our report prior to the bid process in order to avoid possible conflicts, misinterpretations, and inadvertent omissions, etc. It should also be noted that the applicability and final evaluation of recommendations presented herein are contingent upon construction phase field monitoring by NV5 in light of the widely acknowledged importance of geotechnical consultant continuity through the various design, planning and construction stages of a project.

NV5 appreciates the opportunity to provide this geotechnical engineering service for this project and looks forward to continuing our role as your geotechnical engineering consultant.

Respectfully submitted,
NV5 West, Inc.

REGISTERED GEOLOGIST
GENE CUSTENBORDER
No. 1319
CERTIFIED ENGINEERING GEOLOGIST
04/11-30-16
STATE OF CALIFORNIA

Gene Custenborder, CEG 1319
Senior Project Geologist

Sam Koohi, PhD
Project Geotechnical Engineer

Guillaume Gau, GE
Senior Engineering Manager

REGISTERED PROFESSIONAL ENGINEER
GUILLAUME GAU
No. GE 2986
Exp. 9/30/16
GEOTECHNICAL
STATE OF CALIFORNIA

GC/SK/GG:ma

Distribution: (3) Addressee, (1) via email

G.I. Report.doc

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Figures

Appendices

1.0 INTRODUCTION

This report presents the results of NV5's geotechnical investigation for Palomar Community College District's proposed temporary parking lot located at the Palomar Community College campus in San Marcos, California. The approximate location of the project area is shown on *Figure 1, Site Location Map*. The purpose of this study was to evaluate the subsurface conditions and to provide geotechnical recommendations for the design and construction of the proposed parking lot and associated improvements. This report summarizes the data collected and presents our findings, conclusions, and recommendations.

This report has been prepared for the exclusive use of the client and their consultants in the design of the proposed project. In particular, it should be noted that this report has not been prepared from the perspective of a construction bid preparation instrument and should be considered by prospective construction bidders only as a source of general information subject to interpretation and refinement by their own expertise and experience, particularly with regard to construction feasibility. Contract requirements as set forth by the project plans and specifications will supersede any general observations and specific recommendations presented in this report.

2.0 SCOPE OF SERVICES

The scope of services for this project consisted of the following tasks:

- Review of readily available background data, including in-house geotechnical data, geotechnical literature, geologic maps, topographic maps, seismic hazard maps, and literature relevant to the subject site.

- A site reconnaissance to observe the general surficial site conditions and to select boring locations.

- A subsurface investigation, including the excavating, logging, and sampling of three exploratory borings located within the project area to depths up to approximately 16.5 feet below the existing ground surface. Soil samples obtained from the borings were transported to NV5's in-house laboratory for observation and testing.

- Laboratory testing of selected soil samples to evaluate their pertinent geotechnical engineering properties.

- An assessment of faulting, seismicity, slope stability and other geologic hazards affecting the area and possible impacts on the subject project.

- Engineering evaluation of the geotechnical data collected to develop geotechnical recommendations for the design and construction of the proposed project.

- Preparation of this report, including reference maps and graphics, summarizing the data collected and presenting our findings, conclusions, and geotechnical recommendations for the design and construction of the proposed project.

3.0 SITE AND PROJECT DESCRIPTION

The proposed temporary parking lot site is currently a baseball field located in the southeast corner of the Palomar Community College campus in San Marcos California. The relatively level graded pad sits at an elevation of approximately 575 feet above mean sea level and is bounded by West Mission Road on the south, a football field and track on the northwest, tennis courts and fitness center on the north and a softball field on the east (refer to *Figure 1, Site Location Map*).

Based on preliminary information it is understood that the proposed project includes removal of the baseball field and construction of a temporary parking lot with a capacity of 376 spaces. The parking area itself will be Class II base with a binding agent to control dust. Railroad ties will be used for tire stops. The access into and out of the lot will be via a new ramp from Comet Circle to the east. This access ramp into the parking lot will be paved along with a small area in the extreme southeast corner of the parking lot. Minimal lighting will be provided via light standards under 30' in height on concrete bases. A small set of stairs is planned in the northwest corner to allow pedestrians easy access to the main campus. A 30-foot high chain link or net fence along the third base side of the remaining softball field is also included.

4.0 FIELD EXPLORATION

Before starting the field exploration program, a field reconnaissance was conducted to observe site conditions and check locations for the planned subsurface explorations. As required by law, Underground Service Alert was notified of the locations of the exploratory borings prior to drilling.

The subsurface conditions were explored by drilling, logging, and sampling three exploratory borings located within the project area to a maximum depth of approximately 16.5 feet below the existing ground surface. The borings were drilled using a track-mounted hollow-stem auger drill rig. Furthermore, since the baseball field is actively being used, to minimize impact of the drilling, the drill rig used is propelled on rubber tracks, and has a footprint pressure of approximately six pounds per square foot. The approximate locations of the exploratory borings are presented on *Figure 2, Geotechnical Map*. Details of the subsurface exploration and logs of the exploratory borings are presented in Appendix A. Subsequent to logging and sampling, the borings were backfilled.

5.0 LABORATORY TESTING

Laboratory testing was performed on selected representative bulk and relatively undisturbed soil samples obtained from the exploratory borings to aid in the soil classification and to evaluate the engineering properties of the soil materials encountered. The following tests were performed:

- In-situ density and moisture content (ASTM D2216)
- Sieve analyses (ASTM D422)
- Direct shear (ASTM D3080)
- Resistance “R”-Value (ASTM D D2844/CTM301)
- Corrosivity series including sulfate content, chloride content, pH-value, and resistivity (California Test Methods 417, 422, and 532/643)

Testing was performed in general accordance with applicable ASTM standards or California Test Methods. The laboratory test results and details of the laboratory-testing program are presented in Appendix B.

6.0 GEOLOGY

Geologic Setting - The site is located in northern San Diego County within the Peninsular Ranges geomorphic province. This province is characterized by a system of predominantly northwest-southeast trending, right-lateral, strike slip faults associated with the San Andreas and related fault systems. Typical stratigraphy includes Mesozoic (between approximately 250 and 65 million years old) igneous intrusive and metamorphic rocks exposed in the eastern and portion of the province Cenozoic (less than 65 million years old) marine and non-marine sedimentary units overlying Mesozoic basement rocks in coastal areas and Quaternary (less than approximately 2 million years old) alluvial deposits overlying older strata in valleys and larger drainages.

Geologic Materials – As encountered in this investigation, the site appears to be underlain by a thin veneer of fill soils which is in turn underlain by a Eocene-aged claystone representing the Santiago Formation. Although not encountered in this investigation, the project site and general campus area are underlain at depth by Cretaceous granitic rocks. *Figure 2, Geotechnical Map* presents the general distribution of geologic units at the site and nearby vicinity. Detailed descriptions of the earth materials encountered are presented on the in *Appendix A, Logs of Exploratory Borings*. Descriptions of the various geologic units encountered are provided below:

- Fill (mapped as Af) - Fill soils were encountered in all of the exploratory borings drilled on the relatively level baseball field. Fill was encountered to a depth of approximately 8 feet. Fill appears to be derived locally from excavations of the granitic rocks in the areas to the north and/or east. As encountered these materials generally consisted brown to red-brown, moist to wet, medium dense silty fine sand.

- Santiago Formation (mapped as Tsa) – Claystone of the Eocene-aged Santiago Formation was encountered underlying the fill soils at a depth of approximately 8 feet below the existing grade. As encountered the Santiago Formation consisted of gray to mottled gray and yellow, wet, firm to stiff clay.

Groundwater – Groundwater was encountered at a depth of 10 feet and 14 feet below the existing ground surface in borings B-2 and B-3, respectively. Groundwater was not encountered in boring B-1. Groundwater is not expected to be a constraint to the proposed construction as we understand it. However, experience indicates that near-surface groundwater conditions or localized seepage zones can develop in areas where no such groundwater conditions previously existed, especially in areas where a substantial increase in surface water infiltration results from landscape irrigation, agricultural activity or unusually heavy precipitation. Seasonal variations in the groundwater levels should be anticipated.

7.0 FAULTING, SEISMICITY AND OTHER GEOLOGIC HAZARDS

The principal seismic considerations for most facilities in Southern California are surface rupturing of fault traces, damage caused by ground shaking or seismically-induced ground settlement or liquefaction. Potential impacts to the project due to faulting, seismicity and other geologic hazards are discussed in the following sections.

Faulting - The numerous faults in southern California include active, potentially active, and inactive faults. As used in this report, the definitions of fault terms are based on those developed for the Alquist-Priolo Special Studies Zones Act (AP) of 1972 and published by the California Division of Mines and Geology (Hart and Bryant, 1997). Active faults are defined as those that have experienced surface displacement within Holocene time (approximately the last 11,000 years) and/or have been included within any of the state-designated Earthquake Fault Zones (previously known as Alquist-Priolo Special Studies Zones). Faults are considered potentially active if they exhibit evidence of surface displacement since the beginning of Quaternary time (approximately two million years ago) but not since the beginning of Holocene time. Inactive faults are those that have not had surface movement since the beginning of Quaternary time.

Review of geologic maps and literature pertaining to the general site area indicates that the site is not located within a state-designated Earthquake Fault Zone. In addition, there are no known major or active faults mapped on the project site. Evidence for active faulting at the site was not observed during the subsurface investigation. The relative location of the site to known active faults in the region is depicted on Figure 3, Regional Fault Map. The distance from the site to the projection of traces of surface rupture along major active earthquake fault zones, that could affect the site are listed in the following Table 1.

Table1
Distance From the site to Major Active Faults

Fault	Distance From Site
Newport-Inglewood/Rose Canyon (Oceanside Section)	12.3 miles
Elsinore (Julian Section)	17 miles
Coronado Bank (Palos Verdes Section)	25 miles
San Diego Trough	36 miles
San Jacinto	43 miles
San Clemente	68 miles
San Andreas	68 miles

Seismic Shaking - The project site is located in southern California which is considered a seismically active area, and as such, the seismic hazard most likely to impact the site is ground shaking resulting from an earthquake along one of the known active faults in the region. The seismic design of the project may be performed using seismic design recommendations in accordance with the 2013 California Building Code (CBC). Recommended seismic design parameters are presented in Section 9.14 of this report.

Fault Rupture - The project site is not located within an Earthquake Fault Zone delineated by the State of California for the hazard of fault surface rupture. The surface traces of known active or potentially active faults are not known to pass directly through, or to project toward the site. Therefore, the potential for damage due to surface rupture of faults at the project site is considered low.

Liquefaction and Seismically-Induced Settlement - Liquefaction of soils can be caused by ground shaking during earthquakes. Research and historical data indicate that loose, relatively clean granular soils are susceptible to liquefaction and dynamic settlement, whereas the stability of the majority of clayey silts, silty clays and clays is not adversely affected by ground shaking. Liquefaction is generally known to occur in saturated cohesionless soils at depths shallower than approximately 50 feet. Pipes constructed in soils that become liquefied may become buoyant.

The planned parking lot is underlain predominately by medium dense sandy fill and firm to stiff claystone deposits. These materials are not considered to be susceptible to liquefaction. It is our opinion that the potential damage to the proposed pipeline and associated improvements due to liquefaction is considered to be very low.

Dynamic settlement due to earthquake shaking can occur in both dry and saturated loose to medium dense sandy soils. These sand particles can become more densely packed and settle when subject to seismic shaking. The medium dense sandy fill soils and firm to stiff claystone underlying the proposed parking lot are typically not prone to dynamic settlement. It is NV5's opinion that the potential for damage to the proposed parking lot due to seismically-induced settlement at the sites is low.

Landslides and Slope Instability - The proposed parking lot project not located adjacent to steep or high slopes. Based on the investigation, there are no known landslides on or near the project site, and the site is not located in the path of any known landslides. It is our opinion that the potential damage to the proposed project due to landsliding or slope instability is considered very low.

Subsidence - The site is not located in an area of known ground subsidence due to the withdrawal of subsurface fluids. Accordingly, the potential for subsidence occurring at the site due to the withdrawal of oil, gas, or water is considered low.

Tsunamis, Inundation Seiches, and Flooding - The site is located approximately 8.5 miles inland from the coast at an elevation at approximately of 575 feet above mean sea level. Therefore, tsunamis (seismic sea waves) are not considered a hazard at the site.

The site is not located downslope of any large body of water that could affect the site in the event of an earthquake-induced failure or seiche (oscillation in a body of water due to earthquake shaking). Therefore, earthquake-induced seiches are not considered a hazard at the site.

8.0 CONCLUSIONS

Based on the data obtained from the subsurface exploration, the associated laboratory test results, engineering analyses, and experience with similar site conditions, it is NV5's opinion that construction of the proposed temporary parking lot and associated improvements is feasible from a geotechnical standpoint, provided that the recommendations in this report are incorporated into the design plans and implemented during construction.

It is anticipated that once the existing grass and other features of the baseball field are removed, the near-surface soils within the upper 12 inches will be disturbed and not uniform in consistency. To provide a uniform subgrade for the parking lot and associated surface improvements, we recommended that these materials be scarified and recompacted in accordance with the recommendations of this report. The near-surface soils are similar to soils in the general area that have a very low expansion potential when tested. Effort should be made during site grading to ensure that soils with a very low to low expansion potential are placed or allowed to remain within the upper three feet below finished grade.

Geotechnically-related recommendations for the design and construction of the proposed project are presented in the following sections.

9.0 DESIGN RECOMMENDATIONS

9.1 General

Locally-derived, medium dense sandy fill soils underlain by firm to stiff formational materials consisting of claystone were encountered at the proposed project site. These materials are generally considered capable of reliably supporting the proposed parking lot and associated improvements. Scarification and recompaction of the upper 12 inches of these materials in accordance with the recommendations of this report are recommended to provide a uniform subgrade for the proposed parking lot and associated improvements. These materials, when properly moisture-conditioned, are considered suitable for reuse as compacted fill.

9.2 Earthwork

Site grading should be performed in accordance with the following recommendations and the Typical Earthwork Guidelines provided in Appendix D. It should be noted that the recommendations presented in Appendix D are general recommendations and as such many of the recommendations may not be applicable to this project. In addition, in the event of conflict, the recommendations presented herein supersede those of Appendix D.

- Clearing and Grubbing - Prior to grading, the project area should be cleared of all significant surface vegetation, demolition rubble, trash, debris, etc. Any buried organic debris or other unsuitable contaminated material encountered during subsequent excavation and grading work should also be removed. Removed material and debris should be properly disposed of offsite. Holes resulting from removal of buried obstruction which extend below finished site grades should be filled with properly compacted soils.

- Subgrade for Parking Lot, Paved Areas and Flatwork: - The subgrade for the proposed parking lot, paved areas and flatwork should be scarified to a minimum depth of 12 inches, brought to near-optimum moisture conditions, and compacted to at least 95 percent relative compaction, based on laboratory standard ASTM D1557.

- Structural Fill Placement – Other areas (outside of the parking lot subgrade) to receive fill and/or surface improvements should be scarified to a minimum depth of 12 inches, brought to near-optimum moisture conditions, and compacted to at least 90 percent relative compaction, based on laboratory standard ASTM D1557. Fill soils should be brought to near-optimum moisture conditions and compacted in uniform lifts to at least 90 percent relative compaction (ASTM D1557). Rocks with a maximum dimension greater than 4 inches should not be placed in the upper 3 feet of pad grade. The optimum lift thickness to produce a uniformly compacted fill will depend on the size and type of construction equipment used. In general, fill should be placed in uniform lifts not exceeding 8 inches in loose thickness. Placement and compaction of fill should be observed and tested by the geotechnical consultant.

- Excavatability – Based on our subsurface exploration, it is anticipated that the on-site soils can be excavated by modern conventional heavy-duty excavating equipment in good operating conditions.

- Graded Slopes – Graded slopes should be constructed at a gradient of 2 to 1 (horizontal to vertical) or flatter. To reduce the potential for surface runoff over slope faces, cut slopes should be provided with brow ditches and berms should be constructed at the top of fill slopes.

- Import Soils - If import soils are needed, proposed import should be sampled and tested for suitability by NV5 prior to delivery to the site. Imported fill materials should consist of clean granular soils free from vegetation, debris, or rocks larger than 3 inches maximum dimension. The Expansion Index value should not exceed a maximum of 20 (i.e., essentially non-expansive).

9.3 Utility Trench Excavations

Temporary, shallow excavations with vertical side slopes less than 4 feet high will generally be stable, although there is a potential for localized sloughing. Vertical excavations greater than 4 feet high should not be attempted without proper shoring to prevent local instabilities. Shoring may be accomplished with hydraulic shores and trench plates, trench boxes, and/or soldier piles and lagging. The actual method of a shoring system should be provided and designed by a contractor experienced in installing temporary shoring under similar soil conditions. All trench excavations should be shored in accordance with CalOSHA regulations. For your planning purposes, on-site soil materials may be considered a Type B soil, as defined by the current CalOSHA soil classification.

Stockpiled (excavated) materials should be placed no closer to the edge of a trench excavation than a distance defined by a line drawn upward from the bottom of the trench at an inclination of 1:1(H:V), but no closer than 4 feet. All trench excavations should be made in accordance with CalOSHA requirements.

9.4 Utility Trench Backfill

Subsurface utility trench backfill, including water, gas, storm drain, sewer, irrigation, telecommunication, and electrical lines should be mechanically compacted. Water jetting should not be used for compaction. The material within the pipe zone (i.e. 6 inches below to 12 inches above pipe) should consist of free-draining sand or small gravel with a minimum sand equivalent of 30. There should be sufficient clearance along the side of the utility pipe or line to allow for compaction equipment. The pipe bedding shall be compacted under the haunches and alongside the pipe.

9.5 Dewatering

Groundwater was encountered at in borings B-2 and B-3 at a depth of 10 and 14 feet, respectively. Dewatering is not generally anticipated during the proposed construction. However, any cases of localized seepage or heavy precipitation should be monitored during construction. If necessary, dewatering may be achieved by means of excavating a series of shallow trenches directed by gradient (i.e., gravity) to sumps with pumps. In any case, the actual means and methods of any dewatering scheme should be established by a contractor with local experience. It is important to note that temporary dewatering, if necessary, will require a permit and plan that complies with RWQCB regulations. If excessive water is encountered, NV5 should be contacted to provide additional recommendations for temporary construction dewatering. Based on the subsurface exploration the onsite soils maybe considered to be relatively permeable.

9.6 Foundations for Lighting Poles and Fence Posts

The lighting poles and fence posts may be founded entirely in the existing compacted fill. The recommendations assume that the soils within the upper three feet of finished grade have a very low to low expansion potential. Expansion index tests should be performed following site grading to verify the expansion potential of the near-surface soils. The recommendations in the following Table 2 are provided for typical proposed cantilever light poles, fence posts or other pier-supported improvements. Foundation dimensions should be designed by a qualified structural or civil engineer. Minimum 2013 California Building Code parameters may also be used in lieu of any of the suggested parameters below, if preferred.

Table 2
Geotechnical Design Parameters
Drilled Piers and Poles

Allowable Vertical Bearing Capacity	Properly Compacted Fill: 2,000 pounds per square foot (psf). Increase of 200 psf for each additional foot of embedment to a maximum of 4000 psf Assumes a minimum embedment of 3 feet and minimum 24 inches in diameter. A one-third increase is allowed for transient live loads from wind or seismic forces.
Estimated Settlement (Total/Differential)	Less than 1-inch
Allowable Lateral Bearing Value	200 psf per foot of depth below 6 inches
Effective Width	2.0 times the width of the foundation (due to passive arching)

Bottom of pier footings should bear into competent material (anticipated to be previously placed fill) with a minimum distance to daylight of ten feet. To verify the presence of satisfactory materials at design elevations, pier excavations should be observed by a representative of NV5. Footing excavations should be deepened as necessary to extend into satisfactory bearing materials; however, NV5 should be notified to approve the proposed change. The bottom of foundation excavations should be cleared of loose soil prior to placing concrete. As indicated and if preferred, lightly loaded upright structures may be designed in accordance with current California Building Code or applicable standards assuming code minimum design values in lieu of the parameters provided above.

9.7 Exterior Concrete Slabs

Exterior concrete flatwork should have a minimum concrete thickness of 4 inches. Concrete slabs should be supported on at least 4 inches of Class 2 aggregate base compacted to at least 95 percent of the maximum dry density. The upper 12 inches of subgrade soil located below the aggregate base should be moisture-conditioned within 2% over the optimum moisture content, and recompacted to a minimum of 90 percent relative compaction (ASTM D1557).

We recommended that narrow strip concrete slabs, such as sidewalks, be reinforced with at least No. 3 reinforcing bars placed longitudinally at 36 inches on-center. Wide exterior slabs should be reinforced with at least No. 3 reinforcing bars placed 36 inches on-center, each way. The reinforcement should be extended through the control joints to reduce the potential for differential movement. Control joints should be constructed in accordance with recommendations from the structural engineer or architect.

9.8 Seismic Design Parameters

Preliminary seismic design parameters for the project site were also developed as per the guidelines outlined in the 2013 CBC (2012 IBC and 2010 ASCE 7-10 Standard with errata as of April 2013. **NV5 should be contacted to provide revisions to these parameters if other codes are specified.** The seismic design parameters for Site Class "D" were developed using a JAVA ™ application, Java Ground Motion Parameter Calculator–Version 5.0.9 available on the USGS website (http://earthquake.usgs.gov). The preliminary seismic design parameters for the project site are presented in Table 3 below.

Table 3
2013 CBC (2012 IBC Seismic Design Parameters
And ASCE 7-10 Standard)

Parameter	Value
Site Class; (Section 11.4.2)	D
Mapped Spectral Accelerations for short periods, S_S ; (Section 11.4.1)	1.017g
Mapped Spectral Accelerations for 1-sec period, S_1 ; (Section 11.4.1)	0.398g
Site Coefficient, F_a; (Table 11.4-1)	1.093
Site Coefficient, F_v; (Table 11.4-2)	1.604
Maximum considered earthquake spectral response acceleration for short periods, S_{MS} adjusted for Site Class (Equation 11.4-1)	1.112g
Maximum considered earthquake spectral response acceleration at 1-sec period, S_{M1} adjusted for Site Class (Equation 11.4-2)	0.638
Five-percent damped design spectral response acceleration at short periods, S_{DS}; (Equation 11.4-3)	0.741g
Five-percent damped design spectral response acceleration at 1-sec period, S_{D1}; (Equation 11.4-4)	0.426g

9.9 Recommend Pavement Section

It is understood that access into and out of the lot will be via a new ramp from Comet Circle to the east. This ramp along with a small area in the extreme southeast corner of the project will be paved with asphaltic concrete (AC) pavement. The parking lot itself will be surfaced with Class II base with a binding agent to control dust.

To develop preliminary recommendations for the pavement section, an R-Value test was performed on a near surface soil sample and resulted in an R-value of 12. Several pavement sections were calculated using an R-Value of 12 and assumed traffic index values ranging from 4.0 to 6.0. The project Architect or Civil Engineer should select the appropriate pavement section based on the anticipated traffic loads. NV5 can provide alternate sections based on other traffic loadings, if requested. Based on these design parameters, analysis in general accordance with the current Cal-Trans Highway Design Manual, and assuming compliance with site preparation recommendations, NV5 recommends the pavement structural sections in Table 4 below:

Table 4
Flexible Asphalt Pavement Sections

Traffic Index (TI)	***Pavement Section***	
	AC(1) *(inches)*	***AB***(2) *(inches)*
4.0	2.5	5.5
4.5	3.0	5.5
5.0	3.0	7.0
5.5	3.0	8.5
6.0	3.0	10.0
6.0	3.5	9.0

(1) Asphalt Concrete;
(2) Crushed Aggregate Base (CAB), Green Book section 200-2.2, compacted to at least 95% relative compaction (ASTM D-1557);
Note: The upper 12-inches of subgrade soils should be compacted to at least 95% relative compaction (ASTM D-1557).

It is recommended that R-value testing be performed on representative soil samples after rough grading operations on the upper 2 feet to confirm applicability of the above pavement sections.

The aggregate base should conform to the Crushed Aggregate Base per Greenbook requirements, Section 200-2.2. The base course should be compacted to a minimum dry density of 95% of the materials maximum density as determined by the ASTM D1557 test procedure. Field testing should be used to verify compaction, aggregate gradation, and compacted thickness.

The asphalt concrete pavement should be compacted to 95% of the unit weight as tested in accordance with the Hveem procedure. The maximum lift thickness should be two inches. The asphalt concrete material shall conform to Type III, Class C2 or C3, 2009 edition of the Greenbook Standard Specifications for Public Works Construction. An approved mix design should be submitted 30 days prior to placement. The mix design should include proportions of materials, maximum density and required lay-down temperature range. Field testing should be used to verify oil content, aggregate gradation, compaction, compacted thickness, and lay-down temperature.

If the paved areas are to be used during construction, or if the type and frequency of traffic is greater than assumed in the design, the pavement section should be re-evaluated for the anticipated traffic.

For the parking lot which will be surfaced with Class II base, prior to placing the Class II base, the upper 12 inches of subgrade should be moisture conditioned to near-optimum moisture conditions, and compacted to at least 95 percent relative compaction, based on laboratory standard ASTM D1557.

The aggregate base should be compacted to a minimum dry density of 95% of the materials maximum density as determined by the ASTM D1557 test procedure. Field testing should be used to verify compaction, aggregate gradation, and compacted thickness.

9.10 Soil Corrosion

Laboratory testing was performed on a representative sample of the on-site soils to evaluate pH, minimum resistivity, and chloride and soluble sulfate content. Table 5 presents the results of the corrosivity testing.

Table 5
Corrosivity Test Results

Test Location	Exploratory Boring B-3
Depth (feet)	2 – 4
pH	8.1
Resistivity (ohm-cm)	510
Chloride Content (ppm)	330
Soluble Sulfate Content (ppm)	180

Based on our experience and various publications including the Caltrans Corrosion Guidelines dated November 2012, the soil conditions are considered to be “not corrosive.

10.0 CONSTRUCTION OBSERVATION AND TESTING

Observation and testing of the placement and compaction of fill, subgrade and base will be important to the performance of the proposed project. Site preparation, removal of unsuitable soils, assessment of imported fill materials, fill placement, and other earthwork operations should be observed and tested. The substrata exposed during the construction may differ from that encountered in the exploratory borings. Continuous observation by a representative of NV5 during construction allows for evaluation of the soil conditions as they are encountered, and allows the opportunity to recommend appropriate revisions where necessary.

11.0 LIMITATIONS

The recommendations and opinions expressed in this report are based on NV5’s review of background documents and on information obtained from field explorations. It should be noted that this study did not evaluate the possible presence of hazardous materials on any portion of the site.

Due to the limited nature of the field explorations, conditions not observed and described in this report may be present on the site. Uncertainties relative to subsurface conditions can be reduced through additional subsurface exploration. Additional subsurface evaluation and laboratory testing can be performed upon request. It should be understood that conditions different from those anticipated in this report may be encountered during construction, and that additional effort may be required to mitigate them.

Site conditions, including groundwater elevation, can change with time as a result of natural processes or the activities of man at the subject site or at nearby sites. Changes to the applicable laws, regulations, codes, and standards of practice may occur as a result of government action or the broadening of knowledge. The findings of this report may, therefore, be invalidated over time, in part or in whole, by changes over which NV5 has no control.

NV5's recommendations for this site are, to a high degree, dependent upon appropriate quality control of construction operations, placement and compaction of backfill, subgrade preparation, etc. Accordingly, the recommendations are made contingent upon the opportunity for NV5 to observe the earthwork operations for the proposed construction. If parties other than NV5 are engaged to provide such services, such parties must be notified that they will be required to assume complete responsibility as the geotechnical engineer of record for the geotechnical phase of the project by concurring with the recommendations in this report and/or by providing alternative recommendations.

This document is intended to be used only in its entirety. No portion of the document, by itself, is designed to completely represent any aspect of the project described herein. NV5 should be contacted if the reader requires additional information or has questions regarding the content, interpretations presented, or completeness of this document.

NV5 has endeavored to perform this geotechnical evaluation using the degree of care and skill ordinarily exercised under similar circumstances by reputable geotechnical professionals with experience in this area in similar soil conditions.

12.0 REFERENCES

ASTM, 2001, Soil and Rock: American Society for Testing and Materials: vol. 4.08 for ASTM test methods D-420 to D-4914; and vol. 4.09 for ASTM test methods D-4943 to highest number.

California Department of Conservation, Division of Mines and Geology, 1997, Guidelines for Evaluation and Mitigation of Seismic Hazards in California: Special Publication 117, 74 pp.

California Department of Conservation, Division of Mines and Geology, 1998, Maps of Known Active Fault Near-Source Zones in California and Adjacent Portions of Nevada: International Conference of Building Officials, dated February, Scale 1 inch = 4 kilometers.

California Geological Survey, Geologic Data Map No. 2; Compilation and Interpretation by: Charles W. Jennings (1977). Updated version by: Carlos Gutierrez, William Bryant, George Saucedo, and Chris Wills. Graphics by: Milind Patel, Ellen Sander, Jim Thompson, Barbara Wanish and Milton Fonseca Plesch, Anndreas et. al., 2007, Community Fault Model (CFM) for Southern California; in the Bulletin of the Seismological Society of America, Vol. 97, No. 6. pp. 1793-1802, dated December.

Hart, E.W., and Bryant, W.A., 1997, Fault-Rupture Hazard Zones in California, Alquist-Priolo Earthquake Fault Zoning Act with Index to Earthquake Fault Zone Maps: California Department of Conservation, Division of Mines and Geology Special Publication 42, 38 pp.,

International Conference of Building Officials, 2012 California Building Code.

Ishihara, K., 1985, Stability of Natural Deposits during Earthquakes: Proceedings, 11th International Conference on Soil Mechanics and Foundation Engineering, Volume 1, pp. 321-376.

Jennings, C.W., 1994, Fault Activity Map of California and Adjacent Areas with Locations and Ages of Recent Volcanic Eruptions: California Department of Conservation, Division of Mines and Geology Geologic Data Map No. 6, scale 1:750,000.

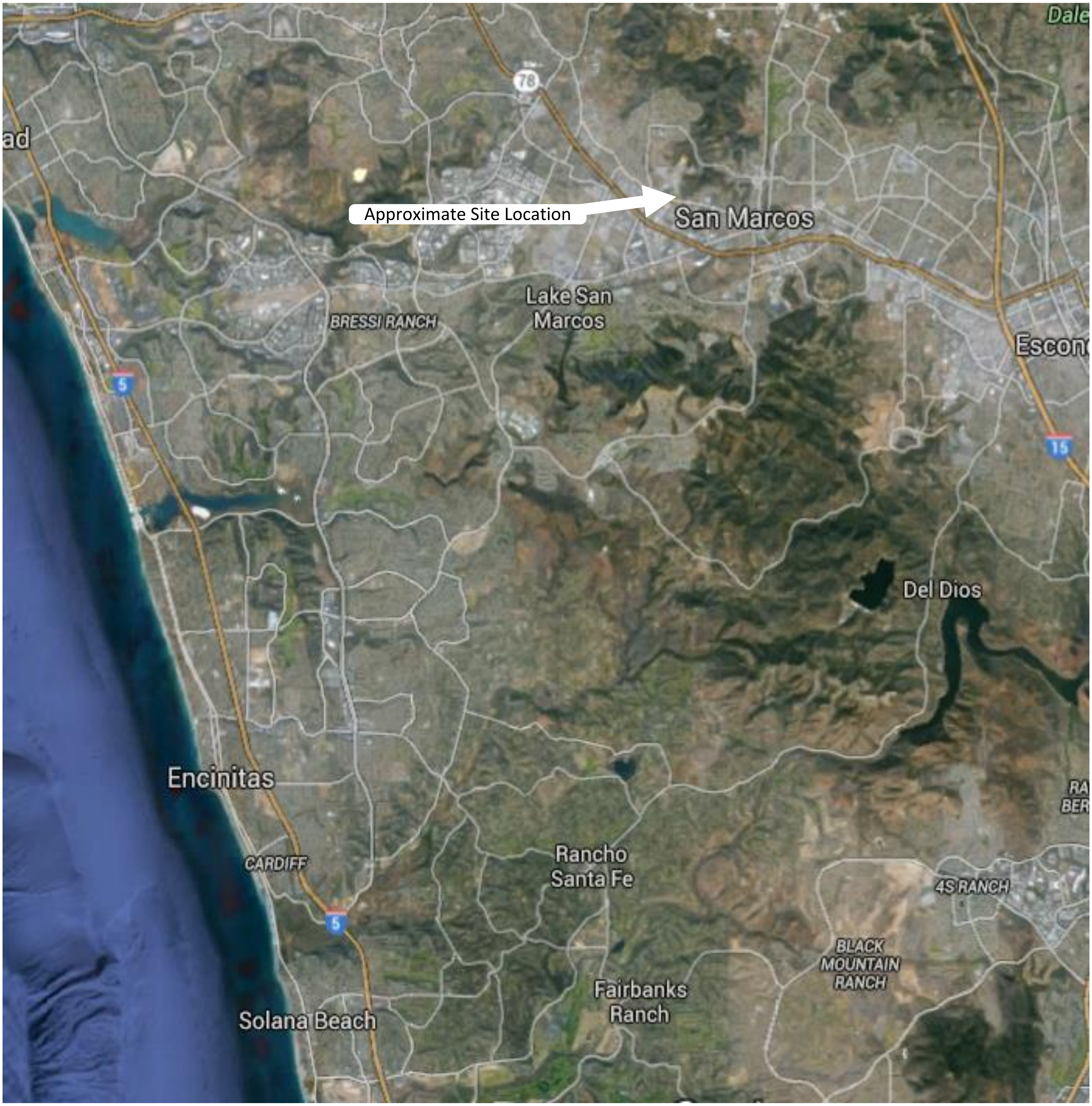
Jennings, C.W., and Saucedo, G. J. 1999, Simplified Fault Activity Map of California, Map Sheet 54, (Revised 2003 by Toppozada, T., and Branum, D.).

Kennedy, Michael P., and Tan, Siang S., Geologic Map of the Oceanside 30’ X 60’ Quadrangle California, Published by the United States Geological Survey, 2005.

International Building Code, dated 2012.

Youd, T.L. and Idriss, I.M., 2001, Liquefaction Resistance of Soils: Summary report of NCEER 1996 and 1998 NCEER/SF Workshops on Evaluation of Liquefaction Resistance of Soils: Journal of Geotechnical and Geoenvironmental Engineering, dated April, pp. 297-313.

Figures



Reference: Google Maps 2015

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No Scale

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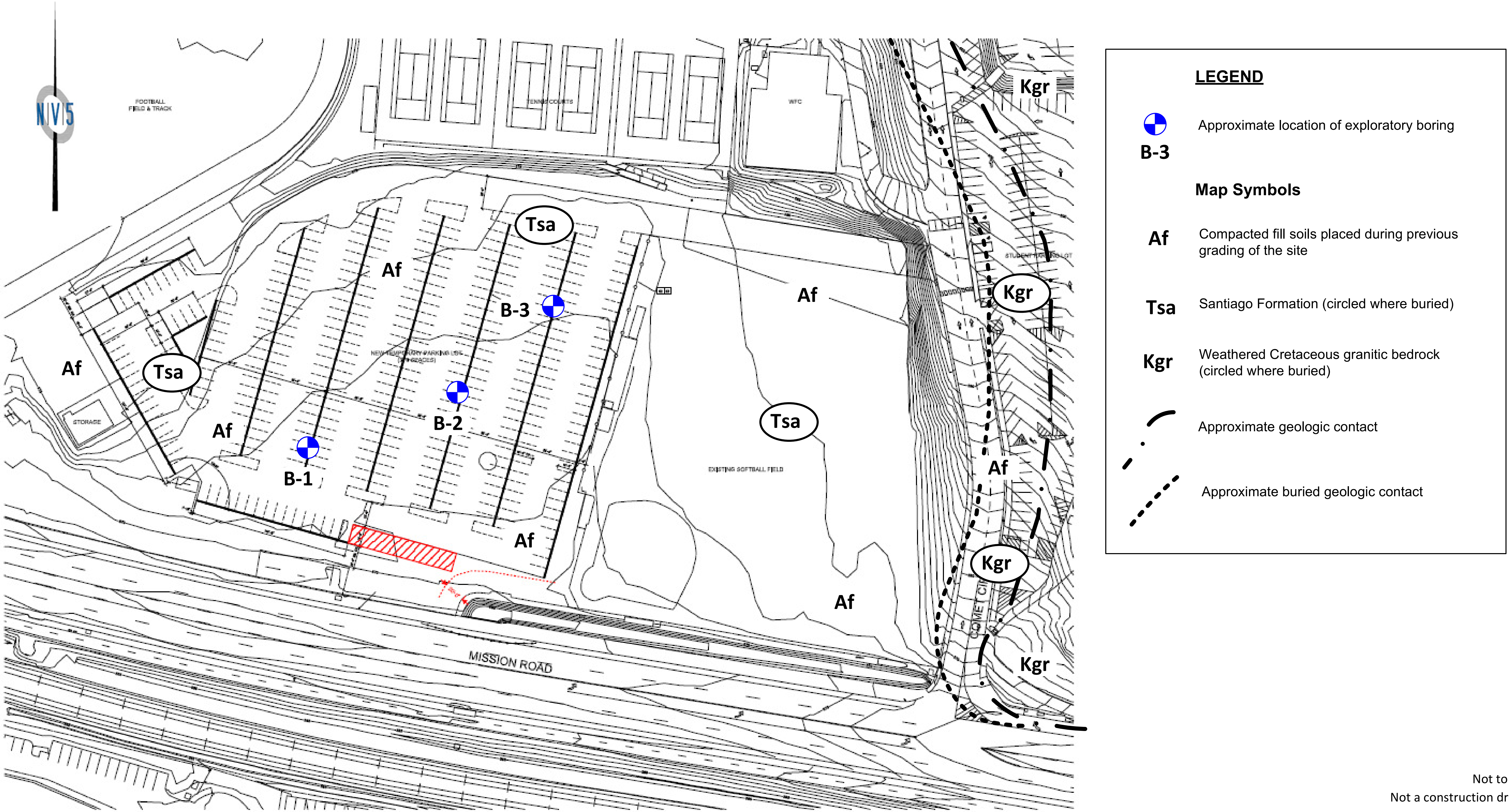
Date:	**September 2015**

Site Location Map
Palomar College Temporary Parking Lot
San Marcos, California

Figure No. 1



Not to scale.

Not a construction drawing.

Base Map: Adapted from "Site Plan, Palomar Community College, Temporary Parking Lot, Palomar College, 1140 W. Mission Rd., San Marcos, CA 92069-1487"' Sheet A-2, prepared by HMC Architects, dated June 22, 2015.

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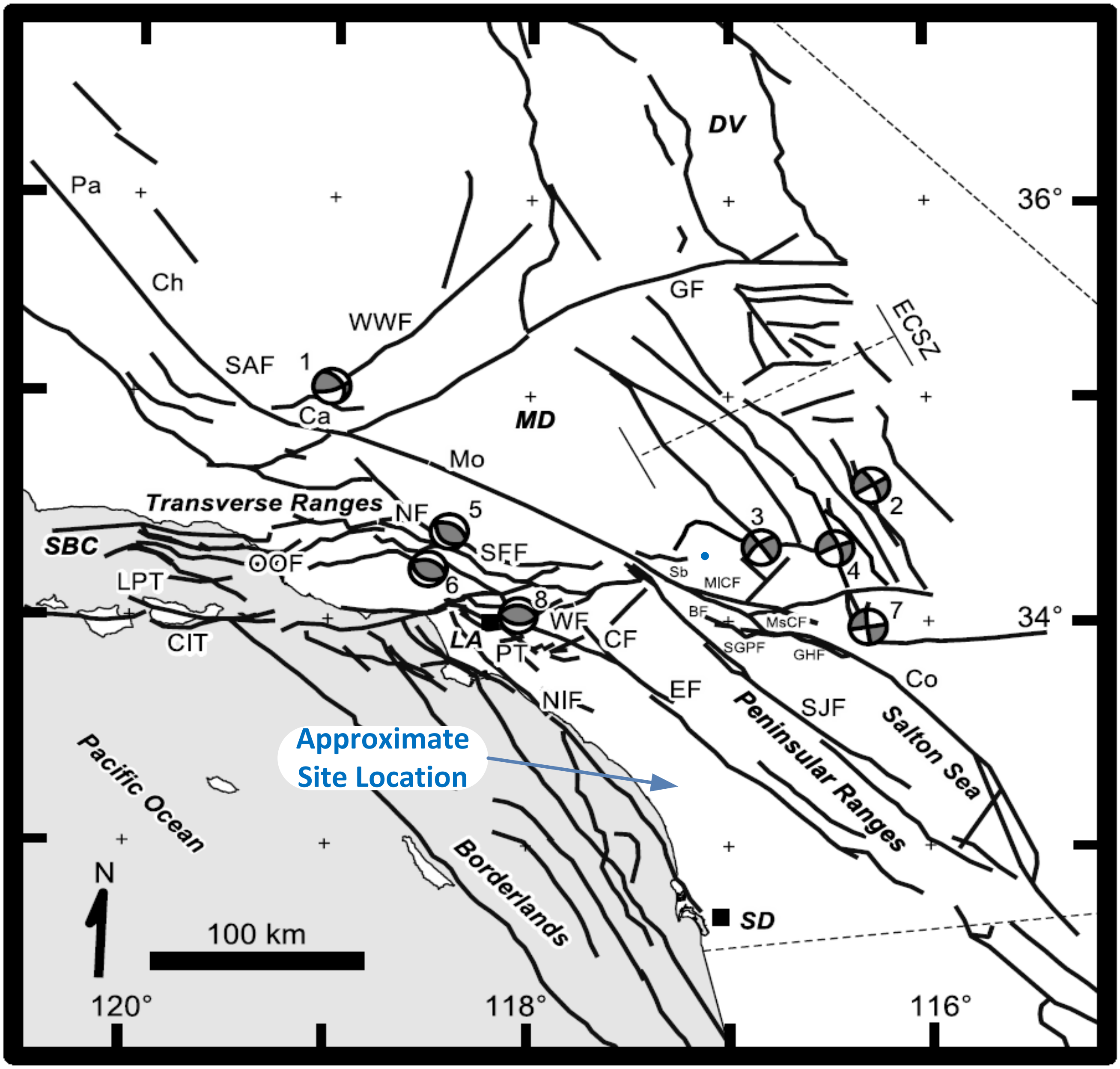
Geotechnical Map
Palomar College Temporary Parking Lot
San Marcos, California

Figure No. 2



Map of southern California showing the geographic regions, faults and focal mechanisms of the more significant earthquakes. **Regions:** Death Valley, DV; Mojave Desert MD; Los Angeles, LA; Santa Barbara Channel, SBC; and San Diego, SD. **Indicated Faults:** Banning fault, BF; Channel Island thrust, CIT; Chino fault, CF; Eastern California Shear Zone, ECSZ; Elsinore fault, EF; Garlock fault, GF; Garnet Hill fault, GHF; Lower Pitas Point thrust, LPT; Mill Creek fault, MICF; Mission Creek fault, MsCF; Northridge fault, NF; Newport Inglewood fault, NIF; offshore Oak Ridge fault, OOF; Puente Hills thrust, PT; San Andreas fault (sections: Parkfield, Pa; Cholame, Ch; Carrizo; Ca; Mojave, Mo; San Bernardino, Sb; and Coachella, Co); San Fernando fault, SFF; San Gorgonio Pass fault, SGPF; San Jacinto fault, SJF; Whittier fault, WF; and White Wolf fault, WWF. **Earthquake Focal Mechanisms:** 1952 Kern County, 1; 1999 Hector Mine, 2; 1992 Big Bear, 3; 1992 Landers, 4; 1971 San Fernando, 5; 1994 Northridge, 6; 1992 Joshua Tree, 7; and 1987 Whittier Narrows, 8.

Reference: Plesch, Andreas et. al., 2007, Community Fault Model (CFM) for Southern California; in the *Bulletin of the Seismological Society of America*, Vol. 97, No. 6. pp. 1793-1802, dated December.

For Schematic Use Only-Not a Construction Drawing

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Regional Fault Map
Palomar College Temporary Parking Lot
San Marcos, California

Figure No. 3

Appendix A

Logs of Exploratory Borings

Logs of Exploratory Borings

Bulk and relatively undisturbed drive samples were obtained in the field during our subsurface evaluation. The samples were tagged in the field and transported to our laboratory for observation and testing. The drive samples were obtained using the Standard Penetration Test (SPT) samplers as described below.

California Modified Split Spoon Sampler

The split barrel drive sampler is driven with a 140-pound hammer allowed to drop freely 30 inches in general accordance with ASTM D1587. The number of blows per foot recorded during sampling is presented in the logs of exploratory borings. The sampler has external and internal diameters of approximately 3.0 and 2.4 inches, respectively, and the inside of the sampler is lined with 1-inch-long brass rings. The relatively undisturbed soil sample within the rings is removed, sealed, and transported to the laboratory for observation and testing.

Standard Penetration Test (SPT) Sampler

The split barrel sampler is driven with a 140-pound hammer allowed to drop freely 30 inches in general accordance with ASTM D1586. The number of blows per foot recorded during sampling is presented in the logs of exploratory borings. The sampler has external and internal diameters of 2.0 and 1.5 inches, respectively. The soil sample obtained in the interior of the barrel is measured, removed, sealed and transported to the laboratory for observation and testing.

LOG SYMBOLS:

▮	Bulk/Bag sample
▯	California sampler (2-1/2 inch outside diameter)
⧗	Modified California sampler (3 inch outside diameter)
☒	Standard penetration Split spoon sampler (2 inch outside diameter)
‖	NX size core barrel (2-5/8 inch outside diameter)
⦀	Shelby tube

▼	Water level (level after completion)
▽	Water level (level where first encountered)

Abbreviations:

SA - Sieve Analysis
P200 - Percent passing #200 sieve
AL - Atterberg Limits
LL - Liquid limit
DS - Direct shear test
'R' - R-value test
CS - Corrosivity test
EI - UBC expansion index
MD - Laboratory compaction test
CN - Consolidation test

General Notes:

1. Lines separating strata on the logs represent approximate boundaries only. Actual transitions may be gradual.
2. No warranty is provided as to the continuity of soil conditions between individual sample locations.
3. Logs represent general soil conditions observed at the point of exploration on the date indicated.
4. In general, unified soil classification designations presented on the logs were evaluated by visual methods only. Therefore, actual designations (based on laboratory tests) may vary.

Consistency criteria based on field tests

Relative density	SPT* (# blows/ft)	Relative density (%)
Very Loose	<4	0 - 15
Loose	4 - 10	15 - 35
Medium Dense	10 - 30	35 - 65
Dense	30 - 50	65 - 85
Very dense	>50	85 - 100

		Torvane	Pocket** penetrometer
Consistency	SPT (# blows/ft)	Undrained shear strength (tsf)	Unconfined compressive strength
Very soft	<2	<0.13	<0.25
Soft	2 - 4	0.13 - 0.25	0.25 - 0.5
Medium stiff	4 - 8	0.25 - 0.5	0.5 - 1.0
Stiff	8 - 15	0.5 - 1.0	1.0 - 2.0
Very stiff	15 - 30	1.0 - 2.0	2.0 - 4.0
Hard	>30	>2.0	>4.0

* Number of blows of 140 pounds hammer falling 30 inches to drive a 2 inch C.D. (1 3/8" I.D.) split barrel samler (ASTM - 1386 standard penetration test)

** Unconfined compressive strength in Tons/ft^2. Read from pocket penetrometer

Moisture content

Description	Field test
Dry	Absence of moisture, dusty, dry to the touch
Moist	Damp but no visible water
Wet	Visible free water, usually soil is below water table

Cementation

Description	Field test
Weakly	Crumbles or breaks with handling or slight finger pressure
Moderately	Crumbles or breaks with considerable finger pressure
Strongly	Will not crumble or break with finger pressure

NV5 West, Inc.
7895 Convoy Court, Ste 18,
San Diego, CA 92111
Tel: (858) 715-5800, Fax: (858) 715-5810

Project No: **766**
Drawn: **GC**
Date: **September 2015**

Title: **Log Legend**
Project: **Palomar College Temporary Parking Lot**
San Marcos, California

Chart No. 1

Soil Classification Chart

Major Divisions			Symbols		Typical Descriptions
			Graph	Letter	
Coarse Grained Soils More than 50& of material is larger than No. 200 sieve size	Gravel and Gravelly soils more than 50% of coarse fraction retained on No. 4 sieve	Clean Gravels (Little or no fines)	●	GW	Well-Graded Gravel, Gravel SAND mixtures, little of no fines
			○	GP	Poorly-Graded Gravels, Gravel - SAND mixtures, little or no fines
		Gravels with fines (Appreciable amount of fines)	○	GM	Silty Gravels, Gravel- SAND-Silt mixture
			▨	GC	Clayey Gravels, Gravel - SAND - Clay mixtures
	Sand and Sandy Soils More than 50% of coarse fraction passing on No.4 sieve	Clean SANDS (Little or no fines)	░	SW	Well-Graded SANDS, Gravelly, SANDS, little or no fines
			░	SP	Poorly - Graded SANDS, Gravelly SAND, little or no fines
		Sands with Fines (Appreciable amount of fines)	░	SM	Silty SANDS, SAND-Silt mixtures
			▨	SC	Clayey SANDS, SAND - Clay mixtures
Fine grained soils More than 50% of material is smaller than No. 200 sieve size	Silts and Clays	Liquid Limit less than 50	▥	ML	Inorganic Silts and very fine SANDS, rock flour, Silty or Clayey fine SANDS or clayey Silts with slight Plasticity
			▨	CL	Inorganic Clays of low to medium Plasticity, Gravelly Clays, Sandy Clays, Silty Clays, Lean Clays
			☰	OL	Organic Silts and organic Silty Clays of low Plasticity
	Silts and Clays	Liquid Limit Greater than 50	▥	MH	Inorganic Silts, micaceous or diatomaceous fine SAND or Silty Soils
			▨	CH	Inorganic Clays of high Plasticity
			▒	OH	Organic Clays of medium to High Plasticity, organic Silts
Highly organic soils			⁂	PT	Peat, Humus, swamp soils with High organic contents

NOTE: Dual symbols are used to indicate borderline soil classifications.

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San Diego, CA 92111
Tel: (858) 715-5800, Fax: (858) 715-5810

Project No: **766**
Drawn: **GC**
Date: **September 2015**

Title: **Log Legend – Soil Classification**
Project: **Palomar College Temporary Parking Lot**
San Marcos, California
Chart No. 2

Project:	Temporary Parking Lot	**Boring B-1**
Project Location:	San Marcos-Palomar College	
Project Number:	766	Sheet 1 of 1

Date(s) Drilled	August 5, 2015	Logged By	G. Custenborder	Checked By	G. Gau
Drilling Method	Hollow Stem Auger	Boring Diameter	6-inch	Approximate Surface Elevation	± 575 feet
Drilling Contractor	Pacific Drilling	Sampling Method	California Split Spoon and Standard Penetration Test	Hammer Data	140 pound, auto chain, 30 inch drop
Drill Rig Type:	LA Fraste	Location: See Geotechnical Map		Lat.: 33.147261	Long.: -117.183265

Depth (ft)	Sample Type	Blows / 6 in. (N)	Sample ID	USCS Class.	**MATERIAL DESCRIPTION** This log is an integral part of the accompanying report and must be used together with the report for relevant interpretation. The descriptions contained hereon apply only at this boring location and at the time of excavation. Subsurface data are a simplified summary of actual conditions encountered and may vary at other locations and with the passage of time.	Moisture Content %	Dry Weight (pcf)	Other Tests and Remarks
0				SM	Grass			
1					Fill - grass, brown, wet, medium dense, silty fine SAND			
2								
3	▩		Bag 1					
4								
5	☒	5 7 16				17.6		
				CL	**Santiago Formation**-@8' gray, wet, firm, CLAY			
10	▩ ▨	8 10 12	Bag 2 Cal 1			20.9	104.8	
15	☒	8 10 15	SPT 2			19.3		
					Total depth: 16.5 feet No refusal Groundwater not encountered Boring backfilled 8-5-15			
20								
25								
30								

Sample Type

▨ Cal. Mod. ☒ SPT ▩ Bulk ◣ Other ● No Recovery

NV5 BEYOND ENGINEERING

Project:	Temporary Parking Lot	**Boring**	**B-2**
Project Location:	San Marcos-Palomar College		
Project Number:	766	Sheet 1 of 1	

Date(s) Drilled	August 5, 2015	Logged By	G. Custenborder	Checked By	G. Gau
Drilling Method	Hollow Stem Auger	Boring Diameter	6-inch	Approximate Surface Elevation	± feet
Drilling Contractor	Pacific Drilling	Sampling Method	California Split Spoon and Standard Penetration Test	Hammer Data	140 pound, auto chain, 30 inch drop
Drill Rig Type:	LA Fraste	Location: See Geotechnical Map		Lat.: 33.147356	Long.: -117.182818

Depth (ft)	Sample Type	Blows / 6 in. (N)	Sample ID	USCS Class.	**MATERIAL DESCRIPTION** This log is an integral part of the accompanying report and must be used together with the report for relevant interpretation. The descriptions contained hereon apply only at this boring location and at the time of excavation. Subsurface data are a simplified summary of actual conditions encountered and may vary at other locations and with the passage of time.	Moisture Content %	Dry Weight (pcf)	Other Tests and Remarks
0				SM	**Fill** - brown, wet, medium dense, silty fine SAND			
1								
2								
3			Bag 1					
4								
5	☒ ▨	7 8 14	Cal 1			15.4	115.9	
				CL	**Santiago Formation**-@8' gray, very moist, firm, CLAY			
10	☒	14 15 16	SPT 1		@ 10 ' ground water	22.5		
15	▨	7 10 11	Cal 2			15.0	114.1	
					Total depth: 16.5 feet No refusal Groundwater at 10 feet Boring backfilled 8-5-15			
20								
25								
30								

Sample Type

▨ Cal. Mod. ☒ SPT ▩ Bulk ◣ Other ● No Recovery

NV5 BEYOND ENGINEERING

Project: Temporary Parking Lot
Project Location: Palomar College - San Marcos
Project Number: 766

Boring B-3

Sheet 1 of 1

Date(s) Drilled	August 5, 2015	Logged By	G. Custenborder	Checked By	G. Gau
Drilling Method	Hollow Stem Auger	Boring Diameter	6-inch	Approximate Surface Elevation	± 575 feet
Drilling Contractor	Pacific Drilling	Sampling Method	California Split Spoon and Standard Penetration Test	Hammer Data	140 pound, auto chain, 30 inch drop
Drill Rig Type:	LA Fraste	Location: See Geotechnical Map		Lat.: 33.147698	Long.: -117.182519

Depth (ft)	Sample Type	Blows / 6 in. (N)	Sample ID	USCS Class.	MATERIAL DESCRIPTION This log is an integral part of the accompanying report and must be used together with the report for relevant interpretation. The descriptions contained hereon apply only at this boring location and at the time of excavation. Subsurface data are a simplified summary of actual conditions encountered and may vary at other locations and with the passage of time.	Moisture Content %	Dry Weight (pcf)	Other Tests and Remarks
0				SM	Grass Fill - grass, brown, wet, medium dense, silty fine SAND			
1								
2	▩		Bag 1					
3								
4								
5	▨	9 15 30	Cal 1			13.5	121.3	
				CL	**Santiago Formation**-@8' gray and yellow, moist, firm, CLAY			
10	☒	5 6 11	[illegible]			29.8		
					@ 14 ' ground water			
15	▨	6 7 10	Cal 2			28.7	91.3	
					Total depth: 16.5 feet No refusal Groundwater at 14 feet Boring backfilled 8-5-15			
20								
25								
30								

Sample Type

▨ Cal. Mod. ☒ SPT ▩ Bulk ◣ Other ● No Recovery

Appendix B

Laboratory Test Results

SUMMARY OF LABORATORY TEST RESULTS

In-situ Moisture and Density Tests

The in-situ moisture contents and dry densities of selected samples obtained from the test borings were evaluated in general accordance with the latest version of D-2216 and D2937 laboratory test methods. The method involves obtaining the moist weight of the sample and then drying the sample to obtain is dry weight. The moisture content is calculated by taking the difference between the wet and dry weights, dividing it by the dry weight of the sample and expressing the result as a percentage. The results of the in-situ moisture content and density tests are presented in the following table and on the logs of exploratory borings in Appendix A.

RESULTS OF MOISTURE CONTENT AND DENSITY TESTS
(ASTM D2216)

Sample Location	Moisture Content (percent)	Dry Density (pounds per cubic foot)
Boring B-1 @ 5 - 6.5’ feet	17.6	density not determined
Boring B-1 @ 11 - 11.5 feet	20.9	104.8
Boring B-1 @ 15 – 16.5 feet	19.3	density not determined
Boring B-2 @ 6 – 6.5 feet	15.4	115.4
Boring B-2 @ 10 – 11.5 feet	22.5	density not determined
Boring B-2 @ 16-16.5 feet	15	114.1
Boring B-3 @ 6-6.5 feet	13.5	121.3
Boring B-3 @ 10-11.5 feet	29.8	density not determined
Boring B-3 @ 16-16.5 feet	28.7	91.3

Classification

Soils were visually and texturally classified in accordance with the Unified Soil Classification System. Soil classifications are indicated on the logs of the exploratory borings in Appendix A.

R Value

R value test was performed in accordance with ASTM D2844. This test was useful in evaluating the resistance R value and expansion pressure of compacted soils. Test results are attached in this appendix.

Direct shear

A direct shear test was performed on a representative undisturbed sample in accordance with ASTM D3080 to evaluate the shear strength characteristics of the on-site materials. The test method consists of placing the soil sample in the direct shear device, applying a series of normal stresses, and then shearing the sample at the constant rate of shearing deformation. The shearing force and horizontal displacements are measured and recorded as the soil specimen is sheared. The shearing is continued well beyond the point of maximum stress until the stress reaches a constant or residual value. The results of the tests are presented in the following table and attached in this appendix.

RESULTS OF DIRECT SHEAR TEST
(ASTM D3080)

Location	**Peak Angle of Internal Friction (degrees)**	**Peak Cohesion Intercept (psf)**	**Notes**
Boring B-3 @ 2-4 feet	40.5	1076	undisturbed

Soil Corrosivity Tests

Soluble sulfate, chloride, resistively and pH tests were performed in accordance with California Test Methods 643, 417 and 422 to assess the degree of corrosivity of the subgrade soils with regard to concrete and normal grade steel. . The results of the tests are presented in the following table and attached in this appendix.

RESULTS OF CORROSIVITY TESTS
(CTM 417, CTM 422)

Sample Location	B-3 @ 3-4 ft
pH	8.1
Resistivity (Ohm-cm)	510
Sulfates (ppm)	330
Chlorides (ppm)	180

Natural Moisture and Density

Date: September 1, 2015

Job No: 766
Client: PALOMAR COMMUNITY COLLEGE DISTRICT
Address: 1140 W. Mission Blvd
San Marcos, CA 92069

Report No: 3937
ENGINEER: Guillaume Gau, GE
Project: Temporary Parking Lot
Sampled By: G. Custenborder
Date Received: 8/6/15

Lab Number	111970	111971	111972	111973	111974
Exploration No.	B-1	B-1	B-1	B-2	B-2
Depth, feet	5-6.5	11-11.5	15-16.5	6-6.5	10-11.5
Moisture Content, %	17.6	20.9	19.3	15.4	22.5
Dry Density, pcf.	- -	104.8	- -	115.9	- -

Lab Number	111975	111977	111978	111979
Exploration No.	B-2	B-3	B-3	B-3
Depth, feet	16-16.5	6-6.5	10-11.5	16-16.5
Moisture Content,	15.0	13.5	29.8	28.7
Dry Density, pcf.	114.1	121.3	- -	91.3

Reviewed by:
Sam Koohi
Project Geotechnical Engineer

NV5

RESISTANCE "R" VALUE TEST

(ASTM D2844/CTM301)

September 1, 2015

Palomar Community College District
2015 Westwind Dr
Bakersfield, CA 93301

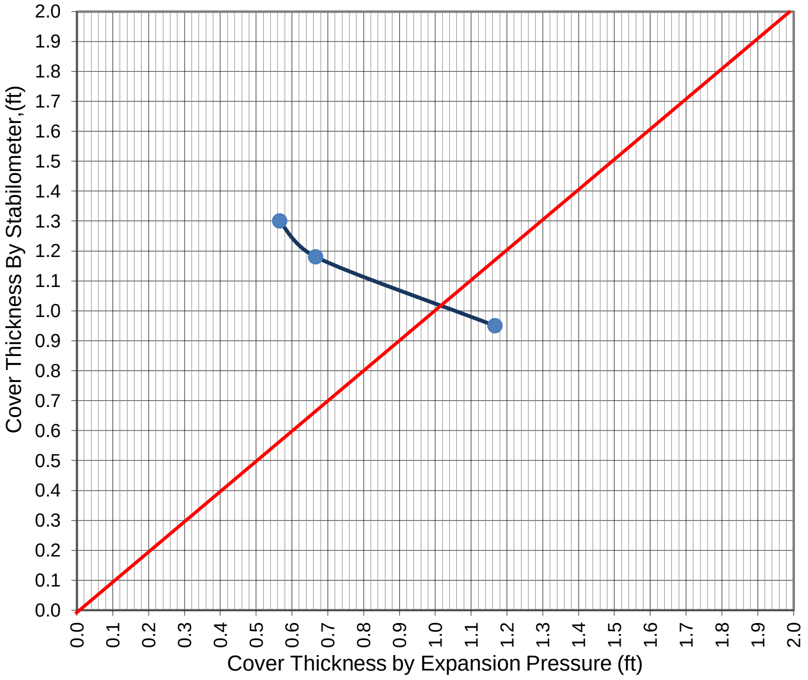
JOB No: **766**
LAB No: 111976

Project: Temporary Parking Lot

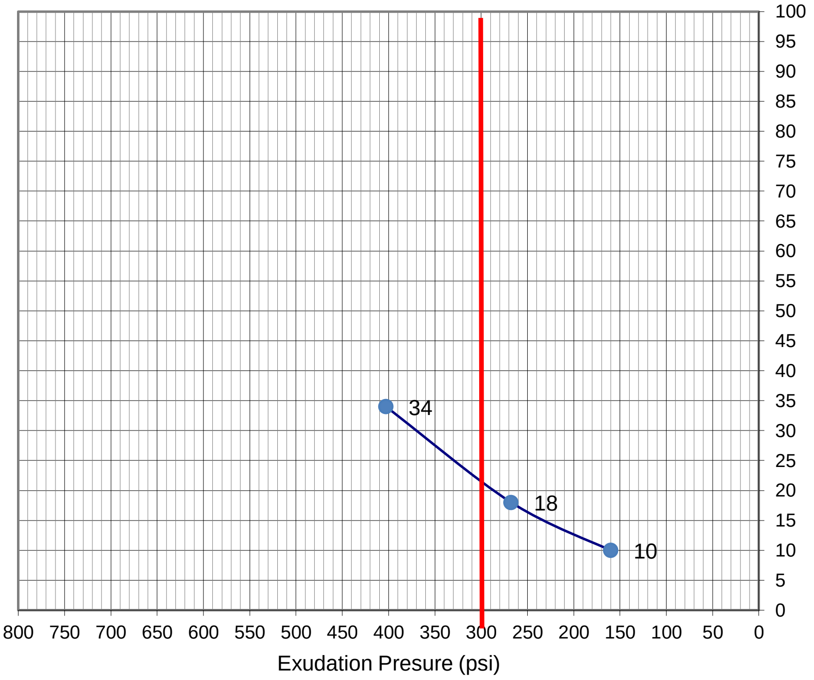
Material: Brown Silty Clayey SAND (SC-SM)
Location: B-3 @ 2-4 ft.
Sampled By: G. Custenborder
Date Received: 8/6/15

Report No: **3937**

EXPANSION PRESSURE CHART



EXUDATION PRESSURE CHART



TEST SPECIMEN	A	B	C	D
COMP. FOOT PRESSURE, psi	295	170	70	
INITIAL MOISTURE %	1.3	1.3	1.3	
MOISTURE @ COMPACTION %	12.2	13.1	14.8	
DRY DENSITY, pcf	121.8	120.0	115.2	
EXUDATION PRESSURE, psi	403	268	160	
STABILOMETER VALUE 'R'	34	18	10	

R-VALUE BY EXUDATION	22
R-VALUE BY EXPANSION	30
R-VALUE AT EQUILIBRIUM	**22**

Reviewed By:
Sam Koohi
Project Geotech Engineer

Project No. **766.00**
Client: **Palomar Community College District**
Proj. Name: **Temporary Parking Lot**
Location: **Palomar Community College, San Marcos, CA**
Sample date **8/5/2015** Sample Location: **2-4 ft.** Boring No: **B-3**

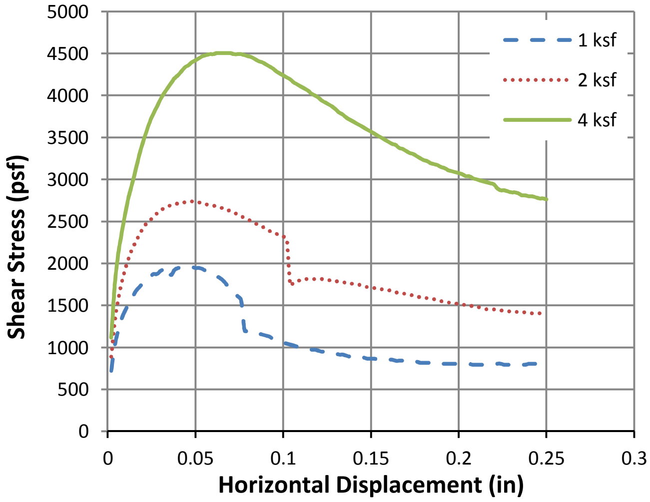
Report No.: **3937**
Lab No.: **111976**
Test Date: **8/10/2015**

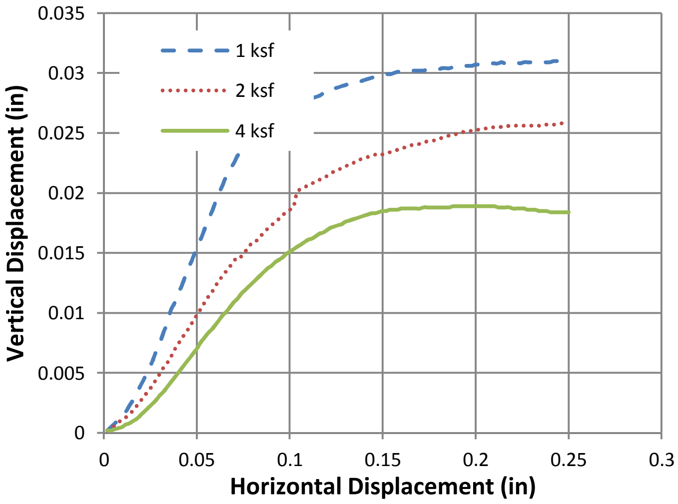
TEST DATA:

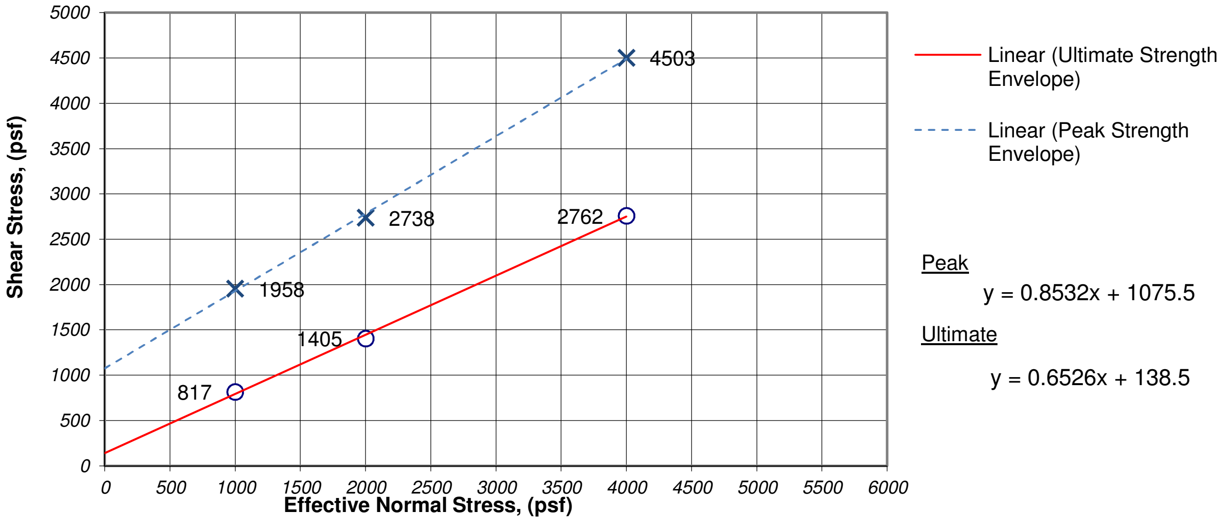
	Sample ID:	1 ksf	2 ksf	4 ksf
Initial	Water Content (%)	11.7	12.8	12.9
	Dry Density (%)	122.2	122.1	123.7
	Saturation (%)	92.8	101.4	107.6
Final	Water Content (%)	17.3	17.2	16.5
	Dry Density (%)	122.2	122.1	123.7
	Saturation (%)	137.5	135.8	138.0
Normal Stress (psf)		1000	2000	4000
Ultimate Shear Stress (psf)		817	1405	2762
Peak Shear Stress (psf)		1958	2738	4503

Sample Type: Undisturbed

Description: Brown Silty Clayey SAND (SM-SC)







Peak Cohesion, C'(psf): **1076**
Peak Friction,Φ' (deg): **40.5**

Ultimate Cohesion, C'(psf): **139**
Ultimate Friction,Φ' (deg): **33.1**

DIRECT SHEAR TEST (ASTM D3080)

NV5
7895 Convoy Court, Suite 18
San Diego CA 92111
p. 858 715 5800 f. 858 715 5810

Reviewed By:

Figure No.:

LABORATORY REPORT

Telephone (619) 425-1993 Fax 425-7917 Established 1928

CLARKSON LABORATORY AND SUPPLY INC.
350 Trousdale Dr. Chula Vista, Ca. 91910 www.clarksonlab.com
ANALYTICAL AND CONSULTING CHEMISTS

Date: August 19, 2015
Purchase Order Number: 15-0339
Sales Order Number: 28087
Account Number: TESE.R1
To:

NV5 West Inc
7895 Convoy Court, Suite 18
San Diego, CA 92111
Attention: Michelle Albrecht

Laboratory Number: SO5768 Customers Phone: 858-715-5800
Fax: 858-715-5810

Sample Designation:

One soil sample received on 08/12/15 at 1:10pm taken on 08/05/15 from Palomar College Temporary Parking Lot Job#766 marked as Boring 3 @ 2-4 Lab#111976 Report#3937.

Analysis By California Test 643, 1999, Department of Transportation Division of Construction, Method for Estimating the Service Life of Steel Culverts.

pH 8.1

Water Added (ml)	Resistivity (ohm-cm)
10	5000
5	1800
5	1100
5	660
5	570
5	540
5	510
5	540
5	560

23 years to perforation for a 16 gauge metal culvert.
30 years to perforation for a 14 gauge metal culvert.
42 years to perforation for a 12 gauge metal culvert.
53 years to perforation for a 10 gauge metal culvert.
65 years to perforation for a 8 gauge metal culvert.

Water Soluble Sulfate	Calif. Test 417	0.018% (180ppm)
Water Soluble Chloride	Calif. Test 422	0.030% (300ppm)

Laura Torres

Laura Torres
LT/ram

APPENDIX B

Exploratory Boring Logs

Sampling Methods

Representative bulk-disturbed and relatively undisturbed drive samples were retrieved during exploratory drilling at selected depths appropriate to the investigation. The samples were labeled in the field and transported to NV5's laboratory for observation, evaluation, and testing. The drive samples were obtained using the California Modified Split Spoon (CAL) and Standard Penetration Test (SPT) samplers, as described below.

California Modified Split Spoon (CAL) Sampler

A split-barrel drive sampler was driven with a 140-pound hammer allowed to drop freely 30 inches in general accordance with ASTM D1587. The sampler has external and internal diameters of approximately 3.0 and 2.4 inches, respectively, and the inside of the sampler is lined with 1-inch-long brass rings. The drive sampler was driven a maximum of 18 inches (or to refusal) and the number of blows per 6-inch interval, or any portion thereof, were recorded during sampling and are presented on the logs of the borings. The relatively undisturbed soil samples within the rings were removed, sealed, and transported to the laboratory for observation and testing.

Standard Penetration Test (SPT) Sampler

A split-barrel drive sampler was driven with a 140-pound hammer allowed to drop freely 30 inches in general accordance with ASTM D1586. The sampler has external and internal diameters of 2.0 and 1.4 inches, respectively. The drive sampler was driven a maximum of 18 inches (or to refusal) and the number of blows per 6-inch interval were recorded during sampling and are presented on the logs of the borings. The numbers of blows for the last two of three 6-inch intervals, or any portion thereof, were recorded during sampling and are presented in the logs of borings (i.e., uncorrected N-value). The soil samples obtained in the interior of the barrel were measured, removed, sealed and transported to the laboratory for observation and testing.

Note:
The penetration resistance (blows/foot) shown on the logs of the exploratory borings represents field penetration that has not been corrected for overburden pressure, sampler size, hammer type, borehole diameter, rod length, sampling method or any other correction factor.

Logging Methods

Earth materials encountered during the field investigation were classified in accordance with the Unified Soil Classification System (USCS/ASTM D2487) and augmented with ASTM Standard Testing for Soil (see Appendix B). The number of blows recorded for the last twelve inches of the drive sampler was used to determine the uncorrected "N-value" in accordance with ASTM D1586. The uncorrected "N-value" was used to determine consistency of cohesive soils (clays and silts) and apparent density of granular soils (sands and gravels) using the following charts (Chart 1 and Chart 2).

SAMPLE/SAMPLER TYPE GRAPHICS

AUGER SAMPLE
STANDARD PENETRATION SPLIT SPOON SAMPLER
BULK / GRAB SAMPLE
CALIFORNIA MODIFIED SPLIT SPOON SAMPLER
SHELBY TUBE SAMPLER
HQ ROCK CORE SAMPLE
NQ ROCK CORE SAMPLE

GROUNDWATER LEVEL GRAPHICS

▼ WATER LEVEL (during drilling operations)
▽ WATER LEVEL (immediately after drilling completion)
▼ WATER LEVEL (additional levels after drilling completion)
≈ OBSERVED SEEPAGE

NOTES

- The report and graphics key are an intergral part of these logs. All data and interpretations in this log are subject to the explanations and limitations stated in the report.
- Lines separating strata on the logs represent approximate boundaries only. Actual transitions my be gradual or differ from those shown.
- No warranty is provided as to the continuity of soil or rock conditions between individual sample locations.
- Logs represent general soil or rock conditions observed at the point of exploration on the date indicated.
- In general, Unified Soil Classification System (USCS) designationspresented on the logs were based on visual classification in thefield and were modified where appropriate based on gradation andindex property testing.
- Fine grained soils that plot within the hatched area on the Plasticity Chart, and coarse grained soils with between 5 and 12% passing the No. 200 sieve require dual USCS symbols, ie., GW-GM, GP-GM, GW-GC, GP-GC, GC-GM, SW-SM, SP-SM, SW-SC, SP-SC, SC-SM.
- If sampler is not able to be driven at least 6 inches then Y/X indicates Y number of blows required to drive the identified sampler X inches with a 140 pound hammer falling 30 inches.

UNIFIED SOIL CLASSIFICATION SYSTEM (ASTM D 2487)

COARSE GRAINED SOILS (More than half of materials is larger than the #200 sieve)	**GRAVELS** (More than half of coarse fraction is larger than the #4 sieve)	CLEAN GRAVEL WITH <5% FINES	Cu≥4 and 1≤Cc≤3	**GW**	WELL-GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE OR NO FINES
			Cu<4 and/or 1>Cc>3	**GP**	POORLY-GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE OR NO FINES
		GRAVELS WITH 5 TO 12% FINES	Cu≥4 and 1≤Cc≤3	**GW-GM**	WELL-GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE FINES
			Cu<4 and/or 1>Cc>3	**GW-GC**	WELL-GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE CLAY FINES
			Cu<4 and/or 1>Cc>3	**GP-GM**	POORLY-GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE FINES
				GP-GC	POORLY-GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE CLAY FINES
		GRAVELS WITH >12% FINES		**GM**	SILTY GRAVELS, GRAVEL-SILT-SAND MIXTURES
				GC	CLAYEY GRAVELS, GRAVEL-SAND-CLAY MIXTURES
				GC-GM	CLAYEY GRAVELS, GRAVEL-SAND-CLAY-SILT MIXTURES
	SANDS (More than half of coarse fraction is smaller than the #4 sieve)	CLEAN SANDS WITH <5% FINES	Cu≥6 and 1≤Cc≤3	**SW**	WELL GRADED SANDS, SAND-GRAVEL MIXTURES WITH LITTLE OR NO FINES
			Cu<6 and/or 1>Cc>3	**SP**	POORLY-GRADED SANDS, SAND-GRAVEL MIXTURES WITH LITTLE OR NO FINES
		SAND WITH 5 TO 12% FINES	Cu≥6 and 1≤Cc≤3	**SW-SM**	WELL-GRADED SANDS, SAND-GRAVEL MIXTURES WITH LITTLE FINES
				SW-SC	WELL-GRADED SANDS, SAND-GRAVEL MIXTURES WITH LITTLE CLAY FINES
			Cu>6 and/or 1<Cc>3	**SP-SM**	POORLY GRADED SANDS, SAND-GRAVEL MIXTURES WITH LITTLE FINES
				SP-SC	POORLY-GRADED SANDS, SAND-GRAVEL MIXTURES WITH LITTLE CLAY FINES
		SANDS WITH >12% FINES		**SM**	SILTY SANDS, SAND-GRAVEL-SILT MIXTURES
				SC	CLAYEY SANDS, SAND-GRAVEL-CLAY MIXTURES
				SC-SM	CLAYEY SANDS, SAND-SILT-CLAY MIXTURES

FINE GRAINED SOILS (More than half of material is smaller than the #200 sieve)	SILTS AND CLAYS (Liquid Limit less than 50)	**ML**	INORGANIC SILTS AND VERY FINE SANDS, SILTY OR CLAYEY FINE SANDS, SILTS WITH SLIGHT PLASTICITY
		CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
		CL-ML	INORGANIC CLAYS-SILTS OF LOW PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SITLY CLAYS, LEAN CLAYS
		OL	ORGANIC SILTS & ORGANIC SILTY CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS (Liquid Limit greater than 50)	**MH**	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SAND OR SILT
		CH	INORGANIC CLAYS OF HIGH PLASTICITY FAT CLAYS
		OH	ORGANIC CLAYS & ORGANIC SILTS OF MEDIUM-TO-HIGH PLASTICITY

NV5
15092 Avenue of Science, Suite 200
San Diego, CA 92128
Tel: (858) 385-0500, Fax: (858) 385-0400

Project No: **113820-0001266.00**

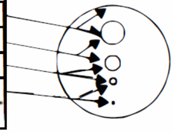
Date: **November 2020**

BORING LOG LEGEND
Proposed Student Union Building Southwestern College – Main Campus
900 Otay Lakes Road
Chula Vista, CA

CHART 1

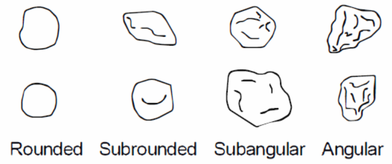
GRAIN SIZE

DESCRIPTION		SIEVE SIZE	GRAIN SIZE	APPROXIMATE SIZE
Boulders		>12 in.	>12 in. (304.8 mm.)	Larger than basketball-sized
Cobbles		3 - 12 in.	3 - 12 in. (76.2 - 304.8 mm.)	Fist-sized to basketball-sized
Gravel	coarse	3/4 - 3 in.	3/4 - 3 in. (19 - 76.2 mm.)	Thumb-sized to fist-sized
	fine	#4 - 3/4 in.	0.19 - 0.75 in. (4.75 - 19 mm.)	Pea-sized to thumb-sized
Sand	coarse	#10 - #4	0.079 - 0.19 in. (2 - 4.75 mm.)	Rock salt-sized to pea-sized
	medium	#40 - #10	0.017 - 0.079 in. (0.43 - 2 mm.)	Sugar-sized to rock salt-sized
	fine	#200 - #40	0.0029 - 0.017 in. (0.074 - 0.43 mm.)	Four-sized to sugar-sized
Fines		Passing #200	<0.0029 in. (0.074 mm.)	Flour-sized and smaller



ANGULARITY

DESCRIPTION	CRITERIA
Angular	Particles have sharp edges and relatively plane sides with unpolished surfaces
Subangular	Particles are similar to angular description but have rounded edges
Subrounded	Particles have nearly plane sides but have well-rounded edges
Rounded	Particles have smoothly curved sides and no edges



PLASTICITY

DESCRIPTION	CRITERIA
Non-plastic	A 1/8-in. (3 mm.) thread cannot be rolled at any water content.
Low (L)	The thread can barely be rolled and the lump or thread cannot be formed when drier than the plastic limit.
Medium (M)	The thread is easy to roll and not much time is required to reach the plastic limit. The thread cannot be rerolled after reaching the plastic limit. The lump or thread crumbles when drier than the plastic limit.
High (H)	It takes considerable time rolling and kneading to reach the plastic limit. The thread can be rerolled several times after reaching the plastic limit. The lump or thread can be formed without crumbling when drier than the plastic limit.

APPARENT DENSITY - COARSE-GRAINED SOIL

APPARENT DENSITY	SPT-N_{60} (#blows/ft)	CALIFORNIA MODIFIED SPLIT SPOON SAMPLER (#blows/ft)
Very Loose	<4	<5
Loose	4 - 10	6 - 15
Medium Dense	11 - 30	16 - 45
Dense	31 - 50	46 - 75
Very Dense	>50	>75

STRUCTURE

DESCRIPTION	CRITERIA
Stratified	Alternating layers of varying material or color with layers at least 1/4-in. (6 mm.) thick, note thickness
Laminated	Alternating layers of varying material or color with layers less than 1/4-in. (6 mm.) thick, note thickness
Fissured	Breaks along definite planes of fracture with little resistance to fracturing
Slickensided	Fracture planes appear polished or glossy, sometimes striated
Blocky	Cohesive soil that can be broken down into smaller angular lumps which resist further breakdown
Lensed	Inclusion of small pockets of different soils, such as small lenses of sand scattered through a mass of clay; note thickness
Homogeneous	Same color and appearance throughout

MOISTURE CONTENT

DESCRIPTION	CRITERIA
Dry	Absence of moisture, dusty, dry to the touch
Moist	Damp but no visible water
Wet	Visible free water, usually soil is below groundwater table

REACTION WITH HYDROCHLORIC ACID

DESCRIPTION	CRITERIA
None	No visible reaction
Weak	Some reaction, with bubbles forming slowly
Strong	Violet reaction, with bubbles forming immediately

CONSISTENCY - FINE-GRAINED SOIL

CONSISTENCY	SPT-N_{60} (#blows/ft)	CRITERIA	P.P. (TSF)
Very Soft	<2	Thumb will penetrate soil more than 1 in. (25 mm.)	<0.25
Soft	2 - 4	Thumb will penetrate soil about 1 in. (25 mm.)	0.25 - 0.5
Medium Stiff	5 - 8	Thumb will indent soil about 1/4-in. (6 mm.)	0.5 - 1.0
Stiff	9 - 15	Thumb will not indent soil but readily indented with thumbnail	1.0 - 2.0
Very Stiff	16 - 30	Readily indented with nail not thumb	2.0 - 4.0
Hard	>30	Thumbnail will not indent soil	>4.0

CEMENTATION

DESCRIPTION	CRITERIA
Weakly	Crumbles or breaks with handling or slight finger pressure
Moderately	Crumbles or breaks with considerable finger pressure
Strongly	Will not crumble or break with finger pressure

15092 Avenue of Science, Suite 200
San Diego, CA 92128
Tel: (858) 385-0500, Fax: (858) 385-0400

Project No: **113820-0001266.00**

Date: **November 2020**

SOIL CLASSIFICATION
Proposed Student Union Building Southwestern College – Main Campus
900 Otay Lakes Road
Chula Vista, CA

CHART
2

Boring Log

Sheet 1 of 1

Date		Project Number	Project	Boring No.
	Started: 3-17-21	113821-0001313.00	Palomar College Athletics	B-1
	Completed: 3-17-21			
	Hammer Efficiency: 71.2%	Rig Type: CME-75 (Baja)	Logged By: A. Hespeler	Reviewed By: M. Chirumalla

Latitude: 33.147842

Longitude: -117.183040

Surface Elevation: 575'

Location: Lower Base Surface Parking Lot

Sample Type

G - Bulk / Grab Sample

SPT - 2" O.D. 1.4" I.D. Tube Sample

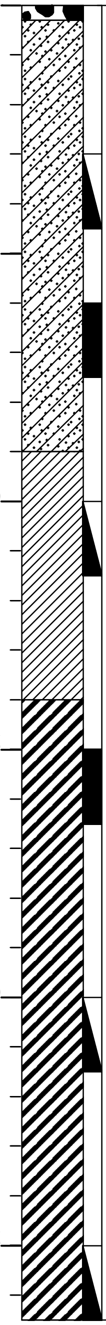
CAL - 3 " O.D. 2.4" I.D. Ring Sample

NR - No Recovery

* - Uncorrected Blow Counts

Groundwater

Depth (ft)	Hour	Date
7	9AM	3-17-21



Groundwater Depth (ft.)	Depth (ft.)	Sample ID	Penetration Resistance (Blows per 6 in.)	Moisture Content (%)	Dry Weight (pcf)	Other Tests and Remarks	USCS Class.	Visual Classification
	0						GW	0.3' Cement-treated Class II base at surface (4 inches). El. 574.7'
							SC	**FILL:** Clayey SAND (SC), red-brown, moist, fine- to medium-grained sand.
		SPT- 1	5 9 13			No. 200 Sieve Wash.		Medium dense.
	5							
▼		CAL- 1	6 25 28	10.9	119.0	Moisture and Density.		Very dense. Wet layer encountered from 7 to 10 ft. bgs.
								9.0' El. 566.0'
	10						CL	**ALLUVIUM (Qa):** Sandy Lean CLAY (CL), gray and red-brown, moist, very stiff, fine- to coarse-grained sand, with fine and coarse subangular gravel.
		SPT- 2	6 15 14			P.P.= +4.5 TSF		
								14.0' El. 561.0'
	15						CH	**SANTIAGO FORMATION (Tsa) - MUDSTONE:** Drills as: Fat CLAY (CH), gray with red iron oxide staining, moist, hard.
		CAL- 2	7 9 15			P.P.= 4.0 TSF		
	20							
		SPT- 3	5 6 6					
	25							
		SPT- 4	5 6 8					26.5' El. 548.5'

Notes: Boring terminated at 26.5 ft. bgs. Groundwater encountered at 7 ft. bgs. Backfilled with hydrated bentonite chips up to 5 ft bgs, then capped with soil cuttings to surface. Drilled with 8-in. O.D. HSA's.

Boring Log

Sheet 1 of 1

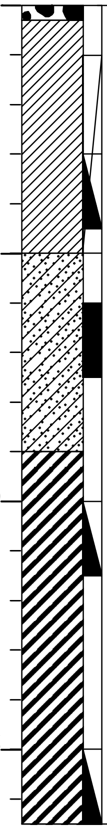
Date Started: 3-17-21 Completed: 3-17-21	**Project Number** 113821-0001313.00	**Project** Palomar College Athletics	**Boring No.** B-2
Hammer Efficiency: 71.2%	**Rig Type:** CME-75 (Baja)	**Logged By:** A. Hespeler	**Reviewed By:** M. Chirumalla
Latitude: 33.147609	**Longitude:** -117.183271	**Surface Elevation:** 575'	

Location: Lower Base Surface Parking Lot

Sample Type
G - Bulk / Grab Sample
SPT - 2" O.D. 1.4" I.D. Tube Sample
CAL - 3 " O.D. 2.4" I.D. Ring Sample
NR - No Recovery
* - Uncorrected Blow Counts

Groundwater

Depth (ft)	Hour	Date



Groundwater Depth (ft.)	Depth (ft.)	Sample ID	Penetration Resistance (Blows per 6 in.)	Moisture Content (%)	Dry Weight (pcf)	Other Tests and Remarks	USCS Class.	Visual Classification
	0						GW	0.3' Cement-treated Class II base at surface (4 inches). El. 574.7'
		G- 1 SPT- 1	5 8 8			Atterberg limits. Expansion Index. pH & Resistivity. Chloride & Sulfate Content.	CL	**FILL:** Sandy Lean CLAY (CL), dark brown, moist, fine- to medium-grained sand. Stiff / very stiff. 5.0' El. 570.0'
	5	CAL- 1	15 21 18	11.8	113.8	Moisture and Density.	SC	**ALLUVIUM (Qa):** Clayey SAND (SC), red-brown, moist, medium dense, fine- to coarse-grained sand, with some fine subangular gravel. 9.0' El. 566.0'
	10	SPT- 2	4 7 9			P.P.= 2.5 TSF	CH	**SANTIAGO FORMATION (Tsa) - MUDSTONE:** Drills as: Sandy Fat CLAY (CH), gray-brown, moist, hard, fine- to medium-grained sand.
	15	SPT- 3	4 8 10			P.P.= 4.0 TSF		16.5' El. 558.5'

Notes: Boring terminated at 16.5 ft. bgs. Groundwater not encountered. Backfilled with hydrated bentonite chips up to 5 ft bgs, then capped with soil cuttings to surface. Drilled with 8-in. O.D. HSA's.

Boring Log

Sheet 1 of 1

Date	Started: 3-17-21 Completed: 3-17-21	Project Number 113821-0001313.00	Project Palomar College Athletics	Boring No. B-3
	Hammer Efficiency: 71.2%	Rig Type: CME-75 (Baja)	Logged By: A. Hespeler	Reviewed By: M. Chirumalla
	Latitude: 33.147331	Longitude: -117.183213	Surface Elevation: 575'	

Location: Lower Base Surface Parking Lot

Sample Type
G - Bulk / Grab Sample
SPT - 2" O.D. 1.4" I.D. Tube Sample
CAL - 3 " O.D. 2.4" I.D. Ring Sample
NR - No Recovery
* - Uncorrected Blow Counts

Groundwater

Depth (ft)	Hour	Date

Groundwater Depth (ft.)	Depth (ft.)	Graphical Log	Sample Taken	Sample ID	Penetration Resistance (Blows per 6 in.)	Moisture Content (%)	Dry Weight (pcf)	Other Tests and Remarks	USCS Class.	Visual Classification
	0								GW	0.3' Cement-treated Class II base at surface (4 inches). El. 574.7'
									CL	**FILL:** Sandy Lean CLAY (CL), dark brown, moist, fine- to coarse-grained sand, with trace fine and coarse subangular gravel.
				G- 1 SPT- 1	4 6 12					Very stiff.
	5									5.0' El. 570.0'
				CAL- 1	9 16 38			P.P.= 4.0 TSF No. 200 Sieve Wash.	CL	**ALLUVIUM (Qa):** Clayey SAND (SC), gray with yellow and red mottling, moist, very stiff, fine- to coarse-grained sand, with trace fine and coarse subangular gravel.
										Wet layer encountered from 8 to 10 ft. bgs.
	10			SPT- 2	6 7 7			P.P.= 2.5 TSF		
										13.0' El. 562.0'
									CH/SC	**SANTIAGO FORMATION (Tsa) - INTERBEDDED MUDSTONE / SANDSTONE:** Drills as: Sandy Fat CLAY (CH) and Clayey SAND (SC), yellow and gray, moist, hard / dense, fine-grained sand.
	15			SPT- 3	6 15 16			P.P.= +4.5 TSF		
										16.5' El. 558.5'

Notes: Boring terminated at 16.5 ft. bgs. Wet layer encountered from 8 to 10 ft. bgs. Backfilled with hydrated bentonite chips up to 5 ft bgs, then capped with soil cuttings to surface. Drilled with 8-in. O.D. HSA's.

Boring Log

Sheet 1 of 1

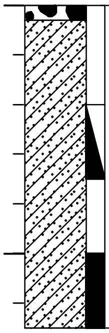
Date Started: 3-17-21 Completed: 3-17-21	**Project Number** 113821-0001313.00	**Project** Palomar College Athletics	**Boring No.** B-4
Hammer Efficiency: 71.2%	**Rig Type: CME-75 (Baja)**	**Logged By: A. Hespeler**	**Reviewed By: M. Chirumalla**
Latitude: 33.147002	**Longitude: -117.183076**	**Surface Elevation: 575'**	

Location: Lower Base Surface Parking Lot

Sample Type
G - Bulk / Grab Sample
SPT - 2" O.D. 1.4" I.D. Tube Sample
CAL - 3 " O.D. 2.4" I.D. Ring Sample
NR - No Recovery
* - Uncorrected Blow Counts

Groundwater

Depth (ft)	Hour	Date



Groundwater Depth (ft.)	Depth (ft.)	Graphical Log	Sample Taken	Sample ID	Penetration Resistance (Blows per 6 in.)	Moisture Content (%)	Dry Weight (pcf)	Other Tests and Remarks	USCS Class.	Visual Classification
	0								GW	0.3' Cement-treated Class II base at surface (4 inches). El. 574.7'
									SC	**FILL:** Clayey SAND (SC), dark brown, moist, loose, fine- to coarse-grained sand, with trace fine and coarse subangular gravel.
				SPT- 1	3 3 2					
	5			CAL- 1	5 11 15					Medium dense.
										6.5' El. 568.5'

Notes: Boring terminated at 6.5 ft. bgs. Groundwater not encountered. Backfilled with soil cuttings. Drilled with 8-in. O.D. HSA's.

NV5

Boring Log

Sheet 1 of 1

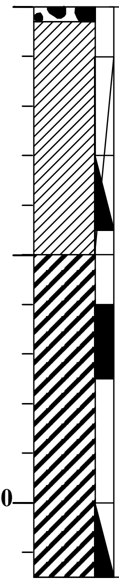
Date			
Started: 3-17-21 Completed: 3-17-21	Project Number 113821-0001313.00	Project Palomar College Athletics	Boring No. B-5
Hammer Efficiency: 71.2%	Rig Type: CME-75 (Baja)	Logged By: A. Hespeler	Reviewed By: M. Chirumalla
Latitude: 33.146999	Longitude: -117.182542	Surface Elevation: 575'	

Location: Lower Base Surface Parking Lot

Sample Type
G - Bulk / Grab Sample
SPT - 2" O.D. 1.4" I.D. Tube Sample
CAL - 3 " O.D. 2.4" I.D. Ring Sample
NR - No Recovery
* - Uncorrected Blow Counts

Groundwater

Depth (ft)	Hour	Date



Groundwater Depth (ft.) Depth (ft.)	Sample ID	Penetration Resistance (Blows per 6 in.)	Moisture Content (%)	Dry Weight (pcf)	Other Tests and Remarks	USCS Class.	Visual Classification
0						GW	0.3' Cement-treated Class II base at surface (4 inches). El. 574.7'
						CL	**FILL:** Sandy Lean CLAY (CL), dark brown, moist, fine- to coarse-grained sand, with trace fine and coarse subangular gravel.
	G- 1						
	SPT- 1	2 3 3					Medium stiff.
5							5.0' El. 570.0'
						CH	**ALLUVIUM (Qa):** Sandy Fat CLAY (CH), dark brown with yellow mottling, moist, very stiff, fine- to coarse-grained sand, with trace fine subangular gravel.
	CAL- 1	9 22 30	14.6	118.8	P.P.= 4.0 TSF Moisture and Density.		
							Becomes very moist at 9 ft. bgs.
10	SPT- 2	6 6 8					
							11.5' El. 563.5'

Notes: Boring terminated at 11.5 ft. bgs. Groundwater not encountered. Backfilled with hydrated bentonite chips to 5 ft. bgs, then capped with soil cuttings to surface. Drilled with 8-in. O.D. HSA's.

Boring Log

Sheet 1 of 1

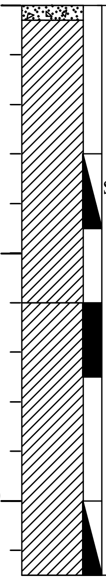
Date Started: 3-17-21 Completed: 3-17-21	**Project Number** 113821-0001313.00	**Project** Palomar College Athletics	**Boring No.** B-6
Hammer Efficiency: 71.2%	**Rig Type: CME-75 (Baja)**	**Logged By: A. Hespeler**	**Reviewed By: M. Chirumalla**
Latitude: 33.146907	**Longitude: -117.182454**	**Surface Elevation: 575'**	

Location: Lower Base Surface Parking Lot

Sample Type
G - Bulk / Grab Sample
SPT - 2" O.D. 1.4" I.D. Tube Sample
CAL - 3 " O.D. 2.4" I.D. Ring Sample
NR - No Recovery
* - Uncorrected Blow Counts

Groundwater

Depth (ft)	Hour	Date



Groundwater Depth (ft.)	Depth (ft.)	Sample ID	Penetration Resistance (Blows per 6 in.)	Moisture Content (%)	Dry Weight (pcf)	Other Tests and Remarks	USCS Class.	Visual Classification
	0							0.3' Concrete sidewalk at surface (3.5 inches). El. ~~574.7'~~
							CL	**FILL:** Sandy Lean CLAY (CL), dark brown, moist, fine- to coarse-grained sand, with trace fine and coarse subangular gravel.
		SPT- N/R	1 1 1					Soft. Becomes very moist from 3 to 6 ft. bgs.
	5							6.0' El. 569.0'
		CAL- 1	6 14 24			P.P.= 3.5 TSF	CL	**ALLUVIUM (Qa):** Sandy Lean CLAY (CL), yellow-brown and gray, moist, very stiff, fine- to coarse-grained sand, with trace fine subangular gravel.
	10	SPT- 1	7 13 16					11.5' El. 563.5'

Notes: Boring terminated at 11.5 ft. bgs. Groundwater not encountered. Backfilled with soil cuttings, and patched with 4-in. of rapid-set concrete. Drilled with 8-in. O.D. HSA's.

Boring Log

Sheet 1 of 1

Date	**Started: 3-18-21** **Completed: 3-18-21**	**Project Number** **113821-0001313.00**	**Project** **Palomar College Athletics**	**Boring No.** **B-7**
	Hammer Efficiency: 71.2%	**Rig Type: CME-75 (Baja)**	**Logged By: A. Hespeler**	**Reviewed By: M. Chirumalla**
	Latitude: 33.147246	**Longitude: -117.182423**	**Surface Elevation: 575'**	

Location: Lower Base Surface Parking Lot

Sample Type
G - Bulk / Grab Sample
SPT - 2" O.D. 1.4" I.D. Tube Sample
CAL - 3 " O.D. 2.4" I.D. Ring Sample
NR - No Recovery
* - Uncorrected Blow Counts

Groundwater

Depth (ft)	Hour	Date

Groundwater Depth (ft.)	Depth (ft.)	Graphical Log	Sample Taken	Sample ID	Penetration Resistance (Blows per 6 in.)	Moisture Content (%)	Dry Weight (pcf)	Other Tests and Remarks	USCS Class.	**Visual Classification**
	0								GW	0.3' Cement-treated Class II base at surface (4 inches). El. 574.7'
									SC	**FILL:** Clayey SAND (SC), dark brown, moist, fine- to medium-grained sand.
				SPT- 1	2 5 5	20.3		Moisture. No. 200 Sieve Wash.		Loose.
	5									5.0' El. 570.0'
									CH	**ALLUVIUM (Qa):** Sandy Fat CLAY (CH), dark brown with yellow mottling, moist, very stiff, fine- to coarse-grained sand, with trace fine and coarse subangular gravel.
				CAL- 1	3 6 11	17.3	112.2	P.P.= 4.0 TSF Moisture & Density.		
	10			SPT- 2	6 10 14			P.P.= +4.5 TSF		Becomes light gray-brown at 10 ft. bgs.
										13.0' El. 562.0'
									CH	**SANTIAGO FORMATION (Tsa) - MUDSTONE:** Drills as: Sandy Fat CLAY (CH), light gray with yellow iron oxide staining, moist, very stiff, fine-grained sand.
	15			SPT- 3	5 8 12			P.P.= +4.5 TSF		
										16.5' El. 558.5'

Notes: Boring terminated at 16.5 ft. bgs. Groundwater not encountered. Backfilled with soil cuttings. Drilled with 8-in. O.D. HSA's.

Boring Log

Sheet 1 of 1

Date		Project Number	Project	Boring No.
Started: 3-18-21 Completed: 3-18-21		113821-0001313.00	Palomar College Athletics	B-8
Hammer Efficiency: 71.2%	Rig Type: CME-75 (Baja)		Logged By: A. Hespeler	Reviewed By: M. Chirumalla
Latitude: 33.147778	Longitude: -117.182662		Surface Elevation: 575'	

Location: Lower Base Surface Parking Lot

Sample Type

G - Bulk / Grab Sample
SPT - 2" O.D. 1.4" I.D. Tube Sample
CAL - 3 " O.D. 2.4" I.D. Ring Sample
NR - No Recovery
* - Uncorrected Blow Counts

Groundwater

Depth (ft)	Hour	Date

Groundwater Depth (ft.)	Depth (ft.)	Graphical Log	Sample Taken	Sample ID	Penetration Resistance (Blows per 6 in.)	Moisture Content (%)	Dry Weight (pcf)	Other Tests and Remarks	USCS Class.	Visual Classification
	0	●							GW	0.3' Cement-treated Class II base at surface (4 inches). El. 574.7'
		▨							CL	**FILL:** Sandy Lean CLAY (CL), dark brown, moist, fine- to coarse-grained sand. 2.0' El. 573.0'
		▨	▲	SPT- 1	3 4 9				CL	**ALLUVIUM (Qa):** Sandy Lean CLAY (CL), gray-brown with red iron oxide staining, moist, stiff to very stiff, fine- to coarse-grained sand, with trace fine and coarse subangular gravel.
	5	▨	■	CAL- 1	5 14 26			P.P.= +4.5 TSF		6.5' El. 568.5'

Notes: Boring terminated at 6.5 ft. bgs. Groundwater not encountered. Backfilled with soil cuttings. Drilled with 8-in. O.D. HSA's.

Boring Log

Sheet 1 of 1

Date			
Started: 3-18-21 Completed: 3-18-21	Project Number 113821-0001313.00	Project Palomar College Athletics	Boring No. B-9
Hammer Efficiency: 71.2%	Rig Type: CME-75 (Baja)	Logged By: A. Hespeler	Reviewed By: M. Chirumalla
Latitude: 33.147456	Longitude: -117.181993	Surface Elevation: 575'	

Location: Softball Practice Field

Sample Type
G - Bulk / Grab Sample
SPT - 2" O.D. 1.4" I.D. Tube Sample
CAL - 3 " O.D. 2.4" I.D. Ring Sample
NR - No Recovery
* - Uncorrected Blow Counts

Groundwater

Depth (ft)	Hour	Date
22	1PM	3-18-21

Groundwater Depth (ft.)	Depth (ft.)	Graphical Log	Sample Taken	Sample ID	Penetration Resistance (Blows per 6 in.)	Moisture Content (%)	Dry Weight (pcf)	Other Tests and Remarks	USCS Class.	Visual Classification
	0								SM	0.5' Lawn grass over topsoil at surface (6-inches). El. 574.5'
		▨							CL	**FILL:** Sandy Lean CLAY (CL), dark gray-brown, moist, fine- to coarse-grained sand. 2.0' El. 573.0'
		▨							CL	**ALLUVIUM (Qa):** Sandy Lean CLAY (CL), red-brown, slightly moist, very stiff, fine- to coarse-grained sand, with fine and coarse subangular gravel.
		▨	▲	SPT- 1	23 30/3"	10.2		Moisture.		
	5	▨	■	CAL- 1	29 44/6"	11.1	121.1	P.P.= 4.0 TSF Moisture & Density.		
	10	▨	▲	SPT- 2	7 12 13					Material turns light gray at 10 ft. bgs. 13.0' El. 562.0'
		▨							CH	**SANTIAGO FORMATION (Tsa) - MUDSTONE:** Drills as: Fat CLAY (CH), gray with red iron oxide staining, moist, hard.
	15	▨	■	CAL- 2	7 17 23			P.P.= +4.5 TSF		20.0' El. 555.0'
	20	▨	▲	SPT- 3	6 15 21				CH/SC	**SANTIAGO FORMATION (Tsa) - INTERBEDDED MUDSTONE / SANDSTONE:** Drills as: Sandy Fat CLAY (CH) and Clayey SAND (SC), gray with red iron oxide staining, moist, hard / dense, fine-grained sand.
▼		▨								Wet layer encountered from 22 to 24 ft. bgs.
	25	▨	▲	SPT- 4	10 40/6"					26.0' El. 549.0'

Notes: Boring terminated at 26 ft. bgs. Wet layer encountered at 22 to 24 ft. bgs. Backfilled with hydrated bentonite chips up to 4 ft bgs, then capped with soil cuttings, and patched with lawn grass plug. Drilled with 8-in. O.D. HSA's.

NV5

Boring Log

Sheet 1 of 1

Date Started: 3-18-21 Completed: 3-18-21	**Project Number** 113821-0001313.00	**Project** Palomar College Athletics	**Boring No.** B-10
Hammer Efficiency: 71.2%	Rig Type: CME-75 (Baja)	Logged By: A. Hespeler	Reviewed By: M. Chirumalla
Latitude: 33.146957	Longitude: -117.181491	Surface Elevation: 575'	

Location: Softball Practice Field

Sample Type
G - Bulk / Grab Sample
SPT - 2" O.D. 1.4" I.D. Tube Sample
CAL - 3 " O.D. 2.4" I.D. Ring Sample
NR - No Recovery
* - Uncorrected Blow Counts

Groundwater

Depth (ft)	Hour	Date

Groundwater Depth (ft.)	Depth (ft.)	Sample ID	Penetration Resistance (Blows per 6 in.)	Moisture Content (%)	Dry Weight (pcf)	Other Tests and Remarks	USCS Class.	Visual Classification
	0						SM	0.5' Lawn grass over topsoil at surface (6-inches). El. 574.5'
							CL	**FILL:** Sandy Lean CLAY (CL), gray-brown, moist, fine- to coarse-grained sand. 5.0' El. 570.0'
	5	CAL- 1	26 50/6"	12.4	105.8	P.P.= 4.0 TSF Moisture and Density. No. 200 Sieve.		**ALLUVIUM (Qa):** Sandy Lean CLAY (CL), light gray with red and yellow iron oxide staining, dry, very stiff, fine- to coarse-grained sand.
	10	SPT- 1	6 14 20	12.2		Moisture.	CL	
	15	SPT- 2	4 15 25					16.5' El. 558.5'

Notes: Boring terminated at 16.5 ft. bgs. Groundwater not encountered. Backfilled with soil cuttings and patched with grass plug. Drilled with 8-in. O.D. HSA's.

APPENDIX C

Laboratory Test Results

LABORATORY TEST RESULTS

In-situ Moisture and Density Tests

The in-situ moisture content and dry density of selected samples obtained from the test borings were evaluated in general accordance with the latest versions of ASTM D2216 and D2937 laboratory test methods. The methods involve obtaining the moist weight of the sample and then drying the sample to obtain its dry weight. The moisture content is calculated by taking the difference between the wet and dry weights, dividing it by the dry weight of the sample and expressing the result as a percentage. The results of the in-situ moisture content and dry density tests are presented in the following table and on the logs of exploratory borings in Appendix A.

RESULTS OF MOISTURE CONTENT AND DENSITY TESTS
(ASTM D2216 and D2937)

Sample Location / Depth (ft.)	Moisture Content (percent)	Dry Density (pounds per cubic foot)
Boring 1 @ 6 – 7.5	10.9	119.0
Boring 2 @ 6 – 7.5	11.8	113.8
Boring 5 @ 6 – 7.5	14.6	118.8
Boring 7 @ 3 – 4.5	20.3	Not tested
Boring 7 @ 6 – 7.5	17.3	112.2
Boring 9 @ 3 – 3.75	10.2	Not tested
Boring 9 @ 6 – 7	11.1	121.1
Boring 10 @ 5 – 6	12.4	105.8
Boring 10 @ 10 – 11.5	12.2	Not tested

Classification

Soils were visually and texturally classified in accordance with the Unified Soil Classification System (USCS/ASTM D2487) and ASTM D2488. Soil classifications are indicated on the logs of the exploratory borings presented in Appendix A.

Particle-size Distribution Tests

An evaluation of the grain-size distribution of selected soil samples was performed in general accordance with the latest versions of ASTM D6913 and ASTM D1140 (No. 200 sieve wash). The test results were utilized in evaluating the soil classifications in accordance with the Unified Soil Classification System. The results of the No. 200 sieve wash tests are presented below and attached in this appendix.

RESULTS OF NO. 200 SIEVE WASH TESTS
(ASTM D1140)

Sample Location / Depth (ft)	Material Type (USCS)	Particles Passing the No. 200 Sieve (%)
Boring 1 @ 3 –3.5 feet	Clayey SAND (SC)	46.0
Boring 3 @ 6 –7.5 feet	Sandy Lean CLAY (CL)	52.8
Boring 7 @ 3 – 3.5 feet	Clayey SAND (SC)	14.3
Boring 10 @ 5 – 6 feet	Sandy Lean CLAY (CL)	87.3

Atterberg Limits Test

An Atterberg limits test was performed on a select soil sample in general accordance with ASTM D4318. The test is useful to aid in classification of soils and in evaluating their expansion and strength characteristics. Test results are summarized below and attached in this appendix.

RESULTS OF ATTERBERG LIMITS TEST
(ASTM D4318)

Sample Location & Depth (ft.)	Material Type (USCS)	Liquid Limit	Plastic Limit	Plasticity Index
B – 2 @ 1 – 5 feet	Brown Sandy Lean CLAY (CL)	40	17	23

Expansion Index Test

Expansion index tests were performed on bulk-disturbed samples of the on-site materials encountered. The tests were performed in general accordance with ASTM D4829. The test results are summarized below and attached in this appendix.

RESULTS OF EXPANSION INDEX TESTS
(ASTM D4829)

Sample Location & Depth (ft.)	Material Type (USCS)	Initial Moisture Content (%)	Final Moisture Content (%)	Dry Density (pcf)	Initial Saturation (%)	Expansion Index	Potential Expansion
B2 @ 1 – 5 feet	Brown Sandy Lean CLAY (CL)	9.3	22.7	111.4	49.3	53	Medium

Soil Corrosivity Tests

Water soluble sulfate & chloride, resistivity and pH tests were performed by Clarkson Laboratory and Supply Inc., in general accordance with California Test Methods 643, 417 and 422, to provide an indication of the degree of corrosivity of the subgrade soils at locations tested with regard to concrete and normal grade steel.

RESULTS OF CORROSIVITY TESTS
(CTM 417, CTM 422 and CTM 643)

Sample Location & Depth (ft)	B – 2 @ 1 – 5 feet
Material Type (USCS)	Brown Sandy Lean CLAY (CL)
pH	8.3
Minimum Resistivity (Ohm-cm)	490
Water Soluble Sulfates (ppm)	170
Water Soluble Chlorides (ppm)	440

NV5

In-Situ Moisture & Density

(ASTM D2216 & ASTM D2937)

Date:	4/15/2021	Job Number:	1310
Client:	Palomar Community College District	Report Number:	8421
Client Address:	San Marcos, CA	Lab Number:	121334-121325
Project Name:	Palomar Community College Athletic Field Improvements		121328-121334
Project Address:	San Marcos, CA		
Date Sampled:	3/17-18/2021	Sampled By:	Adam Hespeler
Date Recieved:	3/30/2021	Submitted By:	Adam Hespeler

Lab Number	121324	121325	121328	121329	121330
Exploration No.	B1	B2	B5	B7	B7
Depth, ft.	6-7.5	6-7.5	6-7.5	3-3.5	6-7.5
Mositure Content, %	10.9	11.8	14.6	20.3	17.3
Dry Density, pcf	119.0	113.8	118.8	-	112.2

Lab Number	121331	121332	121333	121334	
Exploration No.	B9	B9	B10	B10	
Depth, ft.	3-3.5	6-7.5	5-6	10-11.5	
Mositure Content, %	10.2	11.1	12.4	12.2	
Dry Density, pcf	-	121.1	105.8	-	

Respectfully Submitted,
NV5 West, Inc.

Carl Henderson

Carl Henderson, PhD, PE, GE
CDO / SoCal CQA Group Director

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Material Finer Than 75µm(No.200) Seive Report

(ASTM D1140)

Date: 4/15/2021
Client: Palomar Community College District
Client Address: San Marcos, CA
Project Name: Palomar
Project Address: San Marcos, CA

Job Number: 1310
Report Number: 8421
Lab Number: 121323-121333

Date Sampled 3/17-18/2021
Date Submitted: 3/30/2020

Sampled By: Adam Hespeler
Submitted By: Adam Hespeler

Lab Number	121323	121327	121329	121333
Material Location	B1 @ 3'-3.5'	B3 @ 6'-7.5'	B7 @ 3'-3.5'	B10 @ 5'-6'
Material Type	Red Brown Clayey SAND (SC)	Yellow Brown Sandy CLAY (CL)	Brown Clayey SAND (SC)	Tan Gray Fine Sandy CLAY (CL)
Percent Passing #200 Sieve (75µm) , %	46.0	52.8	14.3	87.3

Respectfully Submitted,
NV5 West, Inc.

Carl Henderson

Carl Henderson, PhD, PE, GE
CDO / SoCal CQA Group Director

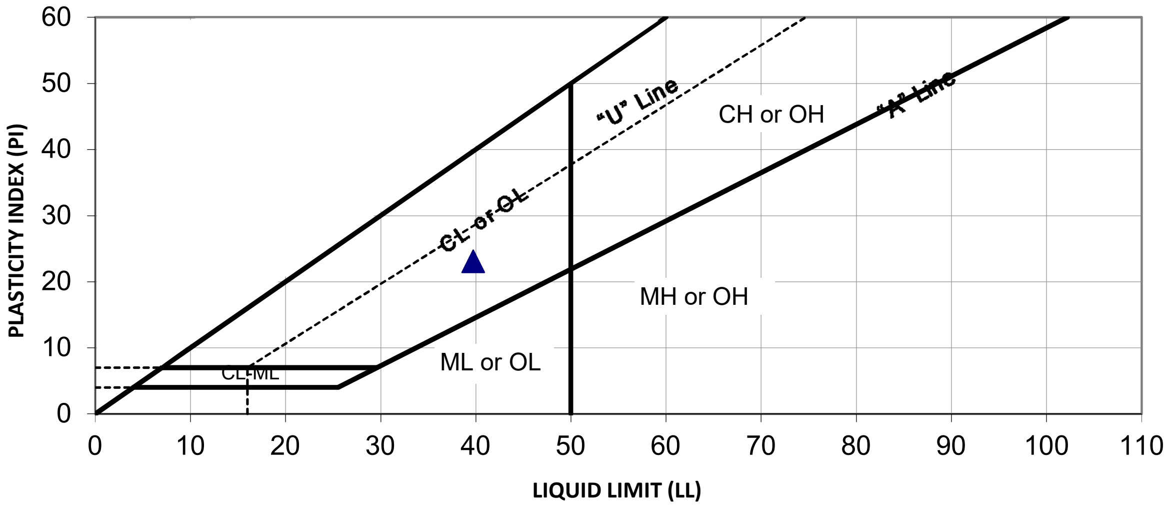
NV5

REPORT OF LIQUID LIMIT, PLASTIC LIMIT & PLASTICITY INDEX TESTS

(ASTM D4318)

Date:	April 13, 2021	Job Number:	1310
Client:	Palomar Community College District	Report Number:	8421
Address:	San Marcos, CA	Lab Number:	121326

Project:	Palomar Community College Athletic Field Improvements
Project Address:	San Marcos, CA
Material:	Brown Sandy Lean CLAY (CL)
Location:	B2 @ 1'-5'
Date Sampled:	3/17-19/2021
Date Submitted:	3/30/2021
Sampled By:	Adam Hespeler
Date Tested:	4/9/2021



SUMMARY OF TEST RESULTS

SAMPLE ID	SOURCE /LOCATION DEPTH	%>#40	TEST RESULT			USCS	
			LL	PL	PI	Class	Group Name
121326	B2 @ 1'-5'	NR	**40**	**17**	**23**	CL	Lean CLAY

NOTE: Classification for material passing #40 sieve only.

Reviewed By: Carl Henderson

Carl Henderson, PhD, PE, GE
CDO / SoCal CQA Group Director

NV5

Expansion Index Test Report
(ASTM D4829)

Date:	4/15/2021		
Client:	Palomar Community College District		
Client Address:	San Marcos, CA	Job Number:	1310
Project Name:	Palomar Community College Athletic Field	Report Number:	8421
Project Address:	San Marcos, CA	Lab Number:	121326
Date Sampled	3/17-18/2021	Sampled By:	Adam Hespeler
Date Submitted:	3/30/2021	Submitted By:	Adam Hespeler

Lab Number	121326			
Material Location	B2 @ 1'-5'			
Material Type	Brown Lean CLAY (CL)			
Material Source	Onsite			
Initial Moisture Content, %	9.3			
Final Moisture Content, %	22.7			
Dry Density, pcf	111.4			
Initial Saturation, %	49.3			
Initial Dial Reading	0.886			
Final Dial Reading	0.939			
Expansion Index	53			
Potential Expansion	**Medium**			

Respectfully Submitted,
NV5 West, Inc.

Carl Henderson

Carl Henderson, PhD, PE, GE
CDO / SoCal CQA Group Director

LABORATORY REPORT

Telephone (619) 425-1993 Fax 425-7917 Established 1928

CLARKSON LABORATORY AND SUPPLY INC.
350 Trousdale Dr. Chula Vista, Ca. 91910 www.clarksonlab.com
ANALYTICAL AND CONSULTING CHEMISTS

Date: April 9, 2021
Purchase Order Number: 21-0628
Sales Order Number: 51320
Account Number: NV5.SD
To:
--
NV5 West Inc
15092 Avenue of Science #200
San Diego, CA 92128
Attention: Michelle Albrecht

Laboratory Number: SO8188 Customers Phone: 858-715-5800
Fax: 858-715-5810
Sample Designation:
--
One soil sample received on 04/05/21 at 2:35pm,
from Palomar College Athletics Project 1310
marked as Lab No 121326, Report No 8421 Location: B2@1'-5'.

Analysis By California Test 643, 1999, Department of Transportation Division of Construction, Method for Estimating the Service Life of Steel Culverts.

pH 8.3

Water Added (ml)	Resistivity (ohm-cm)
20	620
5	570
5	540
5	520
5	510
5	490
5	490
5	500
5	520

23 years to perforation for a 16 gauge metal culvert.
30 years to perforation for a 14 gauge metal culvert.
41 years to perforation for a 12 gauge metal culvert.
52 years to perforation for a 10 gauge metal culvert.
64 years to perforation for a 8 gauge metal culvert.

Water Soluble Sulfate Calif. Test 417	0.017% (170 ppm)
Water Soluble Chloride Calif. Test 422	0.044% (440 ppm)

Rosa Bernal
RMB/dbb

APPENDIX D

Typical Earthwork Guidelines

TYPICAL EARTHWORK GUIDELINES

1. GENERAL

These guidelines and the standard details attached hereto are presented as general procedures for earthwork construction for sites having slopes less than 10 feet high. They are to be utilized in conjunction with the project grading plans. These guidelines are considered a part of the geotechnical report, but are superseded by recommendations in the geotechnical report in the case of conflict. Evaluations performed by the consultant during the course of grading may result in new recommendations which could supersede these specifications and/or the recommendations of the geotechnical report. It is the responsibility of the contractor to read and understand these guidelines as well as the geotechnical report and project grading plans.

1.1. The contractor shall not vary from these guidelines without prior recommendations by the geotechnical consultant and the approval of the client or the client's authorized representative. Recommendations by the geotechnical consultant and/or client shall not be considered to preclude requirements for approval by the jurisdictional agency prior to the execution of any changes.

1.2. The contractor shall perform the grading operations in accordance with these specifications, and shall be responsible for the quality of the finished product notwithstanding the fact that grading work will be observed and tested by the geotechnical consultant.

1.3. It is the responsibility of the grading contractor to notify the geotechnical consultant and the jurisdictional agencies, as needed, prior to the start of work at the site and at any time that grading resumes after interruption. Each step of the grading operations shall be observed and documented by the geotechnical consultant and, where needed, reviewed by the appropriate jurisdictional agency prior to proceeding with subsequent work.

1.4. If, during the grading operations, geotechnical conditions are encountered which were not anticipated or described in the geotechnical report, the geotechnical consultant shall be notified immediately and additional recommendations, if applicable, may be provided.

1.5. An as-graded report shall be prepared by the geotechnical consultant and signed by a registered engineer and registered engineering geologist. The report documents the geotechnical consultants' observations, and field and laboratory test results, and provides conclusions regarding whether or not earthwork construction was performed in accordance with the geotechnical recommendations and the grading plans. Recommendations for foundation design, pavement design, subgrade treatment, etc., may also be included in the as-graded report.

1.6. For the purpose of evaluating quantities of materials excavated during grading and/or locating the limits of excavations, a licensed land surveyor or civil engineer shall be retained.

2. SITE PREPARATION

Site preparation shall be performed in accordance with the recommendations presented in the following sections.

2.1. The client, prior to any site preparation or grading, shall arrange and attend a pre-grading meeting between the grading contractor, the design engineer, the geotechnical consultant, and representatives of appropriate governing authorities, as well as any other involved parties. The parties shall be given two working days notice.

2.2. Clearing and grubbing shall consist of the substantial removal of vegetation, brush, grass, wood, stumps, trees, tree roots greater than 1/2-inch in diameter, and other deleterious materials from the areas to be graded. Clearing and grubbing shall extend to the outside of the proposed excavation and fill areas.

2.3. Demolition in the areas to be graded shall include removal of building structures, foundations, reservoirs, utilities (including underground pipelines, septic tanks, leach fields, seepage pits, cisterns, etc.), and other manmade surface and subsurface improvements, and the backfilling of mining shafts, tunnels and surface depressions. Demolition of utilities shall include capping or rerouting of pipelines at the project perimeter, and abandonment of wells in accordance with the requirements of the governing authorities and the recommendations of the geotechnical consultant at the time of demolition.

2.4. The debris generated during clearing, grubbing and/or demolition operations shall be removed from areas to be graded and disposed of off site at a legal dump site. Clearing, grubbing, and demolition operations shall be performed under the observation of the geotechnical consultant.

2.5. The ground surface beneath proposed fill areas shall be stripped of loose or unsuitable soil. These soils may be used as compacted fill provided they are generally free of organic or other deleterious materials and evaluated for use by the geotechnical consultant. The resulting surface shall be evaluated by the geotechnical consultant prior to proceeding. The cleared, natural ground surface shall be scarified to a depth of approximately 8 inches, moisture conditioned, and compacted in accordance with the specifications presented in Section 4 of these guidelines.

3. REMOVALS AND EXCAVATIONS

Removals and excavations shall be performed as recommended in the following sections.

3.1. Removals

3.1.1. Materials which are considered unsuitable shall be excavated under the observation of the geotechnical consultant in accordance with the recommendations contained herein. Unsuitable materials include, but may not be limited to, dry, loose, soft, wet, organic, compressible natural soils, fractured, weathered, soft bedrock, and undocumented or otherwise deleterious fill materials.

3.1.2. Materials deemed by the geotechnical consultant to be unsatisfactory due to moisture conditions shall be excavated in accordance with the recommendations of the geotechnical consultant, watered or dried as needed, and mixed to generally uniform moisture content in accordance with the specifications presented in Section 4 of this document.

3.2. Excavations

3.2.1. Temporary excavations no deeper than 4 feet in firm fill or natural materials may be made with vertical side slopes. To satisfy California Occupational Safety and Health Administration (Cal OSHA) requirements, any excavation deeper than 4 feet shall be shored or laid back at a 1:1 inclination or flatter, depending on material type, if construction workers are to enter the excavation.

4. COMPACTED FILL

Fill shall be constructed as specified below or by other methods recommended by the geotechnical consultant. Unless otherwise specified, fill soils shall be compacted to 90 percent relative compaction, as evaluated in accordance with ASTM Test Method D 1557.

4.1. Prior to placement of compacted fill, the contractor shall request an evaluation of the exposed ground surface by the geotechnical consultant. Unless otherwise recommended, the exposed ground surface shall then be scarified to a depth of approximately 8 inches and watered or dried, as needed, to achieve a generally uniform moisture content at or near the optimum moisture content. The scarified materials shall then be compacted to 90 percent relative compaction. The evaluation of compaction by the geotechnical consultant shall not be considered to preclude any requirements for observation or approval by governing agencies. It is the contractor's responsibility to notify the geotechnical consultant and the appropriate governing agency when project areas are ready for observation, and to provide reasonable time for that review.

4.2. Excavated on-site materials which are in general compliance with the recommendations of the geotechnical consultant may be utilized as compacted fill provided they are generally free of organic or other deleterious materials and do not contain rock fragments greater than 6 inches in dimension. During grading, the contractor may encounter soil types other than those analyzed during the preliminary geotechnical study. The geotechnical consultant shall be consulted to evaluate the suitability of any such soils for use as compacted fill.

4.3. Where imported materials are to be used on site, the geotechnical consultant shall be notified three working days in advance of importation in order that it may sample and test the materials from the proposed borrow sites. No imported materials shall be delivered for use on site without prior sampling, testing, and evaluation by the geotechnical consultant.

4.4. Soils imported for on-site use shall preferably have very low to low expansion potential (based on UBC Standard 18-2 test procedures). Lots on which expansive soils may be exposed at grade shall be undercut 3 feet or more and capped with very low to low expansion potential fill. In the event expansive soils are present near the ground surface, special design and construction considerations shall be utilized in general accordance with the recommendations of the geotechnical consultant.

4.5. Fill materials shall be moisture conditioned to near optimum moisture content prior to placement. The optimum moisture content will vary with material type and other factors. Moisture conditioning of fill soils shall be generally uniform in the soil mass.

4.6. Prior to placement of additional compacted fill material following a delay in the grading operations, the exposed surface of previously compacted fill shall be prepared to receive fill. Preparation may include scarification, moisture conditioning, and recompaction.

4.7. Compacted fill shall be placed in horizontal lifts of approximately 8 inches in loose thickness. Prior to compaction, each lift shall be watered or dried as needed to achieve near optimum moisture condition, mixed, and then compacted by mechanical methods, using sheepsfoot rollers, multiple-wheel pneumatic-tired rollers, or other appropriate compacting rollers, to the specified relative compaction. Successive lifts shall be treated in a like manner until the desired finished grades are achieved.

4.8. Fill shall be tested in the field by the geotechnical consultant for evaluation of general compliance with the recommended relative compaction and moisture conditions. Field density testing shall conform to ASTM D 1556-00 (Sand Cone method), D 2937-00 (Drive-Cylinder method), and/or D 2922-96 and D 3017-96 (Nuclear Gauge method). Generally, one test shall be provided for approximately every 2 vertical feet of fin placed, or for approximately every 1000 cubic yards of fill placed. In addition, on slope faces one or more tests shall be taken for approximately every 10,000 square feet of slope face and/or approximately every 10 vertical feet of slope height. Actual test intervals may vary as field conditions dictate. Fill found to be out of conformance with the grading recommendations shall be removed, moisture conditioned, and compacted or otherwise handled to accomplish general compliance with the grading recommendations.

4.9. The contractor shall assist the geotechnical consultant by excavating suitable borings for removal evaluation and/or for testing of compacted fill.

4.10. At the request of the geotechnical consultant, the contractor shall "shut down" or restrict grading equipment from operating in the area being tested to provide adequate testing time and safety for the field technician.

4.11. The geotechnical consultant shall maintain a map with the approximate locations of field density tests. Unless the client provides for surveying of the test locations, the locations shown by the geotechnical consultant will be estimated. The geotechnical consultant shall not be held responsible for the accuracy of the horizontal or vertical locations or elevations.

4.12. Grading operations shall be performed under the observation of the geotechnical consultant. Testing and evaluation by the geotechnical consultant does not preclude the need for approval by or other requirements of the jurisdictional agencies.

4.13. Fill materials shall not be placed, spread or compacted during unfavorable weather conditions. When work is interrupted by heavy rains, the filling operation shall not be resumed until tests indicate that moisture content and density of the fill meet the project specifications. Regrading of the near-surface soil may be needed to achieve the specified moisture content and density.

4.14. Upon completion of grading and termination of observation by the geotechnical consultant, no further filling or excavating, including that planned for footings, foundations, retaining walls or other features, shall be performed without the involvement of the geotechnical consultant.

4.15. Fill placed in areas not previously viewed and evaluated by the geotechnical consultant may have to be removed and recompacted at the contractor's expense. The depth and extent of removal of the unobserved and undocumented fill will be decided based upon review of the field conditions by the geotechnical consultant.

4.16. Off-site fill shall be treated in the same manner as recommended in these specifications for on-site fills. Off-site fill subdrains temporarily terminated (up gradient) shall be surveyed for future locating and connection.

5. OVERSIZED MATERIAL

Oversized material shall be placed in accordance with the following recommendations.

5.1. During the course of grading operations, rocks or similar irreducible materials greater than 6 inches in dimension (oversized material) may be generated. These materials shall not be placed within the compacted fill unless placed in general accordance with the recommendations of the geotechnical consultant.

5.2. Where oversized rock (greater than 6 inches in dimension) or similar irreducible material is generated during grading, it is recommended, where practical, to waste such material off site, or on site in areas designated as "nonstructural rock disposal areas." Rock designated for disposal areas shall be placed with sufficient sandy soil to generally fill voids. The disposal area shall be capped with a 5-foot thickness of fill which is generally free of oversized material.

5.3. Rocks 6 inches in dimension and smaller may be utilized within the compacted fill, provided they are placed in such a manner that nesting of rock is not permitted. Fill shall be placed and compacted over and around the rock. The amount of rock greater than ¾-inch in dimension shall generally not exceed 40 percent of the total dry weight of the fill mass, unless the fill is specially designed and constructed as a "rock fill."

5.4. Rocks or similar irreducible materials greater than 6 inches but less than 4 feet in dimension generated during grading may be placed in windrows and capped with finer materials in accordance with the recommendations of the geotechnical consultant and the approval of the governing agencies. Selected native or imported granular soil (Sand Equivalent of 30 or higher) shall be placed and flooded over and around the windrowed rock such that voids are filled. Windrows of oversized materials shall be staggered so that successive windrows of oversized materials are not in the same vertical plane. Rocks greater than 4 feet in dimension shall be broken down to 4 feet or smaller before placement, or they shall be disposed of off site.

6. SLOPES

The following sections provide recommendations for cut and fill slopes.

6.1. Cut Slopes

6.1.1. The geotechnical consultant shall observe cut slopes during excavation. The geotechnical consultant shall be notified by the contractor prior to beginning slope excavations.

6.1.2. If, during the course of grading, adverse or potentially adverse geotechnical conditions are encountered in the slope which were not anticipated in the preliminary evaluation report, the geotechnical consultant shall evaluate the conditions and provide appropriate recommendations.

6.2. Fill Slopes

6.2.1. When placing fill on slopes steeper than 5:1 (horizontal:vertical), topsoil, slope wash, colluvium, and other materials deemed unsuitable shall be removed. Near-horizontal keys and near-vertical benches shall be excavated into sound bedrock or fine fill material, in accordance with the recommendation of the geotechnical consultant. Keying and benching shall be accomplished. Compacted fill shall not be placed in an area subsequent to keying and benching until the area has been observed by the geotechnical consultant. Where the natural gradient of a slope is less than 5:1, benching is generally not recommended. However, fill shall not be placed on compressible or otherwise unsuitable materials left on the slope face.

6.2.2. Within a single fill area where grading procedures dictate two or more separate fills, temporary slopes (false slopes) may be created. When placing fill adjacent to a temporary slope, benching shall be conducted in the manner described in Section 6.2.1. A 3-foot or higher near-vertical bench shall be excavated into the documented fill prior to placement of additional fill.

6.2.3. Unless otherwise recommended by the geotechnical consultant and accepted by the Building Official, permanent fill slopes shall not be steeper than 2:1 (horizontal:vertical). The height of a fill slope shall be evaluated by the geotechnical consultant.

6.2.4. Unless specifically recommended otherwise, compacted fill slopes shall be overbuilt and cut back to grade, exposing firm compacted fill. The actual amount of overbuilding may vary as field conditions dictate. If the desired results are not achieved, the existing slopes shall be overexcavated and reconstructed in accordance with the recommendations of the geotechnical consultant. The degree of overbuilding may be increased until the desired compacted slope face condition is achieved. Care shall be taken by the contractor to provide mechanical compaction as close to the outer edge of the overbuilt slope surface as practical.

6.2.5. If access restrictions, property line location, or other constraints limit overbuilding and cutting back of the slope face, an alternative method for compaction of the slope face may be attempted by conventional construction procedures including backrolling at intervals of 4 feet or less in vertical slope height, or as dictated by the capability of the available equipment, whichever is less. Fill slopes shall be backrolled utilizing a conventional sheepsfoot-type roller. Care shall be taken to maintain the specified moisture conditions and/or reestablish the same, as needed, prior to backrolling.

6.2.6. The placement, moisture conditioning and compaction of fill slope materials shall be done in accordance with the recommendations presented in Section 5 of these guidelines.

6.2.7. The contractor shall be ultimately responsible for placing and compacting the soil out to the slope face to obtain a relative compaction of 90 percent as evaluated by ASTM D 1557 and a moisture content in accordance with Section 4. The geotechnical consultant shall perform field moisture and density tests at intervals of one test for approximately every 10,000 square feet of slope.

6.2.8. Backdrains shall be provided in fill as recommended by the geotechnical consultant.

6.3. Top-of-Slope Drainage

6.3.1. For pad areas above slopes, positive drainage shall be established away from the top of slope. This may be accomplished utilizing a berm and pad gradient of 2 percent or steeper at the top-of-slope areas. Site runoff shall not be permitted to flow over the tops of slopes.

6.3.2. Gunite-lined brow ditches shall be placed at the top of cut slopes to redirect surface runoff away from the slope face where drainage devices are not otherwise provided.

6.4. Slope Maintenance

6.4.1. In order to enhance surficial slope stability, slope planting shall be accomplished at the completion of grading. Slope plants shall consist of deep-rooting, variable root depth, drought-tolerant vegetation. Native vegetation is generally desirable. Plants native to semiarid and mid areas may also be appropriate. Large-leafed ice plant should not be used on slopes. A landscape architect shall be consulted regarding the actual types of plants and planting configuration to be used.

6.4.2. Irrigation pipes shall be anchored to slope faces and not placed in trenches excavated into slope faces. Slope irrigation shall be maintained at a level just sufficient to support plant growth. Property owners shall be made aware that over watering of slopes is detrimental to slope stability. Slopes shall be monitored regularly and broken sprinkler heads and/or pipes shall be repaired immediately.

6.4.3. Periodic observation of landscaped slope areas shall be planned and appropriate measures taken to enhance growth of landscape plants.

6.4.4. Graded swales at the top of slopes and terrace drains shall be installed and the property owners notified that the drains shall be periodically checked so that they may be kept clear. Damage to drainage improvements shall be repaired immediately. To reduce siltation, terrace drains shall be constructed at a gradient of 3 percent or steeper, in accordance with the recommendations of the project civil engineer.

6.4.5. If slope failures occur, the geotechnical consultant shall be contacted immediately for field review of site conditions and development of recommendations for evaluation and repair.

7. TRENCH BACKFILL

The following sections provide recommendations for backfilling of trenches.

7.1. Trench backfill shall consist of granular soils (bedding) extending from the trench bottom to 1 foot or more above the pipe. On-site or imported fill which has been evaluated by the geotechnical consultant may be used above the granular backfill. The cover soils directly in contact with the pipe shall be classified as having a very low expansion potential, in accordance with UBC Standard 18-2, and shall contain no rocks or chunks of hard soil larger than 3/4-inch in diameter.

7.2. Trench backfill shall, unless otherwise recommended, be compacted by mechanical means to 90 percent relative compaction as evaluated by ASTM D 1557. Backfill soils shall be placed in loose lifts 8-inches thick or thinner, moisture conditioned, and compacted in accordance with the recommendations of Section 4 of these guidelines. The backfill shall be tested by the geotechnical consultant at vertical intervals of approximately 2 feet of backfill placed and at spacings along the trench of approximately 100 feet in the same lift.

7.3. Jetting of trench backfill materials is generally not a recommended method of densification, unless the on-site soils are sufficiently free-draining and provisions have been made for adequate dissipation of the water utilized in the jetting process.

7.4. If it is decided that jetting may be utilized, granular material with a sand equivalent greater than 30 shall be used for backfilling in the areas to be jetted. Jetting shall generally be considered for trenches 2 feet or narrower in width and 4 feet or shallower in depth. Following jetting operations, trench backfill shall be mechanically compacted to the specified compaction to finish grade.

7.5. Trench backfill which underlies the zone of influence of foundations shall be mechanically compacted to 90 percent or greater relative compaction, as evaluated by ASTM D 1557-02. The zone of influence of the foundations is generally defined as the roughly triangular area within the limits of a 1:1 (horizontal:vertical) projection from the inner and outer edges of the foundation, projected down and out from both edges.

7.6. Trench backfill within slab areas shall be compacted by mechanical means to a relative compaction of 90 percent, as evaluated by ASTM D 1557. For minor interior trenches, density testing may be omitted or spot testing may be performed, as deemed appropriate by the geotechnical consultant.

7.7. When compacting soil in close proximity to utilities, care shall be taken by the grading contractor so that mechanical methods used to compact the soils do not damage the utilities. If the utility contractors indicate that it is undesirable to use compaction equipment in close proximity to a buried conduit, then the grading contractor may elect to use light mechanical compaction equipment or, with the approval of the geotechnical consultant, cover the conduit with clean granular material. These granular materials shall be jetted in place to the top of the conduit in accordance with the recommendations of Section 7.4 prior to initiating mechanical compaction procedures. Other methods of utility trench compaction may also be appropriate, upon review by the geotechnical consultant and the utility contractor, at the time of construction.

7.8. Clean granular backfill and/or bedding materials are not recommended for use in slope areas unless provisions are made for a drainage system to mitigate the potential for buildup of seepage forces or piping of backfill materials.

7.9. The contractor shall exercise the specified safety precautions, in accordance with OSHA Trench Safety Regulations, while conducting trenching operations. Such precautions include shoring or laying back trench excavations at 1:1 or flatter, depending on material type, for trenches in excess of 5 feet in depth. The geotechnical consultant is not responsible for the safety of trench operations or stability of the trenches.

8. DRAINAGE

The following sections provide recommendations pertaining to site drainage.

8.1. Roof, pad, and slope drainage shall be such that it is away from slopes and structures to suitable discharge areas by nonerodible devices (e.g., gutters, downspouts, concrete swales, etc.).

8.2. Positive drainage adjacent to structures shall be established and maintained. Positive drainage may be accomplished by providing drainage away from the foundations of the structure at a gradient of 2 percent or steeper for a distance of 5 feet or more outside the building perimeter, further maintained by a graded swale leading to an appropriate outlet, in accordance with the recommendations of the project civil engineer and/or landscape architect.

8.3. Surface drainage on the site shall be provided so that water is not permitted to pond. A gradient of 2 percent or steeper shall be maintained over the pad area and drainage patterns shall be established to remove water from the site to an appropriate outlet.

8.4. Care shall be taken by the contractor during grading to preserve any berms, drainage terraces, interceptor swales or other drainage devices of a permanent nature on or adjacent to the property. Drainage patterns established at the time of finish grading shall be maintained for the life of the project. Property owners shall be made very clearly aware that altering drainage patterns may be detrimental to slope stability and foundation performance.

9. SITE PROTECTION

The site shall be protected as outlined in the following sections.

9.1. Protection of the site during the period of grading shall be the responsibility of the contractor unless other provisions are made in writing and agreed upon among the concerned parties. Completion of a portion of the project shall not be considered to preclude that portion or adjacent areas from the need for site protection, until such time as the project is finished as agreed upon by the geotechnical consultant, the client, and the regulatory agency.

9.2. The contractor is responsible for the stability of temporary excavations. Recommendations by the geotechnical consultant pertaining to temporary excavations are made in consideration of stability of the finished project and, therefore, shall not be considered to preclude the responsibilities of the contractor. Recommendations by the geotechnical consultant shall also not be considered to preclude more restrictive requirements by the applicable regulatory agencies.

9.3. Precautions shall be taken during the performance of site clearing, excavation, and grading to protect the site from flooding, ponding, or inundation by surface runoff. Temporary provisions shall be made during the rainy season so that surface runoff is away from and off the working site. Where low areas cannot be avoided, pumps shall be provided to remove water as needed during periods of rainfall.

9.4. During periods of rainfall, plastic sheeting shall be used as needed to reduce the potential for unprotected slopes to become saturated. Where needed, the contractor shall install check dams, desilting basins, riprap, sandbags or other appropriate devices or methods to reduce erosion and provide recommended conditions during inclement weather.

9.5. During periods of rainfall, the geotechnical consultant shall be kept informed by the contractor of the nature of remedial or precautionary work being performed on site (e.g., pumping, placement of sandbags or plastic sheeting, other labor, dozing, etc.).

9.6. Following periods of rainfall, the contractor shall contact the geotechnical consultant and arrange a walk-over of the site in order to visually assess rain-related damage. The geotechnical consultant may also recommend excavation and testing in order to aid in the evaluation. At the request of the geotechnical consultant, the contractor shall make excavations in order to aid in evaluation of the extent of rain-related damage.

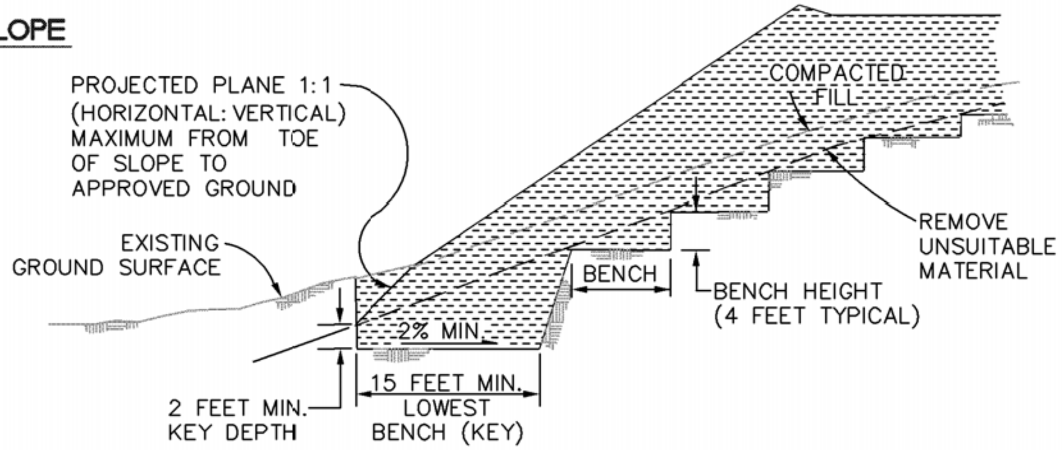
9.7. Rain or irrigation related damage shall be considered to include, but may not be limited to, erosion, silting, saturation, swelling, structural distress, and other adverse conditions noted by the geotechnical consultant. Soil adversely affected shall be classified as "Unsuitable Material" and shall be subject to overexcavation and replacement with compacted fill or to other remedial grading as recommended by the geotechnical consultant.

9.8. Relatively level areas where saturated soils and/or erosion gullies exist to depths greater than 1 foot shall be overexcavated to competent materials as evaluated by the geotechnical consultant. Where adverse conditions extend to less than 1 foot in depth, saturated and/or eroded materials may be processed in-place. Overexcavated or in-place processed materials shall be moisture conditioned and compacted in accordance with the recommendations provided in Section 4. If the desired results are not achieved, the affected materials shall be overexcavated, moisture conditioned, and compacted until the specifications are met.

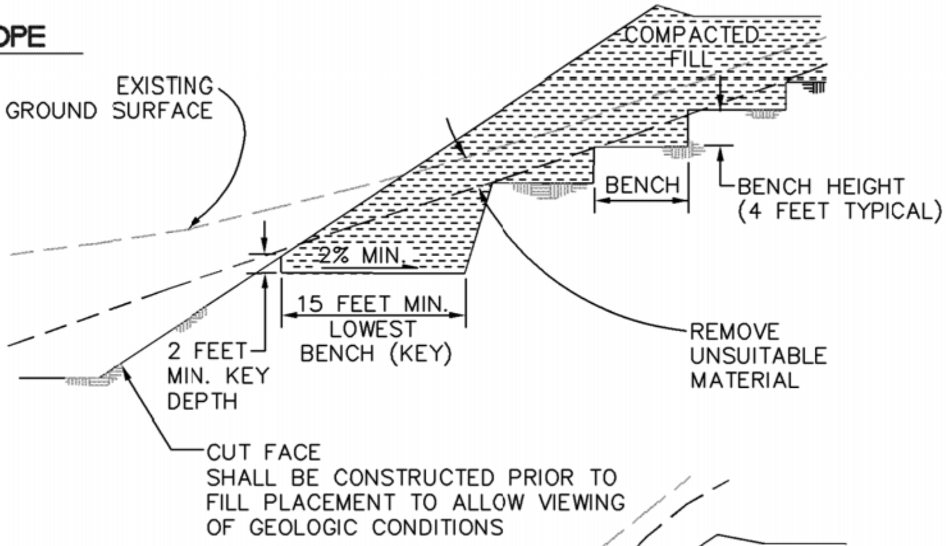
9.9. Slope areas where saturated soil and/or erosion gullies exist to depths greater than 1 foot shall be overexcavated and replaced as compacted fill in accordance with the applicable specifications. Where adversely affected materials exist to depths of l foot or less below proposed finished grade, remedial grading by moisture conditioning in-place and compaction in accordance with the appropriate specifications may be attempted. If the desired results are not achieved, the affected materials shall be overexcavated, moisture conditioned, and compacted until the specifications are met. As conditions dictate, other slope repair procedures may also be recommended by the geotechnical consultant.

9.10. During construction, the contractor shall grade the site to provide positive drainage away from structures and to keep water from ponding adjacent to structures. Water shall not be allowed to damage adjacent properties. Positive drainage shall be maintained by the contractor until permanent drainage and erosion reducing devices are installed in accordance with project plans.

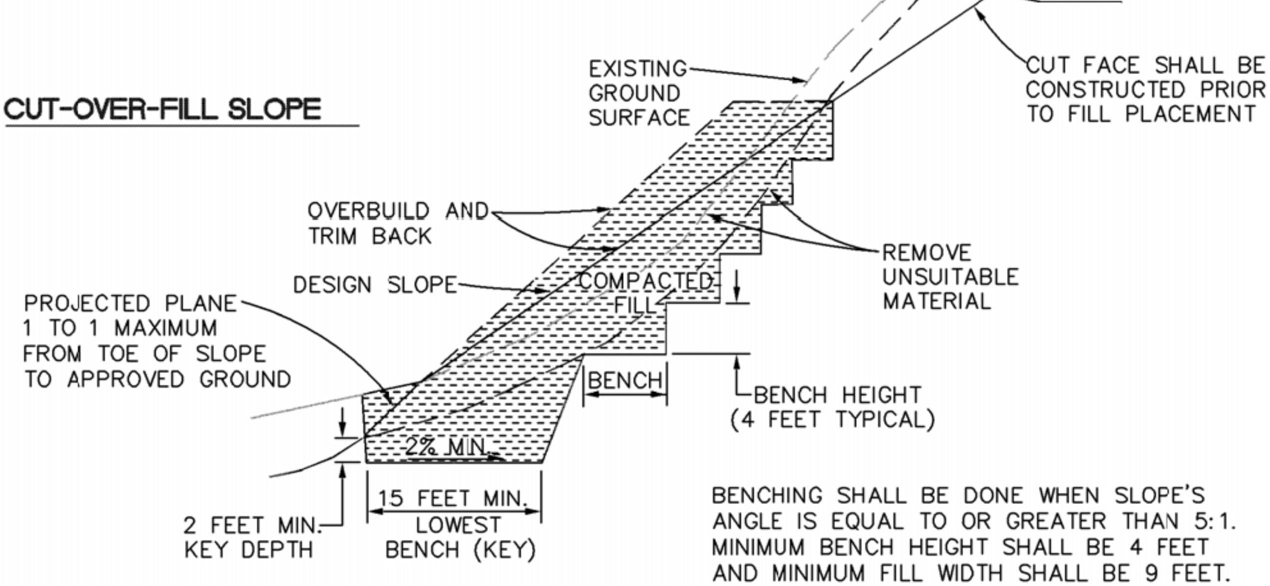
FILL SLOPE



FILL-OVER-CUT SLOPE



CUT-OVER-FILL SLOPE

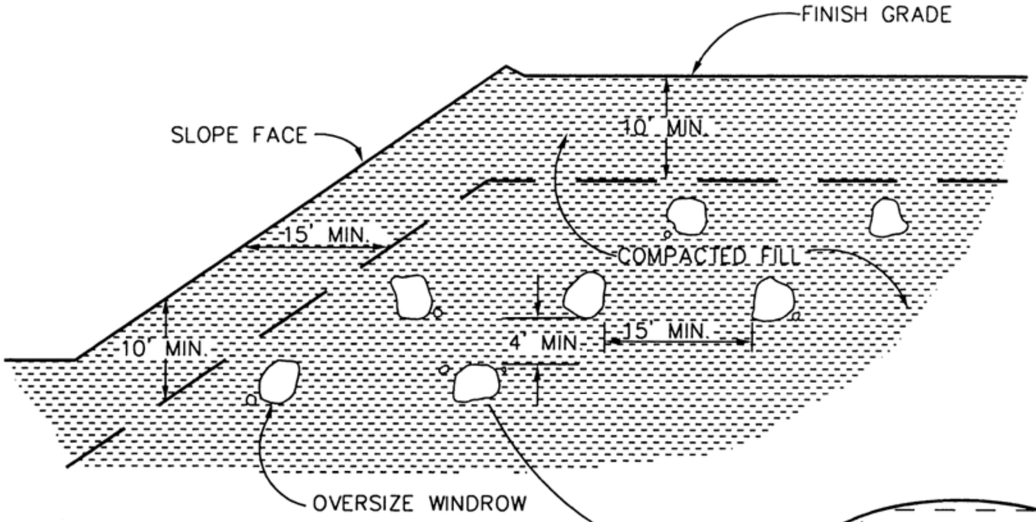


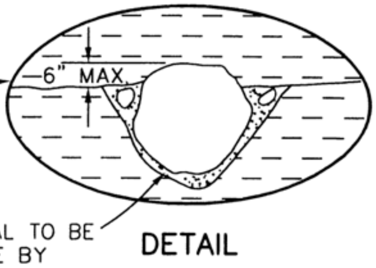
BENCHING SHALL BE DONE WHEN SLOPE'S ANGLE IS EQUAL TO OR GREATER THAN 5:1. MINIMUM BENCH HEIGHT SHALL BE 4 FEET AND MINIMUM FILL WIDTH SHALL BE 9 FEET.

KEYING AND BENCHING

GENERAL EARTHWORK AND
GRADING SPECIFICATIONS
STANDARD DETAIL A

NV5

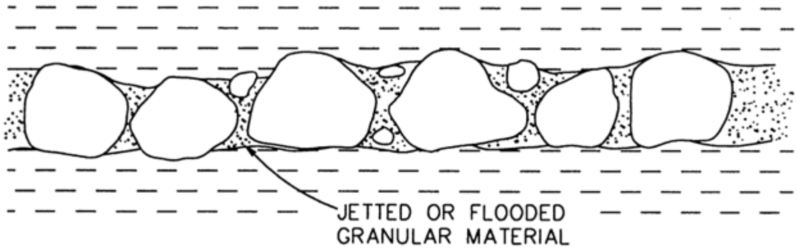




GRANULAR MATERIAL TO BE DENSIFIED IN PLACE BY FLOODING OR JETTING.

DETAIL

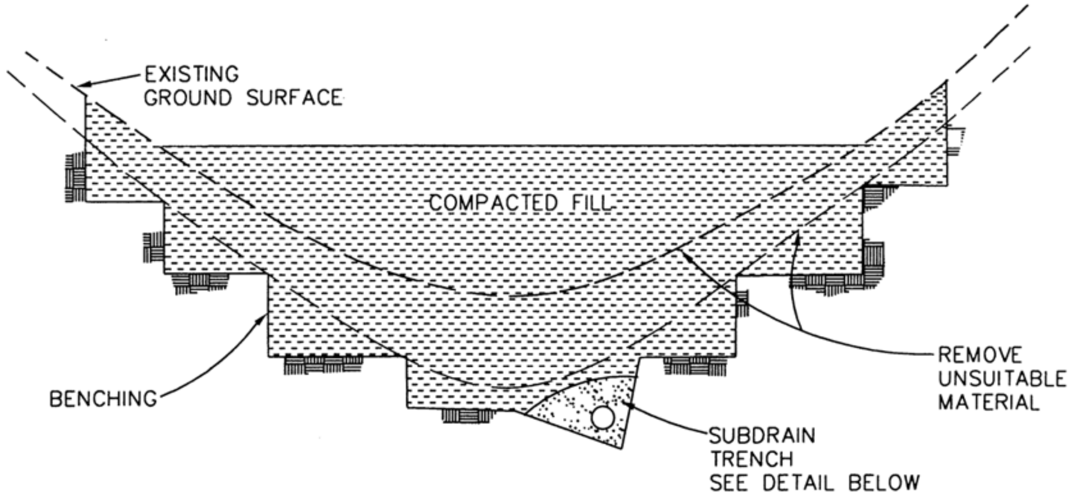
- OVERSIZE ROCK IS LARGER THAN 8 INCHES IN LARGEST DIMENSION.
- EXCAVATE A TRENCH IN THE COMPACTED FILL DEEP ENOUGH TO BURY ALL THE ROCK.
- BACKFILL WITH GRANULAR SOIL JETTED OR FLOODED IN PLACE TO FILL ALL THE VOIDS.
- DO NOT BURY ROCK WITHIN 10 FEET OF FINISH GRADE.
- WINDROW OF BURIED ROCK SHALL BE PARALLEL TO THE FINISHED SLOPE.

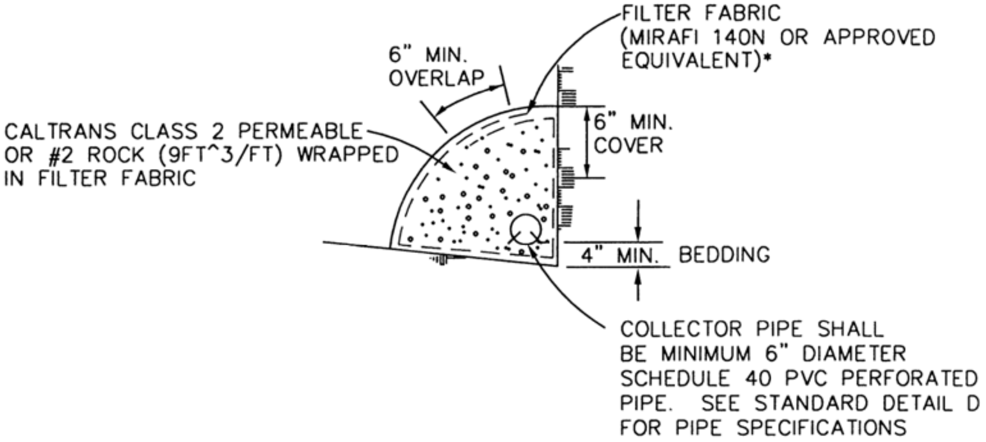


TYPICAL PROFILE ALONG WINDROW

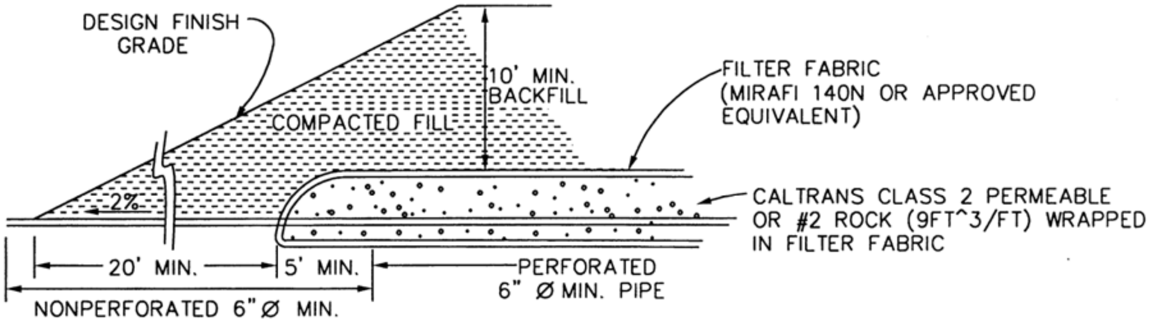
OVERSIZE ROCK DISPOSAL

GENERAL EARTHWORK AND
GRADING SPECIFICATIONS
STANDARD DETAIL B





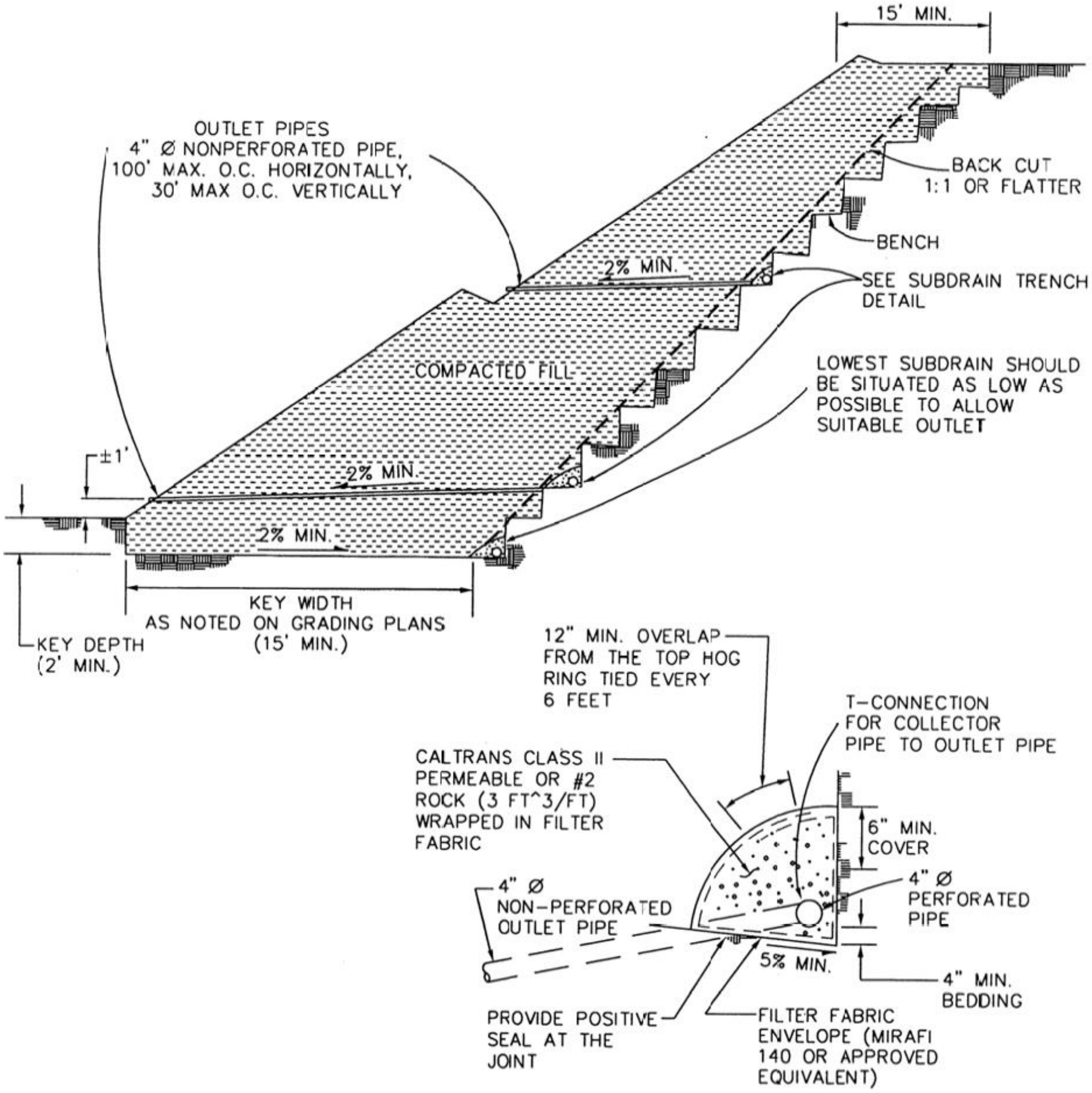
SUBDRAIN DETAIL



DETAIL OF CANYON SUBDRAIN OUTLET

CANYON SUBDRAINS

GENERAL EARTHWORK AND
GRADING SPECIFICATIONS
STANDARD DETAIL C



SUBDRAIN TRENCH DETAIL

SUBDRAIN INSTALLATION – subdrain collector pipe shall be installed with perforation down or, unless otherwise designated by the geotechnical consultant. Outlet pipes shall be non-perforated pipe. The subdrain pipe shall have at least 8 perforations uniformly spaced per foot. Perforation shall be 1/4" to 1/2" if drill holes are used. All subdrain pipes shall have a gradient of at least 2% towards the outlet.

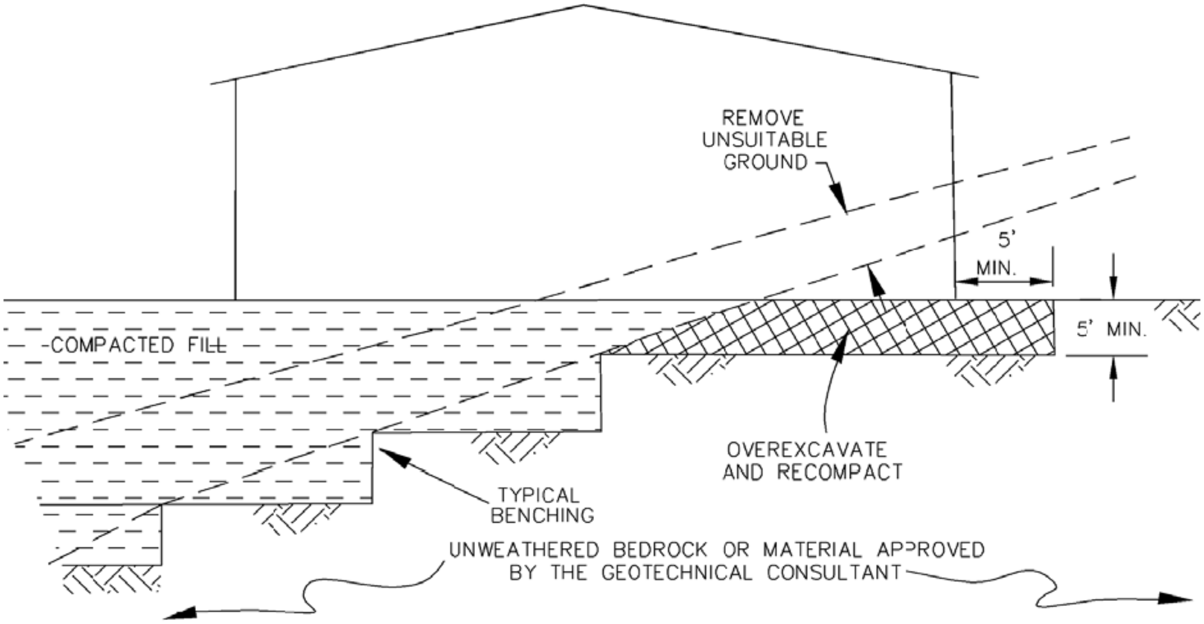
SUBDRAIN PIPE – Subdrain pipe shall be ASTM D2751, SDR 23.5 or ASTM D1527, Schedule 40, or ASTM D3034, SDR 23.5, Schedule 40 Polyvinyl Chloride Plastic (PVC) pipe.

All outlet pipe shall be placed in a trench no wider than twice the subdrain pipe.

BUTTRESS OR REPLACEMENT FILL SUBDRAINS

GENERAL EARTHWORK AND
GRADING SPECIFICATIONS
STANDARD DETAIL D

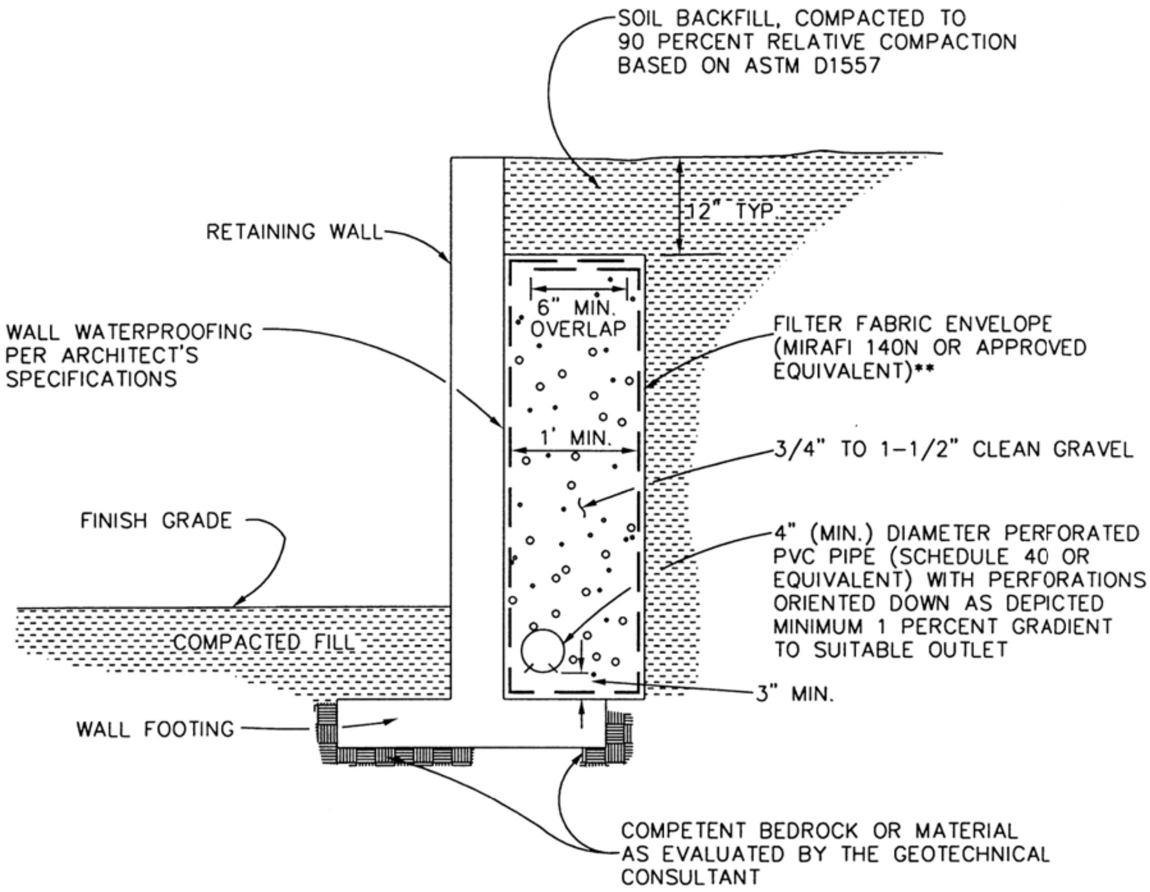
CUT–FILL TRANSITION LOT OVEREXCAVATION



TRANSITION LOT FILLS

GENERAL EARTHWORK AND
GRADING SPECIFICATIONS
STANDARD DETAIL E

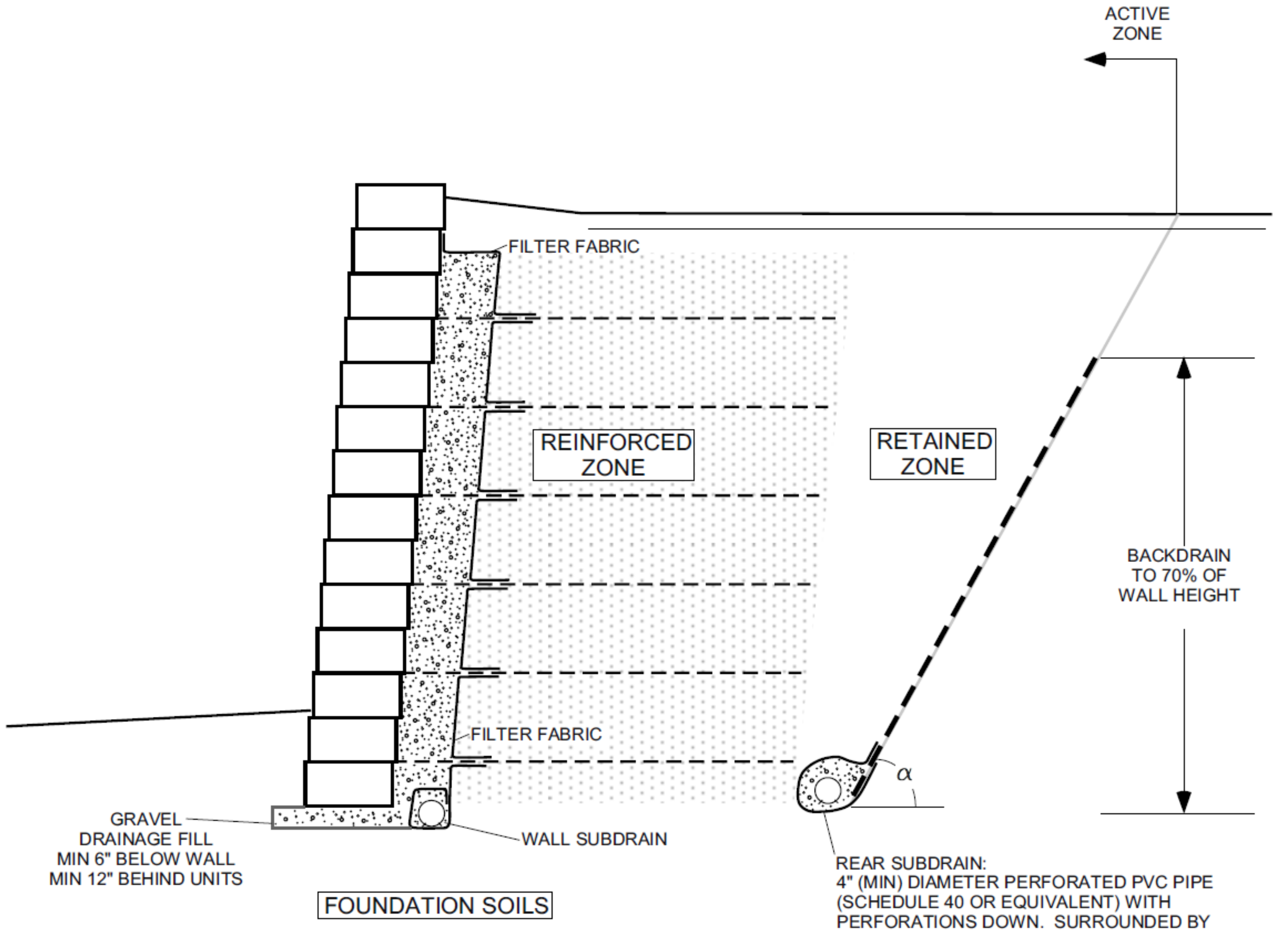
NV5



NOTE: UPON REVIEW BY THE GEOTECHNICAL CONSULTANT, COMPOSITE DRAINAGE PRODUCTS SUCH AS MIRADRAIN OR J-DRAIN MAY BE USED AS AN ALTERNATIVE TO GRAVEL OR CLASS 2 PERMEABLE MATERIAL. INSTALLATION SHOULD BE PERFORMED IN ACCORDANCE WITH MANUFACTURER'S SPECIFICATIONS.

RETAINING WALL DRAINAGE

GENERAL EARTHWORK AND
GRADING SPECIFICATIONS
STANDARD DETAIL F



REAR SUBDRAIN:
4" (MIN) DIAMETER PERFORATED PVC PIPE
(SCHEDULE 40 OR EQUIVALENT) WITH
PERFORATIONS DOWN. SURROUNDED BY
1 CU. FT/FT OF 3/4" GRAVEL WRAPPED IN
FILTER FABRIC (MIRAFI 140N OR EQUIVALENT)

OUTLET SUBDRAINS EVERY 100 FEET, OR CLOSER,
BY TIGHTLINE TO SUITABLE PROTECTED OUTLET

NOTES:

1) MATERIAL GRADATION AND PLASTICITY

<u>REINFORCED ZONE:</u>

SIEVE SIZE	% PASSING
1 INCH	100
NO. 4	20-100
NO. 40	0-60
NO. 200	0-35

FOR WALL HEIGHT < 10 FEET, PLASTICITY INDEX < 20
FOR WALL HEIGHT 10 TO 20 FEET, PLASTICITY INDEX < 10
FOR TIERED WALLS, USE COMBINED WALL HEIGHTS
WALL DESIGNER TO REQUEST SITE-SPECIFIC CRITERIA FOR WALL HEIGHT > 20 FEET

GRAVEL DRAINAGE FILL:

SIEVE SIZE	% PASSING
1 INCH	100
3/4 INCH	75-100
NO. 4	0-60
NO. 40	0-50
NO. 200	0-5

2) CONTRACTOR TO USE SOILS WITHIN THE RETAINED AND REINFORCED ZONES THAT MEET THE STRENGTH REQUIREMENTS OF WALL DESIGN.

3) GEOGRID REINFORCEMENT TO BE DESIGNED BY WALL DESIGNER CONSIDERING INTERNAL, EXTERNAL, AND COMPOUND STABILITY.

3) GEOGRID TO BE PRETENSIONED DURING INSTALLATION.

4) IMPROVEMENTS WITHIN THE ACTIVE ZONE ARE SUSCEPTIBLE TO POST-CONSTRUCTION SETTLEMENT. ANGLE $\alpha = 45 + \Phi/2$, WHERE Φ IS THE FRICTION ANGLE OF THE MATERIAL IN THE RETAINED ZONE.

5) BACKDRAIN SHOULD CONSIST OF J-DRAIN 302 (OR EQUIVALENT) OR 6-INCH THICK DRAINAGE FILL WRAPPED IN FILTER FABRIC. PERCENT COVERAGE OF BACKDRAIN TO BE PER GEOTECHNICAL REVIEW.

SEGMENTAL RETAINING WALLS

GENERAL EARTHWORK AND
GRADING SPECIFICATIONS
STANDARD DETAIL G

APPENDIX E

GBA - Important Information About This Geotechnical Engineering Report

Important Information about This

Geotechnical-Engineering Report

Subsurface problems are a principal cause of construction delays, cost overruns, claims, and disputes.

While you cannot eliminate all such risks, you can manage them. The following information is provided to help.

The Geoprofessional Business Association (GBA) has prepared this advisory to help you – assumedly a client representative – interpret and apply this geotechnical-engineering report as effectively as possible. In that way, clients can benefit from a lowered exposure to the subsurface problems that, for decades, have been a principal cause of construction delays, cost overruns, claims, and disputes. If you have questions or want more information about any of the issues discussed below, contact your GBA-member geotechnical engineer. Active involvement in the Geoprofessional Business Association exposes geotechnical engineers to a wide array of risk-confrontation techniques that can be of genuine benefit for everyone involved with a construction project.

Geotechnical-Engineering Services Are Performed for Specific Purposes, Persons, and Projects

Geotechnical engineers structure their services to meet the specific needs of their clients. A geotechnical-engineering study conducted for a given civil engineer will not likely meet the needs of a civil-works constructor or even a different civil engineer. Because each geotechnical-engineering study is unique, each geotechnical-engineering report is unique, prepared *solely* for the client. *Those who rely on a geotechnical-engineering report prepared for a different client can be seriously misled.* No one except authorized client representatives should rely on this geotechnical-engineering report without first conferring with the geotechnical engineer who prepared it. *And no one – not even you – should apply this report for any purpose or project except the one originally contemplated.*

Read this Report in Full

Costly problems have occurred because those relying on a geotechnical-engineering report did not read it *in its entirety*. Do not rely on an executive summary. Do not read selected elements only. *Read this report in full.*

You Need to Inform Your Geotechnical Engineer about Change

Your geotechnical engineer considered unique, project-specific factors when designing the study behind this report and developing the confirmation-dependent recommendations the report conveys. A few typical factors include:

- the client's goals, objectives, budget, schedule, and risk-management preferences;
- the general nature of the structure involved, its size, configuration, and performance criteria;
- the structure's location and orientation on the site; and
- other planned or existing site improvements, such as retaining walls, access roads, parking lots, and underground utilities.

Typical changes that could erode the reliability of this report include those that affect:

- the site's size or shape;
- the function of the proposed structure, as when it's changed from a parking garage to an office building, or from a light-industrial plant to a refrigerated warehouse;
- the elevation, configuration, location, orientation, or weight of the proposed structure;
- the composition of the design team; or
- project ownership.

As a general rule, *always* inform your geotechnical engineer of project changes – even minor ones – and request an assessment of their impact. *The geotechnical engineer who prepared this report cannot accept responsibility or liability for problems that arise because the geotechnical engineer was not informed about developments the engineer otherwise would have considered.*

This Report May Not Be Reliable

Do not rely on this report if your geotechnical engineer prepared it:

- for a different client;
- for a different project;
- for a different site (that may or may not include all or a portion of the original site); or
- before important events occurred at the site or adjacent to it; e.g., man-made events like construction or environmental remediation, or natural events like floods, droughts, earthquakes, or groundwater fluctuations.

Note, too, that it could be unwise to rely on a geotechnical-engineering report whose reliability may have been affected by the passage of time, because of factors like changed subsurface conditions; new or modified codes, standards, or regulations; or new techniques or tools. *If your geotechnical engineer has not indicated an "apply-by" date on the report, ask what it should be*, and, in general, *if you are the least bit uncertain* about the continued reliability of this report, contact your geotechnical engineer before applying it. A minor amount of additional testing or analysis – if any is required at all – could prevent major problems.

Most of the "Findings" Related in This Report Are Professional Opinions

Before construction begins, geotechnical engineers explore a site's subsurface through various sampling and testing procedures. *Geotechnical engineers can observe actual subsurface conditions only at those specific locations where sampling and testing were performed.* The data derived from that sampling and testing were reviewed by your geotechnical engineer, who then applied professional judgment to form opinions about subsurface conditions throughout the site. Actual sitewide-subsurface conditions may differ – maybe significantly – from those indicated in this report. Confront that risk by retaining your geotechnical engineer to serve on the design team from project start to project finish, so the individual can provide informed guidance quickly, whenever needed.

This Report's Recommendations Are Confirmation-Dependent

The recommendations included in this report – including any options or alternatives – are confirmation-dependent. In other words, *they are not final*, because the geotechnical engineer who developed them relied heavily on judgment and opinion to do so. Your geotechnical engineer can finalize the recommendations *only after observing actual subsurface conditions* revealed during construction. If through observation your geotechnical engineer confirms that the conditions assumed to exist actually do exist, the recommendations can be relied upon, assuming no other changes have occurred. *The geotechnical engineer who prepared this report cannot assume responsibility or liability for confirmation-dependent recommendations if you fail to retain that engineer to perform construction observation.*

This Report Could Be Misinterpreted

Other design professionals' misinterpretation of geotechnical-engineering reports has resulted in costly problems. Confront that risk by having your geotechnical engineer serve as a full-time member of the design team, to:

- confer with other design-team members,
- help develop specifications,
- review pertinent elements of other design professionals' plans and specifications, and
- be on hand quickly whenever geotechnical-engineering guidance is needed.

You should also confront the risk of constructors misinterpreting this report. Do so by retaining your geotechnical engineer to participate in prebid and preconstruction conferences and to perform construction observation.

Give Constructors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can shift unanticipated-subsurface-conditions liability to constructors by limiting the information they provide for bid preparation. To help prevent the costly, contentious problems this practice has caused, include the complete geotechnical-engineering report, along with any attachments or appendices, with your contract documents, *but be certain to note conspicuously that you've included the material for informational purposes only*. To avoid misunderstanding, you may also want to note that "informational purposes" means constructors have no right to rely on the interpretations, opinions, conclusions, or recommendations in the report, but they may rely on the factual data relative to the specific times, locations, and depths/elevations referenced. Be certain that constructors know they may learn about specific project requirements, including options selected from the report, *only* from the design drawings and specifications. Remind constructors that they may perform their own studies if they want to, and *be sure to allow enough time* to permit them to do so. Only then might you be in a position to give constructors the information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions. Conducting prebid and preconstruction conferences can also be valuable in this respect.

Read Responsibility Provisions Closely

Some client representatives, design professionals, and constructors do not realize that geotechnical engineering is far less exact than other engineering disciplines. That lack of understanding has nurtured unrealistic expectations that have resulted in disappointments, delays, cost overruns, claims, and disputes. To confront that risk, geotechnical engineers commonly include explanatory provisions in their reports. Sometimes labeled "limitations," many of these provisions indicate where geotechnical engineers' responsibilities begin and end, to help others recognize their own responsibilities and risks. *Read these provisions closely*. Ask questions. Your geotechnical engineer should respond fully and frankly.

Geoenvironmental Concerns Are Not Covered

The personnel, equipment, and techniques used to perform an environmental study – e.g., a "phase-one" or "phase-two" environmental site assessment – differ significantly from those used to perform a geotechnical-engineering study. For that reason, a geotechnical-engineering report does not usually relate any environmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated subsurface environmental problems have led to project failures*. If you have not yet obtained your own environmental information, ask your geotechnical consultant for risk-management guidance. As a general rule, *do not rely on an environmental report prepared for a different client, site, or project, or that is more than six months old.*

Obtain Professional Assistance to Deal with Moisture Infiltration and Mold

While your geotechnical engineer may have addressed groundwater, water infiltration, or similar issues in this report, none of the engineer's services were designed, conducted, or intended to prevent uncontrolled migration of moisture – including water vapor – from the soil through building slabs and walls and into the building interior, where it can cause mold growth and material-performance deficiencies. Accordingly, ***proper implementation of the geotechnical engineer's recommendations will not of itself be sufficient to prevent moisture infiltration.*** Confront the risk of moisture infiltration by including building-envelope or mold specialists on the design team. ***Geotechnical engineers are not building-envelope or mold specialists.***

GEOPROFESSIONAL
BUSINESS
ASSOCIATION

Telephone: 301/565-2733
e-mail: info@geoprofessional.org www.geoprofessional.org

Delivering Solutions
Improving Lives