FENAGH
ENGINEERING AND TESTING

GEOTECHNICAL INVESTIGATION AND GEOHAZARD REPORT

JUNE 28, 2024



PROJECT

PROPOSED SPORTS COMPLEX
14777 HOPLAND STREET
VICTORVILLE, CALIFORNIA

PREPARED FOR

MS. SHEERE BISHOP-GRIEGO
VICTOR ELEMENTARY SCHOOL

REPORT NO.
7656-01

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June 28, 2024

Ms. Sheere Bishop-Griego
Victor Elementary School District

Subject: **Geotechnical Investigation and Geohazard Report**
Challenger School of Sports and Fitness Complex
14777 Hopland St.
Victorville, CA 92394

Dear Ms. Bishop-Griego,

Fenagh Engineering and Testing Services (Fenagh) is pleased to present this Geotechnical Exploration and Geohazard Report for the proposed development located at 14777 Hopland Street in Victorville, 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



Scott T. Prince, PE
Project Engineer



Emil Rudolph, GE
Geotechnical Engineer

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1 INTRODUCTION

1.1 GENERAL

The following Geotechnical Exploration and Geohazard Report is presented as follows for the proposed development located at 14777 Hopland St. in Victorville, California. The purpose of this investigation 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

Based on the review of the Victor Elementary School District for Challenger Elementary School proposal and scope of services contract, we understand that Victor Elementary School District plans to construct a new sports complex area. The proposed development will consist of a new natural field with surrounding synthetic track, bleachers on both sides of the field, stadium lighting, fencing & gates and a modular building (approximately 30 feet by 50 feet in floor plan area) with restrooms, storage and warming kitchen. The development is also assumed to include site improvements which may consist of stormwater retention, underground utilities, landscaping, as well as miscellaneous flatwork and hardscape features. No below grade basement is anticipated for the proposed development scope.

The project site in relation to local developments and cityscape features is indicated on the attached Vicinity Map, Plate 1. Details and features of the planned development are indicated on the attached Exploration Maps, Plates 2 and 3.

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

Modular Building -
- 1 to 2 kips per lineal foot wall loads for dead plus live load conditions
- 50-kip column loads for dead plus live load conditions

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 February 9, 2024 (Fenagh Proposal FP4858). The scope of services generally includes the following:

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

- Site reconnaissance to observe and evaluate site features including geologic conditions;
- Coordination with Underground Service Alert (USA) to locate underground utilities in the vicinity of subsurface explorations;
- Coordination with client to locate underground utilities not covered by USA;
- Subsurface exploration consisting of 8 borings using hollow-stem auger drilling methodologies to depths of up to 51-1/5 feet below ground surface. Samples logged to characterize subsurface conditions and collected for laboratory testing;
- Laboratory testing on selected samples to evaluate the in-situ moisture content, dry density, grain size distribution, compaction characteristics, expansion properties, 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, compilation of field and laboratory data collected, including 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 Challenger School of Sports and Fitness Campus is located at 14777 Hopland Street, Victorville, California. The school property borders Hopland Street to the north, Cahuenga Road to the east, vacant lots to the south and west. The school campus is located on a property identified by San Bernardino County Assessor's Parcel Numbers (APN) 0395244140000, and 0395244170000, with a combined land area of approximately 16-3/4 acres. According to available online aerial photographs (NETRonline: Historic Aerials) the main buildings of the Challenger School of Sports and Fitness were built prior to 2005. The project site ranges in elevation from approximately 2,892 feet above MSL in the east to approximately 2,890 feet in the west.

The project site is located in the vacant lot to the south of the playfield area within the Challenger School of Sports and Fitness Campus as indicated on the attached Exploration Maps, Plates 2 and 3.

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

34.5404 N Latitude
-117.3409 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 8 borings identified as Borings B-1 through B-8 to depths ranging from approximately 8 feet to 51-1/2 feet below the ground surface (bgs).

Borings B-1 through B-6 and B-8 were drilled using a truck-mounted CME-75 drill rig equipped with 8-inch outer diameter hollow-stem augers with a carbide tooth and blade drill bit. Boring B-7 was hand-augured with manual labor using hand operated tools and equipment. Table 2-1 below summarizes the boring depths and approximate locations. The boring Logs indicating subsurface geologic conditions observed during site exploration are presented in Appendix A.

Table 2 1: Exploration Depths and Locations

Exploration ID	Depth of Exploration (ft)	Approximate Latitude	Approximate Longitude	General Location
B-1	26-1/2	34.540961°	-117.4341156°	Western portion of the North Bleachers
B-2	11-1/2	34.540712°	-117.341438°	Western portion of the Field
B-3	21-1/2	34.540234°	-117.341368°	Western portion of the South Bleachers
B-4	26-1/2	34.539905°	-117.340693°	Eastern portion of the South Bleachers
B-5	8	34.540203°	-117.340322°	Eastern portion of the Field
B-6	51-1/2	34.540537°	-117.340230°	Eastern portion of the North Bleachers
B-7	10	34540539°	-117340089°	Eastern portion of the Modular Building
B-8	21-1/2	34.540572°	-117.340183°	Western portion of the Modular Building

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.

2.2 LABORATORY TEST RESULTS

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

- Moisture Content (ASTM D2216)
- Dry Density (ASTM D2937)
- Amount of Material Finer than 75-mm (ASTM D1140)
- Laboratory Compaction of Soils Using Modified Effort (ASTM D1557)
- Direct Shear Test of Soils under Consolidated Drained Conditions (ASTM D3080)
- One-Dimensional Consolidation Properties – Incremental Loading (ASTM D2435)
- Expansive Index (ASTM D4829)
- R-Value (ASTM D2844)
- pH and Electrical Resistivity (CT643)
- Sulfate and Chloride Content (CT417 and CT422)

The general results of the tests performed above, and characterization of site soils are discussed in the Subsurface Conditions section of this report (Section 3.3). Laboratory test results are provided in Appendix B.

3 FINDINGS

3.1 REGIONAL GEOLOGIC SETTING

The project site is located in the informally designated High Desert region of Southern California, which resides in the northern part of San Bernardino County. The High Desert region area is located within the southwestern portion of the Mojave Desert Geomorphic Province. This province is characterized by broad isolated mountain ranges separated by expanses of desert plains. The Mojave Desert province is wedged between the Garlock Fault to the north and the San Andreas Fault to the west.

3.2 SITE GEOLOGY

The Geologic Map of the Victorville 7.5-minute Quadrangle indicates the site is underlain by alluvium of the ancestral Mojave River (Qoam). Geologic materials obtained from the onsite subsurface investigation by this firm were consistent with research findings. Alluvial soils were encountered to a maximum explored depth of 51-1/2 feet beneath existing ground surface. Regional and local geology for the project site is indicated on Plates 4 and 5, respectively. Details of the subsurface conditions encountered during onsite exploration are provided in Section 3.3 as follows.

3.3 SUBSURFACE CONDITIONS

Borings B-1 through B-8 were drilled on April 30 and May 1, 2024, in the footprint of the proposed sports complex northern and southern bleachers and in the west and east sections of the field. Borings B-7 and B-8 were drilled to the west and east of the proposed modular building to be located approximately 25 feet to the north of the north bleachers as indicated by the attached Exploration Maps, Plates 2 and 3.

Surface layers observed during onsite exploration consisted of fill. Fill depths were observed to extend to depths ranging from approximately 3-1/2 feet to 4 feet below ground surface. The fill

consists primarily of Silty Sand to Sand, which is light brown in color, medium dense to dense, slightly moist to dry, and fine to medium grained with occasional gravel.

Alluvial soil underlies the fill which consists of Poorly Graded Sand with gravel (SP) to depths of approximately 10 feet then transitions to Silty Sand (SM) and Sandy Silt (ML) to an approximate depth of 35 feet and finally transitions to Poorly Graded Sand (SP) at depths deeper than 35 feet below ground surface. The alluvium was observed to be predominantly light gray to light brown in color, moist to dry, and fine to coarse grained with occasional gravel. The blowcount values indicate medium dense to very dense soil conditions which were confirmed by laboratory testing. Locations of drilled Borings B-1 through B-8 are indicated on the Exploration Maps, Plates 2 and 3. Graphical presentations of the boring logs are provided in Appendix A.

3.4 GROUNDWATER CONDITIONS

Groundwater was not encountered during site exploration to a maximum excavated depth of 51-1/2 feet. Groundwater is not expected to impact the design or construction of the project. Historically highest groundwater levels are approximated based on water well data obtained from nearby monitoring wells as indicated below.

Fenagh 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. Based on water level data obtained from 1992 to 2014, the historically highest groundwater is estimated to extend deeper than 150 feet below existing ground surface.

Table 3 1: Regional Groundwater Conditions (lowest site elevation approx. 2,890 feet)

Site Code	Ground Surface Elevation (ft, msl)	Distance/Direction from Site	Reported Approx. Range in Depth to Groundwater/ Record Date
345441N1173320W001	2,859	3,000 ft east	2,688’ to 2,725’ 1992 to 2014
345294N1173464W001	2,938	3,800 ft southwest	2,724’ to 2,727’ 2006 to 2010

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.

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 proposed sports complex 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, medium dense to dense, Sand (SP) to Silty Sand

(SM) underlain by alluvium soils consisting of medium dense to dense and very dense, Silty Sand (SM), Sandy Silts (ML), Poorly Graded Sand with silt (SP) and Poorly Graded Sand with gravels. 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 boring logs within areas of planned development to depths of approximately 3-1/2 feet to 4 feet below exiting site grade. The presence of undocumented fill would likely cause non-uniform bearing support which could result in intolerable settlement for foundations or building slabs placed thereon. The fill encountered generally exhibited medium dense to dense characteristics for the depth profile encountered. Intermittent loose zones were observed at ground surface during site exploration.

Depending on the limits and strength characteristics of undocumented fill, differential settlement could impact the proposed structures. Undocumented fills are generally addressed with complete removal and replacement with certified engineered fill. Based on these considerations, we recommend that undocumented fill is removed and recompacted as certified fill to create an engineered building pad for uniform support of the building slab-on-grade and conventional foundations for the proposed modular building, stadium bleachers, and playfield areas. The recommended engineered building pad fill should extend a minimum of 4 feet below the existing grade and a minimum of 3 feet below the bottom surface of foundations, whichever is more conservative. In addition, the recompacted fill pad should be over-excavated a minimum of 5 feet horizontally beyond the outer edge of the foundations face, or a distance equal to the depth of fill below the foundations, whichever is greater. 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 placing 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 increased maintenance costs should be anticipated.

5.3 EARTHWORK

5.3.1 Site Preparation

Prior to site grading, existing flatwork, pavements, or existing structures and associated foundations requiring demolition should be removed from the construction limits. Demolition debris may be crushed onsite to use in the ongoing grading operations subject to the following limitations (the environmental ramifications of this option, if any, should be considered by the team). The demolition debris should be limited to concrete and other non-deleterious materials. All deleterious materials should be removed including, but not limited to, paper, garbage, ceramic materials, and wood. For structural fill applications, materials should be crushed to 2 inches in maximum dimension or smaller. The crushed materials should be thoroughly blended and mixed with onsite soils prior to placement as compacted fill and the amount of blended crushed material should not exceed 20 percent by volume of placed fill. The blended and mixed materials should be tested by this office prior to placement to ensure the fill is suitable for compaction purposes. In addition, the blended and mixed materials should be tested by this firm (Fenagh) during placement and grading to ensure that the engineered fill has been compacted in a suitable manner.

The site should also be stripped of vegetation, organics, debris, and topsoil. Where roots are less than 1/4-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 site improvements (i.e., foundations or slabs-on-grade), should be scarified to a depth of at least 8 inches in development areas. 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.

Excavations should be observed by the project Geotechnical Engineer or their designated representative to verify the fill has been removed and the subgrade meets the general recommendations provided in this report.

5.3.2 Engineered Fill

On-site or imported 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.

In addition, import material should comply with the general environmental sampling and testing guidelines as recommended by the Department of Toxic Substances Control (DTSC). Specific Recommendations are provided in the “DTSC Information Advisory Clean Imported Fill Material Fact Sheet”.

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 2 feet of subgrade material, the maximum size shall be 3 inches. Specific recommendations for engineered fill including the applicable test procedures to verify suitability are presented in the following Table 5-1.

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	Greater than 5	ASTM[1] D422 or 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 40		ASTM[1] D4829
R Value		Test Procedures
Greater than 40		Caltrans Test 301

1 – American Society for Testing and Materials Standards

5.3.3 Compaction Criteria Engineered Fill

Engineered fill within building and bleacher footprints, trench backfill, flatwork, including playfield and track areas 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 in accordance with the recommendations presented in Table 5-2 as follows:

Table 5 2: Recommended Compaction & Moisture Conditioning: Subgrade & Eng. Fills

Material Type and Location	Compaction and Moisture Content per Modified Proctor ASTM D1557		
	Minimum Compaction Requirement, %	Range of Moisture Contents Relative to Optimum	
		Minimum	Maximum
Subgrade to Receive Fill	90	Optimum	+4%
Trench Backfill	90	Optimum	+4%
Structural Fill for Support of Foundations	90	Optimum	+4%
Subgrade for Exterior Flatwork	90	Optimum	+4%
Retaining Wall Backfill	90	Optimum	+4%
Subgrade/Base Traffic Load Pavement	95	Optimum	+4%
Subgrade for Playfield	95	Optimum	+4%
Aggregate Base for Playfield	92 to 95	Optimum	+4%

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. It is anticipated that the onsite soils will generally consist of alluvial soils and fill material with occasional gravel.

5.3.5 Wet Weather Construction and/or Unstable Soil Conditions

The in-situ moisture content of site soils may increase after long periods of rainfall and soil subgrades may become saturated due to exposure to wet weather conditions. When wet soils are encountered, they should be remediated by the following mitigation methods: Aeration, removing and replacing saturated material with drier material, subgrade stabilization with crushed angular gravel driven into subgrade soil medium, 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 Shrinkage

Shrinkage results when a volume of soil removed at one density is compacted to a higher density. A shrinkage factor between 5 and 10 percent should be anticipated when excavating

and recompacting the existing fill and underlying alluvial soils on site to an average comparative compaction of 92 percent.

5.4 TEMPORARY EXCAVATIONS

Excavations for building pads, foundations, and miscellaneous hardscape features can be performed with typical conventional excavating machines generally in use for such projects. During construction, temporary excavations should not exceed 3 feet in height. Temporary cuts deeper/higher than 3 feet should be sloped back at maximum gradient of 1 horizontal to 1 vertical, 1(h) to 1(v), or stabilized by shoring in accordance with OSHA regulations (Note: Soils within 3 feet of the ground surface exhibit OSHA Type C). 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. Temporary excavations should proceed under observation by a representative of this firm.

5.5 FOUNDATION RECOMMENDATIONS AND DESIGN PARAMETERS

Stadium bleachers and modular building structure may be supported by conventional spread foundations bearing in certified recompacted fill. Existing fill within the footprint of the structures should be removed and recompacted as certified compacted fill for support of building slab-on-grade and conventional foundations. We recommend that undocumented fill is removed and recompacted as certified fill to create an engineered building pad for uniform support of the proposed building slab-on-grade and conventional foundations. The recommended uniform building fill pad should extend a minimum of 3 feet below the bottom surface of the conventional foundations. In addition, the proposed building pad should be over-excavated a minimum of 5 feet horizontally beyond the outer edge of foundations or for a distance equal to the depth of fill below the foundations, whichever is greater.

Drilled cast-in-place pier foundations (drilled piers) may be utilized for support of the stadium light poles anticipated as part of the proposed development as presented in Section 5.6.

5.5.1 Allowable Bearing Capacity

Conventional spread footing foundations may be utilized for support of the stadium bleachers and modular building structure. An allowable bearing capacity of 2,300 pounds per square foot (psf) should be used for the design of spread footings supported by certified engineered fill. The minimum recommended width and embedment depths are indicated as follows.

Table 5 3: Minimum Footing Dimensions

Stadium Bleachers and Modular Building Structure		
Footing Type	Minimum Depth BGS (inches)[1]	Minimum Width (Inches)
Continuous (Bldg. Interior)	12	12
Continuous (Exterior)	18	12
Isolated (Bldg. Interior)	12	18
Isolated (Exterior)	18	18

1 – The embedment depth shown is the vertical distance between the base of the foundation and the ground surface or lowest

Allowable bearing capacity may be increased at a rate of 20 percent for each additional foot of embedment to a maximum bearing capacity of 4,600 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.

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 0.3 inches. Seismic settlement is estimated at 0.2 inches for a total combined static and dynamic estimated settlement 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. A static and seismic differential settlement of approximately 1/3 inch is anticipated over a span of 30 feet.

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 0.35 may be used for dead load forces between proposed cast-in-place concrete footings and certified compacted fill. Allowable passive pressure from engineered fill may be taken as equivalent to the pressure exerted by a fluid pressure of 250 pounds per square foot, per foot of depth, (psf/ft or pcf) with a maximum earth pressure of 2,500 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 Surcharge of Existing Foundations, Retaining Walls or Undocumented Fill

Foundations for the proposed development should not be allowed to surcharge existing subterranean retaining walls, foundations of an adjacent development or existing undocumented fill. The surcharge zone of a proposed foundation is defined as a line drawn down and away at a declined slope of 1 to 1 (horizontal to vertical) from the outer bottom surface of a foundation. Where necessary, the proposed foundations in near proximity to subterranean retaining walls, deep foundations or undocumented fill should be deepened such that the existing retaining walls, foundations, or undocumented fills do not fall within the surcharge zone of the proposed foundation. Where new foundations are proposed immediately adjacent to an existing foundation, new foundations should be deepened to match the depth of the adjacent foundation.

5.5.6 Controlled Low Strength Material (CLSM)

Deepened foundations may be required to prevent surcharge of adjacent foundations, utilities, etc. 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 the excavation should be cleaned of loose materials prior to placement of 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 this firm 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.

5.5.7 Grade Beams

Grade beams may be utilized to unify and secure foundations to improve stability of conventional foundations where needed. Grade beams should be supported by certified compacted fill in accordance with the Earthwork section of this report. Structural slabs may be supported by grade beams as required.

Lateral Capacity of Grade Beams

Lateral loads may be resisted by passive pressure of certified fill placed against side of grade beam. The allowable passive resistance may be assumed as an equivalent fluid having a density of 250 pounds per cubic foot with a maximum earth pressure of 2,500 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 for resistance to lateral loads.

5.6 DRILLED PIER FOUNDATIONS

Drilled cast-in-place pier foundations may be utilized for support of the 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 native 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 should be embedded a minimum of 10 feet into alluvial soils. Historically highest groundwater depth of 100 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.

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 2022 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 500 psf/ft to a maximum of 3,000 psf may be used in determining the required depth for isolated drilled pier foundations. 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 6 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 6 pier diameters. The pile spacing in the direction perpendicular to the direction of the load should be at least 3 times the pile diameter. In general, the group reduction factor for center-to-center spacing can be determined as recommended below.

Table 5 4: Group Reduction Factors Lateral Capacity

	Lateral Capacity Reduction Factor			
Pier Spacing (c-c)	3D	4D	5D	≥6D
Lead Row	0.70	0.85	1.00	1.00
2nd Row	0.50	0.65	0.85	1.00
3rd and Higher Row	0.35	0.50	0.70	1.00

Where D = diameter of piers
Ref: U.S. Department of Transportation, Federal Highway Administration, Pub. FHWA-NHI-10-016.

5.6.4 Construction Considerations for Drilled Pier Foundations

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 that 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 or downhole camera, etc. Steel reinforcement should be placed the same day the concrete will be placed. Additionally, drilled pier excavations should 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.

5.7 RETAINING WALLS

Miscellaneous retaining walls may be required for the proposed development. Retaining walls up to 6 feet in height may be designed for lateral earth pressures in accordance with recommendations presented in this section.

5.7.1 Active and At-Rest Pressure

It is recommended the retaining structures be designed for active pressure for cantilever conditions or at-rest pressure if it is braced at the top (as in a roof connection). These earth pressures are presented in Table 5-5 and are expressed in terms of equivalent fluid pressures.

Table 5 5: Lateral Earth Pressures		
Condition	Drained Case[1] (pcf)	Undrained Case (pcf)
Active Case	51	88
At – Rest Case	54	90

[1]Drained case assumes fully drained conditions and level backfill.

No additional surcharge stresses were included in the pressures noted above. Surcharge pressures will depend on the load conditions (i.e., equipment and construction loads such as material or soil stockpiles, and distance from wall where load is applied, etc.). If specific surcharge pressures need to be considered, additional analysis will likely be required with the load conditions given.

In general, walls subject to surcharge loads should be designed for an additional uniform lateral load pressure equal to one-third the anticipated surcharge loads for unrestrained walls and one-half the anticipated surcharge loads for restrained walls. The project engineer should be consulted to confirm applicable values.

5.7.2 Seismic Design for Retaining Walls

The maximum dynamic active pressure is equal to the sum of the initial static pressure and the dynamic (seismic) pressure increment. Under the most recent building code, seismic earth pressure is required in the design of restraining walls which support over 6 feet of earth. The anticipated retaining walls are less than 6 feet in height, and therefore the dynamic earth pressure may be omitted.

5.7.3 Retaining Wall Drainage

Where retaining walls are designed to be drained, drainage may be provided using a 4-inch-diameter perforated pipe embedded in Caltrans Class 2 permeable material, or free-draining gravel surrounded by synthetic filter fabric. The thickness of the drain blanket should be at least 12 inches. As an alternative, prefabricated synthetic wall drain panels may be used. The drain blanket should extend from the bottom of the wall to about two feet below the finished grade at the top of the wall. The upper two feet of wall backfill should consist of onsite compacted soils. Drainage should be collected by a perforated pipe and directed to an outlet approved by the civil engineer. Subdrain pipe, drain blanket and synthetic filter fabric should meet the minimum requirements presented herein. Clay soils should not be utilized as retaining wall backfill.

5.7.4 Wall Backfill

Any required backfill should be mechanically compacted in layers not more than 8 inches thick, to at least 90 percent of the maximum laboratory density obtainable by the most recent revision of ASTM D1557 method of compaction. Flooding is not permitted. Compaction within five feet, measured horizontally, behind a retaining structure should be achieved by use of light weight, hand operated compaction equipment.

Proper compaction of the backfill will be necessary to reduce settlement of overlying walkways and paving. Some settlement of required backfill should be anticipated and any utilities supported therein should be designed to accommodate differential settlement, particularly at points of entry to a structure.

5.8 SPORTS FIELD AND RUNNING TRACK

General grading and drainage recommendations for support of the proposed sports field and running track are provided in the following sections. Specific grading, drainage and aggregate base section details, geogrid, etc., may be specified by the specialty designer and contractor retained for the project provided this firm can review, evaluate, and verify recommendations included therein.

5.8.1 Subgrade Preparation

Subgrade preparation for support of aggregate base associated with sports field and running track should consist of 24 inches of removal and recompaction to create an engineered fill pad for uniform support of the proposed development features. In addition, the engineered fill pad should extend 2 feet beyond the outside limits of the running track. Compacted fill should be placed in loose layers not to exceed 8 inches in thickness and compacted to a minimum of 95 percent relative compaction based on ASTM D1557 specifications. Prior to construction of the engineered fill pad, the subgrade of fill should be scarified 8 inches and compacted to at least 95 percent relative compaction as determined by ASTM D1557. Clayey or expansive soil or geologic material should not be utilized as subgrade support for sports field or running track. Grading should proceed in general accordance with recommendations provided in the Earthwork section of this report.

5.8.2 Impermeable Liner

It is recommended by this firm that the engineered fill pad is overlain by an impermeable liner followed by aggregate base layer (including drainage system constructed within base layer section). Placement of an impermeable liner would serve to protect the underlying engineered fill pad and alluvial soils from saturation which could lead to expansion bulging and ponding at playfield surface.

5.8.3 Drainage Considerations

Drainage of the sports field and running track is anticipated to consist of an aggregate base layer integrated with flat drain collectors or perforated piping as specified by the civil engineer or specialty consultant retained for sports field and track design. Drainage pipes are normally designed so that the drain collectors are placed at the perimeter of the field and lateral piping is installed at an angle across the slope of the field (herringbone pattern). Lateral drains are normally 15 feet apart with a consistent slope gradient to the outlet of approximately 0.5 percent.

5.8.4 Aggregate Base Layer

Aggregate may fragment during transportation and construction operations which may result in higher fines content. In addition, excessive processing of aggregate base including loading, unloading, spreading and compacting, including stockpiling of aggregate can lead to fines migration and settlement. Moisture conditioning of aggregate during transportation and processing can mitigate fines migration and settlement effects. Gradation testing per ASTM C136 may be periodically performed for verification purposes at the discretion of the project team.

Aggregate should be compacted to between 92 and 95 percent relative compaction as determined by ASTM D1557. Note: Under-compaction may lead to unfavorable settlement or ponding and compromise subgrade modulus while over-compaction may cause reduction in infiltration rate. Test strips may be constructed outside the area of planned development for testing purposes to verify gradation and infiltration performance prior to full-scale production grading.

5.8.5 Running Track - Pavement Section Recommendations

Pavement and aggregate base section recommendations may be obtained from section 5.17 “Flexible and Rigid Pavements” as needed based on performance criteria required for running track. Note: A separate synthetic force-reduction and wear layers with enough cross section to shed water should cover the underlying asphalt or concrete (if selected). Proper drainage adjacent to the track is essential to prevent soil settlement due to water penetration below the ground surface. Curbs should extend a minimum of 12 inches below finish grade to serve as a barrier for water migration below the proposed running track.

5.9 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 5-6 presents the seismic design parameters for the site in accordance with the CBC (2022) and ASCE7-16 guidelines using the SEAOC/OSHPD Seismic Design Maps Tool based on Site Class D site classification:

Table 5 6: Seismic Design Criteria per 2022 California Building Code and ASCE 7 16			
Reference		**Seismic Parameter**	**Value**
ASCE 7-16	Google Earth	North Latitude	34.5404
	Google Earth	West Longitude	-117.3409
	Table 20.3-1	Site Class	D
	Table 1.5-1	Risk Category	I, II, III
	Table 11.4-1	Site Coefficient for Short Period, F_A	1.05
	Table 11.4-2	Site Coefficient for Long Period, F_v	Section 11.4.8[1]
	Figure 22-7	Peak Ground Acceleration, PGA	0.483g
	Table 11.8-1	Site Amplification Factor, F_{PGA}	1.117
	Equation 11.8-1	Peak Ground Acceleration, PGA_M	0.539
	Figure 22-1	Mapped MCE_R Spectral Response Acceleration at 0.2-second period, S_s	1.124g
	Figure 22-2	Mapped MCE_R Spectral Response Acceleration at 1.0-second period, S_1	0.436g
	Equation 11.4-1	Site-Adjusted MCE_R Spectral Acceleration at 0.2-second period, S_{MS}	1.181g
	Equation 11.4-2	Site-Adjusted MCE_R Spectral Acceleration at 1.0-second period, S_{M1}	Section 11.4.8[1]
	Equation 11.4-3	Design Spectral Response Acceleration at 0.2-second period, S_{DS}	0.787
	Equation 11.4-4	Design Spectral Response Acceleration at 1.0-second period, S_{D1}	Section 11.4.8[1]
	Table 11.6-1	Seismic Design Category for Short Period Response Acceleration	Section 11.4.8[1]
	Table 11.6-2	Seismic Design Category for 1-s Period Response Acceleration	Section 11.4.8[1]

[1]The values presented in the table assume the structural engineer will utilize the exceptions allowed in 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.

5.10 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 5-7.

Table 5 7: Evaluation for Soil Corrosion		
Corrosion Property	**Test Result**	**Test Method**
Soil pH	7.9	CA DOT Test #643
Minimum Resistivity	2,211 ohm-cm to 13,400 ohm-cm	CA DOT Test #643
Chloride	33.3 ppm	CA DOT Test #422
Sulfate	110.9 ppm	CA DOT Test #417

Based on the Caltrans Highway Design Manual corrosion criteria (Caltrans, 2020, 2022), corrosive soils are defined as soils with an electrical resistivity of 1,500 ohm-cm or less, more than 500 ppm chlorides, more than 0.2 percent sulfates, and a pH less than 5.5. Based on laboratory testing results included herein (Appendix B), the on-site soils would not be classified as corrosive.

Fenagh Engineering and Testing does not practice in the field of corrosion engineering and mitigation. If corrosion sensitive improvements are planned, it is recommended that a corrosion engineer be retained to evaluate corrosion potential at the project site and incorporate the necessary precautions to avoid premature corrosion of buried metal pipes and utilities in direct contact with site soils.

5.11 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 indicated water-soluble sulfate content of 0.0111 percent by weight (110.9 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.

5.12 INTERIOR CONCRETE SLAB-ON-GRADE

It is the understanding of this firm that the proposed modular structure may 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 E1745 Class A requirements. Placement of the vapor retarder and “welding” of overlaps should follow the manufacturer’s guidelines and is the responsibility of the foundation contractor.

Some cracking of the slabs-on-grade may be anticipated at the site because of concrete shrinkage and potential expansive soils which may exist on the site. 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 requires 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 base or concrete slabs-on grade.

5.13 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 ASTM E1745-97, Class A; 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 2022 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.

5.14 EXTERIOR FLATWORK

Exterior slabs should be at least 4 inches thick and placed over a subgrade prepared in accordance with the recommendations of this report. The design professional should 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 12 inches of soil should be prepared per the Earthwork recommendations presented in this report. This may require removal of expansive soils in the upper 12 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.

5.15 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.

5.16 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.

5.17 FLEXIBLE AND RIGID PAVEMENTS

5.17.1 Flexible (Asphalt Concrete) Pavements

Laboratory testing from one (1) bulk soil sample taken from the proposed development area resulted in an R-Value (Resistance Values) of 70. Asphalt and base course materials should meet the requirements of the *Caltrans Standard Specifications, latest edition*. The pavement sections are based on a subgrade R-value equal to 70.

Table 5 8: Recommended Paving Sections

Service Level	Flexible Asphalt Pavement		Rigid Concrete Pavement	
	Asphalt Pavement Thickness (Inches)	Aggregate Base Course (Inches)	Concrete Pavement Thickness (Inches)	Aggregate Base Course (Inches)
Passenger Cars (TI=4)	3	4	6	4
Moderate Trucks (TI=6)	4	6	6	4
Heavy Trucks (TI=8)	5	9	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 (or 24 inches for running track) below the Class II aggregate base and compacted to a minimum 95 percent of ASTM D1557 laboratory maximum dry density at or above 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 always be 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.

5.17.2 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, trash enclosure slabs and possibly running track, we recommend using Portland cement concrete paving as indicated in Table 5.8. 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.

5.17.3 Construction Considerations for Pavements

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

- Aggregate base materials used should comply with the requirements outlined in Section 26 of the State Standard Specifications.
- Aggregate base 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 ASTM D1557 test method.
- Proof rolling of subgrade and of aggregate base 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 aggregate base following final grading. Proof rolling of subgrade should occur immediately (i.e., less than 24 hours) before placement of compacted fill or aggregate base.

5.18 STORMWATER DISPOSAL

Regulatory agencies have been requiring the disposal of a certain amount of stormwater generated on a site by infiltration into the site soils. Increasing the moisture content of a soil can cause it to lose internal shear strength and increase its compressibility, resulting in a change in the designed engineering properties. This means that any overlying structure, including buildings, pavements and concrete flatwork, could sustain damage due to saturation of the subgrade soils. Structures serviced by subterranean levels could be adversely impacted by stormwater disposal by increasing the design fluid pressures on retaining walls and causing leaks in the walls. Proper site drainage is critical to the performance of any structure in the built environment.

5.18.1 Percolation Testing

Borings B-2 (Percolation Test P-1) and B-5 (Percolation Test P-2) were drilled to depths ranging from 8 feet to 11-1/2 feet below the existing site grade with the aid of a truck-mounted drilling machine, equipped with 8-inch diameter hollowstem augers. Fill materials were encountered in the borings to a depth of 4 feet below the existing site grade. The fill is in turn underlain by native alluvial soils, consisting predominantly of Silty Sands and Sands with occasional gravel.

The geologic materials encountered during excavation are presented in Appendix A on the enclosed Boring Logs B-2 and B-5.

At the completion of drilling, a 3-inch diameter casing was placed within the center of the boreholes for the purpose of conducting percolation testing. The casing consisted of perforated PVC pipes to the maximum excavated depth of 8 feet and 11-1/2 feet. Gravel was poured into the annular space around the perforated pipe casing.

After the casing pipes were installed, the boreholes were filled with water for the purpose of pre-soaking for a minimum 24 hours. After presoaking, the boreholes were refilled with water, and the rate of drop in the water level was measured. The percolation test readings were recorded a minimum of 6 times or until a stabilized rate of drop was obtained, whichever occurred first. The percolation testing was performed within the native alluvial soil strata in accordance with County of San Bernardino Guidelines.

Based on results of the percolation testing, an infiltration rate of 22.2 inches per hour may be utilized for design of the proposed deep infiltration dry well systems. No safety factors or reduction factors have been applied to this infiltration rate. The civil engineer should apply the required factors of safety to the infiltration rate provided herein as required.

5.18.2 The Proposed System

The infiltration system is currently anticipated to underlie the playfield area. The client must be aware that repeatedly saturation of onsite soils may potentially cause settlement to occur. The settlement may manifest itself as cracking in the pavement, flatwork and other improvements. These improvements may require increased maintenance and have a shorter design life.

The soils encountered on the site should allow stormwater to infiltrate in a generally vertical manner. Therefore, there is minimal potential for creating a perched water condition. Laboratory testing of onsite soils indicates very low expansion potential. The onsite soils are not susceptible to significant hydro consolidation as indicated by enclosed laboratory testing results. In addition, the development is not located in a hillside area with no nearby slopes. The project will not be serviced by below grade retaining walls. Infiltration into onsite fill materials is not recommended.

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, & 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, including engineering assessment and analyses. The field explorations were located in the field by 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 three 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 or our inability to observe the construction and verify our assumptions.

8 REFERENCES

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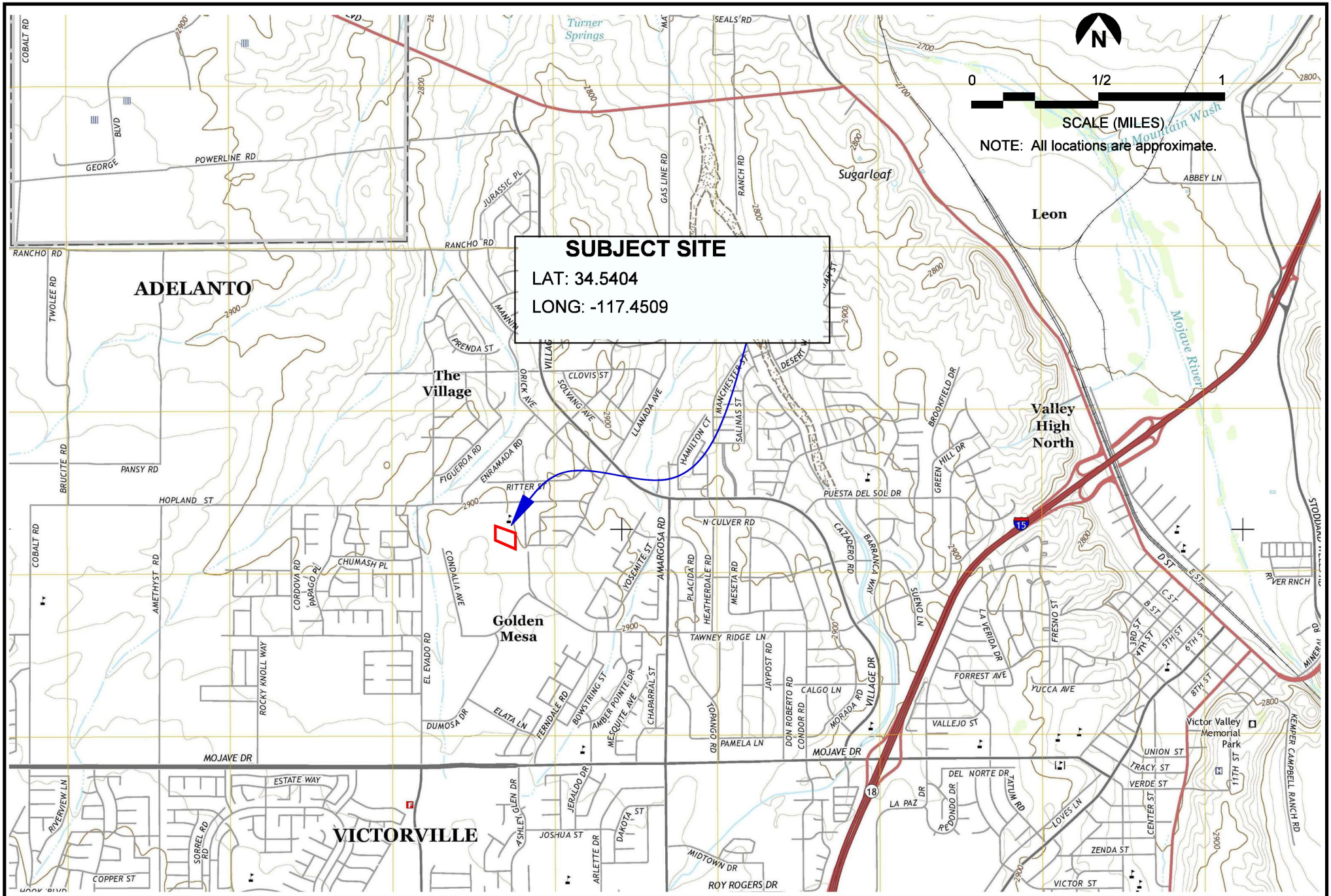
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PLATES

Plate 1: Vicinity Map
Plate 2: Exploration Map – Campus View
Plate 3: Exploration Map – Expanded View
Plate 4: Regional Geology Map
Plate 5: Local Geology Map
Plate 6: Local Fault Map
Plate 7: Seismic Hazard Zone Map
Plate 8: Flood Insurance Rate Map



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VICINITY MAP
CHALLENGER SPORT COMPLEX
VICTORVILLE, CALIFORNIA

REFERENCE: VICTORVILLE QUADRANGLE CALIFORNIA -SAN BERNARDINO CO. 7.5 - MINUTES SERIES

Date: 6/14/24
By: LM
Job No.: J7656

PLATE: 1



LEGEND:

- School Campus Property
- Project Site
- B-1 — Location and Boring Number

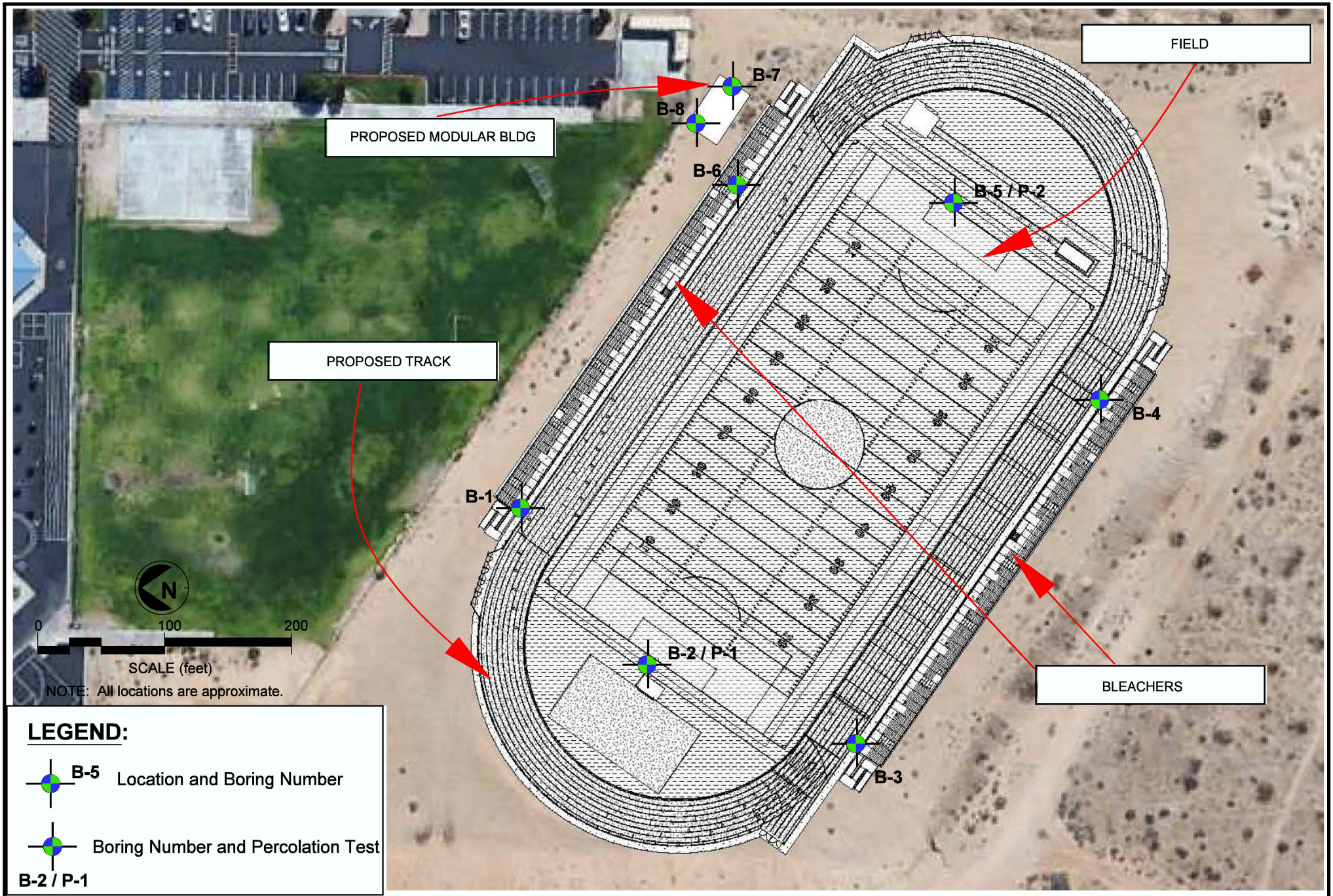
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EXPLORATION MAP - CAMPUS VIEW
CHALLENGER SPORTS COMPLEX
VICTORVILLE, CALIFORNIA

REFERENCE: GOOGLE EARTH

Date: 6/14/2024
By: LM
Job No.: J7656

PLATE: 2



LEGEND:

B-5 Location and Boring Number

Boring Number and Percolation Test

B-2 / P-1

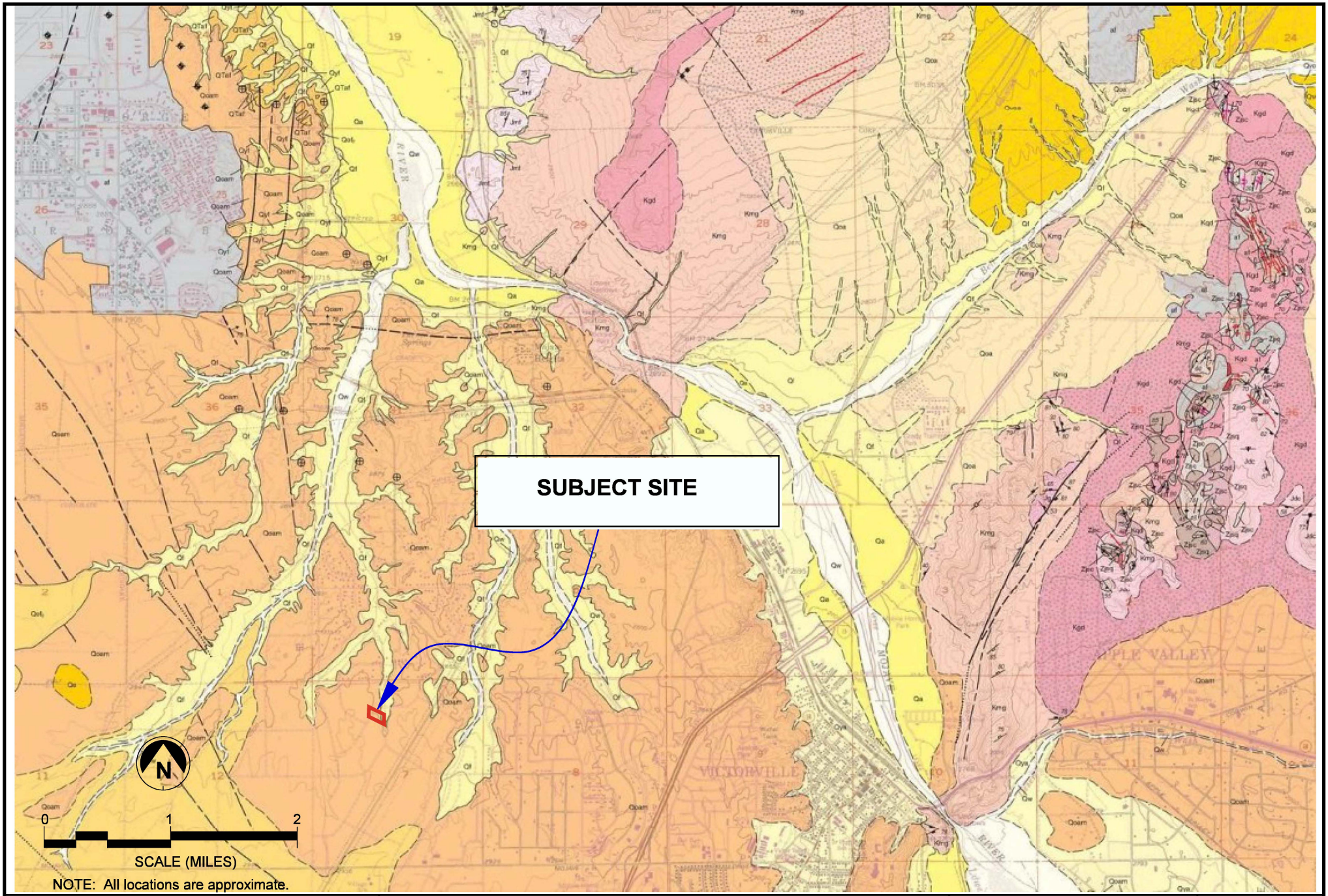
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EXPLORATION MAP - EXPANDED VIEW
CHALLENGER SPORTS COMPLEX
VICTORVILLE, CALIFORNIA

REFERENCE: SITE PLAN PROVIDED BY VEUSD

Date: 6/14/2024
By: LM
Job No.: J7656

PLATE: 3



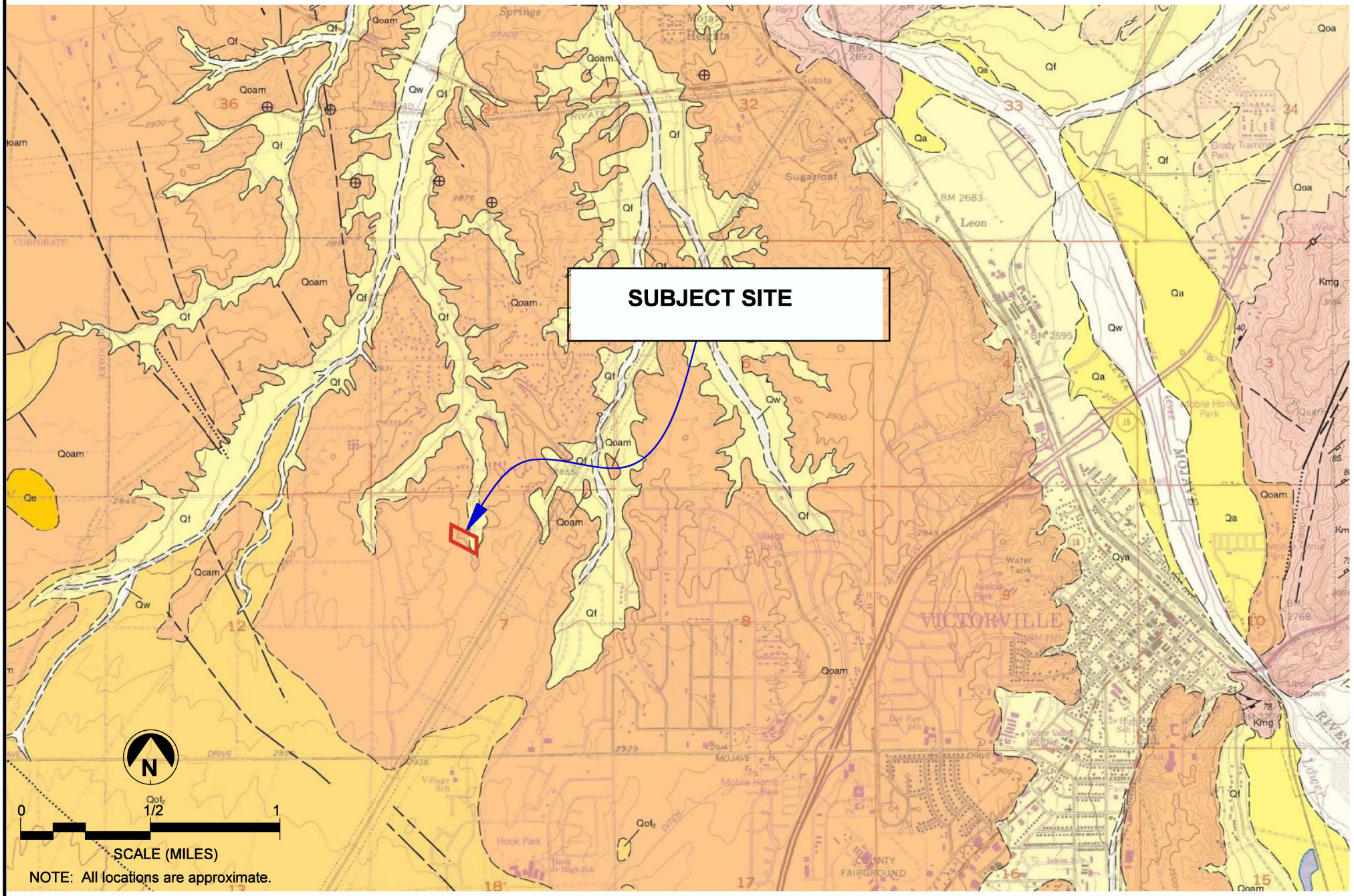
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REGIONAL GEOLOGY MAP
CHALLENGER SPORTS COMPLEX
VICTORVILLE, CALIFORNIA

REFERENCE: GEOLOGIC MAP OF THE VICTORVILLE 7.5' QUADRANGLE SAN BERNARDINO CO., CA

Date: 6/17/2024
By: LM
Job No.: J7656

PLATE: 4



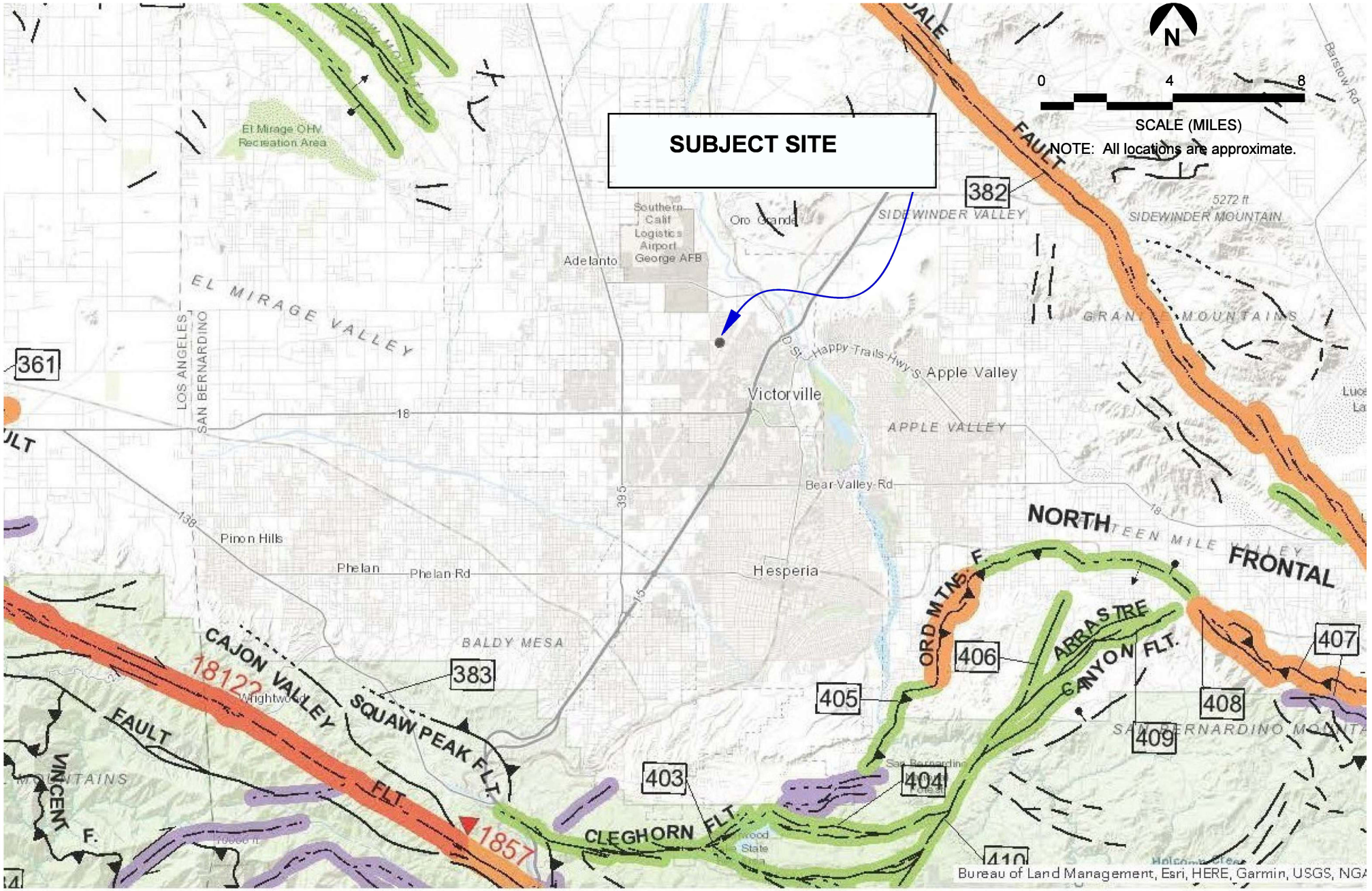
FENAGH
ENGINEERING AND TESTING

LOCAL GEOLOGY MAP
CHALLENGER SPORTS COMPLEX
VICTORVILLE, CALIFORNIA

REFERENCE: GEOLOGIC MAP OF THE VICTORVILLE 7.5' QUADRANGLE SAN BERNARDINO CO., CA

Date: 6/17/2024
By: LM
Job No.: J7656

PLATE:
5



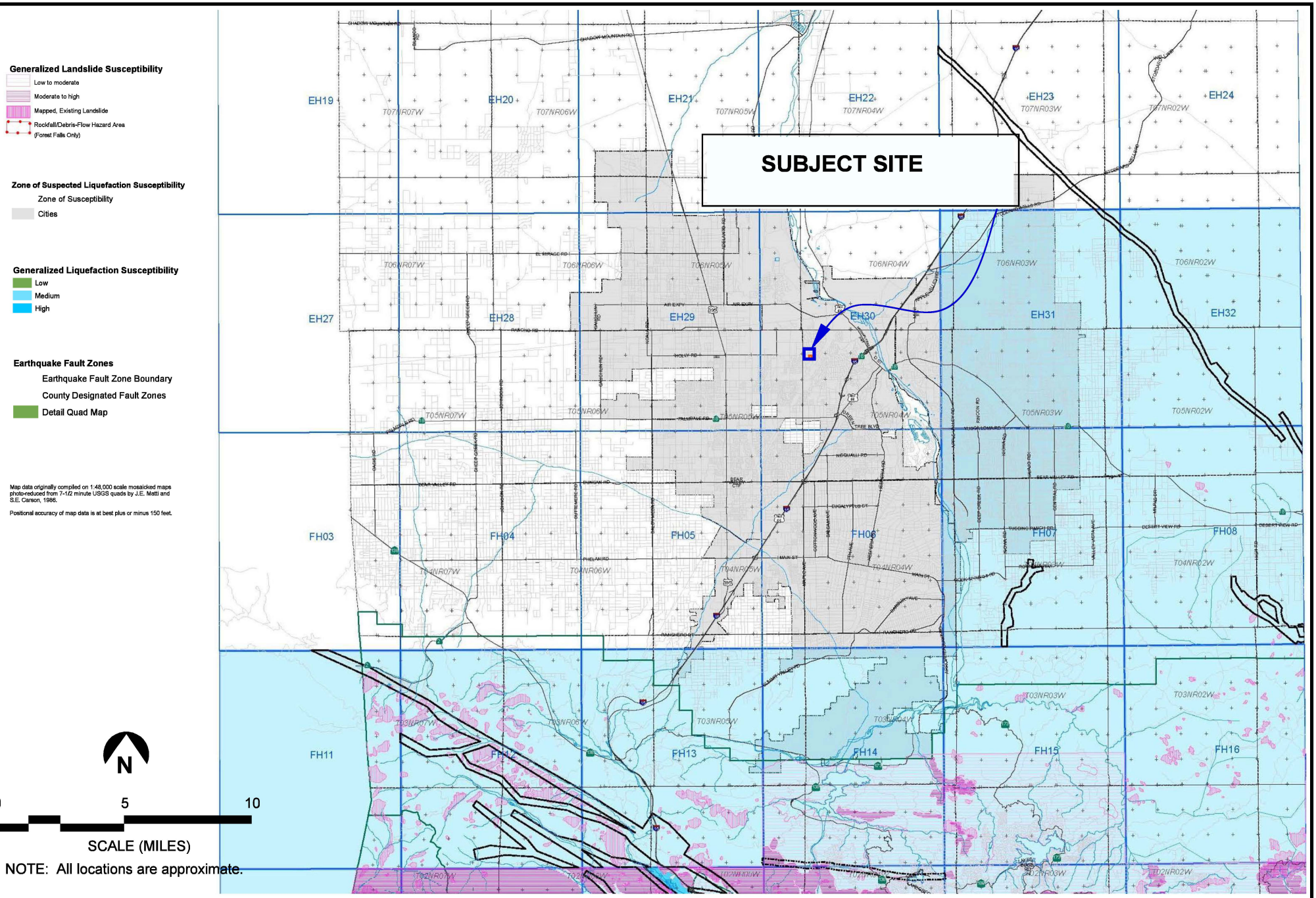
FENAGH
ENGINEERING AND TESTING

LOCAL FAULT MAP
CHALLENGER SPORTS COMPLEX
VICTORVILLE, CALIFORNIA

REFERENCE: FAULT ACTIVITY MAP - CALIFORNIA DEPARTMENT OF CONSERVATION

Date: 6/17/2024
By: LM
Job No.: J7656

PLATE: 6



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ENGINEERING AND TESTING

SEISMIC HAZARD ZONE MAP
CHALLENGER SPORTS COMPLEX
VICTORVILLE, CALIFORNIA

REFERENCE: SAN BERNARDINO CO. LAND USE PLAN GENERAL PLAN GEOLOGIC HAZARD OVERLAYS

Date: 6/17/2024
By: LM
Job No.: J7656

PLATE: 7

117°20'8"W34°32'11"N



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

1:6,000

Legend

SEEFISREPORTFORDETAILEDLEGENDANDINDEXMAPFORFIRMPANELLAYOUT

SPECIALFLOOD HAZARDAREAS
- WithoutBaseFloodElevation(BFE) *ZoneA,V,A99*
- WithBFEorDepth *ZoneAE,AO,AH,VE,AR*
- RegulatoryFloodway

OTHERAREASOF FLOODHAZARD
- 0.2%AnnualChanceFloodHazard,Areas of1%annualchancefloodwithaverage depthlessthanonefootorwithdrainage areasoflessthanonesquaremile *ZoneX*
- FutureConditions1%Annual ChanceFloodHazard *ZoneX*
- AreawithReducedFloodRiskdueto Levee.SeeNotes. *ZoneX*
- AreawithFloodRiskduetoLevee *ZoneD*

OTHERAREAS
- NOSCREEN AreaofMinimalFloodHazard *ZoneX*
- EffectiveLOMRs
- AreaofUndeterminedFloodHazard *ZoneD*

GENERAL STRUCTURES
- Channel,Culvert,orStormSewer
- Levee,Dike,orFloodwall

OTHER FEATURES
- 20.2 CrossSectionswith1%AnnualChance
- 17.5 WaterSurfaceElevation
- 8 CoastalTransect
- 513 BaseFloodElevationLine(BFE)
- LimitofStudy
- JurisdictionBoundary
- CoastalTransectBaseline
- ProfileBaseline
- HydrographicFeature

MAPPANELS
- DigitalDataAvailable
- NoDigitalDataAvailable
- Unmapped

N

Thepindisplayedonthemapisanapproximate pointselectedbytheuseranddoesnotrepresent anauthoritativepropertylocation.

Thismapcomplieswith FEMA'sstandardsfortheuseof digitalfloodmapsifitisnotvoidasdescribedbelow. Thebasemapshowncomplieswith FEMA'sbasemap accuracystandards

Thefloodhazardinformationisderiveddirectlyfromthe authoritativeNFHLwebservicesprovidedbyFEMA.Thismap wasexportedon 6/12/2024at5:17PM anddoesnot reflectchangesoramendmentssubsequenttothisdateand time.TheNFHLandeffectiveinformationmaychangeor becomesupersededbynewdataovertime.

Thismapimageisvoidiftheoneormoreofthefollowingmap elementsdonotappear:basemapimagery,floodzonelabels, legend,scalebar,mapcreationdate,communityidentifiers, FIRMpanelnumber,andFIRMeffectivedate.Mapimagesfor unmappedandunmodernizedareascannotbeusedfor regulatorypurposes.

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FLOOD INSURANCE RATE MAP CHALLENGER SPORTS COMPLEX VICTORVILLE, CALIFORNIA

REFERENCE: FEMA - BASEMAP IMAGERY SOURCE: USGS NATIONAL MAP 2023

Date: 6/17/2024

By: LM

Job No.: J7656

PLATE: 8

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:

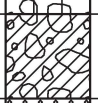
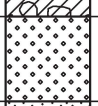
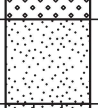
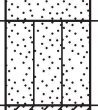
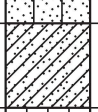
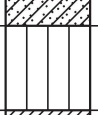
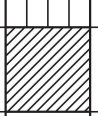
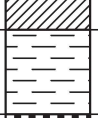
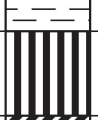
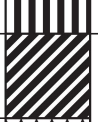
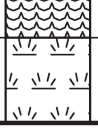
27-7 Drilling, Inc.
1840 Commerce Street
Norco, CA 92860

Borings Logged by:

Luis Mondragon, Fenagh Engineering and Testing

Borings Checked by:

Scott T. Prince, Fenagh Engineering and Testing

MAJOR DIVISIONS					TYPICAL NAMES
COARSE-GRAINED SOILS MORE THAN HALF IS COARSER THAN NO. 200 SIEVE	GRAVELS MORE THAN HALF COARSE FRACTION IS LARGER THAN NO. 4 SIEVE	CLEAN GRAVELS WITH LESS THAN 15% FINES	GW		WELL-GRADED GRAVELS WITH OR WITHOUT SAND
			GP		POORLY GRADED GRAVELS WITH OR WITHOUT SAND
		GRAVELS WITH 15% OR MORE FINES	GM		SILTY GRAVELS WITH OR WITHOUT SAND
			GC		CLAYEY GRAVELS WITH OR WITHOUT SAND
	SANDS MORE THAN HALF COARSE FRACTION IS FINER THAN NO. 4 SIEVE SIZE	CLEAN SANDS WITH LESS THAN 15% FINES	SW		WELL-GRADED SANDS WITH OR WITHOUT GRAVEL
			SP		POORLY GRADED SANDS WITH OR WITHOUT GRAVEL
		SANDS WITH 15% OR MORE FINES	SM		SILTY SANDS WITH OR WITHOUT GRAVEL
			SC		CLAYEY SANDS WITH OR WITHOUT GRAVEL
FINE-GRAINED SOILS MORE THAN HALF IS FINER THAN NO. 200 SIEVE	SILTS AND CLAYS LIQUID LIMIT 50% OR LESS		ML		INORGANIC SILTS OF LOW TO MEDIUM PLASTICITY WITH OR WITHOUT SAND OR GRAVEL
			CL		INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY WITH OR WITHOUT SAND OR GRAVEL
			OL		ORGANIC SILTS OR CLAYS OF LOW TO MEDIUM PLASTICITY WITH OR WITHOUT SAND OR GRAVEL
	SILTS AND CLAYS LIQUID LIMIT GREATER THAN 50%		MH		INORGANIC SILTS OF HIGH PLASTICITY WITH OR WITHOUT SAND OR GRAVEL
			CH		INORGANIC CLAYS OF HIGH PLASTICITY WITH OR WITHOUT SAND OR GRAVEL
			OH		ORGANIC SILTS OR CLAYS OF HIGH PLASTICITY WITH OR WITHOUT SAND OR GRAVEL
HIGHLY ORGANIC SOILS			PT		PEAT AND OTHER HIGHLY ORGANIC SOILS

SAMPLE SYMBOLS

SAMPLE TYPES

- Bulk Sample
- Modified California Sampler
- Standard Penetration Test

LABORATORY SYMBOLS

AL - ATTERBERG LIMITS
CON - CONSOLIDATION
COR - CORROSIVITY TESTING
DS - DIRECT SHEAR
EI - EXPANSION INDEX
MAX - MAXIMUM DENSITY
RV - R-VALUE
PD - PARTICLE-SIZE DISTRIBUTION
UC - UNCONFINED COMPRESSIVE STRENGTH
WA - No. 200 WASH (% PASSING No. 200 SIEVE)

GROUNDWATER SYMBOLS

WATER LEVEL AT TIME OF EXCAVATION OR AS INDICATED

RELATIVE DENSITY OF COHESIONLESS SOILS		CONSISTENCY OF COHESIVE SOILS		
RELATIVE DENSITY	SPT N60 BLOWS/FOOT	CONSISTENCY	SPT N60 BLOWS/FOOT	POCKET PENETROMETER MEASUREMENT (TSF)
VERY LOOSE	0 - 4	VERY SOFT	0 - 2	0 - 0.25
LOOSE	4 - 10	SOFT	2 - 4	0.25 - 0.50
MEDIUM DENSE	10 - 30	MEDIUM STIFF	4 - 8	0.50 - 1.0
DENSE	30 - 50	STIFF	8 - 15	1.0 - 2.0
VERY DENSE	OVER 50	VERY STIFF	15 - 30	2.0 - 4.0
		HARD	OVER 30	OVER 4.0

NUMBER OF BLOWS OF 140 LB HAMMER FALLING 30 INCHES TO DRIVE A 2 INCH O.D. (1-3/8 INCH I.D.) SPLIT-BARREL SAMPLER THE LAST 12 INCHES OF AN 18-INCH DRIVE (ASTM-1586 STANDARD PENETRATION TEST).
IF THE SEATING INTERVAL (1st 6 INCH INTERVAL) IS NOT ACHEIVED, N IS REPORTED AS REF.

FENAGH ENGINEERING AND TESTING
Fenagh Engineering & Testing
9070 Center Avenue
Rancho Cucamonga, CA 91730

SUBSURFACE EXPLORATION LEGEND

BORING NUMBER B-1

Fenagh Engineering

CLIENT Victorville Elementary School District
PROJECT NAME Challenger Sports Complex
PROJECT NUMBER 7656
PROJECT LOCATION Victorville, CA
DATE STARTED 4/30/2024 **COMPLETED** 4/30/2024
GROUND ELEVATION _____ **HOLE SIZE** 8 inches
DRILLING CONTRACTOR 24-7 Drilling
GROUND WATER LEVELS:
DRILLING METHOD Hollow Stem Auger
AT TIME OF DRILLING ---
LOGGED BY LM **CHECKED BY** SP
AT END OF DRILLING ---
NOTES _____
AFTER DRILLING ---

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	POCKET PEN (tsf)	BLOW COUNTS (N VALUE)	UNCONFINED STRENGTH (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	ATTERBERG LIMITS: PLASTIC LIMIT	ATTERBERG LIMITS: PLASTICITY INDEX	FINES CONTENT (%)
0		FILL (Qaf): Poorly Graded SAND with Silt (SP-SM) light brown, dry, medium dense, fine to coarse grained, trace of gravel										
5		ALLUVIUM (Qf): Poorly Graded SAND with Gravel (SP), brown, slightly moist, very dense, fine to coarse grained			15-30-45 (75)		121.7	3.0				
10		increase in coarse content			50/5"		127.2	1.6				
15		Silty SAND (SM), light brown, moist, dense, fine grained			8-17-30 (47)		109.5	22.3				
20												

(Continued Next Page)

BORING LOG WITH ELEVATIONS - GINT STD US LAB.GDT - 6/19/24 11:37 - E:\7656 CHALLENGER\7656 - CHALLENGER SPORTS COMPLEX.GPJ

BORING NUMBER B-1

Fenagh Engineering

CLIENT Victorville Elementary School District **PROJECT NAME** Challenger Sports Complex

PROJECT NUMBER 7656 **PROJECT LOCATION** Victorville, CA

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	POCKET PEN (tsf)	BLOW COUNTS (N VALUE)	UNCONFINED STRENGTH (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	ATTERBERG LIMITS: PLASTIC LIMIT	ATTERBERG LIMITS: PLASTICITY INDEX	FINES CONTENT (%)
20		Poorly Graded SAND with Silt (SP-SM), gray, moist, very dense, fine to medium grained			8-23-35 (58)		115.2	6.0				
25		decrease in moisture, increase in coarse content			10-29-45 (74)		109.6	2.6				

Bottom of borehole at 26.5 feet.

BORING LOG WITH ELEVATIONS - GINT STD US LAB.GDT - 6/19/24 11:37 - E:\7656 CHALLENGER\7656 - CHALLENGER SPORTS COMPLEX.GPJ

BORING NUMBER B-2

Fenagh Engineering

CLIENT Victorville Elementary School District
PROJECT NAME Challenger Sports Complex
PROJECT NUMBER 7656
PROJECT LOCATION Victorville, CA
DATE STARTED 4/30/2024 **COMPLETED** 4/30/2024
GROUND ELEVATION _____ **HOLE SIZE** 8 inches
DRILLING CONTRACTOR 24-7 Drilling
GROUND WATER LEVELS:
DRILLING METHOD Hollow Stem Auger
AT TIME OF DRILLING ---
LOGGED BY LM **CHECKED BY** SP
AT END OF DRILLING ---
NOTES Boring Hole converted to percolation test
AFTER DRILLING ---

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	POCKET PEN (tsf)	BLOW COUNTS (N VALUE)	UNCONFINED STRENGTH (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	ATTERBERG LIMITS: PLASTIC LIMIT	ATTERBERG LIMITS: PLASTICITY INDEX	FINES CONTENT (%)
0		FILL (Qaf): Poorly Graded SAND with Silt (SP-SM), light brown, dry, medium dense, fine to coarse grained, trace of gravel										
5		ALLUVIUM (Qf): Poorly Graded SAND with Gravel (SP), light brown, slighlty moist, medium dense, fine to coarse grained,			1-3-8 (11)			3.2				
		increase in gravel and rock content										
10		very dense			6-28-32 (60)			2.5				

Bottom of borehole at 11.5 feet.

BORING LOG WITH ELEVATIONS - GINT STD US LAB.GDT - 6/19/24 11:37 - E:\7656 CHALLENGER\7656 - CHALLENGER SPORTS COMPLEX.GPJ

BORING NUMBER B-3

Fenagh Engineering

CLIENT Victorville Elementary School District
PROJECT NAME Challenger Sports Complex
PROJECT NUMBER 7656
PROJECT LOCATION Victorville, CA
DATE STARTED 4/30/2024 **COMPLETED** 4/30/2024
GROUND ELEVATION _____ **HOLE SIZE** 8 inches
DRILLING CONTRACTOR 24-7 Drilling
GROUND WATER LEVELS:
DRILLING METHOD Hollow Stem Auger
AT TIME OF DRILLING ---
LOGGED BY LM **CHECKED BY** SP
AT END OF DRILLING ---
NOTES
AFTER DRILLING ---

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	POCKET PEN (tsf)	BLOW COUNTS (N VALUE)	UNCONFINED STRENGTH (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	ATTERBERG LIMITS: PLASTIC LIMIT	ATTERBERG LIMITS: PLASTICITY INDEX	FINES CONTENT (%)
0		FILL (Qaf): Silty SAND (SM), brown, dry, medium dense, fine to coarse grained										
5		ALLUVIUM (Qf): Poorly Graded SAND with Gravel (SP), light brown to tan, slightly moist, dense, medium to coarse grained			7-14-25 (39)		115.4	3.1				
		increase in gravel content										
10					9-50/6"		106.3	3.7				
15		dense, decrease in moisture content			9-20-23 (43)		111.9	3.2				
		layer of fine sand										
20												

(Continued Next Page)

BORING LOG WITH ELEVATIONS - GINT STD US LAB.GDT - 6/19/24 11:37 - E:\7656 CHALLENGER\7656 - CHALLENGER SPORTS COMPLEX.GPJ

BORING NUMBER B-3

FENAGH Fenagh Engineering

CLIENT Victorville Elementary School District **PROJECT NAME** Challenger Sports Complex

PROJECT NUMBER 7656 **PROJECT LOCATION** Victorville, CA

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	POCKET PEN (tsf)	BLOW COUNTS (N VALUE)	UNCONFINED STRENGTH (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS			FINES CONTENT (%)
									LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX	
20												
		Poorly Graded SAND with Silt (SP-SM), light brown, moist, dense, fine grained			5-12-25 (37)		115.1	10.8				

Bottom of borehole at 21.5 feet.

BORING LOG WITH ELEVATIONS - GINT STD US LAB.GDT - 6/19/24 11:37 - E:\7656 CHALLENGER\7656 - CHALLENGER SPORTS COMPLEX.GPJ

BORING NUMBER B-4

Fenagh Engineering

CLIENT Victorville Elementary School District
PROJECT NAME Challenger Sports Complex
PROJECT NUMBER 7656
PROJECT LOCATION Victorville, CA
DATE STARTED 4/30/2024 **COMPLETED** 4/30/2024
GROUND ELEVATION _____ **HOLE SIZE** 8 inches
DRILLING CONTRACTOR 24-7 Drilling
GROUND WATER LEVELS:
DRILLING METHOD Hollow Stem Auger
AT TIME OF DRILLING ---
LOGGED BY LM **CHECKED BY** SP
AT END OF DRILLING ---
NOTES
AFTER DRILLING ---

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	POCKET PEN (tsf)	BLOW COUNTS (N VALUE)	UNCONFINED STRENGTH (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	ATTERBERG LIMITS: PLASTIC LIMIT	ATTERBERG LIMITS: PLASTICITY INDEX	FINES CONTENT (%)
0		FILL (Qaf): Silty SAND (SM), light brown, slightly moist, dense, fine to coarse grained, trace of gravel										
		ALLUVIUM (Qf): Poorly Graded SAND with Gravel (SP), light brown, dry, medium dense, fine to coarse grained,			4-7-15 (22)			2.1				
5		increase in gravel content										
		slight difficulty drilling										
10		dense			14-18-21 (39)		117.8	1.7				
15		Silty SAND (SM), light brown, moist, dense, fine grained			11-19-19 (38)			9.7				
20												

BORING LOG WITH ELEVATIONS - GINT STD US LAB.GDT - 6/19/24 11:37 - E:\7656 CHALLENGER\7656 - CHALLENGER SPORTS COMPLEX.GPJ

(Continued Next Page)

BORING NUMBER B-4

Fenagh Engineering

CLIENT Victorville Elementary School District **PROJECT NAME** Challenger Sports Complex

PROJECT NUMBER 7656 **PROJECT LOCATION** Victorville, CA

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	POCKET PEN (tsf)	BLOW COUNTS (N VALUE)	UNCONFINED STRENGTH (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	ATTERBERG LIMITS: PLASTIC LIMIT	ATTERBERG LIMITS: PLASTICITY INDEX	FINES CONTENT (%)
20		Poorly Graded SAND with SILT (SP-SM), tan to light brown, moist, medium dense, fine grained			2-10-19 (29)		103.4	9.8				
25		medium to fine grained, increase in coarse content			7-13-13 (26)			8.4				

Bottom of borehole at 26.5 feet.

BORING LOG WITH ELEVATIONS - GINT STD US LAB.GDT - 6/19/24 11:37 - E:\7656 CHALLENGER\7656 - CHALLENGER SPORTS COMPLEX.GPJ

BORING NUMBER B-5

Fenagh Engineering

CLIENT Victorville Elementary School District
PROJECT NAME Challenger Sports Complex
PROJECT NUMBER 7656
PROJECT LOCATION Victorville, CA
DATE STARTED 4/30/2024 **COMPLETED** 4/30/2024
GROUND ELEVATION _____ **HOLE SIZE** 8 inches
DRILLING CONTRACTOR 24-7 Drilling
GROUND WATER LEVELS:
DRILLING METHOD Hollow Stem Auger
AT TIME OF DRILLING ---
LOGGED BY LM **CHECKED BY** SP
AT END OF DRILLING ---
NOTES Boring Hole converted to percolation test
AFTER DRILLING ---

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	POCKET PEN (tsf)	BLOW COUNTS (N VALUE)	UNCONFINED STRENGTH (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	ATTERBERG LIMITS: PLASTIC LIMIT	ATTERBERG LIMITS: PLASTICITY INDEX	FINES CONTENT (%)
0		FILL (Qaf): Poorly Graded SAND with Silt (SP-SM), brown, moist, dense, fine to medium grained										
5		ALLUVIUM (Qf): Poorly Graded SAND with Gravel (SP), light brown, slightly moist, very dense, medium to coarse grained			6-35-20 (55)		110.5	1.8				

Bottom of borehole at 8.0 feet.

BORING LOG WITH ELEVATIONS - GINT STD US LAB.GDT - 6/19/24 11:37 - E:\7656 CHALLENGER\7656 - CHALLENGER SPORTS COMPLEX.GPJ

BORING NUMBER B-6

Fenagh Engineering

CLIENT Victorville Elementary School District
PROJECT NAME Challenger Sports Complex
PROJECT NUMBER 7656
PROJECT LOCATION Victorville, CA
DATE STARTED 5/1/2024 **COMPLETED** 5/1/2024
GROUND ELEVATION _____ **HOLE SIZE** 8 inches
DRILLING CONTRACTOR 24-7 Drilling
GROUND WATER LEVELS:
DRILLING METHOD Hollow Stem Auger
AT TIME OF DRILLING ---
LOGGED BY LM **CHECKED BY** SP
AT END OF DRILLING ---
NOTES _____
AFTER DRILLING ---

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	POCKET PEN (tsf)	BLOW COUNTS (N VALUE)	UNCONFINED STRENGTH (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	ATTERBERG LIMITS: PLASTIC LIMIT	ATTERBERG LIMITS: PLASTICITY INDEX	FINES CONTENT (%)
0		FILL (Qaf): Silty SAND (SM), brown, dry, medium dense, fine to coarse grained, trace of gravel										
5		ALLUVIUM (Qf): Poorly Graded SAND with Gravel (SP), light brown, slightly moist, medium dense, fine to coarse grained			3-4-10 (14)			2.5				6.9
		difficulty drilling										
		increase in coarse content			10-15-18 (33)		126.8	4.1				
10		Silty SAND (SM), light brown, slightly moist, dense, fine grained			11-15-19 (34)			13.8				40.5
		very dense			20-35-50 (85)		112.2	4.9				
15		medium dense			7-9-10 (19)			19.6				50.6
		layer of poorly graded Sand with silt, very dense			12-25-50 (75)		122.2	6.4				
20												

(Continued Next Page)

BORING LOG WITH ELEVATIONS - GINT STD US LAB.GDT - 6/19/24 11:37 - E:\7656 CHALLENGER\7656 - CHALLENGER SPORTS COMPLEX.GPJ

BORING NUMBER B-6

Fenagh Engineering

CLIENT Victorville Elementary School District **PROJECT NAME** Challenger Sports Complex

PROJECT NUMBER 7656 **PROJECT LOCATION** Victorville, CA

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	POCKET PEN (tsf)	BLOW COUNTS (N VALUE)	UNCONFINED STRENGTH (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	ATTERBERG LIMITS: PLASTIC LIMIT	ATTERBERG LIMITS: PLASTICITY INDEX	FINES CONTENT (%)
20		Poorly Graded SAND with Gravel (SP), light brown, moist, dense, fine to medium grained			12-20-21 (41)			6.3				11.9
					13-32-50 (82)		110.2	1.8				
25		Sandy SILT (ML), light brown, moist, medium dense, fine grained			5-7-11 (18)			26.6				94.9
					5-16-20 (36)		112.1	20.2				
30		Silty SAND (SM), light brown, moist, medium dense, fine to medium grained, trace of clay			10-8-21 (29)			14.7				42.1
					17-33-50 (83)		118.8	2.1				
35		Poorly Graded SAND with Gravel (SP), light gray, dry, very dense, fine to coarse grained			21-25-40 (65)			1.7				7.1
		increase in coarse content, medium dense			10-12-14 (26)		109.7	21.3				
40												

(Continued Next Page)

BORING LOG WITH ELEVATIONS - GINT STD US LAB.GDT - 6/19/24 11:37 - E:\7656 CHALLENGER\7656 - CHALLENGER SPORTS COMPLEX.GPJ

BORING NUMBER B-6

Fenagh Engineering

CLIENT Victorville Elementary School District **PROJECT NAME** Challenger Sports Complex

PROJECT NUMBER 7656 **PROJECT LOCATION** Victorville, CA

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	POCKET PEN (tsf)	BLOW COUNTS (N VALUE)	UNCONFINED STRENGTH (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	ATTERBERG LIMITS: PLASTIC LIMIT	ATTERBERG LIMITS: PLASTICITY INDEX	FINES CONTENT (%)
40		Silty SAND (SM), brown, slightly moist, medium dense, fine to medium grained			4-5-7 (12)			21.3				70.7
		dense			13-23-25 (48)		112.5	14.2				
45		thin layer of clay			20-21-12 (33)			16.7				18.6
					35-50/6"		128.1	6.0				
50		Well Graded Sand with Gavel (SW), light brown, dry, dense, fine to coarse grained,			15-24-26 (50)			3.4				8.0

Bottom of borehole at 51.5 feet.

BORING LOG WITH ELEVATIONS - GINT STD US LAB.GDT - 6/19/24 11:37 - E:\7656 CHALLENGER\7656 - CHALLENGER SPORTS COMPLEX.GPJ

BORING NUMBER B-7

Fenagh Engineering

CLIENT Victorville Elementary School District
PROJECT NAME Challenger Sports Complex
PROJECT NUMBER 7656
PROJECT LOCATION Victorville, CA
DATE STARTED 5/1/2024 **COMPLETED** 5/1/2024
GROUND ELEVATION _____ **HOLE SIZE** 8 inches
DRILLING CONTRACTOR _____
GROUND WATER LEVELS:
DRILLING METHOD Hand Auger
AT TIME OF DRILLING ---
LOGGED BY LM **CHECKED BY** SP
AT END OF DRILLING ---
NOTES _____
AFTER DRILLING ---

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	POCKET PEN (tsf)	BLOW COUNTS (N VALUE)	UNCONFINED STRENGTH (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	ATTERBERG LIMITS: PLASTIC LIMIT	ATTERBERG LIMITS: PLASTICITY INDEX	FINES CONTENT (%)
0		FILL (Qaf): Silty SAND (SM), light brown, slightly moist, medium dense, fine to medium grained										
		ALLUVIUM (Qf): Silty SAND (SM), light brown, moist, medium dense, medium to fine grained,										
5		Poorly Graded SAND with Gravel (SP), tan to reddish brown, slightly moist, loose, fine to coarse grained										
		medium dense, increase in moisture content, increase in gravel content										
10												

Bottom of borehole at 10.0 feet.

BORING LOG WITH ELEVATIONS - GINT STD US LAB.GDT - 6/19/24 11:37 - E:\7656 CHALLENGER\7656 - CHALLENGER SPORTS COMPLEX.GPJ

BORING NUMBER B-8

Fenagh Engineering

CLIENT Victorville Elementary School District
PROJECT NAME Challenger Sports Complex
PROJECT NUMBER 7656
PROJECT LOCATION Victorville, CA
DATE STARTED 5/1/2024 **COMPLETED** 5/1/2024
GROUND ELEVATION _____ **HOLE SIZE** 8 inches
DRILLING CONTRACTOR 24-7 Drilling
GROUND WATER LEVELS:
DRILLING METHOD Hollow Stem Auger
AT TIME OF DRILLING ---
LOGGED BY LM **CHECKED BY** SP
AT END OF DRILLING ---
NOTES _____
AFTER DRILLING ---

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	POCKET PEN (tsf)	BLOW COUNTS (N VALUE)	UNCONFINED STRENGTH (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	ATTERBERG LIMITS: PLASTIC LIMIT	ATTERBERG LIMITS: PLASTICITY INDEX	FINES CONTENT (%)
0		FILL (Qaf): Silty SAND (SM), light brown, slightly moist, dense, fine to coarse grained										
5		ALLUVIUM (Qf): Poorly Graded SAND with Gravel (SP), light brown, moist, medium dense, fine to coarse grained			13-13-17 (30)		109.8	3.2				
10		Silty SAND (SM), reddish brown, moist, medium dense, fine to mediun grained			10-12-12 (24)			12.5				
15		Poorly Graded SAND with Silt (SP-SM), light brown, moist, medium dense, fine to medium grained			6-11-14 (25)		102.4	25.7				
20												

(Continued Next Page)

BORING LOG WITH ELEVATIONS - GINT STD US LAB.GDT - 6/19/24 11:37 - E:\7656 CHALLENGER\7656 - CHALLENGER SPORTS COMPLEX.GPJ

BORING NUMBER B-8

Fenagh Engineering

CLIENT Victorville Elementary School District **PROJECT NAME** Challenger Sports Complex

PROJECT NUMBER 7656 **PROJECT LOCATION** Victorville, CA

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	POCKET PEN (tsf)	BLOW COUNTS (N VALUE)	UNCONFINED STRENGTH (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	ATTERBERG LIMITS: PLASTIC LIMIT	ATTERBERG LIMITS: PLASTICITY INDEX	FINES CONTENT (%)
20		Poorly Graded SAND with Silt (SP-SM), light brown, moist, dense, fine to medium grained *(continued)*			12-19-25 (44)			4.0				
		light gray, dry, dense, decrease in fine content										

Bottom of borehole at 21.5 feet.

BORING LOG WITH ELEVATIONS - GINT STD US LAB.GDT - 6/19/24 11:37 - E:\7656 CHALLENGER\7656 - CHALLENGER SPORTS COMPLEX.GPJ

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.

Amount of Material Finer than 75-µm
Sieve testing (No. 200 Sieve) was performed on selected representative soil samples in general accordance with ASTM DD1140. Test results are indicated on the boring logs provided in Appendix A.

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.

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.

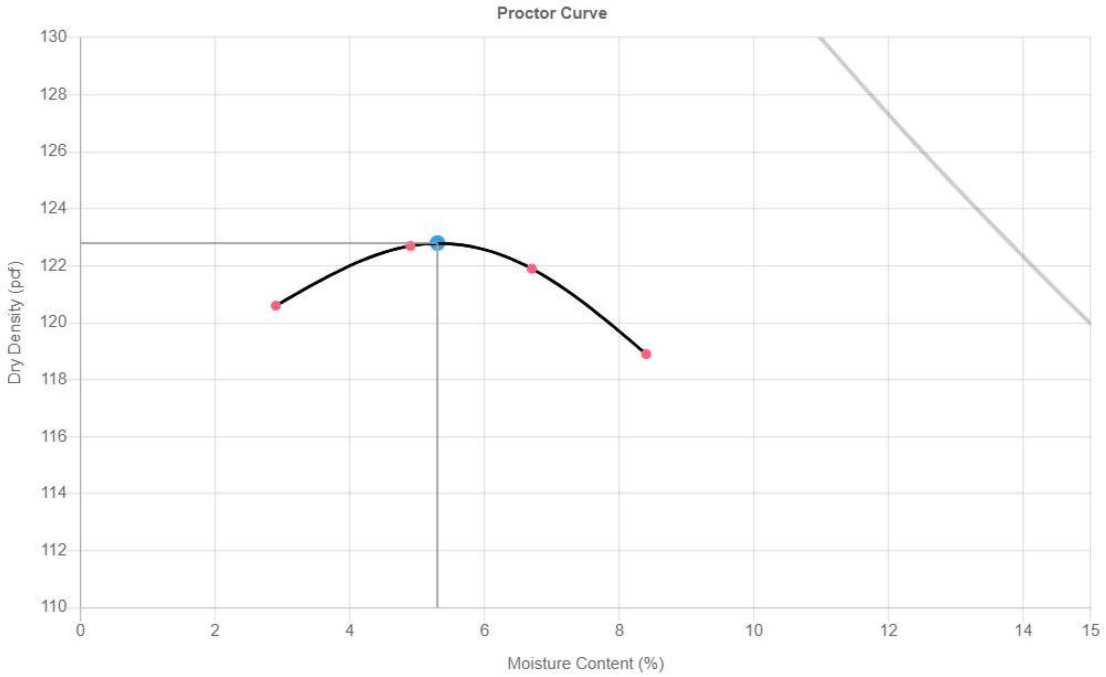
Expansion Index
Expansion Index testing was performed on selected representative soil samples in general accordance with ASTM D4829.

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

TESTING
INSPECTION
ENGINEERING
Rancho Cucamonga Lab: 9070 Center Ave, Rancho Cucamonga, CA 91730

Soils Test Report

Project Name: Sports Complex Project at Challenger Elementary School GI	**Place of Sampling :** B-8 at 0'-5'	**Permit # :**
Project No.:7656	**Sampled by:** Edgar Robles	**Report Date:** 05/17/2024
Client: Victor Elementary School District	**Date of Sampling:** 05/01/2024	**Lab Log No.:** 40127 (S-1351)



Max Density
122.8 pcf @ 5.30%

As Received Moisture
2.9%

Source
B-8 0'-5'

Color
Tan

Description
Sand with some Gravel

Specific Gravity
2.70 Estimated

Sample ID
S-1351

Date Received: 05/01/2024

Date Tested: 05/17/2024

Tested by: Mark Tabb

Desc of Rammer: Mechanical

Material: Sand with some Gravel

Preparation Method: Moist

Remarks: Received Sample Condition: Acceptable and/or Satisfactory

Reviewed by: Magdalena Acevedo, Staff Engineer

05/17/2024
Date

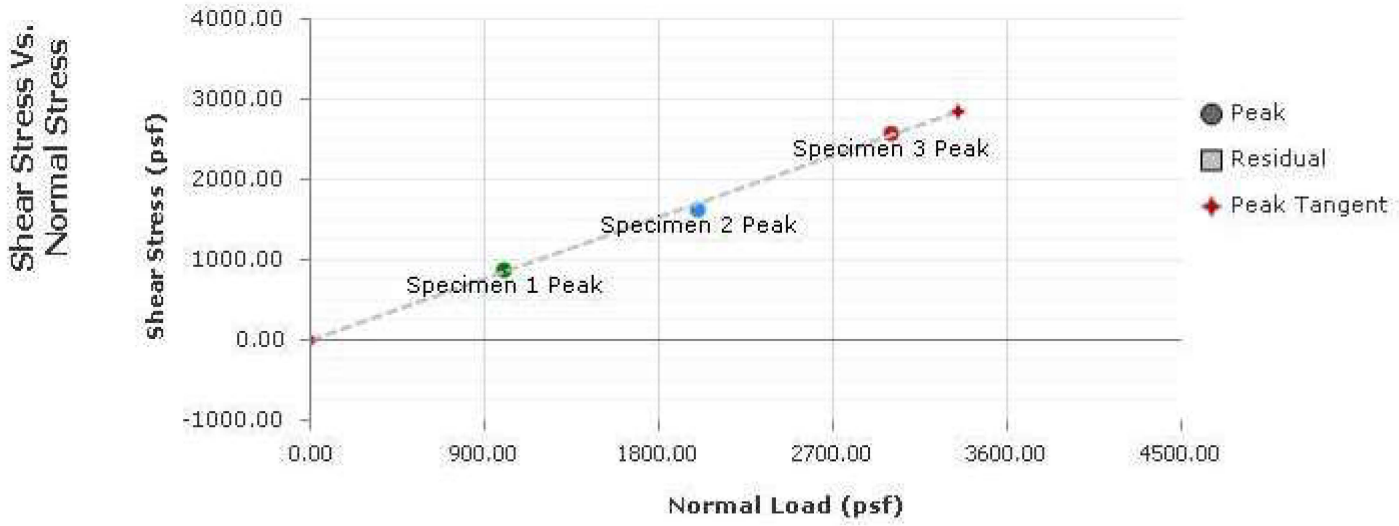
ASTM Standards Used: Unless otherwise noted ASTM D-1557, C-136, D4318, D2216, D1140,Method-C

Testing was performed by qualifed personnel in accordance with generally accepted industry practice, material testing consultants procedures and the above reference standards. This report is applicable only to the items listed herein. The test performed and in this report are not intended to be considered as any guarantee or warranty of suitability for service or fitness of use of items tested and it should not be relied on as such. The report has been prepared for the exclusive use of the client and any partial or whole reproduction without the consent of the client is prohibited.

Direct Shear Test

ASTM D3080

Project: Challenger Sports Complex
Project Number: J7656
Location: B-8 at 0'-5'
Client Name: Victor Elementary School District



C (psf): -11.66
Phi (°): 40.4

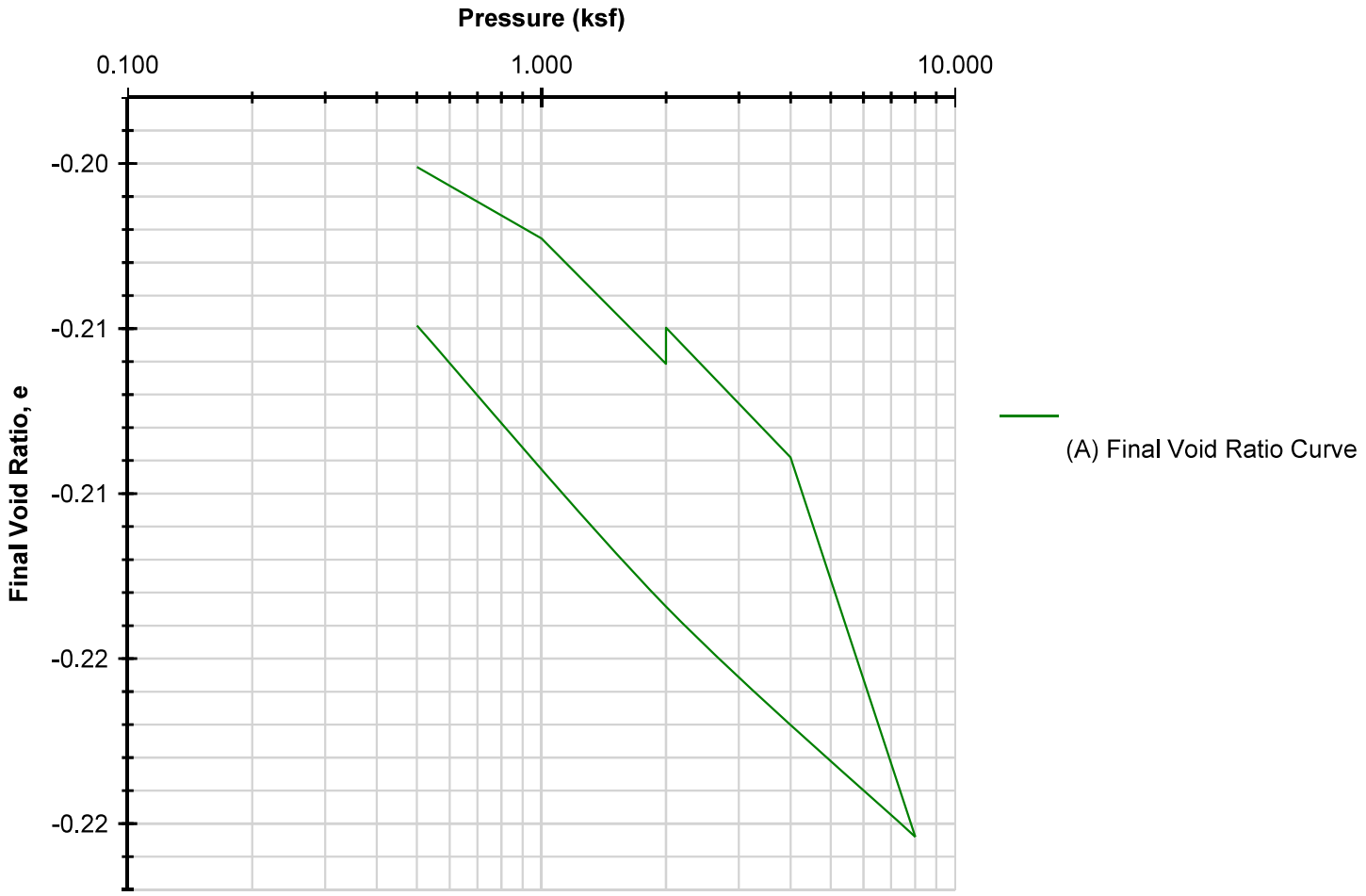
Residual C (psf): NA
Residual Phi (°): NA

	Specimen Number							
Initial	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**
Moisture (%):	5.3	5.3	5.3					
Dry Density (pcf):	109.8	109.8	109.8					
Void Ratio:	0.546	0.546	0.546					
Saturation (%):	26.2	26.2	26.2					
Diameter (in):	2.4200	2.4200	2.4200					
Height (in):	1.0000	1.0000	1.0000					
Final	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**
Moisture (%):	17.3	15.4	15.7					
Dry Density (pcf):	109.2	109.3	109.7					
Void Ratio:	0.556	0.553	0.549					
Saturation (%):	84.9	75.7	77.7					
Height (in):	0.9978	0.9985	0.9978					
Normal Stress (psf):	1000.00	2000.00	3000.00					
Peak Shear Stress (psf):	875.44	1622.82	2579.79					
Residual Stress (psf):	NA	NA	NA					
Horizontal Deformation (%):	1.9	1.5	1.7					
Rate (in/min):	0.005000	0.005000	0.005000					

Checked By: _____ Date: _06/13/24__

Final Voids [Log]

ASTM D2435



Preconsolidation Stress (ksf)	NA	**Cc** 0.026	**Cr** 0.010

	BEFORE	AFTER			
			Liquid Limits	0	**Test Date** 6/18/2024
Moisture (%)	25.8	25.4	**Plastic Limits**	0	
Dry Density (pcf)	98.2	100.0			
Saturation (%)	96.3	99.1			
Void Ratio	0.73	0.70	**Specific Gravity**	2.72	ASSUMED

Sample Description				
Project Number	J7656	**Depth (ft)**	15	**Remarks**
Sample Number	S-1351	**Boring Number**	B-8	
Project	Challenger Sports Complex			
Client	Victor Elementary School District			
Location				

FENAGH
ENGINEERING AND TESTING

Expansion Index of Soils
ASTM D4829

Project Name:	Sports Complex Project at Challenger E.S.	**Date Received:**	5/1/2024
Project No.:	7656	**Tested By:**	M. Tabb
Date Sampled:	4/30/24 / 5/1/24	**Sample Description:**	Brown Silty Sand
Sampled By:	E.R. / L.M.	**Date Tested:**	5/21/2024
Sample ID:	S-1351	**Lab Log / ET No.:**	40127

Location: B-5 at 0'-5'

Description: Brown Silty Sand

Expansion Index = 0

EI Classification = Very Low

Remarks: Received Sample Condition: Acceptable and/or Satisfactory

Magdalena Acevedo

5/21/24

Reviewed By Magdalena Acevedo — Date
Staff Engineer

9070 Center Avenue, Rancho Cucamonga, CA 91730 | (909) 587-6374 office

ARAGÓN GEOTECHNICAL, INC.
16801 Van Buren Blvd.
Riverside, California 92504
(951) 776-0345

Report of R-Value Lab Testing

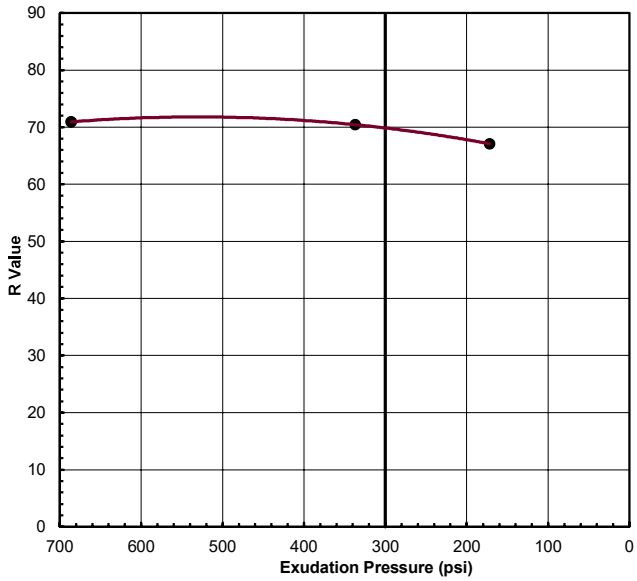
Project Name:	Sports Complex			**Lab No.:**	24-1531
Project No:	4899-TR [Client Project No. 7656]	**Sampled By:**	Client	**Report Date:**	May 14, 2024
Client:	Fenagh Engineering and Testing 9070 Center Avenue Rancho Cucamonga, CA 91730	**Date of Sampling:**	May 1, 2024		
Tested By:	A. Garcia			**Date Received:**	5/6/2024
Location Sampled:	B-2 @ 0-5'			**Date Tested:**	5/13/2024
Specifications:	No specifications provided			**Source:**	-
Sample Description:	Yellowish Red M-C Sand w/ Silt				

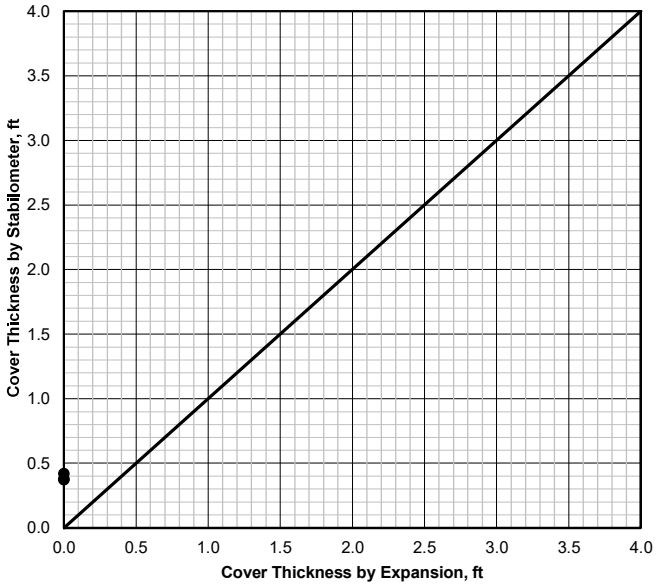
Specimen Identification	A	B	C
Moisture Content	8.0%	8.5%	9.3%
Compaction Pressure (psi)	350	350	350
Specimen Height (inches)	2.49	2.54	2.49
Dry Density (pcf)	121.5	120.3	121.7
Horiz. Pres. @ 1000lbs (psi)	16	17	18
Horiz. Pres. @ 2000lbs (psi)	28	29	31
Displacement	4.83	4.74	5.10
Expansion Pressure (psf)	0	0	0
Exudation Pressure (psi)	686	337	172
R-Value by Stabliometer (Uncorrected)	71	70	67
R-Value by Stabilometer (Corrected)	71	70	67
Thickness by Stabilometer	0.37	0.38	0.42
Thickness by Expansion	0.00	0.00	0.00
R-Value by Expansion	-	-	-

Assumptions:

Traffic Index	5.0
Gf	1.25

R-Value = 70
By Exudation





Remarks: No modifications made to test method, followed exact test procedure. R-Value at equilibrium is the lowest of the two R-Values between exudation and expansion.

- [x] All materials were sampled and tested per the project approved documents.
- [ ] All materials tested met project approved plans & specifications.
- [ ] All materials tested did not meet project approved plans & specifications.

Reviewed by: James Burling, EIT
Staff Engineer

May 14, 2024
Date:

ASTM/CTM Standards Used:	Material was sampled by client. Sample tested in accordance with ASTM D2844.

Testing was performed by qualified personnel in accordance with generally accepted industry practice, material testing consultants procedures and the above reference standards. This report is applicable only to the items listed herein. The tests performed and in this report are not intended to be considered as any guarantee or warranty of suitability for service or fitness of use of items tested and it should not be relied on as such. The report has been prepared for the exclusive use of the client and any partial or whole reproduction without the consent of the client is prohibited. AGI makes no representation or opinion to origin or composition of the above material.

Soil Analysis Lab Results

Job Name: Sports Complex, Challenger
Sample Date: May 1, 2024
Job Number: 7656
Project X Job Number: S240509A
May 10, 2024

	Method	ASTM D4327		ASTM D4327		ASTM G187		ASTM G51
Bore# / Description	**Depth**	**Sulfates** SO_4^{2-}		**Chlorides** Cl^-		**Resistivity** As Rec'd | Minimum		**pH**
	(ft)	(mg/kg)	(wt%)	(mg/kg)	(wt%)	(Ω-cm)	(Ω-cm)	
B-7	10.0	110.9	0.0111	33.3	0.0033	13,400	2,211	7.9

Cations and Anions, except Sulfide and Bicarbonate, tested with Ion Chromatography
mg/kg = milligrams per kilogram (parts per million) of dry soil weight
ND = 0 = Not Detected | NT = Not Tested | Unk = Unknown
Chemical Analysis performed on 1:3 Soil-To-Water extract
PPM = mg/kg (soil) = mg/L (Liquid)

Note: Sometimes a bad sulfate hit is a contaminated spot. Typical fertilizers are Potassium chloride, ammonium sulfate or ammonium sulfate nitrate (ASN). So this is another reason why testing full corrosion series is good because we then have the data to see if those other ingredients are present meaning the soil sample is just fertilizer-contaminated soil. This can happen often when the soil samples collected are simply surface scoops. This is why it's best to dig in a foot, throw away the top and test the deeper stuff. Dairy farms are also notorious for these items.

If one sample pops up much more corrosive than all others, we would recommend collecting more samples surrounding the problem sample location to determine if the peak is isolated to it. This allows us to conclude it was a contaminated sample and able to declare it an outlier.

Try out our new online forms: SOIL CORROSIVITY & THERMAL RESISTIVITY LAB REQUEST FORM & IN-SITU WENNER 4 PIN QUOTE REQUEST FORM

APPENDIX C
CALCULATIONS SHEETS

Dynamic Dry Settlement
Drilled Pole Foundation Capacity
C-3 Percolation Testing Results
C-4 Percolation Testing Results

FENAGH
ENGINEERING AND TESTING

FILE NO.: J7656
PROJECT: Challenger School of Sports and Fitness
BORING B-6

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

INPUT:

EARTHQUAKE INFORMATION:

Earthquake Magnitude:	7.9
Peak Horiz. Acceleration (g):	0.54

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	Fill	2.5	129.3	0.16	0.11	0.057	N/A	N/A	N/A	N/A	0.0	0.0	0.0	0.0	0.000	#DIV/0!	N/A	########	N/A	19.1695	0.0000	Fill
10.0	5.0	SM	7.5	132.0	0.49	0.33	0.170	34	1.3	100.0	1.60	70.7	40.5	5.6	76.3	1084.457	1.36E-04	2.50E-04	2.50E-02	1.00E-03	19.1695	0.0011	0.001
15.0	5.0	SM	12.5	117.7	0.80	0.54	0.276	19	1.3	80.0	1.32	32.6	50.6	5.6	38.2	1102.456	2.01E-04	3.70E-04	3.70E-02	1.20E-02	19.1695	0.0134	0.016
20.0	5.0	SP	17.5	130.0	1.11	0.74	0.377	41	1.3	100.0	1.16	61.8	11.9	3.0	64.8	1548.557	1.82E-04	3.30E-04	3.30E-02	2.00E-03	19.1695	0.0022	0.003
25.0	5.0	ML	22.5	112.2	1.41	0.95	0.471	18	1.3	69.0	1.11	26.0	94.9	5.6	31.6	1374.530	2.40E-04	5.00E-04	5.00E-02	2.10E-02	19.1695	0.0235	0.028
30.0	5.0	SM	27.5	134.7	1.72	1.15	0.561	29	1.3	83.0	0.95	35.8	42.1	5.6	41.4	1660.868	2.23E-04	4.00E-04	4.00E-02	1.00E-02	19.1695	0.0112	0.013
35.0	5.0	SP	32.5	121.3	2.04	1.37	0.647	65	1.3	100.0	0.87	73.5	7.1	2.0	75.5	2209.619	1.84E-04	2.70E-04	2.70E-02	1.00E-03	19.1695	0.0011	0.001
40.0	5.0	SM	37.5	133.1	2.36	1.58	0.725	12	1.3	50.0	0.79	12.3	70.7	5.6	17.9	1470.805	2.97E-04	6.20E-04	6.20E-02	6.00E-02	19.1695	0.0670	0.080
45.0	5.0	SM	42.5	128.5	2.69	1.80	0.797	33	1.3	78.0	0.78	33.5	18.6	5.6	39.1	2034.759	2.28E-04	3.50E-04	3.50E-02	1.00E-02	19.1695	0.0112	0.013
50.0	5.0	SW	47.5	135.8	3.02	2.02	0.862	50	1.3	89.0	0.75	48.8	8.0	5.6	54.4	2407.213	2.02E-04	2.70E-04	2.70E-02	3.80E-03	19.1695	0.0042	0.005

Total Calculated Dynamic Dry Settlement (inches) 0.16

Fenagh Engineering & Testing

Project: Challenger School of Sports and Fitness
File No.: 7656
Description: Isolated Pole Foundations

Drilled Pole Foundation Capacity Calculation

Input Data:

Unit Weight of Overlying Soil Layer	γ_1	125	pcf
Thickness of Overlying Soil Layer	H_1	0	feet
Unit Weight of Bearing Strata	γ_2	125	pcf
Friction Angle of Bearing Strata	ϕ_2	35	degrees
Friction Angle between Pile and Soil	δ	26.25	degrees
Cohesion of Bearing Strata	c_2	0	psf
Adhesion	c_A	0	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	100	feet

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

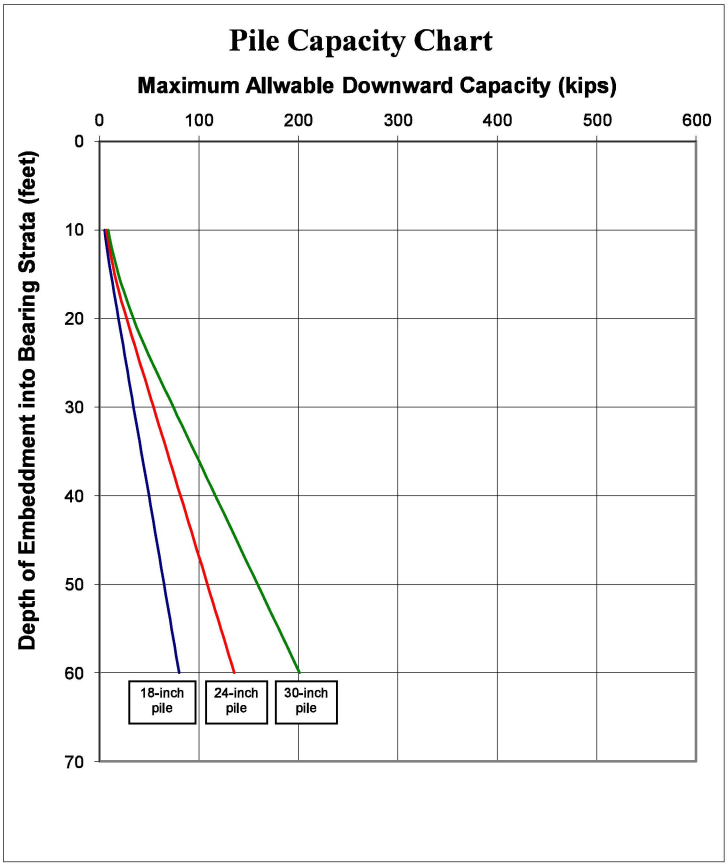
Pile Dimension:
18 inch diameter pile
24 inch diameter pile
30 inch diameter pile

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** $fs/FS = c_A/FS$

Pile Capacity:

Total Depth of Pile (feet)	Depth of Embeddment into Bearing Strata (feet)	Maximum Allowable Downward Pile Capacity: Capacity of 18 inch diameter pile (kips)	Capacity of 24 inch diameter pile (kips)	Capacity of 30 inch diameter pile (kips)
10	10	5.1	6.8	8.5
11	11	6.2	8.2	10.3
12	12	7.3	9.8	12.2
13	13	8.6	11.5	14.3
14	14	10.0	13.3	16.6
15	15	11.4	15.3	19.1
16	16	13.0	17.4	21.7
17	17	14.5	19.6	24.5
18	18	16.0	22.0	27.5
19	19	17.5	24.5	30.6
20	20	19.1	27.1	33.9
21	21	20.6	29.8	37.4
22	22	22.1	32.5	41.0
23	23	23.6	35.2	44.8
24	24	25.2	38.0	48.8
25	25	26.7	40.7	53.0
26	26	28.2	43.4	57.2
27	27	29.7	46.1	61.4
28	28	31.3	48.8	65.7
29	29	32.8	51.5	69.9
30	30	34.3	54.2	74.1
31	31	35.8	56.9	78.4
32	32	37.4	59.6	82.6
33	33	38.9	62.4	86.8
34	34	40.4	65.1	91.1
35	35	41.9	67.8	95.3
36	36	43.5	70.5	99.6
37	37	45.0	73.2	103.8
38	38	46.5	75.9	108.0
39	39	48.0	78.6	112.3
40	40	49.6	81.3	116.5
41	41	51.1	84.0	120.7
42	42	52.6	86.8	125.0
43	43	54.1	89.5	129.2
44	44	55.7	92.2	133.4
45	45	57.2	94.9	137.7
46	46	58.7	97.6	141.9
47	47	60.2	100.3	146.2
48	48	61.8	103.0	150.4
49	49	63.3	105.7	154.6
50	50	64.8	108.4	158.9
51	51	66.3	111.2	163.1
52	52	67.9	113.9	167.3
53	53	69.4	116.6	171.6
54	54	70.9	119.3	175.8
55	55	72.4	122.0	180.0
56	56	74.0	124.7	184.3
57	57	75.5	127.4	188.5
58	58	77.0	130.1	192.7
59	59	78.5	132.8	197.0
60	60	80.1	135.6	201.2



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

Report of Falling Head Borehole Percolation Testing

Storm Water Infiltration

Project Name:	Challenger Sport Complex			Test Number:	**P-1**
Job Number:	J7656			Tested By:	LM
Date Drilled:	4/30/2024	Pipe Interval:	-	Date Tested:	5/1/2024
Drilling Method:	Hollow Stem	Pipe Diameter:	3 inches	Method:	SB County
Drilled Depth:	10.0 feet	Gravel Pack:	Yes	Presoak Time:	24 hours
Hole Diameter:	8 inches	Gravel Size:	3/8"	Test Head:	78 inches

Trial No.	Time	Time Interval, ΔT (min)	Initial Water Height, H_o (in)	Final Water Height, H_f (in)	Change in Water Height, ΔH (in)	Percolation Rate (min/in)
1	10:35 AM 10:45 AM	0:10	78.0	1.8	76.2	0.1
2	10:53 AM 11:03 AM	0:10	76.8	3.0	73.8	0.1
3	11:06 AM 11:16 AM	0:10	78.0	4.8	73.2	0.1
4	11:19 AM 11:29 AM	0:10	75.6	2.4	73.2	0.1
5	11:32 AM 11:42 AM	0:10	78.0	2.4	75.6	0.1
6	11:45 AM 11:55 AM	0:10	78.0	1.2	76.8	0.1
				Observed Rate:	0.1	min/in
				Gravel Correction Factor:	1.00	
				Corrected Rate:	0.1	min/in
				***Tested Infiltation Rate, I_t:**	**22.2**	**in/hr**

*Tested infiltration rate using the Porchet Method:

$$I_t = \frac{\Delta H(60r)}{\Delta T(r + 2H_{avg})}$$

$$I_t = \frac{76.8(60)(4)}{10(4+2(39.6))}$$

$$I_t = 22.2 \text{ in/hr}$$

I_t = Tested infiltration rate [in/hr]
ΔH = Change in water head height over the time interval [in]
r = Test hole radius [in]
ΔT = Time interval [min]
H_{avg} = Average water head height over the time interval = $12(H_o + H_f)/2$ [in]

FENAGH ENGINEERING AND TESTING

14777 Hopland St
Victorville, CA 92392

By:	LM	Date:	June-24
Job No:	J7656	Figure:	C-3

Report of Falling Head Borehole Percolation Testing

Storm Water Infiltration

Project Name:	Challenger Sport Complex			Test Number:	**P-2 (Boring B-5)**
Job Number:	J7656			Tested By:	LM
Date Drilled:	4/30/2024	Pipe Interval:	-	Date Tested:	5/1/2024
Drilling Method:	Hollow Sten	Pipe Diameter:	3 inches	Method:	SB County
Drilled Depth:	8.0 feet	Gravel Pack:	Yes	Presoak Time:	24 hours
Hole Diameter:	8 inches	Gravel Size:	3/8"	Test Head:	75 inches

Trial No.	Time	Time Interval, ΔT (min)	Initial Water Height, H_o (in)	Final Water Height, H_f (in)	Change in Water Height, ΔH (in)	Percolation Rate (min/in)
1	12:05 PM 12:15 PM	0:10	68.4	0.0	68.4	0.1
2	12:18 PM 12:28 PM	0:10	75.6	0.0	75.6	0.1
3	12:32 PM 12:42 PM	0:10	76.8	0.0	76.8	0.1
4	12:45 PM 12:55 PM	0:10	75.6	0.0	75.6	0.1
5	12:58 PM 1:08 PM	0:10	74.4	0.0	74.4	0.1
6	1:12 PM 1:22 PM	0:10	80.0	0.0	80.0	0.1
				Observed Rate:	0.1	min/in
				Gravel Correction Factor:	1.00	
				Corrected Rate:	0.1	min/in
				***Tested Infiltation Rate, I_t:**	**22.8**	**in/hr**

*Tested infiltration rate using the Porchet Method:

$$I_t = \frac{\Delta H(60r)}{\Delta T(r + 2H_{avg})}$$

$$I_t = \frac{74.4(60)(4)}{10(4+2(37.2))}$$

$$I_t = 22.8 \text{ in/hr}$$

I_t = Tested infiltration rate [in/hr]
ΔH = Change in water head height over the time interval [in]
r = Test hole radius [in]
ΔT = Time interval [min]
H_{avg} = Average water head height over the time interval = $12(H_o + H_f)/2$ [in]

APPENDIX D
GEOHAZARDS ASSESSMENT REPORT

4 GEOLOGIC HAZARD
4.1 GEOLOGY
4.2 GROUNDWATER
4.3 SEISMIC HAZARDS
4.3.1 Faulting and Ground Rupture
4.3.2 Liquefaction
4.3.3 Lateral Spreading
4.3.4 Dynamic Dry Settlement
4.3.5 Tsunamis and Seiches
4.3.6 Historic Seismicity
4.4 FLOODING AND DAM INUNDATION
4.5 SLOPE STABILITY AND LANDSLIDING
4.6 SUBSIDENCE DUE TO FLUID WITHDRAWAL
4.7 UNDOCUMENTED FILL
4.8 EXPANSIVE SOILS
4.9 HYDRO-CONSOLIDATION
4.10 METHANE
4.11 OTHER POTENTIAL HAZARDS
4.12 CLOSURE
4.13 REFERENCES

APPENDIX D

4 GEOLOGIC HAZARDS

Geologic hazards relevant to the proposed development and in general accordance with Note 48 of the State of California Checklist for the Review of Engineering Geology and Seismology Reports for California Public Schools, Hospitals, and Essential Services Buildings (CGS, 2022) are addressed in this appendix. Table 4-1 including Plates 1, 2 and 3 in appendix of this report indicate the approximate location of the site and of the exploratory borings performed during onsite investigation.

Table 4-1: Exploration Depths and Locations

Exploration ID	Depth of Exploration (ft)	Approximate Latitude	Approximate Longitude	General Location
B-1	26-1/2	34.540961°	-117.4341156°	Western portion of the North Bleachers
B-2	11-1/2	34.540712°	-117.341438°	Western portion of the Field
B-3	21-1/2	34.540234°	-117.341368°	Western portion of the South Bleachers
B-4	26-1/2	34.539905°	-117.340693°	Eastern portion of the South Bleachers
B-5	8	34.540203°	-117.340322°	Eastern portion of the Field
B-6	51-1/2	34.540537°	-117.340230°	Eastern portion of the North Bleachers
B-7	10	34540539°	-117340089°	Eastern portion of the modular Building
B-8	21-1/2	34.540572°	-117.340183°	Western portion of the modular Building

Notes:
1. Latitude and longitude were estimated from Google Maps.

4.1 GEOLOGY

The Geologic Map of the Victorville 7.5-minute Quadrangle indicates the site is underlain by alluvium of the ancestral Mojave River (Qoam) (Hernandez et al, 2008). Geologic materials obtained from the onsite subsurface investigation by this firm were consistent with research findings. Alluvial soils were encountered to a maximum explored depth of 51-1/2 feet beneath existing ground surface. Regional and local geology for the project site is indicated on Plates 4 and 5, respectively. Details of the subsurface conditions encountered during onsite exploration are provided in Section 3.3 and Appendix A.

4.2 GROUNDWATER

Groundwater was not encountered during site exploration to a maximum excavated depth of 51-1/2 feet. Groundwater is not expected to impact the design or construction of the project.

Historically highest groundwater levels are not well defined for this site by the State of California Department of Conservation. Accordingly, the historically highest groundwater level was

approximated based on water level readings obtained from the Department of Water Resources Water Data Library. Based on the obtained water level data from nearby water monitoring wells, the historically highest groundwater level is estimated at deeper than 150 feet below existing ground surface. Details of groundwater levels obtained from nearby groundwater monitoring wells are provided in Section 3.4 of this report.

Fluctuations of the groundwater level, localized zones of perched water, and increased soil moisture content should be anticipated during and following the rainy season. Irrigation of landscaped areas on or adjacent to the site can also cause a fluctuation of local groundwater levels.

4.3 SEISMIC HAZARDS

Seismic hazards considered for this investigation include the potential for ground rupture due to faulting, seismic ground shaking, liquefaction, lateral spreading, dynamic dry settlement, tsunamis and seiches, and historical seismicity are discussed below. Other geologic hazards including flooding and dam inundation, slope stability and landslides, subsidence due to flood withdrawal, undocumented fills, expansive soils, hydro-consolidation, methane, and other potential hazards are also discussed in the following sections.

4.3.1 Faulting and Ground Rupture

Based on a review of available published geologic maps and reports, active faults have not been mapped at (or trending toward) the project site. Per the California Geological Survey (CGS, 2024a), the closest known active or potentially active faults are the Ord Mountains fault which is approximately 12-1/2 miles southeast of the site and the Helendale fault which is 12-1/2 miles to the northeast of the site. The Mirage Valley fault is also nearby at 13 miles to the northwest. Based on our understanding of the current geologic framework, the potential for surface rupture of active faults onsite is considered low. A copy of the Local Fault Map, Plate 6, which indicates nearby faults in proximity to the project site is enclosed herein.

4.3.2 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, poorly 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 liquefaction settlement distress observed at the ground surface.

Liquefaction susceptibility determination for a site is based on groundwater depth records, soil type and distance to a fault capable of producing a substantial earthquake. According to the San Bernardino County land Use Plan, liquefaction susceptibility at the project site is considered remote - see attached Seismic Hazard Zone Map, Plate 7. Groundwater was not encountered to a maximum excavated depth of 51-1/2 feet below ground surface during site exploration and the historically highest groundwater level for the subject site is estimated to be deeper than 150 feet below existing ground surface as discussed in section 3.4.

Based on the estimated historically highest groundwater level underlying the site and the calculated relative density of the underlying geologic material, it is the assessment of this firm that the potential for liquefaction at the site is considered remote.

4.3.3 Lateral Spreading

The site and immediate vicinity exhibit a generally flat topography. As previously discussed, liquefaction of site soils within 150 feet of ground surface is considered remote due to historically highest groundwater levels obtained from nearby groundwater monitoring wells. Lateral spreading distress is not anticipated to affect the proposed development during a design based seismic event as liquefaction within the site is not considered likely to occur.

4.3.3 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.

Site specific calculations indicate that seismically induced dynamic dry settlement will be on the order of 0.2 inches. This settlement estimate is considered negligible and well within the tolerance of a properly designed structure.

4.3.3 Tsunamis and Seiches

Tsunamis are waves generated in large bodies of water by fault displacement or major ground movement. Seiches are waves generated in enclosed bodies of water (e.g. large lakes or reservoirs) often in response to seismic waves although they can also be a response to changes in atmospheric conditions. Per the most recent Tsunami hazard maps published by the California Geological Survey (2024b) the project site is not located within a mapped tsunami hazard zone. Based on the inland location of the site and its distance and elevation from contained bodies of water, seiches and tsunamis are not considered a hazard to the project site.

4.3.4 Historic Seismicity

Research into the historical seismicity record was performed using data available on the United States Geological Survey website (USGS, 2024a) of recorded historical regional seismic events (those that have been recorded since the mid-1700s) with respect to the school site. Based on the available data it appears that the school has been exposed to relatively significant seismic events in the few decades as is much of southern California. Recent earthquakes in the region include the 1992 Landers and 1999 Hector Mine earthquakes.

4.4 FLOODING AND DAM INUNDATION

From a review of the United States Federal Emergency Management Agency (FEMA; 2024) the site is located in a region defined as an “Area of Minimal Flood Hazard”. A copy of the FEMA flood hazard map indicating flood hazard potential is provided in the appendix (Plate 8).

4.5 SLOPE STABILITY AND LANDSLIDING

No significant slopes are present or planned near the proposed development. As such, slope stability evaluation including development of static and dynamic strength parameters, pseudo-static slope stability coefficients, dynamic site conditions evaluation, and slope stability mitigation is not warranted for this project. In addition, as mapped by the State of California Geological Survey, the site is not located within an area designated as being subject to landslide hazards (CGS, 2024c).

4.6 SUBSIDENCE DUE TO FLUID WITHDRAWAL

Subsidence refers to ground settlement due to withdrawal of liquid from the underlying earth materials (such as water or oil). The site is not mapped within designated oil fields indicated by the California Department of Conservation online resources (DOGGR, Well Finder).

According to the USGS Areas of Land Subsidence in California Interactive Map (USGS, 2024b), the project site is not located in an area susceptible to significant subsidence problems.

4.7 UNDOCUMENTED FILL

Undocumented fill material was encountered during site exploration to depths ranging from 3-1/2 feet to 4 feet below existing ground surface. The observed fill consisted of medium dense, Sand to Silty Sand (SP/SM). The fill was observed to be dark yellowish brown, moist, and fine to medium grained with trace gravel.

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 compacted fill.

4.8 EXPANSIVE SOILS

The existence of expansive soils has the potential to impact a 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 can reduce this shrink-swell capability.

The near-surface soils observed during onsite exploration are classified as medium dense Sand to Silty Sand (SP/SM). Expansion Index testing yielded an EI of 0 (zero) indicating very low expansion potential for onsite soils. Accordingly, expansive soils are not expected to impact the development and is not a design consideration.

4.9 HYDRO-CONSOLIDATION

Hydro-consolidation is a phenomenon wherein soils lose volume when saturated. This can result in settlement of structures bearing thereon. The hydro-consolidation potential of the site soils was considered in the provision of consolidation testing. Hydro-consolidation was not observed during laboratory testing in accordance with ASTM D2435, one-dimension consolidation laboratory testing provided in Appendix B. Based on included laboratory testing including in-situ relative density estimations during onsite exploration, it is the assessment of this firm that the potential for damaging settlement due to hydro-collapse is not significant and not a design consideration.

4.10 METHANE

This office has reviewed the Department of Conservation Oil and Gas Well Finder Map (DOGGR, Well Finder). Based on this review it appears that the project site is not located within an oil field as designated by the state. A methane mitigation system is not likely required for the proposed development.

4.11 OTHER POTENTIAL HAZARDS

The following naturally occurring hazards are not believed to exist at the site nor in the vicinity: hydrogen-sulfide gas, tar seeps, volcanic eruption, radon-22 gas, and naturally occurring asbestos in geologic formations associated with serpentine rock.

4.12 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.

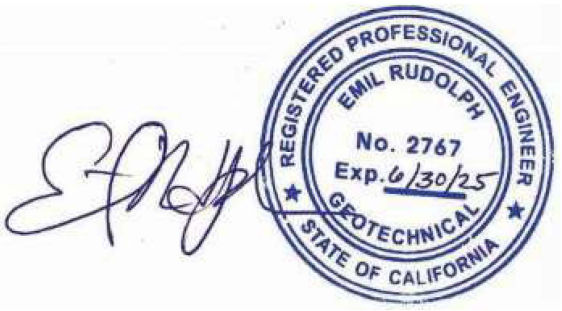
Sincerely,



Scott T. Prince, PE
Project Engineer



Jonathan Goodmacher, PG, CEG, HG
Certified Engineering Geologist



Emil Rudolph PE, GE
Principal Geotechnical Engineer

4.13 REFERENCES

California Geological Survey, California Department of Conservation, Fault Activity Map of California, 2024a, https://maps.conservation.ca.gov/cgs/fam/, accessed May 2024.

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