

PRELIMINARY GEOTECHNICAL INVESTIGATION REPORT

October 8, 2025

Prepared For:

Long Beach Unified School District

Ms. Gricelda Perez
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Long Beach, California 90810



N|V|5

Beyond Engineering

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Long Beach Polytechnic High School
Polytechnic High School Improvements Project
1545 Long Beach Boulevard
Long Beach, California 90813

Project No.: 1400125-0009800.00

Long Beach Unified School District
Facilities Development and Planning
2425 Webster Avenue
Long Beach, California 90810

October 8, 2025
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Subject: Preliminary Geotechnical Investigation Report

Project: Long Beach Unified School District Polytechnic High School
Polytechnic High School Improvements Project
1545 Long Beach Boulevard
Long Beach, California

Dear Ms. Perez,

This report presents the results of NV5, Inc.'s (NV5) geotechnical investigation for the Long Beach Unified School District (LBUSD) Polytechnic High School Improvements Project located in Long Beach, California.

The purpose of this investigation was to evaluate the subsurface conditions at the proposed project site and to provide geotechnical recommendations pertaining to the design and construction of the Polytechnic High School Project. The accompanying report includes a discussion of the subsurface soil conditions observed during NV5's study, a review of available relevant geotechnical documents and geotechnical engineering analyses. Based on the results of the subsurface exploration, subsequent testing of the retrieved soil samples, 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.

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, Inc.

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1.0 INTRODUCTION

This report provides the results of NV5's geotechnical engineering investigation for the Polytechnic High School Project located in Long Beach, California, see **Figure 1, Site Location Map**. The general purpose of this study was to evaluate the subsurface soil conditions at the project site and to provide geotechnical recommendations for the design and construction of the project. This report summarizes the data collected and presents NV5's findings, conclusions and geotechnical design recommendations.

This report has been prepared for the exclusive use of the client and their consultants in the design of the proposed new structures. 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 plans, geotechnical maps and literature pertaining to the vicinity.
- A site reconnaissance to observe the general surficial site conditions and to select boring locations.
- Coordination with entities having an interest in the field exploration activities including the Project Design Team, Long Beach Unified School District (LBUSD) staff, the private underground utility locator GPRS, the exploration subcontractor (24-7 Drilling), and Underground Service Alert (USA).
- Field exploration activities consisted of:
 - Conducting a subsurface investigation, which included the drilling, logging, and sampling of five (5) exploratory borings (B-01 through B-05) located within the project site. The final depths of the exploratory borings ranged from approximately 5 to 25 feet below ground surface (bgs).
 - Conducting three (3) percolation tests at the project site to evaluate infiltration characteristics at a depth of 5 feet bgs.
- Performing laboratory testing on selected representative bulk and relatively undisturbed soil samples obtained during the field exploration program 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 of the geotechnical data collected to develop geotechnical recommendations for the design and construction of the proposed project. Specifically, the following items were addressed:
 - Evaluation of project feasibility and suitability of on-site soils for foundation support.

- Evaluation of general subsurface conditions and description of types, distribution and engineering characteristics of subsurface materials.
- General recommendations for earthwork, including site preparation, excavation, percolation testing, site drainage and the placement of compacted fill.
- Recommendations for temporary slopes/cuts and shoring.
- Recommendations for design of suitable foundation systems including allowable bearing capacity, lateral resistance, settlement estimates and slab-on-grade construction.
- Determination of seismic design parameters.
- Recommendations for subgrade preparation within proposed exterior flatwork.
- 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

The proposed project site is located at the Poly Academy of Achievers and Leaders "PAAL", located at 1545 Long Beach Boulevard, in Long Beach, California. The project site is bounded by East 16th Street to the north, Long Beach Boulevard to the east, by North Palmer Court on the west, and by a private business on the south. The site location, with respect to the surrounding roadways, development and other features is shown on **Figure 1, Site Location Map** and **Figure 2, Site Vicinity Map**.

The existing structures were constructed in 1996 and service as an annex to Poly High School. The site is approximately 1.9 acres in size with the existing structures encompassing approximately 30,550 square feet. It is understood the site will be redeveloped into a softball/multipurpose field for use by Polytechnic High School.

Based on NV5's review of the LBUSD Request for Proposal, dated May 30, 2025, and the 50% Construction Documents for the referenced project prepared by PBK Architects, NV5 understands the proposed development includes the following:

- The demolition of nine existing modular buildings, shade structures and any associated site work.
- New softball field that meets CIF field regulations and associated softball field fencing and gates.
- New multi-purpose field that meets CIF soccer field regulations over the softball field and associated field goal posts.
- Incorporate new backstop, bleachers, dugouts, lighting (6 lights), scoreboard, and fencing and gates. Including new restroom and storage structures.
- The program is to include:
 - (2) bullpens/batting cages combo, (1) for home side and (1) for visitor side
 - 3-5 row bleacher with a 120-spectator capacity
 - Ticketing, concessions and restrooms building
 - Enclosed outdoor storage containers

- Associated chain link perimeter fencing and gates
- Associated sports field lighting
- Associated scoreboard

The majority of the project site is relatively level with a gentle slope to the northeast. Surface runoff appears to mostly occur as sheet flow and directed into several small landscaping drain inlets across the site or to inlets along the adjoining streets. Current site elevations range from a high of approximately 41 feet above mean sea level (MSL) in the southwestern corner of the campus to a low of approximately 35 feet near in the northeastern portion of the campus.

4.0 FIELD EXPLORATION PROGRAM

Prior to starting the field exploration program, a field reconnaissance was conducted to observe site conditions and mark out the locations for the planned subsurface explorations. As required by law, USA was notified of the locations of the exploratory borings prior to drilling. In addition, NV5 conducted a private utility clearance of the proposed exploratory boring locations with Ground Penetrating Radar Systems (GPRS). NV5 coordinated the drilling schedule with the project team, LBUSD and Polytechnic High School staff.

4.1 EXPLORATORY DRILLING

The subsurface conditions at the project site were explored on September 17th, 2025, by drilling, logging, and sampling five (5) exploratory borings (B-01 to B-05) located within the project site. Additionally, three (3) borings were drilled to 5 feet bgs (P-01, P-02, and P-04) and were used for percolation testing. The approximate locations of the borings and percolation test holes are shown on **Figures 3A, Exploration Location Map With Existing Buildings** and **Figure 3B, Exploration Location Map On Proposed Development Map**.

The exploratory borings were drilled with 8-inch diameter hollow-stem augers, utilizing a truck-mounted drilling rig (Diedrich D-70) to depths between 5 and 25 feet bgs. The materials encountered in the exploratory borings were continually observed, classified, and logged by an NV5 engineer in general accordance with the Unified Soil Classification System. The exploratory boring logs are presented in **Appendix A, Exploratory Boring Logs**.

Representative bulk samples and relatively undisturbed drive samples of the soils encountered were obtained in the field during the subsurface evaluation. The samples were labeled in the field and transported to NV5's laboratory for observation and testing. Subsequent to logging and sampling the exploratory borings were backfilled with soil cuttings and patched to match the existing location condition. The drive samples were obtained using the California Modified (CAL) and Standard Penetration Test (SPT) split-barrel samplers.

4.2 FIELD PERCOLATION TESTING

On September 18th and 19th, 2025, three (3) percolation tests were performed at the project site to evaluate the infiltration characteristics of the onsite soils as it relates to the feasibility of storm water

infiltration. The percolation test was conducted in 8-inch diameter borings drilled adjacent to borings B-01, B-02, and B-04, to a final depth of approximately 5 feet bgs. The percolation tests performed in general accordance with the *Los Angeles County Public Works Geotechnical Engineering Division Administrative Manual GS200.1 Procedure*. Tests performed using a 3-inch PVC, schedule 40 casing, with ½ inch slots with 3/8 inch gravel used for gravel pack. The approximate location of the percolation tests are presented on **Figure 3A, Exploration Location Map With Existing Buildings** and **Figure 3B, Exploration Location On Proposed Development Map**.

Water level measurements were taken at time intervals during the percolation tests in accordance with the referenced procedure. The results of the percolation tests are presented in the following Table.

Table 1 - Percolation Test Results

Percolation Test Location	Depth Below Ground Surface	Soil Description	Measured Final Infiltration Rate (inch/hour)	Adjusted Infiltration Rate* (inch/hour)
P-01	5 feet	Sandy CLAY (CL)	1.2	0.4
P-02	5 feet	Silty SAND (SM)	1.2	0.4

*Rate adjusted for type of boring, site variability, and long-term siltation/maintenance per GS200.1

At location P-04, percolation testing demonstrated extremely high and unreliable flow rates in the vicinity. As such, the data set is omitted from the observed and design percolation test rates. The percolation test data suggest infiltration rates that are considered unfavorable (i.e. very low transmissivity of groundwater). Percolation test data is attached to this report in **Appendix C, Percolation Test Results**. In addition, the site is generally underlain in the upper 5 feet by sandy clay soil. Based on these conditions, the project site may lead to the build-up and subsequent lateral migration of infiltration water. It is NV5's opinion that the subject site is considered **not** suitable for infiltration of storm water runoff in any amount. If desired, stormwater BMPs may be designed with an impermeable liner and piped to a suitable discharge outlet.

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, and cementation).

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 material classifications and to evaluate engineering properties of the materials encountered. The following tests were performed:

- Moisture content and density (ASTM D2216 and D2937);
- Sieve analyses (ASTM D6913);
- Atterberg Test (ASTM D4318)
- Expansion Index (ASTM D4829);
- Direct shear (ASTM D3080); and

- Corrosivity test series including sulfate content, chloride content, pH-value, and electrical resistivity (CTM 643 and ASTM D516).

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

6.0 GEOLOGY

6.1 GEOLOGIC SETTING

The project site is located in southeastern Los Angeles County within the western coastal margin 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. Nearby Holocene-active faults include the Newport-Inglewood-Rose Canyon fault zone, located approximately 1.52 miles northeast of the site and the Palos Verdes Connected fault zone, located approximately 4.6 miles southwest of the site.

Typical stratigraphy of the project area includes Quaternary-age old shallow marine and non-marine sedimentary deposits.

6.2 GEOLOGIC MATERIALS

Geologic materials encountered during the subsurface exploration include fill, associated with the development of the school, underlain by Quaternary-Age old marine deposits. **Figure 4, Regional Geologic Map**, presents the distribution of geologic units in the vicinity of the site. Detailed descriptions of the earth materials encountered are presented on the exploratory boring logs in **Appendix A, Exploratory Boring Logs**. The lateral distribution of the earth materials encountered during the field investigation is presented in **Figures 5A and 5B (Geologic Cross Section A-A' and Geologic Cross Section B-B', respectively)**. Generalized descriptions of the units encountered during the field exploration are provided below:

- **Artificial Fill (Af):** Artificial fill (Af) soils were encountered in all five (5) of the exploratory borings at interpreted thickness of approximately 2 to 15 feet. As encountered during the subsurface investigation, the fill soils generally consisted of light brown to olive brown, dry to moist, loose to dense, fine- to coarse-grained silty sand (SM), poorly graded sand (SP), and clayey sand (SC), to sandy silt (ML), and sandy lean clay to lean clay with sand (CL) with varied amounts of fine- to coarse-grained sand. Based on the low expansive characteristics (as evidenced by lab testing) of this material, much of it can likely be reused provided it is properly conditioned and recompacted. Recommendations for preparation and compaction engineered fill is provided later in this report.
- **Quaternary Old Shallow Marine Deposits (Qom):** Shallow marine deposits were encountered in three (3) of the borings below the artificial fill materials. These materials consisted of brown to olive brown, moist, medium dense to dense, silty sand (SM), poorly graded sand (SP), and poorly graded sand with silt (SP-SM), to sandy lean clay (CL), and sandy silt (ML).

6.3 GROUNDWATER

Indications of a static, near-surface groundwater table were not observed or encountered during the subsurface exploration. Review of available geotechnical documents, in the vicinity of the subject site, on the State of California's Sustainable Groundwater Management Act (SGMA) Data viewer tool website indicates that the static groundwater table is on the order of 25 to 35 feet bgs within the vicinity of the subject site area. During our field investigation, we did not encounter static groundwater. However, based on potential fluctuations noted in our review historical information provided above and our experience, 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.

NV5 has researched the *California State Waterboard GeoTracker website* (<https://geotracker.waterboards.ca.gov/>) to obtain additional historical information regarding groundwater levels in the general vicinity of the site. Review of the reported groundwater level in the researched reports indicate that the reported high groundwater level in monitoring wells located approximately 0.35-miles southwest of the site (132 Anaheim Street West) in October 2010 was reported to be at an elevation of 3 feet above MSL.

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 fault zone located approximately 1.5 miles northeast of the site. Other important active faults that could affect the Long Beach area and their distance to the site are included in the following Table 2. In addition, **Figure 6, Regional Fault and Seismicity Map** depicts the site location in relation to known active faults in the region.

Table 2 – Summary of Major Active Faults

Fault	Distance From the Site (mi)
Newport Inglewood Connected	1.5 miles
Palos Verdes Connected	4.6 miles
Puente Hills (LA)	8.8 miles

San Joaquin Hills	16 miles
Elsinore	16 miles
Newport Inglewood Offshore	21 miles
Santa Monica Connected	23 miles
Verdugo	24 miles
San Jose	25 miles
Malibu Coast	26 miles
Anacapa-Dume	26 miles
Sierra Madre Connected	28 miles
Chino	30 miles
Cucamonga	35 miles
San Gabriel	37 miles
Northridge	38 miles
Coronado Bank	38 miles
Santa Susana	41 miles
Simi-Santa Rosa	46 miles
San Andreas	49 miles
San Jacinto	50 miles
Oak Ridge	52 miles
Cleghorn	55 miles
San Cayetano	56 miles
Santa Cruz Island	63 miles
Rose Canyon	64 miles
Channel Islands Thrust	64 miles
North Frontal West	64 miles
Pitas Point Connected	67 miles
Santa Ynez Connected	69 miles
Mission Ridge-Arroyo Parida Santa Ana	73 miles
Red Mountain	74 miles
Garlock	82 miles
Helendale-So Lockhart	83 miles
Pinto Mountain	86 miles
Lenwood – Lockhart – Old Woman Springs	96 miles

Source: https://earthquake.usgs.gov/cfusion/hazfaults_2008_search/

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 known active faults are not known to pass directly through the site. The Alquist-Priolo (AP) mapped Newport-Inglewood Connected fault zone approximately 1.5 miles to the northeast and does not trend towards the site. Based on the distance to the mapped trace of the fault and the distance to other faults in the vicinity of the site, the potential for damage due to surface rupture of faults at the project site is considered 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 that 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 2022 California Building Code (CBC) guidance document. Using the Structural Engineers Association of California's U.S. Seismic Design Maps Online Calculator (<https://seismicmaps.org/>), based on site latitude = 33.786822 degrees North and longitude = -118.189914 degrees West.

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 area are presented in the following Table 3. NV5 should be contacted to provide revisions to these parameters if other codes are specified. Per ASCE Chapter 20, the Site Class was determined as "D".

Table 3 - Recommended 2022 CBC Seismic Design Parameters

Design Parameter	Recommended Value	Reference
Site Class	D	CBC Section 1613.3.2
Mapped Spectral Accelerations for short periods, S_s	1.86g	CBC Section 1613.2.1
Mapped Spectral Accelerations for 1-sec period, S_1	0.73g	CBC Section 1613.2.1
⁽¹⁾ MCE_R (5% damped) spectral response acceleration for short periods adjusted for site class, S_{MS}	1.95	CBC Section 1613.2.3
⁽¹⁾ MCE_R (5% damped) spectral response acceleration at 1-second period adjusted for site class, S_{M1}	2.29g*	CBC Section 1613.2.3
Design spectral response acceleration (5% damped) at short periods, S_{DS}	1.3g	CBC Section 1613.2.4
Design spectral response acceleration (5% damped) at 1-second period, S_{D1}	1.53g*	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	0.76g	ASCE 7-16 Section 11.8.3

(1) MCE_R = Risk-adjusted Maximum Considered Earthquake

MCE_G = Geometric-mean Maximum Considered Earthquake

* Value increased by 50% per CBC 2022, Supplement 3

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. The potential for liquefaction under the same conditions of ground shaking intensity and duration will decrease for sands that are more well-graded, more irregular and gritty, coarser and denser. Also, a pronounced decrease in liquefaction potential will occur with the increase in fine-grained (i.e., silt and clay) content. Seed and others have suggested that a non-liquefiable classification be assigned if the clay fraction is 15 percent or greater (*Guidelines for Evaluating and Mitigating Seismic Hazards in California, Special Publication 117, CDMG, Ch. 6, 1997*). Dynamic settlement due to earthquake shaking can occur in both dry and saturated sands. The potential consequences of liquefaction to engineered structures include loss of bearing capacity, buoyancy forces on underground structures (including pipelines), increased lateral earth pressures on retaining walls, and lateral spreading. Pipes constructed in soils that become liquefied may become buoyant.

The site is underlain by medium dense to dense, old marine deposits at shallow depths. Groundwater was not encountered to maximum depth explored. As noted on **Figure 7, Regional Liquefaction Hazard Map**, the California Geological Survey (CGS) Online Liquefaction Hazard Zone Viewer indicates the project site is not within a liquefaction zone. Therefore the potential for liquefaction and associated ground deformation occurring beneath the structural areas is considered low.

7.4 LANDSLIDES AND SLOPE INSTABILITY

The subject property has relatively low relief with no major or steep slopes. There are no known landslides on or near the property, and the site is not located in the path of any known landslides. Indications of slope instability were not observed at the time of the field exploration, and the geologic materials underlying the site are not known to be prone to slope instability in properly engineered slopes. It is NV5's opinion that the potential damage to the planned improvements due to landsliding or slope instability is considered low.

7.5 SUBSIDENCE

The Long Beach area is a region generally known for historic ground subsidence. Subsidence has been attributed to regional geologic processes, fluid withdrawal associated with agricultural production, and oil extraction. The subsidence occurs when groundwater (near the surface or in a deep aquifer) is lowered past its historical level. This occurrence results in an increase of effective stress within a soil layer which typically translates into additional soil consolidation. Based on research of the State of California's SGMA Data Viewer (<https://sgma.water.ca.gov/webgis/>) and the United States Geological Survey (USGS) Areas of Land Subsidence in California Online Tool (https://ca.water.usgs.gov/land_subsidence/california-subsidence-areas.html), subsidence in the project area is known to be an on-going hazard and likely to continue. The proposed construction should be designed to factor in continued regional subsidence in the project area. The proposed project location is presented on the attached **Figure 8, Regional Subsidence Hazard Map**.

7.6 TSUNAMIS, INUNDATION SEICHE, AND FLOODING

The site is located at an elevation of approximately 36 feet above MSL. Its lowest point is located approximately 1.24 miles from the Los Angeles River and 1.6 miles from the Pacific Ocean. 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 low.

Based on a review of Federal Emergency Management Agency (FEMA) flood insurance rate map (FIRM), the site is located within an area of minimal flood hazard (Zone X). The proposed project location is presented on the attached **Figure 9, FEMA Flood Hazard Map**. Based on the map review, the potential for significant flooding of the site is considered to be low. Site drainage should be addressed by the project civil engineer in accordance with the recommendations of this report.

7.7 EXPANSIVE SOILS

Improvement including foundations and slabs in contact with earth materials with a high potential for expansion can be expected to be subject to distress based on the potential for volume change associated with highly expansive soil. Soils such as these should not be relied upon for foundation bearing.

Existing artificial fill material was encountered in a majority of the exploratory borings (B-01 through B-05) drilled at the site, at depths ranging from approximately 2 to 15 feet bgs, overlying alluvial materials to the total depths explored. Expansion index test results of soil samples of the near surface fill (obtained in borings B-04, depths between 1 to 5 feet) resulted in a “low” expansion potential of the tested material. Without adequate accommodation for these soils in the foundation/s 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 given in the grading and earthwork recommendations section. These materials are generally considered suitable for use, at a depth of greater than 5 feet, as compacted fills for underground pipeline and utility backfill, but are generally considered unsuitable for use as backfill for retaining walls or pipe bedding. Since site grading may redistribute the on-site soils, potential expansive soil properties should be verified at the completion of rough grading.

However, it is noted that variability in soil conditions may occur between boring locations, and further observation and testing of near surface soils may be required if differing conditions are encountered during construction. If medium or highly expansive soils are encountered during site grading and excavation, they may not be suitable for use as structural fill or trench backfill.

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. 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, retaining walls and corrosion potential are provided in the following report sections.

8.2 GRADING AND EARTHWORK

Grading using conventional cut-and-fill earthwork is anticipated for the preparation of the proposed buildings and other improvement areas. Site preparation and grading should be performed in accordance with applicable grading codes, grading permits, project plans and specifications and the recommendations provided herein. Grading observation and testing will be required by NV5 in order to certify fills in conformance with grading permit requirements. Site grading and other miscellaneous construction should be performed in accordance with the following recommendations and the *Typical Earthwork Guidelines* provided in **Appendix D, Typical Earthwork Guidelines**. In the event of conflict, the recommendations presented herein supersede those in **Appendix D, Typical Earthwork Guidelines**.

- Clearing and Grubbing - 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 obstruction which extend below finished site grades should be filled with properly compacted soils. Any utilities within the footprint of planned structural improvements should be appropriately abandoned.
- Site Grading – Areas to receive surface improvements or fill soils should be treated as follows:
 - Removals Below Proposed New Building and Retaining Walls – It is understood that the proposed building will be completely supported at wall footings, with a raised slab. For this building, prior to any fill placement, the existing fill soils underlying the proposed structures footings should be excavated down to a depth of 3 feet below bottom of planned building footings, unless otherwise directed by the Geotechnical Engineer. The excavation should extend laterally a distance of at least 1 foot on either side from the footing edge or 4 feet wide, whichever is greater. The extent and depths of removals and overexcavations should be evaluated by NV5 representative in the field. The soils exposed in the bottom of the excavation should be scarified to a depth of 6 inches, moisture conditioned to near optimum and uniformly recompacted to at least 90 percent of the soil maximum density (based on ASTM D1557). A cut-fill transition condition should not be allowed underlying proposed structures.
 - Removals Below Proposed New Flatwork and Turf Field – Prior to any fill placement, the existing fill soils underlying the proposed flatwork and turf field should be over-excavated to a depth of 2 feet below the current grade unless otherwise directed by the Geotechnical Engineer. The excavation should extend laterally a distance of at least 2 feet beyond the footprint where feasible. The extent and depths of removals and overexcavations should be evaluated by NV5 representative in the field. The soils

exposed in the bottom of the excavation should be scarified to a depth of 6 inches, moisture conditioned to near optimum and uniformly recompacted to at least 90 percent of the soil maximum density (based on ASTM D1557).

- Excavatability – Based on the subsurface exploration, it is anticipated that the on-site soils can be excavated by modern conventional heavy-duty excavating equipment in good operating condition.
- Material for Fills – Fill materials should be free of deleterious or oversized materials. Any rocks with a maximum dimension greater than 4 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. Fine grained, plastic soils should not be used for foundation support. The removed soils should be mixed thoroughly, prior to placement, to achieve relevant uniformity and consistence for the planned engineered fill.

As noted, laboratory testing of onsite soils indicated low expansion potential. Onsite soil materials are considered suitable for reuse as compacted fills as long as they meet project specifications and have an Expansive Index of less than 50. Since site grading may redistribute the on-site soils, potential expansive soil properties should be verified at the completion of rough grading.

It is noted that variability in soil conditions may occur between boring locations, and further observation and testing of near surface soils may be required if differing conditions are encountered during construction. If highly expansive soils are encountered during site grading and excavation, they would not be suitable for use as structural fill or trench backfill.

- 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 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 (2022) 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.
- Structural Fill Placement - Areas to receive fill and/or surface improvements should be scarified to a minimum depth of 6 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 at least 1 percent over optimum moisture conditions and compacted in uniform lifts to at least 90 percent relative compaction (ASTM D1557). 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.

- Graded Slopes – Graded slopes should be constructed at a gradient of 2:1 (H:V) 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.

8.3 TEMPORARY EXCAVATIONS AND SHORING

Although shoring is not anticipated for the project, the following recommendations are provided on a preliminary basis. Vertical excavations greater than 4 feet high should not be attempted without proper shoring to prevent instabilities. 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. The design and construction of temporary slopes and excavations, as well their maintenance and monitoring during construction, is the responsibility of the contractor. The contractor should have a competent person evaluate the soil or rock conditions encountered during excavation to determine permissible temporary slope inclinations and other measures as required by California Occupational Safety and Health Administration (Cal-OSHA). The contractor's competent person should observe temporary slopes at regular intervals to assess their need for maintenance and stability.

For planning purposes, the fill materials may be considered as Type C with temporary excavation slopes of 1.5:1 (horizontal: vertical) and the native soil materials may be considered as Type B with temporary excavation slopes of 1:1, 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 structures. 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:

$$P = K \cdot \gamma \cdot H + \text{Surcharge 1}$$

Where:

- H = height of the excavation
- γ = soil unit weight, where for above water ground is 120 pcf, and for below water level is $\gamma' = 62.6$ pcf
- $K_0 = 0.5$ at-rest earth pressures should be assumed for the geotechnical design, where the wall support does not allow lateral displacement
- $K_a = 0.3$ 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.

8.4 DEWATERING

Groundwater was not encountered within the maximum explored depth of 25 feet bgs and is not anticipated to be a significant concern during site development. As indicated previously, the groundwater table is subject to fluctuations in response to a number of factors. 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 Regional Water Quality Control Board (RWQCB) regulations. Any cases of seepage or heavy precipitation should be monitored during construction.

8.5 FOUNDATIONS FOR PROPOSED STRUCTURES

Foundations for proposed structures should be supported on compacted granular fill prepared in accordance with *Section 8.2*. Recommendations for the design and construction of the structural mat foundation system are presented below.

8.5.1 Shallow Footing Design Parameters

Assuming subgrade is compacted as noted previously, foundations for the proposed building should be designed using the geotechnical design parameters presented in the following Table 4. 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.

Table 4
Geotechnical Design Parameters For Shallow Foundations

Minimum Foundation Dimensions	Continuous footings should be 18 inches in width and be embedded 24 inches below the lowest adjacent grade. Footing bottoms should bear on at least 3 feet of engineered fill.
Allowable Bearing Capacity (dead-plus-live load)	2,000 pounds per square foot (psf) 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	0.25 0.10 in the event a vapor barrier is used.
Allowable Lateral Passive Resistance (Equivalent Fluid Pressure)	100 pounds per cubic foot (pcf) equivalent fluid pressure. 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.

8.5.2 Settlement

Estimated settlements will depend on the foundation size and depth, and the loads imposed and the allowable bearing values used for design. For preliminary design purposes, the total static settlement for foundations loaded to accordance with the allowable bearing capacities recommended above is estimated to be 1-inch. Differential static settlements are anticipated to be 1/2-inch over a distance of 40 feet.

8.5.3 Shallow Foundation Observation

To verify the presence of satisfactory materials at design elevations, footing excavations should be observed to be clean of loosened soil and debris before placing steel or concrete and probed for soft areas. If soft or loose 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. Controlled

low-strength material (CLSM) should display a minimum 28-day compressive strength of 50 psi. If required, sampling, testing and inspection of a CLSM should be completed in accordance with DSA IR 18-1 (issued 07/15/2025). Footing excavations should be deepened as necessary to extend into satisfactory bearing materials; however, NV5 should be notified to review the proposed change.

8.5.4 Isolated Pier Footing Parameters

It is assumed that isolated pier foundations will be needed for stadium light poles, backstops, and similar tall lightly loaded structures. It is assumed that the lateral demands of the piles will control the design depth, therefore the existing fills can remain in place.

Table 5
Geotechnical Design Parameters for Pier Foundations

Foundation Dimensions	Minimum pier diameter should be 18 inches and be embedded a minimum of 6 feet.
Allowable Skin Friction (unit)	150 psf per foot of depth
Allowable Uplift (unit)	100 psf per foot of depth
Reinforcement	Reinforce in accordance with requirements as provided by the project Structural Engineer.
Allowable Lateral Passive Resistance (Equivalent Fluid Pressure)	<p>100 pounds per cubic foot (pcf) equivalent fluid pressure, up to a maximum of 3,000 psf.</p> <p>A one-third ($1/3$) increase in passive resistance value may be used for wind and seismic loads.</p> <p>The passive resistance in the upper 1 feet of soil should be neglected unless it is removed and constrained by concrete at the surface.</p>

Total settlement of pole footings as recommended above is estimated to be 1 inch.

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 for caving conditions. 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 soil 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 be used during the pile construction. The bottom of holes should also be cleaned of loose soil and gravel. Drilling difficulty should be anticipated in cemented zones and if cobbles and boulders are encountered.

8.5.5 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.

8.5.6 Interior Concrete Slabs-on-Grade

While not anticipated to be used for this site, interior concrete slabs-on-grade may be supported at grade on at least 2 feet of compacted fill with very low to low expansion potential. For design of these concrete slabs, a modulus of subgrade reaction (k) of 100 pounds per cubic inch (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 at least 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 approved equivalent meeting the requirements of ASTM E1745-17, 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. The upper 12 inches of subgrade soil located below the vapor retarder should be moisture-conditioned within 2 percent over the optimum moisture content and compacted to a minimum of 90 percent relative compaction (ASTM D1557).

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.

8.5.7 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 90 percent of the maximum dry density. The upper 12 inches of subgrade soil located below the aggregate base should be moisture-conditioned within 2 percent over the optimum moisture content, and recompacted to a minimum of 90 percent relative compaction (ASTM D1557).

Any driveway slab areas and connecting sidewalks under vehicular loading should have a minimum concrete thickness of 6 inches. The driveway concrete slab 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 within 2 percent over the optimum moisture content, and recompacted to a minimum of 95 percent relative compaction (ASTM D1557).

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.

8.6 UTILITY TRENCH BACKFILL

All 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 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.

8.7 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 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 $\frac{3}{4}$ -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.

8.8 CORROSION POTENTIAL

General recommendations to address the corrosion potential of the on-site soils are provided below. 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.

8.8.1 Caltrans Criteria

The California Department of Transportation (Caltrans) Corrosion Guidelines (Version 3.2, dated May 2021) 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 1,500 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 Table 7 below and presented in **Appendix B, Laboratory Test Results**.

Table 7 - Corrosivity Test Results

Boring Location and Sample Depth	pH	Minimum Electrical Resistivity (Ohm-cm)	Soluble Sulfate Content (%)	Soluble Chloride Content (%)
B-01 @ 6-6.5'	8.0	1,320	0.04	0.02

Based on experience and the Caltrans Corrosion Guidelines, the site soils are not considered to be corrosive to steel and concrete foundation elements based on the sulfide and chloride test results.

8.8.2 ACI Criteria

Based on a review of the American Concrete Institute (ACI) 318, Section 19.3, the following is noted:

- Per Table 19.3.1.1 and Table 19.3.2.1 – Sulfate Content Test Results
 - The tested site soils are in a “Class S0” Exposure Category. A minimum concrete compressive strength of 2,500 psi would be required with no specific requirement for water cement ratio.

- Per Table 19.3.2.1 – Chloride Content Test Results
 - The tested site soils are in a “Class C1” Exposure Category (assuming a potential for encountering water). A minimum concrete compressive strength of 2,500 psi would be required with no specific requirement for cement ratio. The maximum water-soluble chloride ion content in non-prestressed concrete is 0.30 percent by weight of cement

8.8.3 Ferrous Pipes Criteria

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 8:

Table 8 - 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 highly 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.

8.9 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 and foundation construction will be important to the performance of the proposed improvements. The following sections present NV5's 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, drilled hole excavation, and other earthwork operations should be observed and tested as needed. 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/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

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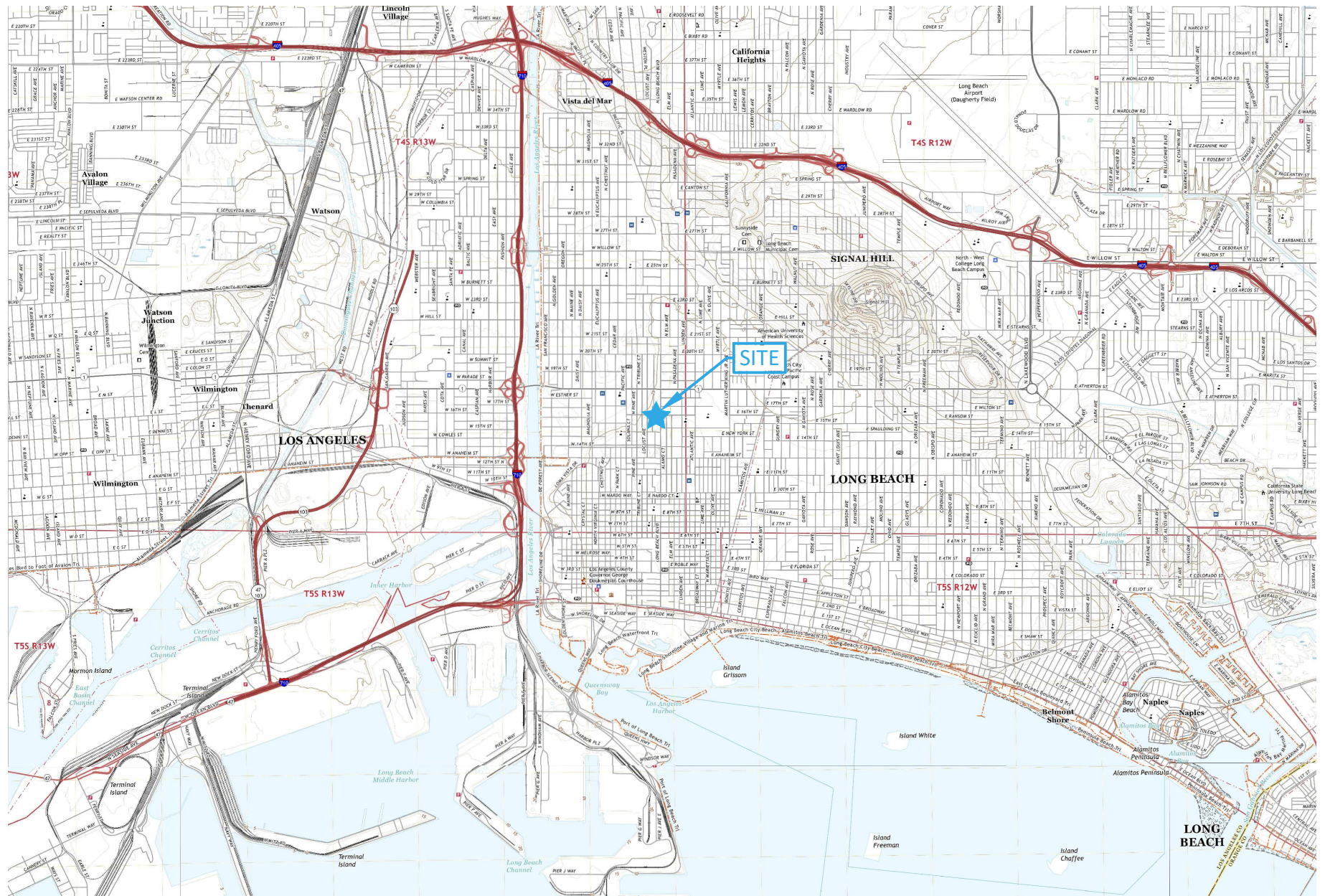
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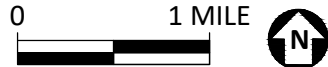
United States Geological Survey (USGS), 2022, Interactive U.S. Fault Map, <https://www.usgs.gov/programs/earthquake-hazards/faults>

United States Geological Survey (USGS), 2025, Earthquake Catalog, <https://earthquake.usgs.gov/earthquakes/search/>

United States Geological Survey (USGS), 2025. Areas of Land Subsidence in California Online Tool, https://ca.water.usgs.gov/land_subsidence/california-subsidence-areas.html



REFERENCE: TORRANCE, LONG BEACH, LOS ALAMITOS, SAN PEDRO, LONG BEACH EO, AND SEAL BEACH, QUADRANGLES USGS 7.5 MINUTE SERIES (2021)

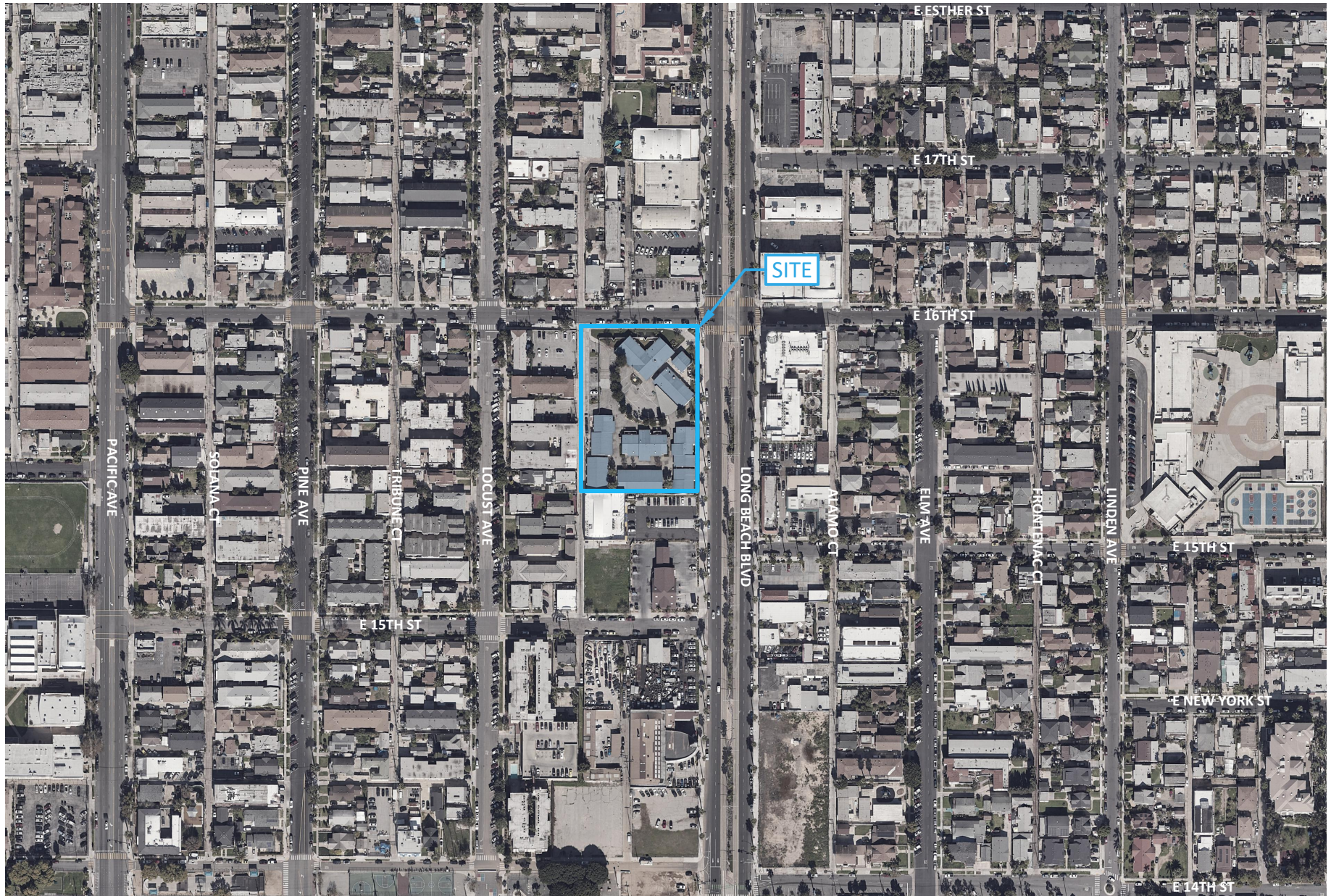


SCALE, LOCATIONS, AND DIRECTIONS ARE APPROXIMATE.

POLYTECHNIC HIGH SCHOOL IMPROVEMENTS PROJECT

SITE LOCATION MAP

NV5 Beyond Engineering	
PROJECT NUMBER	FIGURE NUMBER
1400125-0009800.00	1



REFERENCE: GOOGLE, INC (2025) GOOGLE EARTH PRO, AERIAL IMAGERY DATED: DECEMBER 4, 2023



SCALE, LOCATIONS, AND DIRECTIONS ARE APPROXIMATE.

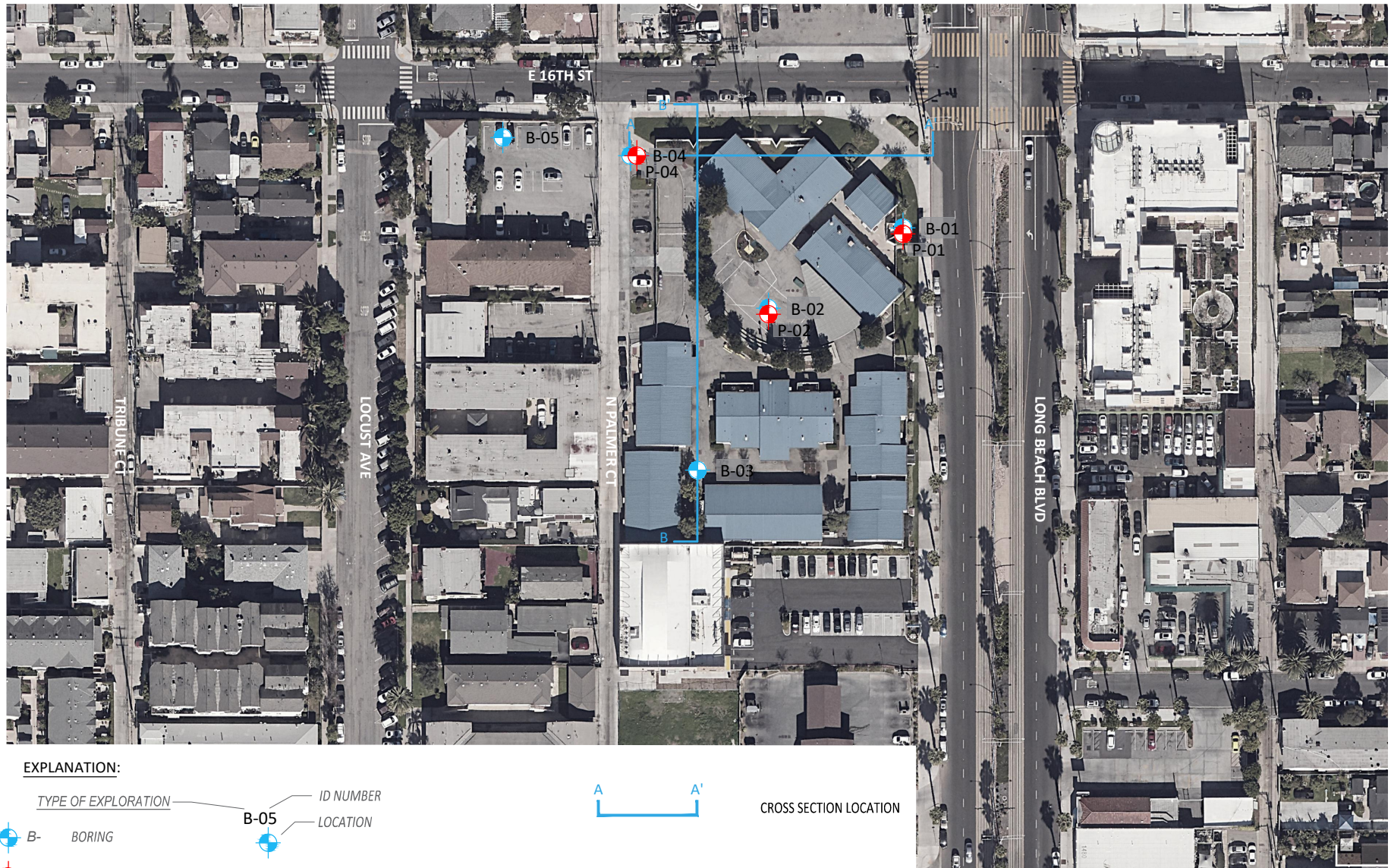
POLYTECHNIC HIGH SCHOOL IMPROVEMENTS PROJECT

SITE VICINITY MAP

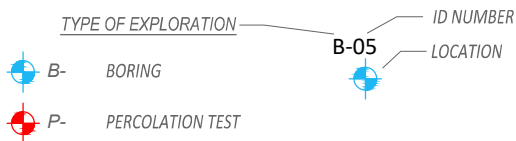
NV5 Beyond Engineering

PROJECT NUMBER
1400125-0009800.00

FIGURE NUMBER
2



EXPLANATION:



CROSS SECTION LOCATION



NOTE: SCALE, LOCATIONS, AND DIRECTIONS ARE APPROXIMATE.

REFERENCE: GOOGLE, INC (2025) GOOGLE EARTH PRO, AERIAL IMAGERY DATED: DECEMBER 4, 2023

**POLYTECHNIC HIGH SCHOOL
IMPROVEMENTS PROJECT**

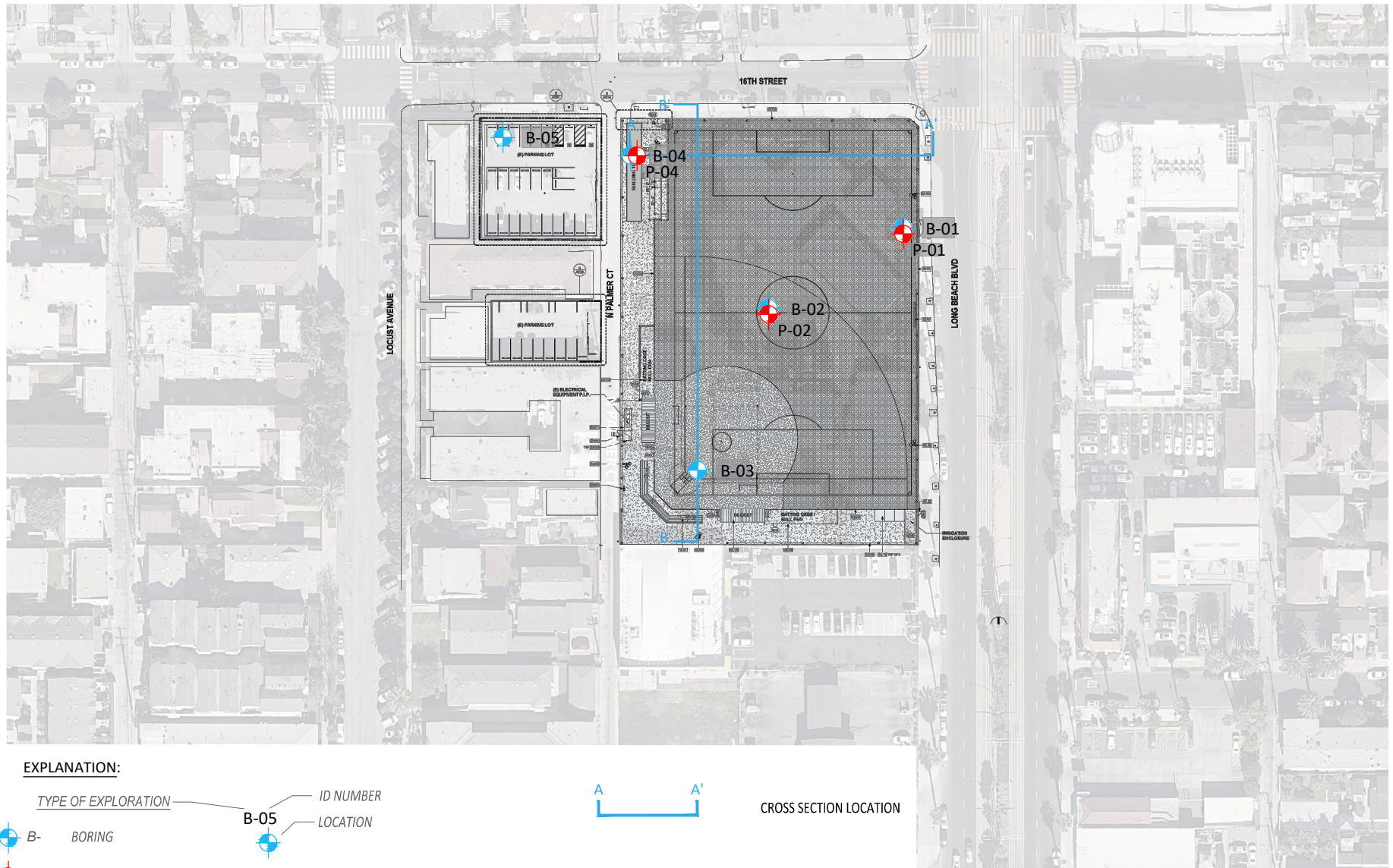
**EXPLORATION LOCATION MAP
WITH EXISTING BUILDING**

NV5

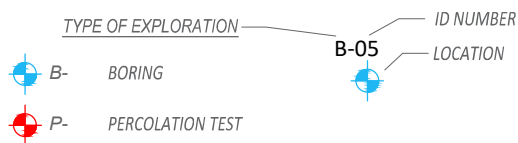
Beyond Engineering

PROJECT NUMBER
1400125-0009800.00

FIGURE NUMBER
3A



EXPLANATION:



CROSS SECTION LOCATION



NOTE: SCALE, LOCATIONS, AND DIRECTIONS ARE APPROXIMATE.

**POLYTECHNIC HIGH SCHOOL
IMPROVEMENTS PROJECT**

**EXPLORATION LOCATION ON
PROPOSED DEVELOPMENT MAP**

REFERENCE: OVERALL SITE PLAN, PBK ARCHITECT, SHEET A010, 06/24/2025
GOOGLE, INC (2025) GOOGLE EARTH PRO, AERIAL IMAGERY DATED: DECEMBER 4, 2023

NV5

Beyond Engineering

PROJECT NUMBER
1400125-0009800.00

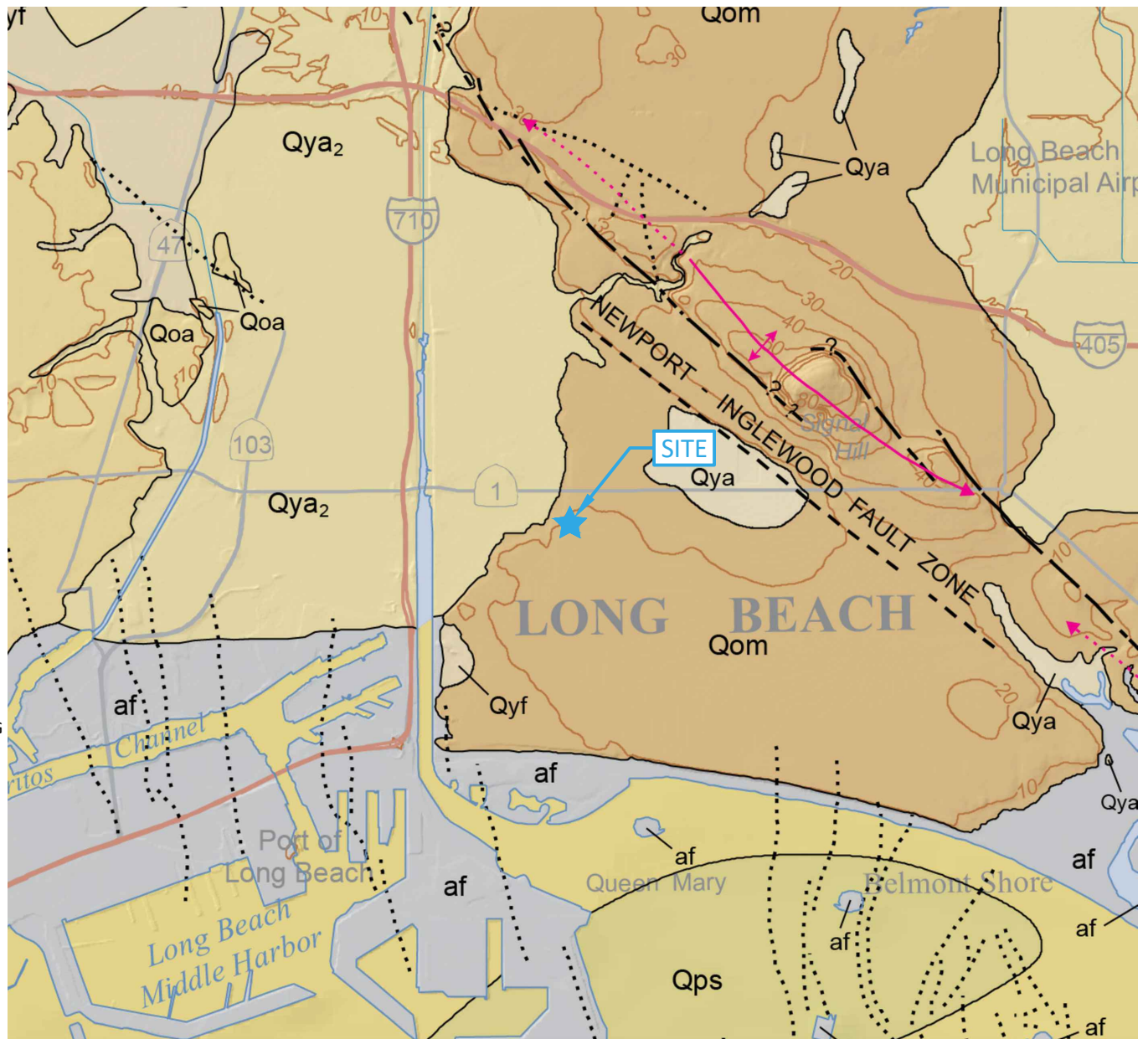
FIGURE NUMBER
3B

ABBREVIATED EXPLANATION:

af	ARTIFICIAL FILL
Qya	YOUNG ALLUVIUM, UNDIVIDED
Qya ₂	YOUNG ALLUVIUM, UNIT 2
Qoa	OLD ALLUVIUM, UNDIVIDED
Qom	OLD SHALLOW MARINE DEPOSITS ON WAVE-CUT SURFACE
Qps	PLEISTOCENE SEDIMENTARY DEPOSITS, UNDIVIDED

SYMBOL EXPLANATION:

— · — · — · — · — ·	CONTACT
$\frac{U}{D}$ — — — — —	FAULT
\longleftrightarrow (dashed line with outward arrows)	ANTICLINE
\longleftrightarrow (dashed line with inward arrows)	SYNCLINAL FOLD
\oplus	HORIZONTAL BEDDING
$\frac{35}{\text{line}}$	INCLINED BEDDING
$+$	VERTICAL BEDDING
\blacktriangle	STRIKE AND DIP
	LANDSLIDES
\curvearrowright	CREEP (OFFSHORE)
\odot	OIL AND/OR GAS SEEP



REFERENCE: GEOLOGIC MAP OF THE LONG BEACH 30'X60' QUADRANGLE, CALIFORNIA BY GREENE, KENNEDY, AND BEZORE, VER 2.0, 2016



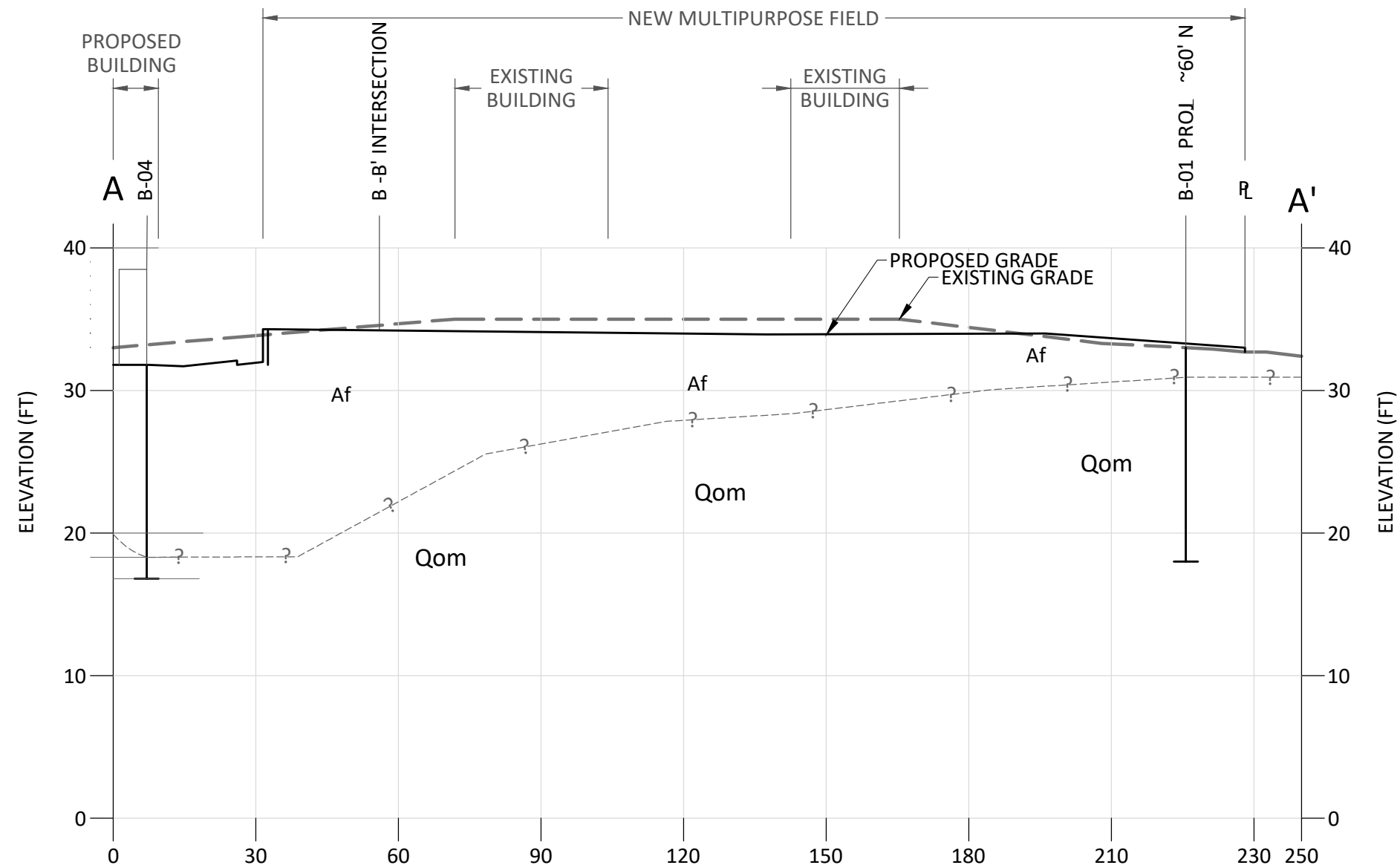
POLYTECHNIC HIGH SCHOOL
IMPROVEMENTS PROJECT

REGIONAL GEOLOGIC MAP

NV5 Beyond Engineering

PROJECT NUMBER
1400125-0009800.00

FIGURE NUMBER
4



EXPLANATION:

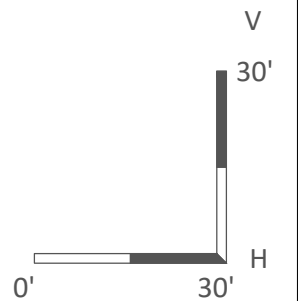
GEOLOGY

- FILL EXISTING UNDOCUMENTED FILL
- Qom OLD SHALLOW MARINE DEPOSITS ON WAVE-CUT SURFACE
- ?-?-? INTERPRETED GEOLOGIC CONTACT (QUERIED WHERE UNCERTAIN)

IMPROVEMENTS

- PROPOSED GRADE
- - - - - EXISTING GRADE

NOTE: SCALE, LOCATIONS, AND DIRECTIONS ARE APPROXIMATE.



REFERENCE: PBK ARCHITECT, SURVEY SHEETS C002, C-003, GRADING AND PAVING PLAN SHEETS C201 AND C202, AUGUST 22, 2025

POLYTECHNIC HIGH SCHOOL
IMPROVEMENTS PROJECT

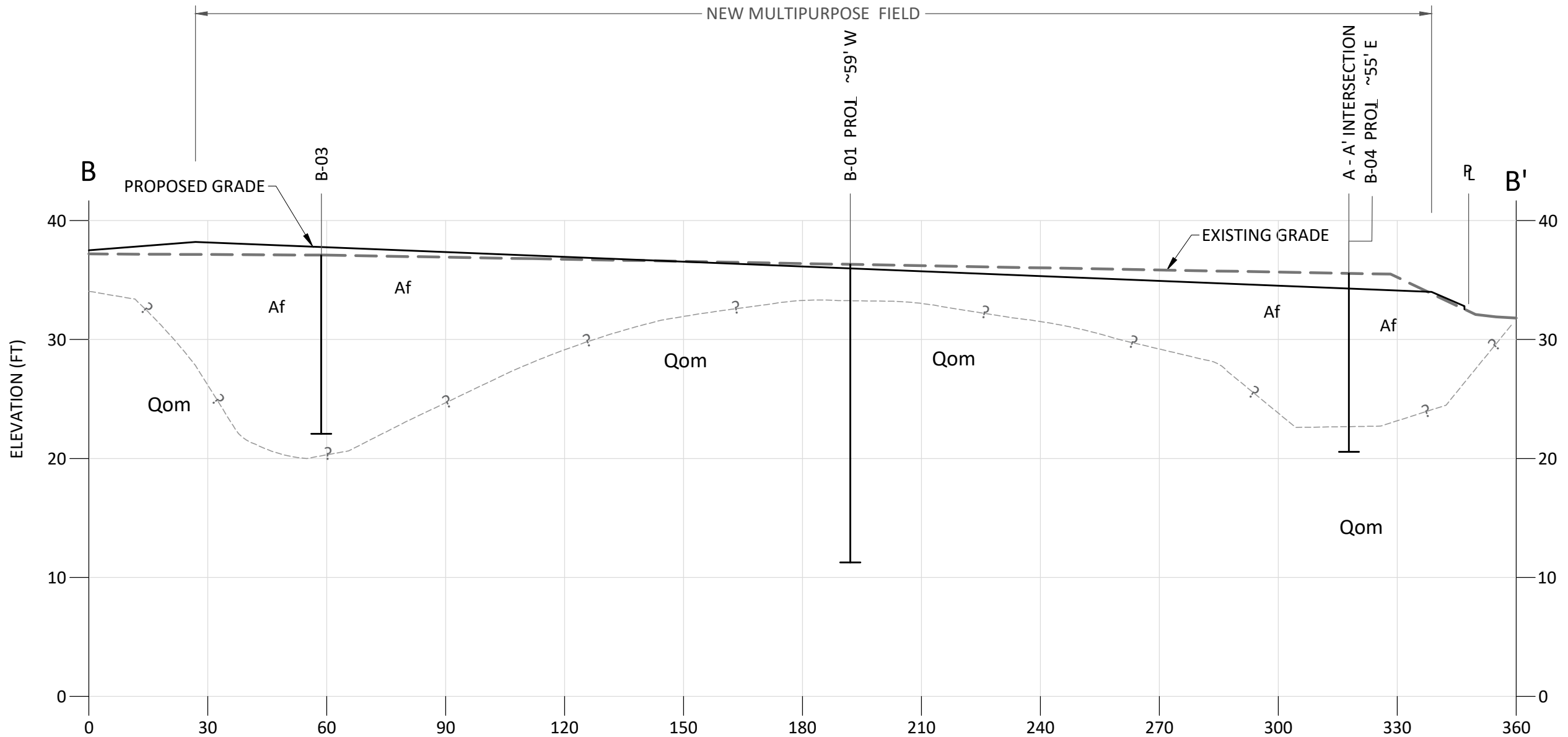
GEOLOGIC CROSS SECTION
A - A'

NV5 Beyond Engineering

PROJECT NUMBER
1400125-0009800.00

FIGURE NUMBER
5A

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EXPLANATION:

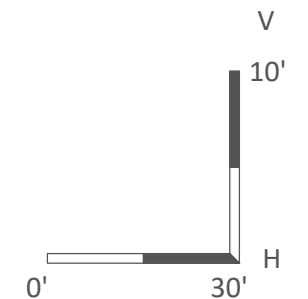
GEOLOGY

- FILL EXISTING UNDOCUMENTED FILL
- Qom OLD SHALLOW MARINE DEPOSITS ON WAVE-CUT SURFACE
- ?-?-? INTERPRETED GEOLOGIC CONTACT (QUERIED WHERE UNCERTAIN)

IMPROVEMENTS

- PROPOSED GRADE
- EXISTING GRADE

NOTE: SCALE, LOCATIONS, AND DIRECTIONS ARE APPROXIMATE.



REFERENCE: PBK ARCHITECT, SURVEY SHEETS C002, C-003, GRADING AND PAVING PLAN SHEETS C201 AND C202, AUGUST 22, 2025

POLYTECHNIC HIGH SCHOOL
IMPROVEMENTS PROJECT

GEOLOGIC CROSS SECTION
B - B'

NV5 Beyond Engineering

PROJECT NUMBER
1400125-0009800.00

FIGURE NUMBER
5B

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REFERENCE: GOOGLE EARTH, IMAGERY DATE, 02/2024
USGS, EARTHQUAKE CATALOG, ACCESSED 07/13/2022
USGS & CGS, QUATERNARY FAULT AND FOLD DATABASE, ACCESSED 7/13/2022

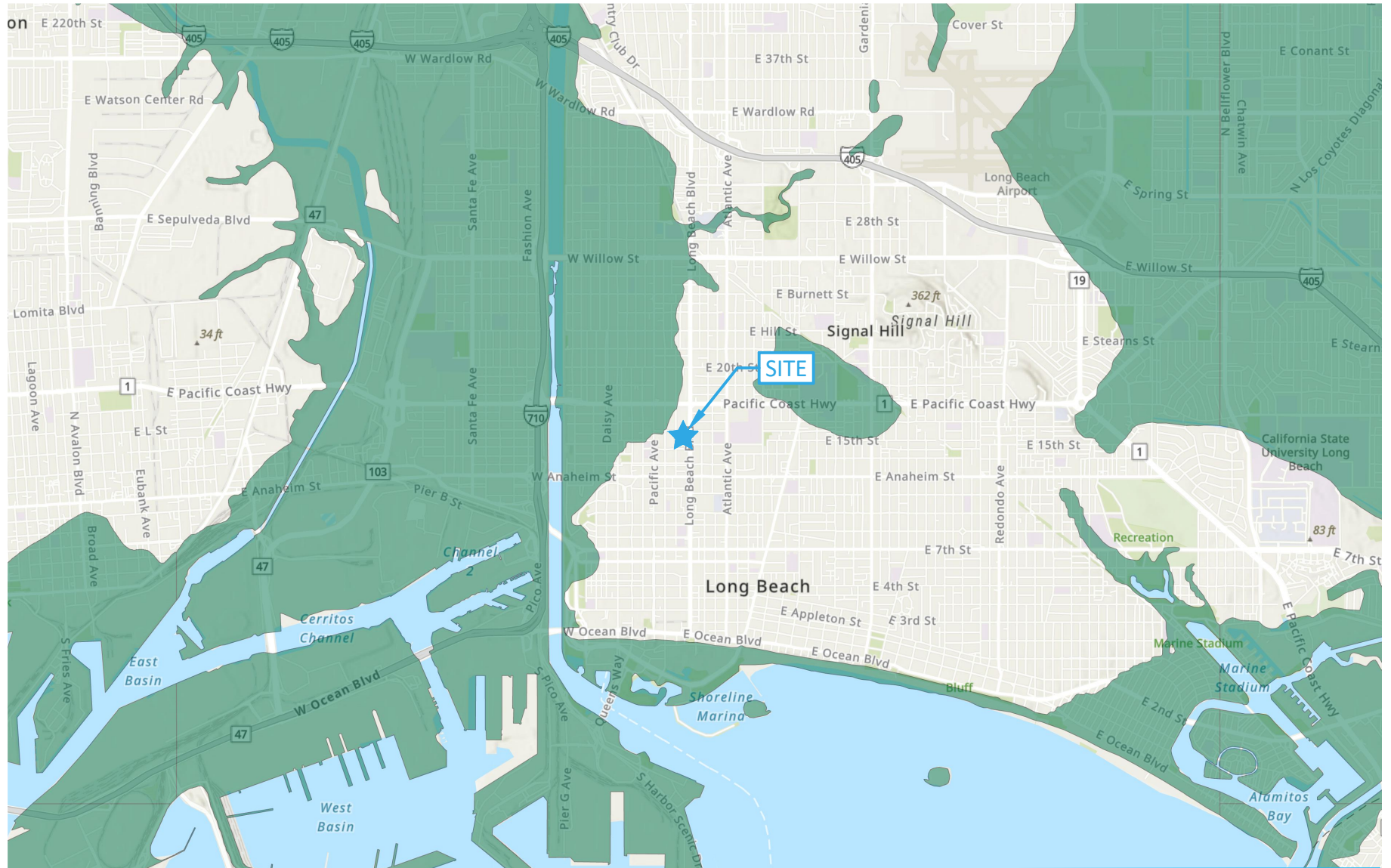


POLYTECHNIC HIGH SCHOOL IMPROVEMENTS PROJECT

REGIONAL FAULT AND SEISMICITY MAP

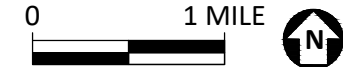
NV5 Beyond Engineering	
PROJECT NUMBER 1400125-0009800.00	FIGURE NUMBER 6

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9/25/2025

CGS Liquefaction Zones
World_Hillshade



SCALE, LOCATIONS, AND DIRECTIONS ARE APPROXIMATE.

REFERENCE: SGS LIQUEFACTION ZONES MAP, RETRIEVED 09/25/2025
FROM THE WEBSITE <https://www.arcgis.com/apps/mapviewer/index.html?layers=fcf1ebfe19be49819a12c404d74b4a7d>

POLYTECHNIC HIGH SCHOOL
IMPROVEMENTS PROJECT

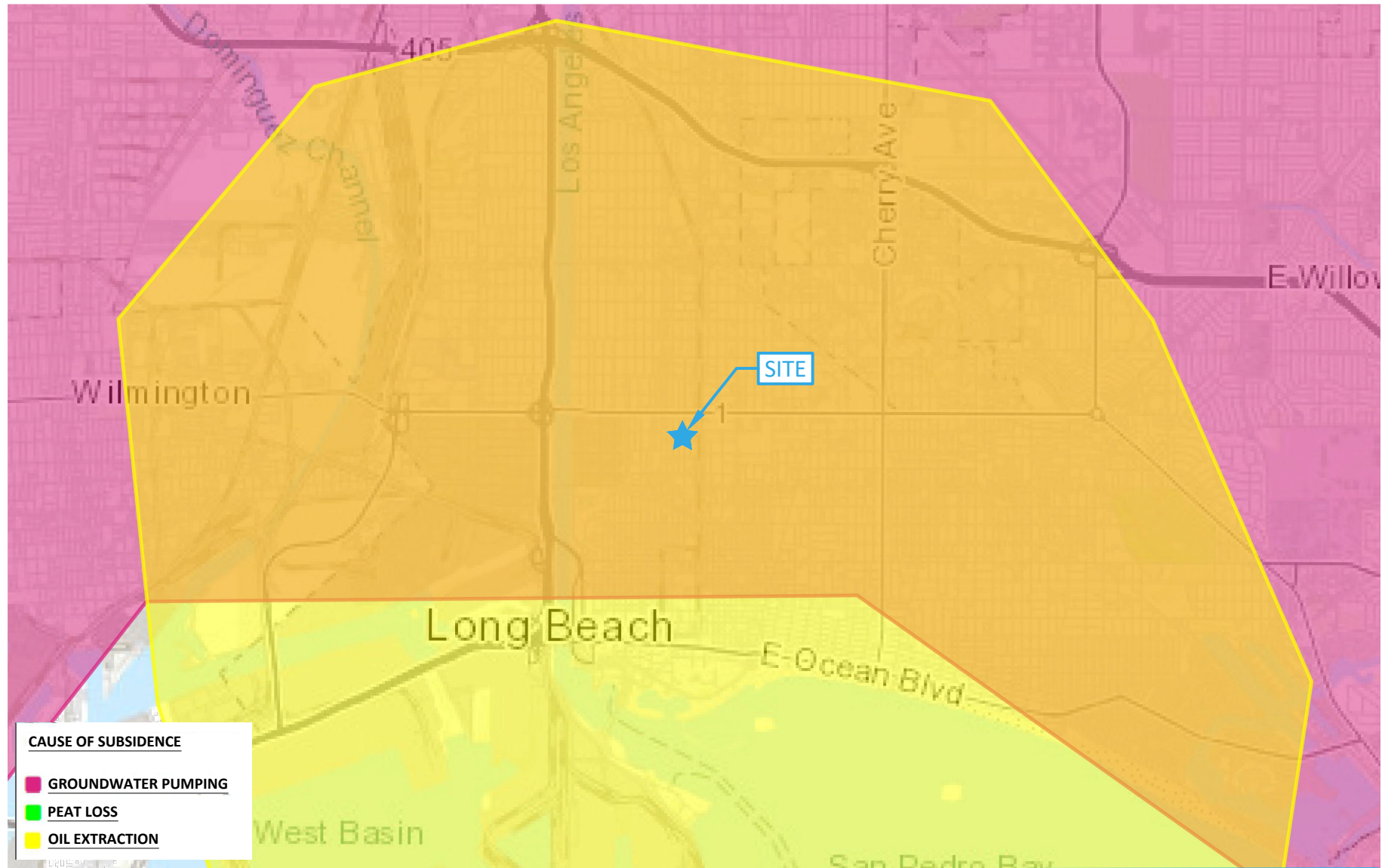
REGIONAL LIQUEFACTION HAZARD MAP

NV5 Beyond Engineering

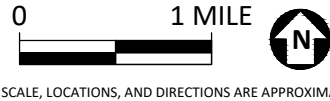
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FIGURE NUMBER
7

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REFERENCE: AREAS OF LAND SUBSIDENCE IN CALIFORNIA, RETRIEVED 09/25/2025
FROM THE WEBSITE https://ca.water.usgs.gov/land_subsidence/california-subsidence-areas.html



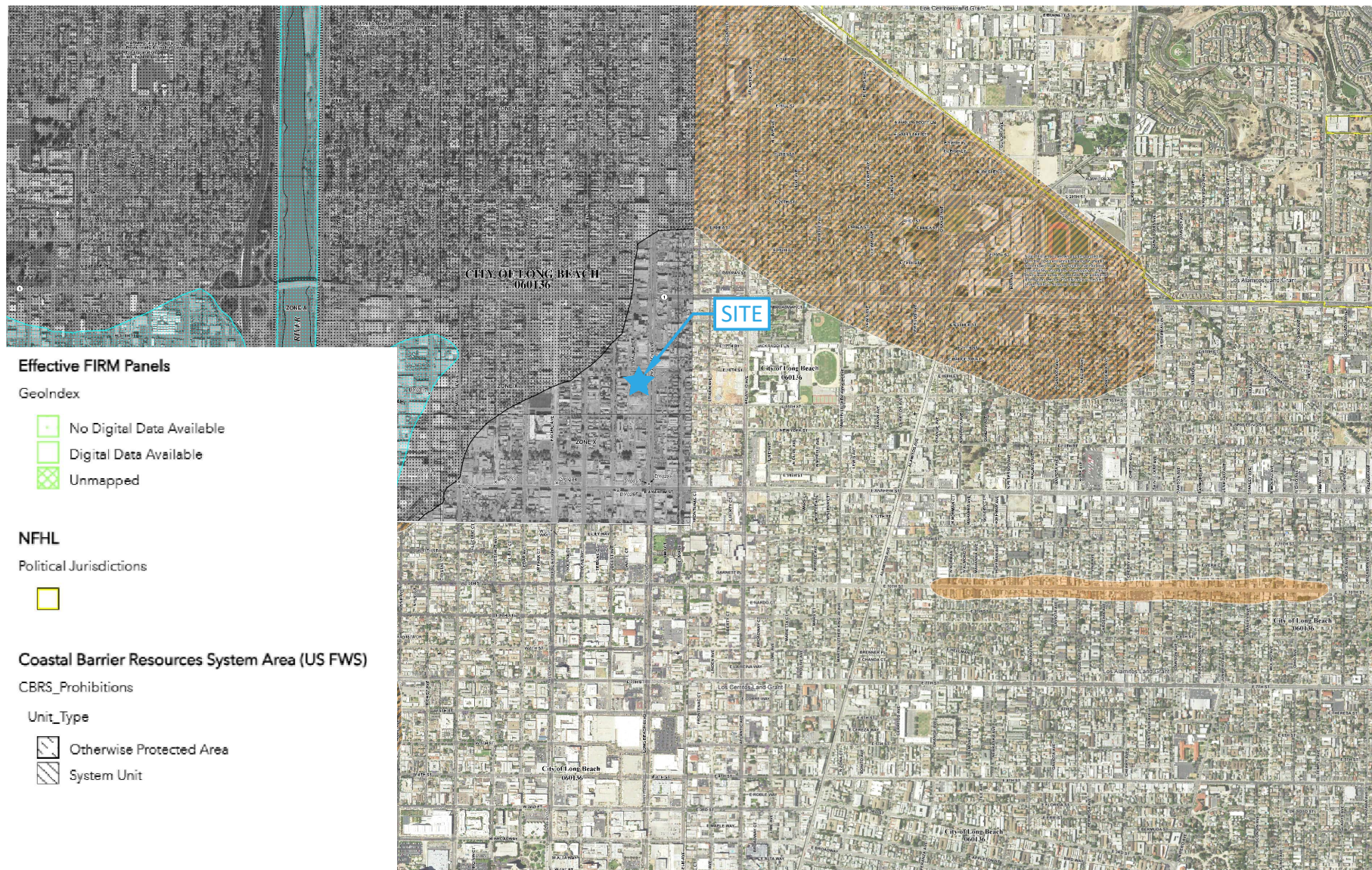
POLYTECHNIC HIGH SCHOOL
IMPROVEMENTS PROJECT

REGIONAL SUBSIDENCE HAZARD MAP

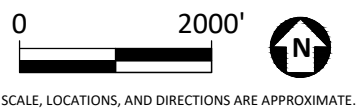
NV5 Beyond Engineering

PROJECT NUMBER
1400125-0009800.00

FIGURE NUMBER
8



REFERENCE: FEMA NATIONAL FLOOD HAZARD 06037C1962F (2008), 06037C1966G (2021), 06037C1964G (2021), 06034C1968G (2021)



POLYTECHNIC HIGH SCHOOL
IMPROVEMENTS PROJECT

FEMA FLOOD HAZARD MAP

NV5 Beyond Engineering

PROJECT NUMBER 1400125-0009800.00	FIGURE NUMBER 9
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APPENDIX A

Exploratory Boring Logs

Sampling Methods

Representative bulk-disturbed and relatively undisturbed drive samples were retrieved during exploratory drilling at selected depths appropriate to our investigation. The samples were labeled in the field and transported to our 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 SPT sampler was driven with a 140-pound hammer (using a 30-inches drop) in general accordance with ASTM D1586. The sampler has external and internal diameters of 2.0 and 1.4 inches, respectively. The SPT sampler was driven 18 inches (or to refusal) and the number of blows per 6-inch interval was recorded on the field boring logs. The uncorrected N-value (numbers of blows for the last two 6-inch intervals or any portion thereof) is presented on the borings logs. The soil samples retrieved from the STP sampler were logged and transported to the laboratory for classification 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) and augmented with ASTM Standard Testing for Soil (see Appendix B). The apparent density of materials was derived from blow counts (ASTM D1586). 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 corrected “N-value” for hammer efficiency (N60) 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

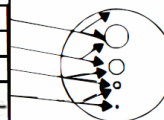
- The report and graphics key are an integral part of these logs. All data and interpretations in this log are subject to the explanations and limitations stated in the report.
- Lines separating strata on the logs represent approximate boundaries only. Actual transitions may be gradual or differ from 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) designations presented on the logs were based on visual classification in the field and were modified where appropriate based on gradation and index 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, i.e., 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)

GRAVELS (More than half of coarse fraction is larger than the #200 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
				GP-GM	POORLY-GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE FINES
		Cu < 4 and/or 1 > Cc > 3		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
COARSE GRAINED SOILS (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, SILTY CLAYS, LEAN CLAYS
	SILTS AND CLAYS (Liquid Limit greater than 50)			OL	ORGANIC SILTS & ORGANIC SILTY CLAYS OF LOW PLASTICITY
				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

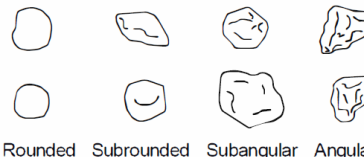
GRAIN SIZE

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	0.075 - 0.19 in. (2 - 4.75 mm.)	Rock salt-sized to pea-sized
	medium #40 - #10	0.017 - 0.075 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.

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

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

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

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

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

SPT CAL

SPT HAMMER ENERGY MEASUREMENTS

Prepared by;

SPT CAL
5512 Belem Dr
Chino Hills, CA 91709

909-730-2161
bc@sptcal.com

Prepared for;
24-7 Drilling
1840 Commerce St
Unit B
Norco, CA 92860

(760) 250-8320

Date: 08/10/24

Project Title: 24-7 Drilling Norco

Project Description: Diedrich D-70 Rig 619

Energy Transfer Ratio = 83.4% @ 41.4 blows per minute

Testing was performed on August 10, 2024 in Norco, California

Hammer Energy Measurements performed per ASTM D4633 using an approved and calibrated SPT Analyzer from Pile Dynamics, Inc. meeting the criteria of ASTM D4633-05 and per the process defined in ASTM D4633-05, The process and equipment requirements followed per ASTM D4633-05 meet the criteria of ASTM D4633-16.

PRESENTATION OF SPT ANALYZER TEST DATA

1. Introduction

This report presents the results of SPT Hammer Energy Measurements recorded with an SPT Analyzer from Pile Dynamics carried out on August 10, 2024 in Norco, California.

2. Field Equipment and Procedures

The drill used is a Deidrich D-70, serial number 022. The operator was Steve of 24-7 Drilling.. It has an attached Deidrich Automatic Hammer. The Diedrich Automatic Hammer uses a 140 lb. weight dropped 30" on to an anvil above the bore hole. The drill rod connects the anvil to a split spoon type soil sampler inside an 8" o.d. hollow stem auger at the designated sample depth. After a seeding blow the sampler is driven 18". The number of blows required to penetrate the last 12" is referred to as the "N value", which is related to soil strength.

The first recording was taken at 5' below ground surface and then every 5' to final recording at 25'.

3. Instrumentation

An SPT Analyzer from Pile Dynamics was used to record and the process the data. The raw data was stored directly in the SPT Analyzer computer with subsequent analysis in the office with PDA-W and PDIPlot software. The measurements and analysis were conducted in general accordance with ASTM D4945 and ASTM D6066 test standards.

The SPT Analyzer is fully compliant with the minimum digital sampling frequency requirements of ASTM D4633-05 (50 kHz) and EN ISO 22476-3:2005 (100 kHz), as well as with the low pass filter, (cutoff frequency of 5000 Hz instead of 3000 Hz) requirements of ASTM D4633-05. All equipment and analysis also conform to ASTM D6066.

A 2' instrumented section of NWJ rod, with two sets of accelerometers and strain transducers mounted on opposite sides of the drill rod, was placed below the anvil. It measured strain and acceleration of every hammer blow. The SPT Analyzer then calculates the amount of energy transferred to the rod by force and velocity measurements.



4. Observations

The drill rig motor is diesel fueled. The drill and sample equipment looked to be well operated and maintained.

5. Results

Results from the SPT Hammer Energy Measurements are summarized on the following pages. It shows the Energy Transfer Ratio (ETR) at each sampling depth. ETR is the ratio of the measured maximum transferred energy to rated energy of the hammer which is the product of the weight of the hammer times the height of the fall. $140 \text{ lb} \times 30'' = 4200 \text{ lb-in} = 0.350 \text{ kip-ft}$. $N_{60} = (ETR/60)N$. The reported average value is a weighted average based on the number of blows at each sample interval.

6. Recommendations

Recalibration of the auto hammer is recommended annually. Recalibration is also recommended for change of operator, engine modifications and repair, hydraulic system modifications and repair, auto hammer adjustments and repair and anything else that may affect speed, function and weight of the auto-hammer

Energy Transfer Ratio = 83.4% @ 41.4 blows per minute

$N_{60} = (ETR/60)N$

Depth	ETR%	BPM
5	81.0	40.3
10	83.3	43.0
15	81.5	39.7
20	83.9	39.8
25	87.3	44.0
Average	83.4	41.4

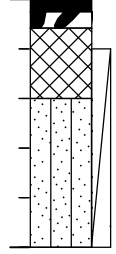
If you have any questions please do not hesitate to call or email.

Thank you,

Brian Serl
Calibration Engineer
SPT CAL
909-730-2161
bc@sptcal.com

Date		Started: 9/17/25		Project Number 1400125-0009800				Project Poly HS Improvements Project				Boring No. B-01	
		Completed: 9/17/25						Logged By: M. Rastegar		Reviewed By: P. Cunningham			
		Hammer Efficiency: 83.4		Rig Type: Diedrich D-70								Surface Elevation: 38.0'	
Latitude: 33.786819°				Longitude: -118.189645°				Location: Northeast region of PAAL Academy campus on lawn					
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.	Groundwater				
									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				
									Depth (ft)	Hour	Date		
Visual Classification													
0													
0.3' Topsoil: approximately 3" thick El. 37.8'													
SM Artificial Fill (Af): Silty SAND (SM); light brown; moist; fine to coarse grained sand El. 36.0'													
2.0'													
CL Quaternary Old Shallow Marine Deposits (Qom): Sandy Lean CLAY (CL); brown; moist													
5													
CAL- 1 8 15 20													
6.5' El. 31.5'													
SM Silty SAND (SM); medium dense; brown; moist													
10													
10.0' El. 28.0'													
ML Sandy SILT (ML); hard; olive brown; moist													
15													
CAL- 1 13 15 16 11.9 129.9 Atterberg Limits Direct shear													
16.5' El. 21.5'													
SM Silty SAND (SM); medium dense; yellowish brown; fine grained sand; moist													
20													
20.0' El. 18.0'													
SP Poorly graded SAND (SP); dense; olive brown; moist													
25													
CAL- 2 11 18 26 5.0 102.3													
25.0' El. 13.0'													
Medium dense													

Notes: Drilled using a 8" O.D. Hollow-Stem Auger. Boring terminated at 25' bgs.
Refusal not encountered. Groundwater not encountered. Spoils Drummed.
Backfilled with grout. Patched to match existng.

Date	Started: 9/17/25		Project Number 1400125-0009800		Project Poly HS Improvements Project		Boring No. B-02									
	Completed: 9/17/25															
	Hammer Efficiency: 83.4		Rig Type: Diedrich D-70		Logged By: M. Rastegar		Reviewed By: P. Cunningham									
Latitude: 33.786639°			Longitude: -118.190008°			Surface Elevation: 37.0'										
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.								
									Location: Center of PAAL Academy on recreation courts							
									Sample Type		Groundwater					
									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		Depth (ft)	Hour	Date			
Visual Classification																
0																
			G- 1		8.7		Sieve									
5																
<p>Notes: Drilled using a 8" O.D. Hollow-Stem Auger. Boring terminated at 5' bgs. Refusal not encountered. Groundwater not encountered. Spoils Drummed. Backfilled with grout. Patched to match existing</p>																

Date	Started: 9/17/25		Project Number 1400125-0009800		Project Poly HS Improvements Project		Boring No. B-03					
	Completed: 9/17/25											
	Hammer Efficiency: 83.4		Rig Type: Diedrich D-70		Logged By: M. Rastegar		Reviewed By: P. Cunningham					
Latitude: 33.786275°			Longitude: -118.190201°			Surface Elevation: 41.0'						
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.	Location: South region of PAAL Academy campus			
									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
										Visual Classification		

Notes: Drilled using a 8" O.D. Hollow-Stem Auger. Boring terminated at 15' bgs.
 Refusal not encountered. Groundwater not encountered. Spoils Drummed.
 Backfilled with grout. Patched to match existing

Date	Started: 9/17/25			Project Number 1400125-0009800				Project Poly HS Improvements Project				Boring No. B-04		
	Completed: 9/17/25													
	Hammer Efficiency: 83.4			Rig Type: Diedrich D-70				Logged By: M. Rastegar			Reviewed By: P. Cunningham			
Latitude: 33.786980°				Longitude: -118.190379°				Surface Elevation: 35.0'						
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.	Location: PAAL Academy Campus Driveway on N. Palmer Ct.					
									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
									Visual Classification					

0		G- 1	16.2	Sieve Expansion Index	CL	0.3' 0.6'	Aphalt: approximately 3" thick	El. 34.8'		
							Base: approximately 4" thick	El. 34.4'		
							Fill (AF): Lean CLAY w/ih Sand (CL); brown; moist			
5		CAL- 1	2 3 3		SC	5.0'	Clayey SAND (SC); loose; brown; moist	El. 30.0'		
10		SPT- 1	8 13 4		ML	10.0'	Sandy SILT (ML); very stiff; brown; moist	El. 25.0'		
15		CAL- 2	16 22 25	3.3	103.3	SP-SM	13.5' 15.0'	Quaternary Old Shallow Marine Deposits (Qom): Poorly graded SAND w/ silt (SP-SM); dense; olive brown; dry	El. 21.5'	

Notes: Drilled using a 8" O.D. Hollow-Stem Auger. Boring terminated at 15' bgs.
Refusal not encountered. Groundwater not encountered. Spoils Drummed.
Backfilled with grout. Patched to match existng

Date	Started: 9/17/25		Project Number 1400125-0009800		Project Poly HS Improvements Project		Boring No. B-05									
	Completed: 9/17/25															
	Hammer Efficiency: 83.4		Rig Type: Diedrich D-70		Logged By: M. Rastegar		Reviewed By: P. Cunningham									
Latitude: 33.787022°			Longitude: -118.190725°			Surface Elevation: 34.0'										
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.								
									Location: Staff Park lot of PAAL Academy on N Palmer Ct.							
									Sample Type		Groundwater					
									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		Depth (ft)	Hour	Date			
Visual Classification																
<div> <div> 0 </div> <div> </div> <div> <div> AC 0.3' AB 0.6' </div> <div> Aphalt: approximately 3" thick Base: approximately 4" thick Fill (AF): Sandy Lean CLAY (CL); brown; moist </div> <div> CL </div> <div> 5.0' </div> <div> El. 33.8' El. 33.4' El. 29.0' </div> </div> </div>																
Notes: Drilled using a 8" O.D. Hollow-Stem Auger. Boring terminated at 5' bgs. Refusal not encountered. Groundwater not encountered. Spoils Drummed. Backfilled with grout. Patched to match existng																

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 D2216 and D2937 laboratory test methods. The method involves 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 density tests are presented in the following table and on the exploratory borings logs in Appendix A.

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

Sample Location	Moisture Content (percent)	Dry Density (pounds per cubic foot)
B-01 @ 10' – 11.5'	18.3	-
B-01 @ 16.0' – 16.5'	11.9	129.9
B-02 @ 24.5' – 25.0'	5.0	102.3
B-02 @ 1.0' – 5.0'	8.7	-
B-03 @ 5.0' – 6.5'	14.1	-
B-03 @ 11.0' – 11.5'	13.6	131.2
B-04 @ 1.0' – 5.0'	16.2	-
B-04 @ 14.5' – 15.0'	3.3	103.3
B-05 @ 1.0' – 5.0'	12.9	-

Classification

Soils were visually and texturally classified in general accordance with the Unified Soil Classification System (ASTM D2487). 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 version of ASTM D6913. These test results were utilized in evaluating the soil classifications in accordance with the Unified Soil Classification System. In addition, material finer than 75 µm (No. 200) sieve was performed in accordance with ASTM C117. Particle size distribution test results are presented on the laboratory test sheets attached in this appendix.

Atterberg Limits

Atterberg limits tests were performed in general accordance with ASTM D4318 on selected soil samples. This test is useful in classification of the soils. Test results are attached in this appendix and summarized below.

RESULTS OF ATTERBERG LIMITS TESTS (ASTM D4318)

Location	USCS Group Name	Liquid Limit	Plastic Limit	Plasticity Index
B-01 @ 15.0' – 16.5'	SILT (ML)	NP	NP	NP

Expansion Index Tests

Expansion index tests were performed on samples of the on-site soils. The tests were performed in general accordance with ASTM D4829. The result of the tests are presented below and attached in this appendix.

RESULTS OF EXPANSION INDEX TESTS (ASTM D4829)

Location	Material Type	Initial Moisture Content, %	Final Moisture Content, %	Dry Density, pcf	Initial Saturation, %	Expansion Index	Potential Expansion
B-04 @ 1.0' – 5.0'	Lean CLAY (CL)	10.0	20.5	110.6	51.6	41	Low

Direct Shear

Direct shear testing was performed on a representative relatively undisturbed sample in general 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 TESTS (ASTM D3080)

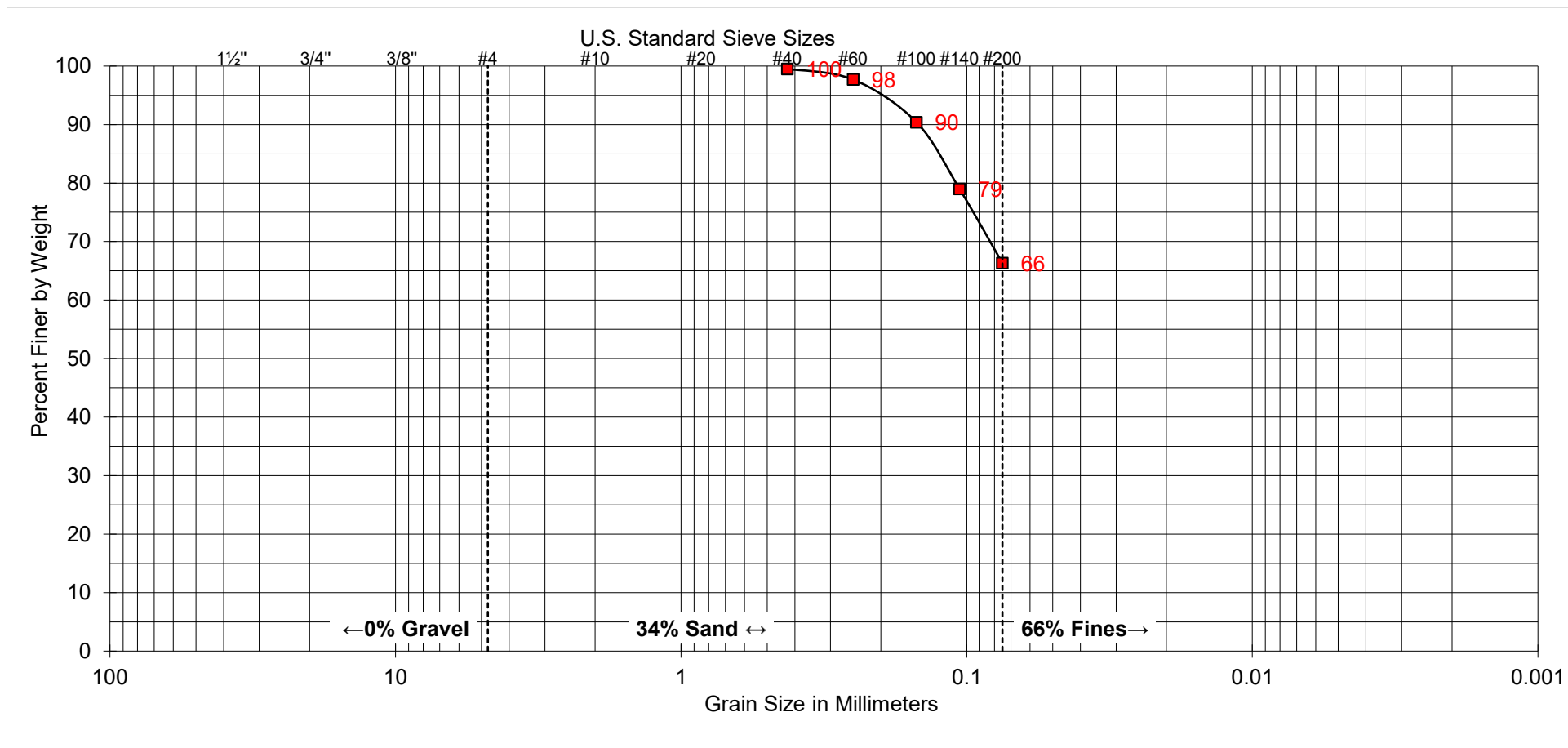
Location	USCS Classification	Peak Friction (degrees)	Ultimate Friction (degrees)	Peak Cohesion (psf)	Ultimate Cohesion (psf)	Notes
B-01 @ 15.0' 16.5'	Silty SAND (SM)	35	35	500	400	Saturated prior to testing

Soil Corrosivity Test

Water soluble sulfate & chloride, resistivity and pH testing was 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	B-01 @ 6.0' - 6.5'
pH	8.0
Minimum Resistivity (Ohm-cm)	1320
Water Soluble Sulfates (ppm)	400
Water Soluble Chlorides (ppm)	200
Material Type	Silty SAND (SM)

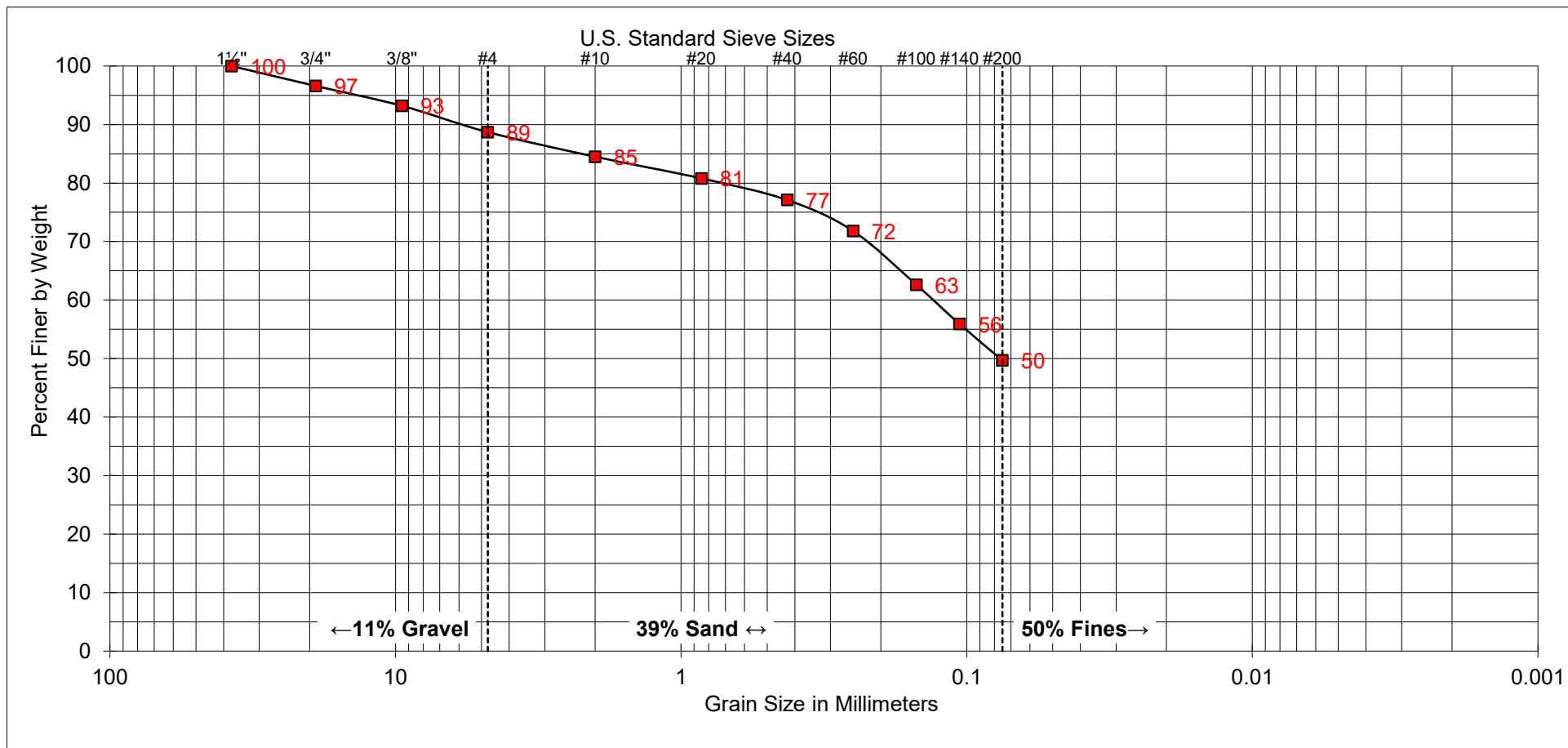


COARSE	FINE	COARSE	MEDIUM	FINE	SILT AND CLAY
GRAVEL		SAND			

SAMPLE	
EXPLORATION ID:	B-1
SAMPLE DEPTH:	6' - 6½

UNIFIED SOIL CLASSIFICATION:	CL
DESCRIPTION:	SANDY LEAN CLAY

ATTERBERG LIMITS	
LIQUID LIMIT:	--
PLASTIC LIMIT:	--
PLASTICITY INDEX:	--



COARSE	FINE	COARSE	MEDIUM	FINE	SILT AND CLAY
GRAVEL		SAND			

SAMPLE	
EXPLORATION ID:	B-2
SAMPLE DEPTH:	1' - 5'

UNIFIED SOIL CLASSIFICATION:	SM
DESCRIPTION:	SILTY SAND

ATTERBERG LIMITS	
LIQUID LIMIT:	--
PLASTIC LIMIT:	--
PLASTICITY INDEX:	--

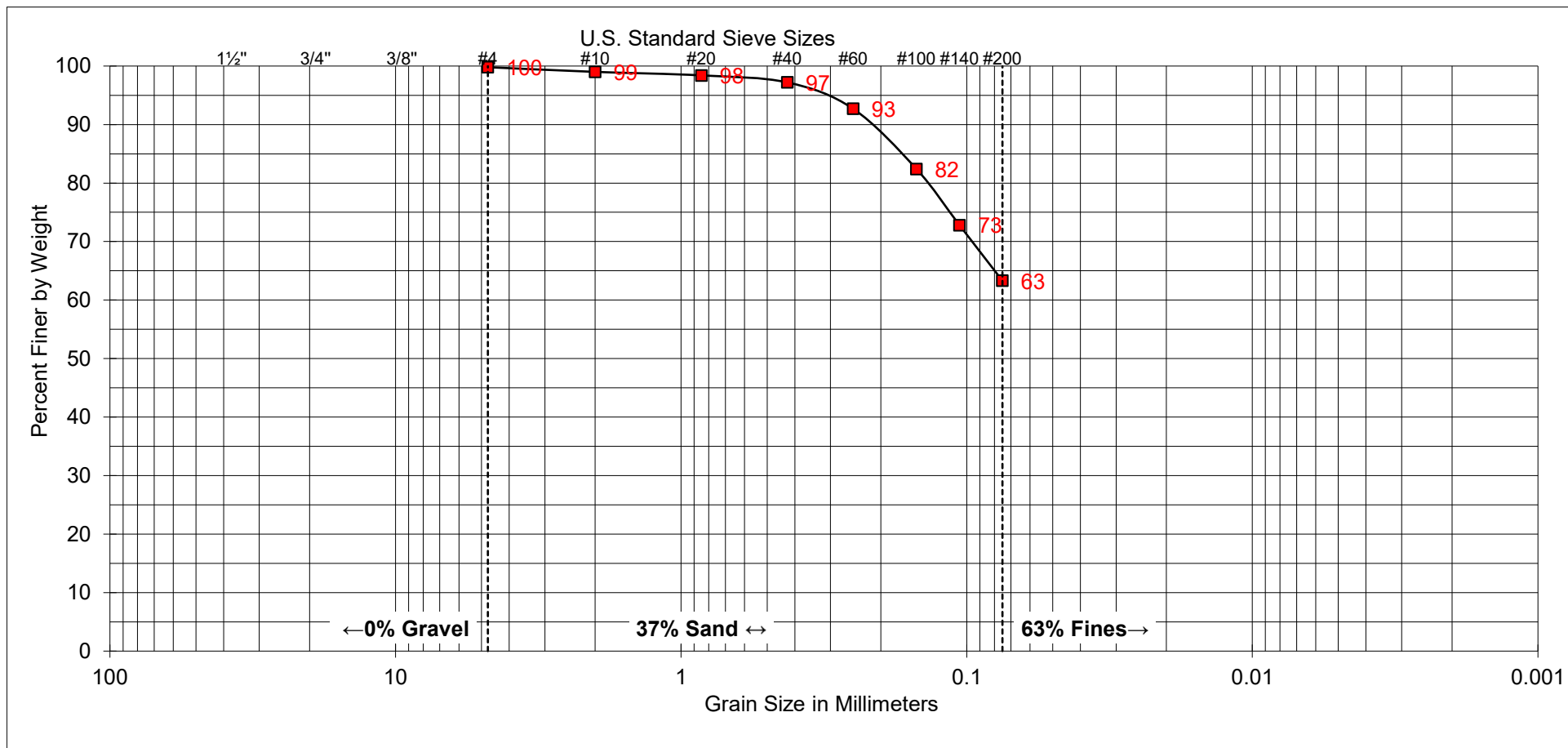


GROUP DELTA

SOIL CLASSIFICATION

Project No. 9800

FIGURE B-1.1

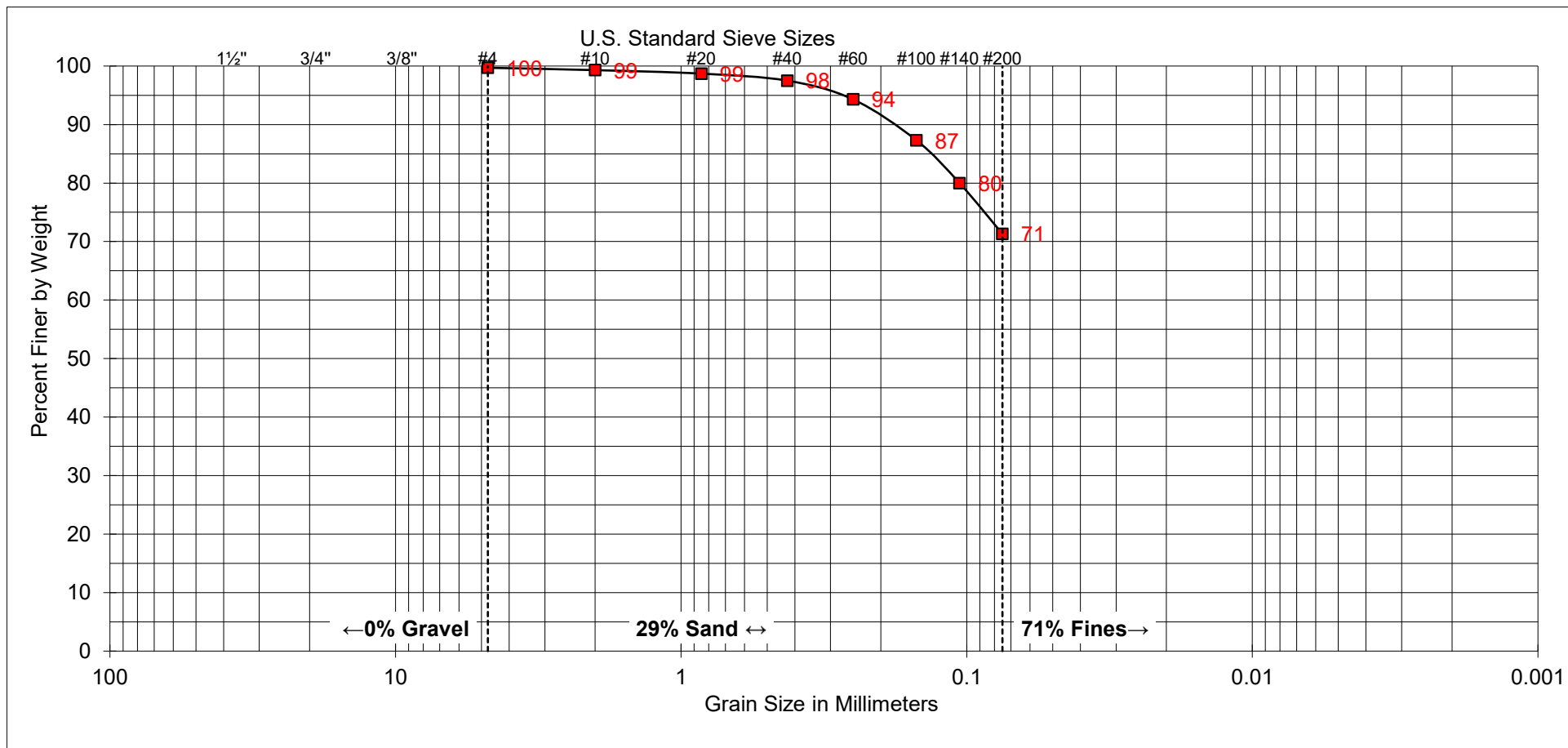


COARSE	FINE	COARSE	MEDIUM	FINE	SILT AND CLAY
GRAVEL		SAND			

SAMPLE	
EXPLORATION ID:	B-3
SAMPLE DEPTH:	1' - 5'

UNIFIED SOIL CLASSIFICATION:	CL
DESCRIPTION:	SANDY LEAN CLAY

ATTERBERG LIMITS	
LIQUID LIMIT:	--
PLASTIC LIMIT:	--
PLASTICITY INDEX:	--



COARSE	FINE	COARSE	MEDIUM	FINE	SILT AND CLAY
GRAVEL		SAND			

SAMPLE	
EXPLORATION ID:	B-4
SAMPLE DEPTH:	1' - 5'

UNIFIED SOIL CLASSIFICATION:	CL
DESCRIPTION:	LEAN CLAY WITH SAND

ATTERBERG LIMITS	
LIQUID LIMIT:	--
PLASTIC LIMIT:	--
PLASTICITY INDEX:	--



GROUP DELTA CONSULTANTS, INC.
ENGINEERS AND GEOLOGISTS
9245 ACTIVITY ROAD, SUITE 103
SAN DIEGO, CALIFORNIA 92126

STANDARD METHOD FOR ATTERBERG LIMITS ASTM D4318

REVISION 0, DATED 1/31/15

Project Name: Poly High School
Project No.: 1400125-0009800
Sample No.: B-1
Sample Location: 15' - 16½

Tested By: J. Krehbiel Date Tested 09/23/25
Data Input By: J. Krehbiel Date: 09/23/25
Checked By: Date:

TEST NO.	PLASTIC LIMIT		LIQUID LIMIT			
	1	2	1	2	3	4
Number of Blows [N]						
Wet Wt. of Soil + Cont. (gm.)						
Dry Wt. of Soil + Cont. (gm.)						
Wt. of Container (gm.)						
Moisture Content (%) [Wn]						

LIQUID LIMIT
PLASTIC LIMIT
PLASTICITY INDEX

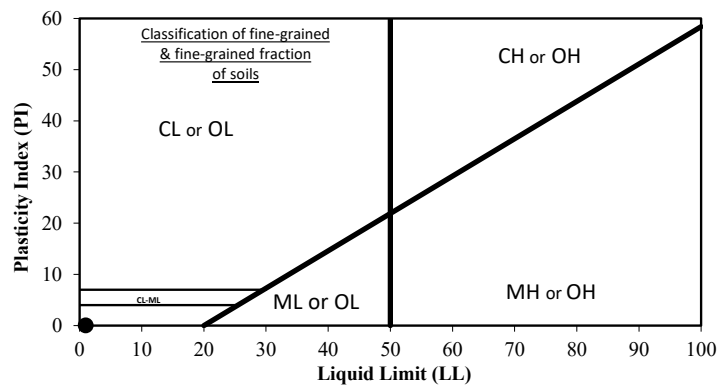
PI at "A" - Line = $0.73(LL-20)$ =

One - Point Liquid Limit Calculation

NP
NP
NP

--

$$LL = W_n(N/25)^{0.7421}$$



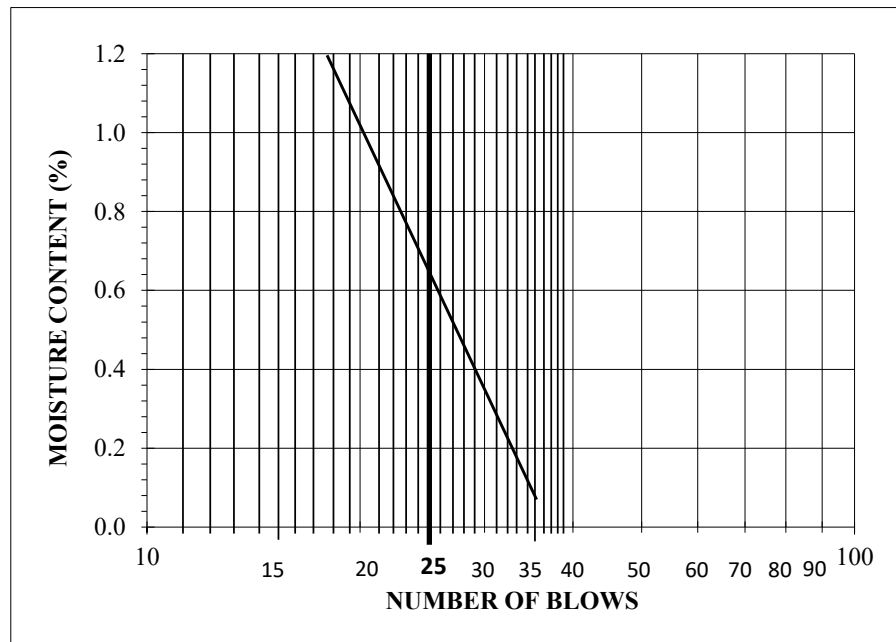
PROCEDURES USED

☐ Wet Preparation
Multipoint Wet Preparation

☒ Dry Preparation
Multipoint Dry Preparation

☒ Procedure A
Multipoint Test

☐ Procedure B
One-point Test





GROUP DELTA CONSULTANTS, INC.
ENGINEERS AND GEOLOGISTS
9245 ACTIVITY ROAD, SUITE 103
SAN DIEGO, CALIFORNIA 92126

STANDARD TEST METHOD FOR EXPANSION INDEX (ASTM D4829)

REV.1, DATED 1/31/15

PROJECT: Poly High School SAMPLE NUMBER: B-4 @ 1' - 5'
PROJECT NO.: 1400125-0009800 SAMPLE DESCRIPTION: Brown Lean Clay with Sand (CL)
TESTED BY: J. Estes DATE TESTED: 9/23/25 CHECKED BY: _____ SAMPLED BY: _____
LOCATION _____ % COARSE: _____ Page _____ of _____

MOISTURE CONTENT

TRIAL NO.

NO. 1

NO. 2

NO. 3

WET SOIL WEIGHT

477.7

g

DRY SOIL WEIGHT

434.2

g

A MOISTURE (((WET - DRY) / DRY) X 100)

10.0%

%

RING PREPARATION

B WET WEIGHT OF SOIL AND RING

604.6

g

C RING WEIGHT

201.3

g

D WET WEIGHT OF SOIL (B - C)

403.3

g

E DRY WEIGHT OF SOIL (D / ((A / 100) + 1))

366.6

g

F DRY DENSITY OF SOIL (E * 0.3016)

110.6

g

G CALCULATE (2.7 * A * F)

2986.2

H CALCULATE (168.5 - F)

57.9

J SAMPLE SATURATION (G / H)

51.6%

%

DIAL READINGS

K INITIAL SETUP READING

0.200 in

L 10 MINUTE DRY READING

0.200 in

M 24 HOUR WET READING

0.241 in

N EXPANSION INDEX ((M - L) * 1000)

41 EI

Remarks (if any) _____

FINAL MOISTURE CONTENT

O WET WEIGHT OF SOIL AND RING

636.4

g

P DRY WEIGHT OF SOIL AND RING

562.4

g

Q WEIGHT OF WATER (O - P)

74

g

R DRY WEIGHT OF SOIL (P - C)

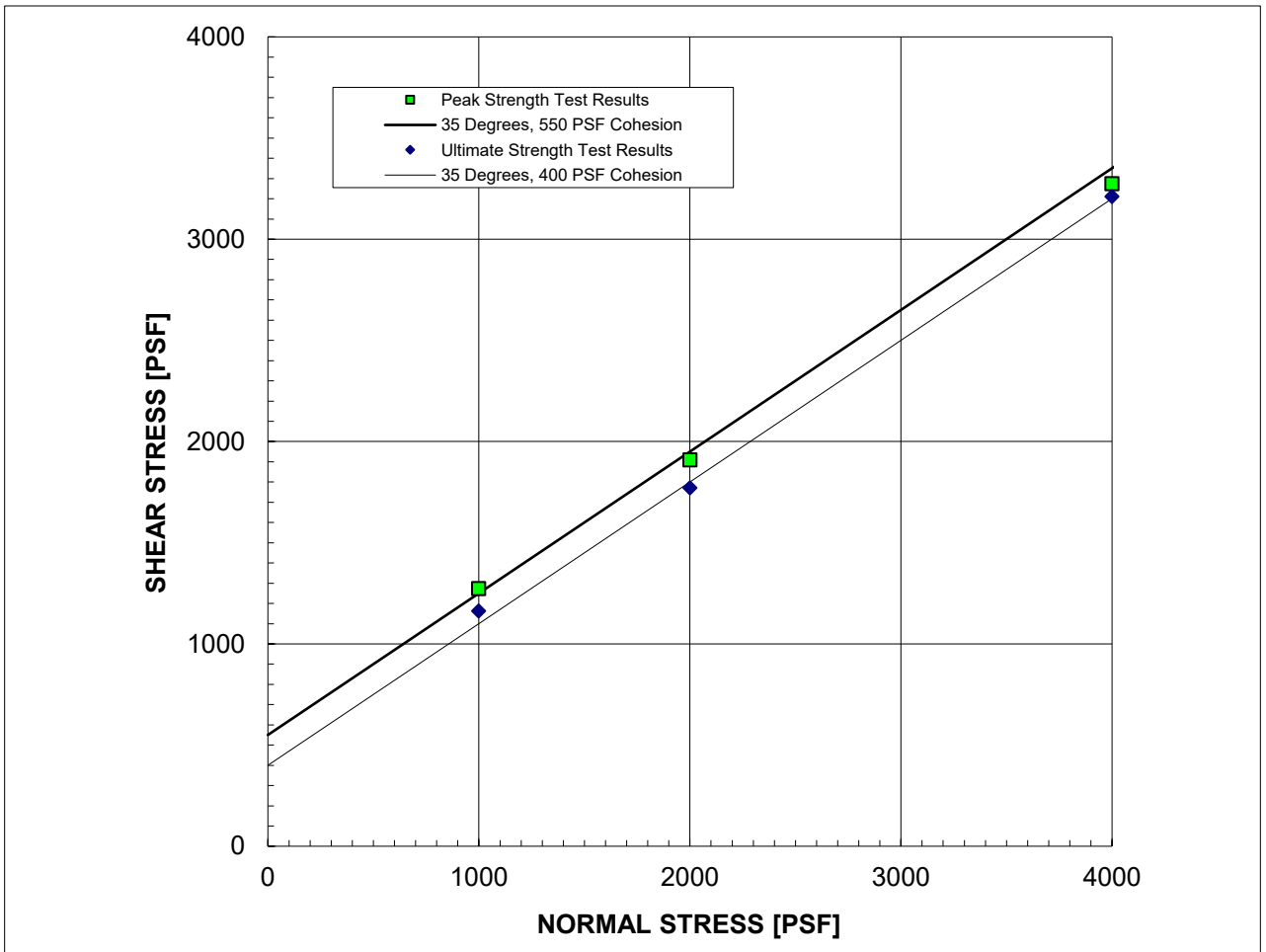
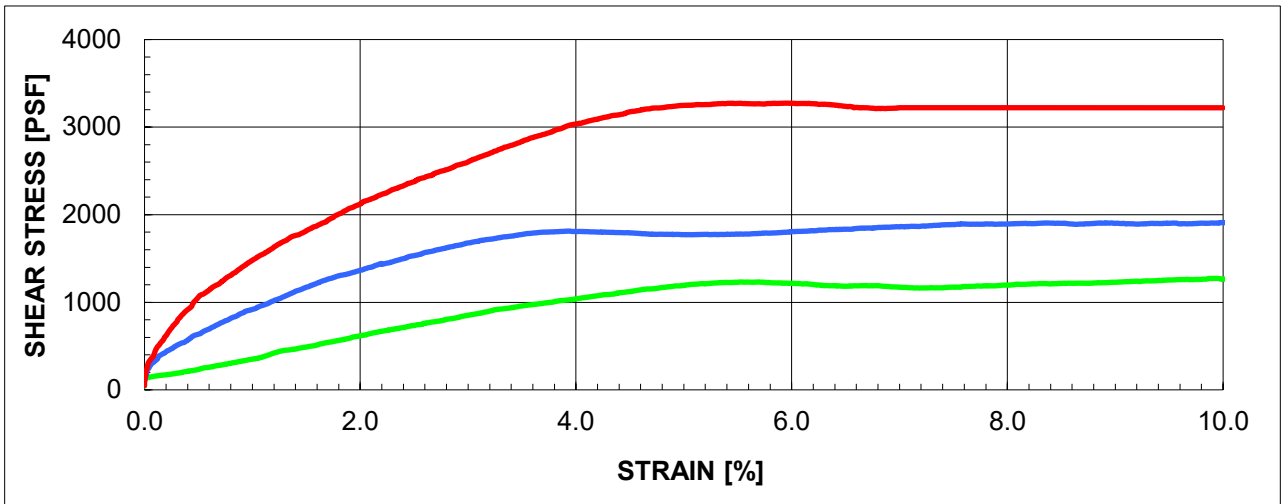
361.1

g

S MOISTURE CONTENT ((Q/R) * 100)

20.5%

%



SAMPLE: B-1 @ 15' - 16½

Description:

Yellowish brown Silty Sand (SM)

PEAK

ϕ'

35 °

C'

550 PSF

ULTIMATE

35 °

400 PSF

IN-SITU

γ_d

106.5 PCF

w_c

11.9 %

AS-TESTED

106.5 PCF

19.6 %

STRAIN RATE: 0.0030 IN/MIN

(Sample was consolidated and drained)



GROUP DELTA

DIRECT SHEAR TEST RESULTS

Project No. 9800

CORROSIVITY TEST RESULTS (ASTM D516, CTM 643)

[illegible]

SULFATE CONTENT (%)	SULFATE EXPOSURE	CEMENT TYPE
0.00 to 0.10	Negligible	--
0.10 to 0.20	Moderate	II, IP(MS), IS(MS)
0.20 to 2.00	Severe	V
Above 2.00	Very Severe	V plus pozzolan

SOIL RESISTIVITY (OHM-CM)	GENERAL DEGREE OF CORROSIVITY TO FERROUS METALS
0 to 1,000	Very Corrosive
1,000 to 2,000	Corrosive
2,000 to 5,000	Moderately Corrosive
5,000 to 10,000	Mildly Corrosive
Above 10,000	Slightly Corrosive

CHLORIDE (Cl) CONTENT (%)	GENERAL DEGREE OF CORROSIVITY TO METALS
0.00 to 0.03	Negligible
0.03 to 0.15	Corrosive
Above 0.15	Severely Corrosive



APPENDIX C

Percolation Test Results

Percolation Test Data

Project: Poly High School Improvements Project
Project No.: 1400125-0009800.00
Date of Testing: 9/19/2025

Date: Sep-25
Reference: Administrative Manual GS200.1 (dated 6/30/21), Guidelines for Geotechnical Investigation and Reporting
 County of Los Angeles Department of Public Works

P-01

Boring Number: P-01
 Hole Diameter (in): 8
 Hours Presaturation: 19
 Time Presoak Initiated: 3:00 PM
 Depth Below Grade (ft): 5
 Strata Peculiarities: NONE
 Name of Tester: Mike Rastegar
 Date Tested: 9/19/2025
 Caving Prevention: Gravel Packed

Hours:Minutes	Hours:Minutes	Minutes	Feet	Feet	Feet	Feet	Inch/Hour	Min/Inch
Initial Time (t _i)	Final Time (t _f)	Elapsed Time (Δt)	Depth to Bottom (d _b)	Initial Water Depth (d _i)	Final Water Depth (d _f)	Water Drop (Δh)	Field Infiltration Rate	Field Percolation Rate
10:15 AM	10:45 AM	30	5	1.15	1.20	0.05	1.2	50.00
10:46 AM	11:16 AM	30	5	1.15	1.20	0.05	1.2	50.00
11:17 AM	11:47 AM	30	5	1.10	1.15	0.05	1.2	50.00
11:48 AM	12:18 PM	30	5	1.10	1.15	0.05	1.2	50.00
12:19 PM	12:49 PM	30	5	1.10	1.15	0.05	1.2	50.00
12:50 PM	13:30 PM	30	5	1.05	1.10	0.05	1.2	50.00
13:31 PM	14:01 PM	30	5	1.10	1.15	0.05	1.2	50.00
14:02 PM	14:32 PM	30	5	1.15	1.20	0.05	1.2	50.00

Reduction Factors

RF_t = 1.00
 RF_v = 1.00
 RF_s = 1.00
 Total Reduction Factor = 3.00

Design Infiltration Rate

0.40

P-02

Boring Number: P-02
 Hole Diameter (in): 8
 Hours Presaturation: 19
 Time Presoak Initiated: 17:30 PM
 Depth Below Grade (ft): 5
 Strata Peculiarities: NONE
 Name of Tester: Mike Rastegar
 Date Tested: 9/19/2025
 Caving Prevention: Gravel Packed

Hours:Minutes	Hours:Minutes	Minutes	Feet	Feet	Feet	Feet	Inch/Hour	Min/Inch
Initial Time (t _i)	Final Time (t _f)	Elapsed Time (Δt)	Depth to Bottom (d _b)	Initial Water Depth (d _i)	Final Water Depth (d _f)	Water Drop (Δh)	Field Infiltration Rate	Field Percolation Rate
12:40 PM	13:10 PM	30	5	1.15	1.20	0.05	1.20	50.00
13:15 PM	13:45 PM	30	5	1.15	1.18	0.03	0.72	83.33
13:50 PM	14:20 PM	30	5	1.10	1.15	0.05	1.20	50.00
14:31 PM	15:01 PM	30	5	1.10	1.15	0.05	1.20	50.00
15:03 PM	15:33 PM	30	5	1.10	1.15	0.05	1.20	50.00
15:35 PM	16:05 PM	30	5	1.15	1.20	0.05	1.20	50.00
16:10 PM	16:40 PM	30	5	1.20	1.25	0.05	1.20	50.00
16:45 PM	17:15 PM	30	5	1.25	1.30	0.05	1.20	50.00

Reduction Factors

RF_t = 1.00
 RF_v = 1.00
 RF_s = 1.00
 Total Reduction Factor = 3.00

Design Infiltration Rate

0.40

Percolation Test Data

Project: Poly High School Improvements Project
Project No.: 1400125-0009800.00
Date of Testing: 9/19/2025

Date: Sep-25
Reference: Administrative Manual GS200.1 (dated 6/30/21), Guidelines for Geotechnical Investigation and Reporting
 County of Los Angeles Department of Public Works

P-04

Boring Number: P-04
 Hole Diameter (in): 8
 Hours Presaturation: 22 Hours
 Time Presoak Initiated: 2:00pm
 Depth Below Grade (ft): 5
 Strata Peculiarities: NONE
 Name of Tester: Alina Hernandez
 Date Tested: 8/27/2025
 Caving Prevention: Gravel Packed

Hours:Minutes	Hours:Minutes	Minutes	Feet	Feet	Feet	Feet	Inch/Hour	Min/Inch
Initial Time (t _i)	Final Time (t _f)	Elapsed Time (Δt)	Depth to Bottom (d _b)	Initial Water Depth (d _i)	Final Water Depth (d _f)	Water Drop (Δh)	Field Infiltration Rate	Field Percolation Rate
12:20 PM	12:21 PM	1	6.25	1.20	6.25	5.05	3636.0	0.02
12:30 PM	12:31 PM	1	6.25	1.20	6.25	5.05	3636.0	0.02
12:55 PM	12:56 PM	1	6.25	1.20	6.25	5.05	3636.0	0.02
13:05 PM	13:06 PM	1	6.25	1.20	6.25	5.05	3636.0	0.02
13:15 PM	13:16 PM	1	6.25	1.20	6.25	5.05	3636.0	0.02

Reduction Factors
 RF_t = 1.00
 RF_v = 1.00
 RF_s = 1.00
 Total Reduction Factor = 3.00

Design Infiltration Rate
 1212.00

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 D1557.

- 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 IBC Standard 1803.5.3). 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 D1556-00 (Sand Cone method), D2937-00 (Drive-Cylinder method), and/or D2922-96 and D3017-96 (Nuclear Gauge method). Generally, one test shall be provided for approximately every 2 vertical feet of fill 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 test pits 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 recompactd 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 $\frac{3}{4}$ -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 sheepfoot-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 D1557 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 IBC section 1803.5.3, 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 D1557. 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 D1557-02. 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 D1557. 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 1 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.

