Appendix B Geotechnical Exploration and Geohazard Report

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GEOTECHNICAL EXPLORATION AND GEOHAZARD REPORT

PATRIOT HIGH SCHOOL – STADIUM IMPROVEMENTS
4355 CAMINO REAL
JURUPA VALLEY, CA

September 20, 2022

Prepared for

Ms. Robin Griffin
Director, Planning & Development
Jurupa Unified School District

Fenagh Job 6484

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September 20, 2022

Ms. Robin Griffin
Jurupa Unified School District

Subject: **Geotechnical Exploration and Geohazard Report**
Patriot High School
Stadium Improvements
4355 Camino Real
Jurupa Valley, CA 92509

Dear Ms. Griffin,

Fenagh Engineering and Testing Services (Fenagh) is pleased to present this Geotechnical Exploration and Geohazard Report for the proposed development located at 4355 Camino Real in Jurupa Valley, California. The purpose of this exploration was to explore and evaluate the subsurface conditions at the site and develop soils engineering conclusions and recommendations for project design and construction as well as provide an assessment of potential geohazards within the project site and surrounding area.

Based on site exploration and analysis, we conclude that the proposed development is feasible for design and construction from a geotechnical engineering perspective. Due to the presence of undocumented fill and the potential for strong ground shaking, designs and construction details related to the geotechnical engineering aspects will be needed to accommodate such effects. A discussion of the subsurface conditions, conclusions, and recommendations for geotechnical and geohazard-related aspects of design and construction for the planned site improvements are presented in the following report.

We appreciate the opportunity to be of service to you over the course of this project. If you have any questions regarding the contents of this report, or if we could provide further assistance, please contact the undersigned.

Sincerely,

Fenagh Engineering & Testing

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TABLE OF CONTENTS

1 INTRODUCTION **1**
1.1 GENERAL **1**
1.2 PROJECT DESCRIPTION **1**
1.3 SCOPE OF SERVICES **2**
1.4 SITE DESCRIPTION **3**
2 FIELD EXPLORATIONS AND LABORATORY TESTING **4**
2.1 FIELD EXPLORATION **4**
2.2 LABORATORY TEST RESULTS **6**
3 FINDINGS **6**
3.1 REGIONAL GEOLOGIC SETTING **6**
3.2 GEOLOGIC LITERATURE REVIEW **6**
3.2.1 California Geological Survey – 2015 / Fault Activity Map of California 6
3.2.2 Earthquake Zones of Required Investigation 7
3.3 SITE GEOLOGY **7**
3.4 SUBSURFACE CONDITIONS **7**
3.4.1 Borings 7
3.5 GROUNDWATER CONDITIONS **8**
3.6 REVIEW OF PREVIOUS GEOTECHNICAL REPORTS **9**
4 GEOLOGIC HAZARDS **10**
5 CONCLUSIONS and RECOMMENDATIONS **10**
5.1 GENERAL **10**
5.2 UNDOCUMENTED FILL SOILS **10**
5.3 EARTHWORK **11**
5.3.1 Site Preparation 11
5.3.2 Engineered Fill 11
5.3.3 Compaction Criteria Engineered Fill 12
5.3.4 Construction Considerations 13
5.3.5 Wet Weather Construction and/or Unstable Soil Conditions 13
5.3.6 Hillside Grading 13
5.4 TEMPORARY EXCAVATIONS **14**
5.5 FOUNDATION Recommendations and design parameters **14**
5.5.1 Allowable Bearing Capacity 15
5.5.2 Estimated Settlement 15
5.5.3 Lateral Resistance 16
5.5.4 Construction Considerations 16
5.5.5 Foundation Setback 16
5.6.1 Axial Capacity 18
5.6.3 Lateral Resistance 19
5.6.4 Other Design Considerations 19
5.7 SEISMIC DESIGN PARAMETERS **20**
5.8 CORROSIVITY **22**
5.9 CONCRETE **22**

3942 Valley Ave. Suite A, Pleasanton, CA 94566 Phone (925) 403-4747
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5.10 INTERIOR CONCRETE SLAB-ON-GRADE 22
5.11 RAISED FLOORS AND BUILDING CRAWLSPACE 23
5.12 EXTERIOR FLATWORK 23
5.13 DRAINAGE 24
5.14 BURIED SHALLOW EXISTING UTILITIES 24
5.15 FLEXIBLE AND RIGID PAVEMENTS 24
5.15.1 Flexible (Asphalt Concrete) Pavements 24
5.15.2 Rigid (Portland Cement Concrete) Pavements 25
5.15.3 Construction Considerations for Pavements 25
6 ADDITIONAL SERVICES 26
6.1 PLAN AND SPECIFICATIONS REVIEW 26
6.2 EARTHWORK OBS, SPECIAL INSP, AND MATERIAL TESTING 26
7 LIMITATIONS 26
8 REFERENCES 28

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Plates

Plate 1	Vicinity Map
Plate 2	Boring Location Map – Elevation Contours
Plate 3	Boring Location Map – Proposed Development
Plate 4	Cross Section A-A’
Plate 5	Cross Section B-B’
Plate 6	Regional Geologic Map
Plate 7	Local Geologic Map
Plate 8	Local Fault Map
Plate 9	Historical Seismic Event Map - Local
Plate 10	Seismic Hazard Zone Map
Plate 11	Flood Insurance Rate Map

Appendix A Field Exploration

A-1	Key to Logs
A-2	Boring Logs B1 through B11

Appendix B Laboratory Testing

B-1	Direct Shear
B-2	Consolidation
B-3	Compaction Test
B-4	Expansion Index
B-5	R-Value
B-6	No. 200 Sieve Test

Appendix C Calculation Sheets

C-1	Liquefaction Analysis
C-2	Dynamic Dry Settlement
C-3	Pole Foundations – Axial Capacity

Appendix D Geohazards Assessment Report

D-1	Geologic Hazards

Appendix E Slope Stability Analysis

E-1	Surficial Slope Stability
E-2	Cross Section A-A’; Bishop; Static
E-3	Cross Section A-A’; Bishop; Pseudo-Static
E-4	Cross Section A-A’; Janbu; Static

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E-5 Cross Section A-A'; Janbu; Pseudo-Static
E-6 Cross Section B-B'; Bishop; Static
E-7 Cross Section B-B'; Bishop; Pseudo-Static
E-8 Cross Section B-B'; Janbu; Static
E-9 Cross Section B-B'; Janbu; Pseudo-Static

Appendix F Previous Geotechnical Investigation: Plot Plan, Borings and Trench Logs

F-1 Plot Plan
F-2 Boring Logs B-01 through B-30
F-3 Trench Logs TR-01 through TR-14

Appendix G Soil Corrosivity Study

G-1 Soil Corrosivity Evaluation Report – Project X Corrosion Engineering

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1 INTRODUCTION

1.1 GENERAL

Presented herein is the Geotechnical Exploration and Geohazard Report for the proposed development located at 4355 Camino Real, Jurupa Valley, California, as indicated on the Vicinity Map, Plate 1. The purpose of this exploration was to explore and evaluate the subsurface conditions at the site and develop geotechnical engineering conclusions and recommendations for project design and construction along with providing an assessment of potential engineering, geologic and seismic hazards within the project site and surrounding area.

1.2 PROJECT DESCRIPTION

The following plans were reviewed during the development of this report:

- HMC Architects
  - Exhibit 1, Track and Field Lighting and Electrical, dated 05/26/2022
  - Sheet A2.10, Enlarged Plans Home Concessions and Restrooms, dated 09/29/2021
  - Drawing No. A1.2-2, Perforated Screen, dated 12/02/2015

Based on review of the Patriot High School Stadium Improvement plans listed above, we understand that the Jurupa Unified School District (JUSD) plans to construct several stadium improvements associated with the existing football field in the southern portion of the Patriot High School Campus as follows:

- Stadium bleachers for spectator seating are planned along the east and west perimeter of the existing football field.
- A single-story concession stand structure including restroom facilities ranging from 2,000 to 2,600 square feet is planned approximately 50 feet north of the running track perimeter.
- An elevated scoreboard will also be constructed in near proximity to the proposed concession stand building.
- Stadium lighting poles are planned around the perimeter of the running track and adjacent to the proposed stadium bleachers.
- An elevated perforated screen located along the south perimeter of the running track
- Underground utilities, concrete flatwork, asphalt paving, and various hardscape features including landscaping.

Details and features of the planned development are indicated on the attached Boring Location Map – Proposed Development, Plate 3.

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Based on correspondence with team members, the stadium bleachers and concession stand building will be lightly loaded and supported on shallow foundations including concrete slab-on-grade floors. The Stadium lighting poles, elevated scoreboard, and perforated screen will be supported by drilled piers extending to depths of approximately 20 to 25 feet.

Final structural loading calculations were not available at the time this report was finalized. The following load conditions are estimated as follows:

Stadium Bleachers -

- 2 to 3 kips per lineal foot wall loads for dead plus live load conditions
- 40-kip column loads for dead plus live load conditions

Concession Stand and Restroom Facilities -

- 2 to 3 kips per lineal foot wall loads for dead plus live load conditions
- 50 to 75-kip column loads for dead plus live load conditions

Elevated Scoreboard, Perforated Screen, and Stadium Lighting Poles -

- 10 to 15-kip column loads for dead plus live load conditions

The load conditions should be verified by the structural engineer. We should be informed if the load conditions increase significantly from that assumed. If the load conditions increase significantly from that assumed, additional analysis may be required.

1.3 SCOPE OF SERVICES

The scope of services is outlined in the Proposal dated June 20, 2022 (Fenagh Proposal FP2679). The scope of services generally includes the following:

- Review of readily available background materials, including geologic maps, aerial photographs, topographic maps, and hazard maps;

- Site reconnaissance by a California Certified Engineering Geologist to observe the site and geologic conditions;

- Coordination with Underground Service Alert (USA) to locate underground utilities in the vicinity of our subsurface explorations;

- Coordination with client to locate underground utilities not covered by USA;

- Subsurface exploration consisting of 11 borings using hollow-stem auger drilling techniques to depths of up to approximately 50 feet below ground surface. Samples logged to characterize subsurface conditions and collected for laboratory testing;

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- Laboratory testing on selected samples to evaluate the in-situ moisture content, dry density, grain size distribution, Expansion Index, soil compressibility, soil corrosion potential and shear strength parameters;

- Review of compliance with the California Geologic Survey – Note 48, Check List for the Review of Engineering Geology and Seismology Reports for California Public Schools, Hospitals and Essential Services Buildings;

- Engineering and Geologic analysis and compilation of field and laboratory data collected, and findings from background research;

- Preparation of this report presenting findings, conclusions, and recommendations related to the geologic hazards and geotechnical conditions observed at the project site including mitigation of geologic and seismic hazards as necessary.

1.4 SITE DESCRIPTION

The Patriot High School Campus is located at 4355 Camino Real, Jurupa Valley, California. The school property borders Mission Boulevard to the north, Camino Real to the east, Jurupa Road to the south, and Garth Street/Bethel Road to the west. The school campus is located on a property identified by Riverside County Assessor's Parcel Numbers (APN) 183-020-008, 183-020-029, and 183-020-035 with a combined land area of approximately 48 1/2-acres. The school is situated on an elevated older alluvial fan terrace formed from sediments derived from the Jurupa Mountains to the north and extending towards the southwest. According to available online aerial photographs (NETRonline: Historic Aerials) the main buildings of the Patriot High School were built prior to 2005. The school property appears to have been originally graded as gently sloping terraced pads with occasional embankments descending toward the southwest. The original permanent school buildings and gymnasium were built on the northern portion of the property where cuts and fills were required to create the building pads. The school property ranges in elevations from approximately 860 feet above MSL in the northeast to approximately 820 feet in the southwest corner of the campus. Residential developments border along the southeast and city streets border the school campus in all directions.

The project site is in the southcentral section of the playfield area within the Patriot High School Campus as indicated on the attached Boring Location Maps, Plates 2 and 3. Embankments which descend to the west and range in height from approximately 6 feet to 11 feet are located along the western perimeter of the project site as indicated on Cross Section A-A' and B-B' (Plates 4 and 5).

The project site is limited to the area surrounding the football field and running track which is currently developed with:

- Football field including an all-weather running track
- Stadium bleachers for spectator seating including restroom and storage facilities
- Asphalt paved access roads and concrete hardscape features

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The coordinates near the center of the project site and area of planned improvements as referenced from Google Earth are:

34.0064 N Latitude
-117.4509 W Longitude

2 FIELD EXPLORATIONS AND LABORATORY TESTING

2.1 FIELD EXPLORATION

Prior to initiating onsite field exploration, the planned boring locations were checked for underground utilities by contacting Underground Service Alert (USA) which located underground and aboveground utilities within the vicinity of the proposed excavations.

The boring exploration program consisted of drilling 11 soil borings identified as Borings B1 through B11 to depths ranging from approximately 25 to 50 feet below the ground surface (bgs).

Borings B1 through B11 were drilled using a truck-mounted CME-75 drill rig equipped with 8-1/2-inch outer diameter hollow-stem augers with a carbide tooth and blade drill bit. Table 2-1 below summarizes the boring depths and approximate locations. Boring Logs for this exploration are presented in Appendix A.

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Table 2-1 – Exploration Depths and Locations				
Exploration ID	Depth of Exploration (ft)	Approximate Latitude	Approximate Longitude	General Location
B1	25 ft	34.006025°	-117.451419°	Southern edge of home team stadium bleachers
B2	25 ft	34.006269°	-117.451466°	Southwest portion of home stadium bleachers
B3	25 ft	34.006483°	-117.451467°	Northwest portion of home stadium bleachers
B4	25 ft	34.006680°	-117.451434°	Northern edge of home team stadium bleachers
B5	25 ft	34.006401°	-117.450412°	Northern portion of visitor stadium bleachers
B6	25 ft	34.006242°	-117.450413°	Central portion of visitor stadium bleachers
B7	25 ft	34.006019°	-117.450393°	Southern edge of visitor stadium bleachers
B8	25 ft	34.006870°	-117.450369°	Northeast perimeter of running track
B9	25 ft	34.005487°	-117.450965°	South perimeter of running track
B10	50 ft	34.007333°	-117.450993°	50 feet northwest of running track
B11	25 ft	34.007329°	-117.450819°	50 feet northeast of running track

Notes:

1. Latitude and longitude were estimated from Google Maps.

Samples were collected from the borings using split barrel soil samplers having nominal outer dimensions of 3.0 inches or standard penetration test sampler (i.e., SPT) without liners which were advanced with a 140-pound hammer free-falling 30 inches. The number of blows required to drive the samplers for the 18-inch sample interval was recorded on the boring logs. The sum of the blow counts for the final 12 inches of driving is recorded as the "N Value". The N Values reported are raw values obtained in the field and are not corrected for overburden, rod length, bore diameter, and hammer energy effects. Relatively undisturbed and bulk samples were collected at select depths from the borings and transported to the laboratory for further analysis and geotechnical testing.

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2.2 LABORATORY TEST RESULTS

Laboratory testing was performed to quantify and evaluate the geotechnical characteristics of the soil samples obtained at the site. The following laboratory tests were performed on selected samples from the borings:

- Moisture Content (ASTM D2216)
- Dry Density (ASTM D2937)
- Laboratory Compaction of Soils Using Modified Effort (ASTM D1557)
- Amount of Material Finer than 75–µm (ASTM D1140)
- Direct Shear Test of Soils under Consolidated Drained Conditions (ASTM D3080)
- One-Dimensional Consolidation Properties of Soils using Incremental Loading (ASTM D2435)
- Expansive Index (ASTM D4829)
- pH and Electrical Resistivity (CT643)
- Sulfate and Chloride Content (CT17 and CT422)
- R-Value (ASTM D2844)

The results of the tests performed above are discussed in the Subsurface Conditions section of this report (Section 3.4). Laboratory test results are provided in Appendix B.

3 FINDINGS

3.1 REGIONAL GEOLOGIC SETTING

The project site is located south of the Jurupa Mountains, in the northern portion Peninsular Range geomorphic province of California. The Peninsular Ranges are bounded by the Transverse Ranges (San Gabriel and San Bernardino Mountains) to the north and the Colorado Desert Geomorphic Province to the east. The province extends westward into the Pacific Ocean and southward to the tip of Baja California. This is an area of complex geology as the relatively northwestward-moving Peninsular Range Province collides with the Transverse Range Province (San Gabriel Mountains) to the north. Several active or potentially active faults have been mapped in the region and are believed to accommodate compression associated with this collision.

Regional and Local Geology for the project site are indicated on Plates 6 and 7, respectively.

3.2 GEOLOGIC LITERATURE REVIEW

The following available published geologic maps and websites pertinent to the site and vicinity were reviewed for the project. Summaries of the maps and websites reviewed are provided below.

3.2.1 California Geological Survey – 2015 / Fault Activity Map of California

According to the Fault Activity Map of California, developed by the California Geological Survey, Department of Conservation, the nearest fault to the project site is an unnamed, "inferred" fault near

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Fontana which is located approximately two miles to the northwest. Additional local faults identified as the Rialto-Colton fault and Loma Linda fault are located three miles to the northeast of the project site. A copy of this map indicating the proximity of local faults in relation to the site is included on Plate 8 entitled "Local Fault Map".

3.2.2 Earthquake Zones of Required Investigation

The California Geologic Survey website, Earthquake Zones of Required Investigation is not available for the subject site or Jurupa Valley. The parcel is not mapped in an earthquake fault zone; however, the site has not been evaluated by the California Geologic Survey for liquefaction or other seismic hazards.

3.3 SITE GEOLOGY

The Jurupa Mountains are underlain by exposures of the northern-most Peninsular Ranges province basement rocks. Pleistocene alluvial-fan deposits flank the south side of the Jurupa Mountains. Most of these deposits are well cemented, brown, sandy deposits containing cobble lenses near the south side of the mountains; clasts are locally derived from the Jurupa Mountains. The lower elevations south of the Jurupa mountains are covered by Pleistocene alluvial-fan deposits. These fans were graded to the location of the present-day course of the Santa Ana River but at a slightly higher elevation than the elevation of the present-day river.

Geologic materials obtained from the onsite subsurface investigation by this firm were consistent with research findings. Uplifted older alluvial soils were encountered to a maximum explored depth of 50 feet beneath existing ground surface.

Granite Bedrock (Kt – Tonalite) was not encountered during this investigation but was observed during a previous onsite investigation by Inland Foundation Engineering, Inc, in several borings at depths ranging from 12 to 35 feet below ground surface and encountered in the northern area of the school property – see Section 3.6. The observed bedrock likely originates from the Perris Block which lies to the north of the project site as indicated by the Local Geologic Map (Plate 7).

Local site geology and regional geology are indicated on Plates 6 and 7. Details of the subsurface conditions encountered are provided below in Section 3.4.

3.4 SUBSURFACE CONDITIONS

3.4.1 Borings

Borings B1 through B11 were drilled on August 3 and August 4, 2022, in the footprint of the proposed stadium bleachers along the east and west perimeter of the running track and the concession stand structure to be located approximately 50 feet north of the existing running track. Additional borings were drilled to the south and northeast of the football field as indicated by the attached Boring Location Maps (Plates 2 and 3). From the subsurface materials encountered, it is likely the property was graded and leveled between 2002 and 2005 during the original construction of the school buildings and playfield

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areas. The southwest portion of the project site appears to be composed of fill material that was pushed out and placed over the existing older alluvial deposits to level the grade for the existing football field.

Surface layers observed during onsite exploration consisted of fill materials and organic laden topsoil. Fill depths were observed to extend to depths ranging from approximately two to five feet - likely due to fill embankments along the west perimeter and various subsurface utility trench backfill in discrete locations. The fill consists primarily of clayey sand, which is reddish brown in color, dense to very dense, moist, and fine to medium grained.

Older alluvial soil underlies the fill which consists of Clayey Sand (SC) to Lean Clay (CL) to depths of approximately 35 feet then transitions to Well Graded Sand (SW), Poorly Graded Sand (SP), and Silty Sand (SM) at depths deeper than 35 feet below ground surface. The alluvium was observed to be predominantly reddish brown in color in the near surface layers and transitions to grayish or yellowish brown at depths of 35 feet. The blowcount values indicate very dense soil conditions which were confirmed by laboratory density testing.

Plates 2 and 3 indicate the locations of the borings. Plates 4 and 5 provide cross-sections of the subsurface conditions encountered near the west perimeter of the site including embankment profiles. Graphical presentations of the boring logs are provided in Appendix A.

3.5 GROUNDWATER CONDITIONS

Groundwater was encountered at a depth of approximately 45 feet in Boring B10 during our explorations. Groundwater was also encountered during the previous investigation by Inland Foundation Engineering, Inc. at depths ranging from 40 feet (Boring B-28) to 45 feet (Boring B-21) below ground surface which correspond closely with our recent exploration observations.

Well water data obtained during an Environmental Site Assessment in 1976 referenced by the previous geotechnical report for the site by Inland Foundation Engineering, Inc. indicates historical groundwater levels as shallow as 16 feet below ground surface. The historically highest groundwater is conservatively defined as 16 feet below ground surface for the purpose of this report.

This firm has reviewed the Department of Water Resources Water Data Library for wells and related depths to groundwater in the vicinity of the site. Table 3-1 below summarizes these findings.

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Table 3-1 – Regional Groundwater Conditions (lowest site elevation = approx. 830 feet)			
Site Code	Ground Surface Elevation (ft, msl)	Distance/Direction from Site	Reported Approx. Range in Depth to Groundwater/ Record Date
340040N1175131W001	730.3	3.5 mile west	63.8' to 160.7' 2010
339950N1174230W001	813.7	2 miles southeast	65.9' to 72.5' 2018

Source: http://wdl.water.ca.gov/waterdatalibrary/

Variations in groundwater levels may occur due to variations in ground surface topography, subsurface geologic conditions and structure, seasonal rainfall, local irrigation practices, new construction, and/or other factors.

3.6 REVIEW OF PREVIOUS GEOTECHNICAL REPORTS

A preliminary geotechnical report previously submitted by Inland Foundation Engineering, Inc. dated July 26, 2001, was provided by the Jurupa Unified School District (client) for review by this firm. Boring Location Map, Boring Logs and Trench Logs are provided in Appendix F. A brief synopsis of the report is presented as follows:

Project Scope: Preliminary geotechnical investigation for a proposed school structure(s) within the Patriot High School Campus -

- Drilling of 30 soil borings to depths ranging between 11 and 51 feet below the ground surface.
- Laboratory testing including moisture content, dry unit weight, plasticity Index, sieve analysis, expansion index, and consolidation testing.
- Recommendations related to the geotechnical aspects of:
    - Site preparation and engineered fill
    - Temporary excavations and trench backfill
    - Foundation design and construction
    - California Building Code seismic site coefficients for use in structural analysis,
    - Concrete slabs and supported-on-grade
    - Site drainage

The borings advanced in the Inland Foundation Engineering, Inc. study generally encountered geologic materials composed of loose to dense silty to clayey sand. Beneath the near surface layers, the borings encountered medium dense to dense Silty Sand, Sandy Silt, and Gravelly Sand. Granite Bedrock (Kt – Tonalite) was reportedly observed during onsite exploration in several borings at depths ranging from 12 to 35 feet below ground surface.

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Stabilized groundwater was encountered at a depth of 45 feet in Boring B-21 and 40 feet in Boring B-26. Well water data obtained during an Environmental Site Assessment in 1976 indicates historical groundwater levels as shallow as 16 feet below ground surface.

The recommendations contained within the report were considered preliminary due to the lack of grading plan and conceptual design details. The final recommendations would be confirmed and finalized once the required information was provided for the proposed development. Final recommendations were not available during preparation of this report.

4 GEOLOGIC HAZARDS

See Appendix D for comprehensive geologic hazards assessment.

5 CONCLUSIONS AND RECOMMENDATIONS

5.1 GENERAL

Based on the results of onsite exploration including findings and analysis, the project is considered feasible for design and construction from a geotechnical engineering perspective. The exploratory borings indicate the near-surface soils are composed of fill materials comprised of low plasticity, dense to very dense, Clayey Sand. The site is also located in an area subject to significant seismic activity. Specific recommendations to accommodate ground shaking due to seismic activity are presented below.

5.2 UNDOCUMENTED FILL SOILS

Undocumented fill was encountered in the borings within the areas of planned development to depths of approximately two feet to five feet below the existing site grade. The presence of undocumented fill will likely cause non-uniform bearing support which can result in intolerable settlement for foundations or building slabs. The fill encountered generally exhibited dense to very dense characteristics for the depth profile encountered. No loose zones were observed in the borings during site exploration.

Depending on the limits and strength characteristics of undocumented fill, intolerable differential settlement could impact the proposed structures. Undocumented fills are generally addressed with complete removal and replacement with certified engineered fill, use of deep foundations extending to competent bearing materials at depth, or use of structural foundation such as a mat foundation. Based on these considerations, we recommend that all undocumented fill is completely removed and recompacted as engineered fill for uniform support of the proposed building slab-on-grade. Foundations for the proposed stadium bleachers and concession stand structure may then be extended through the certified fill to bear in undisturbed alluvial soils. The Geotechnical Engineer of Record (or representative) should be present during grading operations to verify the undocumented fill materials are removed to the target depth prior to backfill.

Complete removal of the existing fill soils beneath outdoor flatwork such as walkways or patio areas, is not required, however, due to the rigid nature of concrete, some cracking, a shorter design life and

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increased maintenance costs should be anticipated. To provide uniform support beneath flatwork, we recommend a 6-inch thick layer of compacted aggregate baserock over a subgrade prepared in accordance with this report. In sitework areas consisting of exterior flatwork, or landscape areas, no overexcavation is required provided the areas are prepared in accordance with the earthwork recommendations presented herein.

5.3 EARTHWORK

5.3.1 Site Preparation

Prior to any site grading, any existing flatwork, pavements, or existing structures and associated foundations requiring demolition should be removed from the construction limits. The site should also be stripped of vegetation, organics, debris, and topsoil. Where roots are less than 1/8-inch diameter, they may remain in place provided they do not comprise more than 3 percent by dry weight of organics in the surrounding native soil. Where trees are removed, the entire tree root ball should also be removed and backfilled with compacted engineered fill.

After stripping and any required overexcavation of undocumented fills to the target depth, the exposed subgrade to receive engineered fill or to be used for future support of structural improvements (i.e., foundations or slabs-on-grade), should be scarified to a depth of at least 8 inches in building areas. Fill material encountered during site exploration should be removed to the target depth. The exposed native subgrade should be moisture conditioned to slightly above the optimum moisture content, and then compacted to no less than 90 percent relative compaction based on the ASTM D1557 test method, latest edition.

All excavations should be observed by the project Geotechnical Engineer or their designated representative to verify the fill has been removed to the target depth and the subgrade meets the intent of this report.

5.3.2 Engineered Fill

On-site or off-site materials can be used as engineered fill provided the fill soils meet the following criteria. If fill is to be imported from off-site, it should meet the requirements of engineered fill below as well as those for Class 3 Subbase in the State of California Standard Specifications, Chapter 25 (latest edition). Any imported fill should be sampled by the project Geotechnical Engineer prior to being imported to evaluate its suitability for its intended use and to perform confirmatory testing listed below, if necessary.

Fill should be nearly free of organic or other deleterious debris, essentially non- to low-plastic, and less than 6 inches in maximum dimension; except that in the upper two feet of subgrade material, the maximum size shall be 3 inches. Specific requirements for engineered fill including the applicable test procedures to verify suitability are presented in the following table.

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Table 5-1 –Suitable Materials for Engineered Fill		
Gradation		
Sieve Size	Percent Passing	Test Procedures
3-Inch	100	ASTM[1] D422 or D6938
No. 200	More than 15	ASTM[1] D422 of D6938/D1140
Atterberg Limits		
Liquid Limit	Plasticity Index	Test Procedures
Less than 30	Less than 12	ASTM[1] D4318
Expansion Index (EI)		Test Procedures
Less than 20		ASTM[1] D4829
R-Value		Test Procedures
Greater than 30		Caltrans Test 301

1 – American Society for Testing and Materials Standards

5.3.3 Compaction Criteria Engineered Fill

Engineered fill within building areas, trench backfill, and flatwork should be placed uniformly in horizontal loose lifts not exceeding 8 inches.

Exposed subgrades and subsequent engineered fill should be uniformly compacted, and moisture conditioned according to the recommended criteria presented in Table 5-2 below.

Table 5-2 – Recommended Compaction and Moisture Conditioning for Subgrade and Engineered Fills			
Material Type and Location	Compaction and Moisture Content per Modified Proctor ASTM D1557		
	Minimum Compaction Requirement, %	Range of Moisture Contents Above Optimum	
		Minimum	Maximum
Subgrade to Receive Fill	90	>/=2%	+4%
Engineered Fill	90	0%	+4%
Exterior Flatwork	90	0%	+4%
Trench and/or Structural Backfill	90	0%	+4%
Traffic Loaded Pavement	95	0%	+4%

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5.3.4 Construction Considerations

Based on our exploratory borings, it is anticipated that excavation may generally be accomplished with typical earthwork grading equipment in good operating condition. Onsite soils will generally consist of clayey sand to sandy clay alluvial materials with occasional gravel.

5.3.5 Wet Weather Construction and/or Unstable Soil Conditions

The in-situ moisture content of the site soils may increase after long periods of rainfall. Soil subgrades may become saturated due to exposure to wet weather conditions. When wet soils are encountered, they should be remediated by aeration, removing, and replacing with drier material, or chemically treated with lime or cement combinations. Although not anticipated, deeper excavations may encounter perched groundwater seepage during the wet season. These conditions may be addressed with localized sump pumps to temporarily facilitate construction.

Fenagh Engineering and Testing should be contacted if these conditions are encountered for assurance of the method selection, specifications, acceptance criteria, and quality assurance.

5.3.6 Hillside Grading

Hillside grading methodologies may be required during recompacted fill placement near the embankments which lie along the west perimeter of the site. All fill materials should be placed in horizontal lifts and should be keyed and benched into undisturbed alluvial soil as follows:

Sidehill fills should have a keyway excavated at the toe of the proposed fill slope. This key should be cut a minimum of 2 feet into undisturbed alluvial soil. The base of the key shall be sloped back into the embankment. Where embankments are steeper than 5:1 (5 horizontal to 1 vertical), horizontal benches shall be cut into alluvial soils to provide both lateral and vertical stability.

Sidehill fills shall have back-drains installed at the compacted fill/alluvium contact to prevent future poor water pressure buildup. Back-drains shall consist of four-inch perforated pipes; placed with perforations facing down. The pipe should be encased with at least one foot (1') of gravel. The minimum cover on the pipe should be one foot (1'). The gravel should consist of three-quarter inch (¾") to one inch (1") crushed rock.

The first drain shall be placed no higher than three feet above the front cut of the key excavation. Additional back-drains shall be placed at intervals roughly equivalent to three feet of vertical rise in elevation or where considered necessary by the representative of this firm.

Each drain shall be placed into a trench excavated along the back of a horizontal bench at the fill/alluvium contact. The trench bottom shall slope downward to each exit drain with a minimum gradient of two percent. The exit pipe shall consist of a four-inch diameter non-perforated pipe. This pipe need not be encased in gravel. It shall exit at a minimum gradient of two percent to the finish face of the fill slope. A

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cutoff wall consisting of concrete or soil cement shall be placed at the junction of the perforated pipe and the exit drains to stop seepage and force the water being removed into the perforated pipe.

Materials excavated uphill from where fills are to be placed, shall not be cast over the slope into the fill area. Materials shall be channeled down a ramp to the area to receive compacted fill and then spread in horizontal layers. As compacted fills are placed, this ramp will be trimmed out to expose the dense, tight materials approved by the soils engineer. The minimum vertical height of bench in approved materials shall be three feet. This will maintain the proper benching, as fill is placed up the slope. The ramp will be shifted periodically during the grading operations to allow for complete removal of the loose fill materials and for the proper benching.

A minimum compaction of 90 percent out to the finish face of fill slopes will be required. Compaction on slopes may be achieved by over building the slope and cutting back to the compacted core or by direct compaction of the slope face with suitable equipment. Direct compaction on the slope faces shall be accomplished by back-rolling the slopes in three foot to four foot increments of elevation gain.

5.4 TEMPORARY EXCAVATIONS

Excavations for foundations can be performed with typical conventional excavating machines generally in use for such projects. During construction, excavations as deep as 5 feet should temporarily stand vertically. Most of the soils will be OSHA Type A. Temporary cuts deeper/higher than 5 feet should be sloped back at maximum 1 horizontal to 1 vertical, 1(h) to 1(v), above the 5-foot level, or stabilized by shoring in accordance with OSHA regulations. The contractor is responsible for providing suitable shoring systems, if required, based on the soils encountered. Deeper excavations may require a shoring system designed by an experienced and licensed civil engineer.

5.5 FOUNDATION RECOMMENDATIONS AND DESIGN PARAMETERS

Stadium bleachers and concession stand structure may be supported by conventional spread foundations bearing in undisturbed alluvial soils. Existing fill within the footprint of the structures shall be removed and recompact as certified recompacted fill for support of building slab-on-grade or structural slabs. Conventional foundations shall extend through the recompacted fill to bear in the underlying undisturbed alluvial soil.

Deepened conventional foundations are recommended for proposed foundations in near proximity to descending embankments located along the west perimeter of the site. It is recommended that the proposed home team stadium bleachers implement this foundation option for foundations positioned near the existing embankments within the building footprint. In addition, foundations within 10 feet (horizontal) of the open face of descending embankments shall be limited to an allowable bearing capacity of 1,000 pounds per square foot (with no incremental capacity increases allowed for width and depth of foundation). See Sections 5.5.5 and 5.5.6 for more details regarding foundations positioned in near proximity to descending embankments.

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Drilled cast-in-place pier foundations (drilled piers) may be utilized for support of the score board, perforated sign, and stadium light poles anticipated as part of the proposed development.

5.5.1 Allowable Bearing Capacity

Conventional spread footing foundations may be utilized for support of the stadium bleachers and concession stand structures. An allowable bearing capacity of 2,500 pounds per square foot (psf) should be used for the design of spread footings supported by undisturbed native alluvial soil. The minimum width and embedment depths are indicated as follows.

Table 5-3 – Minimum Footing Dimensions

Performing Arts Center		
Footing Type	Minimum Depth BGS (inches)[1]	Minimum Width (Inches)
Continuous	24	12
Isolated	24	24

1 – The embedment depth shown is the vertical distance between the base of the foundation and the ground surface or lowest adjacent subgrade, whichever is lower.

Allowable bearing capacity may be increased at a rate of 20 percent for each additional foot of embedment to a maximum of 5,000 pounds per square foot. The allowable bearing capacity is a net value so the weight of the foundation extending below grade may be disregarded when computing dead loads. The allowable bearing capacity is based on a factor of safety of 3 and applies to dead- plus live load conditions. The allowable bearing capacity may be increased by 1/3 for short-term loading due to wind or seismic forces.

As previously stated, stadium bleacher foundations placed within 10 feet (horizontal) of the open face of descending embankments shall be limited to a maximum allowable bearing capacity of 1,000 pounds per square foot with no incremental capacity increases for depth or width of foundation. See Section 5.5.5 and 5.5.6 for more details regarding foundations placed in near proximity to descending embankments.

5.5.2 Estimated Settlement

Total static and any minor anticipated seismic settlement may vary depending on the plan dimensions of the conventional foundation and the actual load supported. Total static settlement of foundations designed in accordance with the recommendations of this report are estimated to be on the order of 1/2 inch. Differential settlements between adjacent footings are expected to be less than 2/3 of the estimated total settlement, provided footings are founded in similar materials. The differential settlement of approximately 1/3 inch is anticipated over a span of 30 feet.

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5.5.3 Lateral Resistance

Resistance to lateral forces may be provided from frictional forces between the bottom of the footing and the underlying soils, and by passive soil resistance against the sides of the foundations. A coefficient of friction equal to 0.37 may be used for dead load forces between proposed cast-in-place concrete footings and the underlying soil. Allowable passive pressure from engineered fill or undisturbed native soil may be taken as equivalent to the pressure exerted by a fluid pressure of 220 pounds per square foot, per foot of depth, (psf/ft or pcf) with a maximum earth pressure of 2,200 pounds per square foot. Additional safety factors may be determined and applied by the project structural engineer.

When combining passive pressure and coefficient of friction for lateral resistance, the passive component should be reduced by one-third. A one-third increase in passive value may be used for wind or seismic loads.

5.5.4 Construction Considerations

Foundation excavations should be firm, neat, and clean of debris, loose, or soft soil, or water prior to placing any reinforcement. All footings excavations should be observed by the project geotechnical engineer or their designated representative just prior to placing reinforcing steel or concrete to verify the recommendations presented herein are implemented during construction.

Additionally, footings may experience an overall loss of bearing capacity or an increased potential for settlement when located near existing or future utility trenches. Further, stresses imposed by the footings on the utility lines may cause cracking, collapse, and/or a loss of serviceability. To reduce this risk, open or backfilled trenches parallel with a footing shall not be below a plane having a downward slope of 2 horizontal to 1 vertical (2:1) slope from a line 9 inches above the bottom edge of the footing and not closer than 18 inches from the face of the footing. When pipes cross under footings, the footings shall be specially designed. Pipe sleeves shall be provided where pipes cross through footings or footing walls and sleeve clearances shall provide for possible footing settlement but not less than 1 inch all around the pipe.

5.5.5 Foundation Setback

California Building Code (CBC, 2019) Section 1808A.7, Foundations on or Adjacent to Slopes, indicates the placement of buildings and structures on or adjacent to slopes steeper than one unit vertical (V) in three units horizontal (H) (33.3-percent slope) shall comply with Section 1808A.7.1 through 1818A.7.5. Section 1808A.7.2 states, where the slope is steeper than 1 unit vertical in 1 unit horizontal (100-percent slope), the required setback shall be measured from an imaginary plane 45 degrees to the horizontal, projected upward from the toe of the slope. An appropriate survey and calculation should be performed to determine the actual percent slope of the embankments located to the west and south of the football field area. Based on contour drawings provided by the client and onsite field measurements, it is the assessment of this firm that the gradient of the embankments within the site generally range from 2H:1V to 3H:1V. See Cross-Sections A-A’ (Plate 4) and B-B’ (Plate 5) for critical embankment profiles affecting the development.

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The final building footprints were not entirely established during this investigation; however, consideration should be given such that the siting of the buildings meet the appropriate minimum setback requirements from the descending slope, see Figure 1808A.7.1 below.

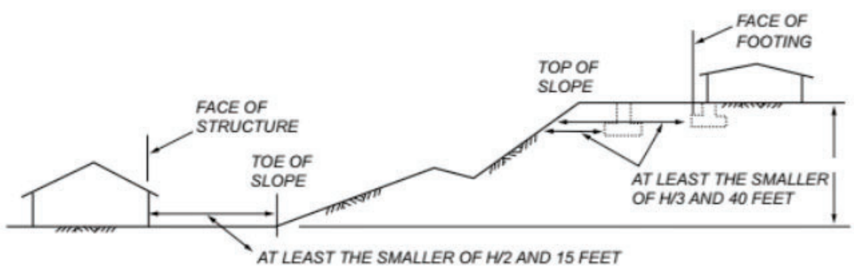

FIGURE 1808A.7.1
FOUNDATION CLEARANCES FROM SLOPES

It is conservatively assumed that the home team bleacher structure's footprints will encroach the embankment limit zones. Accordingly, a slope stability analysis was implemented for this investigation to determine if the building surcharges would impact the stability of the slope. See sections 4.3 (Appendix D) and 5.5.6 for complete analysis, findings, and design recommendations.

5.5.6 Foundations Adjacent to Embankments (West Perimeter of Site)

Embankments to a maximum height of 11 feet are located along the western perimeter of the site in an area designated for the home-team stadium bleachers. Bleacher foundations placed within 10 feet (horizontal) of the open face of descending embankments shall be limited to a maximum allowable bearing capacity of 1,000 pounds per square foot with no incremental capacity increases for depth or width of foundation. In addition, foundations within 10 feet of descending embankments shall be deepened to a minimum of 1/2 the total vertical height of the embankment face and shall bear in undisturbed alluvial soil. Foundations placed within the embankment face shall be deepened to a minimum of 2 feet below the toe elevation of the embankment.

5.5.7 Controlled Low Strength Material (CLSM)

Deepened foundations will likely be required to achieve foundation embedment into alluvial soils and to ensure stability for foundations in near proximity to embankments. The deepened portion of the foundation may be backfilled with Controlled Low Strength Material (or CLSM) as permitted under CBC code section 1803.7. The bottom of excavation shall be cleaned of loose materials prior to placement of

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CLSM. Once the CLSM has cured, the proposed foundations may then be formed and poured thereon. The interface between the CLSM and bottom foundation surface should be dowelled to ensure stability during lateral loading conditions.

Compressive strength of CLSM samples should be tested by a competent materials testing agency in accordance with ASTM D4832, "Standard Test Method for Preparation and Testing of Controlled Low Strength Material Test Cylinders" or equivalent. The CLSM test results should indicate 28-day strength of 100 pounds per square inch or higher.

5.5.8 Grade Beams

Grade beams are recommended to improve stability for foundations in near proximity to embankments. Grade beams should be utilized to unify and secure all foundation in the X and Y direction for structures positioned within 10 feet of the slope face of descending embankments located along the west perimeter of the site. Grade beams shall be supported by undisturbed native alluvial soils or certified compacted fill. Structural slabs may be supported by grade beams as required.

Lateral Capacity

Lateral loads may be resisted by passive pressure of alluvial soil or certified fill placed against side of grade beam. The allowable passive resistance may be assumed as an equivalent fluid having a density of 220 pounds per cubic foot with a maximum earth pressure of 2,200 pounds per square foot. Additional safety factors may be determined and applied by the project structural engineer. Note: Friction coefficient may not be utilized for bottom surface of grade beam in contact with site soils.

5.6 DRILLED PIER FOUNDATIONS

Drilled cast-in-place pier foundations may be utilized for support of the score board, perforated sign, and stadium light poles anticipated as part of the proposed development.

5.6.1 Axial Capacity

Drilled cast-in-place pier foundations utilized for pole-supported structures may derive vertical capacity from older alluvial soils underlying the site. Vertical capacities or 18, 24, and 30-inch diameter drilled cast-in-place pier foundations are provided in the "Drilled Pole Foundation Capacity Calculation" charts included herein (Appendix C). Drilled pier foundations shall be embedded a minimum of 10 feet into older alluvial soils. A historically highest groundwater depth of 16 feet was conservatively assumed as part of the axial capacity calculations.

Uplift capacity of pier foundations may be designed using 50% of downward capacity indicated in the enclosed chart. Drilled pier foundations shall be spaced a minimum of three diameters on center. If pier foundations are so spaced, no reduction in downward or upward capacities need be considered due to group action. A one-third increase may be used for transient loading due to wind or seismic forces.

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5.6.2 Estimated Settlement

Estimated settlement of structures supported by drilled pier foundations is anticipated to be less than 1/2 inch. Differential settlements are typically less than about one-half of the total settlement.

5.6.3 Lateral Resistance

The depth of drilled piers required to resist lateral loads may be determined using the design criteria established in Section 1807A.3.2 of the 2019 California Building Code. For areas where no lateral constraint is provided at the ground surface, such as by rigid floor or pavement, the nonconstrained formula (Equation 18A-1) may be used. For areas where lateral constraint is provided at the ground surface, such as by rigid floor or pavement, the constrained formula (Equation 18A-2 or 18A-3) may be used. The allowable lateral bearing pressure of 440 psf/ft to a maximum of 3,000 psf may be used in determining the required depth for isolated drilled pier foundations. The upper 24 inches of pier foundation embedment should be neglected for pier foundations constructed within an embankment slope face. A one-third increase may be used for transient loading such as wind or seismic forces. Arching effects for passive pressure should not be assumed for lateral capacity of drilled pier foundations.

The lateral resistance computed is based on an isolated, single pier. Where drilled piers are spaced at least 8 pier diameters center-to-center perpendicular to the direction of the load, the piers may be assumed to act as single isolated piers and no reduction will be required. Lateral resistance should be reduced by a multiplier to account for group action effects where spacing is less than 8 pier diameters. The pile spacing in the direction perpendicular to the direction of the load should be at least 2.5 times the pile diameter. In general, the group reduction factor for center-to-center spacing can be determined as indicated below.

Table 5-5 – Group Reduction Factors

Center-to-Center Spacing in Direction of Load	Group Reduction Factor
3d	0.25
4d	0.40
6d	0.70
8d	1.00

5.6.4 Other Design Considerations

Drilled pier foundation excavations should be firm, neat, plumb, and clean of debris, loose or soft soil, or water prior to placing any reinforcement. All pier excavations should be observed by the project Geotechnical Engineer's representative just prior to placing reinforcing steel or concrete to verify the recommendations presented herein are implemented during construction. The inspections should also verify immediately excessive sloughing and/or caving has not reduced the required hole depth. This may be accomplished by using a weighted tape measure or similar measuring device. Steel reinforcement should be placed the same day the concrete will be placed. Additionally, drilled pier excavations should

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be scheduled to allow concrete in each pier to set over night before drilling adjacent holes that are closer than 4 diameters center to center.

Concrete used for drilled pier construction should be discharged vertically into the drilled holes to reduce aggregate segregation. The pier concrete should not be allowed to free fall against the steel reinforcement or sides of the excavation. Sufficient space should be provided in the pier reinforcement cage during fabrication to allow insertion of a pump hose or tremie tube for concrete placement. The pier reinforcement cage should be installed and the concrete pumped and vibrated during placement immediately after drilling is completed.

To develop the skin friction values, concrete used for drilled pier construction should have a slump of 4 to 6 inches for dry placement methods or at least 8 inches if slurry drilling is used. The concrete mix should be designed by a registered design professional to include admixtures and/or water cement ratios to achieve the recommended slumps. It is not recommended to add water to achieve slump.

If slurry methods are used for pier construction, tremie concrete should be performed in accordance with American Concrete Institute (ACI) 204R requirements. The tremie pipe should be rigid and remain several feet below the surface of the in-place concrete at all times. This will allow for a seal to be formed between the water or slurry and the fresh concrete. The upper concrete seal will likely be contaminated with water and should be pumped out until fresh concrete is exposed at the top of the pier.

If casing is considered, it should be removed from the hole as concrete is being placed. The bottom of the casing should be maintained at least 5 feet below the top of the concrete during casing withdrawal and concrete placement operations.

Additionally, footings may experience an overall loss of bearing capacity or an increased potential for settlement when located in near proximity to existing or future utility trenches. Further, stresses imposed by the footings on the utility lines may cause cracking, collapse, and/or a loss of serviceability. To reduce this risk, footings should be extended below a 2 horizontal to 1 vertical, 2(h) to 1(v), plane projected upward from the closest bottom corner of the trench. Foundation excavations within clay soils that are left exposed for extended periods of time may shrink and result in cracking at the surface. They should be kept moist to seal the cracks prior to placing reinforcing steel and concrete.

5.7 SEISMIC DESIGN PARAMETERS

The design of the proposed improvements should be performed in accordance with the requirements of governing jurisdictions and applicable building codes. The structural engineer should confirm the appropriate values to use for the development. Map-based design criteria presented in this section are based on entering the site coordinates (latitude and longitude), the risk category, and Site Class.

Based on the data from the soil borings from the interpreted blow counts, the generalized profile may be classified as Site Class D corresponding to a "Stiff Soil" profile in accordance with Table 20.3-1 of ASCE 7-16. Table 5.5 presents the seismic design parameters for the site in accordance with the CBC (2019) and

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ASCE7-16 guidelines using the SEAOC/OSHPD Seismic Design Maps Tool based on Site Class D site classification:

Table 5.5–Seismic Design Criteria per 2019 California Building Code and ASCE 7-16			
Reference		**Seismic Parameter**	**Value**
ASCE 7-16	Google Earth	North Latitude	34.0064
	Google Earth	West Longitude	-117.4509
	Table 20.3-1	Site Class	D
	Table 1.5-1	Risk Category	II
	Table 11.4-1	Site Coefficient for Short Period, F_A	1.0
	Table 11.4-2	Site Coefficient for Long Period, F_v	1.7*
	Figure 22-7	Peak Ground Acceleration, PGA	0.529
	Table 11.8-1	Site Amplification Factor, F_{PGA}	1.1
	Equation 11.8-1	Peak Ground Acceleration, PGA_M	0.581
	Figure 22-1	Mapped MCE_R Spectral Response Acceleration at 0.2-second period, S_s	1.5g
	Figure 22-2	Mapped MCE_R Spectral Response Acceleration at 1.0-second period, S_1	0.6g
	Equation 11.4-1	Site-Adjusted MCE_R Spectral Acceleration at 0.2-second period, S_{MS}	1.5g
	Equation 11.4-2	Site-Adjusted MCE_R Spectral Acceleration at 1.0-second period, S_{M1}	1.02g*
	Equation 11.4-3	Design Spectral Response Acceleration at 0.2-second period, S_{DS}	1.0
	Equation 11.4-4	Design Spectral Response Acceleration at 1.0-second period, S_{D1}	0.68g*
	Table 11.6-1	Seismic Design Category for Short Period Response Acceleration	D*
	Table 11.6-2	Seismic Design Category for 1-s Period Response Acceleration	D*

*According to requirements in the 2019 California Building Code a site-specific ground motion hazard analysis (SHA) and a site response analysis (SRA) should be performed for Site Classes D, E & F unless the project Structural Engineer intends on applying for applicable Exception No. 2 as allowed in ASCE 7-16, Section 11.4.8.

The values presented in the table assume the structural engineer will utilize the exceptions allowed in the 2016 ASCE 7-16, Section 11.4.8 for the Site Class D structure. The structural engineer is responsible for the selection of the appropriate spectral acceleration values used in design. Should a site-specific hazard analysis be required, please notify this firm accordingly.

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5.8 CORROSIVITY

Laboratory testing was performed on a representative sample of the on-site earth materials to evaluate pH and electrical resistivity, as well as chloride and sulfate contents. These laboratory test results are summarized below in Table 5-6 and presented in the Soil Corrosivity Evaluation Report in Appendix G.

Table 5-6 – Evaluation for Soil Corrosion

Corrosion Property	Test Result	Test Method
Soil pH	6.8 to 9.1	CA DOT Test #643
Minimum Resistivity	1,608 ohm-cm to 4,288 ohm-cm	CA DOT Test #643
Chloride	60 ppm to 162 ppm	CA DOT Test #422
Sulfate	110 ppm to 286 ppm	CA DOT Test #417

Based on the Caltrans Highway Design Manual corrosion criteria (Caltrans, 2020), corrosive soils are defined as soils with an electrical resistivity of 1,000 ohm-cm or less, more than 500 ppm chlorides, more than 0.2 percent sulfates, and a pH less than 5.5. Based on the Minimum Resistivity results, the on-site soils would not be classified as corrosive. See attached Soil Corrosivity Evaluation Report (Appendix G) for discussion of results and recommended mitigation measures.

5.9 CONCRETE

Concrete in contact with soil or water that contains high concentrations of water-soluble sulfates can be subject to premature chemical and/or physical deterioration. The soil sample tested in this evaluation by Project X Corrosion Engineers indicated water-soluble sulfate content of 0.0286 percent by weight (286 ppm). According to American Concrete Institute (ACI) 318, the potential for sulfate attack is negligible for water-soluble sulfate contents in soil ranging from 0.00 to 0.10 percent by weight (i.e., 0 to 1,000 ppm). Therefore, the site soils may be considered to have a negligible potential for sulfate attack. However, due to the potential variability of site soils, we recommend using Type II/V cement for concrete structures in contact with soil with a water-cement ratio no higher than 0.45 by weight for normal-weight aggregate concrete for the project.

5.10 INTERIOR CONCRETE SLAB-ON-GRADE

It is the understanding of this firm that the proposed concession stand structure will likely have slab-on-grade floors. The subgrade should be prepared in accordance with the earthwork recommendations included herein as well as the following:

- Concrete slabs should be at least 5 inches thick. The slab reinforcing and thickness should be designed and verified by the structural engineer. As a minimum, the slab reinforcement should consist of No. 4 bars spaced 16 inches on-center each way.

- Where moisture sensitive floor coverings are used, a vapor retarding membrane a minimum of 15 mils thick and in conformance with ASTM E 1745-97 Class A requirements. Placement of the

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vapor retarder and "welding" of overlaps should follow the manufacturer's guidelines and is the responsibility of the foundation contractor.

- A layer of crushed rock at least 4 inches thick should underlie the vapor retarding membrane. The rock shall be clean, crushed, and free-draining having a nominal 1-inch maximum size with less than 3 percent passing the no. 200 sieve.

Some cracking of the slabs-on-grade may be anticipated at the site because of concrete shrinkage and potentially expansive nature of the onsite soils. Frequent control joints should be provided to control the cracking. As a general guideline, control joints should be spaced at distances equal to 24 to 36 times the slab thickness. Joint spacing that is greater than 15 feet require the use of load transfer devices (dowels or diamond plates). Added steel or slab thickness would also serve to improve the performance of the slabs. Subgrade materials should not be allowed to desiccate between grading and the construction of the concrete slabs. Moisture content of subgrade soils shall be maintained until they are covered with aggregate baserock or concrete slabs-on grade.

5.11 RAISED FLOORS AND BUILDING CRAWLSPACE

For buildings constructed with raised floors and underlying crawl space areas, there are risks of excessive ground moisture and water vapor leading to wood damage, mold, mildew, etc. Irrigation practices around the structure, presence of perched groundwater, depressed crawlspace, and poor site drainage can lead to high ground moisture conditions in the crawlspace areas. To reduce the potential for high ground moisture conditions, it is recommended that measures are implemented to control moisture below and around the structures.

Building pads shall be graded to promote positive drainage away from the building. Surface drain inlets shall be installed at low portions of the crawl space to collect and divert surface water, in sealed pipes, away from the structure. Perimeter subdrains shall be installed around the building perimeter. Utility trenches entering the foundation area shall be provided with low permeability trench "plugs" to reduce moisture migration via pervious trench backfill materials. Crawlspace ground surface shall be covered with a durable vapor retarder/liner conforming to Class A of ASTM E1745-97; vapor retarders shall be installed in accordance with manufacturer's recommendations, including sealing seams, pipe penetrations, and attachment to perimeter concrete stem-walls.

At a minimum, it is recommended that ventilation openings be provided through foundation walls or exterior walls for the under-floor space, between the bottom of the floor joists and the earth under the building, in accordance with 2019 CBC. Additionally, the locations of the ventilation openings shall be around all sides of the foundation perimeter. Consultation with a crawlspace ventilation specialist may be warranted to verify the recommendations provided are adequate based on the site conditions observed.

5.12 EXTERIOR FLATWORK

Exterior slabs should be at least 4 inches thick and placed over at least 6 inches of aggregate base over a subgrade prepared in accordance with the recommendations of this report. The design professional should

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determine the final slab thickness, reinforcing, and joint spacing based upon the anticipated loads. Slab support may be derived from extra reinforcement in slabs. The upper 6 inches of soil should be prepared per the Earthwork recommendations presented in this report. This may require removal of expansive soils in the upper 6 inches and replacement with non-expansive soil or aggregate base. Slab reinforcement should be supported on dobie blocks or similar. Due to the potentially expansive soils, slabs should be provided with contraction joints on a rectangular pattern, no greater than 15 feet square and with a length-to-width ratio not exceeding 3. Tee-joints should be avoided. Place trimmer bars at least 4 feet long diagonally across L-corners. Provide expansion joints in the playground paving at 30-foot maximum centers to accommodate expansive soil and thermal expansion. These should have ½” or thicker joint board and greased dowels.

5.13 DRAINAGE

To minimize moisture intrusion into foundation and slab subgrades, we recommend the ground surface should slope away from building pad and pavement areas in accordance with jurisdictional and/or California Building Code requirements toward the appropriate drop inlets or other surface drainage devices. These grades should be maintained for the life of the project. Building pads should also be designed such that the lowest adjacent grade surrounding the building is at or below the elevation of the building pad surface (at or below the bottom of the capillary break material beneath the floor slab. Downspouts should be directed to discharge away from the building to an appropriate catch basin.

Landscaping after construction should not promote ponding of water adjacent to structures. Landscaping adjacent to foundations should include vegetation with low water demands and irrigation should be limited to that which is needed to sustain the plants. Trees should be restricted from the areas adjacent to foundations a distance equivalent to the canopy radius of the mature tree. Stormwater management facilities that percolate water into the subgrade soil should not be located within a distance of 20 feet from structure foundations.

5.14 BURIED SHALLOW EXISTING UTILITIES

Based on this firm’s experience at existing school sites, buried shallow existing utilities may be present near the upper 12 to 18 inches of subgrades that may impact site grading. Provisions should be considered to allow for a modified section over the pipe to allow for protection during construction. A suitable means to protect pipes in place during pavement subgrade preparation would be to encase the pipe with CLSM or utilize geotextile (i.e., Mirifi Rs380i, or approved equivalent) over the pipe to provide additional stability to the pavement section. A separate line-item unit price should be provided by the contractor to allow for such conditions should they occur.

5.15 FLEXIBLE AND RIGID PAVEMENTS

5.15.1 Flexible (Asphalt Concrete) Pavements

Laboratory testing from one (1) bulk soil samples taken from the proposed pavement area resulted in R-Values (Resistance Values) of 31. Asphalt and base course materials should meet the requirements of the

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Caltrans Standard Specifications, latest edition. The pavement sections are based on a subgrade R-value equal to 31.

Table 5-7 – Recommended Paving Sections				
Service Level	Flexible Asphalt Pavement		Rigid Concrete Pavement	
	Asphalt Pavement Thickness (Inches)	Asphalt Pavement Base Course (Inches)	Concrete Pavement Thickness (Inches)	Concrete Pavement Base Course (Inches)
Passenger Cars (TI = 4)	3	5	6	4
Moderate Truck (TI = 6)	4	8	6	4
Heavy Trucks (TI = 8)	5	11	7-1/2	4

If adverse conditions are encountered during the preparation of subgrade materials, special construction methods may need to be employed. Subgrade materials should be processed to a minimum depth of 12 inches below the Class II aggregate base and compacted to a minimum 95 percent of ASTM D1557 laboratory maximum dry density at or near the optimum moisture content. Class II Aggregate Base material should be compacted to 95 percent of ASTM D1557 laboratory maximum dry density at or near optimum moisture content. The base should meet the quality requirements outlined in Section 26 of the Caltrans Standard Specifications.

The pavement section is intended as a minimum. Positive site drainage should be always maintained. Water should not be allowed to pond or seep into the ground. If the traffic service level increases beyond that intended, as reflected by the assumed traffic designation, increased maintenance could be required for the pavement section. The project Civil Engineer should determine the Traffic Index appropriate for the project.

5.15.2 Rigid (Portland Cement Concrete) Pavements

Where rigidity of pavement is desired for areas designed for, high volume vehicular traffic, heavy maintenance or equipment traffic, entry driveways or trash enclosure slabs, we recommend using Portland cement concrete paving as indicated in Table 5.7. In addition, the driveway slabs should be designed with thickened edges at least twice the slab thickness. The design and thickness of rigid pavement slabs should be confirmed by the design professional.

5.15.3 Construction Considerations for Pavements

Additional requirements and/or assumptions for pavements are outlined below:

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- Baserock materials used should comply with the requirements outlined in Section 26 of the State Standard Specifications. We strongly recommend that baserock be a virgin, crushed aggregate product.
- Baserock should be firm and stable prior to placing asphalt and compacted to a minimum of 95 percent based on the ASTM D1557 test method.
- Subgrade beneath paved areas shall be compacted to a minimum of 95 percent based on the ASTM D1557 test method.
- Proof rolling of subgrade and of baserock with fully loaded water truck, or equivalent, should be performed under observation of our field representatives to detect for any instabilities of pavement subgrade and baserock following final grading. Proof rolling of subgrade should occur immediately (i.e., less than 24 hours) before placement of baserock. Baserock should be proofrolled immediately prior to placement of tack coat.
- Subgrade preparation is performed as outlined in the Earthwork sections of this report.

6 ADDITIONAL SERVICES

6.1 PLAN AND SPECIFICATIONS REVIEW

The preparation of the geotechnical investigation for design purposes is a portion of the services Fenagh Engineering & Testing can provide. It is essential that Fenagh Engineering & Testing be requested to perform a general review of the plans and specifications to evaluate if the recommendations contained in this report are properly interpreted and implemented during the design phase. Fenagh will not be responsible for any misinterpretation of our recommendations if we are not retained to perform this recommended task.

6.2 EARTHWORK OBSERVATIONS, SPECIAL INSPECTIONS, AND MATERIALS TESTING

To provide project continuity, it is essential that Fenagh Engineering & Testing be retained to observe earthwork construction, to evaluate exposed foundation soils for appropriate bearing capacity, and provide special inspections and materials testing. Construction services of Fenagh are essential to observe grading operations during site preparation, test trench backfill, engineered fill, and other related construction, observe surface and subsurface conditions during foundation excavation, evaluate the applicability of the recommendations contained in this report, and recommend appropriate changes in construction procedures if conditions are found to differ from those encountered during this investigation.

Separate proposals and estimates can be provided for each of the additional services described above when requested. Fenagh Engineering & Testing can also prepare a master agreement for providing these services.

7 LIMITATIONS

The conclusions and recommendations provided in this report are based on our understanding of the proposed improvements, data developed from the results of our field and laboratory testing program laboratory testing, and our engineering analyses. The field explorations were located in the field by

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pacing from available landmarks as surveying was not part of our work scope. It is possible that actual subsurface conditions can vary between the points of exploration provided during this investigation. If this is found to be the case, Fenagh should be notified and requested to review the changes and provide appropriate modifications to our recommendations if needed.

We have strived to prepare this report in substantial accordance with generally accepted geotechnical engineering practice as it exists in the local area at the time of the work. No warranty, express or implied, is made. This report may be used by the Client, for the purposes stated, for a maximum of two years from the date of the report. If construction is delayed, or if the final construction varies from that stated herein, and land use or other factors modify site and subsurface conditions beyond our control, additional field explorations, laboratory testing, and an updated analysis and report may be required. Fenagh Engineering & Testing shall be released from any liability resulting from any misuse of the report by the authorized party.

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PLATES

Plate 1: Vicinity Map
Plate 2: Boring Location Map – Elevation Contours
Plate 3: Boring Location Map – Proposed Development
Plate 4: Cross Section A-A’
Plate 5: Cross Section B-B’
Plate 6: Regional Geologic Map
Plate 7: Local Geologic Map
Plate 8: Local Fault Map
Plate 9: Historical Seismic Event Map - Local
Plate 10: Seismic Hazard Zone Map
Plate 11: Flood Insurance Rate Map

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REFERENCE:
USGS TOPOGRAPHIC MAPS, 7.5 MINUTE SERIES
FONTANA & RIVERSIDE WEST QUADRANGLE

VICINITY MAP		
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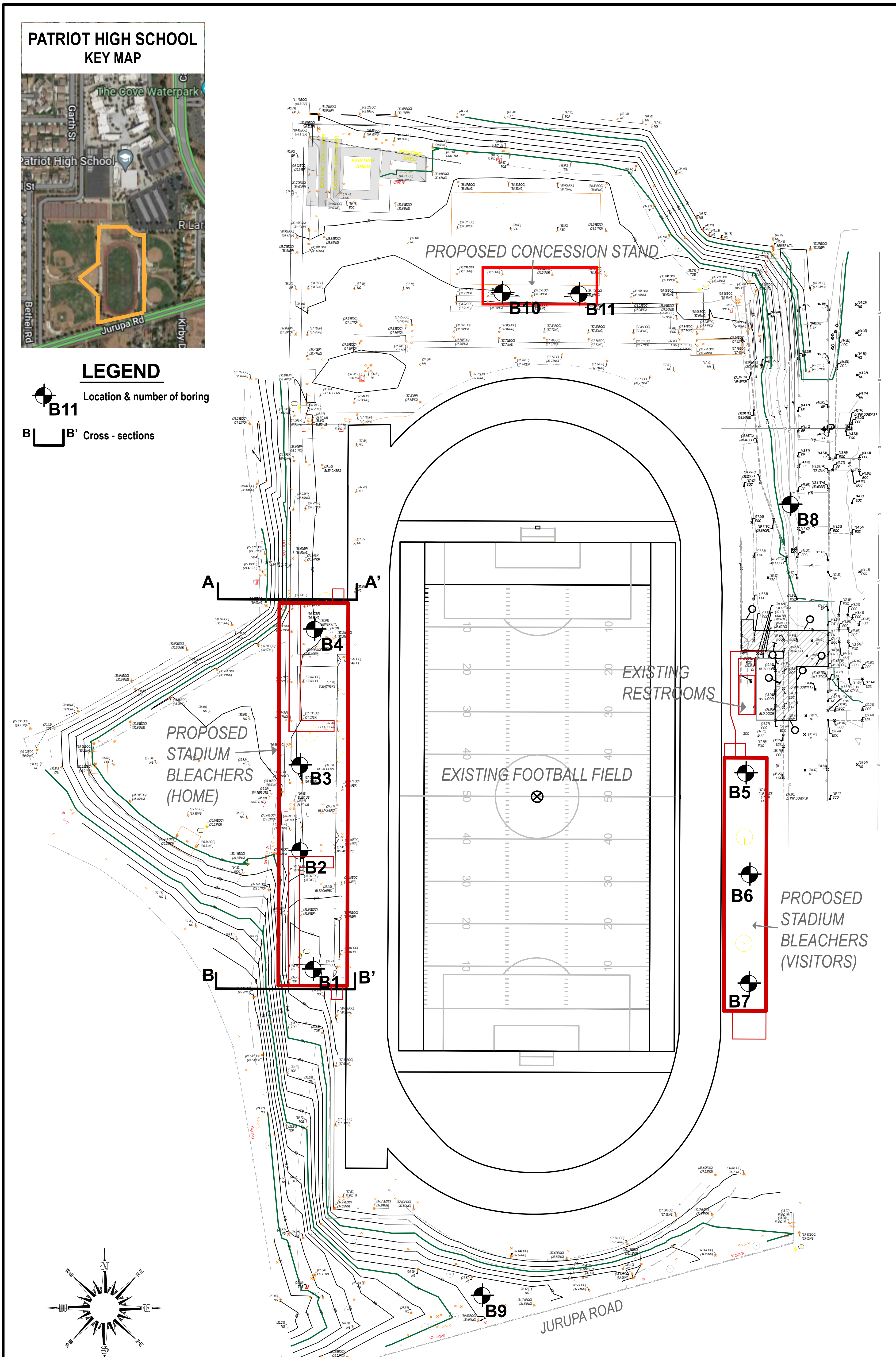
	FILE NO. 6484	1



BORING LOCATION PLAN - ELEVATION CONTOURS

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FILE NO. 6484

PLATE **2**

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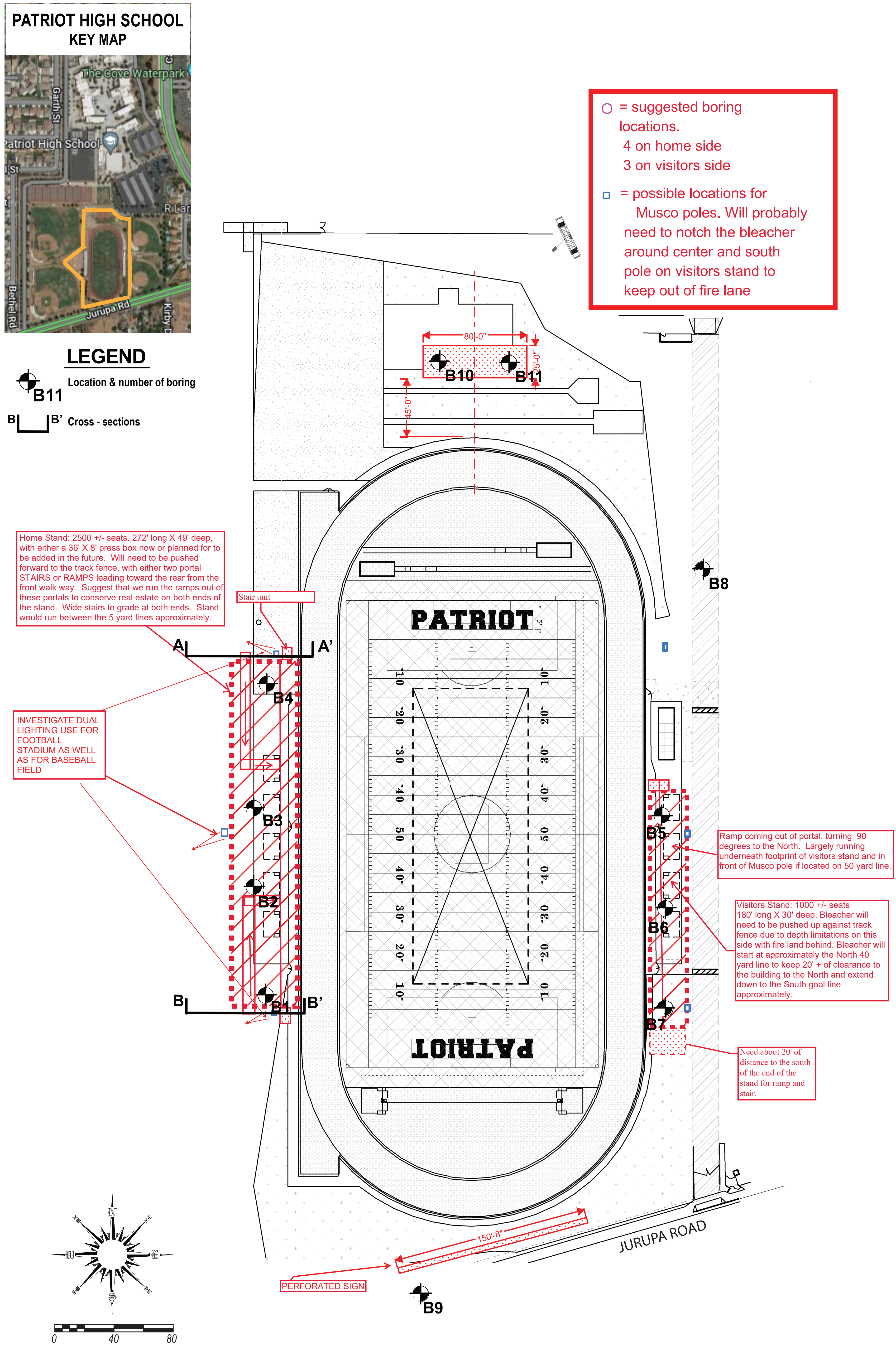

REFERENCE: PATRIOT HIGH SCHOOL TRACK AND FIELD PHASE II - LIGHTING AND ELECTRICAL,HMC ARCHITECTS CONSTRUCTION DRAWINGS, A2.10, 09/29/2021

BORING LOCATION MAP - PROPOSED DEVELOPMENT		
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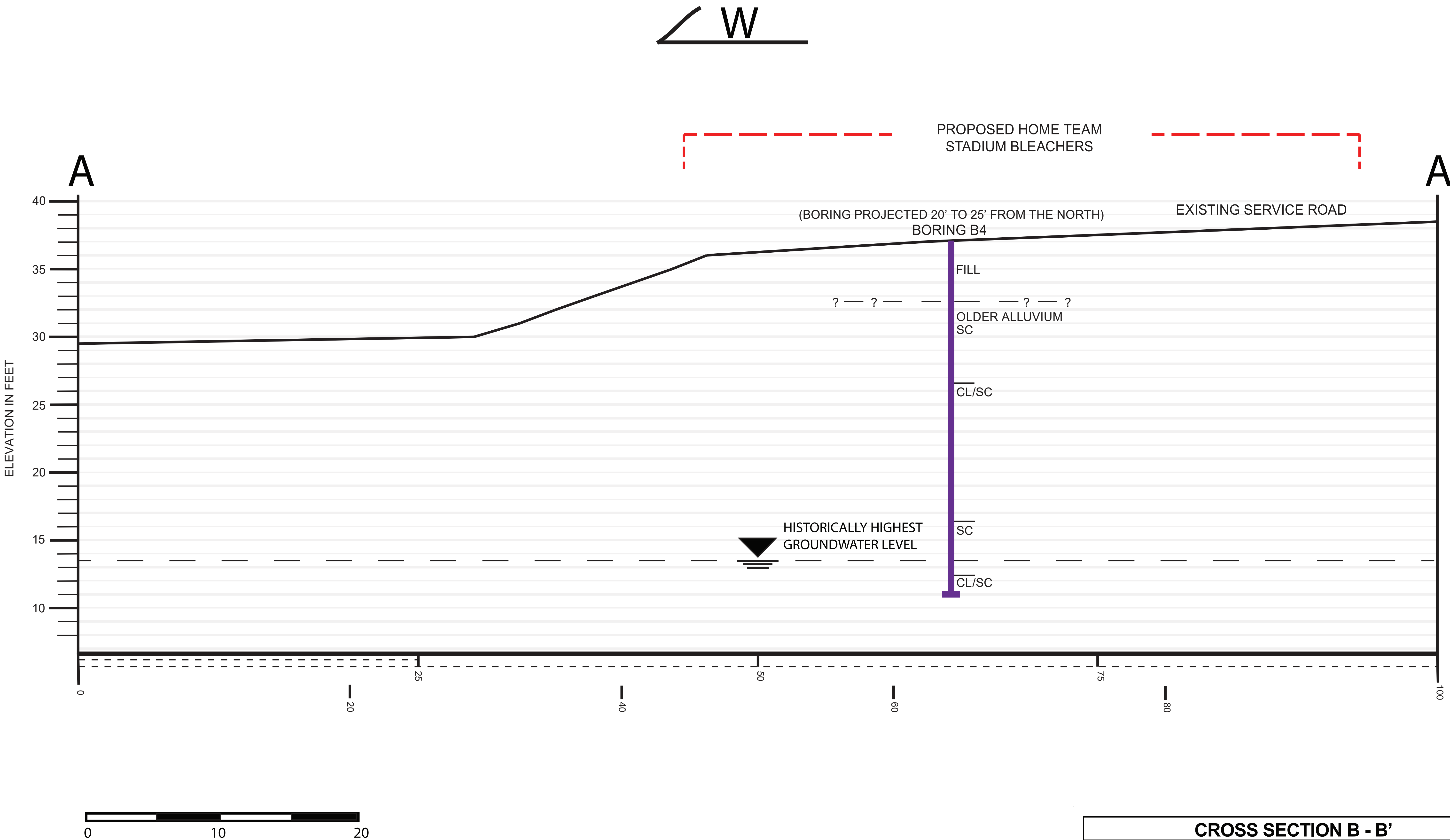

CROSS SECTION B - B'

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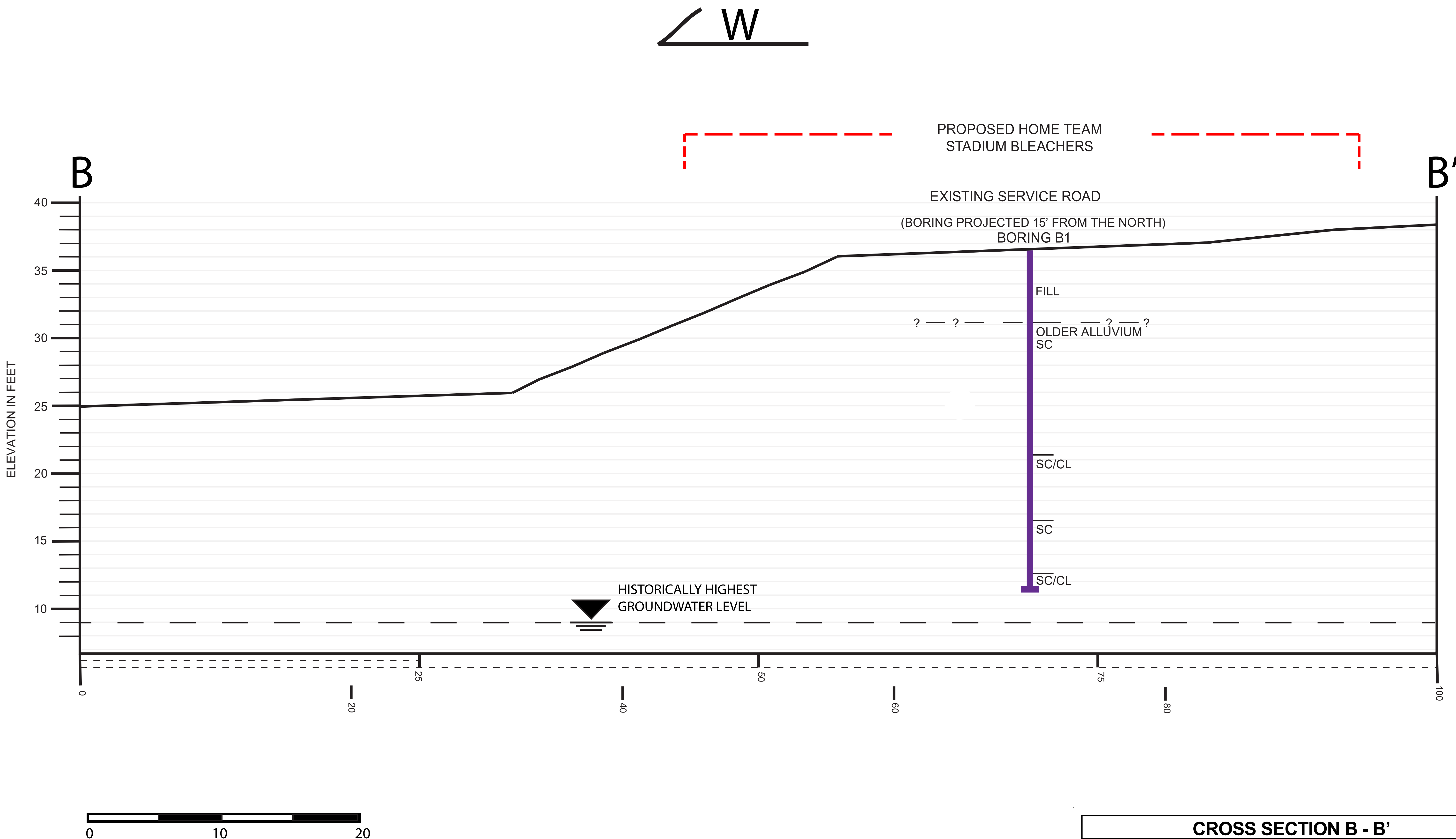
PLATE
4



CROSS SECTION B - B'

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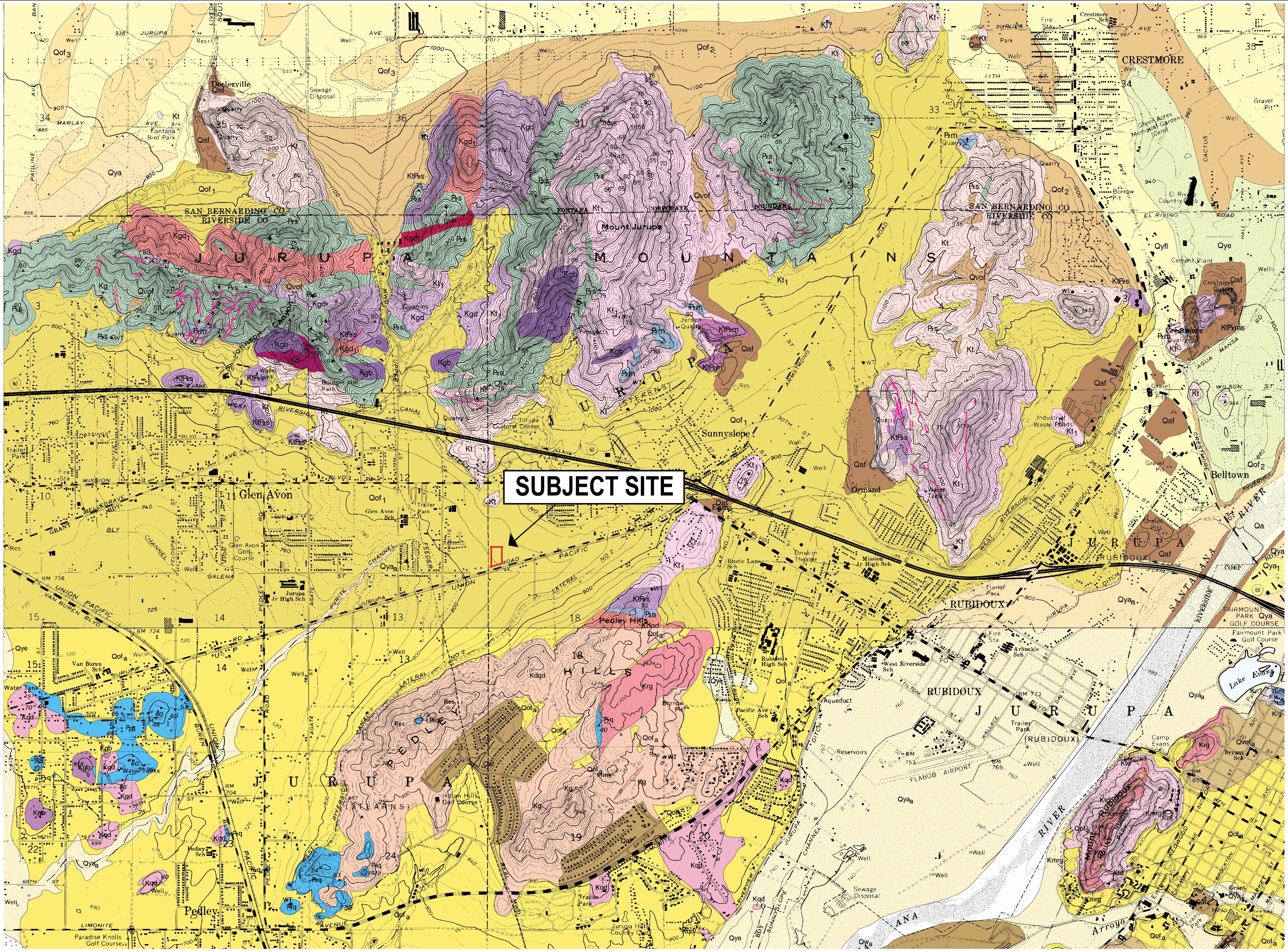
FILE NO. 6484

PLATE
5



LEGEND

Qaf - Artificial Fill
Qa - Alluvium
Qf - Alluvial-Fan Deposits
Qof - Old Alluvial-Fan Deposits
Qoa - Old Alluvium
Kt - Tonalite
Kt1 - Tonalite, Unite 1
Kdqd - Diorite and tonalite, undifferentiated
PzS - Schist
KtPzS - Intermixed tonalite and marble

Contact

Kg - Granitic dikes

REFERENCE:
MORTON, DOUGLAS M., 2003, PRELIMINARY GEOLOGIC MAP OF THE FONTANA 7.5' QUADRANGLE, RIVERSIDE AND SAN BERNARDINO COUNTIES, CALIFORNIA: U.S. GEOLOGICAL SURVEY OPEN-FILE REPORT 03-418

REGIONAL GEOLOGIC MAP

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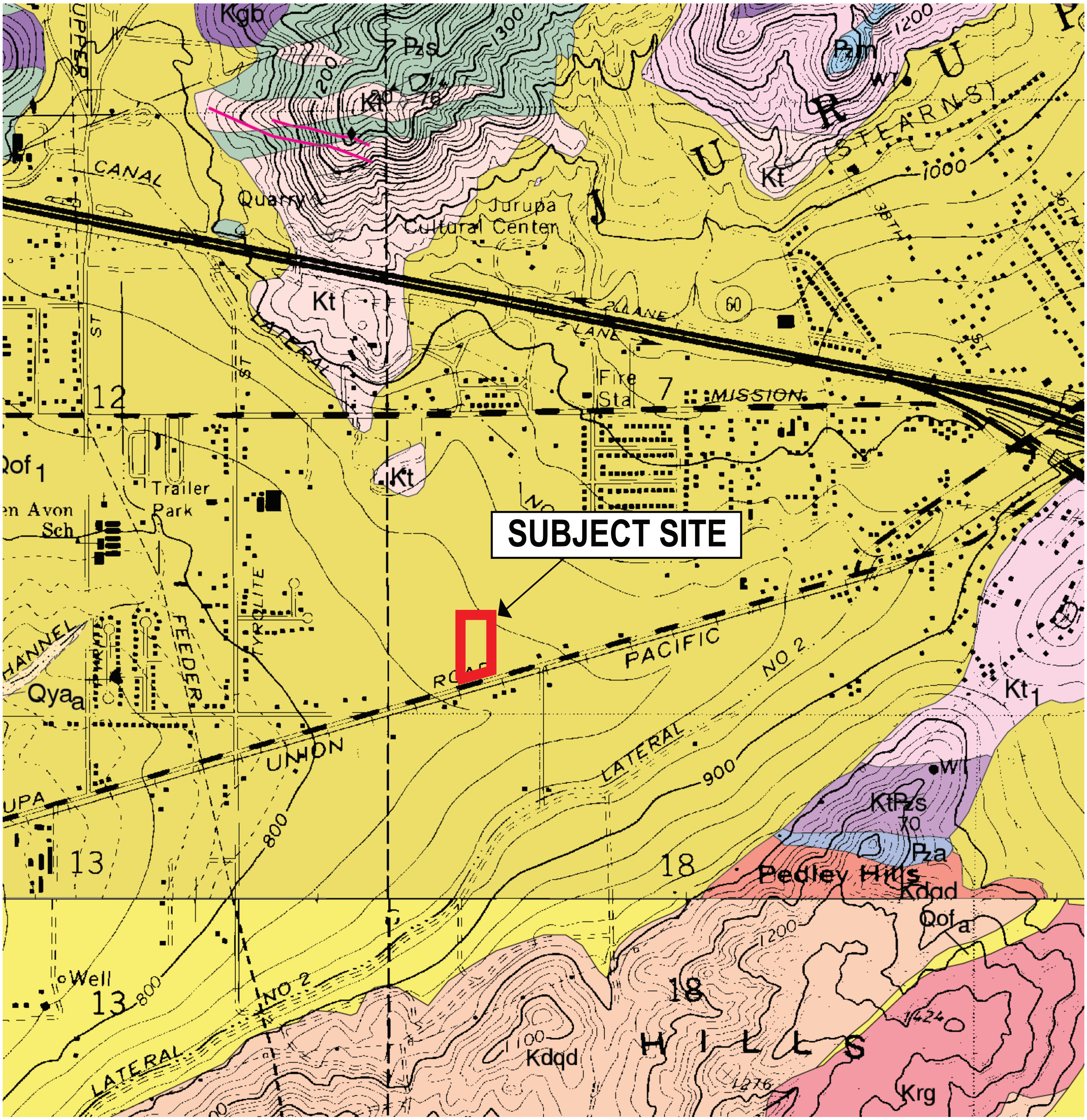
FILE NO. 6484

PLATE 6



SCALE 1:24,000

0 — 1 MILE

CONTOUR INTERVAL 20 FEET

N

LEGEND

Qya - Young axial channel Deposits
Qof - Old Alluvial-Fan Deposits
Krg - Granite of Riverside Area
Kt - Tonalite
Kt1 - Tonalite, Unite 1
Kdqd - Diorite and tonalite, undifferentiated
PzS - Schist
KtPzS - Intermixed tonalite and marble

Contact

Kg - Granitic dikes

REFERENCE:
MORTON, DOUGLAS M., 2003, PRELIMINARY GEOLOGIC MAP OF THE FONTANA 7.5' QUADRANGLE, RIVERSIDE AND SAN BERNARDINO COUNTIES, CALIFORNIA: U.S. GEOLOGICAL SURVEY OPEN-FILE REPORT 03-418
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LOCAL GEOLOGIC MAP		
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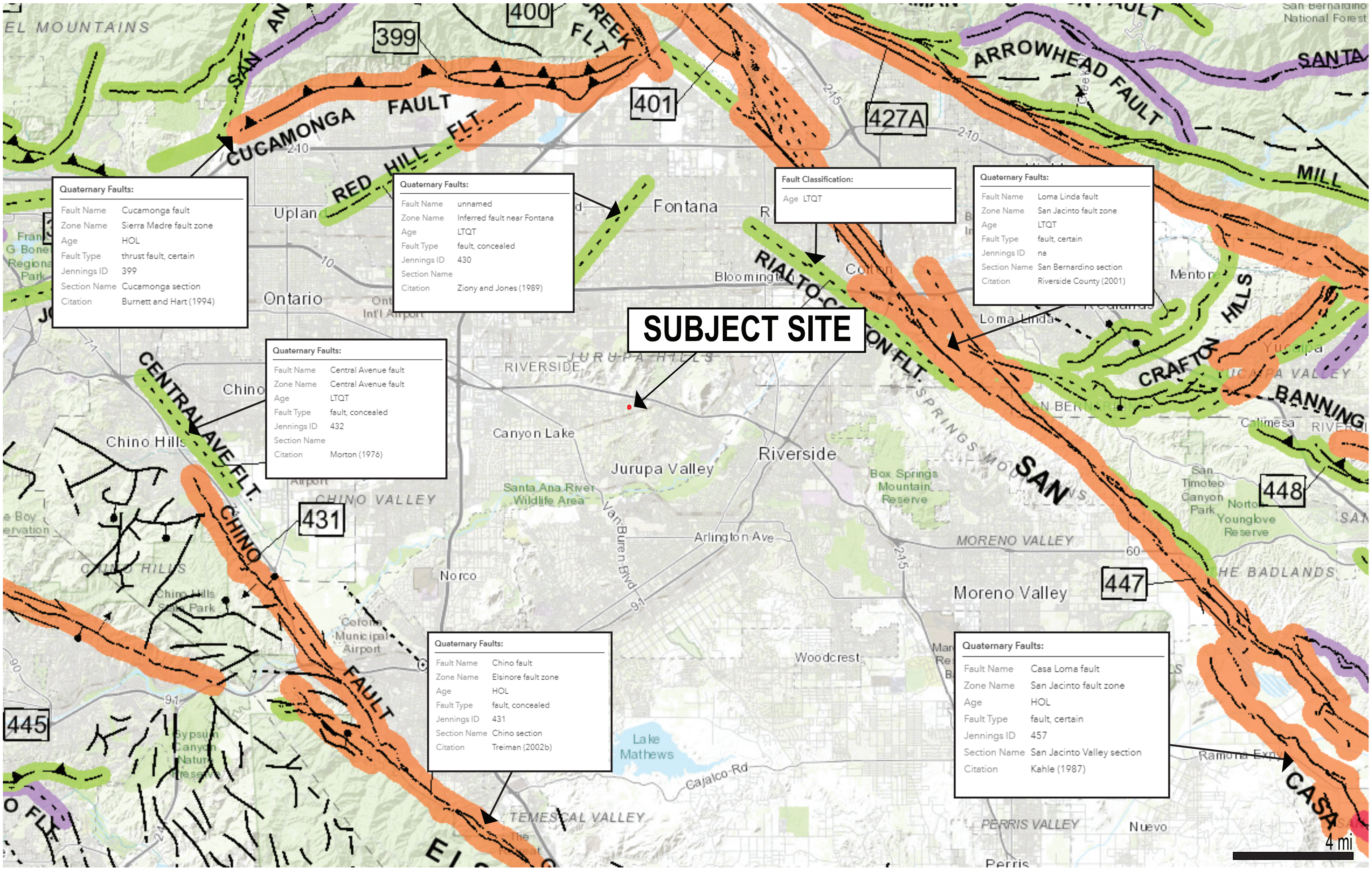
	FILE NO. 6484	



LOCAL FAULT MAP

FENAGH ENGINEERING AND TESTING

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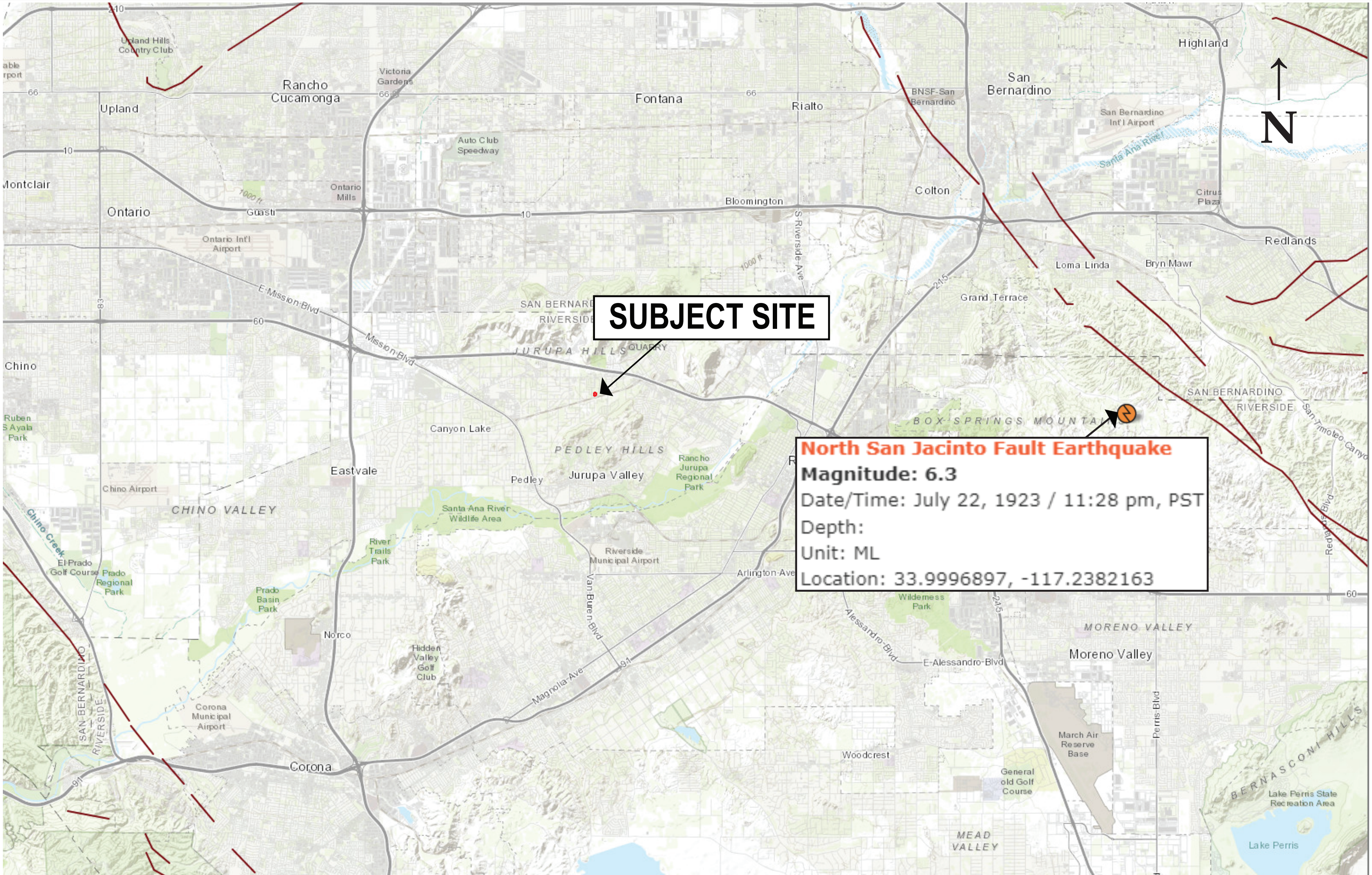
PLATE 8

REFERENCE:
FAULT ACTIVITY MAP OF CALIFORNIA (2015)
CALIFORNIA GEOLOGICAL SURVEY, CALIFORNIA DEPARTMENT OF CONSERVATION



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REFERENCE:
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SOUTHEN CALIORNIA EARTHQUAKE DATA CENTER, CALTECH

HISTORICAL SEISMIC EVENT MAP - LOCAL		
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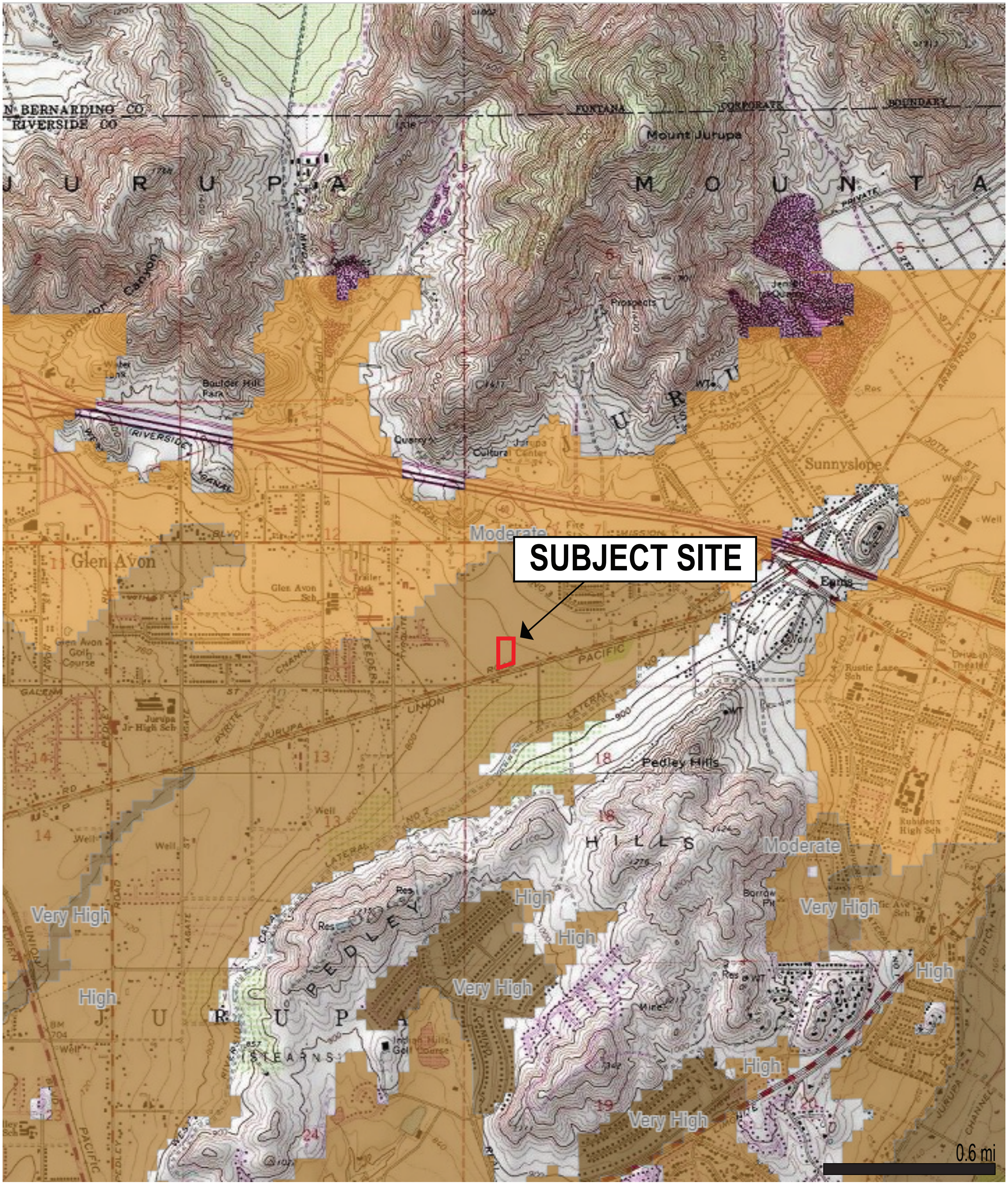
	FILE NO. 6484	9



N

LIQUIFACTION SUSCEPTIBILITY

- VERY HIGH
- HIGH
- MODERATE

REFERENCE:
RIVERSIDE COUNTY MAPPING PORTAL
(WWW.GISOPENDATA@RIVCO.ORG)

SEISMIC HAZARD ZONE MAP		
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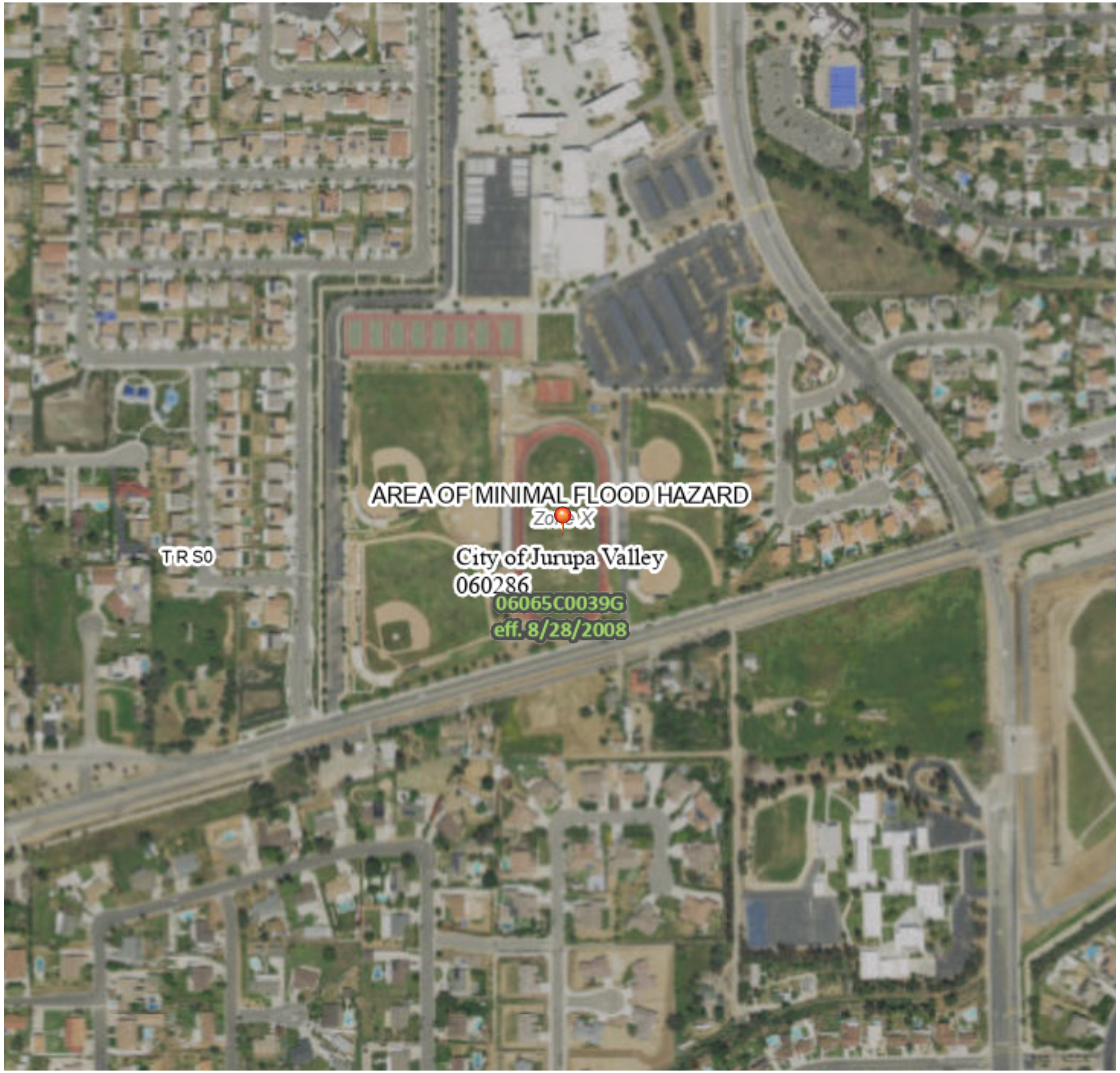
	FILE NO. 6484	10

117°27'22"W 34°0'38"N



117°26'45"W 34°0'8"N

N

Legend

SEE FIS REPORT FOR DETAILED LEGEND AND INDEX MAP FOR FIRM PANEL LAYOUT

SPECIAL FLOOD HAZARD AREAS		Without Base Flood Elevation (BFE)
		With BFE or Depth
		Regulatory Floodway
OTHER AREAS OF FLOOD HAZARD		0.2% Annual Chance Flood Hazard, Areas of 1% annual chance flood with average depth less than one foot or with drainage areas of less than one square mile
		Future Conditions 1% Annual Chance Flood Hazard
		Area with Reduced Flood Risk due to Levee. See Notes.
		Area with Flood Risk due to Levee
	NO SCREEN	Area of Minimal Flood Hazard
		Effective LOMRs
OTHER AREAS		Area of Undetermined Flood Hazard
GENERAL STRUCTURES		Channel, Culvert, or Storm Sewer
		Levee, Dike, or Floodwall
OTHER FEATURES	B 20.2 17.5	Cross Sections with 1% Annual Chance Water Surface Elevation
	8	Coastal Transect
	513	Base Flood Elevation Line (BFE)
		Limit of Study
		Jurisdiction Boundary
		Coastal Transect Baseline
		Profile Baseline
		Hydrographic Feature
MAP PANELS		Digital Data Available
		No Digital Data Available
		Unmapped
		The pin displayed on the map is an approximate point selected by the user and does not represent an authoritative property location.

This map complies with FEMA's standards for the use of digital flood maps if it is not void as described below. The basemap shown complies with FEMA's basemap accuracy standards

The flood hazard information is derived directly from the authoritative NFHL web services provided by FEMA. This map was exported on 8/23/2022 at 4:51 PM and does not reflect changes or amendments subsequent to this date and time. The NFHL and effective information may change or become superseded by new data over time.

This map image is void if the one or more of the following map elements do not appear: basemap imagery, flood zone labels, legend, scale bar, map creation date, community identifiers, FIRM panel number, and FIRM effective date. Map images for unmapped and unmodernized areas cannot be used for regulatory purposes.

Basemap: USGS National Map: Orthoimagery: Data refreshed October, 2020

FEMA

Feet
0 250 500 1,000 1,500 2,000

1:6,000

FLOOD INSURANCE RATE MAP

FENAGH ENGINEERING AND TESTING	Patriot High School 4355 Camino Real, Jurupa Valley FILE NO. 6484	PLATE 11

APPENDIX A
FIELD EXPLORATION

Collection of Field Samples

Bulk Samples

Bulk samples of representative earth materials were obtained from the exploratory borings

The Standard Penetration Test (SPT) Sampler

Disturbed drive soil samples were obtained by means of a Standard Penetration Test sampler. The sampler is composed of a split barrel with an external diameter of 2 inches and an unlined internal diameter of 1-3/8 inches. The sampler was driven into the ground 6 to 18 inches with a 140-pound hammer free-falling from a height of 30 inches in general accordance with ASTM D1586. The blow counts were recorded for every 6 inches of penetration; the blow counts reported on the logs are those for the last 12 inches of penetration.

Modified Split Barrel Samplers

Relatively undisturbed soil samples were obtained using split barrel soil samplers with external diameters of 3.0 inches, and 2.5 inches both lined with 1-inch tall thin brass rings with inside diameters of approximately 2.4 inches. The sample barrels were driven into the ground with the weight of a hammer in general accordance with ASTM D3550.

Drilling Contractor:

Choice Drilling, Inc.
11029 Sutter Avenue
Pacoima, CA 91331

Borings Logged by:

Clarissa Jones, Fenagh Engineering and Testing

Borings Checked by:

Scott T. Prince, Fenagh Engineering and Testing

BORING LOG NUMBER 1

Patriot High School Jurupa Valley **Date: 08/03/22**

File No. 6484 **Method: 8-inch diameter Hollow Stem Auger**

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Barren Ground w/ 6” Organic Topsoil
				0		Organic Topsoil
				1		FILL: Clayey Sand, Reddish and Grayish Brown, Moist, Very Dense, Fine Grained.
2.5	64	10.3	129.3	2.5		
5	90	14.2	122.0	5	SC	OLDER ALLUVIUM: Clayey Sand, Reddish Brown, Moist, Very Dense, Fine Grained, Caliche
7.5	50-4"	6.7	133.2	7.5		
10	84	5.0	131.3	10		
15	50-2"	11.3	121.4	15	SC/CL	Clayey Sand to Sandy and Silty Clay, Reddish Brown, Very Dense, Very Stiff, Fine Grained
20	50-6"	7.3	125.2	20	SC	Clayey Sand, Reddish and Grayish Brown, Moist, Very Dense, Fine Grained
				24	SC/CL	Clayey Sand to Sandy and Silty Clay, Reddish and Grayish Brown, Moist, Very Dense, Very Stiff, Fine Grained
25	50-5"	9.3	129.0	25		Total Depth: 25 Feet Fill to 5 Feet, No Water Encountered

BORING LOG NUMBER 2

Patriot High School Jurupa Valley | **Date: 08/03/22**

File No. 6484 | **Method: 8-inch diameter Hollow Stem Auger**

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Asphalt 4", No Base
				0		Asphalt 4"
				1		FILL: Clayey Sand, Grayish Brown, Moist, Dense, Fine Grained.
2.5	29	14.6	123.4	2–3		
5	50-3"	9.8	127.3	4–5	SC	OLDER ALLUVIUM: Clayey Sand, Reddish Brown, Moist, Very Dense, Fine Grained
7.5	50-6"	10.4	130.0	8		Caliche
10	50-6"	12.7	110.0	10	SC/CL	Clayey Sand to Sandy and Silty Clay, Reddish Brown, Very Dense, Very Stiff, Fine Grained
15	50-3"	9.9	122.2	15	SC	Clayey Sand, Reddish to Grayish Brown, Moist, Very Dense, Fine Grained
20	42 50-5"	10.2	129.7	20	SC/CL	Clayey Sand to Sandy and Silty Clay, Reddish Brown Moist, Very Dense, Very Stiff, Fine Grained
				24		Red to Greyish Brown
25	50-6"	10.9	127.3	25		Total Depth: 25 Feet Fill to 4 Feet, No Water Encountered

Fenagh Engineering & Testing

BORING LOG NUMBER 3

Patriot High School Jurupa Valley | **Date: 08/03/22**

File No. 6484 | **Method: 8-inch diameter Hollow Stem Auger**

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Asphalt 4", No Base
				0		Asphalt 4”
				1–3		FILL: Clayey Sand, Grayish Brown, Moist, Dense, Fine Grained.
2.5	26 50-5"	9.9	132.2			
				3–10	SC	OLDER ALLUVIUM: Clayey Sand, Reddish Brown, Moist Very Dense, Fine Grained
5	77	8.0	136.4			
7.5	50-3"	9.6	108.0			
10	50-4"	11.1	108.7	10–15	SC/CL	Clayey Sand to Sandy and Silty Clay, Reddish Brown, Very Dense, Very Stiff, Fine Grained
15	50-4"	10.7	131.0	15–20	SC	Clayey Sand, Reddish and Grayish Brown, Moist, Very Dense, Fine Grained
20	50-3"	10.4	130.4	20–25	SC/CL	Clayey Sand to Sandy and Silty Clay, Reddish and Grayish Brown, Moist, Very Dense, Very Stiff, Fine Grained
25	50-6"	10.8	128.3	25		Total Depth: 25 Feet Fill to 3 Feet, No Water Encountered

Fenagh Engineering & Testing

BORING LOG NUMBER 4

Patriot High School Jurupa Valley — **Date: 08/03/22**

File No. 6484 — **Method: 8-inch diameter Hollow Stem Auger**

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Barren Ground
				0 --		FILL: Clayey Sand, Grayish Brown, Moist, Dense, Fine Grained, Cobble Fragment
				1 --		
				2 --		
2.5	50-6"	10.8	130.2	-		
				3 --		
				4 --		
				-	SC	OLDER ALLUVIUM: Clayey Sand, Reddish Brown, Moist Very Dense, Fine Grained
5	50-3"	12.3	122.6	5 --		
				6 --		
				7 --		
7.5	50-6"	10.4	126.1	-		
				8 --		
				9 --		
10	50-6"	8.8	133.9	10 --		
				-	CL/SC	Sandy Clay to Clayey Sand, Reddish Brown, Very Dense, Very Stiff, Fine Grained. Caliche
				11 --		
				12 --		
				13 --		
				14 --		
15	50-6"	9.9	128.5	15 --		
				-		Cobble
				16 --		
				17 --		
				18 --		
				19 --		
20	50-6"	10.4	124.2	20 --		
				-	CL	Sandy Clay, Reddish and Grayish Brown, Moist, Very Dense, Fine Grained, Clayey Sand layer 2" rock discovered
				21 --		
				22 --		
				23 --		
				24 --	CL/SC	Sandy Clay to Clayey Sand, Reddish and Grayish Brown, Moist, Very Dense, Very Stiff, Fine Gravel
25	50'6"	13.6	116.7	25 --		
				-		Total Depth: 25 Feet Fill to 4 Feet, No Water Encountered

Fenagh Engineering & Testing

BORING LOG NUMBER 5

Patriot High School Jurupa Valley **Date: 08/04/22**

File No. 6484 **Method: 8-inch diameter Hollow Stem Auger**

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Concrete 5 1/2", No Base
				0		Concrete 5 1/2"
				1		FILL: Clayey Sand, Grayish Brown, Moist, Dense, Fine Grained, Caliche
				2		
2.5	50-4"	7.8	123.6			
				3	SC	OLDER ALLUVIUM: Clayey Sand, Reddish Brown, Moist, Very Dense, Fine Grained
				4		
5	50-6"	9.5	123.3	5		
				6		
				7		
7.5	42 50-5"	9.3	127.4			
				8		
				9		
10	50-6"	11.0	119.1	10	SC/CL	Clayey Sand to Sandy and Silty Clay, Reddish Brown, Moist Very Dense, Very Stiff, Fine Grained, Caliche
				11		
				12		
				13		
				14		
15	50-6"	9.0	126.0	15		
				16		
				17		
				18		
				19		
20	45 50-5"	5.3	122.2	20	SP-SM	Poorly Graded Sand with Silt, Reddish to Brown, Moist, Very Dense, Fine to Medium Grained
				21		
				22		
				23		
				24	SC	Clayey Sand, Reddish Brown, Moist, Very Dense, Fine Grained
25	50-6"	6.9	116.9	25		Total Depth: 25 Feet Fill to 3 Feet, No Water Encountered

Fenagh Engineering & Testing

BORING LOG NUMBER 6

Patriot High School Jurupa Valley **Date: 08/04/22**

File No. 6484 **Method: 8-inch diameter Hollow Stem Auger**

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Concrete 5 1/2", No Base
				0		Concrete 5"
				1		FILL: Clayey Sand, Tan to Reddish Brown, Dry to Moist Very Dense, Fine Grained, Organics, Few Coarse Gravel
				2		
2.5	50-6"	7.0	109.8			
				3	SC/CL	OLDER ALLUVIUM: Clayey Sand to Sandy and Silty Clay Reddish Brown, Moist, Very Dense, Very Stiff, Fine Grained Few Coarse Gravel
				4		
5	50-4"	9.0	117.5	5		
				6		
				7		
7.5	50-6"	8.6	120.8			
				8		
				9		
10	70	9.6	126.0	10	SC	Clayey Sand, Reddish Brown, Moist, Very Dense, Fine Grained Caliche
				11		
				12		
				13		
				14		
15	50-6"	6.7	124.6	15	SC/CL	Clayey Sand to Sandy and Silty Clay, Reddish Brown, Moist, Very Dense, Very Stiff, Fine Grained
				16		
				17		
				18		
				19		
20	50-6"	2.5	123.9	20	SP-SM	Poorly Graded Sand with Silt, Reddish Brown, Moist, Very Dense, Medium to Fine Grained, Fine Gravel
				21		
				22		
				23		
				24	SC	Clayey Sand, Reddish Brown, Moist, Dense, Fine Grained
25	50-6"	6.2	126.5	25		Total Depth: 25 Feet Fill to 3 Feet, No Water Encountered

Fenagh Engineering & Testing

BORING LOG NUMBER 7

Patriot High School Jurupa Valley **Date: 08/04/22**

File No. 6484 **Method: 8-inch diameter Hollow Stem Auger**

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Barren Ground
				0		FILL: Clayey Sand, Tan to Reddish Brown, Dry to Moist Very Dense, Fine Grained, Organics
				1		
				2		
2.5	50-6"	7.0	109.9			
				3		
				4	SC	OLDER ALLUVIUM: Clayey Sand, Reddish Brown, Moist, Very Dense, Fine Grained
5	50-4"	8.0	134.7	5		
				6		
				7		
7.5	50-6"	7.5	134.6			Caliche
				8		
				9		
10	70	7.9	132.7	10		
				11		
				12		
				13		
				14		
15	50-6"	6.0	115.0	15	SW/SM	Well-Graded Sand with Silt and Silty Sand, Reddish Brown, Moist, Very Dense, Fine Grained
				16		
				17		
				18		
				19		
20	50-6"	5.4	119.7	20	SC/CL	Clayey Sand to Sandy and Silty Clay, Reddish Brown, Moist, Very Dense, Very Stiff, Fine Grained, Caliche, Clay layer @ 20'
				21		
				22		
				23		
				24	SC	Clayey Sand, Reddish Brown, Moist, Very Dense Fine Grained, Caliche
25	50-6"	5.7	115.0	25		Total Depth: 25 Feet Fill to 4 Feet, No Water Encountered

Fenagh Engineering & Testing

BORING LOG NUMBER 8

Patriot High School Jurupa Valley | **Date: 08/04/22**

File No. 6484 | **Method: 8-inch diameter Hollow Stem Auger**

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Barren Ground
				0		FILL: Clayey Sand, Reddish Brown, Moist, Dense, Fine Grained
				1		
				2		
2.5	50-6"	4.8	133.0			
				3		
				4	SC/CL	OLDER ALLUVIUM: Clayey Sand to Sandy and Silty Clay, Reddish Brown, Moist, Very Dense, Very Stiff, Fine Grained
5	50-3"	11.3	120.3	5		
				6		
				7		
7.5	50-3"	8.5	124.9		SC	Clayey Sand, Reddish Brown, Moist, Very Dense Fine Grained
				8		
				9		
10	39-4"	8.2	124.6	10		Caliche
				11		
				12		
				13		
				14		
15	50-2"	4.0	124.9	15	SC/CL	Clayey Sand to Sandy and Silty Clay, Reddish Brown, Moist, Very Dense, Very Stiff, Fine Grained, Few Fine Gravel
				16		
				17		
				18		
				19		
20	50-5"	3.8	125.0	20	SC	Clayey Sand, Reddish Brown, Moist, Very Dense, Fine Grained
				21		
				22		
				23		
				24	SP-SM	Poorly Graded Sand with Silt, Reddish to Greyish Brown, Moist, Very Dense, Fine to Medium Grained
25	50-3"	2.6	115.2	25		Total Depth: 25 Feet Fill to 4 Feet, No Water Encountered

Fenagh Engineering & Testing

BORING LOG NUMBER 9

Patriot High School Jurupa Valley **Date: 08/04/22**

File No. 6484 **Method: 8-inch diameter Hollow Stem Auger**

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Barren Ground
				0 --		FILL: Clayey Sand, Reddish Brown, Moist, Dense, Fine Grained
				1 --		
				2 --		
2.5	50-3"	8.7	128.9	-	SC/CL	OLDER ALLUVIUM: Clayey Sand to Sandy and Silty Clay, Reddish Brown, Moist, Very Dense, Very Stiff, Fine Grained
				3 --		
				4 --		
5	50-3"	5.8	132.1	5 --		Caliche
				6 --		
				7 --		
7.5	50-5"	6.6	129.4	-		Cobble Encountered
				8 --		
				9 --		
10	50-5"	5.3	133.5	10 --		Caliche
				11 --		
				12 --		
				13 --		
				14 --		
15	50-2"	9.5	118.5	15 --		
				16 --		
				17 --		
				18 --		
				19 --		
20	50-6"	8.3	121.1	20 --		Mottled w/ Grayish Brown Few Fine Gravel
				21 --		
				22 --		
				23 --		
				24 --		Few Coarse Gravel
25	50-2"	11.3	128.8	25 --		Total Depth: 25 Feet Fill to 2 Feet, No Water Encountered

Fenagh Engineering & Testing

BORING LOG NUMBER 10

Patriot High School Jurupa Valley **Date: 08/04/22**

File No. 6484 **Method: 8-inch diameter Hollow Stem Auger**

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Barren Ground
				0 --		FILL: Clayey Sand, Reddish Brown, Moist, Very Dense Fine Grained
				-		
				1 --		
				-		
				2 --		
2.5	84	6.5	133.7	-		
				3 --	SC	OLDER ALLUVIUM: Clayey Sand, Reddish Brown, Moist, Very Dense, Fine Grained
				-		
				4 --		
				-		
5	29	7.9	SPT	5 --		
				-		
				6 --		
				-		
				7 --		
7.5	69	7.6	122.6	-		
				8 --	SM	Silty Sand, Dark Reddish Brown, Caliche
				-		
				9 --		
				-		
10	22	4.0	SPT	10 --		
				-	SW/SM	Well-Graded Sand with Silt and Silty Sand, Reddish Brown, Moist, Very Dense, Fine Grained
				11 --		
				-		
				12 --		
12.5	50-5"	8.7	125.8	-		
				13 --	SC/CL	Clayey Sand to Sandy and Silty Clay, Reddish Brown, Moist Very Dense, Very Stiff, Fine Grained
				-		
				14 --		
				-		
15	50'6"	9.2	SPT	15 --		
				-		
				16 --		
				-		
				17 --		
17.5	50-6"	8.1	129.0	-		
				18 --	SC	Clayey Sand, Reddish Brown, Moist, Very Dense Fine Grained
				-		
				19 --		
				-		
20	58	7.4	SPT	20 --		
				-		
				21 --		
				-		
				22 --		
22.5	50-6"	6.8	121.7	-		
				23 --		
				-		
				24 --		
				-		
25	72	7.3	SPT	25 --		
				-		

Fenagh Engineering & Testing

BORING LOG NUMBER 10

Patriot High School Jurupa Valley

File No. 6484

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				-	SC	Clayey Sand, Reddish Brown, Moist, Very Dense
				26 --		Fine Grained
				-		
				27 --		
27.5	50-6"	7.1	123.3	-		
				28 --		Yellowish to Reddish Brown
				-		
				29 --		
				-		
30	48	5.5	SPT	30 --		
				-		
				31 --		
				-		
				32 --		
32.5	50-5"	6.3	128.0	-		
				33 --		
				-		
				34 --		
				-		
35	50-5"	9.3	SPT	35 --		
				-	SC/SM	Clayey Sand to Silty Sand w/ Gravel, Dark Yellowish Brown
				36 --		Very Moist, Fine Grained, Very Dense, Sharp transition to
				-		Gravelly Sand
				37 --		
37.5	50-4"	5.4	122.7	-		
				38 --	SW/SM	Well-Graded Sand to Silty Sand, Grayish Brown,
				-		to Dark Yellowish Brown, Moist, Some Rock to
				39 --		Fine Grain, Dense
				-		
40	65	13.4	SPT	40 --		
				-	SM	Silty Sand, Dark Yellowish Brown, Moist, Very Dense,
				41 --		Fine Grained
				-		
				42 --		
42.5	50-6"	11.2	128.6	-		
				43 --		Yellowish and Reddish Brown w/ Gray Mottling
				-		Very Moist, Fine to Medium Grained, Pyritic
				44 --		
				-		
45	50-6"	15.4	SPT	45 --		
				-		WET
				46 --		
				-		
				47 --		
47.5	50-6"	16.5	119.9	-		
				48 --		Reddish and Grayish Brown
				-		
				49 --		
				-		
50	62	18.0	SPT	50 --		
				-		Total Depth: 50 Feet
						Fill to 2.5 Feet, Water at 45’

BORING LOG NUMBER 11

Patriot High School Jurupa Valley — **Date: 08/04/22**

File No. 6484 — **Method: 8-inch diameter Hollow Stem Auger**

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Barren Ground
				0		FILL: Clayey Sand, Reddish Brown, Moist, Very Stiff, Fine Grained
				1		
				2		
2.5	81	4.8	127.5			
				3	SC/CL	OLDER ALLUVIUM: Clayey Sand to Sandy and Silty Clay, Reddish Brown, Moist, Very Dense, Very Stiff, Fine Grained
				4		
5	66	6.4	133.9	5		
				6		
				7		
7.5	50-6"	10.3	131.2			
				8		Caliche
				9		
10	50-3"	10.2	133.0	10		
				11		
				12		
				13		
				14		
15	50-2"	11.8	120.9	15		
				16		
				17		Cobbles
				18		
				19		
20	50-6"			20		No Recovery
				21		
				22		
				23		
				24	SC	Clayey Sand, Reddish Brown, Moist, Very Dense, Fine Grained,
25	50'6"	10.3	120.9	25		
						Total Depth: 25 Feet Fill to 2.5 Feet, No Water Encountered

Fenagh Engineering & Testing

APPENDIX B
LABORATORY TESTING

Classification
Soils were classified in accordance with the Unified Soil Classification System (USCS) in general accordance with ASTM D2488 and modified as necessary in general accordance with ASTM D2487 based on laboratory results. The classifications are indicated on the boring logs in Appendix A.

In-Place Moisture and Density Tests
The moisture content and dry density of relatively undisturbed samples obtained from the exploratory borings were evaluated in general accordance with ASTM D2937. The test results are indicated on the boring logs in Appendix A.

Particle Size Distribution
Gradation analysis testing was performed on selected representative soil samples in general accordance with ASTM D422.

Compaction Test
Maximum density compaction tests are performed on representative samples to determine compactive characteristics and used to evaluate the relative compaction of in-place earth materials. Compaction tests are performed in accordance with the latest version of ASTM D1557.

Expansion Index
Expansion Index testing was performed on selected representative soil samples in general accordance with ASTM D4829.

R-Value
Resistance Value (R-Value) testing by Stabilometer was performed on a select representative soil sample in general accordance with CT301.

Direct Shear Test
Direct single-shear tests were performed on representative undisturbed and remolded samples to determine strength properties. Loads are applied in increasing load increments and results recorded. Soil samples were inundated to replicate saturated soil conditions. Results are plotted and provided on lab results sheets. Shear Tests were performed in accordance with the latest version of ASTM D3080.

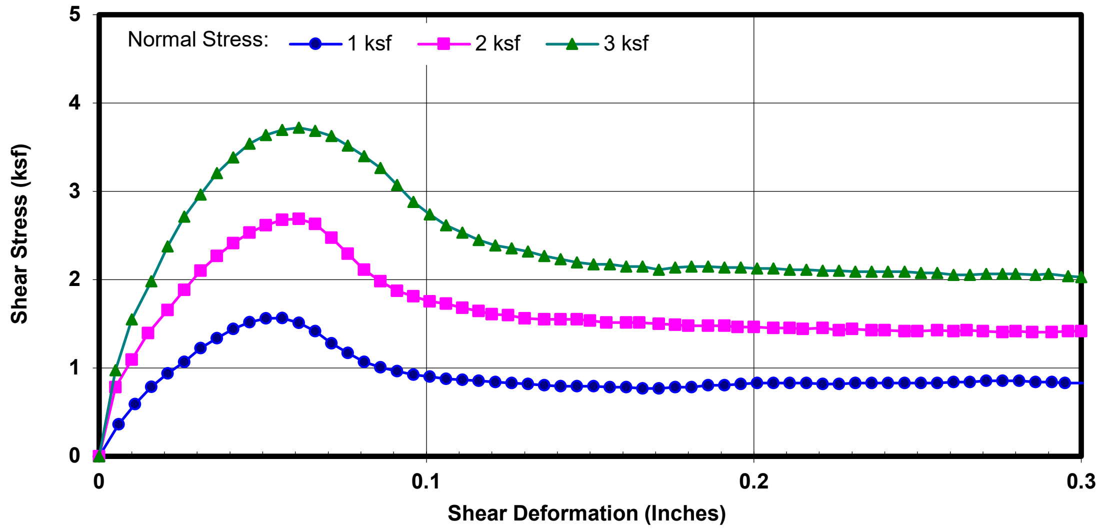
Consolidation
Consolidation tests were performed on undisturbed samples to predict the soils compressive properties under specific load conditions. Confinement loads are applied in increasing increments and results recorded. Samples are inundated at 2,000 pounds per square foot to evaluate saturation response of soils. Results of test are provided on consolidation curve sheets. Tests are performed in accordance with the latest version of ASTM D2435.

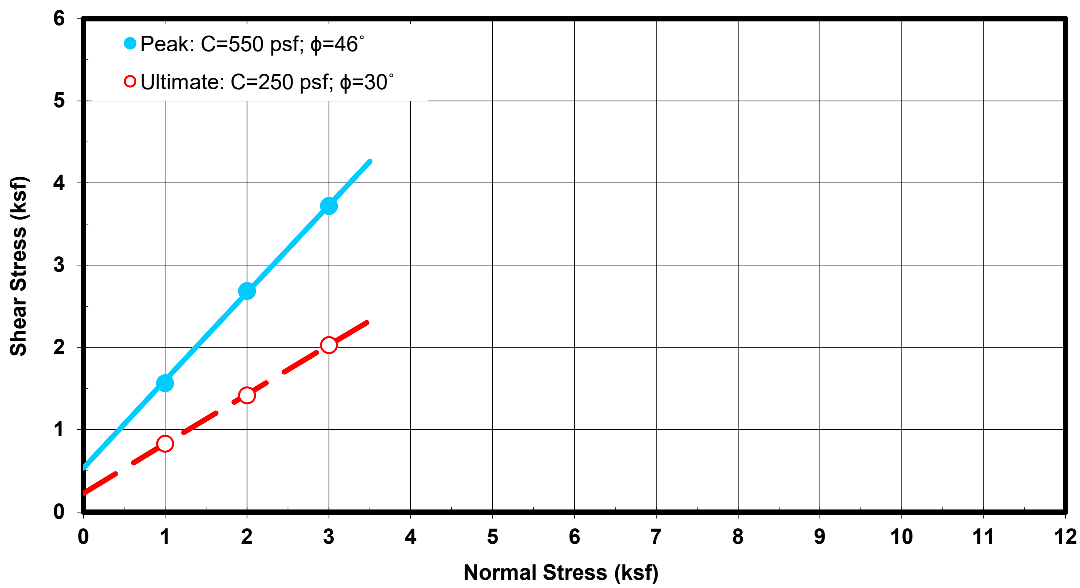
AP Engineering and Testing, Inc.
DBE|MBE|SBE
2607 Pomona Boulevard | Pomona, CA 91768
t. 909.869.6316 | f. 909.869.6318 | www.aplaboratory.com

DIRECT SHEAR TEST RESULTS
ASTM D 3080

Client:	Fenagh Engineering & Testing	**Tested By:**	ST	**Date:**	08/19/22
Project Name:	Patriot H.S. Stadium Improvements	**Computed By:**	JP	**Date:**	08/22/22
Project No.:	6484	**Checked by:**	AP	**Date:**	08/22/22
Boring No.:	B1				
Sample No.:	- **Depth (ft):** 10				
Sample Type:	Mod. Cal.				
Soil Description:	Clayey Sand				
Test Condition:	Inundated **Shear Type:** Regular				

Wet Unit Weight (pcf)	Dry Unit Weight (pcf)	Initial Moisture Content (%)	Final Moisture Content (%)	Initial Degree Saturation (%)	Final Degree Saturation (%)	Normal Stress (ksf)	Peak Shear Stress (ksf)	Ultimate Shear Stress (ksf)
133.1	125.6	5.9	12.6	47	100	1	1.565	0.828
						2	2.688	1.416
						3	3.720	2.028





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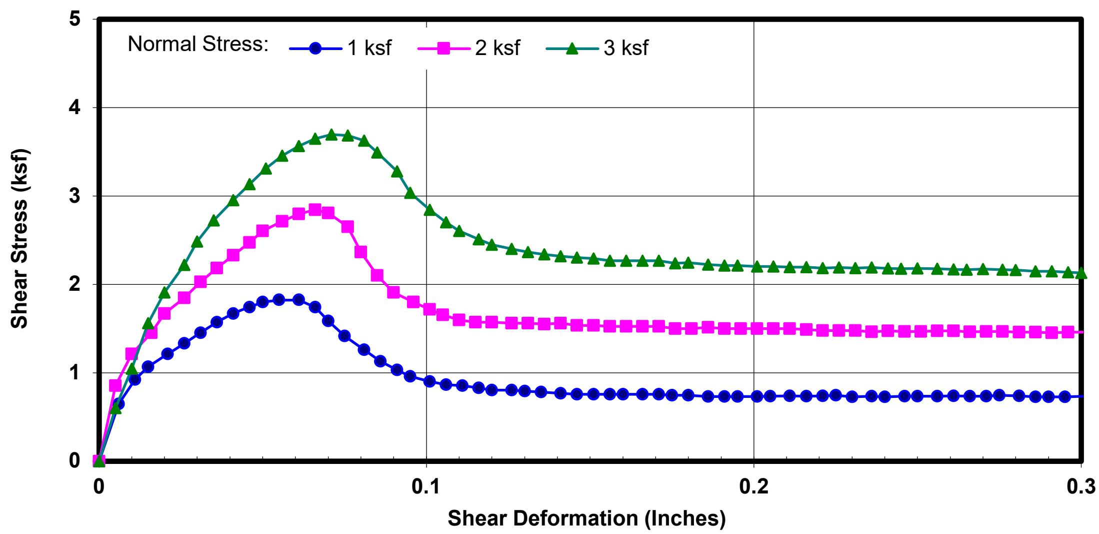
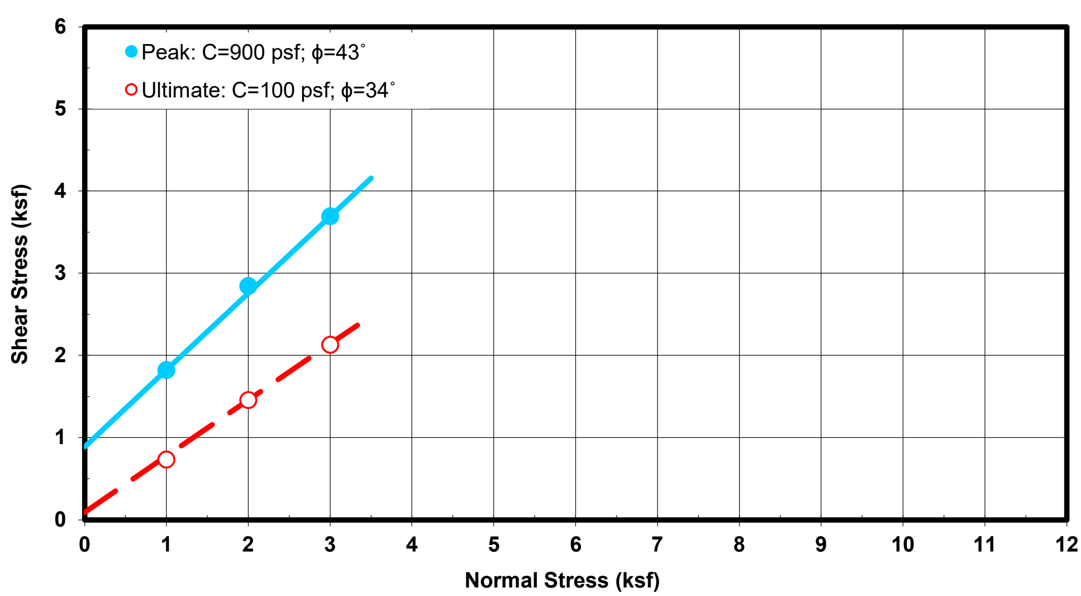
DIRECT SHEAR TEST RESULTS
ASTM D 3080

Client: Fenagh Engineering & Testing
Project Name: Patriot H.S. Stadium Improvements
Project No.: 6484
Boring No.: B3
Sample No.: - **Depth (ft):** 15
Sample Type: Mod. Cal.
Soil Description: Clayey Sand
Test Condition: Inundated **Shear Type:** Regular

Tested By: ST **Date:** 08/19/22
Computed By: JP **Date:** 08/22/22
Checked by: AP **Date:** 08/22/22

Wet Unit Weight (pcf)	Dry Unit Weight (pcf)	Initial Moisture Content (%)	Final Moisture Content (%)	Initial Degree Saturation (%)	Final Degree Saturation (%)	Normal Stress (ksf)	Peak Shear Stress (ksf)	Ultimate Shear Stress (ksf)
141.7	128.0	10.7	11.6	92	99	1	1.824	0.734
						2	2.844	1.457
						3	3.696	2.128





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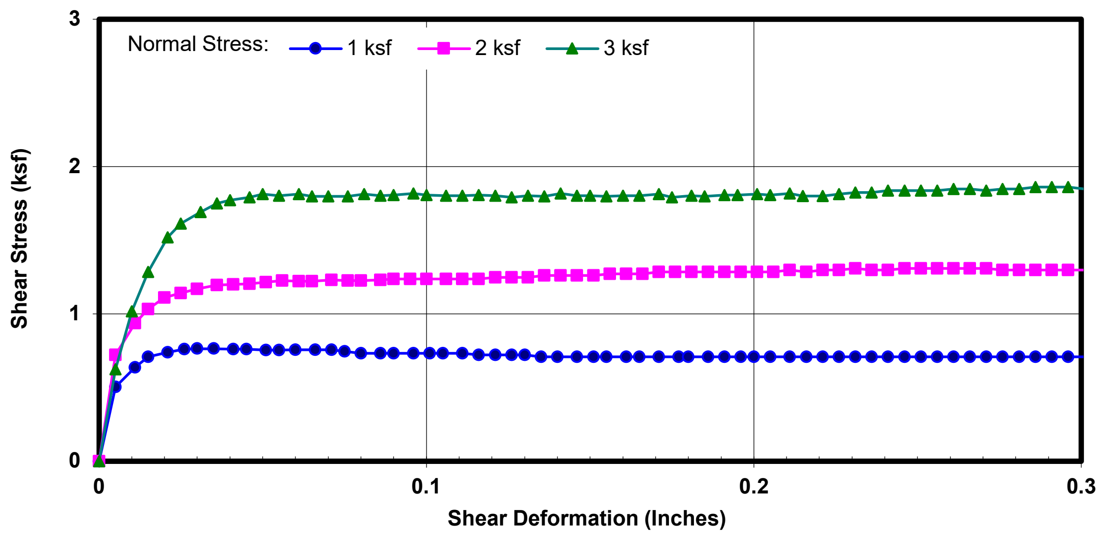
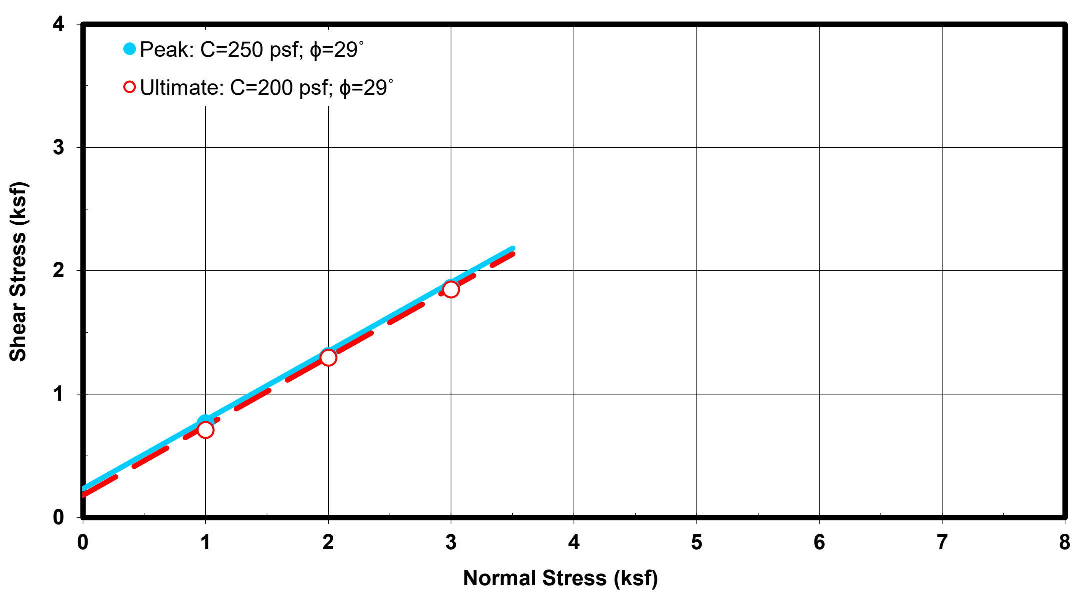
DIRECT SHEAR TEST RESULTS
ASTM D 3080

Client:	Fenagh Engineering & Testing	**Tested By:**	LS	**Date:**	08/20/22
Project Name:	Patriot H.S. Stadium Improvements	**Computed By:**	JP	**Date:**	08/22/22
Project No.:	6484	**Checked by:**	AP	**Date:**	08/22/22
Boring No.:	B4				
Sample Type:	Bulk **Depth (ft):** 1-5				
Remold Cond.:	Remolded to 90% RC at opt. MC				
Soil Description:	Clayey Sand				
Test Condition:	Inundated **Shear Type:** Regular				

Wet Unit Weight (pcf)	Dry Unit Weight (pcf)	Initial Moisture Content (%)	Final Moisture Content (%)	Initial Degree Saturation (%)	Final Degree Saturation (%)	Normal Stress (ksf)	Peak Shear Stress (ksf)	Ultimate Shear Stress (ksf)
129.5	118.6	9.1	15.2	59	98	1	0.764	0.708
						2	1.308	1.296
						3	1.860	1.848





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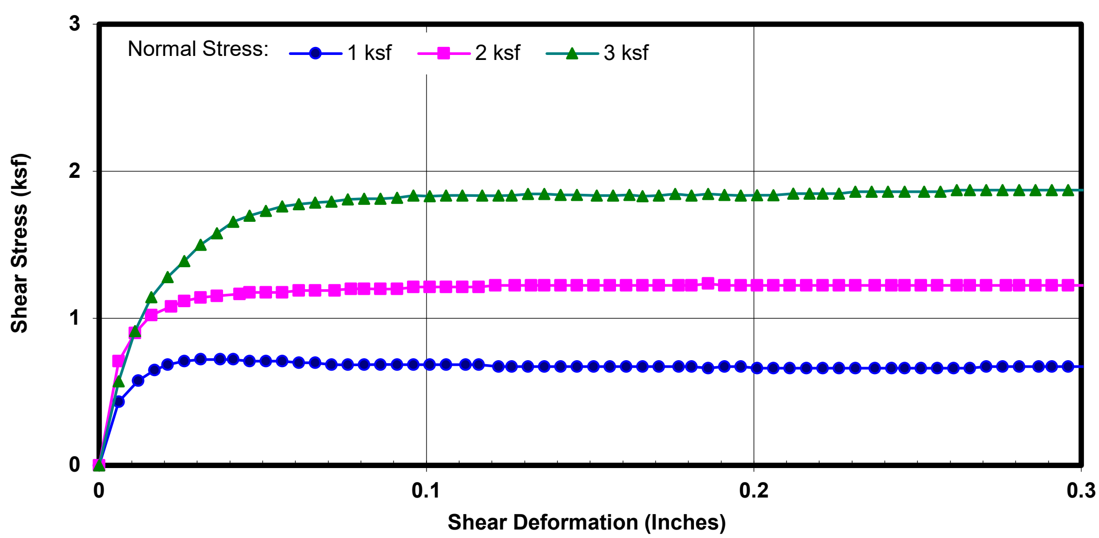
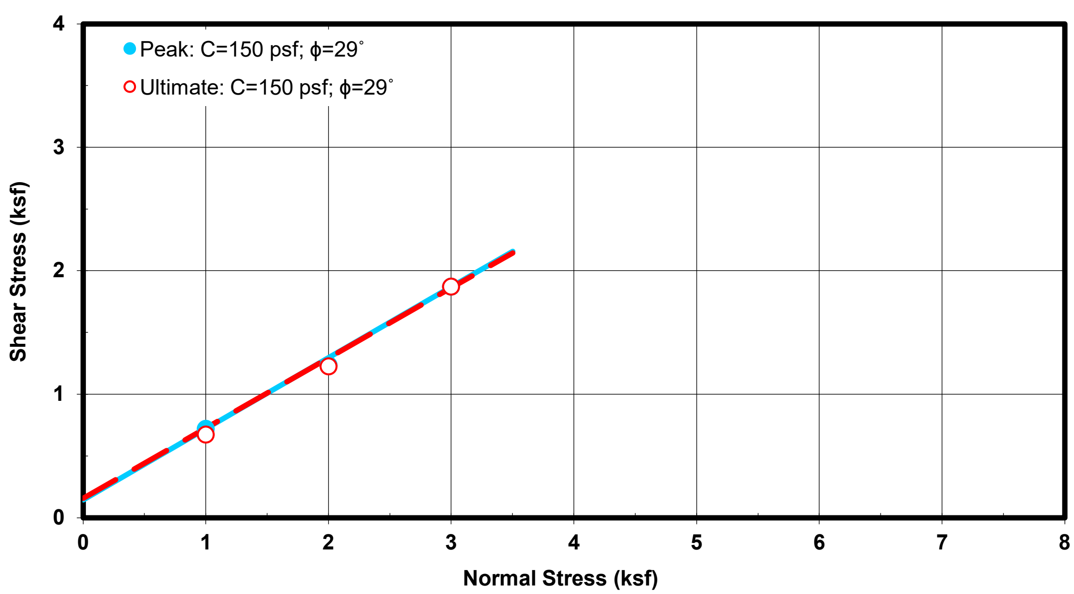
DIRECT SHEAR TEST RESULTS
ASTM D 3080

Client:	Fenagh Engineering & Testing	**Tested By:**	LS	**Date:**	08/20/22
Project Name:	Patriot H.S. Stadium Improvements	**Computed By:**	JP	**Date:**	08/22/22
Project No.:	6484	**Checked by:**	AP	**Date:**	08/22/22
Boring No.:	B5				
Sample Type:	Bulk **Depth (ft):** 1-5				
Remold Cond.:	Remolded to 90% RC at opt. MC				
Soil Description:	Clayey Sand				
Test Condition:	Inundated **Shear Type:** Regular				

Wet Unit Weight (pcf)	Dry Unit Weight (pcf)	Initial Moisture Content (%)	Final Moisture Content (%)	Initial Degree Saturation (%)	Final Degree Saturation (%)	Normal Stress (ksf)	Peak Shear Stress (ksf)	Ultimate Shear Stress (ksf)
127.0	116.1	9.3	16.7	56	100	1	0.720	0.672
						2	1.236	1.224
						3	1.872	1.872





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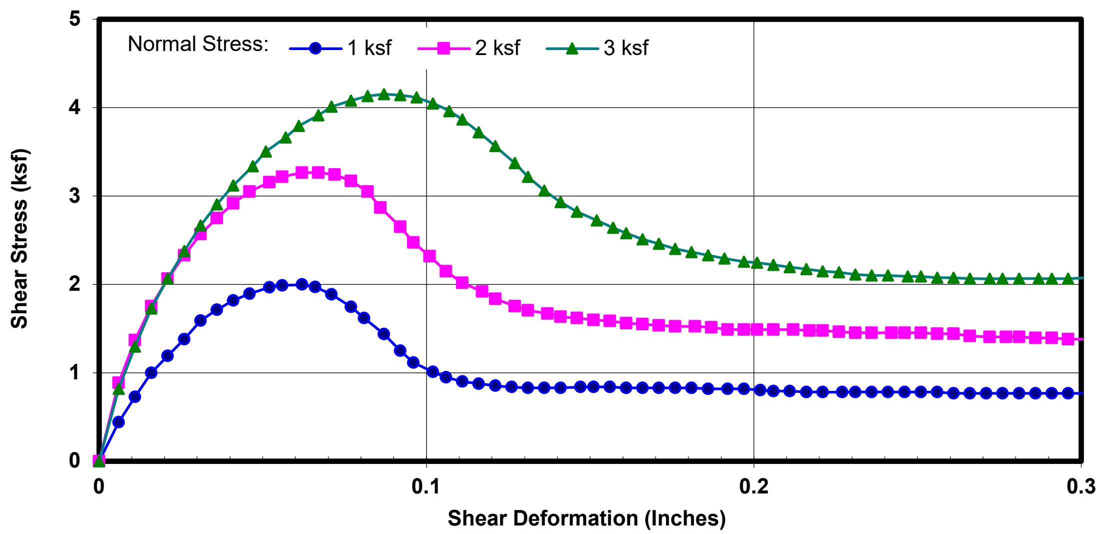
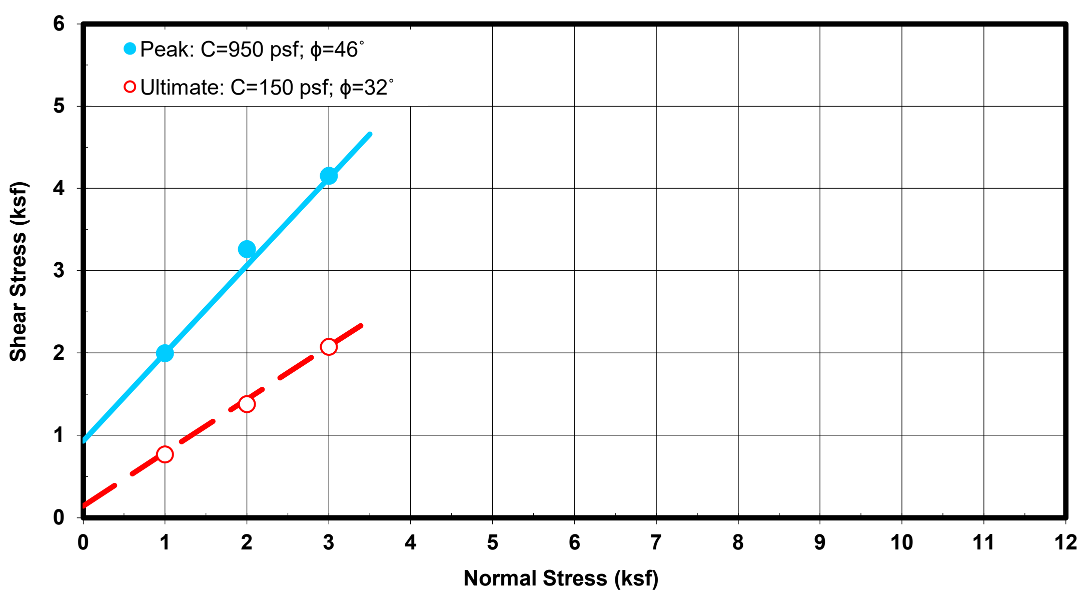
DIRECT SHEAR TEST RESULTS
ASTM D 3080

Client: Fenagh Engineering & Testing
Project Name: Patriot H.S. Stadium Improvements
Project No.: 6484
Boring No.: B7
Sample No.: - **Depth (ft):** 5
Sample Type: Mod. Cal.
Soil Description: Clayey Sand
Test Condition: Inundated **Shear Type:** Regular

Tested By: ST **Date:** 08/19/22
Computed By: JP **Date:** 08/22/22
Checked by: AP **Date:** 08/22/22

Wet Unit Weight (pcf)	Dry Unit Weight (pcf)	Initial Moisture Content (%)	Final Moisture Content (%)	Initial Degree Saturation (%)	Final Degree Saturation (%)	Normal Stress (ksf)	Peak Shear Stress (ksf)	Ultimate Shear Stress (ksf)
145.1	134.7	7.7	9.2	83	99	1	1.997	0.768
						2	3.264	1.380
						3	4.152	2.076





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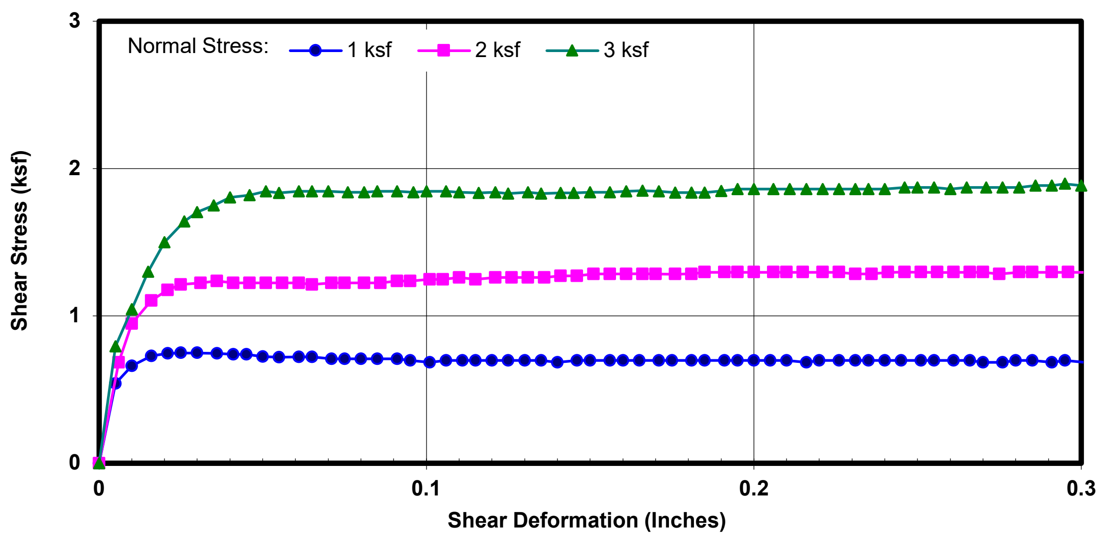
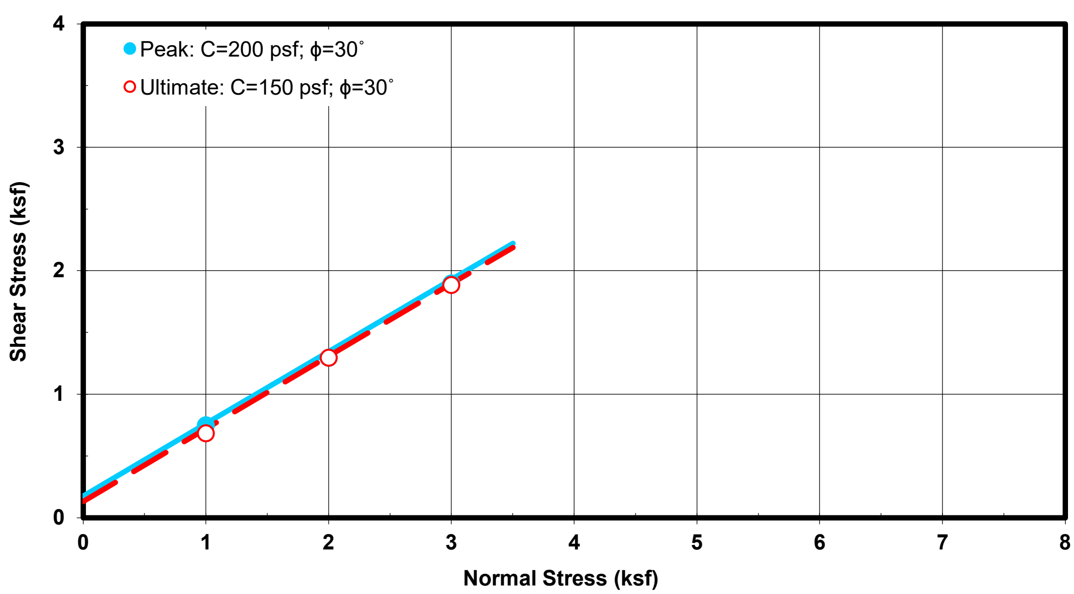
DIRECT SHEAR TEST RESULTS
ASTM D 3080

Client: Fenagh Engineering & Testing
Project Name: Patriot H.S. Stadium Improvements
Project No.: 6484
Boring No.: B10
Sample Type: Bulk **Depth (ft):** 1-5
Remold Cond.: Remolded to 90% RC at opt. MC
Soil Description: Clayey Sand
Test Condition: Inundated **Shear Type:** Regular

Tested By: LS **Date:** 08/20/22
Computed By: JP **Date:** 08/22/22
Checked by: AP **Date:** 08/22/22

Wet Unit Weight (pcf)	Dry Unit Weight (pcf)	Initial Moisture Content (%)	Final Moisture Content (%)	Initial Degree Saturation (%)	Final Degree Saturation (%)	Normal Stress (ksf)	Peak Shear Stress (ksf)	Ultimate Shear Stress (ksf)
129.9	120.3	8.0	14.7	54	99	1	0.749	0.684
						2	1.296	1.296
						3	1.896	1.884





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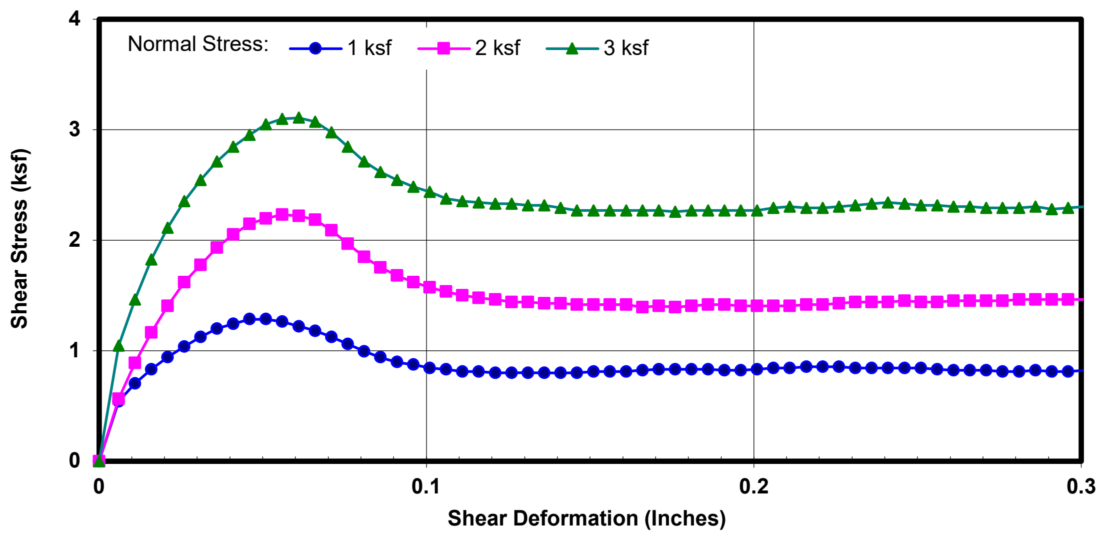
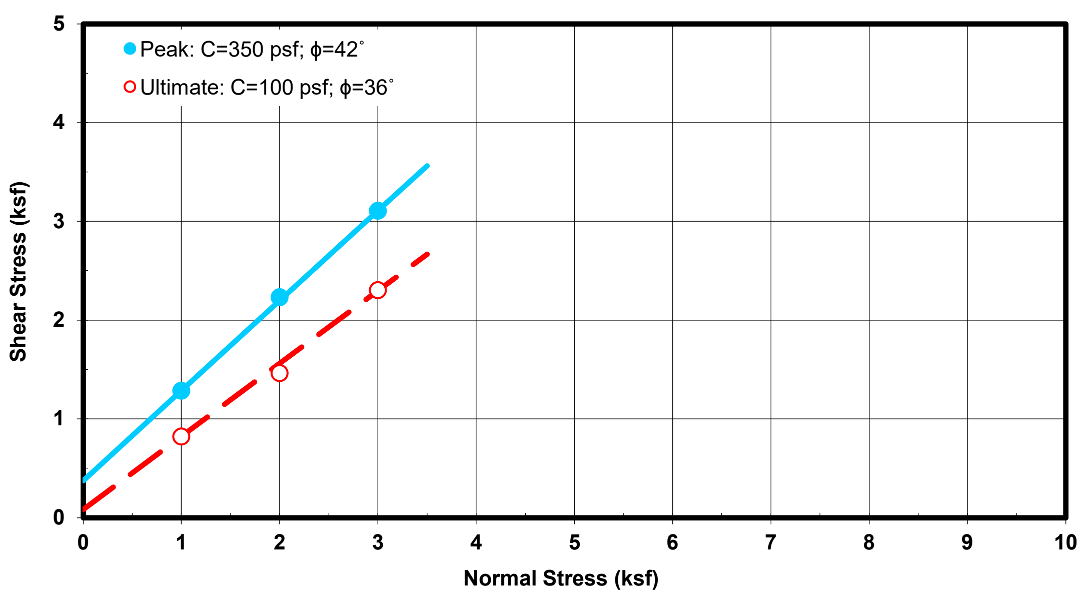
DIRECT SHEAR TEST RESULTS
ASTM D 3080

Client: Fenagh Engineering & Testing
Project Name: Patriot H.S. Stadium Improvements
Project No.: 6484
Boring No.: B10
Sample No.: - **Depth (ft):** 7.5
Sample Type: Mod. Cal.
Soil Description: Silty Sand
Test Condition: Inundated **Shear Type:** Regular

Tested By: ST **Date:** 08/19/22
Computed By: JP **Date:** 08/22/22
Checked by: AP **Date:** 08/22/22

Wet Unit Weight (pcf)	Dry Unit Weight (pcf)	Initial Moisture Content (%)	Final Moisture Content (%)	Initial Degree Saturation (%)	Final Degree Saturation (%)	Normal Stress (ksf)	Peak Shear Stress (ksf)	Ultimate Shear Stress (ksf)
127.9	118.2	8.2	15.7	52	100	1	1.285	0.821
						2	2.232	1.464
						3	3.108	2.304





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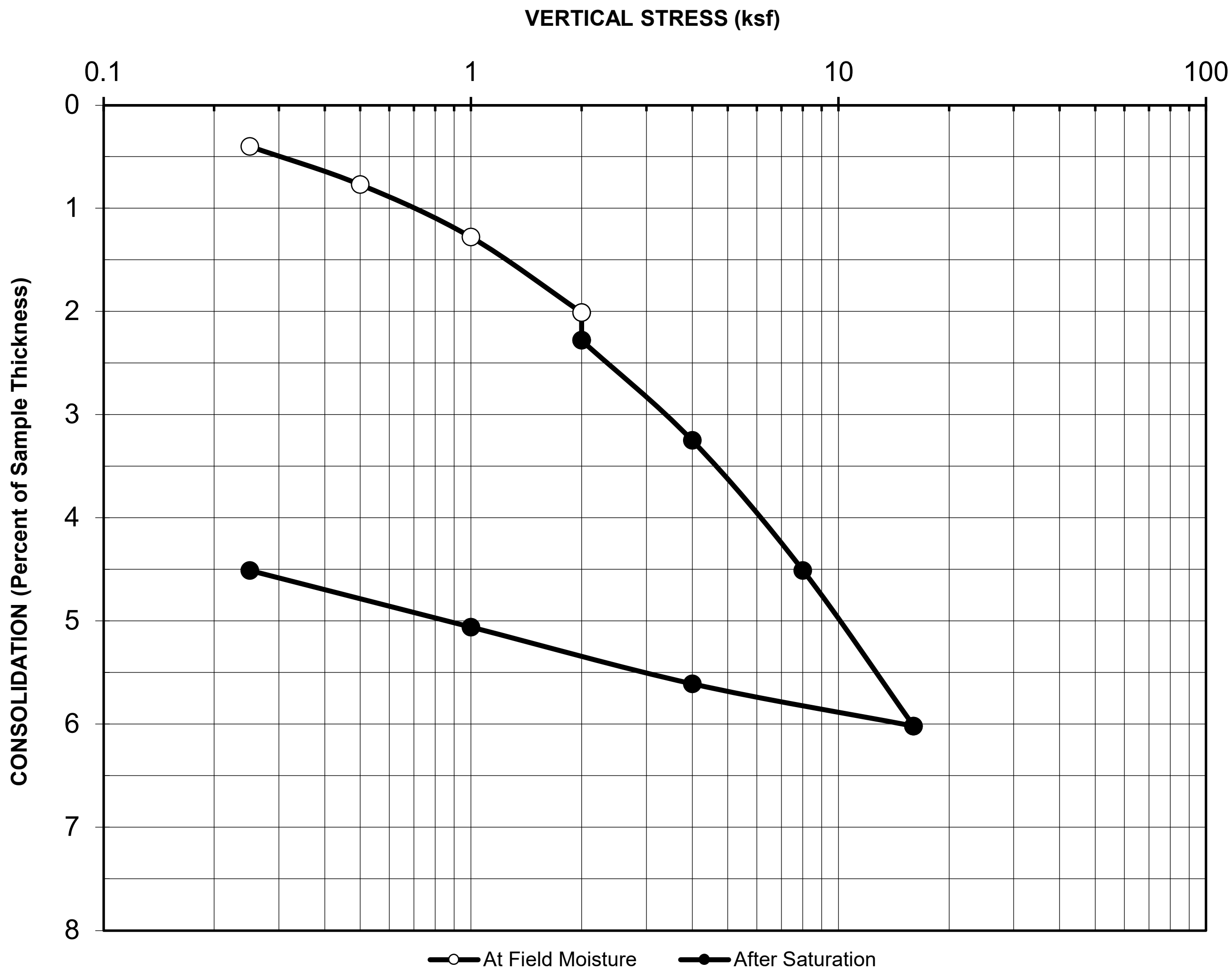

Boring No. :	B2	Initial Dry Unit Weight (pcf):	110.0
Sample No.:	-	Initial Moisture Content (%):	13.3
Depth (feet):	10	Final Moisture Content (%):	18.4
Sample Type:	Mod Cal	Assumed Specific Gravity:	2.7
Soil Description:	Clayey Sand	Initial Void Ratio:	0.53
Remarks:	Collapse= 0.27% upon inundation		

CONSOLIDATION CURVE
ASTM D 2435

Project Name:	Patriot H.S. Stadium Improvements
Project No.:	6484
Date:	8/15/2022
AP No:	22-0837 **Sheet No:** 1

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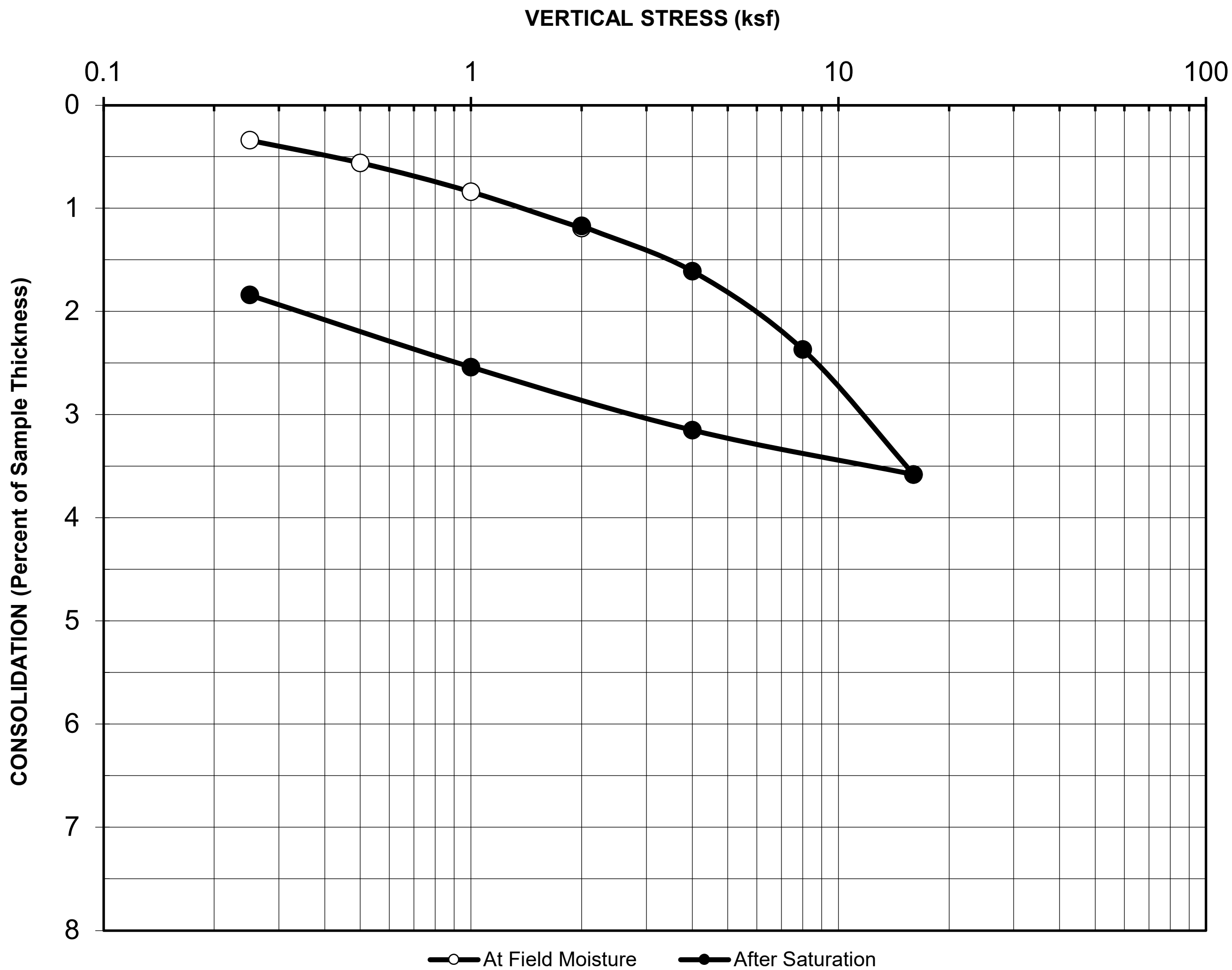
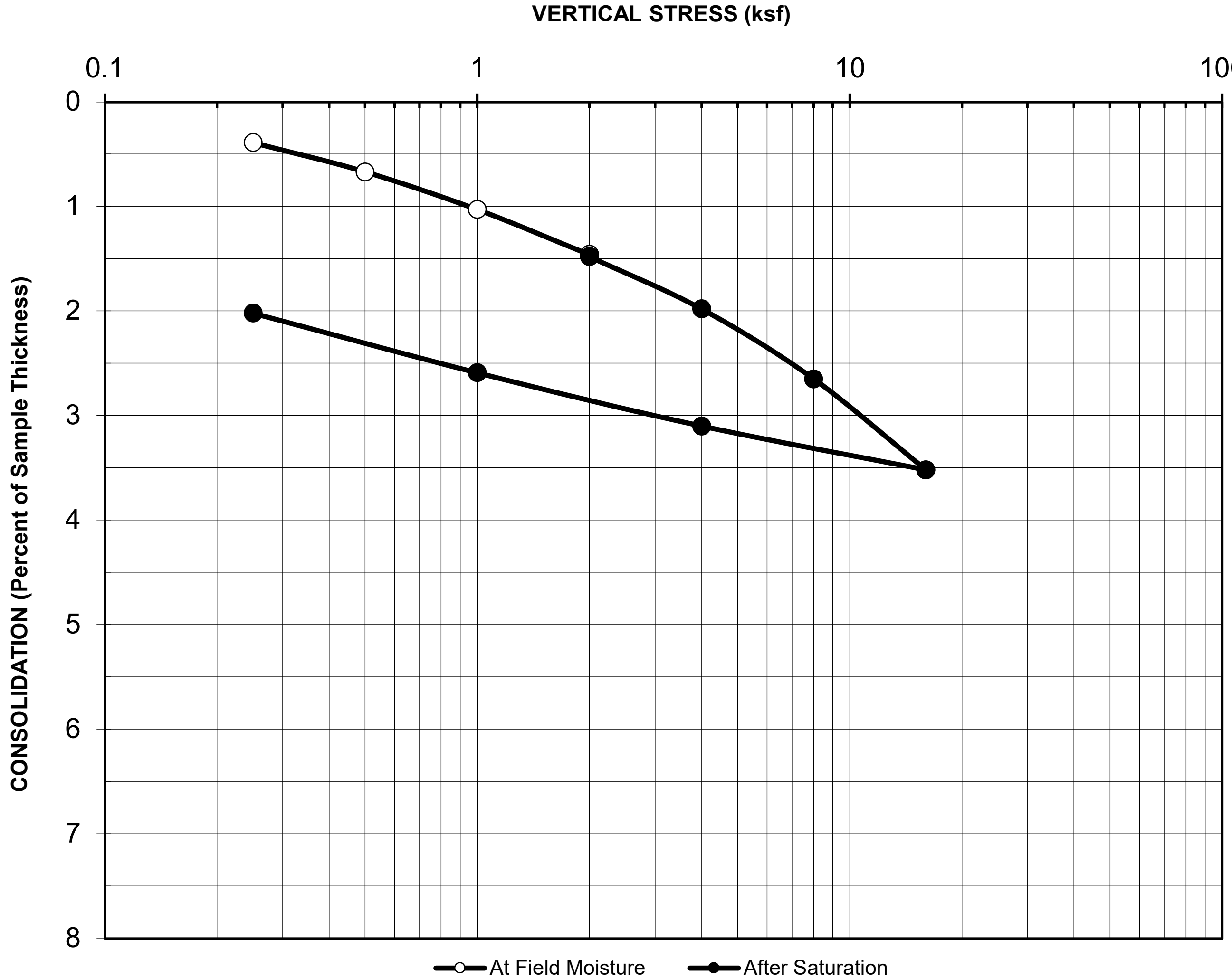


Boring No. :	B4	Initial Dry Unit Weight (pcf):	125.6
Sample No.:	-	Initial Moisture Content (%):	10.0
Depth (feet):	20	Final Moisture Content (%):	13.0
Sample Type:	Mod Cal	Assumed Specific Gravity:	2.7
Soil Description:	Sandy Clay	Initial Void Ratio:	0.34
Remarks:	Swell= 0.02% upon inundation		

CONSOLIDATION CURVE
ASTM D 2435

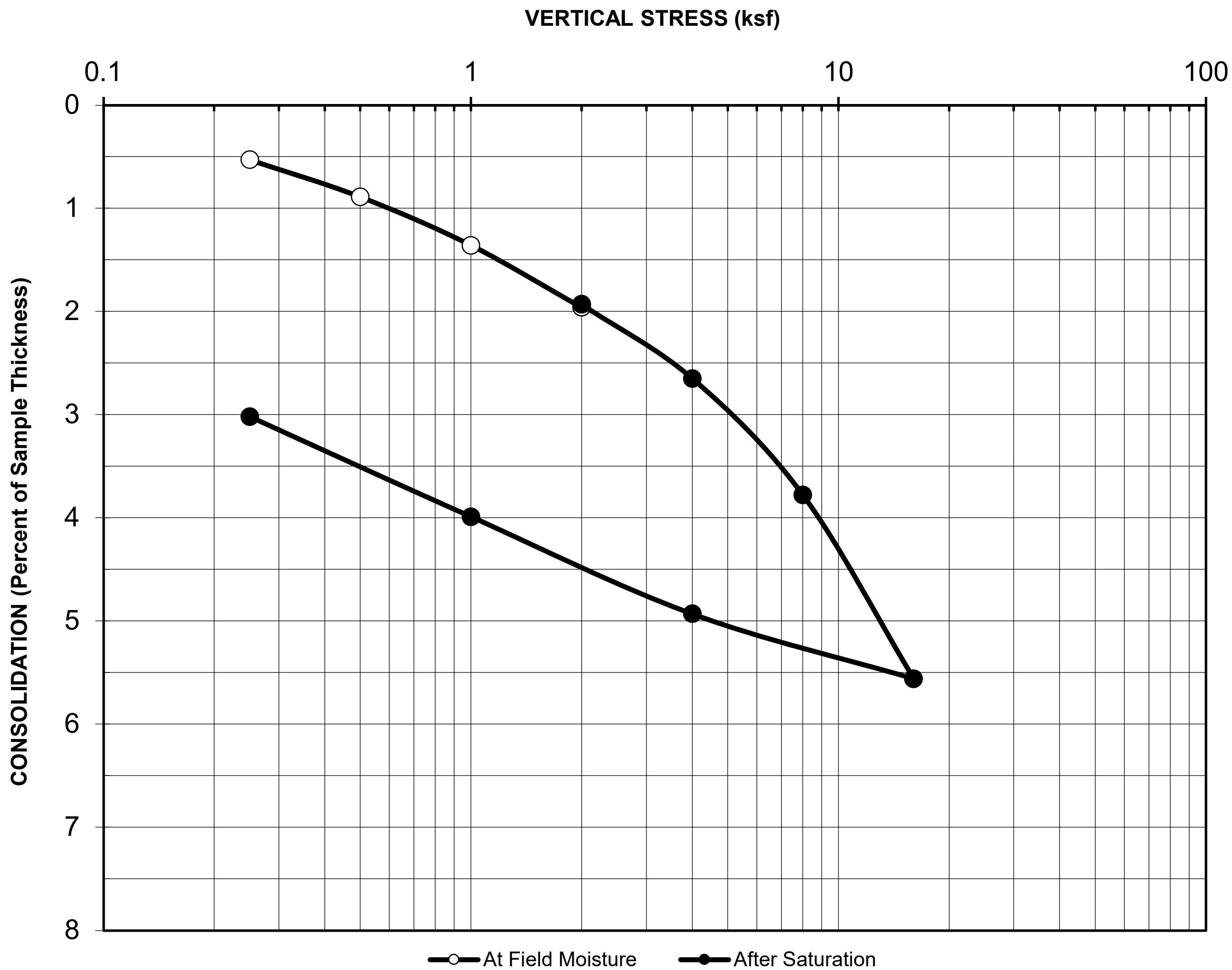
Project Name: Patriot H.S. Stadium Improvements
Project No.: 6484
Date: 8/15/2022
AP No: 22-0837 **Sheet No:** 1



Boring No. :	B6	Initial Dry Unit Weight (pcf):	118.0
Sample No.:	-	Initial Moisture Content (%):	9.5
Depth (feet):	5	Final Moisture Content (%):	15.0
Sample Type:	Mod Cal	Assumed Specific Gravity:	2.7
Soil Description:	Clayey Sand	Initial Void Ratio:	0.43
Remarks:	Collapse= 0.02% upon inundation		

CONSOLIDATION CURVE
ASTM D 2435

Project Name: Patriot H.S. Stadium Improvements
Project No.: 6484
Date: 8/15/2022
AP No: 22-0837 **Sheet No:** 1



Boring No. :	B11	Initial Dry Unit Weight (pcf):	118.3
Sample No.:	-	Initial Moisture Content (%):	12.1
Depth (feet):	15	Final Moisture Content (%):	15.7
Sample Type:	Mod Cal	Assumed Specific Gravity:	2.7
Soil Description:	Clay	Initial Void Ratio:	0.42
Remarks:	Swell= 0.03% upon inundation		

CONSOLIDATION CURVE
ASTM D 2435

Project Name: Patriot H.S. Stadium Improvements
Project No.: 6484
Date: 8/15/2022
AP No: 22-0837 **Sheet No:** 1

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COMPACTION TEST

Client: Fenagh Engineering & Testing — AP Number: 22-0837
Project Name: Patriot H.S. Stadium Improvements — Tested By: LS — Date: 08/18/22
Project No. : 6484 — Calculated By: JP — Date: 08/19/22
Boring No.: B4 — Checked By: AP — Date: 08/22/22
Sample Type: Bulk — Depth (ft.): 1-5
Visual Sample Description: Clayey Sand

Compaction Method: [X] ASTM D1557 [ ] ASTM D698
Preparation Method: [ ] Moist [X] Dry

METHOD: A
MOLD VOLUME (CU.FT): 0.0333

Wt. Comp. Soil + Mold (gm.)	3942	4006	3913	3824		
Wt. of Mold (gm.)	1830	1830	1830	1830		
Net Wt. of Soil (gm.)	2113	2177	2084	1995		
Container No.						
Wt. of Container (gm.)	149.54	145.39	146.76	144.45		
Wet Wt. of Soil + Cont. (gm.)	670.25	696.51	642.92	642.56		
Dry Wt. of Soil + Cont. (gm.)	634.10	649.23	589.22	616.30		
Moisture Content (%)	7.46	9.38	12.14	5.57		
Wet Density (pcf)	139.72	143.95	137.80	131.91		
Dry Density (pcf)	130.02	131.60	122.88	124.96		

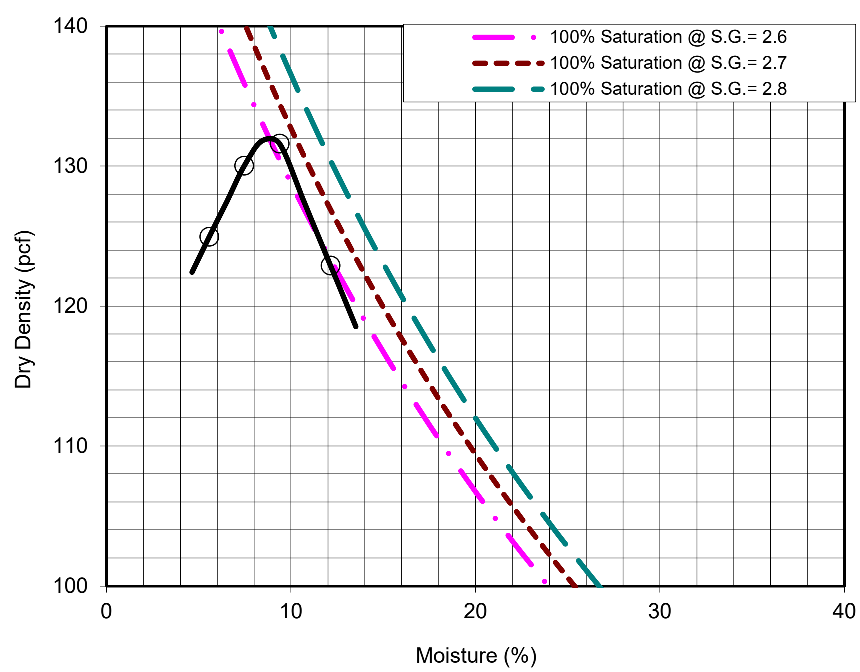
Maximum Dry Density (pcf): **131.9**
Maximum Dry Density w/ Rock Correction (pcf): **N/A**
Optimum Moisture Content (%): **8.8**
Optimum Moisture Content w/ Rock Correction (%): **N/A**

PROCEDURE USED

[X] *METHOD A: Percent of Oversize:* *1.5%*
Soil Passing No. 4 (4.75 mm) Sieve
Mold : 4 in. (101.6 mm) diameter
Layers : 5 (Five)
Blows per layer : 25 (twenty-five)

[ ] *METHOD B: Percent of Oversize:* *N/A*
Soil Passing 3/8 in. (9.5 mm) Sieve
Mold : 4 in. (101.6 mm) diameter
Layers : 5 (Five)
Blows per layer : 25 (twenty-five)

[ ] *METHOD C: Percent of Oversize:* *N/A*
Soil Passing 3/4 in. (19.0 mm) Sieve
Mold : 6 in. (152.4 mm) diameter
Layers : 5 (Five)
Blows per layer : 56 (fifty-six)



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COMPACTION TEST

Client:	Fenagh Engineering & Testing			AP Number:	22-0837
Project Name:	Patriot H.S. Stadium Improvements	Tested By:	TV	Date:	08/18/22
Project No. :	6484	Calculated By:	JP	Date:	08/19/22
Boring No.:	B5	Checked By:	AP	Date:	08/22/22
Sample Type:	Bulk	Depth (ft.):	1-5		
Visual Sample Description:	Clayey Sand				

Compaction Method: [X] ASTM D1557 [ ] ASTM D698

Preparation Method: [ ] Moist [X] Dry

METHOD	A
MOLD VOLUME (CU.FT)	0.0333

Wt. Comp. Soil + Mold (gm.)	3921	3974	3884	3794		
Wt. of Mold (gm.)	1830	1830	1830	1830		
Net Wt. of Soil (gm.)	2092	2144	2054	1965		
Container No.						
Wt. of Container (gm.)	143.04	137.57	152.04	153.22		
Wet Wt. of Soil + Cont. (gm.)	755.23	648.67	785.44	759.99		
Dry Wt. of Soil + Cont. (gm.)	709.58	600.81	713.27	726.24		
Moisture Content (%)	8.06	10.33	12.86	5.89		
Wet Density (pcf)	138.33	141.80	135.85	129.93		
Dry Density (pcf)	128.01	128.52	120.37	122.70		

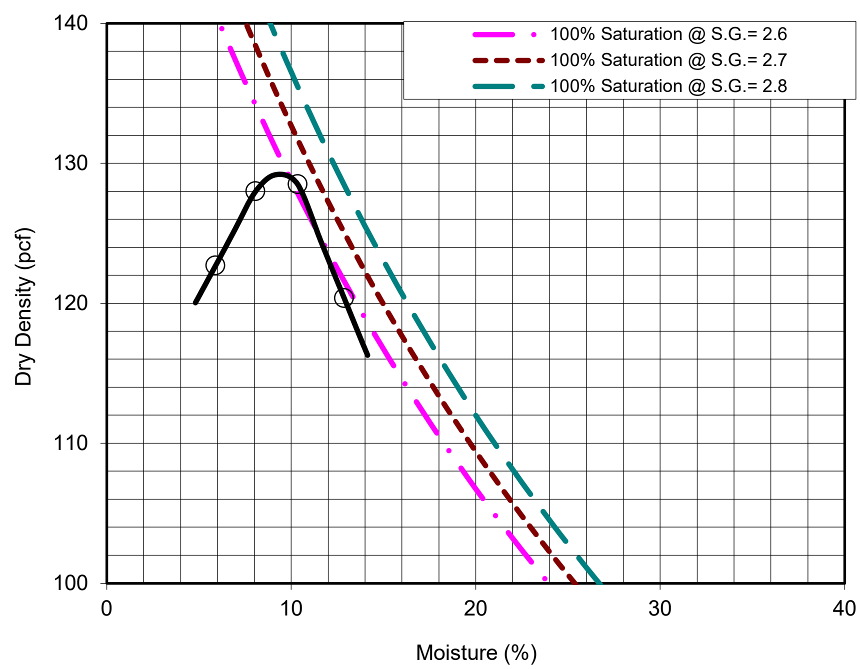
Maximum Dry Density (pcf)	**129.2**	Optimum Moisture Content (%)	**9.5**
Maximum Dry Density w/ Rock Correction (pcf)	**N/A**	Optimum Moisture Content w/ Rock Correction (%)	**N/A**

PROCEDURE USED

[X] *METHOD A: Percent of Oversize:* *0.5%*
Soil Passing No. 4 (4.75 mm) Sieve
Mold : 4 in. (101.6 mm) diameter
Layers : 5 (Five)
Blows per layer : 25 (twenty-five)

[ ] *METHOD B: Percent of Oversize:* *N/A*
Soil Passing 3/8 in. (9.5 mm) Sieve
Mold : 4 in. (101.6 mm) diameter
Layers : 5 (Five)
Blows per layer : 25 (twenty-five)

[ ] *METHOD C: Percent of Oversize:* *N/A*
Soil Passing 3/4 in. (19.0 mm) Sieve
Mold : 6 in. (152.4 mm) diameter
Layers : 5 (Five)
Blows per layer : 56 (fifty-six)



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COMPACTION TEST

Client:	Fenagh Engineering & Testing			AP Number:	22-0837
Project Name:	Patriot H.S. Stadium Improvements	Tested By:	SM	Date:	08/18/22
Project No. :	6484	Calculated By:	JP	Date:	08/19/22
Boring No.:	B10	Checked By:	AP	Date:	08/22/22
Sample Type:	Bulk	Depth (ft.):	1-5		
Visual Sample Description:	Clayey Sand				

Compaction Method: [X] ASTM D1557 [ ] ASTM D698

Preparation Method: [ ] Moist [X] Dry

METHOD: A
MOLD VOLUME (CU.FT): 0.0333

Wt. Comp. Soil + Mold (gm.)	3983	4015	3947	3854		
Wt. of Mold (gm.)	1830	1830	1830	1830		
Net Wt. of Soil (gm.)	2154	2186	2118	2025		
Container No.						
Wt. of Container (gm.)	125.47	135.86	150.09	161.76		
Wet Wt. of Soil + Cont. (gm.)	549.57	478.27	637.29	642.76		
Dry Wt. of Soil + Cont. (gm.)	521.92	449.40	584.74	621.50		
Moisture Content (%)	6.97	9.21	12.09	4.63		
Wet Density (pcf)	142.43	144.54	140.05	133.90		
Dry Density (pcf)	133.14	132.36	124.94	127.98		

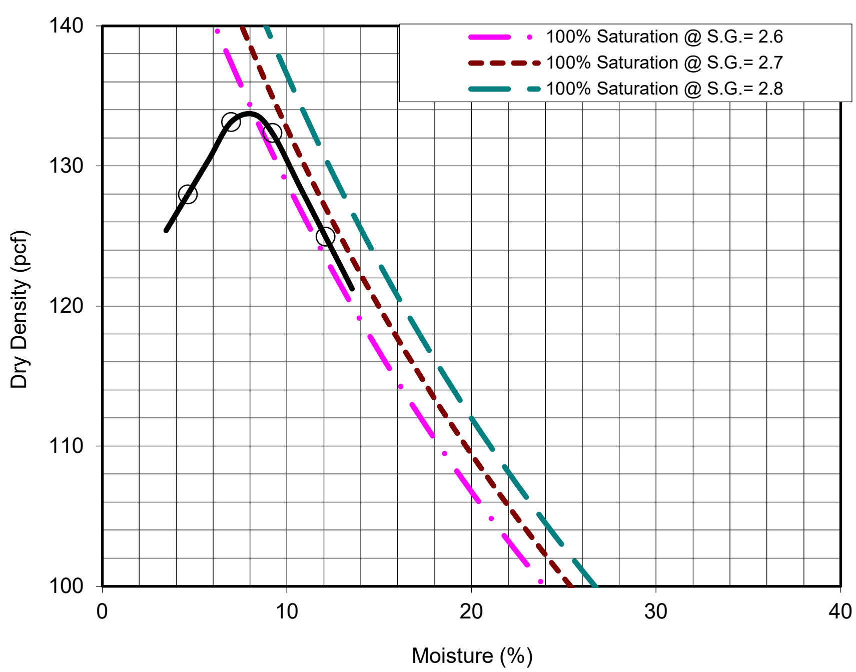
Maximum Dry Density (pcf)	**133.7**	Optimum Moisture Content (%)	**8.0**
Maximum Dry Density w/ Rock Correction (pcf)	**N/A**	Optimum Moisture Content w/ Rock Correction (%)	**N/A**

PROCEDURE USED

[X] *METHOD A: Percent of Oversize:* *2.3%*
Soil Passing No. 4 (4.75 mm) Sieve
Mold : 4 in. (101.6 mm) diameter
Layers : 5 (Five)
Blows per layer : 25 (twenty-five)

[ ] *METHOD B: Percent of Oversize:* *N/A*
Soil Passing 3/8 in. (9.5 mm) Sieve
Mold : 4 in. (101.6 mm) diameter
Layers : 5 (Five)
Blows per layer : 25 (twenty-five)

[ ] *METHOD C: Percent of Oversize:* *N/A*
Soil Passing 3/4 in. (19.0 mm) Sieve
Mold : 6 in. (152.4 mm) diameter
Layers : 5 (Five)
Blows per layer : 56 (fifty-six)



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EXPANSION INDEX TEST RESULTS

ASTM D 4829

Client Name: Fenagh Engineering & Testing
Project Name: Patriot H.S. Stadium Improvements
Project No.: 6484

AP Job No.: 22-0837
Date: 22-0837

Boring No.	Sample Type	Depth (ft)	Soil Description	Molded Dry Density (pcf)	Molded Moisture Content (%)	Init. Degree Saturation (%)	Measured Expansion Index	Corrected Expansion Index
B4	Bulk	1-5	Clayey Sand	119.7	7.8	51.9	32	33
B5	Bulk	1-5	Clayey Sand	117.7	7.8	49.0	27	27
B10	Bulk	1-5	Clayey Sand	124.9	6.5	50.3	12	12

ASTM EXPANSION CLASSIFICATION

Expansion Index	Classification
0-20	V. Low
21-50	Low
51-90	Medium
91-130	High
>130	V. High

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R-VALUE TEST DATA

ASTM D2844

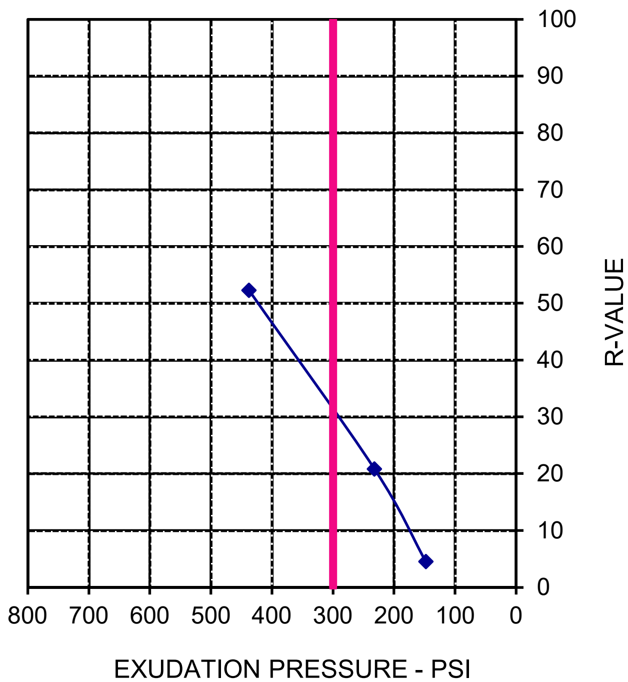
Project Name:	Patriot H.S. Stadium Improvements	Tested By: ST	Date: 08/17/22
Project Number:	6484	Computed By: KM	Date: 08/18/22
Boring No.:	B4	Checked By: AP	Date: 08/22/22
Sample Type:	Bulk	Depth (ft.): 1-5	
Location:	N/A		
Soil Description:	Clayey Sand		

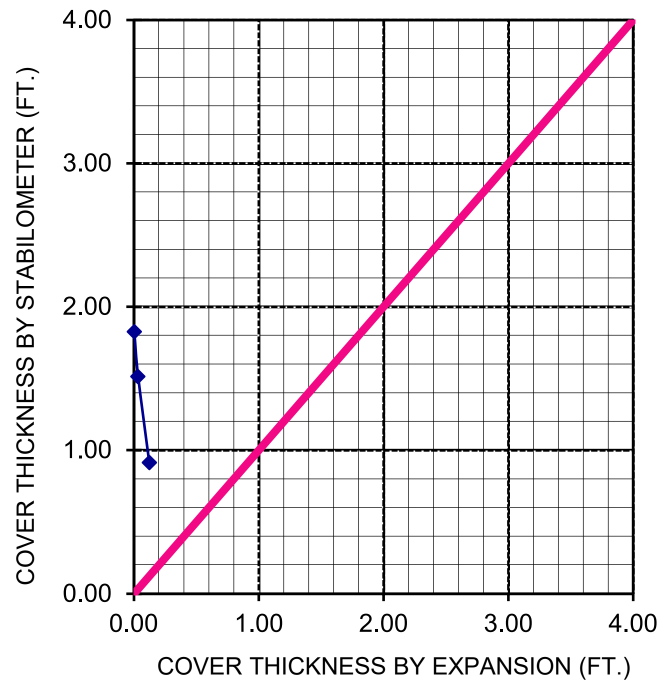
Mold Number	D	E	F	
Water Added, g	51	35	20	
Compact Moisture(%)	13.2	11.4	9.9	
Compaction Gage Pressure, psi	50	100	250	
Exudation Pressure, psi	148	232	438	
Sample Height, Inches	2.4	2.4	2.4	
Gross Weight Mold, g	3041	3082	2985	
Tare Weight Mold, g	1964	1954	1869	
Net Sample Weight, g	1077	1127	1116	
Expansion, inchesx10^{-4}	1	10	37	
Stability 2,000 (160 psi)	42/140	38/100	18/46	
Turns Displacement	6.35	5.27	5.11	
R-Value Uncorrected	5	22	55	
R-Value Corrected	5	21	52	
Dry Density, pcf	120.2	127.8	128.2	
Traffic Index	8.0	8.0	8.0	
G.E. by Stability	1.82	1.51	0.91	
G.E. by Expansion	0.00	0.03	0.12	

R-VALUE		
By Exudation:	31
By Expansion:	*N/A
At Equilibrium: (by Exudation)	31

Remarks

Gf = 1.34, and 1.1 % Retained on the ¾"
*Not Applicable

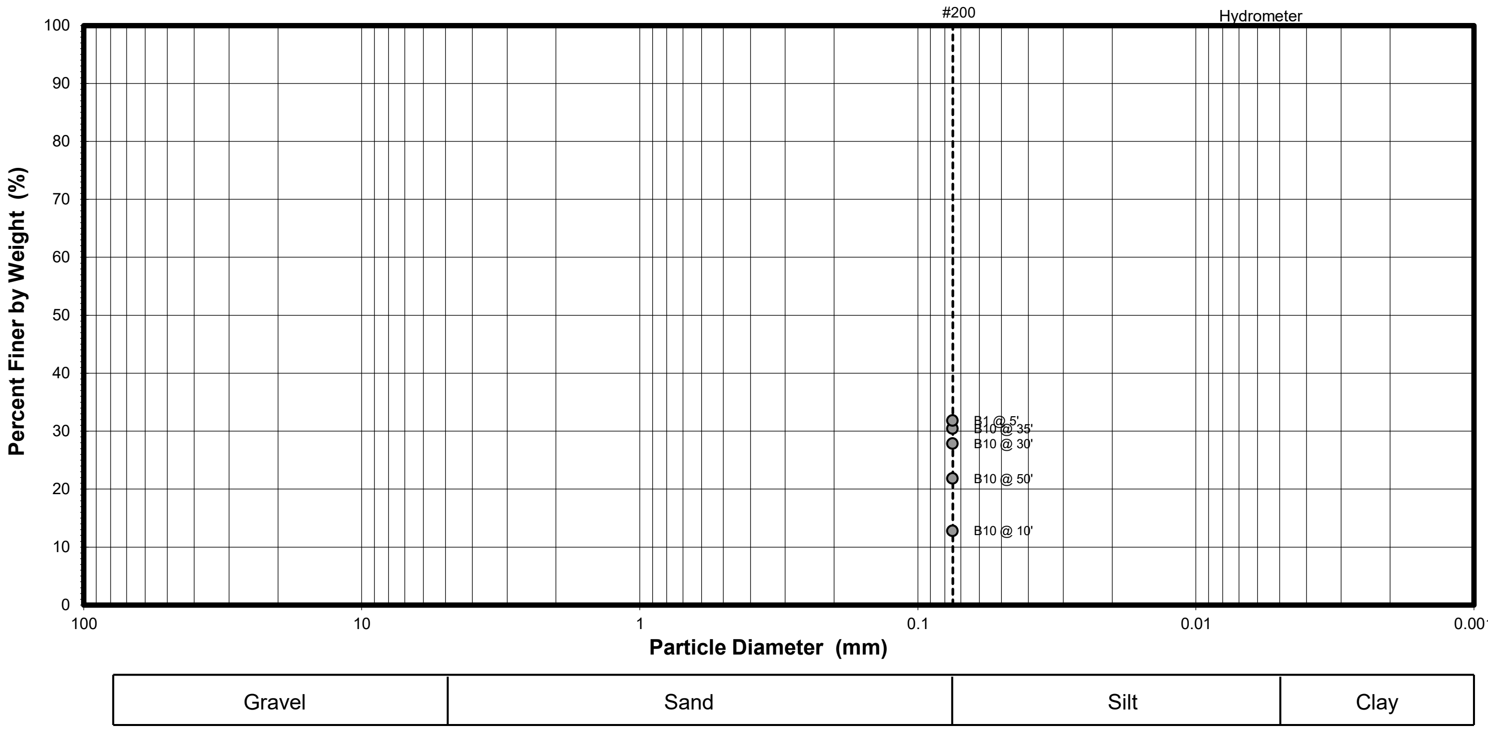




Amount of Material in Soils Finer than No. 200 Sieve - ASTM D 1140

FENAGH ENGINEERING AND TESTING

Project:	Patriot High School, Jurupa Valley	**Date:**	8/23/2022
File No:	6484	**Soil Type:**	Various
Sample #:	Various	**Lab Tech:**	M. Tabb



Test Method (A or B):	B

Sample Number	Soil Type (USCS)	A Mass of Dry Test Sample (g)	B Mass of Tare (g)	C Mass Ret. on #200 Sieve + Tare (g)	D=C-B D Mass Ret. on #200 Sieve (g)	E=[(A-D)/A]*100 E Percent Passing #200 Sieve (%)
B1 @ 5'	SC	195.30	0.00	133.10	133.10	31.8%
B10 @ 10'	SM/SC	228.50	0.00	199.20	199.20	12.8%
B10 @ 30'	SC	152.90	0.00	110.30	110.30	27.9%
B10 @ 35'	SC/SM	181.90	0.00	126.50	126.50	30.5%
B10 @ 50'	SM	188.80	0.00	147.50	147.50	21.9%

APPENDIX C
CALCULATIONS SHEETS

Liquefaction Analysis
Dynamic Dry Settlement
Drilled Pole Foundation Capacity

FENAGH
ENGINEERING AND TESTING

Project: Patriot High School, Jurupa Valley
File No.: 6484
Description: Liquefaction Analysis (PGA$_M$)
Boring No: B10

LIQUEFACTION EVALUATION (Idriss & Boulanger, EERI NO 12)

EARTHQUAKE INFORMATION:

Earthquake Magnitude (M):	7.0
Peak Ground Horizontal Acceleration, PGA (g):	0.58
Calculated Mag.Wtg.Factor:	1.141

GROUNDWATER INFORMATION:

Current Groundwater Level (ft):	45.0
Historically Highest Groundwater Level* (ft):	16.0
Unit Weight of Water (pcf):	62.4

* Based on California Geological Survey Seismic Hazard Evaluation Report

BOREHOLE AND SAMPLER INFORMATION:

Borehole Diameter (inches):	8
SPT Sampler with room for Liner (Y/N):	Y

LIQUEFACTION BOUNDARY:

Plastic Index Cut Off (PI):	12
Minimum Liquefaction FS:	1.3

Depth to Base Layer (feet)	Total Unit Weight (pcf)	Current Water Level (feet)	Historical Water Level (feet)	Field SPT Blowcount N	Depth of SPT Blowcount (feet)	Fines Content #200 Sieve (%)	Plastic Index (PI)	Vetical Stress σ_{vc}, (psf)	Effective Vert. Stress σ_{vc}', (psf)	Fines Corrected $(N_1)_{60-cs}$	Stress Reduction Coeff, r_d	Cyclic Shear Ratio CSR	Cyclic Resistance Ratio (CRR)	Factor of Safety CRR/CSR (F.S.)	Liquefaction Settlment ΔS_i (inches)
1	142.4	Unsaturated	Unsaturated	29	5	31.8	0	142.4	142.4	74.5	1.00	0.379	2.000	Non-Liq.	0.00
2	142.4	Unsaturated	Unsaturated	29	5	31.8	0	284.8	284.8	74.3	1.00	0.378	2.000	Non-Liq.	0.00
3	142.4	Unsaturated	Unsaturated	29	5	31.8	0	427.2	427.2	67.3	1.00	0.376	2.000	Non-Liq.	0.00
4	142.4	Unsaturated	Unsaturated	29	5	31.8	0	569.6	569.6	62.8	1.00	0.375	2.000	Non-Liq.	0.00
5	142.4	Unsaturated	Unsaturated	29	5	31.8	0	712.0	712.0	63.1	0.99	0.374	2.000	Non-Liq.	0.00
6	142.4	Unsaturated	Unsaturated	29	5	31.8	0	854.4	854.4	60.4	0.99	0.373	2.000	Non-Liq.	0.00
7	142.4	Unsaturated	Unsaturated	29	5	31.8	0	996.8	996.8	58.2	0.98	0.371	2.000	Non-Liq.	0.00
8	131.8	Unsaturated	Unsaturated	29	5	31.8	0	1128.6	1128.6	56.5	0.98	0.370	2.000	Non-Liq.	0.00
9	131.8	Unsaturated	Unsaturated	29	5	31.8	0	1260.4	1260.4	58.2	0.98	0.369	2.000	Non-Liq.	0.00
10	131.8	Unsaturated	Unsaturated	22	10	12.8	0	1392.2	1392.2	42.0	0.97	0.367	2.000	Non-Liq.	0.00
11	131.8	Unsaturated	Unsaturated	22	10	12.8	0	1524.0	1524.0	41.0	0.97	0.366	2.000	Non-Liq.	0.00
12	131.8	Unsaturated	Unsaturated	22	10	12.8	0	1655.8	1655.8	40.1	0.97	0.364	2.000	Non-Liq.	0.00
13	136.6	Unsaturated	Unsaturated	50	15	0.0	0	1792.4	1792.4	82.9	0.96	0.362	2.000	Non-Liq.	0.00
14	136.6	Unsaturated	Unsaturated	50	15	0.0	0	1929.0	1929.0	81.3	0.96	0.361	2.000	Non-Liq.	0.00
15	136.6	Unsaturated	Unsaturated	50	15	0.0	0	2065.6	2065.6	89.3	0.95	0.359	2.000	Non-Liq.	0.00
16	136.6	Unsaturated	Unsaturated	50	15	0.0	0	2202.2	2202.2	87.8	0.95	0.358	2.000	Non-Liq.	0.00
17	136.6	Unsaturated	Saturated	50	15	0.0	0	2338.8	2276.4	86.4	0.94	0.366	2.000	5.5	0.00
18	139.4	Unsaturated	Saturated	58	20	0.0	0	2478.2	2353.4	98.7	0.94	0.373	2.000	5.4	0.00
19	139.4	Unsaturated	Saturated	58	20	0.0	0	2617.6	2430.4	97.3	0.93	0.379	2.000	5.3	0.00
20	139.4	Unsaturated	Saturated	58	20	0.0	0	2757.0	2507.4	96.0	0.93	0.385	2.000	5.2	0.00
21	139.4	Unsaturated	Saturated	58	20	0.0	0	2896.4	2584.4	94.7	0.92	0.391	2.000	5.1	0.00
22	139.4	Unsaturated	Saturated	58	20	0.0	0	3035.8	2661.4	93.6	0.92	0.395	2.000	5.1	0.00
23	130.0	Unsaturated	Saturated	58	20	0.0	0	3165.8	2729.0	92.5	0.91	0.400	2.000	5.0	0.00
24	130.0	Unsaturated	Saturated	58	20	0.0	0	3295.8	2796.6	91.6	0.91	0.404	1.982	4.9	0.00
25	130.0	Unsaturated	Saturated	72	25	0.0	0	3425.8	2864.2	112.5	0.90	0.408	1.956	4.8	0.00
26	130.0	Unsaturated	Saturated	72	25	0.0	0	3555.8	2931.8	111.4	0.90	0.411	1.930	4.7	0.00
27	130.0	Unsaturated	Saturated	72	25	0.0	0	3685.8	2999.4	110.4	0.89	0.414	1.906	4.6	0.00
28	132.0	Unsaturated	Saturated	72	25	0.0	0	3817.8	3069.0	115.1	0.89	0.417	1.883	4.5	0.00
29	132.0	Unsaturated	Saturated	72	25	0.0	0	3949.8	3138.6	114.1	0.88	0.419	1.860	4.4	0.00
30	132.0	Unsaturated	Saturated	48	30	27.9	0	4081.8	3208.2	80.7	0.88	0.421	1.838	4.4	0.00
31	132.0	Unsaturated	Saturated	48	30	27.9	0	4213.8	3277.8	80.0	0.87	0.423	1.816	4.3	0.00
32	132.0	Unsaturated	Saturated	48	30	27.9	0	4345.8	3347.4	79.4	0.87	0.424	1.795	4.2	0.00
33	136.1	Unsaturated	Saturated	48	30	27.9	0	4481.9	3421.1	78.8	0.86	0.426	1.775	4.2	0.00
34	136.1	Unsaturated	Saturated	48	30	27.9	0	4618.0	3494.8	78.3	0.86	0.426	1.754	4.1	0.00
35	136.1	Unsaturated	Saturated	50	35	30.5	0	4754.1	3568.5	80.8	0.85	0.427	1.735	4.1	0.00
36	136.1	Unsaturated	Saturated	50	35	30.5	0	4890.2	3642.2	80.3	0.84	0.428	1.716	4.0	0.00
37	136.1	Unsaturated	Saturated	50	35	30.5	0	5026.3	3715.9	79.7	0.84	0.428	1.697	4.0	0.00
38	129.2	Unsaturated	Saturated	50	35	30.5	0	5155.5	3782.7	79.2	0.83	0.428	1.680	3.9	0.00
39	129.2	Unsaturated	Saturated	50	35	30.5	0	5284.7	3849.5	78.8	0.83	0.429	1.664	3.9	0.00
40	143.1	Unsaturated	Saturated	65	40	0.0	0	5427.8	3930.2	94.7	0.82	0.428	1.646	3.8	0.00
41	143.1	Unsaturated	Saturated	65	40	0.0	0	5570.9	4010.9	94.1	0.82	0.428	1.628	3.8	0.00
42	143.1	Unsaturated	Saturated	65	40	0.0	0	5714.0	4091.6	93.5	0.81	0.427	1.611	3.8	0.00
43	143.1	Unsaturated	Saturated	65	40	0.0	0	5857.1	4172.3	92.8	0.81	0.426	1.594	3.7	0.00
44	143.1	Unsaturated	Saturated	65	40	0.0	0	6000.2	4253.0	92.3	0.80	0.426	1.578	3.7	0.00
45	143.1	Unsaturated	Saturated	50	45	0.0	0	6143.3	4333.7	70.5	0.79	0.425	1.562	3.7	0.00
46	143.1	Saturated	Saturated	50	45	0.0	0	6286.4	4414.4	70.3	0.79	0.424	1.553	3.7	0.00
47	143.1	Saturated	Saturated	50	45	0.0	0	6429.5	4495.1	70.1	0.78	0.423	1.545	3.7	0.00
48	139.6	Saturated	Saturated	50	45	0.0	0	6569.1	4572.3	69.8	0.78	0.421	1.537	3.6	0.00
49	139.6	Saturated	Saturated	50	45	0.0	0	6708.7	4649.5	69.6	0.77	0.420	1.529	3.6	0.00
50	139.6	Saturated	Saturated	62	50	21.9	0	6848.3	4726.7	90.8	0.77	0.419	1.521	3.6	0.00

Total Liquefaction Settlement, S = 0.00 inches

FENAGH
ENGINEERING AND TESTING

FILE NO.: 6484
PROJECT: Patriot High School, Jurupa Valley
BORING B10

EVALUATION OF EARTHQUAKE-INDUCED SETTLEMENTS IN DRY SANDY SOILS (Existing Water Level Conditions)

INPUT:

EARTHQUAKE INFORMATION:

Earthquake Magnitude:	7.0
Peak Horiz. Acceleration (g):	0.58

Depth of Base of Strata (ft)	Thickness of Layer (ft)	USCS Soil Type	Depth of Mid-point of Layer (ft)	Soil Unit Weight (pcf)	Overburden Pressure at Mid-point (tsf)	Mean Effective Pressure at Mid-point (tsf)	Average Cyclic Shear Stress [Tav]	Field SPT [N]	Correction Factor [Cer]	Relative Density [Dr] (%)	Correction Factor [Cn]	Corrected [N1]60	Percent Passing 200 Sieve	ΔN for Fines Content	Fines Corrected [N1]60	Maximum Shear Mod. [Gmax] (tsf)	[geff]*[Geff] / [Gmax]	[geff]	[geff]*100%	Volumetric Strain [E15} (%)	Number of Strain Cycles [Nc]	Corrected Vol. Strains [Ec]	Settlement [S] (inches)
5.0	5.0	SC	2.5	142.4	0.18	0.12	0.067	29	1.3	100.0	1.60	60.3	31.8	5.6	65.9	623.583	1.02E-04	5.00E-04	5.00E-02	3.00E-03	10.8481	0.0026	0.003
10.0	5.0	SM/SC	7.5	131.8	0.52	0.35	0.195	22	1.3	91.0	1.48	42.3	12.8	3.5	45.8	944.865	1.80E-04	6.00E-04	6.00E-02	1.40E-02	10.8481	0.0121	0.015
15.0	5.0	SC/CL	12.5	136.6	0.86	0.57	0.318	50	1.3	100.0	1.28	83.2	0.0	0.0	83.2	1478.040	1.73E-04	4.00E-04	4.00E-02	1.00E-03	10.8481	0.0009	0.001
20.0	5.0	SC	17.5	139.4	1.20	0.80	0.439	58	1.3	100.0	1.10	82.9	0.0	0.0	82.9	1748.838	1.88E-04	4.20E-04	4.20E-02	1.00E-03	10.8481	0.0009	0.001
25.0	5.0	SC	22.5	130.0	1.54	1.03	0.552	72	1.3	100.0	0.98	91.7	0.0	0.0	91.7	2046.399	1.89E-04	4.00E-04	4.00E-02	1.00E-03	10.8481	0.0009	0.001
30.0	5.0	SC	27.5	132.0	1.87	1.25	0.654	48	1.3	100.0	0.91	56.8	27.9	5.0	61.8	1975.610	2.19E-04	4.80E-04	4.80E-02	3.80E-03	10.8481	0.0033	0.004
35.0	5.0	SC/SM	32.5	136.1	2.20	1.47	0.750	50	1.3	97.0	0.83	54.0	30.5	5.6	59.6	2119.557	2.23E-04	5.00E-04	5.00E-02	4.00E-03	10.8481	0.0035	0.004
40.0	5.0	SM	37.5	129.2	2.53	1.70	0.837	65	1.3	100.0	0.80	67.6	0.0	0.0	67.6	2371.813	2.13E-04	4.70E-04	4.70E-02	2.00E-03	10.8481	0.0017	0.002
45.0	5.0	SM	42.5	143.1	2.87	1.92	0.917	50	1.3	90.0	0.77	50.1	0.0	0.0	50.1	2285.348	2.33E-04	5.00E-04	5.00E-02	8.00E-03	10.8481	0.0069	0.008
50.0	5.0	SM	47.5	139.6	3.23	2.16	0.992	62	1.3	97.0	0.73	58.8	21.9	4.5	63.3	2619.581	2.13E-04	4.00E-04	4.00E-02	2.50E-03	10.8481	0.0022	0.003

Total Calculated Dynamic Dry Settlement (inches) 0.04

Fenagh Engineering & Testing
Project: Patriot High School, Jurupa Valley
File No.: 6484
Description: Isolated Pole Foundations

Drilled Pole Foundation Capacity Calculation

Input Data:

Unit Weight of Overlying Soil Layer	γ_1	130	pcf
Thickness of Overlying Soil Layer	H_1	5	feet
Unit Weight of Bearing Strata	γ_2	130	pcf
Friction Angle of Bearing Strata	ϕ_2	29	degrees
Friction Angle between Pile and Soil	δ	21.75	degrees
Cohesion of Bearing Strata	c_2	100	psf
Adhesion	c_A	75	psf
Minimum Embedment into Bearing Strata	H_2	10	feet
Unit Weight of Water	γ_w	62.4	pcf
Depth to Groundwater from Pile Cap	H_w	16	feet

Pile Design:
Drilled <<Driven/Drilled
Circular <<Circular/Square Pile

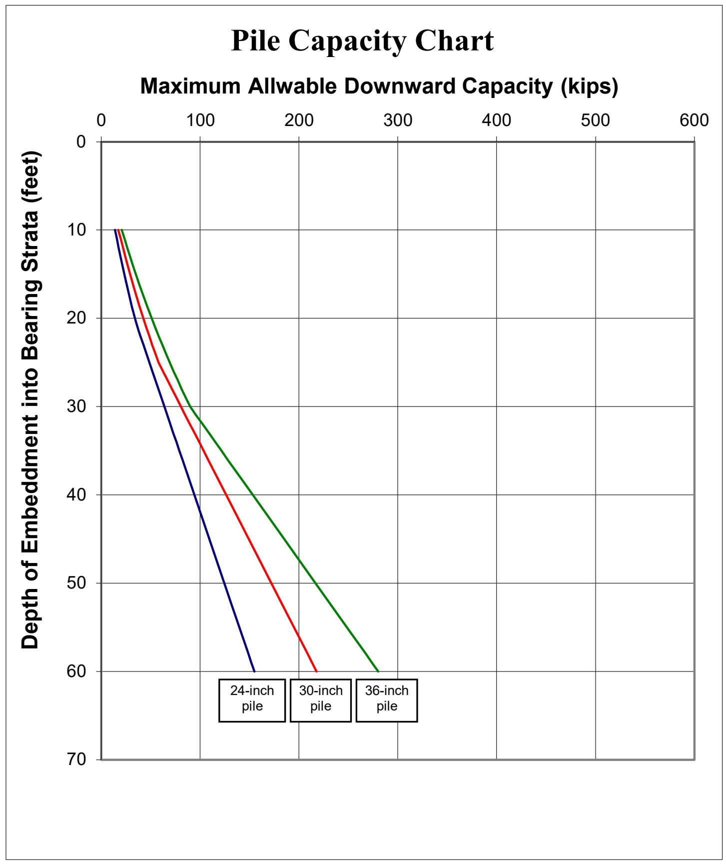
Pile Dimension:
24 inch diameter pile
30 inch diameter pile
36 inch diameter pile

Critical Depth Limit (Dc):
10 B

Lateral Earth Pressure Coefficient: $K_{HC} = 0.70$
Applied Factor of Safety: FS = 2
Factored Skin Friction $f_s/FS = [K_{HC} * \sigma'_v * (\tan \delta)]/FS$ **or** $f_s/FS = c_A/FS$

Pile Capacity:

Total Depth of Pile (feet)	Depth of Embedment into Bearing Strata (feet)	Maximum Allowable Downward Pile Capacity Capacity of 24 inch diameter pile (kips)	Capacity of 30 inch diameter pile (kips)	Capacity of 36 inch diameter pile (kips)
15	10	13.8	17.2	20.6
16	11	15.8	19.7	23.6
17	12	17.6	21.9	26.3
18	13	19.4	24.3	29.1
19	14	21.3	26.6	32.0
20	15	23.3	29.1	34.9
21	16	25.3	31.6	38.0
22	17	27.4	34.2	41.1
23	18	29.5	36.9	44.3
24	19	31.7	39.7	47.6
25	20	34.0	42.5	51.0
26	21	36.5	45.4	54.5
27	22	39.6	48.4	58.1
28	23	42.6	51.4	61.7
29	24	45.6	54.6	65.5
30	25	48.7	57.8	69.3
31	26	51.7	62.3	73.2
32	27	54.7	66.9	77.3
33	28	57.8	71.5	81.4
34	29	60.8	76.0	85.6
35	30	63.8	80.6	89.8
36	31	66.9	85.2	96.2
37	32	69.9	89.8	102.5
38	33	72.9	94.3	108.9
39	34	76.0	98.9	115.2
40	35	79.0	103.5	121.6
41	36	82.0	108.0	127.9
42	37	85.1	112.6	134.2
43	38	88.1	117.2	140.6
44	39	91.1	121.8	146.9
45	40	94.2	126.3	153.3
46	41	97.2	130.9	159.6
47	42	100.2	135.5	165.9
48	43	103.3	140.1	172.3
49	44	106.3	144.6	178.6
50	45	109.3	149.2	185.0
51	46	112.3	153.8	191.3
52	47	115.4	158.3	197.7
53	48	118.4	162.9	204.0
54	49	121.4	167.5	210.3
55	50	124.5	172.1	216.7
56	51	127.5	176.6	223.0
57	52	130.5	181.2	229.4
58	53	133.6	185.8	235.7
59	54	136.6	190.3	242.0
60	55	139.6	194.9	248.4
61	56	142.7	199.5	254.7
62	57	145.7	204.1	261.1
63	58	148.7	208.6	267.4
64	59	151.8	213.2	273.7
65	60	154.8	217.8	280.1



Note: 1. Minimum pile embeddment depth of 10 feet
2. Uplift capacity may be designed using 50% of the downward capacity
3. Pile should be spaced a minimum of 3 diameters on center
4. See text of report for pile details and installation recommendations

APPENDIX D
GEOHAZARDS ASSESSMENT REPORT

4 GEOLOGIC HAZARD
4.1 SEISMIC HAZARDS
4.1.1 Historic Seismicity
4.1.2 Faulting and Ground Rupture
4.1.3 Strong Ground Motion
4.1.4 Liquefaction
4.1.5 Tsunamis and Seiches
4.2 FLOOD HAZARD
4.3 SLOPE STABILITY
4.3.1 General
4.3.2 Surficial Slope Stability
4.3.3 Gross Slope Stability
4.3.4 Analysis Findings
4.3.5 Conclusions
4.4 NATURALLY OCCURRING ASBESTOS
4.5 UNDOCUMENTED FILL
4.6 EXPANSIVE SOILS
4.7 RADON – 222 GAS
4.8 DYNAMIC DRY SETTLEMENT
4.9 CLOSURE

APPENDIX D

4 GEOLOGIC HAZARDS

Geologic hazards relevant to the proposed development have been considered, including seismic hazards, flood hazards, landsliding, slope stability, and expansive soils. These hazards are presented and discussed in the following subsections.

4.1 SEISMIC HAZARDS

Seismic hazards considered for this investigation include the potential for ground rupture due to faulting, seismic ground shaking, liquefaction, slope stability, tsunamis, and seiches. These potential hazards are discussed below.

4.1.1 Historic Seismicity

Significant seismic event earthquakes (>4.0 Mag) in proximity to the site (for incident dates later than 1923) are indicated on the attached Plate 9 entitled "Historical Seismic Events Map – Local". Notable earthquake events close to the project site are discussed as follows:

North San Jacinto Fault Earthquake -

The North San Jacinto Fault Earthquake took place at 11:28 pm on July 22, 1923, with a recorded magnitude of 6.3. Damage from this quake, although minor, was predominantly observed in the San Bernardino and Redlands areas. The San Bernardino County Hospital and State Hospital at Patton incurred significant damage. In general, however, buildings which sustained damage exhibited poor construction. Ground shaking was experienced as far as the cities of Needles and Santa Barbara.

4.1.2 Faulting and Ground Rupture

The site is not located within an Alquist-Priolo Earthquake Fault Zone (formally known as Special Studies Zone) established by the State Geologist, California Geologic Survey. Alquist-Priolo earthquake fault zones are regulatory zones surrounding the surface traces of active faults in California. A trace is a line on the earth's surface defining a fault. Wherever an active surface fault trace exists on a property, a structure for human occupancy cannot be placed over the fault and must be a minimum distance of 50 feet from the fault. An active fault, per the Alquist-Priolo Act, is one that has ruptured in the last 11,700 years.

As discussed above in report Section 3.2 and as indicated on Plate 8, an "unnamed" fault is mapped as a dotted line approximately two miles northwest of the site. Additional local faults identified as the Rialto-Colton Fault and Loma Linda fault are located three miles to the northeast of the project site. The dashed line indicates the location of the fault is inferred.

Based on our review of the referenced geologic maps and websites, no known faults encroach on the boundaries of the site or cross the school property and are not aligned in the direction of the proposed new structures. No known faults are mapped crossing the property and the property is not located within an Alquist-Priolo fault zone. **Therefore, the probability of damage due to surface rupture of a fault is considered low and not a design consideration.**

4.1.3 Strong Ground Motion

The peak ground acceleration (PGA_M) and modal magnitude were obtained from the USGS website using the Probabilistic Seismic Hazard Deaggregation program (USGS, 2022) and the Structural Engineers Association of California in collaboration with the Office of Statewide Health Planning and Development (SEAOCC/OSHPD, 2022), ground motion utility tool. A Site Class "D" ("Stiff Soil" Profile) was utilized in the USGS seismic and SEAOCC/OSHPD ground motion utility tools. A modal magnitude (MW) of 7.0 was obtained using the USGS Probabilistic Seismic Hazard Deaggregation program (USGS, 2021). A peak ground acceleration PGA_M of 0.58g, corresponding to a seismic event with a mean return interval of 2,475 years (2% exceedance in 50 years) was obtained using the SEAOCC/OSHPD seismic hazard utility tool. These parameters were utilized in the enclosed liquefaction analysis.

4.1.4 Liquefaction

Soil liquefaction results from loss of strength during cyclic loading, such as imposed by earthquakes. Soils most susceptible to liquefaction are clean, loose, saturated, uniformly graded sand below the groundwater table. Empirical evidence indicates that low plasticity silt and clay are also potentially liquefiable, though this phenomenon is commonly referred to as cyclic softening. When seismic ground shaking occurs, the soil is subjected to cyclic shear stresses that can cause excess hydrostatic pressures to develop. This can lead to lateral spreading of sloping or unconfined ground. Sand boils can also develop and lead to subsidence of the ground surface.

According to the Riverside County Mapping Portal Website (www.gisopendata@rivco.org, 2022) the site is located within a potentially liquefiable zone of high susceptibility. This determination is based on groundwater depth records, soil type and distance to a fault capable of producing a substantial earthquake. A copy of the Seismic Hazard Zone Map (Plate 10) indicating liquefaction susceptibility zones is included in the appendix of this report.

A site-specific liquefaction analysis was performed in accordance with the Recommended Procedures for Implementation of the California Geologic Survey Special Publication 117A, Guidelines for Analyzing and Mitigating Seismic Hazards in California (CGS, 2008), and the EERI Monograph (MNO-12) by Idriss and Boulanger (2008). This semi-empirical method is based on a correlation between measured values of Standard Penetration Test (SPT) resistance and field performance data.

Groundwater was encountered in Boring B10 at a depth of 45 feet below ground surface during site exploration. According to a previous site investigation by Inland Foundation Engineering, Inc. dated July 26, 2001, the historically high groundwater level for the subject site was estimated at 16 feet below ground surface. A groundwater level of 16 feet below ground surface was conservatively utilized in the liquefaction analysis.

The enclosed "Empirical Estimation of Liquefaction Potential" discussed below is based on Boring 10. Standard Penetration Test (SPT) data were collected at 5-foot intervals. Samples of the collected materials were conveyed to the laboratory for testing and analysis. The percent passing a number 200 sieve of representative samples of the soils encountered in the exploratory boring are presented on the enclosed laboratory data sheets in the Appendix B. Based on CGS Special Publication 117A (CDMG, 2008), liquefaction hazards are typically associated with sandy soils and silty soils of low plasticity.

Based on the adjusted blow count data, results of laboratory testing, and the calculated factor of safety against the occurrence of liquefaction, it is the assessment of this firm that the potential for liquefaction at the site is low.

4.1.5 Tsunamis and Seiches

Tsunamis are large ocean waves, generated by displacements of vertical faulting beneath the ocean floor, which can reach great heights when they encounter shorelines. **Based on this and the site elevation of approximately 840 feet above msl, tsunamis are not likely to affect the site and not a design consideration.**

Seiches result when earthquake ground motion causes an enclosed or restricted body of water, such as a lake, bay, reservoir, or river to oscillate and generate large waves. **Based on the site elevation and lack of nearby reservoirs or lakes, it is concluded the risk of seiches at the site is low and not a design consideration.**

4.2 FLOOD HAZARD

Our review of the Federal Emergency Management Agency (FEMA) Flood Insurance Rate Maps for the property address (Map # 06065C0039G, dated 08/28/2008) indicate the site is in Zone X which is defined as an area of minimal flood hazard. A copy of the Flood Insurance Rate Map (Plate 11) is included in the appendix. **Based on the mapping, it is concluded the risk of flooding at the site is low and not a design consideration.**

4.3 LANDSLIDES AND SLOPE STABILITY

According to the U.S. Landslide Inventory Map provided by the USGS website, no specific information regarding landslide conditions were indicated in the vicinity of the project site. During onsite investigation, evidence of past landslides at or in the immediate vicinity of the site was not observed. The onsite geologic materials were observed to be dense and cohesive. Due to the proximity of the proposed home field stadium bleachers to the descending embankments along the western perimeter of the site, the following slope stability analysis was performed.

4.3.1 General

The school property slopes gently down from the northeast to the southwest with intermittent terraced embankments throughout the campus grounds. Embankments extending to a height of 11 feet with slope gradients ranging from 3H:1V to 2H:1V are located along the western and southern perimeter of the football field in the area designated for construction of the home team stadium bleachers as indicated by the Boring Location Map – Elevation Contours (Plate 2) and Cross-Sections A-A' (Plate 4) and B-B' (Plate 5). Surficial stability and deep-seated gross slope stability were considered during analysis.

4.3.2 Surficial Slope Stability

The method of analysis utilized in the included surficial stability is based on the "parallel seepage model" recommended by the American Society of Civil Engineers (ASCE). The parallel seepage model is based on: A uniform planar slope, uniform density and shear strength, and uniform seepage parallel with slope surface. As with any model, the validity of the analysis is determined in part by how closely the assumptions represent actual field conditions.

Based on the enclosed surficial stability calculations, the slope associated with Cross-Sections A-A' and B-B' indicate a factor of safety of 1.78. These calculations are based on saturated residual shear strengths of soil samples anticipated near the slope face with an assumed saturated surface thickness of 3 feet.

The provided calculations were performed in accordance with the recommendations established by CGS Special Publication 117A (CDMG, 2008) and Blake and others (2002). Blake and others (2002) suggest a factor of safety against surficial failure should be greater than 1.3 to 1.5, depending on the method of shear testing. The attached surficial stability calculations indicate a factor of safety in compliance with the recommended minimum criteria. Although surficial slope surfaces are assessed to by stable, mitigation measures such as the use of proper drainage systems and landscape development of slope surfaces is recommended.

4.3.3 Gross Slope Stability

Slope stability analysis was performed using the computer software application GSLOPE (Version 5.13) for the most critical slope sections (A-A' and B-B') based on slope gradient, slope height, geologic material, and structural surcharge conditions. GSLOPE utilizes Bishop's modified Method and Janbu Simplified Method to determine critical surfaces and safety factors based on force equilibrium methodologies. Analysis inputs included geotechnical parameters including soil density, cohesion, friction angle, building load surcharge, seismic loading and piezometric surfaces. A discussion of analysis input parameters is presented as follows:

Groundwater was encountered in Boring B10 at a depth of 45 feet during onsite investigation on August 4, 2022. According to previous site investigation by Inland Foundation Engineering, well water data obtained during an Environmental Site Assessment in 1976 indicates historical groundwater levels as shallow as 16 feet below ground surface. Groundwater or seepage was not encountered in the embankments under analysis as indicated on Borings B1 through B4. Due to the depth of the actual and historical groundwater depths relative the embankment height of 11 feet, groundwater parameters and piezometric lines were not included in the slope stability analysis.

For selection of the pseudo-static seismic coefficient, CGS Note 48 recommends the procedure outlined in CGS special Publication 117A. The screening analysis by Blake and others (2002) was utilized in the slope stability analysis whereby Keq is derived from the maximum peak ground acceleration at a return period of 475 years multiplied by a factor, feq. related to the seismicity of the site. The corresponding Keq was determined to be 0.27 in accordance with SP117-A guidelines based on the seismic parameters obtained for the site and a displacement threshold value of 5 cm.

Based on information provided by the stadium bleacher design-build contractor, column loading is estimated at 40 kips to be distributed by a conventional spread footing with a dimension of approximately 6 foot by 6 foot square yielding a bearing pressure of approximately 1,000 pounds per square foot. The anticipated footing configuration and bearing pressure was represented in the slope stability analysis model with the footing conservatively positioned on the uppermost surface immediately adjacent to the slope face surface. Actual footings placed during construction shall be deepened to alluvial soil and sufficiently setback away from the slope surface in accordance with CBC requirements.

Material strength parameters utilized during slope stability analysis are indicated on the following tables based on soils obtained and tested in near proximity to embankments:

Table 1: Material Properties & Strength Parameters – Cross Section A-A'			
Material Description	Material Density (pcf)	Cohesion (psf)	Phi Angle(deg)
Footing Surcharge	1,000 (psf)	N/A	N/A
Fill Soils	129.5	200	29
Older Alluvium	141.7	100	34

Table 2: Material Properties & Strength Parameters – Cross Section B-B’			
Material Description	Material Density (pcf)	Cohesion (psf)	Phi Angle(deg)
Footing Surcharge	1,000 (psf)	N/A	N/A
Fill Soils	129.5	200	29
Older Alluvium	133.1	250	30

4.3.4 Analysis Findings

The west perimeter slopes of Cross-Sections A-A’ and B-B’ have been analyzed for both static and pseudo-static deep seated gross stability utilizing Modified Bishop Method and Simplified Janbu Method. A footing surcharge boundary load of 1,000 pounds per square foot was applied at the top of the slope to simulate structural load surcharge from the proposed stadium bleachers. A seismic coefficient of 0.27g was utilized for the pseudo-static slope stability analysis.

The analysis model printouts are provided in Appendix E. The slope stability analysis findings and results are summarized in the following table:

Results of Gross Slope Stability Analysis			
Cross Section	Analysis Model	Condition	Factor of Safety
Section A-A’	Modified Bishop Method	Static	2.054
		Pseudo-Static	1.526
	Simplified Janbu Method	Static	1.797
		Pseudo-Static	1.140
Section B-B’	Modified Bishop Method	Static	2.263
		Pseudo-Static	1.583
	Simplified Janbu Method	Static	2.002
		Pseudo-Static	1.248

4.3.5 Conclusions

For permanent slopes, minimum factors of safety of 1.5 (static condition) and 1.1 (pseudo-static condition) are generally recommended. Based on safety factors obtained from computer-aided slope stability analysis provided herein, the analyzed embankments have been calculated to be stable for deep-seated gross stability provided the allowable bearing capacity for foundations within 10 feet of the embankment face is limited to 1,000 pounds per square foot.

4.4 NATURALLY OCCURRING ASBESTOS

Naturally Occurring Asbestos (NOA), now known to be hazardous to humans, includes six regulated naturally occurring minerals: Actinolite, amosite, anthophyllite, chrysotile, crocidolite, and tremolite. In California, asbestos minerals are most associated with ultramafic rocks and their derivatives, including Serpentine rock. Ultramafic rocks are igneous rocks composed mainly of iron-magnesium silicates minerals that crystallize deep in the earth’s interior. By the time they are exposed at the Earth’s surface, ultramafic rocks have typically undergone metamorphism, a process in which the mineralogy or the rock changes in response to the changing chemical and physical conditions. Asbestos is classified as a known human cancer-causing substance by local, State, and Federal health agencies and is known to cause chronic respiratory diseases. Asbestos fibers may be released into the air because of activities that disturb NOA-containing rocks or soils. Asbestos minerals can fragment into small fibers that readily suspend in the air and are of a size visible only under a microscope. Breathing these small fiber fragments may result in an increased risk of respiratory disease or cancer in exposed individuals.

The Department of Toxic Substances Control (DTSC) has developed the Interim Guidance, Naturally Occurring Asbestos at School Sites, revised 9/24/2004. The guidance document provides a four-step process to assist school districts and their consultants in conducting environmental assessments, investigations, and response actions (if needed) at new or expanding school sites with potential NOA. Step 1 is the potential identification of NOA through the performance of a Phase I Environmental Site Assessment (Phase I ESA). If NOA is potentially identified, environmental sampling and analysis will be needed as part of the development of a Preliminary Environmental Assessment (PEA.) The guidance document continues to a mitigation phase and long-term operation and maintenance of the site.

Based on the review of the geologic maps and literature discussed above in Section 3, no ultramafic rocks are mapped in the vicinity of the property. **It is concluded that anticipated NOA a not a concern for the project.**

4.5 UNDOCUMENTED FILL

Undocumented fill material was encountered during site exploration to depths ranging from two feet to five feet below existing ground surface. The observed fill consisted of very dense clayey sand. The fill was observed to be reddish brown, moist, and fine to medium grained.

Undocumented fill may cause non-uniform bearing support and potential for intolerable foundation or slab differential settlement. The existing fill soils are not suitable for support of foundations, floor slabs or additional fill but may be reused as certified recompacted fill.

4.6 EXPANSIVE SOILS

Expansive soils are common in the area and have the potential to impact the development where fluctuations in the moisture contents can cause unacceptable shrinkage and/or swell beneath buildings and/or flatwork. The climate, with dry summers and wet winters, may cause these clays to cyclically shrink as they dry and then swell as they become wetter. Controlling this moisture change will reduce this shrink-swell capability.

The near-surface soils at our exploratory borings are classified as medium dense to dense Clayey Sand (SC). Expansion Index testing result in values ranging from 12 to 33 indicating very low to low expansion potential.

On this basis, expansive soils are not expected to impact the site and are not a design consideration.

4.7 RADON – 222 GAS

Radon is produced naturally as Radon-222 in gas form. Radon is a byproduct of the natural decay of uranium that is present in small quantities in several rock types of the Transverse Ranges. Radon is soluble and can be transported in groundwater. When water-containing radon is exposed to air (by pumping or through a tap), radon can diffuse into the air where it can be inhaled.

The U.S. Environmental Protection Agency (EPA) lists Riverside County in Zone 2, moderate potential radon hazard (between 2 and 4 pCi/L) (U.S. EPA, n.d.).

The California Geological Survey has collaborated with the California Department of Public Health Radon Program since 1989 to identify areas of California with increased potential for elevated indoor radon levels. A review of the Interactive Radon Map developed by the California Geological Survey and California Department of Public Health indicates the site is in an area of Riverside County wherein radon levels are not indicated.

Based on this information, it is the assessment of this firm that the risk of naturally occurring radon at the site is categorized as moderate. Should radon risk be a concern for the development, a consultant specializing in radon site investigation and mitigation should be retained.

4.8 DYNAMIC DRY SETTLEMENT

Seismically induced settlement or compaction of dry or moist, cohesionless soils can be an effect related to earthquake ground motion. Such settlements are typically most damaging when the settlements are differential in nature across the length of structures.

Some seismically induced settlement of the proposed structures should be expected due to strong ground-shaking, however, due to the uniform nature of the underlying geologic materials, excessive differential settlements are not expected to occur.

Calculations indicate that seismically induced settlement on the site will be on the order of 0.1 inches. This settlement estimate is considered negligible and well within the tolerance of a properly designed structure.

4.9 CLOSURE

Fenagh Engineering & Testing appreciates the opportunity to be of service to you over the course of this project. If you have any questions regarding the contents of this report, or if we can provide additional assistance, please contact the undersigned.

Sincerely,

Scott T. Prince, PE
Project Engineer

Jonathan Goodmacher, PG, CEG, HG
Certified Engineering Geologist

Bradford Quon, GE
Principal Geotechnical Engineer

APPENDIX E
SLOPE STABILITY ANALYSIS

E-1 Surficial Slope Stability
E-2 Cross Section A-A’; Bishop; Static
E-3 Cross Section A-A’; Bishop; Pseudo-Static
E-4 Cross Section A-A’; Janbu; Static
E-5 Cross Section A-A’; Janbu; Pseudo-Static
E-6 Cross Section A-A’; Bishop; Static
E-7 Cross Section A-A’; Bishop; Pseudo-Static
E-8 Cross Section A-A’; Janbu; Static
E-9 Cross Section A-A’; Janbu; Pseudo-Static

Project: Patriot High School, Jurupa Valley
File No.: 6484
Description: Fill

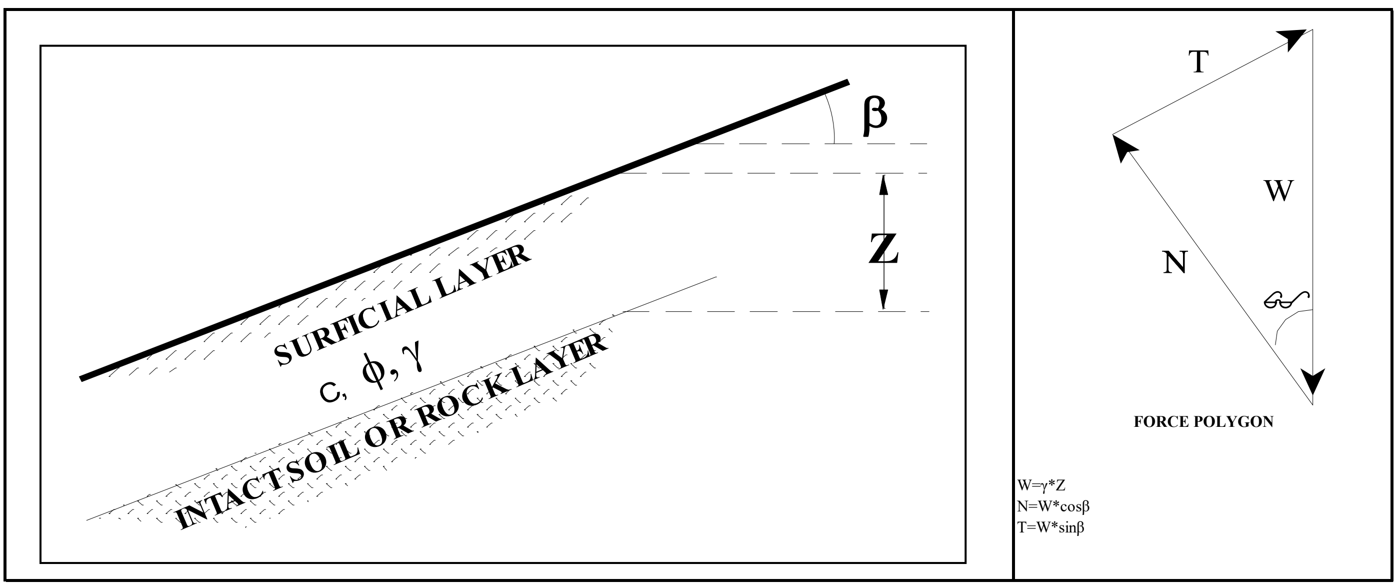
SURFICIAL SLOPE STABLITY FOR INFINITE SLOPE

Input Slope Properties:

Vertical Thickness of Surficial Materials	(Z)	3.0	feet		
Slope Angle	(β)	27.0	degrees	0.4712389	radians
Saturated Thickness	(h_s)	3.0	feet		

Input Soil Properties:

Unit Weight of Saturated Surficial Soils	(γ)	140.0	pcf		
Friction Angle of Surficial Soils	(ϕ)	29.0	degrees	0.50614548	radians
Cohesion of Surficial Soils	(c)	200.0	psf		
Density of Water	(γ_ω)	62.4	pcf		



Equation

$$F = \frac{c' + (\gamma - m*\gamma w)*z*\cos^2\beta*\tan\varphi}{\gamma*z*\sin\beta\cos\beta}$$

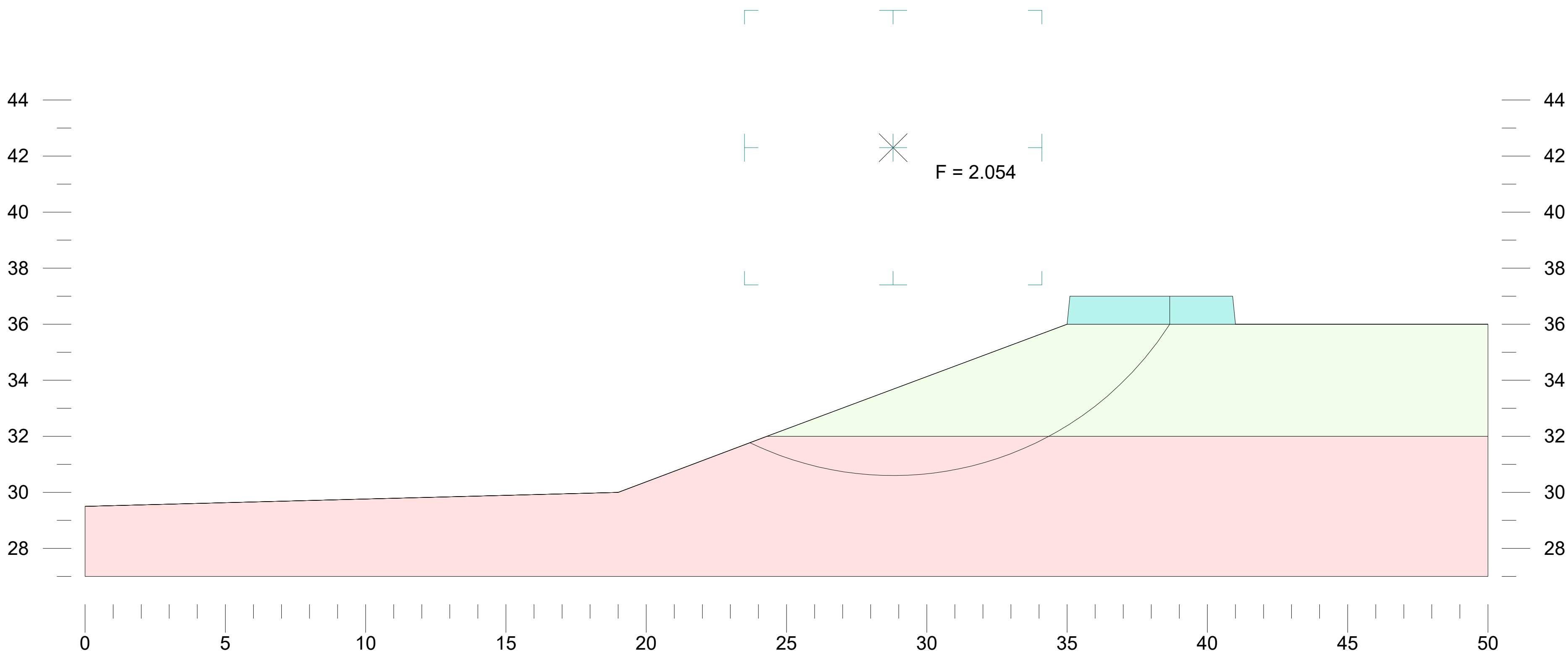
Factor of Safety	**1.78**

Ref:

Blake, T.F., Hollingsworth, R.A., and Stewart, J.P., 2002, Recommended Procedures for Implementation of DMG Special Publication 117 Guidelines for analyzing and Mitigating Landslide Hazards in California, Southern California Earthquake Center

	Gamma pcf	C psf	Phi deg	Piezo Surf.
Footing Surcharge	1000	0	0	0
Fill Soils	129.5	200	29	0
Older Alluvium	141.7	100	34	0

Fenagh Engineering & Testing
Project No. 6484
Patriot H.S., Jurupa Valley
September 7, 2022
Cross Section A-A'
Bishop, Static

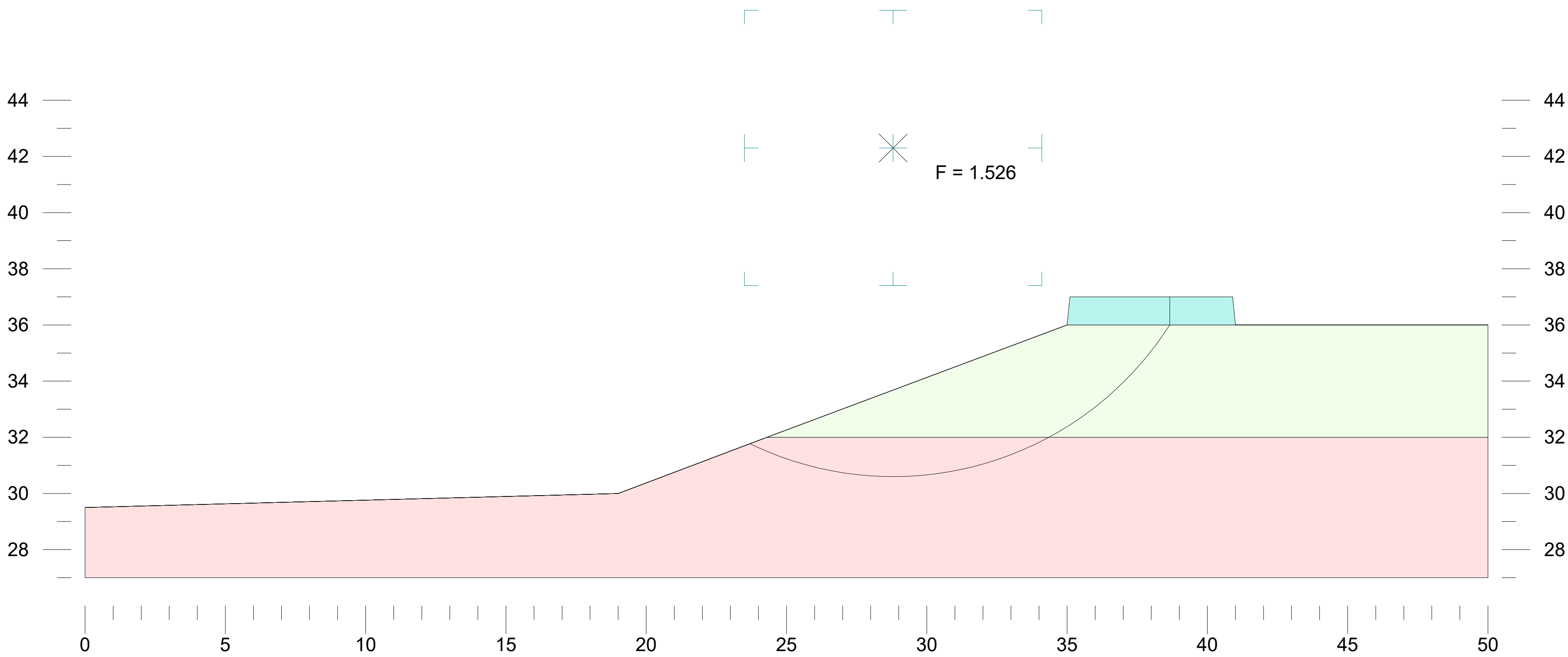


9/13/2022 5:29:55 PM C:\Users\sprince\OneDrive - Fenagh Engineering and Testing\Desktop\6484 Slope Stability\Cross-Section A-A' Bishop Static.gsl Fenagh Engineering & Testing F = 2.054

	Gamma pcf	C psf	Phi deg	Piezo Surf.
Footing Surcharge	1000	0	0	0
Fill Soils	129.5	200	29	0
Older Alluvium	141.7	100	34	0

Seismic coefficient = 0.27

Fenagh Engineering & Testing
Project No. 6484
Patriot H.S., Jurupa Valley
September 7, 2022
Cross Section A-A'
Bishop, Pseudo-Static



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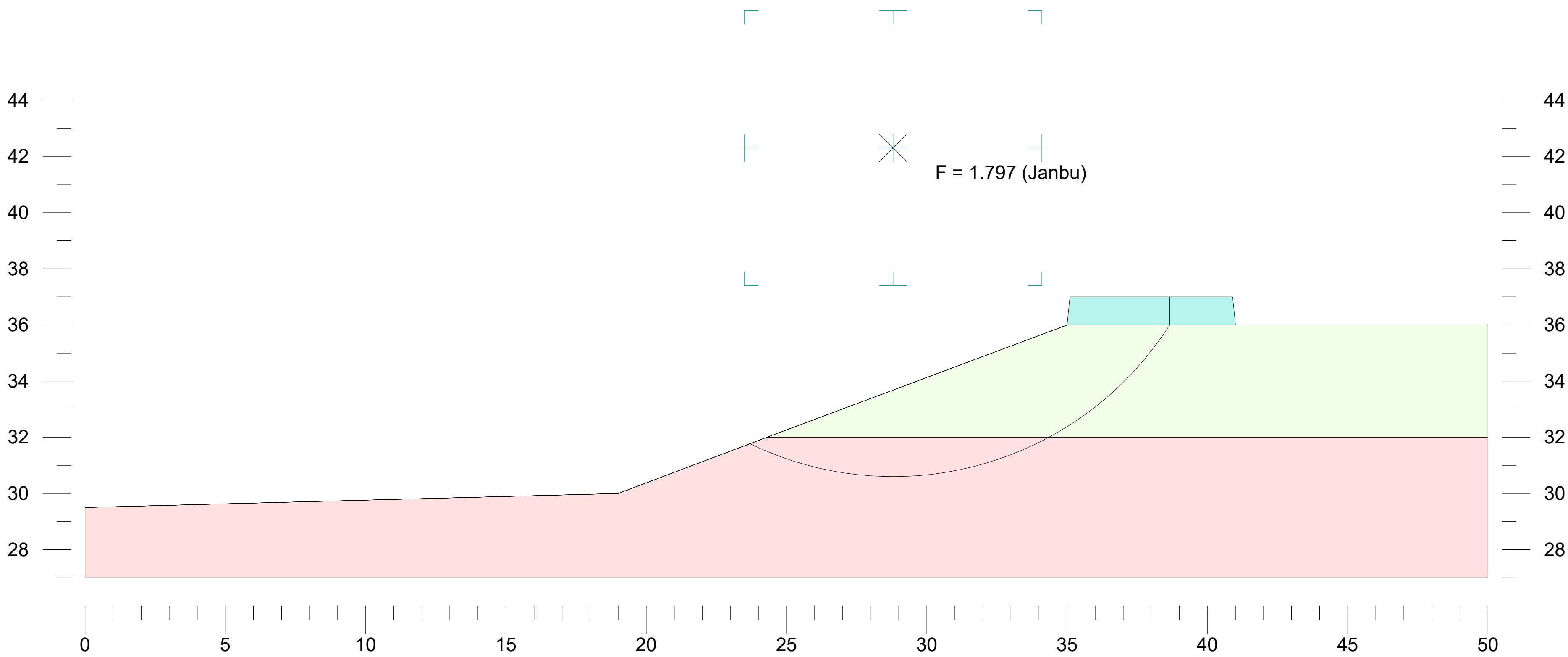
	Gamma pcf	C psf	Phi deg	Piezo Surf.
Footing Surcharge	1000	0	0	0
Fill Soils	129.5	200	29	0
Older Alluvium	141.7	100	34	0

Fenagh Engineering & Testing
Project No. 6484
Patriot H.S., Jurupa Valley
September 7, 2022
Cross Section A-A'
Janbu, Static



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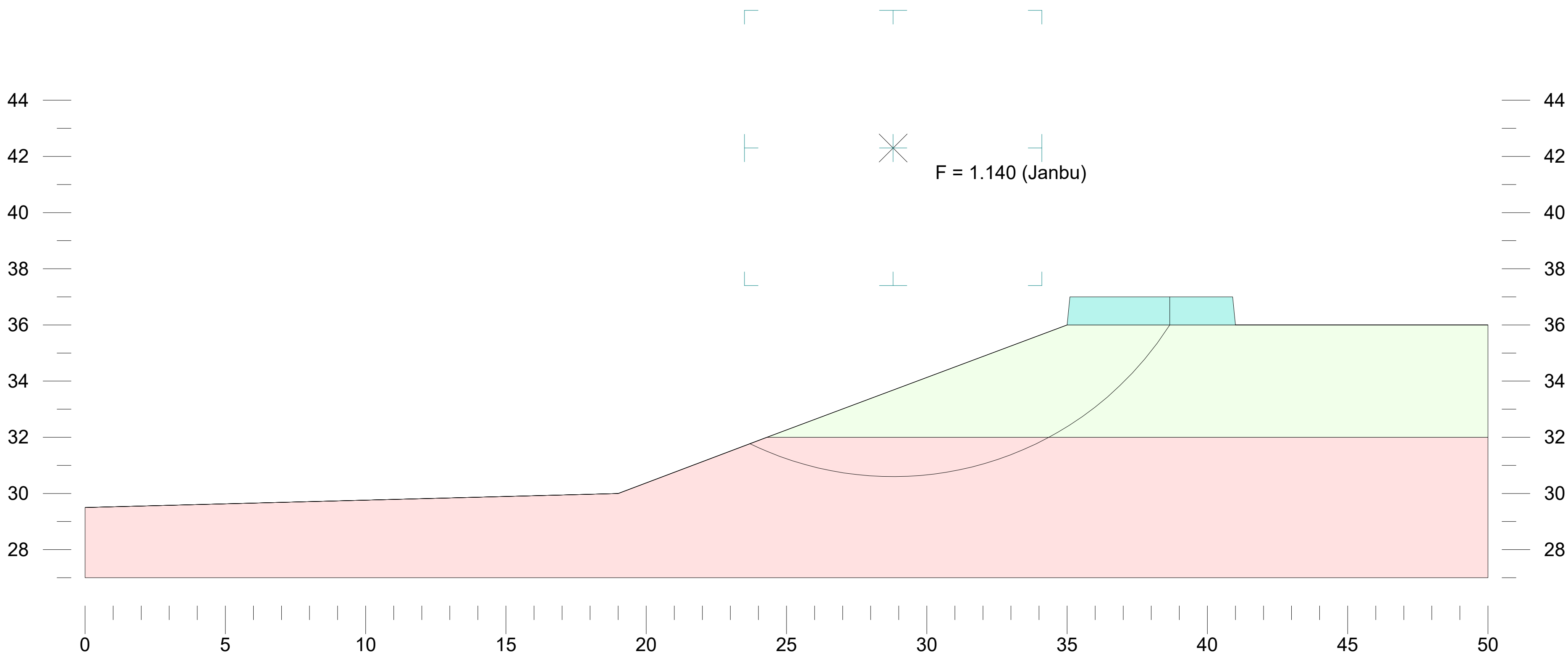
	Gamma pcf	C psf	Phi deg	Piezo Surf.
Footing Surcharge	1000	0	0	0
Fill Soils	129.5	200	29	0
Older Alluvium	141.7	100	34	0

Seismic coefficient = 0.27

Fenagh Engineering & Testing
Project No. 6484
Patriot H.S., Jurupa Valley
September 7, 2022
Cross Section A-A'
Janbu, Pseudo-Static



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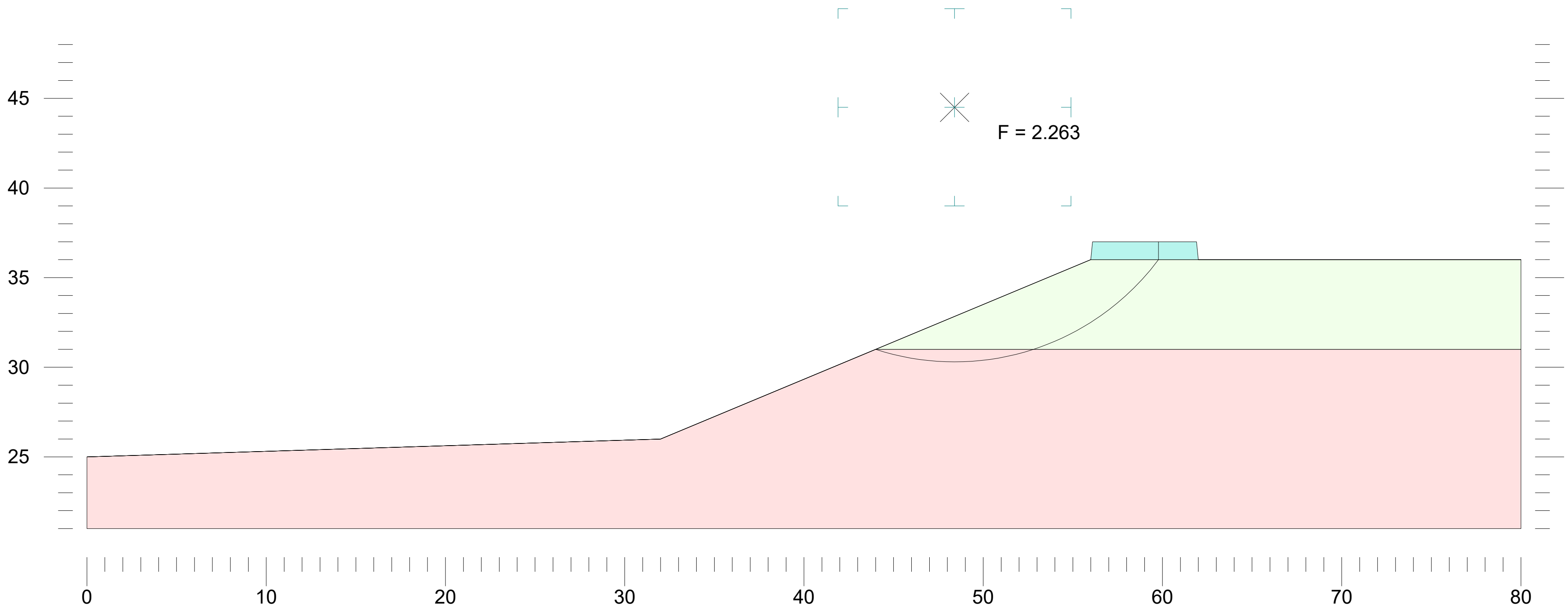
	Gamma pcf	C psf	Phi deg	Piezo Surf.
Footing Surcharge	1000	0	0	0
Fill Soils	129.5	200	29	0
Older Alluvium	133.1	250	30	0

Fenagh Engineering & Testing
Project No. 6484
Patriot H.S., Jurupa Valley
September 6, 2022
Cross Section B-B'
Bishop, Static

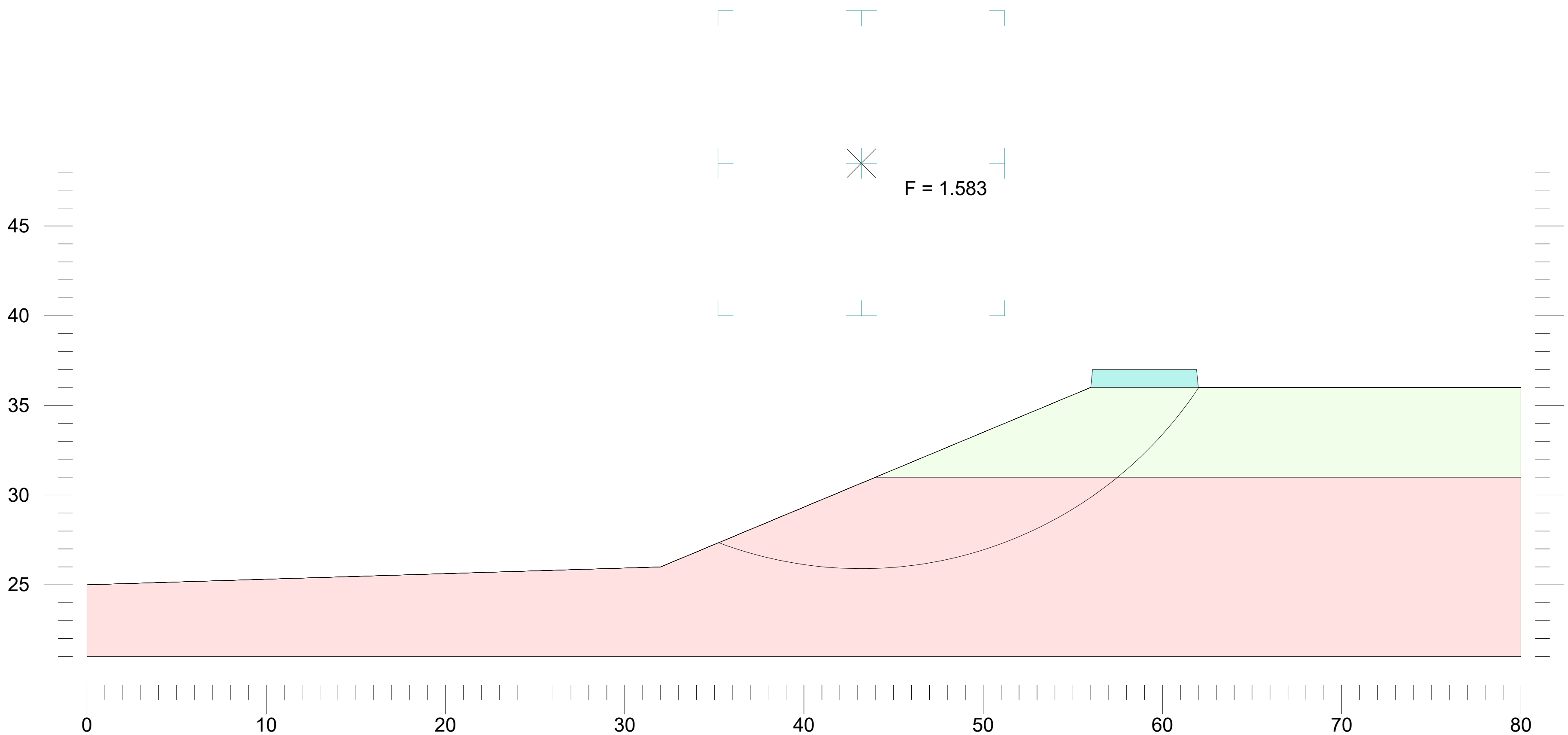


9/13/2022 5:34:47 PM C:\Users\sprince\OneDrive - Fenagh Engineering and Testing\Desktop\6484 Slope Stability\Cross-Section B-B' Bishop Static.gsl Fenagh Engineering & Testing F = 2.263

	Gamma pcf	C psf	Phi deg	Piezo Surf.
Footing Surcharge	1000	0	0	0
Fill Soils	129.5	200	29	0
Older Alluvium	133.1	250	30	0

Seismic coefficient = 0.27

Fenagh Engineering & Testing
Project No. 6484
Patriot H.S., Jurupa Valley
September 6, 2022
Cross Section B-B'
Bishop, Pseudo-Static



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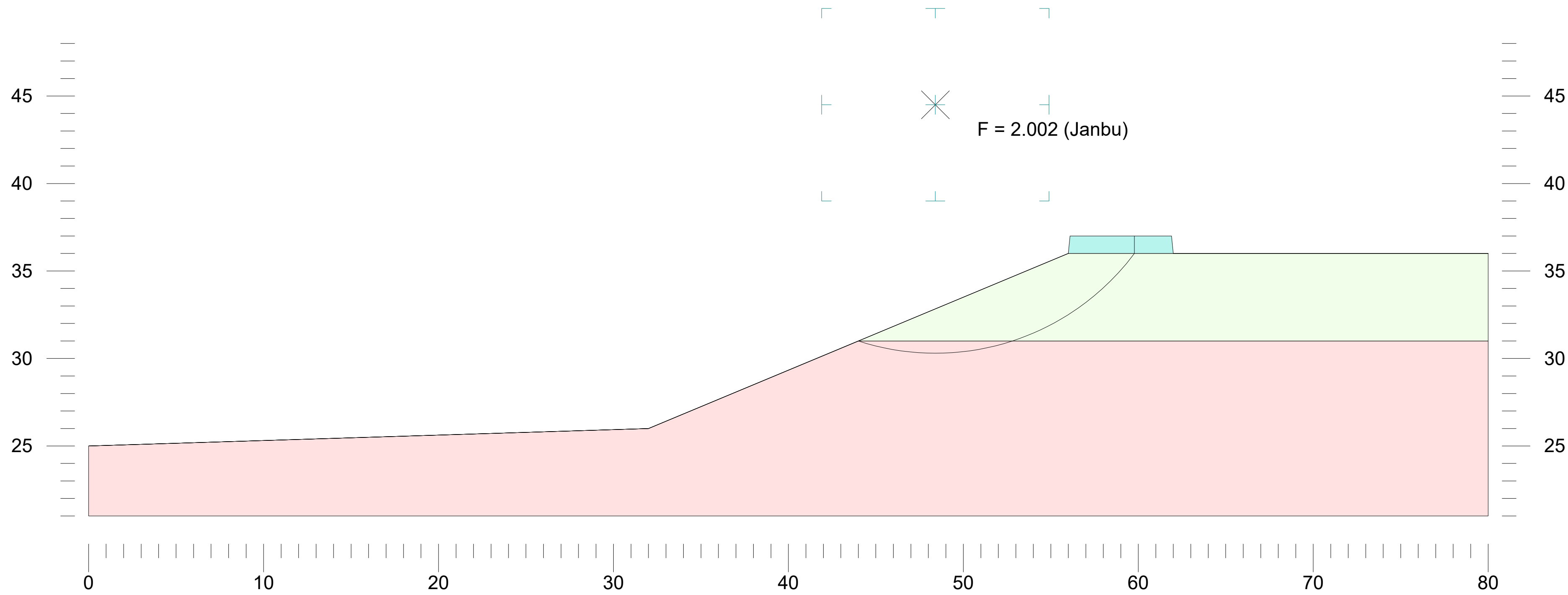
	Gamma pcf	C psf	Phi deg	Piezo Surf.
Footing Surcharge	1000	0	0	0
Fill Soils	129.5	200	29	0
Older Alluvium	133.1	250	30	0

Fenagh Engineering & Testing
Project No. 6484
Patriot H.S., Jurupa Valley
September 6, 2022
Cross Section B-B'
Janbu, Static



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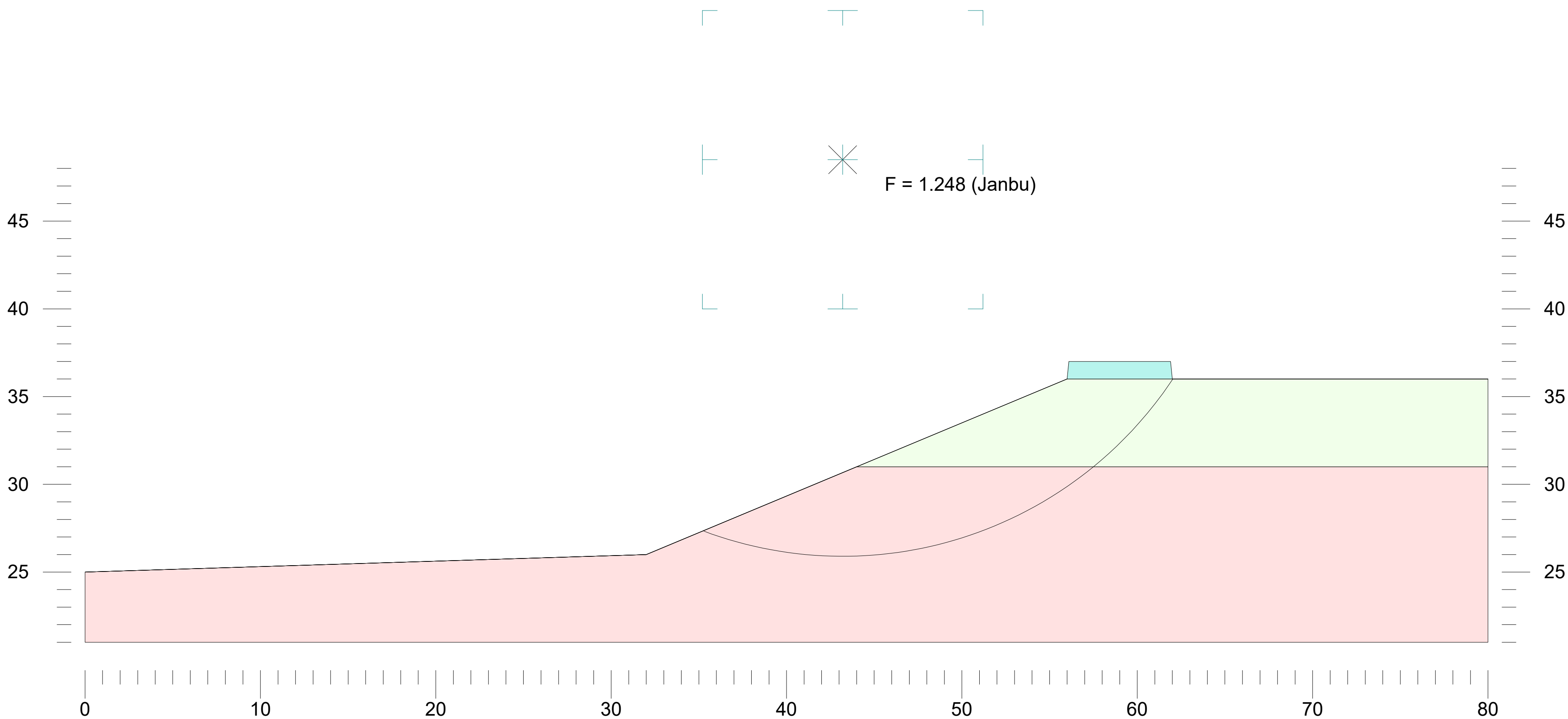
	Gamma pcf	C psf	Phi deg	Piezo Surf.
Footing Surcharge	1000	0	0	0
Fill Soils	129.5	200	29	0
Older Alluvium	133.1	250	30	0

Seismic coefficient = 0.27

Fenagh Engineering & Testing
Project No. 6484
Patriot H.S., Jurupa Valley
September 6, 2022
Cross Section B-B'
Janbu, Pseudo-Static



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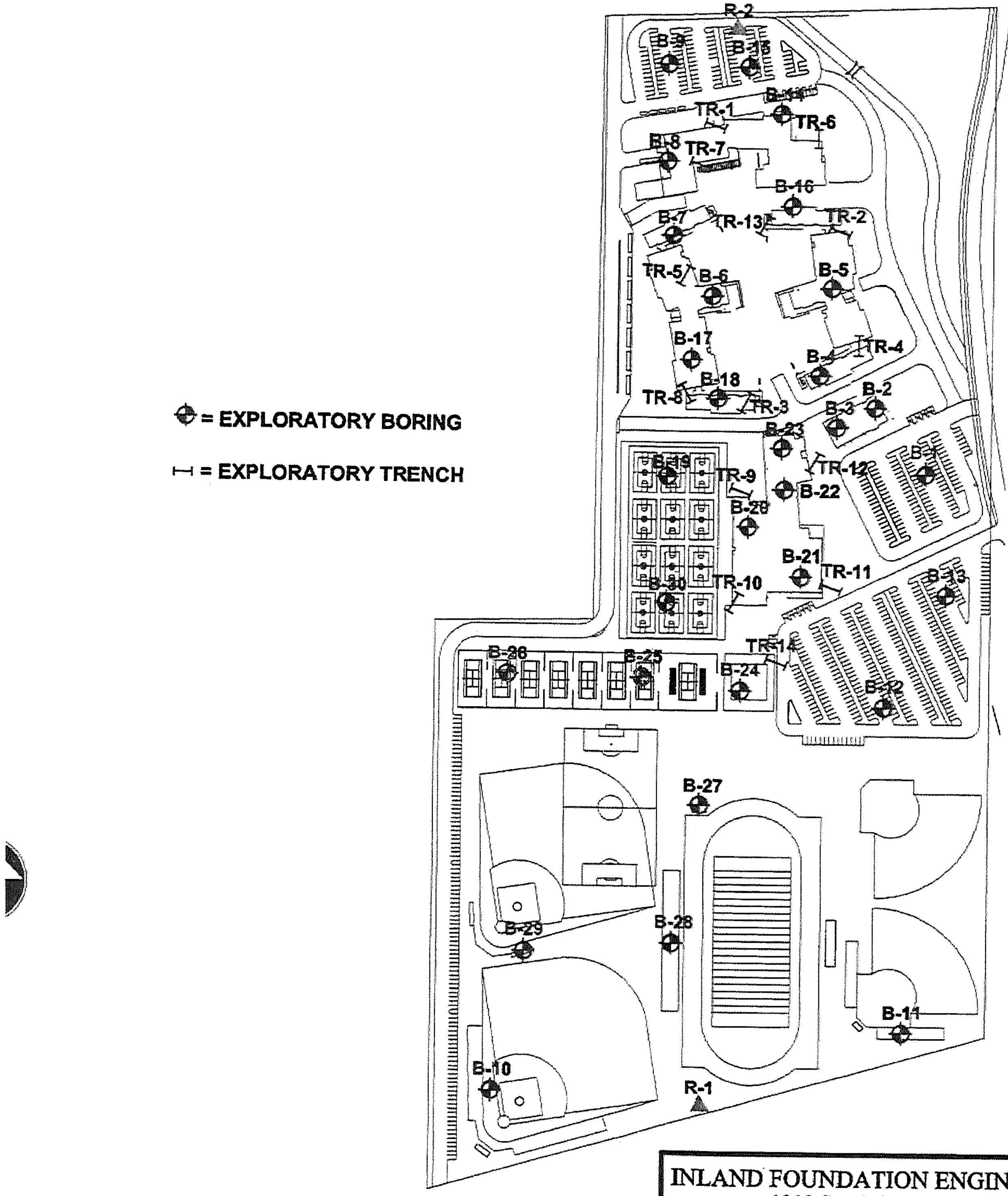
Fenagh Job 6484
Patriot High School, Jurupa Valley

APPENDIX F
PREVIOUS GEOTECHNICAL INVESTIGATION: PLOT PLAN, BORINGS & TRENCH LOGS

F-1 Plot Plan
F-2 Boring Logs B-01 through B-30
F-3 Trench Logs TR-01 through TR-14

- INVESTIGATION



INLAND FOUNDATION ENGINEERING, INC.
1310 South Santa Fe Avenue
San Jacinto, California
(909) 654-1555 FAX (909) 654-0551

PROPOSED HIGH SCHOOL NO. 3			
JURUPA RD; GLEN AVON AREA, RIV. CO., CALIF.			
DRAWN BY:	MC	JOB NO.:	J117-004
SCALE:	NTS	DATE:	JULY 2001

LOG OF BORING B-01

Elevation:	860.0	Date(s) Drilled:	6/20/01
Drilling Method:	Rotary Auger	Logged by:	M. Sullivan
rilling Rig:	CME 55	Hammer Type:	Auto-trip
Boring Diameter:	8-inches	Hammer Weight:	140 lb.
		Hammer Drop:	30-inches

SUMMARY OF SUBSURFACE CONDITIONS

This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered and is representative of interpretations made during drilling. Contrasting data derived from laboratory analysis may not be reflected in these representations.

DEPTH (ft)	GRAPHIC	USCS	Description	SAMPLES: DRIVE SAMPLE	SAMPLES: BULK SAMPLE	SAMPLES: SAMPLE TYPE	BLOWS/6"	MOISTURE (%)	DRY UNIT WT. (pcf)	RELATIVE COMPACTION (%)
		SM	SILTY SAND, fine to medium grained with coarse grained sand, brown, slightly moist, medium dense			B				
		SM	SILTY SAND, fine grained with medium grained sand and trace clay, brown, moist, medium dense			B SS	10 19	10	126	
5						SS	14 16	7	127	
10		SM	SILTY SAND, fine to medium grained with coarse grained sand, brown, slightly moist, medium dense			B SS	10 13	2	114	
15						SS	11 18	2	112	
			End of boring at 16.5 feet. No groundwater or mottling encountered.							

INLAND FOUNDATION ENGINEERING, INC.

Geotechnical Investigation
Jurupa Road
Glen Avon Area, CA
Project No. J117-004

Figure No. A-2

LOG OF BORING B-02

Elevation:	885.0	Date(s) Drilled:	6/20/01	Logged by:	M. Sullivan
Drilling Method:	Rotary Auger			Hammer Type:	Auto-trip
illing Rig:	CME 55			Hammer Weight:	140 lb.
Boring Diameter:	8-inches			Hammer Drop:	30-inches

SUMMARY OF SUBSURFACE CONDITIONS

This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered and is representative of interpretations made during drilling. Contrasting data derived from laboratory analysis may not be reflected in these representations.

DEPTH (ft)	USCS	Description	SAMPLE TYPE	BLOWS/6"	MOISTURE (%)	DRY UNIT WT. (pcf)	RELATIVE COMPACTION (%)
	SM	SILTY SAND, fine to medium grained with trace clay, dark red-brown, slightly moist, medium dense	B				
	SC	CLAYEY SAND, fine grained, red-brown, slightly moist, medium dense	B SS	9 10	11	126	
5			SS	10 14	9	121	
10		SANDY CLAY, fine grained, red-brown, slightly moist, hard	SS	30 50/4"	8	121	
15			SS	16 28	17	104	
20			SS	30 50/3"	12	126	
25	BR	GRANITE, slightly weathered	SPT	37 50	7		

End of boring at 28 feet. No groundwater or mottling encountered.

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Geotechnical Investigation
Jurupa Road
Glen Avon Area, CA
Project No. J117-004

Figure No. **A-3**

LOG OF BORING B-03

Elevation:	875.0	Logged by:	M. Sullivan
Date(s) Drilled:	6/20/01	Hammer Type:	Auto-trip
Drilling Method:	Rotary Auger	Hammer Weight:	140 lb.
rilling Rig:	CME 55	Hammer Drop:	30-inches
Boring Diameter:	8-inches		

SUMMARY OF SUBSURFACE CONDITIONS

This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered and is representative of interpretations made during drilling. Contrasting data derived from laboratory analysis may not be reflected in these representations.

DEPTH (ft)	GRAPHIC	USCS	Description	SAMPLE TYPE	BLOWS/6"	MOISTURE (%)	DRY UNIT WT. (pcf)	RELATIVE COMPACTION (%)
		SM	SILTY SAND, fine to medium grained with clay, dark red-brown, slightly moist, medium dense, porous	B				
				SS		11	125	
5		SC	CLAYEY SAND, fine grained, red-brown, moist, medium dense	B				
				SS	11 17	11	124	
				SS	9 13	9	132	
10		SM	SILTY SAND, fine to medium grained, red-brown, slightly moist, dense	SS	16 23	6	100	
15				SS	26 50/4"	8	115	
20				SPT	23 40	9		
		BR	GRANITE, slightly weathered					
25				SPT	39 50/4"	10		
			End of boring at 26 feet. No groundwater or mottling encountered.					

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Geotechnical Investigation
Jurupa Road
Glen Avon Area, CA
Project No. J117-004

Figure No. A-4

LOG OF BORING B-04

Elevation:	888.0	Date(s) Drilled:	6/20/01
Drilling Method:	Rotary Auger	Logged by:	M. Sullivan
Drilling Rig:	CME 55	Hammer Type:	Auto-trip
Boring Diameter:	8-inches	Hammer Weight:	140 lb.
		Hammer Drop:	30-inches

SUMMARY OF SUBSURFACE CONDITIONS

This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered and is representative of interpretations made during drilling. Contrasting data derived from laboratory analysis may not be reflected in these representations.

DEPTH (ft)	USCS	SUMMARY OF SUBSURFACE CONDITIONS	SAMPLE TYPE	BLOWS/6"	MOISTURE (%)	DRY UNIT WT. (pcf)	RELATIVE COMPACTION (%)
0	SM	SILTY SAND, fine grained with clay, red-brown, slightly moist, dense	B				
			SS	20 50/4"	10	124	
5	SM	SILTY SAND, fine to medium grained with trace clay, red-brown, slightly moist, dense	B SS	35 50/5"	7	130	
10			SS	24 25	5	120	
15	SM	SILTY SAND, fine grained with clay, red-brown, slightly moist, dense to very dense	SS	20 50/4"	7	115	
20			SPT	14 16	11		
25			SPT	23 42	11		
30			SPT	27 30	11		
35	BR	GRANITE, slightly weathered End of boring at 35 feet. No groundwater or mottling encountered.	SPT	50			

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Geotechnical Investigation
Jurupa Road
Glen Avon Area, CA
Project No. J117-004

Figure No. A-5

LOG OF BORING B-05

Elevation:	879.0	Logged by:	M. Sullivan
Date(s) Drilled:	6/20/01	Hammer Type:	Auto-trip
Drilling Method:	Rotary Auger	Hammer Weight:	140 lb.
Drilling Rig:	CME 55	Hammer Drop:	30-inches
Boring Diameter:	8-inches		

SUMMARY OF SUBSURFACE CONDITIONS

This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered and is representative of interpretations made during drilling. Contrasting data derived from laboratory analysis may not be reflected in these representations.

DEPTH (ft)	USCS	Description	Sample Type	BLOWS/6"	MOISTURE (%)	DRY UNIT WT. (pcf)	RELATIVE COMPACTION (%)
0	SM	SILTY SAND, fine grained with clay, red-brown, slightly moist, loose	B				
			SS	4 4	6	112	
5			SS	16	12	116	
	SM	SILTY SAND, fine to medium grained with clay and trace gravel, red-brown, slightly moist, dense	B	50/1"			
			SS	14 50	7	123	
10	SC	CLAYEY SAND, fine grained, red-brown, moist, very dense	SS	40 50/4"	10	120	
15			SS	50	8	113	
20			SPT	34 24	9		
	BR	GRANITE, slightly weathered					

End of boring at 23 feet. No groundwater or mottling encountered.

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Geotechnical Investigation
Jurupa Road
Glen Avon Area, CA
Project No. J117-004

Figure No. A-6

LOG OF BORING B-06

Elevation:	870.0	Date(s) Drilled:	6/20/01	Logged by:	M. Sullivan
Drilling Method:	Rotary Auger			Hammer Type:	Auto-trip
Drilling Rig:	CME 55			Hammer Weight:	140 lb.
Boring Diameter:	8-inches			Hammer Drop:	30-inches

SUMMARY OF SUBSURFACE CONDITIONS

This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered and is representative of interpretations made during drilling. Contrasting data derived from laboratory analysis may not be reflected in these representations.

DEPTH (ft)	GRAPHIC	USCS	SUMMARY OF SUBSURFACE CONDITIONS	SAMPLES: DRIVE SAMPLE	SAMPLES: BULK SAMPLE	SAMPLES: SAMPLE TYPE	BLOWS/6"	MOISTURE (%)	DRY UNIT WT. (pcf)	RELATIVE COMPACTION (%)
0		SM	SILTY SAND, fine grained with clay, red-brown, slightly moist, loose		■	B				
				☒	■	SS	3 4	8	120	
5				☒	■	SS	3 3	6	104	
10		SM	SILTY SAND, fine to medium grained with trace clay, red-brown, slightly moist, dense	☒		SS	15 24	6	119	
15		BR	GRANITE, slightly weathered	☒		SS	16 50	8	105	

End of boring at 17.5 feet. No groundwater or mottling encountered.

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Geotechnical Investigation
Jurupa Road
Glen Avon Area, CA
Project No. J117-004

Figure No. **A-7**

LOG OF BORING B-07

Elevation:	865.0	Date(s) Drilled:	6/20/01	Logged by:	M. Sullivan
Drilling Method:	Rotary Auger			Hammer Type:	Auto-trip
rilling Rig:	CME 55			Hammer Weight:	140 lb.
Boring Diameter:	8-inches			Hammer Drop:	30-inches

SUMMARY OF SUBSURFACE CONDITIONS

This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered and is representative of interpretations made during drilling. Contrasting data derived from laboratory analysis may not be reflected in these representations.

DEPTH (ft)	GRAPHIC	USCS	SUMMARY OF SUBSURFACE CONDITIONS	SAMPLES: DRIVE SAMPLE	SAMPLES: BULK SAMPLE	SAMPLES: SAMPLE TYPE	BLOWS/6"	MOISTURE (%)	DRY UNIT WT. (pcf)	RELATIVE COMPACTION (%)
		SM	SILTY SAND, fine grained with medium grained sand and trace clay, dark brown, slightly moist, loose to dense			B				
						SS	4 2	4	104	
5										
						SS	4 6	8	117	
10										
						SS	16 50	9	127	
		GP	GRAVEL, medium dense, slightly moist							
		BR	GRANITE, highly to moderately weathered, light brown.							
15			End of boring at 15.2 feet. No groundwater or mottling encountered.			NR	50/2"			

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Geotechnical Investigation
Jurupa Road
Glen Avon Area, CA
Project No. J117-004

Figure No. A-8

LOG OF BORING B-08

Elevation:	865.0	Date(s) Drilled:	6/20/01
Drilling Method:	Rotary Auger	Logged by:	M. Sullivan
Drilling Rig:	CME 55	Hammer Type:	Auto-trip
Boring Diameter:	8-inches	Hammer Weight:	140 lb.
		Hammer Drop:	30-inches

SUMMARY OF SUBSURFACE CONDITIONS

This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered and is representative of interpretations made during drilling. Contrasting data derived from laboratory analysis may not be reflected in these representations.

DEPTH (ft)	GRAPHIC	USCS	SUMMARY OF SUBSURFACE CONDITIONS	SAMPLES: DRIVE SAMPLE	SAMPLES: BULK SAMPLE	SAMPLES: SAMPLE TYPE	BLOWS/6"	MOISTURE (%)	DRY UNIT WT. (pcf)	RELATIVE COMPACTION (%)
		SM	SILTY SAND, fine to medium grained with trace clay, red-brown, slightly moist, loose			B				
						SS	3 4	4	110	
5						SS	2 2	9	116	
						SS	5 10	6	132	
10		SM	GRANITE, highly weathered							
		BR	GRAVEL, medium dense, slightly moist			SS	25 50/4"	14	121	
15			End of boring at 15 feet. No groundwater or mottling encountered.							

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Geotechnical Investigation
Jurupa Road
Glen Avon Area, CA
Project No. J117-004

Figure No. A-9

LOG OF BORING B-09

Elevation:	855.0	Date(s) Drilled:	6/20/01
Drilling Method:	Rotary Auger	Logged by:	M. Sullivan
Drilling Rig:	CME 55	Hammer Type:	Auto-trip
Boring Diameter:	8-inches	Hammer Weight:	140 lb.
		Hammer Drop:	30-inches

SUMMARY OF SUBSURFACE CONDITIONS

This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered and is representative of interpretations made during drilling. Contrasting data derived from laboratory analysis may not be reflected in these representations.

DEPTH (ft)	GRAPHIC	USCS	DESCRIPTION	DRIVE SAMPLE	BULK SAMPLE	SAMPLE TYPE	BLOWS/6"	MOISTURE (%)	DRY UNIT WT. (pcf)	RELATIVE COMPACTION (%)
		SM	SILTY SAND, fine to medium grained, brown, slightly moist, loose			B				
						SS	2 11	5	110	
5						SPT	2 2	8		
10		BR	GRANITE, highly weathered			SS	30 50	6	127	
			End of boring at 11 feet. No groundwater or mottling encountered.							

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Geotechnical Investigation
Jurupa Road
Glen Avon Area, CA
Project No. J117-004

Figure No. A-10

LOG OF BORING B-10

Elevation:	810.0	Date(s) Drilled:	6/20/01
Logged by:	M. Sullivan		
Drilling Method:	Rotary Auger	Hammer Type:	Auto-trip
Drilling Rig:	CME 55	Hammer Weight:	140 lb.
Boring Diameter:	8-inches	Hammer Drop:	30-inches

SUMMARY OF SUBSURFACE CONDITIONS

This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered and is representative of interpretations made during drilling. Contrasting data derived from laboratory analysis may not be reflected in these representations.

DEPTH (ft)	USCS	Description	SAMPLE TYPE	BLOWS/6"	MOISTURE (%)	DRY UNIT WT. (pcf)	RELATIVE COMPACTION (%)
	SM	SILTY SAND, fine grained with coarse grained and trace clay, brown, slightly moist, medium dense	B				
		SANDY CLAY, fine grained, brown-red, slightly moist, hard, moderately cemented	SS	12	7	122	
			B	50			
5			SS	50	6	102	
10			SPT	50	10		
15		End of boring at 15.2 feet. No groundwater or mottling encountered.	NR	50/2"			

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Geotechnical Investigation
Jurupa Road
Glen Avon Area, CA
Project No. J117-004

Figure No. **A-11**

LOG OF BORING B-11

Elevation:	805.0	Date(s) Drilled:	6/20/01
Drilling Method:	Rotary Auger	Logged by:	M. Sullivan
Drilling Rig:	CME 55	Hammer Type:	Auto-trip
Boring Diameter:	8-inches	Hammer Weight:	140 lb.
		Hammer Drop:	30-inches

SUMMARY OF SUBSURFACE CONDITIONS

This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered and is representative of interpretations made during drilling. Contrasting data derived from laboratory analysis may not be reflected in these representations.

DEPTH (ft)	GRAPHIC	USCS	SUMMARY OF SUBSURFACE CONDITIONS	SAMPLES: DRIVE SAMPLE	SAMPLES: BULK SAMPLE	SAMPLES: SAMPLE TYPE	BLOWS/6"	MOISTURE (%)	DRY UNIT WT. (pcf)	RELATIVE COMPACTION (%)
0		SM	SILTY SAND, fine grained, red-brown, slightly moist, dense to very dense, moderately cemented		X	B				
				X	X	SS	21 21	7	128	
5				X	X	SS	50	7	121	
		SM	SILTY SAND, fine grained with clay, red-brown, slightly moist, moderately cemented							
10				X		SS	11 14	4	117	
15				X		SS	30 50/3"	9	123	
			End of boring at 16.3 feet. No groundwater or mottling encountered.							

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Geotechnical Investigation
Jurupa Road
Glen Avon Area, CA
Project No. J117-004

Figure No. A-12

LOG OF BORING B-12

Elevation:	810.0	Date(s) Drilled:	6/22/01	Logged by:	M. Sullivan
Drilling Method:	Rotary Auger			Hammer Type:	Auto-trip
Drilling Rig:	CME 55			Hammer Weight:	140 lb.
Boring Diameter:	8-inches			Hammer Drop:	30-inches

SUMMARY OF SUBSURFACE CONDITIONS

This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered and is representative of interpretations made during drilling. Contrasting data derived from laboratory analysis may not be reflected in these representations.

DEPTH (ft)	GRAPHIC	USCS	SUMMARY OF SUBSURFACE CONDITIONS	SAMPLES: DRIVE SAMPLE	SAMPLES: BULK SAMPLE	SAMPLES: SAMPLE TYPE	BLOWS/6"	MOISTURE (%)	DRY UNIT WT. (pcf)	RELATIVE COMPACTION (%)
0		SM	SILTY SAND, fine to medium grained with trace clay, red-brown, slightly moist, very dense, moderately cemented			B				
						SS	50	9	125	
5						SS	50	8	107	
10		SM	SILTY SAND, fine to coarse grained, brown, moist, medium dense			SS	21 22	9	121	
15						SS	12 19	2	123	
			End of boring at 16.5 feet. No groundwater or mottling encountered.							

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Geotechnical Investigation
Jurupa Road
Glen Avon Area, CA
Project No. J117-004

Figure No. A-13

LOG OF BORING B-13

Elevation:	825.0	Date(s) Drilled:	6/22/01
Drilling Method:	Rotary Auger	Logged by:	M. Sullivan
Drilling Rig:	CME 55	Hammer Type:	Auto-trip
Boring Diameter:	8-inches	Hammer Weight:	140 lb.
		Hammer Drop:	30-inches

SUMMARY OF SUBSURFACE CONDITIONS

This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered and is representative of interpretations made during drilling. Contrasting data derived from laboratory analysis may not be reflected in these representations.

DEPTH (ft)	GRAPHIC	USCS	SUMMARY OF SUBSURFACE CONDITIONS	SAMPLES: DRIVE SAMPLE	SAMPLES: BULK SAMPLE	SAMPLES: SAMPLE TYPE	BLOWS/6"	MOISTURE (%)	DRY UNIT WT. (pcf)	RELATIVE COMPACTION (%)
		SM	SILTY SAND, fine grained with trace clay, brown, slightly moist, medium dense			B				
			SANDY CLAY, fine grained, red-brown, slightly moist, stiff, moderately cemented			SS	15	12	119	
5						B	50/4"			
						SS	16 33	12	116	
10						SS	25 50/5"	9	123	
		SM	SILTY SAND, fine grained, brown, slightly moist, dense							
15						SS	25 35	6	115	
			End of boring at 16.5 feet. No groundwater or mottling encountered.							

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Geotechnical Investigation
Jurupa Road
Glen Avon Area, CA
Project No. J117-004

Figure No. A-14

LOG OF BORING B-14

Elevation:	859.0	Date(s) Drilled:	6/22/01
Drilling Method:	Rotary Auger	Logged by:	M. Sullivan
Drilling Rig:	CME 55	Hammer Type:	Auto-trip
Boring Diameter:	8-inches	Hammer Weight:	140 lb.
		Hammer Drop:	30-inches

SUMMARY OF SUBSURFACE CONDITIONS

This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered and is representative of interpretations made during drilling. Contrasting data derived from laboratory analysis may not be reflected in these representations.

DEPTH (ft)	USCS	Description	SAMPLE TYPE	BLOWS/6"	MOISTURE (%)	DRY UNIT WT. (pcf)	RELATIVE COMPACTION (%)
	SM	SILTY SAND, fine grained, brown, slightly moist, medium dense	B				
			SS	8 9	5	112	
5	SM	SILTY SAND, fine grained with clay, red-brown, slightly moist, medium dense	B				
			SS	4 10	5	114	
			SS	25 50/5"	5	128	
10	SC	CLAYEY SAND, fine to medium grained with trace coarse grained sand, red-brown, dry, very dense, highly cemented					
			SS	36 40	7	123	
15	SM	SILTY SAND, fine grained with clay, brown, moist, very dense	SPT	30 36	11		
20	BR	GRANITE, moderately weathered	SPT	20 50/5"	10		

End of boring at 24 feet. No groundwater or mottling encountered.

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Geotechnical Investigation
Jurupa Road
Glen Avon Area, CA
Project No. J117-004

Figure No. A-15

LOG OF BORING B-15

Elevation: 859.0 | Date(s) Drilled: 6/22/01 | Logged by: M. Sullivan
Drilling Method: Rotary Auger | Hammer Type: Auto-trip
Drilling Rig: CME 55 | Hammer Weight: 140 lb.
Boring Diameter: 8-inches | Hammer Drop: 30-inches

SUMMARY OF SUBSURFACE CONDITIONS

This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered and is representative of interpretations made during drilling. Contrasting data derived from laboratory analysis may not be reflected in these representations.

DEPTH (ft)	USCS	Description	SAMPLE TYPE	BLOWS/6"	MOISTURE (%)	DRY UNIT WT. (pcf)	RELATIVE COMPACTION (%)
0	SC	CLAYEY SAND, fine to medium grained, red-brown, slightly moist, very dense, highly cemented	B				
			SS	10 50/3"	6	117	
5	SM	SILTY SAND, fine to coarse grained, brown, slightly moist, very dense	B SS	31 50/4"	6	124	
10			SS	30 50/5"	4	117	
	SM	SILTY SAND, fine to medium grained with trace gravel, red-brown, slightly moist, very dense					
15			SS	27 50/4"	11	123	
20	BR	GRANITE, moderately weathered	SS	12 23	14		

End of boring at 24 feet. No groundwater or mottling encountered.

INLAND FOUNDATION ENGINEERING, INC.

Geotechnical Investigation
Jurupa Road
Glen Avon Area, CA
Project No. J117-004

Figure No. A-16

LOG OF BORING B-16

Elevation:	875.0	Date(s) Drilled:	6/22/01
Drilling Method:	Rotary Auger	Logged by:	M. Sullivan
Drilling Rig:	CME 55	Hammer Type:	Auto-trip
Boring Diameter:	8-inches	Hammer Weight:	140 lb.
		Hammer Drop:	30-inches

SUMMARY OF SUBSURFACE CONDITIONS

This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered and is representative of interpretations made during drilling. Contrasting data derived from laboratory analysis may not be reflected in these representations.

DEPTH (ft)	USCS	Description	SAMPLE TYPE	BLOWS/6"	MOISTURE (%)	DRY UNIT WT. (pcf)	RELATIVE COMPACTION (%)
	SM	**SILTY SAND**, fine to medium grained with clay, red-brown, slightly moist, medium dense	B				
			SS	10 15	8	109	
5	SC	**CLAYEY SAND**, fine to medium grained, red-brown, moist, dense to very dense	B				
			SS	50/3"	10	108	
			SS	30 35	4	123	
10			SS	25 35	3	128	
15	SM	**SILTY SAND**, fine to medium grained with clay, red-brown, dry, very dense	SS	34 50/4"	2	136	
20			SPT	37 47	9		
25			SPT	32 50/4"	8		
	BR	**GRANITE**, moderately weathered					

End of boring at 26.3 feet. No groundwater or mottling encountered.

INLAND FOUNDATION ENGINEERING, INC.	Geotechnical Investigation Jurupa Road Glen Avon Area, CA Project No. J117-004	Figure No. A-17

LOG OF BORING B-17

Elevation:	860.0	Date(s) Drilled:	6/22/01
Drilling Method:	Rotary Auger	Logged by:	M. Sullivan
Drilling Rig:	CME 55	Hammer Type:	Auto-trip
Boring Diameter:	8-inches	Hammer Weight:	140 lb.
		Hammer Drop:	30-inches

SUMMARY OF SUBSURFACE CONDITIONS

This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered and is representative of interpretations made during drilling. Contrasting data derived from laboratory analysis may not be reflected in these representations.

DEPTH (ft)	USCS	Description	SAMPLE TYPE	BLOWS/6"	MOISTURE (%)	DRY UNIT WT. (pcf)	RELATIVE COMPACTION (%)
	SM	SILTY SAND, fine grained with medium grained sand, brown, dry, medium dense	B				
			SS	20 44	12	117	
	SM	SILTY SAND, fine to medium grained with coarse grained sand, brown, slightly moist, medium dense	B				
5			SS	13 13	3	123	
			SS	17 22	7	117	
10			SS	16 24	3	119	
15			SS	36 26	3	132	
	GP	GRAVEL, brown, slightly moist, dense					
	SM	SILTY SAND, fine to medium grained with trace clay, red-brown, slightly moist, dense					
20			SPT	20 50/2"	68		
	BR	GRANITE, moderately weathered					
25			SPT	40 50/4"	3		
		End of boring at 25.8 feet. No groundwater or mottling encountered.					

INLAND FOUNDATION ENGINEERING, INC.

Geotechnical Investigation
Jurupa Road
Glen Avon Area, CA
Project No. J117-004

Figure No. A-18

LOG OF BORING B-18

Elevation:	820.0	Date(s) Drilled:	6/22/01
Drilling Method:	Rotary Auger	Logged by:	M. Sullivan
Drilling Rig:	CME 55	Hammer Type:	Auto-trip
Boring Diameter:	8-inches	Hammer Weight:	140 lb.
		Hammer Drop:	30-inches

SUMMARY OF SUBSURFACE CONDITIONS

This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered and is representative of interpretations made during drilling. Contrasting data derived from laboratory analysis may not be reflected in these representations.

DEPTH (ft)	USCS	DESCRIPTION	SAMPLE TYPE	BLOWS/6"	MOISTURE (%)	DRY UNIT WT. (pcf)	RELATIVE COMPACTION (%)
0	SM	SILTY SAND, fine grained with clay, brown, slightly moist, medium dense	B				
	SC	CLAYEY SAND, fine to medium grained, red-brown, slightly moist, very dense, moderately cemented	B	16	10	117	
			SS	22			
5			SS	37 50	7	116	
10			SS	19 30	3	121	
	SM	SILTY SAND, fine to medium grained with trace gravel, brown, slightly moist, dense					
15			SS	34 50	8	129	
20	BR	GRANITE, moderately weathered	SPT	50/1"			
		End of boring at 20.1 feet. No groundwater or mottling encountered.					

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Geotechnical Investigation
Jurupa Road
Glen Avon Area, CA
Project No. J117-004

Figure No. A-19

LOG OF BORING B-19

Elevation:	825.0	Date(s) Drilled:	6/22/01
Drilling Method:	Rotary Auger	Logged by:	M. Sullivan
Drilling Rig:	CME 55	Hammer Type:	Auto-trip
Boring Diameter:	8-inches	Hammer Weight:	140 lb.
		Hammer Drop:	30-inches

SUMMARY OF SUBSURFACE CONDITIONS

This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered and is representative of interpretations made during drilling. Contrasting data derived from laboratory analysis may not be reflected in these representations.

DEPTH (ft)	USCS	DESCRIPTION	SAMPLE TYPE	BLOWS/6"	MOISTURE (%)	DRY UNIT WT. (pcf)	RELATIVE COMPACTION (%)
	SM	SILTY SAND, fine grained with clay, brown, slightly moist, medium dense					
	SC	CLAYEY SAND, fine to medium grained, red-brown, slightly moist, very dense, moderately cemented	B				
			SS	12 18	7	114	
5			SS	27 50	8	135	
10			SS	27 37	7	124	
15			SPT	13 16	9		
		End of boring at 16.5 feet. No groundwater or mottling encountered.					

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Geotechnical Investigation
Jurupa Road
Glen Avon Area, CA
Project No. J117-004

Figure No. A-20

LOG OF BORING B-20

Elevation:	825.0	Date(s) Drilled:	6/22/01
Drilling Method:	Rotary Auger	Logged by:	M. Sullivan
Drilling Rig:	CME 55	Hammer Type:	Auto-trip
Boring Diameter:	8-inches	Hammer Weight:	140 lb.
		Hammer Drop:	30-inches

SUMMARY OF SUBSURFACE CONDITIONS

This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered and is representative of interpretations made during drilling. Contrasting data derived from laboratory analysis may not be reflected in these representations.

DEPTH (ft)	USCS	SUMMARY OF SUBSURFACE CONDITIONS	SAMPLE TYPE	BLOWS/6"	MOISTURE (%)	DRY UNIT WT. (pcf)	RELATIVE COMPACTION (%)
0	SM	SILTY SAND, fine grained, brown, dry, loose	B				
	SC	CLAYEY SAND, fine to medium grained, red-brown, slightly moist, very dense, highly cemented	B				
			SS	9 50/5"	8	115	
5			SS	32 50	7	126	
10			SS	10 16	5	122	
15	SM	SILTY SAND, fine to medium grained with trace gravel, red-brown, dry, very dense	SS	21 50/5"	9	118	
20	BR	GRANITE, moderately weathered	SPT	30 50	8		
		End of boring at 21.5 feet. No groundwater or mottling encountered.					

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Geotechnical Investigation
Jurupa Road
Glen Avon Area, CA
Project No. J117-004

Figure No. **A-21**

LOG OF BORING B-21

Elevation:	800.0	Date(s) Drilled:	6/25/01
Drilling Method:	Rotary Auger	Logged by:	M. Sullivan
Drilling Rig:	CME 55	Hammer Type:	Auto-trip
Boring Diameter:	8-inches	Hammer Weight:	140 lb.
		Hammer Drop:	30-inches

SUMMARY OF SUBSURFACE CONDITIONS

This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered and is representative of interpretations made during drilling. Contrasting data derived from laboratory analysis may not be reflected in these representations.

DEPTH (ft)	GRAPHIC	USCS	Description	SAMPLES: DRIVE SAMPLE	SAMPLES: BULK SAMPLE	SAMPLES: SAMPLE TYPE	BLOWS/6"	MOISTURE (%)	DRY UNIT WT. (pcf)	RELATIVE COMPACTION (%)
0		SM	SILTY SAND, fine to medium grained with coarse grained sand, red-brown, slightly moist, dense to very dense		X	B				
5				X	X	SS	25 50/4"	6	130	
10		SM	SILTY SAND, fine to medium grained with coarse grained sand, red-brown, slightly moist, very dense	X		SS	50	8	103	
15		SM	SILTY SAND, fine to coarse grained, red-brown, slightly moist, very dense	X		SPT	25 50/4"	7		
20				X		SPT	22 34	6		
25		SM SW	SAND, fine to coarse grained, gray-brown, dry, very dense	X		SPT	26 34	2		
30				X		SPT	25 26	3		
35		SM	SILTY SAND, fine to medium grained with coarse grained sand, brown, dry to slightly moist, very dense	X		SPT	27 49	4		
40		ML	SANDY SILT, fine to medium grained, brown, moist, stiff	X		SPT	14 14	19		
45		SM	SILTY SAND, fine to coarse grained with trace clay, brown, moist, very dense	X		SPT	21 50	13		
50				X		SPT	31 50/5"	12		
			End of boring at 50.9 feet. No groundwater or mottling encountered.							

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Geotechnical Investigation
Jurupa Road
Glen Avon Area, CA
Project No. J117-004

Figure No. A-22

LOG OF BORING B-22

Elevation:	835.0	Date(s) Drilled:	6/25/01
Drilling Method:	Rotary Auger	Logged by:	M. Sullivan
Drilling Rig:	CME 55	Hammer Type:	Auto-trip
Boring Diameter:	8-inches	Hammer Weight:	140 lb.
		Hammer Drop:	30-inches

SUMMARY OF SUBSURFACE CONDITIONS

This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered and is representative of interpretations made during drilling. Contrasting data derived from laboratory analysis may not be reflected in these representations.

DEPTH (ft)	USCS	Description	SAMPLE TYPE	BLOWS/6"	MOISTURE (%)	DRY UNIT WT. (pcf)	RELATIVE COMPACTION (%)
0	SM	SILTY SAND, fine to medium grained, red-brown, slightly moist, loose to medium dense	B				
5			SS	6 7	5	117	
5	SM	SILTY SAND, fine to coarse grained with gravel, red-brown, slightly moist, dense	B				
			SS	30 30	3	135	
8	SM	SILTY SAND, fine to medium grained with clay, red-brown, slightly moist, dense					
10			SS	21 30	7	133	
15		grading to coarse grained	SS	17 20	6	127	
20			SS	19 30	6	131	
25			SS	23 50/5"	6	128	
30			SPT	14 50/5"	12		
35	BR	GRANITE, moderately weathered	SPT	28 46	9		

End of boring at 39 feet. No groundwater or mottling encountered.

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Geotechnical Investigation
Jurupa Road
Glen Avon Area, CA
Project No. J117-004

Figure No. A-23

LOG OF BORING B-23

Elevation:	840.0	Date(s) Drilled:	6/25/01	Logged by:	M. Sullivan
Drilling Method:	Rotary Auger			Hammer Type:	Auto-trip
Drilling Rig:	CME 55			Hammer Weight:	140 lb.
Boring Diameter:	8-inches			Hammer Drop:	30-inches

SUMMARY OF SUBSURFACE CONDITIONS

This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered and is representative of interpretations made during drilling. Contrasting data derived from laboratory analysis may not be reflected in these representations.

DEPTH (ft)	USCS	Description	Sample Type	Blows/6"	Moisture (%)	Dry Unit Wt. (pcf)	Relative Compaction (%)
0	SC	CLAYEY SAND, fine to medium grained, red-brown, moist, very dense	B				
			SS	13 50/5"	12	128	
5	SM	SILTY SAND, fine to medium grained, red-brown, slightly moist, dense to very dense	B				
			SS	23 50	7	111	
10			SS	30 50	7	118	
15			SS	16 41	9	122	

End of boring at 16.5 feet. No groundwater or mottling encountered.

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Geotechnical Investigation
Jurupa Road
Glen Avon Area, CA
Project No. J117-004

Figure No. A-24

LOG OF BORING B-24

Elevation:	795.0	Date(s) Drilled:	6/25/01
Drilling Method:	Rotary Auger	Logged by:	M. Sullivan
Drilling Rig:	CME 55	Hammer Type:	Auto-trip
Boring Diameter:	8-inches	Hammer Weight:	140 lb.
		Hammer Drop:	30-inches

SUMMARY OF SUBSURFACE CONDITIONS

This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered and is representative of interpretations made during drilling. Contrasting data derived from laboratory analysis may not be reflected in these representations.

DEPTH (ft)	GRAPHIC	USCS	SUMMARY OF SUBSURFACE CONDITIONS	DRIVE SAMPLE	BULK SAMPLE	SAMPLE TYPE	BLOWS/6"	MOISTURE (%)	DRY UNIT WT. (pcf)	RELATIVE COMPACTION (%)
0		SM	SILTY SAND, fine to medium grained with coarse grained sand, red-brown, slightly moist, dense		X	B				
5				X	X	SS	22 50	2	139	
8		SM	SILTY SAND, fine to coarse grained, red-brown, slightly moist, medium dense							
10				X		SS	9 12	3	119	
15				X		SS	14 22	3	124	
			End of boring at 16.5 feet. No groundwater or mottling encountered.							

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Geotechnical Investigation
Jurupa Road
Glen Avon Area, CA
Project No. J117-004

Figure No. A-25

LOG OF BORING B-25

Elevation:	800.0	Date(s) Drilled:	6/25/01
Drilling Method:	Rotary Auger	Logged by:	M. Sullivan
Drilling Rig:	CME 55	Hammer Type:	Auto-trip
Boring Diameter:	8-inches	Hammer Weight:	140 lb.
		Hammer Drop:	30-inches

SUMMARY OF SUBSURFACE CONDITIONS

This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered and is representative of interpretations made during drilling. Contrasting data derived from laboratory analysis may not be reflected in these representations.

DEPTH (ft)	USCS	DESCRIPTION	SAMPLE TYPE	BLOWS/6"	MOISTURE (%)	DRY UNIT WT. (pcf)	RELATIVE COMPACTION (%)
0	SM	SILTY SAND, fine to coarse grained, red-brown, slightly moist, dense	B				
5			SS	21 50	8	111	
7	SM	SILTY SAND, fine to medium grained, red-brown, slightly moist, dense					
10			SS	22 35	7	120	
15			SS	25 39	6	120	
		End of boring at 16.5 feet. No groundwater or mottling encountered.					

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Geotechnical Investigation
Jurupa Road
Glen Avon Area, CA
Project No. J117-004

Figure No. A-26

LOG OF BORING B-26

Elevation:	810.0	Date(s) Drilled:	6/25/01
Logged by:	M. Sullivan		
Drilling Method:	Rotary Auger	Hammer Type:	Auto-trip
rilling Rig:	CME 55	Hammer Weight:	140 lb.
Boring Diameter:	8-inches	Hammer Drop:	30-inches

SUMMARY OF SUBSURFACE CONDITIONS

This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered and is representative of interpretations made during drilling. Contrasting data derived from laboratory analysis may not be reflected in these representations.

DEPTH (ft)	GRAPHIC	USCS	SUMMARY OF SUBSURFACE CONDITIONS	SAMPLE TYPE	BLOWS/6"	MOISTURE (%)	DRY UNIT WT. (pcf)	RELATIVE COMPACTION (%)
		SM	SILTY SAND, fine to medium grained with trace clay, red-brown, slightly moist, medium dense	B				
				SS	6 9	9	116	
5		SM	SILTY SAND, fine to coarse grained, red-brown, slightly moist, medium dense	B SS	8 27	5	129	
10		SM	SILTY SAND, fine to medium grained, red-brown, slightly moist, dense	SS	15 22	8	125	
15				SS	18 42	7	127	
			End of boring at 16.5 feet. No groundwater or mottling encountered.					

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Geotechnical Investigation
Jurupa Road
Glen Avon Area, CA
Project No. J117-004

Figure No. A-27

LOG OF BORING B-27

Elevation:	800.0	Date(s) Drilled:	6/25/01
Drilling Method:	Rotary Auger	Logged by:	M. Sullivan
Drilling Rig:	CME 55	Hammer Type:	Auto-trip
Boring Diameter:	8-inches	Hammer Weight:	140 lb.
		Hammer Drop:	30-inches

SUMMARY OF SUBSURFACE CONDITIONS

This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered and is representative of interpretations made during drilling. Contrasting data derived from laboratory analysis may not be reflected in these representations.

DEPTH (ft)	GRAPHIC	USCS	SUMMARY OF SUBSURFACE CONDITIONS	DRIVE SAMPLE	BULK SAMPLE	SAMPLE TYPE	BLOWS/6"	MOISTURE (%)	DRY UNIT WT. (pcf)	RELATIVE COMPACTION (%)
		SM	SILTY SAND, fine to medium grained with trace clay, red-brown, slightly moist, dense			B				
						SS	23 50/5"	6	131	
5		SM	SILTY SAND, fine to coarse grained with trace coarse grained sand, red-brown, slightly moist, medium dense			B SS	21 32	8	124	
10						SS	16 26	5	126	
15						SS	18 20	8	117	
			End of boring at 16.5 feet. No groundwater or mottling encountered.							

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Geotechnical Investigation
Jurupa Road
Glen Avon Area, CA
Project No. J117-004

Figure No. A-28

LOG OF BORING B-28

Elevation:	810.0	Date(s) Drilled:	6/25/01
Drilling Method:	Rotary Auger	Logged by:	M. Sullivan
Drilling Rig:	CME 55	Hammer Type:	Auto-trip
Boring Diameter:	8-inches	Hammer Weight:	140 lb.
		Hammer Drop:	30-inches

SUMMARY OF SUBSURFACE CONDITIONS

This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered and is representative of interpretations made during drilling. Contrasting data derived from laboratory analysis may not be reflected in these representations.

DEPTH (ft)	USCS	DESCRIPTION	SAMPLE TYPE	BLOWS/6"	MOISTURE (%)	DRY UNIT WT. (pcf)	RELATIVE COMPACTION (%)
0	SM	SILTY SAND, fine to medium grained, red-brown, slightly moist, very dense	B				
			SS	50	11	107	
5			SS	50	8	119	
		gravels					
10	SM	SILTY SAND, fine to coarse grained, red-brown, slightly moist, very dense	SS	37 50/4"	8	122	
15	SM	SILTY SAND, fine to medium grained with coarse grained sand, red-brown, slightly moist, very dense	SS	25 50/5"	6	131	
20		SILTY SAND, fine to medium grained, red-brown, slightly moist, very dense	SPT	25 50	9		
25			SPT	23 50	10		
30			SPT	25 23	8		
35	SM	SILTY SAND, fine to medium grained with trace clay and trace coarse grained sand, brown, moist, dense	SPT	19 16	14		
40			SPT	16 27	13		
45			SPT	14 20	22		
	SM	SILTY SAND, fine to coarse grained with trace clay, red-brown, moist, dense					
50			SPT	22 28	16		
		End of boring at 51.5 feet. No groundwater or mottling encountered.					

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Geotechnical Investigation
Jurupa Road
Glen Avon Area, CA
Project No. J117-004

Figure No. A-29

LOG OF BORING B-29

Elevation:	812.0	Date(s) Drilled:	6/25/01
Drilling Method:	Rotary Auger	Logged by:	M. Sullivan
Drilling Rig:	CME 55	Hammer Type:	Auto-trip
Boring Diameter:	8-inches	Hammer Weight:	140 lb.
		Hammer Drop:	30-inches

SUMMARY OF SUBSURFACE CONDITIONS

This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered and is representative of interpretations made during drilling. Contrasting data derived from laboratory analysis may not be reflected in these representations.

DEPTH (ft)	GRAPHIC	USCS	DESCRIPTION	SAMPLE TYPE	BLOWS/6"	MOISTURE (%)	DRY UNIT WT. (pcf)	RELATIVE COMPACTION (%)
0		SM	SILTY SAND, fine to medium grained with trace clay, red-brown, slightly moist, dense	B				
				SS	24 50/5"	11	122	
5		SM	SILTY SAND, fine to coarse grained, red-brown, slightly moist, dense to very dense	B SS	27 50/4"	8	128	
10				SS	50	10		
			gravels					
15				SS	24 38	8	119	
			End of boring at 16.5 feet. No groundwater or mottling encountered.					

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Geotechnical Investigation
Jurupa Road
Glen Avon Area, CA
Project No. J117-004

Figure No. A-30

LOG OF BORING B-30

Elevation:	795.0	Date(s) Drilled:	6/25/01	Logged by:	M. Sullivan
Drilling Method:	Rotary Auger			Hammer Type:	Auto-trip
Drilling Rig:	CME 55			Hammer Weight:	140 lb.
Boring Diameter:	8-inches			Hammer Drop:	30-inches

SUMMARY OF SUBSURFACE CONDITIONS

This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered and is representative of interpretations made during drilling. Contrasting data derived from laboratory analysis may not be reflected in these representations.

DEPTH (ft)	GRAPHIC	USCS	SUMMARY OF SUBSURFACE CONDITIONS	DRIVE SAMPLE	BULK SAMPLE	SAMPLE TYPE	BLOWS/6"	MOISTURE (%)	DRY UNIT WT. (pcf)	RELATIVE COMPACTION (%)
		SM	SILTY SAND, fine to medium grained, red-brown, slightly moist, dense		■	B				
5				◸	■	SS	26 50	6	132	
		SM	SILTY SAND, fine to coarse grained, red-brown, slightly moist, dense to very dense							
10				◸		SS	20 50	7	114	
15				◸		SS	14 50	7	113	
			End of boring at 16.5 feet. No groundwater or mottling encountered.							

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Geotechnical Investigation
Jurupa Road
Glen Avon Area, CA
Project No. J117-004

Figure No. A-31

LOG OF TRENCH TR-01

Elevation: 865.0
Excavation Method: Backhoe
Equipment:

Logged by: DRL
Date(s): 1/3/01

DEPTH (ft)	GRAPHIC	USCS	SUMMARY OF SUBSURFACE CONDITIONS This summary applies only at the location of the trench and at the time of digging. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	REMARKS
1		SM	SILTY SAND, fine to coarse grained, brown, dry, loose.	
2 3		SM	SILTY SAND, fine to coarse grained with trace clay, reddish-brown, slightly moist, medium dense to dense, well cemented.	
			End of Trench. No groundwater, mottling or refusal encountered.	

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Geotechnical Investigation
Jurupa Road
Glen Avon Area, CA
Project No. J117-004

Figure No. A-32

LOG OF TRENCH TR-02

Elevation: 880.0 | Logged by: DRL
Excavation Method: Backhoe | Date(s) : 1/3/01
Equipment:

DEPTH (ft)	GRAPHIC	USCS	SUMMARY OF SUBSURFACE CONDITIONS This summary applies only at the location of the trench and at the time of digging. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	REMARKS
		SM	SILTY SAND, fine to medium grained with trace clay, brown, dry, loose.	
1		SM	SILTY SAND, fine to medium grained with clay, reddish-brown, slightly moist, medium dense to dense, well cemented.	
2				
3				
			End of Trench. No groundwater, mottling or refusal encountered.	

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Geotechnical Investigation
Jurupa Road
Glen Avon Area, CA
Project No. J117-004

Figure No. A-33

LOG OF TRENCH TR-03

Elevation: 825.0
Excavation Method: Backhoe
Equipment:

Logged by: DRL
Date(s) : 1/3/01

DEPTH (ft)	GRAPHIC	USCS	SUMMARY OF SUBSURFACE CONDITIONS This summary applies only at the location of the trench and at the time of digging. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	REMARKS
		SM	SILTY SAND, fine to medium grained with trace clay, brown, dry, loose.	
1		SM	SILTY SAND, fine to medium grained with clay, reddish-brown, slightly moist, medium dense to dense, well cemented.	
2				
		SC	CLAYEY SAND, fine to medium grained, reddish-brown, slightly moist, medium dense to dense, well cemented.	
3				
			End of Trench. No groundwater, mottling or refusal encountered.	

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Geotechnical Investigation
Jurupa Road
Glen Avon Area, CA
Project No. J117-004

Figure No. A-34

LOG OF TRENCH TR-04

Elevation: 880.0
Excavation Method: Backhoe
Equipment:

Logged by: DRL
Date(s) : 1/3/01

DEPTH (ft)	GRAPHIC	USCS	SUMMARY OF SUBSURFACE CONDITIONS This summary applies only at the location of the trench and at the time of digging. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	REMARKS
		SM	SILTY SAND, fine to medium grained with trace clay, brown, dry, loose.	
1		SM	SILTY SAND, fine to medium grained with clay, reddish-brown, slightly moist, medium dense to dense, well cemented.	
2				
3			End of Trench. No groundwater, mottling or refusal encountered.	

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Geotechnical Investigation
Jurupa Road
Glen Avon Area, CA
Project No. J117-004

Figure No. A-35

LOG OF TRENCH TR-05

Elevation: 870.0
Excavation Method: Backhoe
Equipment:

Logged by: DRL
Date(s) : 1/3/01

SUMMARY OF SUBSURFACE CONDITIONS

This summary applies only at the location of the trench and at the time of digging. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.

DEPTH (ft)	GRAPHIC	USCS	SUMMARY OF SUBSURFACE CONDITIONS	REMARKS
0–1		SM	SILTY SAND, fine to medium grained with trace clay, brown, dry, loose.	
1–3.5		SM	SILTY SAND, fine to medium grained with clay, reddish-brown, slightly moist, loose to medium dense.	
			End of Trench. No groundwater, mottling or refusal encountered.	

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Geotechnical Investigation
Jurupa Road
Glen Avon Area, CA
Project No. J117-004

Figure No. A-36

LOG OF TRENCH TR-06

Elevation: 875.0
Excavation Method: Backhoe
Equipment:

Logged by: DRL
Date(s) : 1/3/01

DEPTH (ft)	GRAPHIC	USCS	SUMMARY OF SUBSURFACE CONDITIONS This summary applies only at the location of the trench and at the time of digging. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	REMARKS
		SM	SILTY SAND, fine to medium grained with trace clay, brown, dry, loose.	
1		SM	SILTY SAND, fine to medium grained with clay, reddish-brown, slightly moist, medium dense, moderately cemented, friable.	
2				
3				
			End of Trench. No groundwater, mottling or refusal encountered.	

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Geotechnical Investigation
Jurupa Road
Glen Avon Area, CA
Project No. J117-004

Figure No. A-37

LOG OF TRENCH TR-07

Elevation: 865.0
Excavation Method: Backhoe
Equipment:

Logged by: DRL
Date(s) : 1/3/01

DEPTH (ft)	GRAPHIC	USCS	SUMMARY OF SUBSURFACE CONDITIONS This summary applies only at the location of the trench and at the time of digging. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	REMARKS
1		SM	SILTY SAND, fine to medium grained with trace clay, brown, dry, loose.	
2 3		SM	SILTY SAND, fine to medium grained with clay, reddish-brown, slightly moist, loose to medium dense, moderately cemented, friable.	
			End of Trench. No groundwater, mottling or refusal encountered.	

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Geotechnical Investigation
Jurupa Road
Glen Avon Area, CA
Project No. J117-004

Figure No. A-38

LOG OF TRENCH TR-08

Elevation: 820.0
Excavation Method: Backhoe
Equipment:

Logged by: DRL
Date(s) : 1/3/01

DEPTH (ft)	GRAPHIC	USCS	SUMMARY OF SUBSURFACE CONDITIONS This summary applies only at the location of the trench and at the time of digging. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	REMARKS
		SM	SILTY SAND, fine to medium grained with trace clay, brown, dry, loose.	
1		SM	SILTY SAND, fine to medium grained with clay, reddish-brown, slightly moist, dense, well cemented.	
2				
3			End of Trench. No groundwater, mottling or refusal encountered.	

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Geotechnical Investigation
Jurupa Road
Glen Avon Area, CA
Project No. J117-004

Figure No. A-39

LOG OF TRENCH TR-09

Elevation: 820.0
Excavation Method: Backhoe
Equipment:

Logged by: DRL
Date(s) : 1/3/01

SUMMARY OF SUBSURFACE CONDITIONS

This summary applies only at the location of the trench and at the time of digging. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.

DEPTH (ft)	GRAPHIC	USCS	DESCRIPTION	REMARKS
0 – 1		SM	SILTY SAND, fine to medium grained, brown, dry, loose.	
1 – 3		SM	SILTY SAND, fine to medium grained, reddish-brown, slightly moist, dense, well cemented.	
3			End of Trench. No groundwater, mottling or refusal encountered.	

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Geotechnical Investigation
Jurupa Road
Glen Avon Area, CA
Project No. J117-004

Figure No. **A-40**

LOG OF TRENCH TR-10

Elevation: 800.0
Excavation Method: Backhoe
Equipment:

Logged by: DRL
Date(s) : 1/3/01

DEPTH (ft)	GRAPHIC	USCS	SUMMARY OF SUBSURFACE CONDITIONS This summary applies only at the location of the trench and at the time of digging. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	REMARKS
		SM	SILTY SAND, fine to medium grained, brown, dry, loose.	
1		SM	SILTY SAND, fine to medium grained, reddish-brown, slightly moist, dense, well cemented.	
2				
3			End of Trench. No groundwater, mottling or refusal encountered.	

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Geotechnical Investigation
Jurupa Road
Glen Avon Area, CA
Project No. J117-004

Figure No. A-41

LOG OF TRENCH TR-11

Elevation: 800.0
Excavation Method: Backhoe
Equipment:

Logged by: DRL
Date(s) : 1/3/01

DEPTH (ft)	GRAPHIC	USCS	SUMMARY OF SUBSURFACE CONDITIONS This summary applies only at the location of the trench and at the time of digging. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	REMARKS
1		SM	SILTY SAND, fine to medium grained, brown, dry, loose.	
2 3		SM	SILTY SAND, fine to medium grained with trace clay, reddish-brown, slightly moist, medium dense, slightly cemented, friable.	
			End of Trench. No groundwater, mottling or refusal encountered.	

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Geotechnical Investigation
Jurupa Road
Glen Avon Area, CA
Project No. J117-004

Figure No. A-42

LOG OF TRENCH TR-12

Elevation: 855.0

Excavation Method: Backhoe

Equipment:

Logged by: DRL

Date(s) : 1/3/01

DEPTH (ft)	GRAPHIC	USCS	SUMMARY OF SUBSURFACE CONDITIONS This summary applies only at the location of the trench and at the time of digging. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	REMARKS
		SM	SILTY SAND, fine to medium grained, reddish-brown, dry, loose.	
1 2 3		SM	SILTY SAND, fine to medium grained with trace clay, reddish-brown, slightly moist, medium dense, slightly cemented, friable.	
			End of Trench. No groundwater, mottling or refusal encountered.	

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Geotechnical Investigation
Jurupa Road
Glen Avon Area, CA
Project No. J117-004

Figure No. A-43

LOG OF TRENCH TR-13

Elevation: 870.0
Excavation Method: Backhoe
Equipment:

Logged by: DRL
Date(s) : 1/3/01

DEPTH (ft)	GRAPHIC	USCS	SUMMARY OF SUBSURFACE CONDITIONS This summary applies only at the location of the trench and at the time of digging. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	REMARKS
		SM	SILTY SAND, fine to medium grained with clay, brown, dry, loose.	
1		SM	SILTY SAND, fine to medium grained with clay, reddish-brown, slightly moist, medium dense to dense, well cemented.	
2				
3				
			End of Trench. No groundwater, mottling or refusal encountered.	

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Geotechnical Investigation
Jurupa Road
Glen Avon Area, CA
Project No. J117-004

Figure No. A-44

LOG OF TRENCH TR-14

Elevation: 795.0

Excavation Method: Backhoe

uipment:

Logged by: DRL

Date(s) : 1/3/01

DEPTH (ft)	GRAPHIC	USCS	SUMMARY OF SUBSURFACE CONDITIONS This summary applies only at the location of the trench and at the time of digging. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	REMARKS
		SM	SILTY SAND, fine to medium grained, brown, dry, loose.	
1		SM	SILTY SAND, fine to medium grained with trace clay, reddish-brown, slightly moist, medium dense to dense, moderately cemented.	
2				
3				
			End of Trench. No groundwater, mottling or refusal encountered.	

INLAND FOUNDATION ENGINEERING, INC.

Geotechnical Investigation
Jurupa Road
Glen Avon Area, CA
Project No. J117-004

Figure No. A-45

FENAGH
ENGINEERING AND TESTING

Fenagh Job 6484
Patriot High School, Jurupa Valley

APPENDIX G
SOIL CORROSIVITY STUDY

Soil Corrosivity Evaluation Report – Project X Corrosion Engineering, Inc.

Soil Corrosivity Evaluation Report for Patriot H.S. Stadium Improvements, Jurupa Valley

August 16, 2022

Prepared for:
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Project X Job #: S220812A
Client Job or PO #: 6484

Contents

1 Executive Summary 4
2 Corrosion Control Recommendations 5
2.1 Cement 5
2.2 Steel Reinforced Cement/ Cement Mortar Lined & Coated (CML&C) 5
2.3 Stainless Steel Pipe/Conduit/Fittings 6
2.4 Steel Post Tensioning Systems 6
2.5 Steel Piles 6
2.5.1 Expected Corrosion Rate of Steel and Zinc in disturbed soil 7
2.5.2 Expected Corrosion Rate of Steel and Zinc in Undisturbed soil 8
2.6 Steel Storage tanks 8
2.7 Steel Pipelines 8
2.8 Steel Fittings 10
2.9 Ductile Iron (DI) & Cast Iron Fittings 10
2.10 Ductile Iron & Cast Iron Pipe 11
2.11 Copper Materials 13
2.11.1 Copper Pipes 13
2.11.2 Brass Fittings 13
2.11.3 Bare Copper Grounding Wire 14
2.12 Aluminum Pipe/Conduit/Fittings 15
2.13 Carbon Fiber or Graphite Materials 15
2.14 Plastic and Vitrified Clay Pipe 15
3 CLOSURE 16
4 Soil analysis lab results 17
5 Corrosion Basics 20
5.1 Pourbaix Diagram – In regards to a material's environment 20
5.2 Galvanic Series – In regards to dissimilar metal connections 20
5.3 Corrosion Cell 23
5.4 Design Considerations to Avoid Corrosion 24
5.4.1 Testing Soil Factors (Resistivity, pH, REDOX, SO, CL, NO3, NH3) 24
5.4.2 Proper Drainage 25
5.4.3 Avoiding Crevices 25
5.4.4 Coatings and Cathodic Protection 26

5.4.5 Good Electrical Continuity 28
5.4.6 Bad Electrical Continuity 29
5.4.7 Corrosion Test Stations 29
5.4.8 Excess Flux in Plumbing 30
5.4.9 Landscapers and Irrigation Sprinkler Systems 30
5.4.10 Roof Drainage splash zones 30
5.4.11 Stray Current Sources 31

1 Executive Summary

A corrosion evaluation of the soils at Patriot H.S. Stadium Improvements, Jurupa Valley was performed to provide corrosion control recommendations for general construction materials. The site is located at 4355 Camino Real, Jurupa Valley, CA 92509. Three (3) samples were tested to a depth of 5.0 ft. Site ground water and topography information was provided by Fenagh Engineering & Testing. Groundwater depth was determined to be 45 feet below finished grade.

Every material has its weakness. Aluminum alloys, galvanized/zinc coatings, and copper alloys do not survive well in very alkaline or very acidic pH environments. Copper and brasses do not survive well in high nitrate or ammonia environments. Steels and irons do not survive well in low soil resistivity and high chloride environments. High chloride environments can even overcome and attack steel encased in normally protective concrete. Concrete does not survive well in high sulfate environments. And nothing survives well in high sulfide and low redox potential environments with corrosive bacteria. This is why Project X tests for these 8 factors to determine a soil's corrosivity towards various construction materials. **Depending solely on soil resistivity or Caltrans corrosion guidelines (which concentrate on concrete/steel highways), will over-simplify descriptions as corrosive or non-corrosive. This approach will not detect these other factors attacking other metals because it is possible to have bad levels of corrosive ions and still have greater than 1,100 ohm-cm soil resistivity. We have observed this fact on thousands of soil samples tested in our laboratory.**

It should not be forgotten that import soil should also be tested for all factors to avoid making your site more corrosive than it was to begin with.

The recommendations outlined herein are not a substitute for any design documents previously prepared for the purpose of construction and apply only to the depth of samples collected.

Soil samples were tested for minimum resistivity, pH, chlorides, sulfates, ammonia, nitrates, sulfides and redox.

As-Received soil resistivities ranged between 6,365 ohm-cm and 9,380.0 ohm-cm. This data would be similar to a Wenner 4 pin test in the field and used in the design of a cathodic protection or grounding bed system. This resistivity can change seasonally depending on the weather and moisture in the ground. This reading alone can be misleading because condensation or minor water leaks will occur underground along pipe surfaces creating a saturated soil environment in the trench on infrastructure surfaces. This is why minimum or saturated soil resistivity measurements are more important than as-received resistivities.

Saturated soil resistivities ranged between 1,608 ohm-cm to 4,288 ohm-cm. The worst of these values is considered to be corrosive to general metals.

PH levels ranged between 6.8 to 9.1 pH. The average pH of these samples is alkaline and can cause accelerated corrosion of copper and aluminum alloys.

Chlorides ranged between 60 mg/kg to 162 mg/kg. Chloride levels in these samples are low and may cause insignificant corrosion of metals.

Sulfates ranged between 110 mg/kg to 286 mg/kg. Sulfate levels in these samples are negligible for corrosion of cement. Any type of cement can be used that does not contain encased metal.

Ammonia ranged between 25.1 mg/kg to 40.1 mg/kg. Nitrates ranged between 1.8 mg/kg to 49.0 mg/kg. Concentrations of these elements were high enough to cause accelerated corrosion of copper and copper alloys such as brass.

Sulfides presence was determined to be negative. REDOX ranged between + 128 mV to + 182 mV. The probability of corrosive bacteria was determined to be low due to the sulfide and positive REDOX levels determined in these samples.

2 Corrosion Control Recommendations

The following recommendations are based upon the results of soil testing.

2.1 Cement

The highest reading for sulfates was 286 mg/kg or 0.0286 percent by weight.

Per ACI 318-14, Table 19.3.1.1, sulfate levels in these samples categorized as S0 and are negligible for corrosion of metals and cement. Per ACI 318-14 Table 19.3.2.1 any type of cement not containing steel or other metal can be used.

2.2 Steel Reinforced Cement/ Cement Mortar Lined & Coated (CML&C)

Chlorides in soil can overcome the corrosion inhibiting property of cement for steel, as it can also break through passivated surfaces of aluminum and stainless steels.[1,2] The highest concentration of chlorides was 162 mg/kg.

Chloride levels in these samples are not significantly corrosive to metals not in tension. Standard cement cover may be used in these soils.

Though soils at some locations are significantly corrosive to various metals, per ACI 318-14 Chapter 19 Table 19.3.1.1, all slabs on this site exposure categories and class for **Corrosion Protection of Reinforcement (C) would be considered C1** as Concrete exposed to moisture [mud/rain] (slab sides and bottom) but not to an external source of chlorides. Though there are chlorides in the soil, ACI 318's definition of "external source of chlorides" consists of deicing chemicals, salt, brackish water, seawater, or spray from these sources. The chloride levels in seawater are typically over 19,000 mg/L or 19,000 ppm.

When concrete is tested for water-soluble chloride ion content, the tests should be made at an age of 28 to 42 days. The limits in Per ACI 318-14 Table 5.3.2.1 are to be applied to chlorides contributed from the concrete ingredients, not those from the environment surrounding the concrete.[3]

[1] Design Manual 303: Cement Cylinder Pipe. Ameron. p.65
[2] *Chapter 19, Table 1904.2.2(1), 2012 International Building Code*
[3] ACI 381-14., BUILDING CODE REQUIREMENTS FOR STRUCTURAL CONCRETE (ACI 318-14) AND COMMENTARY (ACI 318R-14)

2.3 Stainless Steel Pipe/Conduit/Fittings

Stainless steels derive their corrosion resistance from their chromium content and oxide layer which needs oxygen to regenerate if damaged. Thus stainless steel is not good for deep soil applications where oxygen levels are extremely low. Stainless steels should not be installed deeper than a plant root zone. Stainless steels typically have the same nobility as copper on the galvanic series and can be connected to copper. If stainless steel must be used, it must be backfilled with soil having greater than 10,000 ohm-cm resistivity and excellent drainage. 304 Stainless steel will also corrode if in contact with carbon materials such as activated carbon. Stainless steel welds should be pickled.

The soil at this site has low probability for anaerobic corrosive bacteria and low chloride levels. Per Nickel Institute guidelines, 304 or 316 Stainless steels can be used in these soils.

2.4 Steel Post Tensioning Systems

The proper sealing of stressing holes is of utmost importance in PT Systems. Cut off excess strand 1/2" to 3/4" back in the hole. Coat or paint exposed anchorage, grippers, and stub of strands with "Rust-o-leum" or equal. After tendons have been coated, the cement contractor shall dry pack blockouts within ten (10) days. A non-shrink, non-metallic, non-porous moisture-insensitive grout (Master EMACO S 488 or equivalent), or epoxy grout shall be used for this purpose. If an encapsulated post-tension system is used, regular non-shrink grout can be used.

Due to the low chloride concentrations measured on samples obtained from this site, post-tensioned slabs should be protected in accordance with soil considered normal (non-corrosive).[4,5] Addition of grease caps to the cut strand at live end anchors can deter construction defect accusations but are not needed.

2.5 Steel Piles

Steel piles are most susceptible to corrosion in disturbed soil where oxygen is available. Further, a dissimilar environment corrosion cell would exist between the steel embedded in cement, such as pile caps and the steel in the soil. In the cell, the steel in the soil is the anode (corroding metal), and the steel in cement is the cathode (protected metal). This cell can be minimized by coating the part of the steel piles that will be embedded in cement to prevent contact with cement and reinforcing steel.

Piles driven into soils without disturbing soils will avoid oxygen introduction and low corrosion rates unless there is a probability for corrosive anaerobic bacteria. Galvanized steel's zinc coating can provide significant protection for driven piles. In corrosive soils in which normal zinc coatings are not enough, the life of piles can be extended by increasing zinc coating thickness, using sacrificial metal, or providing a combination of epoxy coatings and cathodic protection. Corrosion has been observed to be extremely localized even at and below underground water tables. Pit depths of this magnitude do not have an appreciable effect on the strength or useful life of piling structures because the reduction in pile cross section is not

[4] *Standard Requirements for Design and Analysis of Shallow Post-Tensioned Concrete Foundations on Expansive Soils, PTI DC10.5-12,Table 4.1, pg 16*
[5] *Specification for Unbonded Single Strand Tendons. Post-tensioning Institute (PTI), Phoenix, AZ, 2000.*

significant.[6] Pitting is of more importance to pipes transporting liquids or gases which should not be leaked into the ground.

The following recommendations are recommended to achieve desired life. We defer to structural engineers to use our estimated corrosion rates and to choose from the corrosion control options listed below.

1) Sacrificial metal by use of thicker piles per non-disturbed soil corrosion rates, or
2) Galvanized steel piles per non-disturbed soil corrosion rates, or
3) Combination of galvanized and sacrificial metal per non-disturbed soil corrosion rates, or
4) For no loss of metal, coat entire pile with abrasion resistant epoxy coating such as 3M Scotchkote 323, or PowercreteDD, or equivalent, or
5) Use high yield steel which will corrode at the same rate as mild steel but have greater yield strength and thus be able to suffer more material loss than mild steel.

2.5.1 Expected Corrosion Rate of Steel and Zinc in disturbed soil

In general, the corrosion rate of metals in soil depends on the electrical resistivity, the elemental composition, and the oxygen content of the soil. Soils can vary greatly from one acre to the next, especially at earthquake faults. The better a soil is for farming; the easier it will be for corrosion to take place. Expansive soils will also be considered disturbed simply because of their nature from dry to wet seasons.

In Melvin Romanoff's NBS Circular 579, the corrosion rates of carbon steels and various metals was studied over long term periods. Various metals were placed in various soil types to gather corrosion rate data of all metals in all soil types. Samples were collected and material loss measured over the course of 20 years in some sites. The following corrosion rates were estimated by comparing the worst results of soils tested with similar soils in Romanoff's studies and Highway Research Board's publications.[7] The corrosion rate of zinc in disturbed soils is determined per Romanoff studies and King Nomograph.[8]

Expected Corrosion Rate for Steel = 1.33 mils/year for one sided attack

Expected Corrosion Rate for Zinc = 0.17 mils/year for one sided attack.

Note: 1 mil = 0.001 inch

In undisturbed soils, a corrosion rate of 1.00 mil/year for steel is expected with little change in the corrosion rate of zinc due to it's low nobility in the galvanic series.

Per CTM 643: Years to perforation of corrugated galvanized steel culverts

- 39.0 Years to Perforation for a 18 gage metal culvert
- 50.7 Years to Perforation for a 16 gage metal culvert
- 62.4 Years to Perforation for a 14 gage metal culvert

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[6] Melvin Romanoff, Corrosion of Steel Pilings in Soils, National Bureau of Standards Monograph 58, pg 20.

[7] Field test for Estimating Service Life of Corrugated Metal Culverts, J.L. Beaton, Proc. Highway Research Board, Vol 41, P. 255, 1962

[8] King, R.A. 1977, Corrosion Nomograph, TRRC Supplementary Report, British Corrosion Journal

- 85.8 Years to Perforation for a 12 gage metal culvert
- 109.2 Years to Perforation for a 10 gage metal culvert
- 132.6 Years to Perforation for a 8 gage metal culvert

2.5.2 *Expected Corrosion Rate of Steel and Zinc in Undisturbed soil*

Expected Corrosion Rate for Steel = 1.00 mils/year for one sided attack

Expected Corrosion Rate for Zinc = 0.17 mils/year for one sided attack.

Note: 1 mil = 0.001 inch

2.6 Steel Storage tanks

Underground fuel tanks must be constructed and protected in accordance with California Underground Storage Tank Regulations, CCR, Title 23, Division 3, Chapter 16. Metals should be protected with cathodic protection or isolated from backfill material with an epoxy coating.

2.7 Steel Pipelines

Though a site may not be corrosive in nature at the time of construction, **installation of corrosion test stations and electrical continuity joint bonding should be performed during construction** so that future corrosion inspections can be performed. If steel pipes with gasket joints or other possibly non-conductive type joints are installed, their joints should be bonded across by welding or pin brazing a #8 AWG copper strand bond cable. Electrical continuity is necessary for corrosion inspections and for cathodic protection.

Corrosion test stations should be installed every 1,000 feet of pipeline.

Test stations shall have two #8 HMWPE copper strand wire test leads welded or pin brazed to the underground pipe, brought up into the test station hand hole and marked CTS. Wires should be brought into test station hand hole at finished grade with 12 inches of wire coiled within test station.

At isolation joints and pipe casings, 4 wire test stations shall be installed using #8 HMWPE copper strand wire test leads. Use different color wires to distinguish which wires are bonded to one side of isolation joint or to casing. Wires should be brought into test station hand hole at finished grade with 12 inches of wire coiled within test station.

Prevent dissimilar metal corrosion cells per NACE SP0286:

1) Electrically isolate dissimilar metal connections
2) Electrically isolate dissimilar coatings (Epoxy vs CML&C) segments connections
3) Electrically isolate river crossing segments
4) Electrically isolate freeway crossing segments
5) Electrically isolate old existing pipelines from new pipelines
6) Electrically isolate aboveground and underground pipe segments with flange isolation joint kits per NACE SP0286 to avoid galvanic corrosion cells. **These are especially important for fire risers.**

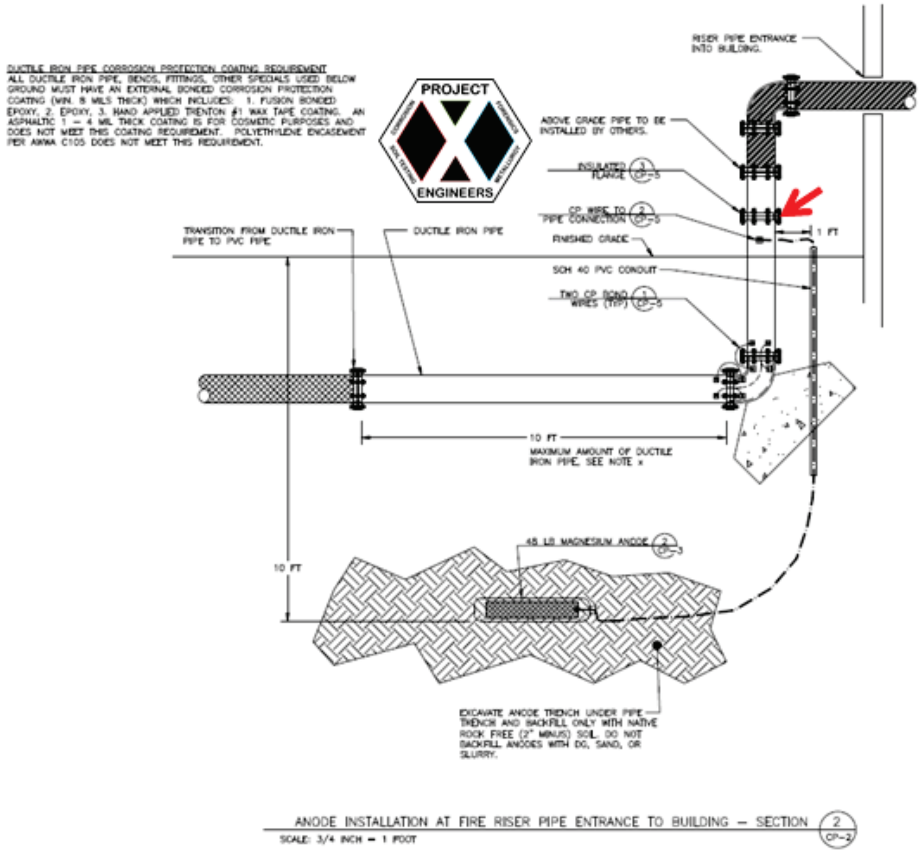

Figure 1- Fire Riser Detail: Install Isolation joint at red arrow

The corrosivity at this site is corrosive to steel. Any piping that must be jack-bored should use abrasion resistant epoxy coating such as 3M Scotchkote 323, or PowercreteDD, or equivalent. The corrosion control options for this site are as follows:

1) Apply impermeable dielectric coating such as minimum 10 mil thick polyethylene, and install cathodic protection system per NACE SP0169, or
2) Wax tape per AWWA C217, or
3) Coal tar enamel per AWWA C203, or
4) Fusion bonded epoxy per AWWA C213, or
5) For bare steel surfaces, such as welded pipe joints, apply 3 inch thick field coating of Type II cement or high pH slurry that will maintain pH higher than 12. Cement is both a corrosion inhibitor and a coating for ferrous metals. Cement naturally holds a pH of 12 or higher for many years if not exposed to high levels of carbon dioxide. (For CML&C pipes, CML&C factory applied 3/4 inch thick coating is equivalent and needs no extra thickness added.)

It is critical for the life of the pipe that the protective wrap contains no openings or holes. Prevent damage to the protective sleeve during backfilling of the pipe trench. Penetrations of any kind within these or other protective materials generally leads to accelerated corrosion failure due to the fact that the corrosion attack is concentrated at the location of these penetrations. Cathodic protection will protect these defects. The better the coating, the less expensive a cathodic protection system will be in anode material and power requirement if needed.

2.8 Steel Fittings

The corrosivity at this site is corrosive to steel. The corrosion control options for this site are as follows:

1) Apply impermeable dielectric coating such as minimum 10 mil thick polyethylene, and install cathodic protection system per NACE SP0169, or
2) Tape coating system per AWWA C214, or
3) Wax tape per AWWA C217, or
4) Coal tar enamel per AWWA C203, or
5) Fusion bonded epoxy per AWWA C213
6) Apply 3 inch coating of Type II cement or high pH slurry that will maintain pH higher than 12. Cement is both a corrosion inhibitor and a coating for ferrous metals. Cement naturally holds a pH of 12 or higher for many years if not exposed to high levels of carbon dioxide.

It is critical for the life of the metal that the protective wrap contains no openings or holes. Prevent damage to the protective sleeve during backfilling of the pipe trench. Penetrations of any kind within these or other protective materials generally leads to accelerated corrosion failure due to the fact that the corrosion attack is concentrated at the location of these penetrations. Cathodic protection will protect these defects. The better the coating, the less expensive a cathodic protection system will be in anode material and power requirement if needed.

2.9 Ductile Iron (DI) & Cast Iron Fittings

AWWA C105 developed a 10 point system to classify sites as aggressive or non-aggressive to ductile iron materials. The 10-point system does not, and was never intended to, quantify the corrosivity of a soil. It is a tool used to distinguish nonaggressive from aggressive soils relative to iron pipe. Soils <10 points are considered nonaggressive to iron pipe, whereas soils ≥ 10 points are considered aggressive. A 15 and a 20 point soil are both considered aggressive to iron pipe, however, because of the nature of the soil parameters measured, the 20 point soil may not necessarily be more aggressive than the 15 point soil. The criterion is based upon soil resistivities, soil drainage, pH, sulfide presence, and reduction-oxidation (REDOX) potential. The soil samples tested for this site resulted in a score of 5 out of 25.5. A score greater or equal to 10 points classifies soils as aggressive to iron materials. The black coating on iron pipes is purely for aesthetic purposes and should not be relied upon for corrosion protection.[9]

The corrosivity at this site is corrosive to iron. The corrosion control options for this site are as follows:

[9] https://www.dipra.org/ductile-iron-pipe-resources/frequently-asked-questions/corrosion-control

1) Apply impermeable dielectric coating such as minimum 10 mil thick polyethylene, and install cathodic protection system per NACE SP0169, or
2) Wax tape per AWWA C217, or
3) Coal tar enamel per AWWA C203, or
4) Fusion bonded epoxy per AWWA C213
5) Apply standard concrete cover of Type II cement or high pH slurry that will maintain pH higher than 12. Cement is both a corrosion inhibitor and a coating for ferrous metals. Cement naturally holds a pH of 12 or higher for many years if not exposed to high levels of carbon dioxide.

It is critical for the life of the metal that the protective wrap contains no openings or holes. Prevent damage to the protective sleeve during backfilling of the pipe trench. Penetrations of any kind within these or other protective materials generally leads to accelerated corrosion failure due to the fact that the corrosion attack is concentrated at the location of these penetrations. Cathodic protection will protect these defects. The better the coating, the less expensive a cathodic protection system will be in anode material and power requirement if needed.

2.10 Ductile Iron & Cast Iron Pipe

AWWA C105 developed a 10 point system to classify sites as aggressive or non-aggressive to ductile iron materials. The 10-point system does not, and was never intended to, quantify the corrosivity of a soil. It is a tool used to distinguish nonaggressive from aggressive soils relative to iron pipe. Soils <10 points are considered nonaggressive to iron pipe, whereas soils ≥ 10 points are considered aggressive. A 15 and a 20 point soil are both considered aggressive to iron pipe, however, because of the nature of the soil parameters measured, the 20 point soil may not necessarily be more aggressive than the 15 point soil. The criterion is based upon soil resistivities, soil drainage, pH, sulfide presence, and reduction-oxidation (REDOX) potential. The soil samples tested for this site resulted in a score of 5 out of 25.5. A score greater or equal to 10 points classifies soils as aggressive to iron materials. The black coating on iron pipes is purely for aesthetic purposes and should not be relied upon for corrosion protection.[10]

Though a site may not be corrosive in nature at the time of construction, <u>**installation of corrosion test stations and electrical continuity joint bonding should be performed during construction**</u> so that future corrosion inspections can be performed. If steel pipes with gasket joints or other possibly non-conductive type joints are installed, their joints should be bonded across by welding or pin brazing a #8 AWG copper strand bond cable. Electrical continuity is necessary for corrosion inspections and for cathodic protection. **If using thermite, perform one test bond using a half-charge then pressure test to confirm excess heat and pinholes were not created.**

Pea gravel is used by plumbers to lay pipes and establish slopes. If the gravel has more than 200 ppm chlorides or is not tested, a 25 mil plastic should be placed between the gravel and pipe to avoid corrosion.

[10] https://www.dipra.org/ductile-iron-pipe-resources/frequently-asked-questions/corrosion-control

Corrosion test stations should be installed every 1,000 feet of pipeline.

Test stations shall have two #8 HMWPE copper strand wire test leads welded or pin brazed to the underground pipe, brought up into the test station hand hole and marked CTS. Wires should be brought into test station hand hole at finished grade with 12 inches of wire coiled within test station.

At isolation joints and pipe casings, 4 wire test stations shall be installed using #8 HMWPE copper strand wire test leads. Use different color wires to distinguish which wires are bonded to one side of isolation joint or to casing. Wires should be brought into test station hand hole at finished grade with 12 inches of wire coiled within test station.

Prevent dissimilar metal corrosion cells per NACE SP0286:

1) Electrically isolate dissimilar metal connections
2) Electrically isolate dissimilar coatings (Epoxy vs CML&C) segments connections
3) Electrically isolate river crossing segments
4) Electrically isolate freeway crossing segments
5) Electrically isolate old existing pipelines from new pipelines
6) Electrically isolate aboveground and underground pipe segments with flange isolation joint kits per NACE SP0286**. These are especially important for fire risers.**

The corrosivity at this site is corrosive to iron. Any piping that must be jack-bored should use abrasion resistant epoxy coating such as 3M Scotchkote 323, or PowercreteDD, or equivalent. The corrosion control options for this site are as follows:

1) Apply impermeable dielectric coating such as minimum 10 mil thick polyethylene, and install cathodic protection system per NACE SP0169, or
2) Wax tape per AWWA C217, or
3) Coal tar enamel per AWWA C203, or
4) Fusion bonded epoxy per AWWA C213
5) Apply 3 inch coating of Type II cement or high pH slurry that will maintain pH higher than 12. Cement is both a corrosion inhibitor and a coating for ferrous metals. Cement naturally holds a pH of 12 or higher for many years if not exposed to high levels of carbon dioxide.

It is critical for the life of the metal that the protective wrap contains no openings or holes. Prevent damage to the protective sleeve during backfilling of the pipe trench. Penetrations of any kind within these or other protective materials generally leads to accelerated corrosion failure due to the fact that the corrosion attack is concentrated at the location of these penetrations. Cathodic protection will protect these defects. The better the coating, the less expensive a cathodic protection system will be in anode material and power requirement if needed.

2.11 Copper Materials

Copper is an amphoteric material which is susceptible to corrosion at very high and very low pH. It is one of the most noble metals used in construction thus typically making it a cathode when connected to dissimilar metals. Copper's nobility can change with temperature, similar to the phenomenon in zinc. When zinc is at room temperature, it is less noble than steel and can provide cathodic protection to steel. But when zinc is at a temperature above 140F such as in a water heater, it becomes more noble than the steel and the steel becomes the sacrificial anode. This is why zinc is not used in steel water heaters or boilers. Cold copper has one native potential, but when heated it develops a more electronegative electro-potential aka open circuit potential. Thus hot and cold copper pipes should be electrically isolated from each other to avoid creation of a thermo-galvanic corrosion cell.

2.11.1 Copper Pipes

The lowest pH for this area was measured to be 6.8. Copper is greatly affected by pH, ammonia and nitrate concentrations[11]. The highest nitrate concentration was 49.0 mg/kg and the highest ammonia concentration was 40.1 mg/kg at this site.

These soils were determined to be corrosive to copper and copper alloys such as brass.

Aboveground, underground, cold water and hot water pipes should be electrically isolated from each other by use of dielectric unions and plastic in-wall pipe supports per NACE SP0286. The following are corrosion control options for underground copper water pipes.

1) Run copper pipes within PVC pipes to prevent soil contact, or
2) Cover piping with a 20 mil epoxy coating, or 8-mil polyethylene sleeve, or encase in double 4-mil thick polyethylene sleeves free of scratches and defects then backfill with clean sand with 2 inch minimum cover above and below tubing. Backfill should have a pH between 6 and 8 with electrical resistivity greater than 2,000 ohm-cm
3) Cover copper pipes with minimum 8 mil polyethylene sleeve or incase in double 4-mil thick polyethylene sleeves over a suitable primer and apply cathodic protection per NACE SP0169

It is critical for the life of the metal that the protective wrap contains no openings or holes. Prevent damage to the protective sleeve during backfilling of the pipe trench. Penetrations of any kind within these or other protective materials generally leads to accelerated corrosion failure due to the fact that the corrosion attack is concentrated at the location of these penetrations. Cathodic protection will protect these defects. The better the coating, the less expensive a cathodic protection system will be in anode material and power requirement if needed.

2.11.2 Brass Fittings

Brass fittings should be electrically isolated from dissimilar metals by use of dielectric unions or isolation joint kits per NACE SP0286.

These soils were determined to be corrosive to copper and copper alloys such as brass.

[11] Corrosion Data Handbook, Table 6, Corrosion Resistance of copper alloys to various environments, 1995

The following are corrosion control options for underground brass.

1) Prevent soil contact by use of impermeable coating system such as wax tape, or
2) Prevent soil contact by use of a 20 mil epoxy coating free of scratches and defects and backfill with clean sand with 4 inch minimum cover above and below brass. Backfill should have a pH between 6 and 8 with electrical resistivity greater than 2,000 ohm-cm, or
3) Cover brass with minimum 10 mil polyethylene sleeve over a suitable primer and apply cathodic protection per NACE SP0169

It is critical for the life of the metal that the protective wrap contains no openings or holes. Prevent damage to the protective sleeve during backfilling of the pipe trench. Penetrations of any kind within these or other protective materials generally leads to accelerated corrosion failure due to the fact that the corrosion attack is concentrated at the location of these penetrations. Cathodic protection will protect these defects. The better the coating, the less expensive a cathodic protection system will be in anode material and power requirement if needed.

2.11.3 Bare Copper Grounding Wire

It is assumed that corrosion will occur at all sides of the bare wire, thus the corrosion rate is calculated as a two sided attack determining the time it takes for the corrosion from two sides to meet at the center of the wire. The estimated life of bare copper wire for this site is the following:[12]

Size (AWG)	Diameter (mils)	Est. Time to penetration (Yrs)
14	64.1	5.5
13	72	6.2
12	80.8	7.0
11	90.7	7.8
10	101.9	8.8
9	114.4	9.9
8	128.5	11.1
7	144.3	12.4
6	162	14.0
5	181.9	15.7
4	204.3	17.6
3	229.4	19.8
2	257.6	22.2
1	289.3	24.9

If the bare copper wire is being used as a grounding wire connected to less noble metals such as galvanized steel or carbon steel, the less noble metals will provide additional cathodic protection to the copper reducing the corrosion rate of the copper.

[12] Soil-Corrosion studies 1946 and 1948: Copper Alloys, Lead, and Zinc, Melvin Romanoff, National Bureau of Standards, Research Paper RP2077, 1950

It is recommended that a corrosion inhibiting and water-repelling coating be applied to aboveground and belowground copper-to-dissimilar metal connections to reduce risk of dissimilar corrosion. This can be wax tape, or other epoxy coating.

Tinned copper wiring or laying copper wire in conductive concrete can protect against chemical attack in soils with high nitrates, ammonia, sulfide and severely low soil electrical resistivity.

2.12 Aluminum Pipe/Conduit/Fittings

Aluminum is an amphoteric material prone to pitting corrosion in environments that are very acidic or very alkaline or high in chlorides.

Conditions at this site are unsafe for aluminum. Soils at this site were determined to be too alkaline for aluminum. Soil contact with aluminum alloys should be avoided at this site. This can be achieved with:

1) Impermeable minimum 20 mil polyethylene coatings, or
2) Epoxy coatings with minimum 20 mil thickness free of scratches and defects, or
3) Wax tape

Aluminum derives its corrosion resistance from its oxide layer which needs oxygen to regenerate if damaged, similar to stainless steels. Thus aluminum is not good for deep soil applications. Since aluminum corrodes at very alkaline environments, it cannot be encased or placed against cement or mortar such as brick wall mortar up against an aluminum window frame.

Aluminum is also very low on the galvanic series scale making it most likely to become a sacrificial anode when in contact with dissimilar metals in moist environments. Avoid electrical continuity with dissimilar metals by use of insulators, dielectric unions, or isolation joints per NACE SP0286. Pooling of water at post bottoms or surfaces should be avoided by integrating good drainage.

2.13 Carbon Fiber or Graphite Materials

Carbon fiber or other graphite materials are extremely noble on the galvanic series and should always be electrically isolated from dissimilar metals. They can conduct electricity and will create corrosion cells if placed in contact within a moist environment with any metal.

2.14 Plastic and Vitrified Clay Pipe

No special precautions are required for plastic and vitrified clay piping from a corrosion viewpoint.

Protect all metallic fittings and pipe restraining joints with wax tape per AWWA C217, cement if previously recommended, or epoxy.

3 CLOSURE

In addition to soils chemistry and resistivity, another contributing influence to the corrosion of buried metallic structures is stray electrical currents. These electrical currents flowing through the earth originate from buried electrical systems, grounding of electrical systems in residences, commercial buildings, and from high voltage overhead power grids. Therefore, it is imperative that the application of protective wraps and/or coatings and electrical isolation joints be properly applied and inspected.

It is the responsibility of the builder and/or contractor to closely monitor the installation of such materials requiring protection in order to assure that the protective wraps or coatings are not damaged.

The recommendations outlined herein are in conformance with current accepted standards of practice that meet or exceed the provisions of the Uniform Building Code (UBC), the International Building Code (IBC), California Building Code (CBC), the American Cement Institute (ACI), Nickel Institute, National Association of Corrosion Engineers (NACE International), Post-Tensioning Institute Guide Specifications and State of California Department of Transportation, Standard Specifications, American Water Works Association (AWWA) and the Ductile Iron Pipe Research Association (DIPRA).

Our services have been performed with the usual thoroughness and competence of the engineering profession. No other warranty or representation, either expressed or implied, is included or intended.

Please call if you have any questions.

Respectfully Submitted,

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4 SOIL ANALYSIS LAB RESULTS

Client: Fenagh Engineering & Testing
Job Name: Patriot H.S. Stadium Improvements, Jurupa Valley
Client Job Number: 6484
Project X Job Number: S220812A
August 16, 2022

	Method	ASTM D4327		ASTM D4327		ASTM G187		ASTM G51	ASTM G200	SM 4500-D	ASTM D4327	ASTM D6919	ASTM D6919	ASTM D6919	ASTM D6919	ASTM D6919	ASTM D6919	ASTM D4327	ASTM D4327
Bore# / Description	Depth	Sulfates SO_4^{2-}		Chlorides Cl^-		Resistivity		pH	Redox	Sulfide S^{2-}	Nitrate NO_3^-	Ammonium NH_4^+	Lithium Li^+	Sodium Na^+	Potassium K^+	Magnesium Mg^{2+}	Calcium Ca^{2+}	Fluoride F_2^-	Phosphate PO_4^{3-}
						As Rec'd	Minimum												
	(ft)	(mg/kg)	(wt%)	(mg/kg)	(wt%)	(Ohm-cm)	(Ohm-cm)		(mV)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)
B1 - Bulk Sample	1-5	286.2	0.0286	161.7	0.0162	6,365	1,608	6.8	182	0.36	49.0	40.1	ND	153.5	11.1	22.7	208.4	3.7	0.1
B5 - Bulk Sample	1-5	196.9	0.0197	73.1	0.0073	9,380	4,288	9.1	130	1.32	1.8	27.7	0.01	116.9	7.2	32.9	164.5	3.4	0.3
B10 - Bulk Sample	1-5	110.1	0.0110	59.9	0.0060	8,040	3,015	8.8	128	1.89	22.3	25.1	0.01	91.6	5.1	22.5	100.6	3.5	6.1

Unk = Unknown
NT = Not Tested
ND = 0 = Not Detected
mg/kg = milligrams per kilogram (parts per million) of dry soil weight
Chemical Analysis performed on 1:3 Soil-To-Water extract
Anions and Cations tested via Ion Chromatograph except Sulfide.

29990 Technology Dr, Suite 13, Murrieta, CA 92563 Tel: 213-928-7213 Fax: 951-226-1720
www.projectxcorrosion.com

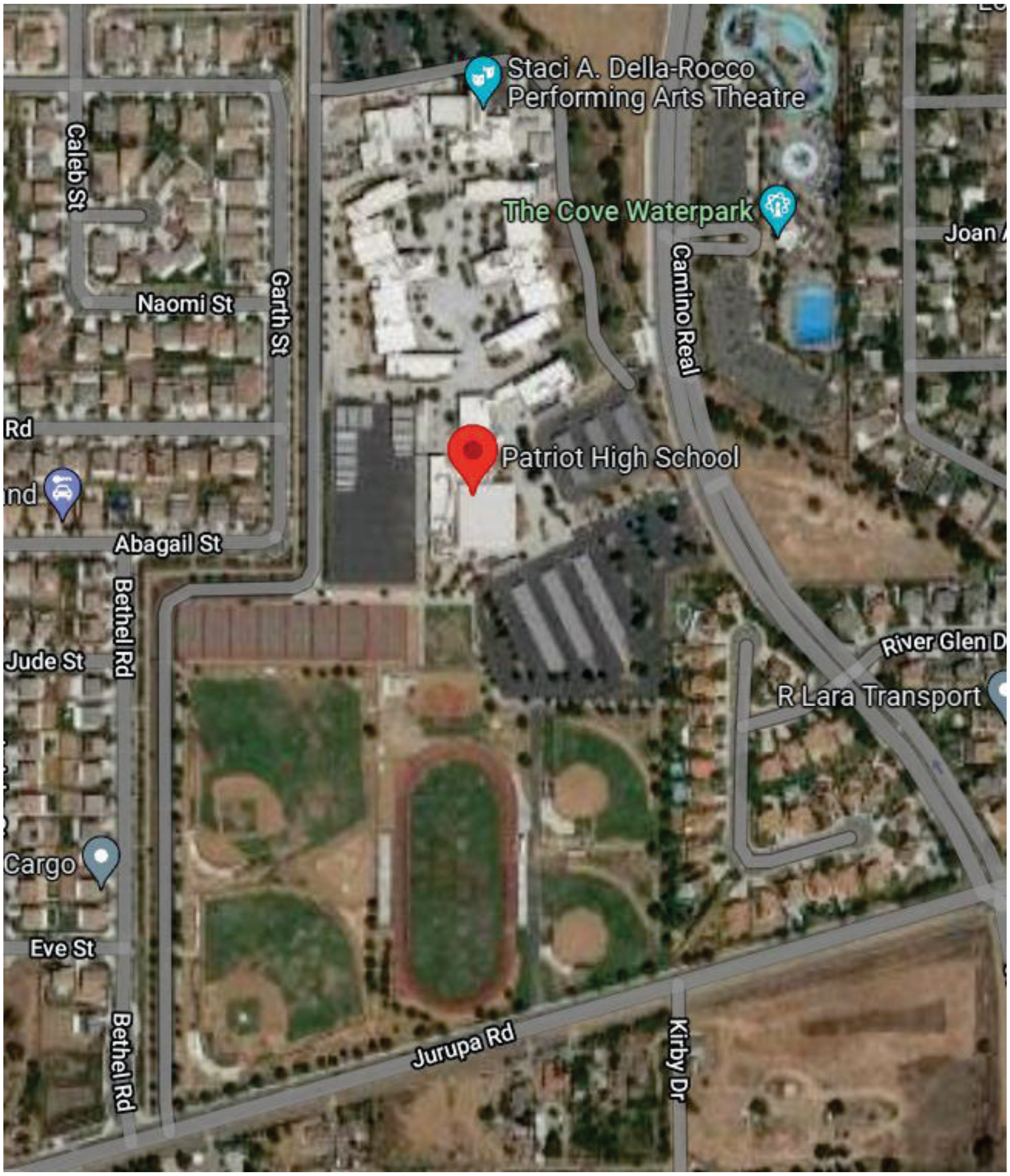


Figure 2- Soil Sample Locations, 4355 Camino Real, Jurupa Valley, CA 92509

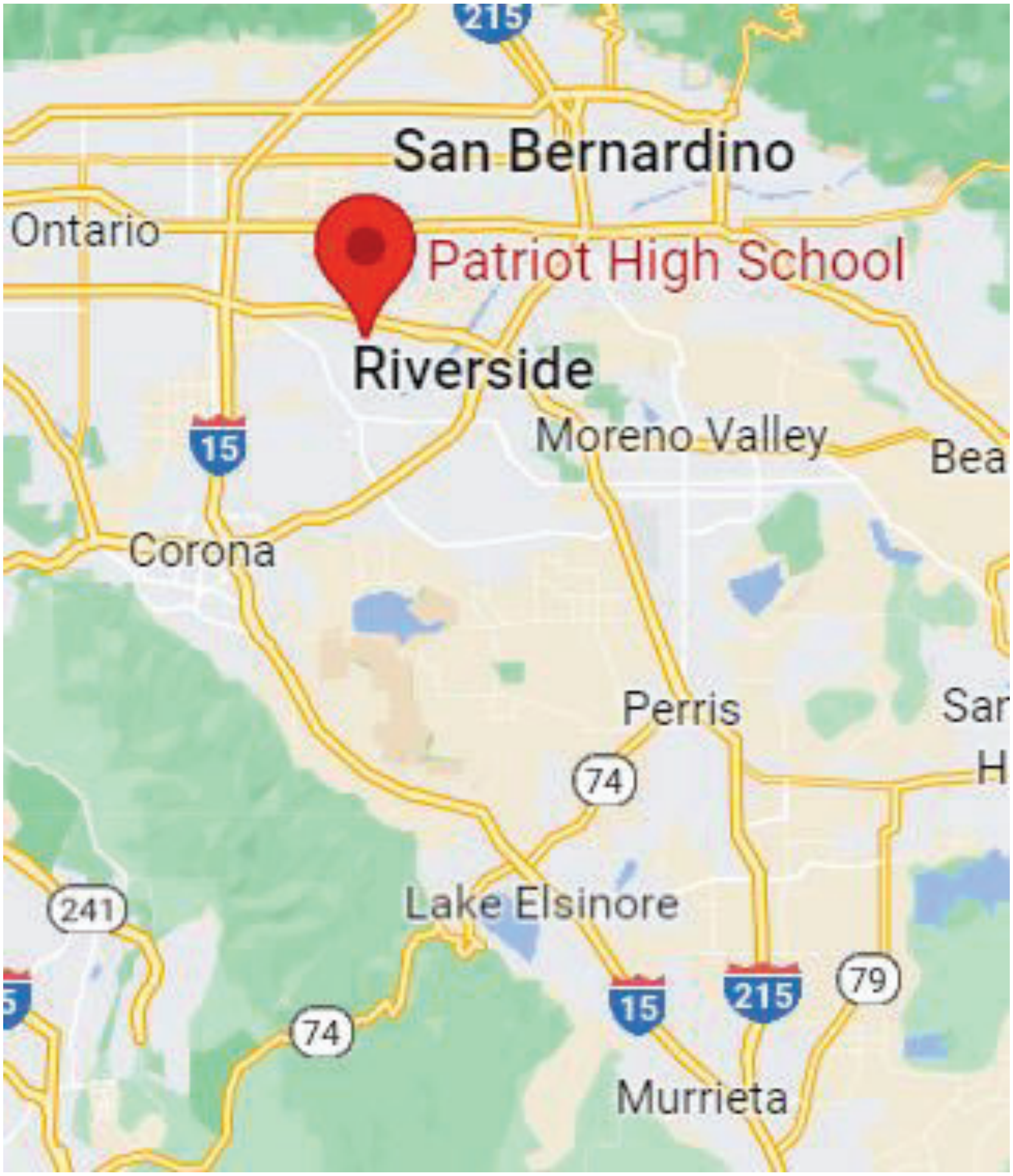


Figure 3- Vicinity Map, 4355 Camino Real, Jurupa Valley, CA 92509

5 Corrosion Basics

In general, the corrosion rate of metals in soil depends on the electrical resistivity, the elemental composition, and the oxygen content of the soil. Soils can vary greatly from one acre to the next, especially at earthquake faults. The better a soil is for farming; the easier it will be for corrosion to take place. Expansive soils should be considered disturbed simply because of their nature from dry to wet seasons.

5.1 Pourbaix Diagram – In regards to a material's environment

All metals are unique and have a weakness. Some metals do not like acidic (low pH) environments. Some metals do not like alkaline (high pH) environments. Some metals don't like either high or low pH environments such as aluminum. These are called amphoteric materials. Some metals become passivated and do not corrode at high pH environments such as steel. These characteristics are documented in Marcel Pourbaix's book "Atlas of electrochemical equilibria in aqueous solutions"

In the mid 1900's, Marcel Pourbaix developed the Pourbaix diagram which describes a metal's reaction to an environment dependent on pH and voltage conditions. It describes when a metal remains passive (non-corroding) and in which conditions metals become soluble (corrode). Steels are passive in pH over 12 such as the condition when it is encased in cement. If the cement were to carbonate and its pH reduce to below 12, the cement would no longer be able to act as a corrosion inhibitor and the steel will begin to corrode when moist.

Some metals such as aluminum are amphoteric, meaning that they react with acids and bases. They can corrode in low pH and in high pH conditions. Aluminum alloys are generally passive within a pH of 4 and 8.5 but will corrode outside of those ranges. This is why aluminum cannot be embedded in cement and why brick mortar should not be laid against an aluminum window frame without a protective barrier between them.

5.2 Galvanic Series – In regards to dissimilar metal connections

All metals have a natural electrical potential. This electrical potential is measured using a high impedance voltmeter connected to the metal being tested and with the common lead connected to a copper copper-sulfate reference electrode (CSE) in water or soil. There are many types of reference electrodes. In laboratory measurements, a Standard Hydrogen Electrode (SHE) is commonly used. When different metal alloys are tested they can be ranked into an order from most noble (less corrosion), to least noble (more active corrosion). When a more noble metal is connected to a less noble metal, the less noble metal will become an anode and sacrifice itself through corrosion providing corrosion protection to the more noble metal. This hierarchy is known as the galvanic series named after Luigi Galvani whose experiments with electricity and muscles led Alessandro Volta to discover the reactions between dissimilar metals leading to the early battery. The greater the voltage difference between two metals, the faster the corrosion rate will be.

Table 1- Dissimilar Metal Corrosion Risk

	Zinc	Galvanized Steel	Aluminum	Cast Iron	Lead	Mild Steel	Tin	Copper	Stainless Steel
Zinc	None	Low	Medium	High	High	High	High	High	High
Galvanized Steel	Low	None	Medium	Medium	Medium	High	High	High	High
Aluminum	Medium	Medium	None	Medium	Medium	Medium	Medium	High	High
Cast Iron	High	Medium	Medium	None	Low	Low	Low	Medium	Medium
Lead	High	Medium	Medium	Low	None	Low	Low	Medium	Medium
Mild Steel	High	High	Medium	Low	Low	None	Low	Medium	Medium
Tin	High	High	Medium	Low	Low	Low	None	Medium	Medium
Copper	High	High	High	Medium	Medium	Medium	Medium	None	Low
Stainless Steel	High	High	High	Medium	Medium	Medium	Medium	Low	None

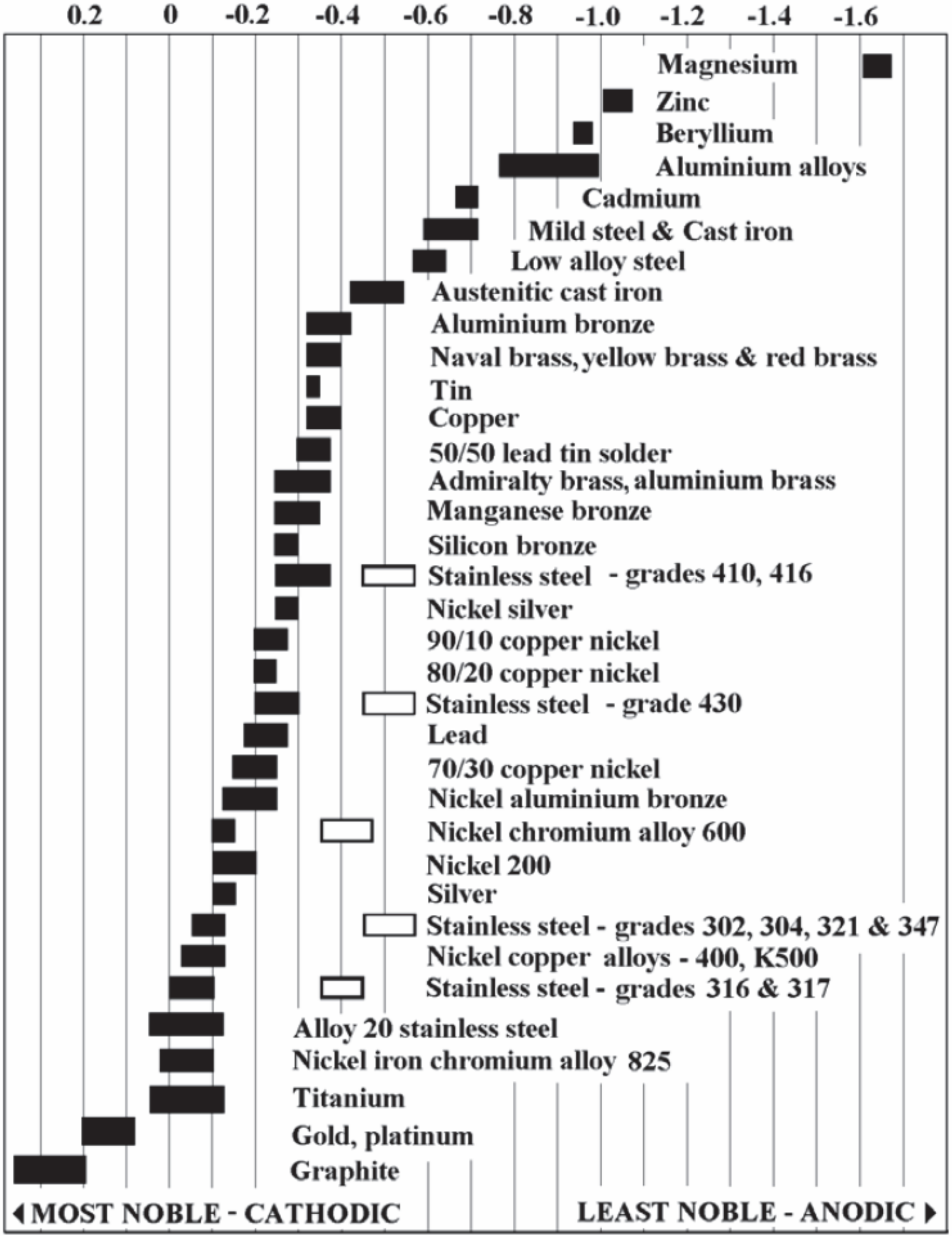
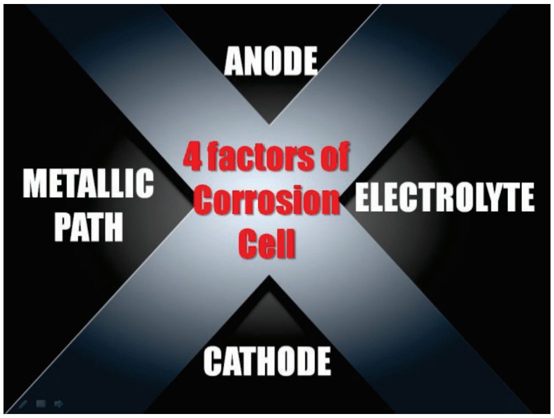


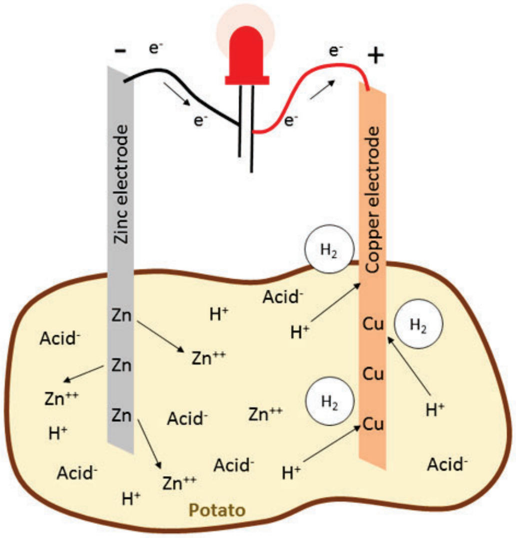
Figure 4 - Galvanic series of metals relative to CSE half cell.

5.3 Corrosion Cell

In order for corrosion to occur, four factors must be present. (1) The anode (2) the cathode (3) the electrolyte and (4) the metallic or conductive path joining the anode and the cathode. If any one of these is removed, corrosion activity will stop. This is how a simple battery produces electricity. An example of a non-metallic yet conductive material is graphite. Graphite is similar in nobility to gold. Do not connect graphite to anything in moist environments.



The anode is where the corrosion occurs, and the cathode is the corrosion free material. Sometimes the anode and cathode are different materials connected by a wire or union. Sometimes the anode and cathode are on the same pipe with one area of the pipe in a low oxygen zone while the other part of the pipe is in a high oxygen zone. A good example of this is a post in the ocean that is repeatedly splashed. Deep underwater, corrosion is minimal, but at the splash zone, the corrosion rate is greatest.



Low oxygen zones and crevices can also harbor corrosive bacteria which in moist environments will lead to corrosion. This is why pipes are laid on backfill instead of directly on native cut soil in a trench. Filling a trench slightly with backfill before installing pipe then finishing the backfill creates a uniform environment around the entire surface of the pipe.

The electrolyte is generally water, seawater, or moist soil which allows for the transfer of ions and electrical current. Pure water itself is not very conductive. It is when salts and minerals dissolve into pure water that it becomes a good conductor of electricity and chemical reactions. Metal ores are turned into metal alloys which we use in construction. They naturally want to return to their natural metal ore state but it requires energy to return to it. The corrosion cell, creates the energy needed to return a metal to its natural ore state.

The metallic or conductive path can be a wire or coupling. Examples are steel threaded into a copper joint, or an electrician grounding equipment to steel pipes inadvertently connecting electrical grid copper grounding systems to steel or iron underground pipes.

The ratio of surface area between the anode and the cathode is very important. If the anode is very large, and the cathode is very small, then the corrosion rate will be very small and the anode may live a long life. An example of this is when short copper laterals were connected to a large and long steel pipeline. The steel had plenty of surface area to spread the copper's attack, thus corrosion was not noticeable. But if the copper was the large pipe and the steel the short laterals, the steel would corrode at an amazing rate.

5.4 Design Considerations to Avoid Corrosion

The following recommendations are based upon typical observations and conclusions made by forensic engineers in construction defect lawsuits and NACE International (Corrosion Society) recommendations.

5.4.1 Testing Soil Factors (Resistivity, pH, REDOX, SO, CL, NO3, NH3)

As previously mentioned, different factors can cause corrosion. The most useful and common test for categorizing a soil's corrosivity has been the measure of soil resistivity which is typically measured in units of (ohm-cm) by corrosion engineers and geologists. Soil resistivity is the ability of soil to conduct or resist electrical currents and ion transfer. The lower the soil resistivity, the more conductive and corrosive it is. The following are "generally" accepted categories but keep in mind, the question is not "Is my soil corrosive?", the question should be, "What is my soil corrosive to?" and to answer that question, soil resistivity and chemistry must be tested. Though **soil resistivity is a good corrosivity indicator for steel materials, high chlorides or other corrosive elements do not always lower soil resistivity, thus if you don't test for chlorides and other water soluble salts, you can get an unpleasant surprise.** The largest contributing factor to a soil's electrical resistivity is its clay, mineral, metal, or sand make-up.

Table 2 - Corrosion Basics- An Introduction, NACE, 1984, pg 191

(Ohm-cm)	Corrosivity Description
0-500	Very Corrosive
500-1,000	Corrosive
1,000-2,000	Moderately Corrosive
2,000-10,000	Mildly Corrosive
Above 10,000	Progressively less corrosive

Testing a soil's pH provides information to reference the Pourbaix diagram of specific metals. Some elements such as ammonia and nitrates can create localized alkaline conditions which will greatly affect amphoteric materials such as aluminum and copper alloys.

Excess sulfates can break-down the structural integrity of cement and high concentrations of chlorides can overcome cement's corrosion inhibiting effect on encased ferrous metals and break down protective passivated surface layers on stainless steels and aluminum.

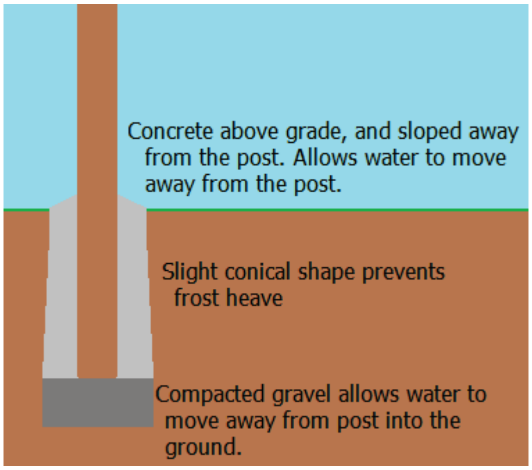
Corrosive bacteria are everywhere but can multiply significantly in anaerobic conditions with plentiful sulfates. The bacteria themselves do not eat the metal but their by-products can form corrosive sulfuric acids. The probability of corrosive bacteria is tested by measuring a soil's oxidation-reduction (REDOX) electro-potential and by testing for the presence of sulfides.

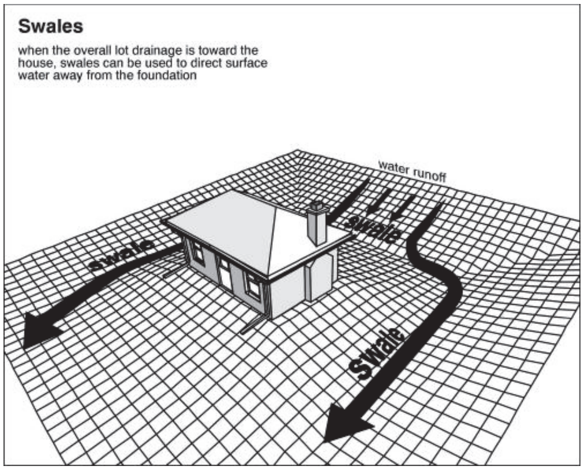
Only by testing a soil's chemistry for minimum resistivity, pH, chlorides, sulfates, sulfides, ammonia, nitrate, and redox potential can one have the information to evaluate the corrosion risk to construction materials such as steel, stainless steel, galvanized steel, iron, copper, brass, aluminum, and concrete.

5.4.2 Proper Drainage

It cannot be emphasized enough that pooled stagnant water on metals will eventually lead to corrosion. This stands for internal corrosion and external corrosion situations. In soils, providing good drainage will lower soil moisture content reducing corrosion rates. Attention to properly sealing polyethylene wraps around valves and piping will avoid water intrusion which would allow water to pool against metals. Above ground structures should not have cupped or flat surfaces that will pond water after rain or irrigation events.

Buildings typically are built on pads and have swales when constructed to drain water away from buildings directing it towards an acceptable exit point such as a driveway where it continues draining to a local storm drain. Many homeowners, landscapers and flatwork contractors appear to not be aware of this and destroy swales during remodeling. The majority of garage floor and finished grade elevations are governed by drainage during design. [13,14]





5.4.3 Avoiding Crevices

Crevices are excellent locations for oxygen differential induced corrosion cells to begin. Crevices can also harbor corrosive bacteria even in the most chemically treated waters. Crevices will also gather salts. If water's total alkalinity is low, its ability to maintain a stable pH can also become more difficult within a crevice allowing the pH to drop to acidic levels continuing a pitting process. Welds in extremely corrosive environments should be complete and well filleted without sharp edges to avoid crevices. Sharp edges should be avoided to allow uniform coating of protective epoxy. Detection of crevices in welds should be treated immediately. If pressures and loads are low, sanding and rewelding or epoxy patching can be suitable repairs. Damaged coatings can usually be repaired with Direct to Metal paints. **Scratches and crevice corrosion are like infections, they should not be left to fester or the infection will spread making things worse.**

[13] https://www.fencedaddy.com/blogs/tips-and-tricks/132606467-how-to-repair-a-broken-fence-post

[14] http://southdownstudio.co.uk/problme-drainage-maison.html

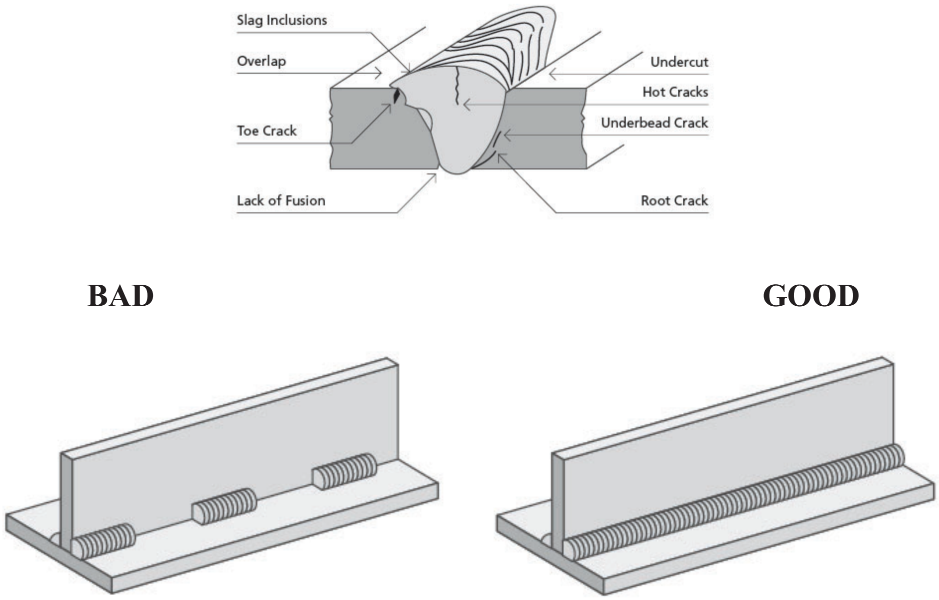


Figure 5- Defects which form weld crevices[15]

5.4.4 *Coatings and Cathodic Protection*

When faced with a corrosive environment, the best defense against corrosion is removing the electrolyte from the corrosion cell by applying coatings to separate the metal from the soil. During construction and installation, there is always some scratch or damage made to a coating. NACE training recommends that coatings be used as a first line of defense and that sacrificial or impressed current cathodic protection is used as a 2nd line of defense to protect the scratched areas. Use of a good coating dramatically reduces the amount of anodes a CP system would need. If CP is not installed as a 2nd line of defense in an extremely corrosive environment, the small scratched zones will suffer accelerated corrosion. CP details such as anode installation instructions must be designed by corrosion engineers or vessel manufacturers on a per project basis because it depends on electrolyte resistivity, surface area of infrastructure to be protected, and system geometry.

There are two types of cathodic protection systems, a Galvanic Anode Cathodic Protection (GACP) system and an Impressed Current Cathodic Protection (ICCP) system. A Galvanic Anode Cathodic Protection (GACP) system is simpler to install and maintain than an Impressed Current Cathodic Protection (ICCP) system. To protect the metals, they must all be electrically continuous to each other. In a GACP system, sacrificial zinc or magnesium anodes are then buried at locations per the CP design and connected by wire to a structure at various points in system. At the connection points, a wire connecting to the structure and the wire from the anode are joined in a Cathodic Protection Test Station hand hole which looks similar in size and shape to an irrigation valve pull box. By coating the underground structures, one can reduce the number of anodes needed to provide cathodic protection by 80% in many instances.

An ICCP system requires a power source, a rectifier, significantly more trenching, and more expensive type anodes. These systems are typically specified when bare metal is requiring protection

[15] http://www.daroproducts.co.uk/makes-good-weld/

in severely corrosive environments in which galvanic anodes do not provide enough power to polarize infrastructure to -850 mV structure-to-soil potential or be able to create a 100 mV potential shift as required by NACE SP169 to control corrosion. In severely corrosive environments, a GACP system simply may not last a required lifetime due to the high rate of consumption of the sacrificial anodes. ICCP system rectifiers must be inspected and adjusted quarterly or at a minimum bi-annually per NACE recommendations. Different anode installations may be possible but for large sites, anodes are placed evenly throughout the site and all anode wires must be trenched to the rectifier. For a large site, it may be beneficial to use two or more rectifiers to reduce wire lengths or trenching.

To simplify, a GACP system can be installed and practically forgotten with minor trenching because the anodes can be installed very close to the structures. An ICCP system must be inspected annually and anode wires run back to the rectifier which itself connects to the pile system. If any type of trenching or development is expected to occur at the site during the life of the site, it is a good idea to inspect the anode connections once a year to make sure wires are not cut and that the infrastructure is still being provided adequate protection. A common situation that occurs with ICCP systems is that a contractor accidently cuts the wires during construction then reconnects them incorrectly, turning the once cathode, into a sacrificing anode.

Design of a cathodic protection system protecting against soil side corrosion requires that Wenner Four Pin ground resistance measurements per ASTM G57 be performed by corrosion engineers at various locations of the site to determine the best depths and locations for anode installations. Ideally, a sample pile is installed and experiments determining current requirement are conducted. Using this data, the decision is made whether a GACP system is feasible or if an ICCP must be used.

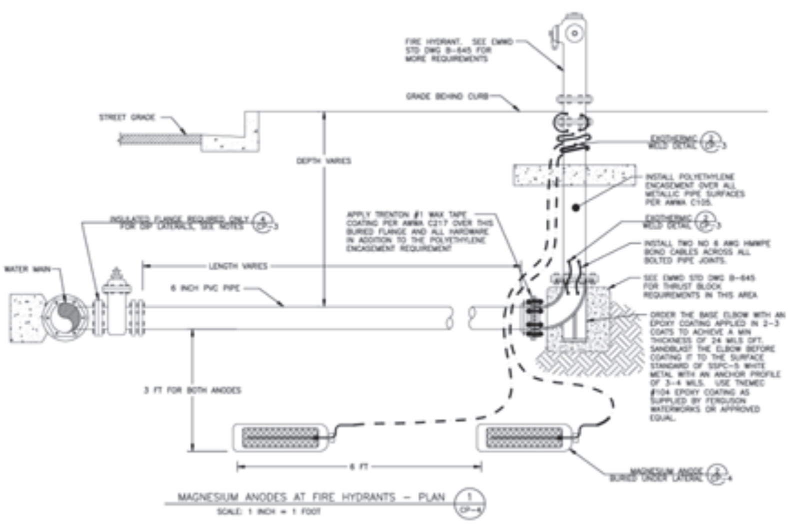


Figure 6- Sample anode design for fire hydrant underground piping

Vessels such as water tanks will have protective interior coatings and anodes to protect the interior surfaces. Anodes can also be buried on site and connected to system skid supports to protect the metal in contact with soil. A good example of a vessel cathodic protection system exists in all home water heaters which contain sacrificial aluminum or magnesium anodes. In environments that exceed 140F, zinc anodes cannot be used with carbon steel because they become the aggressor (Cathodic) to the steel instead of sacrificial (anodic). Anodes in vessels containing extremely brackish water with chloride levels over 2,000 ppm should inspect or change out their anodes every 6 months.

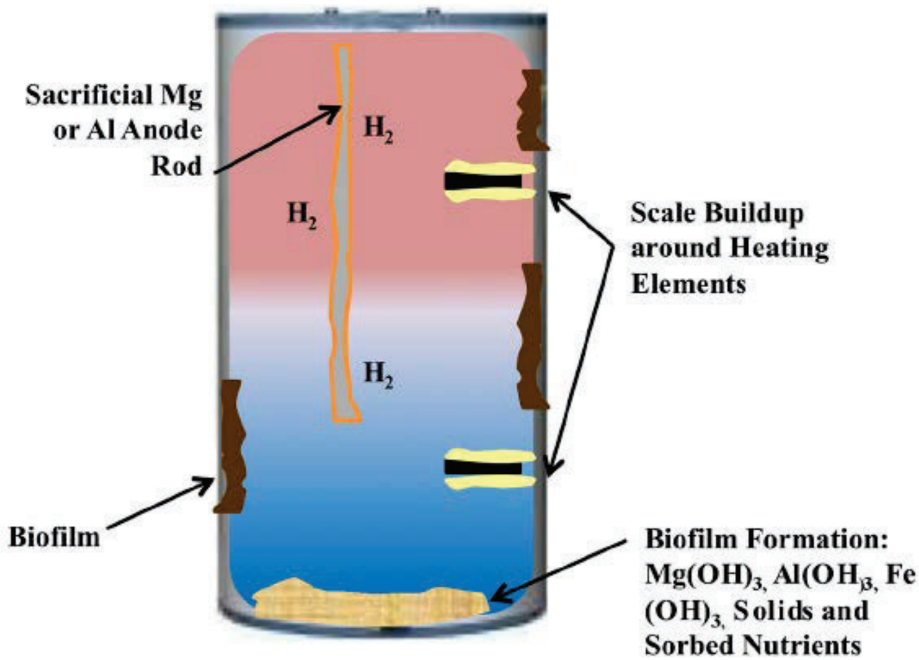


Figure 7- Cross section of boiler with anode

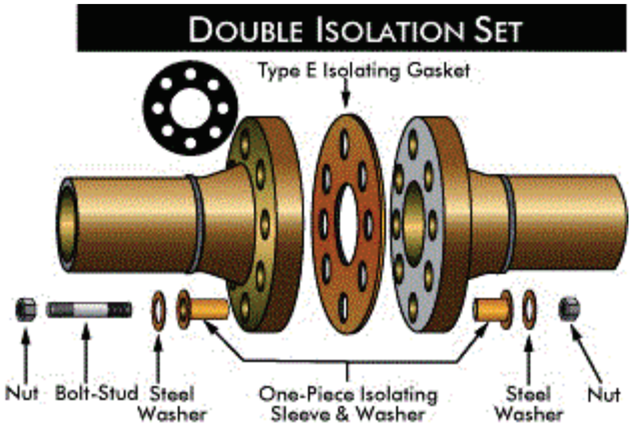
Cathodic protection can only protect a few diameters within a pipeline thus it is not recommended for small diameter pipelines and tubing internal corrosion protection. Anodes are like a lamp shining light in a room. They can only protect along their line of sight.

5.4.5 Good Electrical Continuity

In order for cathodic protection to protect a long pipeline or system of pipes from external soil side corrosion, they must all be electrically continuous to each other so that the electric current from the anode can travel along the pipes, then return through the earth to the anode. Electrical continuity is achieved by welding or pin brazing #8 AWG copper strand bond cable to the end of pipe sticks which have rubber gaskets at bell and spigots. If steel pipes are joined by full weld, bonding wires are not needed.

Electrical continuity between dissimilar metals is not desirable. Isolation joints or di-electric unions should be installed between dissimilar metals, such as steel pipes connecting to a brass valve per NACE SP0286. Bonding wires should then be welded onto the steel pipes by-passing the brass valve so that the cathodic protection system's current can continue to travel along the steel piping but isolate the brass valve from the steel pipeline. Another option would be to provide a separate cathodic protection system for steel pipes on both sides of the brass valve.

Typically, water heater inlets and outlets, gas meters and water meters have dielectric unions installed in them to separate utility property from homeowner property. This also protects them in the case that a home owner somehow electrically connects water pipes or gas pipes to a neighborhood electrical grounding system which can potentially have less noble steel in soil now connected to much more noble copper in soil which will then create a corrosion cell. This is exactly how a lemon powered clock works when a galvanized zinc nail and a steel nail are inserted into a lemon then connected to a clock. The clock is powered by the corrosion cell created.



5.4.6 Bad Electrical Continuity

Bad electrical continuity is when two different materials or systems are made electrically continuous (aka shorted) when they were not designed to be electrically continuous. Examples of this would be when gas lines are shorted to water lines or to electrical grounding beds. Very often, fire risers are shorted to electrical grounding systems, and water pipes at business parks. Since fire risers usually have a very short ductile iron pipe in the ground which connects to PVC pipe systems, they tend to experience leaks after 7 to 10 years of being attacked by underground copper systems.

It is absolutely imperative that any copper water piping or other metal conduits penetrating cement slab or footings, not come in contact with the reinforcing steel or post-tensioning tendons to avoid creation of galvanic corrosion cells.

5.4.7 Corrosion Test Stations

Corrosion test stations should be installed every 1,000 feet along pipelines in order to measure corrosion activity in the future. For a simple pipeline, two #8 AWG copper strand bond cable welded or pin brazed onto the pipeline are run up to finished grade and left in a hand hole. Corrosion test stations are used to measure pipe-to-soil electro potential relative to a copper copper-sulfate reference electrode to determine if the pipe is experiencing significant corrosion activity. By measuring test stations along a pipeline, hot spots can be determined, if any. The wires also allow for electrical continuity testing, condition assessment, and a multitude of other types of tests.

At isolation joints and pipe casings, two wires should be welded to either side of the isolation joint for a total of 4 wires to be brought up to the hand hole. This allows for future tests of the isolation joint, casing separation confirmation, and pipe-to-soil potential readings during corrosion surveys.

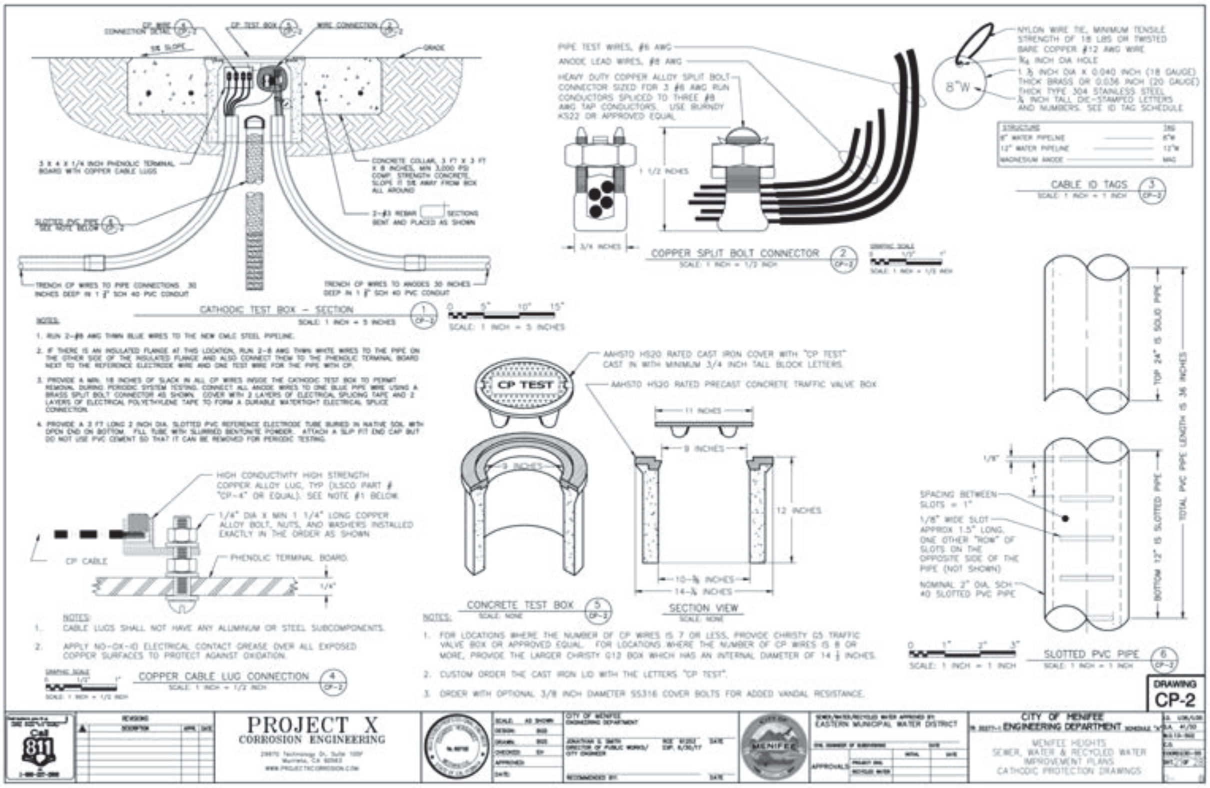


Figure 8- Sample of corrosion test station specification drawing

5.4.8 *Excess Flux in Plumbing*

Investigations of internal corrosion of domestic water plumbing systems almost always finds excess flux to be the cause of internal pitting of copper pipes. Some people believe that there is no such thing as too much flux. Flux runs have been observed to travel up to 20 feet with pitting occurring along the flux run. Flushing a soldered plumbing system with hot water for 15 minutes can remove significant amounts of excess flux left in the pipes. If a plumbing system is expected to be stagnant for some time, it should be drained to avoid stagnant water conditions that can lead to pitting and dezincification of yellow brasses.

5.4.9 *Landscapers and Irrigation Sprinkler Systems*

A significant amount of corrosion of fences is due to landscaper tools scratching fence coatings and irrigation sprinklers spraying these damaged fences. Recycled water typically has a higher salt content than potable drinking water, meaning that it is more corrosive than regular tap water. The same risk from damage and water spray exists for above ground pipe valves and backflow preventers. Fiber glass covers, cages, and cement footings have worked well to keep tools at an arm’s length.

5.4.10 *Roof Drainage splash zones*

Unbelievably, even the location where your roof drain splashes down can matter. We have seen drainage from a home’s roof valley fall directly down onto a gas meter causing it’s piping to corrode at an accelerated rate reaching 50% wall thickness within 4 years. It is the same effect as a splash zone in the ocean or in a pool which has a lot of oxygen and agitation that can remove material as it corrodes.

5.4.11 Stray Current Sources

Stray currents which cause material loss when jumping off of metals may originate from direct-current distribution lines, substations, or street railway systems, etc., and flow into a pipe system or other steel structure. Alternating currents may occasionally cause corrosion. The corrosion resulting from stray currents (external sources) is similar to that from galvanic cells (which generate their own current) but different remedial measures may be indicated. In the electrolyte and at the metal-electrolyte interfaces, chemical and electrical reactions occur and are the same as those in the galvanic cell; specifically, the corroding metal is again considered to be the anode from which current leaves to flow to the cathode. Soil and water characteristics affect the corrosion rate in the same manner as with galvanic-type corrosion.

However, stray current strengths may be much higher than those produced by galvanic cells and, as a consequence, corrosion may be much more rapid. Another difference between galvanic-type currents and stray currents is that the latter are more likely to operate over long distances since the anode and cathode are more likely to be remotely separated from one another. Seeking the path of least resistance, the stray current from a foreign installation may travel along a pipeline causing severe corrosion where it leaves the line. Knowing when stray currents are present becomes highly important when remedial measures are undertaken since a simple sacrificial anode system is likely to be ineffectual in preventing corrosion under such circumstances.[16] Stray currents can be avoided by installing proper electrical shielding, installation of isolation joints, or installation of sacrificial jump off anodes at crossings near protected structures such as metal gas pipelines or electrical feeders.

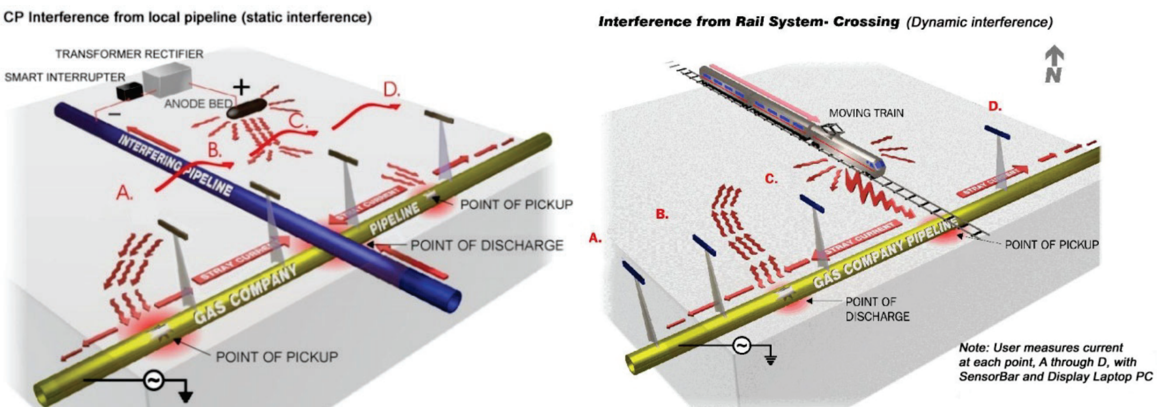


Figure 9- Examples of Stray Current[17]

[16] http://corrosion-doctors.org/StrayCurrent/Introduction.htm

[17] http://www.eastcomassoc.com/