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GEOTECHNICAL ENGINEERING INVESTIGATION REPORT

PATRICK HENRY ELEMENTARY SCHOOL MODERNIZATION
1123 WEST ROMNEYA DRIVE
ANAHEIM, CALIFORNIA 92801

PREPARED FOR:
ANAHEIM ELEMENTARY SCHOOL DISTRICT
1001 S. EAST STREET, ANAHEIM, CA 92805

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PROJECT NO. 22-0161
AUGUST 22, 2022

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Anaheim Elementary School District
1001 S. East Street
Anaheim, CA 92805

SUBJECT: Geotechnical Engineering Investigation
Patrick Henry Elementary School
1123 West Romneya Drive
Anaheim, California

1. INTRODUCTION

This report presents the results of a Geotechnical Engineering Investigation & Percolation Testing performed by Koury Engineering & Testing, Inc. (Koury) for the proposed one and two-story buildings within the Patrick Henry Elementary School located at 1123 West Romneya Drive, City of Anaheim, California. The investigation was performed to evaluate the subsurface soil conditions in the area of the proposed buildings in order to provide geotechnical recommendations for design and construction. This report contains our findings and recommendations for the design and construction of the proposed buildings and associated improvements from a geotechnical standpoint.

The recommendations provided within this submittal are based on the results of our field exploration, laboratory testing and engineering analyses. Our services were performed in general accordance with our Proposal No. 22-0161, dated March 11, 2022.

Our professional services have been performed using the degree of care and skill ordinarily exercised, under similar circumstances, by reputable geotechnical consultants practicing in this or similar localities. No other warranty, expressed or implied, is made as to the professional advice included in this report. This report has been prepared exclusively for Anaheim Elementary School District and their consultants for the subject project. The report has not been prepared for use by other parties, and may not contain sufficient information for the purposes of other parties or other uses.

2. SITE CONDITIONS

The Patrick Henry Elementary School campus is located approximately 520 feet south of the 91 Freeway, and more specifically about 110 feet from the northeast corner of West Romneya Drive and North Lombard Drive. The campus is bounded on the east, north, and west by residential buildings and on the south by West Romneya Drive. The entrance to the campus is provided through West Romneya Drive. A site Vicinity Map with approximate ground contour elevations is presented in Appendix A as Figure A-1.

The areas for the proposed buildings are located within the eastern, southern and western portions of the campus and more specifically south of the existing playfield lawn and volleyball and basketball hardcourt areas. Following demolition of the existing buildings, it is anticipated that the new buildings will encroach partially on the existing parking lot and playfield area on the west and north and hardscape play areas on the north. The areas of the proposed improvements are currently covered partially with asphalt concrete, playfield lawn, and existing buildings. The campus ranges in elevations from about 143 to 151 feet (NAVD88) and generally slopes gently to the west. Drainage is generally by sheet flow to the west and south and is intercepted by concrete swales and storm drain, where present.

3. PROPOSED IMPROVEMENTS

Koury understands the Anaheim Elementary School District is planning to construct 2 one-story buildings (possibly combined into one building) and 2 two-story buildings within the eastern, southern and western portion of the Patrick Henry Elementary School campus. Based on the Proposed Master Plan provided to us, a two-story classroom building will be constructed along the eastern portion of the campus and will replace the existing lunch shelter and two side by side buildings oriented in the north-south direction. This building will include several classrooms for Kinder & Pre-K student on the first floor and for 3rd and 4th grade students on the second floor. The building will have dimensions of about 80 feet in width and 180 feet in length, and will be oriented in the north-south direction.

About 25 south of the building, it is planned to construct a kindergarten playground area. About 35 feet west of the kindergarten playground, there will be a one-story administration building with dimensions of about 80 by 85 feet. About 25 feet west of the Administration building, there will be the combined one-story building containing the Library, the Multipurpose Room and Kitchen facilities. The proposed building measures about 90 by 190 feet in plan and has an east-west orientation.

The second two-story building will be located about 20 feet north side of the kitchen facilities. This two-story building will accommodate several classrooms for 1st and 2nd graders on the first floor and 3rd and 4th graders on the second floor. The proposed building will have dimensions on the order 80 feet in width and 180 feet in length, and will be oriented in a north-south direction.

The modernization at the campus will also include a new parking lot and a student drop off zone along West Romneya Drive, a quad zone with playground area between the 2 two-story classroom buildings, fire lanes, playfields and hardcourt areas on the northeastern and northwestern portions of the campus, respectively.

Further modernization at the Patrick Henry Elementary Campus will possibly require also new walkways, underground utilities, asphalt and concrete pavement, low site walls, irrigation lines, and miscellaneous landscape improvements. The access to the campus will be provided through two driveways on the southeast and southwest corners of the campus just off West Romneya Drive.

No building loads were available to Koury for the preparation of this report. We have assumed that the buildings will consist of wood-framed structures with stucco façade, slab-on-grade floors and wood roof framing. Koury assumed that the buildings wall loads will not exceed 5 kips per linear foot (live plus dead loads). We have also assumed that the buildings column loads will not exceed about 100 kips.

4. FIELD EXPLORATION

The field exploration program consisted of drilling thirteen soil test borings on July 18 and 19, 2022 using a truck-mounted hollow-stem auger drill rig. These borings were drilled to depths ranging from 6½ and 51½ feet. In addition, two percolation borings (P-1 and P-2) were drilled with truck-mounted hollow-stem auger equipment to depths of 5 and 5½ feet. The locations of the borings and percolation tests are shown on the Boring Location Map, Figure A-2, Appendix A.

Standard penetration test samples, California ring samples, and/or bulk samples were obtained from selected depths for laboratory testing. The depths, blow counts and description of the samples are shown on the attached boring logs presented in Appendix B of this report. The contractor used a 140-lbs automatic hammer to drive the samplers 18 inches into the soils.

5. LABORATORY TESTING

Laboratory tests, including moisture content, dry unit weight, #200 sieve wash, gradation, direct shear and consolidation were performed on selected samples obtained from the borings to aid in the classification of the materials encountered and to evaluate their engineering properties. Sulfate, chloride, resistivity, and PH tests (corrosivity tests) were also performed on selected samples. The results of the laboratory tests are presented on the boring logs in Appendix B, and/or in Appendix C.

6. SOIL CONDITIONS

The subsurface soil profile within the proposed building footprint areas consists of fill underlain by alluvial deposits. The fill depths observed at the boring locations range from about 2½ to 4½ feet; deeper fill may be present at other locations, including utilities. The fill soils encountered consist of silty sand and poorly graded sand with silt, and are generally loose to medium dense and dry to moist.

The alluvium underlying the fill encountered consists predominantly of alternating layers of silty sand, poorly graded sand and sandy silt. A 2-foot thick layer of sandy lean clay was noted in Boring B-2 at a depth of approximately 16 to 18 feet. The alluvial sand is generally slightly moist to moist and loose to medium dense. The moisture contents of the alluvial sand range from about 0.8 to 21.9 percent (average 8.2%). The clay and sandy silt alluvium have moisture contents ranging from about 18.5 to 23.2 percent (average about 20%). The clay and silt are generally moist with a stiff consistency.

The silty sand tested has between about 20 and 48 percent fines (average 29%). Two tests on the silt indicated 52 and 67 percent fines.

With the exception of Boring B-1 at the 25 feet depth, the standard penetration test blow counts and equivalent blow counts from the modified California sampler indicated sand blow counts ranging from about 6 to 38 with an average around 18 blows per foot of the sampler penetration, thus confirming the presence of some loose to medium dense sand deposits. Boring B-1 has a blow count of 54 at the depth of 25 feet. The standard penetration test blow counts and equivalent blow counts from the modified California sampler for the soil classified as silt indicate blow counts ranging from about 10 to 18 with an average of about 13 blows per foot of the sampler penetration.

The sampler blow counts are generally lower at shallow depths. For the upper 12 feet of soil, the blow counts for the sand material range from about 6 to 15 with an average of about 11 blows per foot of sampler penetration.

One direct shear test indicated peak and ultimate friction angles of about 21 and 23 degrees with corresponding cohesion of 156 and 24 psf, respectively. The consolidation tests on silty sand material indicated hydrocollapse ranging from about 1 to 1½ percent and moderate to high consolidation characteristics following the collapse.

Variations in the soil conditions as well as detailed descriptions are indicated on the attached boring logs in Appendix B. The soil conditions described in this report are based on the soils observed in the test borings drilled for this investigation and the laboratory test results. Variations between and beyond the borings should be anticipated.

7. GROUNDWATER

The ground surface at the proposed improvement locations has elevations ranging from about 143 to 151 feet (NAVD88). No groundwater was encountered in our exploratory borings drilled to a maximum depth of 51½ feet. The map provided in the "Seismic Hazard Zone Report 03, for the Anaheim Quadrangle", published by the California Department of Conservation, Division of Mines and Geology (1997), indicates that the historic high groundwater may be deeper than 50 feet below the existing ground surface (see Figure A-4, Appendix A). Based on our findings, other than nuisance surface water infiltration from rain or irrigation, it is unlikely that groundwater will be encountered during the course of construction.

8. SITE GEOLOGY

The site is located in the Peninsular Ranges Geomorphic Province, and within the Central Block of the Los Angeles Basin. The Central Block is characterized by thick layers of alluvium overlying predominantly sedimentary rock of Pleistocene through Cretaceous age. Holocene age (up to 11,000 years old) alluvial deposits, which become increasingly older with depth, underlie the site. The Holocene alluvial material is reported to consist primarily of unconsolidated gravel, sand, silt and clay (See Figure A-3 for a Geologic Map). The borings drilled during our investigation on July 18 and 19, 2022 encountered alluvium materials consisting predominantly of silty sand, poorly graded sand with silt, and sandy silt.

9. OIL WELLS

The site is located about 2.4 miles east of the Buena Park Oil/Gas Field, about 2½ miles south of the Coyote East Oil/Gas Field, 3.6 miles southwest of the Richfield Oil/Gas Field, 3.3 miles west of the Olive Oil/Gas Field and 2½ mile north of the Anaheim Oil/Gas Field. The nearest active well is located about 3.9 miles west of the site. According to the California Division of Oil, Gas and Geothermal Resources, the closest plugged dry hole is located about 0.85 miles west of the school site and there is an idle well located approximately 2.3 miles north of the site (Figure A-9). No evidence of hazardous materials related to oil field was encountered during our field investigation. It is our opinion that no hazardous materials associated with active oil fields should be present on site based on readily available oil/gas well information.

10. SEISMIC CONSIDERATIONS

10.1. General

Patrick Henry Elementary School, like the rest of Southern California, is located within a seismically active region as a result of being located near the active margin between the North American and Pacific tectonic plates. The principal source of seismic activity is movement along the northwest-trending regional faults such as the San Andreas, San Jacinto, Newport-Inglewood and Whittier-Elsinore fault zones.

By definition of the California Geological Survey (CGS), an active fault is one which has had surface displacement within the Holocene Epoch (roughly the last 11,000 years). The CGS has defined a pre-Holocene fault as any fault which has been active during the Quaternary Period (approximately the last 2,000,000 years, excluding the Holocene). These definitions are used in delineating Earthquake Fault Zones as mandated by the Alquist-Priolo Geologic Hazard Zones Act of 1972 and as subsequently revised in 1997 as the Alquist-Priolo Earthquake Fault Zones. The intent of the act is to require fault investigations for sites located within Special Studies Zone to preclude new construction of certain inhabited structures across the trace of active faults.

The subject site is not located within an Alquist-Priolo Earthquake Fault Zone. Based on California Geological Survey maps, the nearest Alquist-Priolo Earthquake Fault Zone is the Whittier Elsinore Fault Zone located approximately 6.6 miles northeast of the site. The Newport-Inglewood Fault Zone is located approximately 11½ miles southwest of the site. The nearest segment of the Puente Hills Blind Trust Fault is located about 1¼ miles north of the site. The El Modeno Fault, which has not indicated signs of movement during the last 11,000 years, is located about 4 miles southeast of the site (Figure A-4). No evidence of active or potentially active faulting was observed on the subject site during our investigation. In our opinion, surface rupture has a low potential to occur.

Based on the information available at this time, according to CGS, there is a potential for an Mw7.5 earthquake on the strike-slip Newport Inglewood Fault Zone, an Mw7.8 earthquake on the strike-slip Whittier Elsinore connected Fault Zone, and a MW6.9 earthquake on the Puente Hills (Coyote Hills) Thrust Fault Zone. Large earthquakes could occur on other faults in the general area, but because of their greater distance and/or lower probability of occurrence, they may be less important to the site from a seismic shaking standpoint.

Due to the proximity of the site to nearby faults, near field effects from strong ground motion associated with a large earthquake along these faults may occur at the site. These near field effects, including "fling" and directivity of strong ground motion, may result in significantly higher accelerations at the site. Figure A-6, presented in Appendix A, shows the approximate locations of some of the nearby active or potentially active faults.

According to the EQSEARCH program, within a search radius of 60 miles, about 67 earthquakes of magnitude 5 or greater have been recorded up to the year 2000. Within that same period, there are records of 15 earthquakes of magnitude 6 or greater, 5 earthquakes of magnitude 6.5 or greater, and 2 earthquakes of magnitude 7 or greater within the same search area. The largest and closest earthquake for the search was reported to have occurred in 1812 at a location about 38 miles from the site. Using the attenuation relationship of Campbell and Bozorgnia for alluvium (1997), the highest acceleration at the site could have been on the order of 0.125g. A summary of the earthquakes with magnitudes 5 and greater is attached in Appendix D.

10.2. Landsliding

The site is not located in a Landslide Hazards Zone on the State of California Seismic Hazards Zones Map (Figure A-5 in Appendix A). No evidence for landsliding was observed on or in the immediate vicinity of the site. Therefore, due to the lack of significant topographic changes at the project site, landsliding is not a potential problem at the site.

10.3. Lateral Spreading

The damaging effect of liquefaction settlement can be exacerbated when the soils are subject to lateral spreading. The site is generally level or very gently sloping. In addition, due to the absence of shallow groundwater, the potential for lateral spreading is remote.

10.4. Liquefaction

Liquefaction may occur when saturated, loose to medium dense, cohesionless soils are densified by ground shaking or vibrations. The densification results in increased pore water pressures if the soils are not sufficiently permeable to dissipate these pressures during and immediately following an earthquake. When the pore water pressure is equal to or exceeds the overburden pressure, liquefaction of the affected soil layers occurs. For liquefaction to occur, three conditions are required:

- Ground shaking of sufficient magnitude and duration;
- Groundwater level at or above the level of the susceptible soils during the ground shaking; and
- Soils that are susceptible to liquefaction.

The Liquefaction Hazards zone on the State of California Seismic Hazards Zones Map (Figure A-5 in Appendix A) indicates that the site is not located in a generalized liquefaction susceptibility zone.

Due to the absence of shallow groundwater, it is our opinion that the potential for liquefaction is remote at the site.

For dry seismic settlement evaluation, we calculated an earthquake magnitude of M7.3 from a seismic-hazard deaggregation using the USGS Unified Hazard Tool. The analysis also utilized a site acceleration of 0.716g (PGA_M) obtained from the USGS Web Site and the SEAOC/OSHPD tool. The seismic settlement calculations were performed for the deepest borings (B-2 & B-7) located within the proposed building areas. The California sampler blow counts were multiplied by a factor of 0.65 to obtain the equivalent SPT blow counts. The SPT tests were performed with an automatic hammer and unlined SPT samplers with an inner diameter of 1.5 inches. We used a hammer energy factor of 1.25 (Ce=1.25), a borehole diameter factor of 1.0 (Cb=1), and sampling method factors of 1.0 and 1.2 for California Ring samples and SPT samples, respectively, in our analyses.

We calculated total dry seismic settlements on the order of 1 inch using the LIQSVS software. The results of seismic settlement analyses are presented in Appendix C of this report. Considering the proposed recommendations for grading and building support presented in this report, and the recommendations in Section 7.66 of the SCEC Guidelines for Implementation of SP 117, it is our opinion that a differential seismic settlement on the order of ½ inch in 40 feet should be considered for the design seismic event.

10.5. Tsunamis and Seiche

The proposed building sites are located at approximate elevation 143 to 151 feet (NAVD88) and about 12 miles away from the coastline. Therefore, tsunamis are not considered to be potential hazards to the site. Since there are no large bodies of water located immediately adjacent to the site, seiches are also not considered a hazard.

11. FLOODING

The campus lies within a flood hazard zone X with a 0.2 percent annual chance flood as shown on the FEMA Flood Map # 06059C0131J, effective date December 3, 2009 (Figure A-7, Appendix A). The County of Orange Safety Elements show the site as being in the Prado Dam inundation zone. Therefore, flooding is considered to be a moderate potential hazard to the site.

12. COLLAPSIBLE SOILS

Soils prone to collapse are generally young sediments deposited by flash floods. There appears to be some low to moderate potential for collapse to occur within the near surface soils. Overexcavation and re-compaction, and drainage measures are recommended to mitigate potential collapse.

13. CONCLUSIONS AND RECOMMENDATIONS

13.1. General

In our opinion, the planned improvements are feasible from a geotechnical engineering point of view provided the geotechnical recommendations presented in this report are followed. The main concerns from a geotechnical standpoint are the presence of undocumented fill, presence of soils with relatively low cohesion and with potential for erosion, the potential for trench caving during construction, and the anticipated seismic settlement and strong seismic shaking.

The following sections contain geotechnical recommendations for the design and construction of the subject buildings and include our recommendations and discussions about bearing capacity, settlement, flatwork, slabs-on-grade, temporary excavations, and utility trenches. It is recommended that a formal review of foundation plans be performed by our office when plans become available, to verify the applicability of the recommendations contained herein.

13.2. Grading

13.2.1. Buildings Pad

The fill encountered during our site investigation within the building pad areas is on the order of 2½ to 4½ feet in thickness. We recommend removing all fill within building pads and foundation areas unless documentation is encountered to attest to the presence of engineered fill.

Any existing pavement, foundation, vegetation, abandoned underground utilities and other debris should be removed from the proposed buildings areas. Except as noted, within the building pad areas, we recommend overexcavating the fill completely and the subgrade to at least 4 feet below the existing grade and 2½ feet below the proposed footings, whichever is deeper for the one-story building and 3 feet for the two-story buildings. Where feasible, the overexcavation should extend laterally at least 5 feet beyond the building perimeters or footing edges, whichever is greater.

Following geotechnical approval, the bottoms of the removal excavations should be scarified to a depth of 10 inches, moisture conditioned to at least optimum moisture content for sand and 120 percent of optimum moisture for clay and recompacted to 93% relative compaction as determined by ASTM D1557. All fill placed below the building areas should be compacted to at least 95% relative compaction with a moisture content within 2½ percent of optimum for sand material; clay should not be used for backfill below the building areas. The fill may consist of excavated onsite silty sand, poorly graded sand with silt and sandy silt; expansive soils should not be used below the buildings and concrete flatwork, if encountered. Non expansive imported granular material may also be used as backfill.

13.2.2. Exterior Flatwork and Pavement Areas

Similarly to the building footprint areas, all abandoned utilities should be removed and the excavations should be backfilled with engineered fill. We recommend overexcavation and placement of at least 2 feet of new engineered fill for the subgrade of all new non-structural concrete or asphalt flatwork. Except for pavement areas, all fill outside the structure areas should be compacted to at least 90% relative compaction for clay and 93% relative compaction for sand soils. Within pavement areas, the upper 12 inches of subgrade should be compacted to 95% relative compaction for sand and 93% relative compaction for clay.

13.3. General Grading Requirements

1. All fill, unless otherwise specifically stated in the report, should be compacted to at least 90 percent of the maximum dry unit weight for clay soils and 95 percent for granular material as determined by ASTM D 1557 Method of Soil Compaction.

2. No fill should be placed until the area to receive the fill has been adequately prepared and approved by the Geotechnical Consultant or his representative.

3. Fill soils should be kept free of debris and organic material.

4. Rocks or hard fragments larger than 2 inches may not be placed in the fill below the buildings or foundations and within one foot of finished subgrade for exterior flatwork without approval of the Geotechnical Consultant or his representative, and in a manner specified for each occurrence.

5. The fill material should be placed in lifts which, when loose, should not exceed 8 inches per lift. Each lift should be spread evenly and should be thoroughly mixed during the spreading to obtain uniformity of material and moisture.

6. The moisture content of the fill material should be at least 120 percent of optimum, whichever is greater, for clay soils and within 2½ percent of optimum for sand. Water should be added and thoroughly dispersed until the soil has a moisture content within the range specified unless recommended otherwise by the Geotechnical Engineer at the time of construction.

7. When the moisture content of the fill material is too high to obtain adequate compaction, the fill material should be aerated by blading or other satisfactory methods until the soil has a moisture content as specified herein.

8. Permanent fill and cut slopes supporting flatwork or structures should not be constructed at gradients steeper than 2:1 (H:V).

13.4. Fill Materials

13.4.1. Onsite Materials

Most of the onsite soils encountered at shallow depth in the borings are considered to have a low expansion potential and are suitable for backfilling purposes, following proper processing, provided they are free of organics, construction debris and deleterious materials. Also, non-expansive import materials (EI less than 20) may be used for backfilling purpose.

Overexcavation and re-compaction will induce fill shrinkage. Many factors such as mixing, relative compaction of the fill, and topographic approximations will affect shrinkage. We cannot estimate the exact amount of shrinkage; however, in our opinion, the shrinkage may be on the order of 15 to 20 percent for native material excavated and recompacted to 95 percent relative compaction. This estimate does not include the material that will be required to fill in the excavations after the removal of any subsurface structures left in-place from the prior use of the site and removal of topsoil.

13.4.2. Import

Import materials should contain sufficient fines (binder material) so as to be relatively impermeable and result in a stable subgrade when compacted. The imported materials should have an expansion index (EI) less than 20 and should be free of organic materials, debris, and cobbles larger than 2½ inches with no more than 35% passing the # 200 sieve. A bulk sample of potential import material, weighing at least 30 pounds, should be submitted to the Geotechnical Consultant at least 48 hours before fill operations. Other than aggregate base and bedding sand, all proposed import materials should be tested for corrosivity, should be environmentally cleared from contamination and should be approved by the Geotechnical Consultant prior to being imported onsite.

13.5. Temporary Excavations

The shallow undisturbed site soils are expected to be temporarily stable when excavated vertically to a depth of 4 feet for unsurcharged alluvium. For deeper excavations up to a depth of 7 feet, we recommend a gradient no steeper than 1:1 (H:V) unless shoring is used. For unsurcharged excavations between the depths of 7 and 10 feet, a 1.5:1 slope gradient should be used.

The top of slopes should be barricaded to prevent vehicles and storage loads within 6 feet of the tops of the slopes. A greater setback may be necessary when considering heavy vehicles, such as concrete trucks and cranes; we should be advised of such heavy vehicle loadings so that specific setback requirements can be established. When excavating adjacent to existing footings or building supports, proper means should be employed to prevent any possible damage to the existing structures. Un-shored excavations should not extend below a 1½:1 (H:V) plane extending downward from the lower edge of adjacent footings and should start at least three feet away from the footing. Where there is insufficient space to slope back an excavation, shoring may be required. All regulations of State and Federal OSHA should be followed. Some sloughing and caving of excavations should be anticipated.

Temporary excavations are assumed to be those that will remain un-shored for a period of time not exceeding one week. In dry weather, the excavation slopes should be kept moist, but not soaked. If excavations are made during the rainy season (normally from November through April), particular care should be taken to protect slopes against erosion. Mitigative measures, such as installation of berms, plastic sheeting, or other devices, may be warranted to prevent surface water from flowing over or ponding at the top of excavations.

13.6. Floor Slabs

13.6.1. General

The grading recommendations for the new building floor slabs are provided in Section 13.2.1. The building floor slabs, as a minimum, should have a nominal thickness of 5 inches and should contain as a minimum No. 4 bars spaced a maximum of 16 inches on centers, in both directions. It is recommended that the compacted subgrade be moistened prior to casting floor slabs. Thicker slabs and additional reinforcement may be required depending on the floor loads and the structural requirements as determined by the Structural Engineer. It is recommended that the compacted subgrade be moistened prior to placement of the vapor retarder.

13.6.2 Moisture Sensitive Floor Coverings

Water vapor transmitted through floor slabs is a common cause of floor covering problems. In areas where moisture-sensitive floor coverings (such as tile, hardwood floors, linoleum or carpeting) are planned, a vapor retarder should be installed below the concrete slabs to reduce excess vapor transmission through the slab.

The function of the recommended relatively impermeable membrane (vapor retarder) is to reduce the amount of soil moisture or water vapor that is transmitted through the floor slab. The membrane should be 15-mil thick, Class A, and care should be taken to preserve the continuity and integrity of the membrane beneath the floor slab. The vapor retarder should conform to ASTM E1745. The vapor retarder should be installed in strict conformance with the manufacture recommendations.

A capillary break may be used at the discretion of the project architect. If a capillary break is used, at least 4 inches of free draining crushed rock, with no more than 2 percent passing the No. 200 sieve, should be placed below the vapor retarder. The crushed rock should be vibrated in place to achieve the compaction required by the project specifications. The gradation for the free draining capillary break material should conform to the requirements for No. 4 Concrete Aggregates as specified in Section 200-1.4 of the Standard Specifications for Public Works Construction (Greenbook) or approved equivalent.

Another factor affecting vapor transmission through floor slabs is the water to cement ratio in the concrete used for the floor slab. A high water to cement ratio increases the porosity of the concrete, thereby facilitating the transmission of water vapor through the slab. The project Structural Engineer should provide recommendations for design of concrete for footings and floor slabs in accordance with the latest version of the applicable codes. We recommend a concrete strength of at least 3500 psi with a water cement ratio not exceeding 0.5 unless indicated otherwise by the Structural Engineer. The placement of sand above the vapor retarder is the purview of the Structural Engineer.

13.7. Seismic Coefficients

Under the Earthquake Design Regulations of Chapter 16, Section 1613A of the CBC 2019, the following coefficients and factors (mapped values) presented in Table 1 were calculated using the USGS web site and the SEAOC/OSHPD tool (see Figure A-8).

Table 1 – Seismic Coefficients and Factors

Site Class (CBC 2019 – 1613A.3.2)	D
Seismic Design Category based on Occupancy Category III (CBC 2019-1604A.5 &1613A.3.5)	D
Mapped Acceleration Parameter for Short Period (0.2 Second), S_S	1.529
Mapped Acceleration Parameter for 1.0 Second, S_1	0.539
Adjusted Maximum Spectral Response Parameter for Short Period (0.2 Second), S_{MS}	1.529
Adjusted Maximum Spectral Response Parameter for 1.0 Second Period, S_{M1}	*
Design Spectral Response Acceleration Parameter, S_{DS}	1.02
Design Spectral Response Acceleration Parameter, S_{D1}	*
Peak Ground Acceleration (PGA_M)	0.716
Period (T_0/T_S)	$^{+}$.124/.620

Project Site Coordinates: Longitude: W-117.932929° Latitude: N33.851286 (WGS84)
$^{+}$Based on F_v of 1.76 for period calculation, otherwise should be based on F_v of 2.5. *See Section 11.4.8 of ASCE 7-16 for calculation requirements, exception 2. It is assumed that the seismic response coefficient C_S will be determined by Equations 12.8-2, 12.8-3 or 12.8-4 of ASCE 7-16.

The site class is determined in accordance with ASCE 7 Chapter 20 using shear wave velocity, SPT blow count or undrained shear strength. For a site to be classified as Site Class D the weighted average SPT blow count should be between 15 and 50 or the average weighted undrained shear strength should be between 1,000 and 2,000 psf within the upper 100 feet of soil. The SPT blow counts presented on the boring logs indicate that the requirements for Class D are met.

13.8. Foundations

General: For the purpose of preparing this report, we assumed that the proposed structures will impose column loads of about 100 kips or less and continuous wall loads of about 5 kips per lineal foot or less.

The proposed footings should be founded on at least 2½ feet of new engineered fill for one-story buildings and on 3 feet of engineered for 2-story buildings. The recommendations for preparation of the soils underlying the footings are provided in the "Grading" section of this report. The Structural Engineer should design foundations and floor slabs in accordance with the requirements of the applicable building code.

Footings supporting the proposed structures should have a minimum width of 2 feet for isolated footings and 1.5 feet for continuous footings. The bottom of footings should be located at least 24 inches below the lowest adjacent finish grade. A net vertical bearing value of 2,000 psf may be used to design the footings. This bearing value may be increased by 250 psf for each additional foot of width or depth up to a maximum of 2,500 psf. If a footing is located within one footing width of an existing foundation, the allowable bearing pressure should be reduced by 30 percent. A one-third increase in the bearing value may be used when considering wind or seismic loads. The footings should be reinforced with at least two No. 5 bars top and bottom or other reinforcement as determined by the Structural Engineer.

Lateral Resistance: Lateral load resistance may be derived from passive resistance along the vertical sides of the foundations, friction acting at the base of the foundations, or a combination of the two. A coefficient of friction of 0.35 may be used between the footings, floor slabs, and the supporting soils comprised of compacted sand materials. The friction coefficient used for a slab supported on the vapor retarder should not exceed 0.1. The passive resistance of level properly compacted fill soils in direct contact with the footings may be assumed to be equal to the pressure developed by a fluid with a density of 250 pcf, to a maximum pressure of 2,500 psf (allowable). A one-third increase in the passive value may be used for wind or seismic loads. The frictional resistance and the passive resistance of the soils may be combined provided that the passive resistance is reduced by one third. We recommend that the upper 12 inches of soil cover be neglected in the passive resistance calculations if the ground surface is not protected from erosion or disturbance by a slab, pavement or in a similar manner.

<u>Estimated Settlement:</u> Based on the results of our analyses and provided that our recommendations in preceding sections of this report are followed, we estimate that the total static settlement of isolated and/or strip footings under sustained loads would be on the order of 1 inch for the estimated maximum structural loads. A large portion of this settlement is anticipated to occur during construction.

For the design earthquake, the calculated seismically-induced dry settlement is anticipated to be on the order of 1 inch with a differential seismic settlement on the order ½ inch over a horizontal distance of 40 feet. The maximum differential static settlement, over a horizontal distance of 40 feet, is also anticipated to be on the order of ½ inch for similarly loaded footings.

13.9. Retaining Walls

Retaining walls in the range of 1½ to 5 feet in height may be associated with the improvements. The pressure behind retaining walls depends primarily on the allowable wall movement, wall inclination, type of backfill materials, backfill slopes, surcharge, and drainage. Determination of whether the active or at-rest condition is appropriate for design will depend on the flexibility of the walls. Walls that are free to rotate at least 0.002 radians at the top (deflection at the top of the wall of at least 0.002 x H, where H is the unbalanced wall height) can be designed for active conditions. The recommended active and at-rest pressures for the site silty sand backfill are presented in the following table. No clay should be used to backfill behind retaining walls.

Table 2 - Earth Pressures for Retaining Walls

Wall Movement	Backfill Condition	Equivalent Fluid Pressure (onsite sand) (pcf)
Free to Deflect	Level	40
Restrained	Level	65

The above lateral earth pressures do not include the effects of surcharge (e.g. traffic, footings, hydrostatic pressure) or compaction. Any surcharge (live, including traffic, or dead load) located within a 1:1 plane drawn upward from the base of the excavation should be added to the lateral earth pressures. The lateral pressure addition of a uniform surcharge load located immediately behind walls may be calculated by multiplying the surcharge by 0.33 for cantilevered walls and 0.5 for restrained walls. For vehicular surcharge adjacent to driveways or parking areas, a uniform lateral pressure of 100 pounds per square foot, acting as a result of an assumed 300 pounds per square foot traffic surcharge, should be used.

A drainage system should be provided behind the walls to reduce the potential for development of hydrostatic pressure. If a drainage system is not installed, the walls should be designed to resist the hydrostatic pressure in addition to the earth pressure.

Walls should be properly drained and waterproofed. Except for the upper 2 feet, the backfill immediately behind retaining walls (minimum horizontal distance of 12 inches) should consist of free-draining ¾-inch crushed rock wrapped with filter fabric. A 4-inch diameter perforated PVC pipe, placed perforations down at the bottom of the crushed rock backfill, leading to a suitable gravity outlet, should be installed.

All undocumented fill should be removed from the footing area for retaining walls taller than 3 feet. The wall footings should be underlain by at least 1½ feet of new engineered fill compacted to 95 percent relative compaction. The footing embedment should be at least 1½ feet below the lowest adjacent grade. The maximum allowable bearing pressure recommended is 2,000 psf.

13.10. Utility Trench Backfill

Bedding material surrounding utility lines and extending to a point 12 inches above the lines should consist of either sand, fine-grained gravel, or sand-cement slurry to support and/or to protect the lines. The bedding material should meet the specifications provided in the latest edition of the "Standard Specifications for Public Works Construction" (Greenbook). Sand or gravel should be compacted in accordance with Greenbook specifications.

Above the bedding, up to finished subgrade in areas other than landscape and up to one foot below flatworks and pavements, utility trenches should be backfilled with onsite sands or imported granular materials and mechanically compacted to at least 93% of the maximum dry density of the soils.

Below pavements, a minimum relative compaction of 95% is required in the upper 12 inches of the subgrade. For utility trenches within the buildings, the backfill should be compacted to the minimum required relative compaction indicated under the "Grading" section of this report. The material should be observed, tested and approved by the Geotechnical Consultant. The trench materials should be placed in accordance with Sections 306-1.2.1 and 306-1.3 of the "Standard Specifications for Public Works Construction" (Greenbook).

When adjacent to any footings, utility trenches and pipes should be laid above an imaginary line measured at a gradient of 1½ :1 (H:V) projected down from the bottom edges of any footings. Otherwise, the pipe should be designed to accept the lateral effect from the footing load, or the footing bottom should be deepened as needed to comply with this requirement. Backfill consisting of 2-sack sand-cement slurry may also be used.

13.11. Drainage

Foundation, slab, flatwork, and pavement performance depend greatly on proper drainage within and along the boundary of the improvements. Perimeter grades around the buildings should be sloped in a manner allowing water to drain away from the buildings and not pond next to the foundations. Roof downdrains should be connected to underground pipes carrying water away from the building areas or have extenders so water does not drain and pond next to the buildings. Per the 2019 CBC, landscape areas within 10 feet of buildings should slope away at gradients of at least 5 percent. Paved areas within 10 feet of buildings should slope away at gradients of at least 2 percent. Proper drainage is recommended for all surfaces to reduce the potential settlement due to water infiltration. We recommend minimizing the size and number of planters adjacent to the buildings and other foundations and using drought resistant planting in order to avoid distress due to settlement.

13.12. Asphalt Concrete (AC) Pavement

The required pavement structural sections depend on the expected wheel loads, volume of traffic, and subgrade soils. The characteristics of subgrade soils are determined by R-value testing. Based on soil classification and prior experience with similar soils, we anticipate an R-value on the order of 35 or better for the silty sand encountered in the borings. The R-value may be confirmed with additional tests, if necessary, at the time of construction. The following pavement sections were calculated based on assumed traffic indices of 4, 5, 6 and 7. The project Civil

Engineer should determine the traffic index to be used for different areas of the site. A traffic index of 6 to 7 is normally utilized to design fire lanes.

Table 3 - Asphalt Pavement Section

Traffic Index	Asphalt Thickness (Inches)	Base Course (CAB) Thickness (Inches)
4	3.0	4.0
5	3.0	5.0
5	3.5	4.0
6	3.5	7.0
6	4.0	6.0
7	4.5	8.0

Base course material should consist of Crushed Aggregate Base (CAB) as defined by Section 200-2.2 of the Standard Specifications for Public Works Construction (“Greenbook”). Base course should be compacted to at least 95 percent of the maximum dry density of that material. Crushed Miscellaneous Base (CMB) may be used only if the supplier can demonstrate that the aggregate does not contain contaminated material.

The grading for flatwork is addressed in Section 13.2.2 of this report. The subgrade should be in a “non-pumping” condition at the time of compaction.

Any onsite surficial organic soils within landscaped/turf areas should not be used as subgrade materials. Where feasible, the overexcavation should extend laterally a minimum of 2 feet beyond the perimeters and edges of parking areas, roadways and curbs. If present, any abandoned footing and/or underground concrete structure within the work limit should be removed entirely and the excavation should be backfilled to grade.

13.13. Portland Cement Concrete (PCC) Vehicular Pavement

The grading recommendations for vehicular PCC pavement are provided in Section 13.2.2 of this report. Base course material, used in the vehicular pavement sections, should consist of Crushed Aggregate Base (CAB) as defined by Section 200-2.2 of the Standard Specifications for Public Works Construction (Greenbook 2018). The aggregate base course should be compacted to at least 95% of the maximum dry density of that material. Crushed Miscellaneous Base (CMB) may be used only if the supplier can demonstrate that the aggregate does not contain contaminated material.

The recommendations presented herein should be used for design and construction of the slabs and pertaining grading work underlying vehicular pavement areas. A minimum modulus of rupture of 550 psi for concrete has been assumed in designing the PCC pavement sections; this corresponds to a concrete compressive strength of approximately 4,000 psi at 28 days. A qualified design professional should specify where heavy duty and standard duty slabs are used based on the anticipated type and frequency of traffic. Fire access roads are normally considered heavy duty pavement. The recommended vehicular PCC pavement sections are provided in the following table.

Table 4 – PCC Pavement Sections

Pavement Type	Portland Cement Concrete Thickness (inches)	Base Course (CAB) Thickness (inches)
Light Duty	6.5	4.0
Heavy Duty	7.0	6.0

These concrete pavement sections should be increased for bus traffic where applicable. Also the following recommendations should be incorporated into the design and construction of PCC pavement section:

- The pavement sections should be reinforced with No. 3 rebars spaced at 18 inches on centers each way to reduce the potential for shrinkage cracking.
- Joint spacing in feet should not exceed twice the slab thickness in inches, e.g., 12 feet for a 6-inch thick slab. Regardless of slab thickness, joint spacing should not exceed 15 feet.
- Layout joints should form square panels. When this is not practical, rectangular panels can be used if the long dimension is no more than 1.5 times the short one.
- Control joints should have a depth of at least 1/4 the slab thickness, e.g., 1 inch for a 4-inch thick slab.
- Pavement section design assumes that proper maintenance such as sealing and repair of localized distress will be performed on a periodic basis.

13.14. Portland Cement Pedestrian Pavement, Pavers and Turf Blocks

The grading recommendations for exterior flatwork are provided in Section 13.2.2. Exterior concrete slabs for pedestrian traffic or landscape should be at least four inches thick. Weakened plane joints should be located at intervals of no more than about 8 feet unless slabs thicker than 4 inches are used. The use of aggregate base below the pedestrian pavement is left to the discretion of the project Civil Engineer.

To prolong the pavement life, a 4-inch layer of aggregate base may be placed below the concrete where frequent use of heavy cart is anticipated. The concrete strength for pedestrian walkways should be at least 2,500 psi unless determined otherwise by the Structural Engineer. If necessary, the pavement sections should be reinforced with No. 3 rebars spaced no further than 18 inches on centers each way to control shrinkage cracking.

If pedestrian pavers are used, they should be supported on one inch of sand underlain by 4 inches of crushed aggregate base (CAB). For light vehicular traffic, the pavers should be underlain by one inch of sand and at least 9 inches of aggregate base (CAB). For heavy duty traffic areas, we recommend increasing the aggregate base thickness to 14 inches. The aggregate base should be underlain by a separation/filter fabric for heavy duty traffic areas. A similar heavy-duty construction may be used for fire truck turf block pavers. For turf block pavers, a filter fabric should also be placed also above the aggregate base to prevent contamination of the base by the planting soils. The aggregate base should extend at least one foot outside the edge of the road where feasible.

14. SOIL EXPANSIVITY

The subsurface soils encountered at shallow depths consist mostly of sands with low cohesion. These types of material generally have a low susceptibility to expansion when facing seasonal cycles of saturation/desiccation. Consequently, the recommendations provided in this report regarding drainage, moisture content during compaction and other pertinent recommendations for site improvements should be incorporated into the design and construction. If encountered, clay should not be used for backfill below the buildings. Proper moisture conditioning is essential to control the potential expansion of clay soils.

15. SOIL CORROSIVITY

The corrosion potential of the onsite materials to steel and buried concrete was preliminarily evaluated. Laboratory testing was performed on a selected soil sample to evaluate pH, minimum resistivity, chloride and soluble sulfate content. The test results are presented in the following table.

Table 5 - Corrosion Test Results

Boring	Depth (ft)	Minimum Resistivity (ohm-cm)	pH	Soluble Sulfate Content (ppm)	Soluble Chloride Content (ppm)
B-5	1-5	7307	7.3	130	67

These tests are only an indicator of soil corrosivity for the samples tested. Other soils found on site may be more, less, or of a similar corrosive nature. Imported fill materials should be tested to confirm their corrosion potential. Based on the minimum resistivity results from the soil tested, some of the near-surface site soils are mildly corrosive towards buried ferrous metals. The concentrations of soluble sulfates indicate that the potential of sulfate attack on concrete in contact with the onsite soils is “negligible” based on ACI 318 Table 4.3.1. Cement Type II may be used in the concrete. Maximum water-cement ratios are not specified for the sulfate concentrations; however, the Structural Engineer should select a concrete with appropriate strength. Further interpretation of the corrosivity test results, including the resistivity value, and providing corrosion design and construction recommendations are the purview of corrosion specialists/consultants.

16. PERCOLATION TESTING

The drilling for the percolation holes and percolation testing were performed on a sunny day. No rain had occurred for several weeks prior to percolation testing. The locations of percolation tests were selected by a District representative. The depths of the holes for percolation tests were about 5 and 5½ feet and the locations are as shown on the Boring Location Map, Figure A-2, Appendix A.

Koury performed the tests in substantial conformance with the boring percolation test procedures of the County of Orange/County of Riverside as described in Appendix A of the Low Impact

Development BMP Design Handbook dated 9/2011. The percolation test procedure consisted of drilling 8¾-inch diameter boreholes to the test depths and placing a 2-inch layer of filter gravel at the bottom of the holes. We also placed a 3-inch diameter perforated pipe in the holes and filter rock to prevent caving.

Percolation testing started with the two pre-test readings to determine the duration of the tests and reading intervals. The pre-test readings indicated more than 6 inches of water dissipation in less than 25 minutes and the fast method of testing was followed. To determine the percolation rates, we used time intervals of 5 and 10 minutes for Percolation Borings P-2 and P-1, respectively.

The procedure involved pre-soaking the percolation zones prior to testing. The holes were filled with water and then the water level was maintained at least 5 times the hole radius above the hole bottom until an additional 5 gallons of water had percolated through the test hole. The water column heights were generally between about 1 and 3½ feet. The water level time measurements were repeated several times until consistent results were noted.

Using fixed reference points, we measured the water level drop for the time interval selected and refilled each time once the time interval was achieved. The following Table 6 summarizes the results of the two percolation tests. Detail test data for each percolation test is presented in Appendix C.

Table 6 – Summary of Falling Head Percolation Testing

Test Number	Depth (ft)	Short Term Infiltration *(in/hour)	Adjusted Long Term Infiltration (in/hour)
P-1	5.5	9.8	3.3
P-2	5.0	13.6	4.5

*No correction factor applied

The percolation tests measure the rate of water progression in the lateral and downward directions while infiltration is the rate of water progression in the downward direction only. The percolation tests include both the bottom surface and the sidewalls of the test hole surface. The conversion from percolation to infiltration data was performed using the "Porchet Method". Table 6 indicates short term infiltration rates in the range of about 9.8 to 13.6 inches per hour.

The field measured infiltration rates must be corrected for site variability, number of tests performed, extensiveness of site investigation (CF_V = 1 to 3), long-term siltation, plugging and maintenance (CF_S = 1 to 3). The design infiltration rate should be approximately equal to the measured infiltration rate divided by the correction factor ($CF = CF_V \times CF_S$). We have selected a combined correction factor of 3 and we obtained a long-term design infiltration rate on the order of 3 inches per hour.

It should be noted that maintenance will affect infiltration. Also, because the type of deposit present at the site often contains localized layers of less pervious soils such as silty sand and the water has to move laterally to bypass these layers, the infiltration may slow down if large volumes of water are disposed of.

Design Consideration

At the anticipated infiltration level, the site soils generally have a very low expansion potential and the effect of infiltration on soil expansion should be negligible. There is moderate potential for settlement of the ground surface due to hydroconsolidation, and some maintenance should be anticipated in the vicinity of the infiltration facilities. The infiltration facilities should be designed to overflow to the storm drain in the event that the drainage capacity is exceeded or in case of future failure to infiltrate sufficiently. The infiltration facilities should extend at least 5 feet below the ground surface, and should be located at least 30 feet away from buildings and 15 feet away from property lines. Utility pipelines should be located well outside the infiltration facilities or special measures should be taken to prevent water from entering the bedding and shading materials placed around utilities.

No infiltration facility should be designed to infiltrate water into fill material except if poorly graded coarse-grained clean sand or clean gravel are used as fill. Any construction method should prevent compaction of the area where infiltration is proposed. Any soil processing and compaction may reduce the infiltration by factors ranging between about 2 and 10. Excavation of infiltration facilities should be performed using an excavator; no rubber tire equipment should be allowed at the bottom of the excavations. No disturbance to the bottom of the excavations should be allowed. If silt or very fine silty sand is encountered at the bottom of the excavations, it should be removed and replaced with coarse clean sand or crushed rock. The filter rock placed at the bottom of the excavation should be vibrated in place and ample water should be used during the vibration/compaction process.

17. OBSERVATION AND TESTING

This report has been prepared assuming that Koury Engineering & Testing, Inc. will perform all geotechnical-related field observations and testing. If the recommendations presented in this report are utilized, and observation of the geotechnical work is performed by others, the party performing the observations must review this report and assume responsibility for the recommendations contained herein. That party would then assume the title of "Geotechnical Consultant of Record". A representative of the Geotechnical Consultant should be present to observe all grading operations as well as all footing excavations.

18. CLOSURE

The findings and recommendations presented in this report were based on the results of our field and laboratory investigations, combined with professional engineering experience and judgment. The report was prepared in accordance with generally accepted engineering principles and practice. We make no other warranty, either expressed or implied. Subsurface variations between borings should be anticipated. Koury should be notified if subsurface conditions are encountered, which differ from those described in this report since revised recommendations may be required. Samples obtained during this investigation will be retained in our laboratory for a period of 45 days from the date of this report and will be disposed after this period.

Should you have any questions concerning this submittal, or the recommendations contained herewith, please do not hesitate to call our office.

Respectfully submitted,

KOURY ENGINEERING & TESTING, INC.

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APPENDICES

Appendix A: Maps and Plans

Vicinity Map – Figure A-1
Boring Location Map – Figure A-2
Geology Map – Figure A-3
Historic High Groundwater Map – Figure A-4
Seismic Hazard Zones Map – Figure A-5
Fault Map – Figure A-6
Flood Map – Figure A-7
Seismic Parameters – Figure A-8
Oil and Gas Map – Figure A-9

Appendix B: Field Exploratory Boring Logs

Borings B-1 through B-13

Appendix C: Laboratory Test Results and Calculations

Appendix D: Historical Earthquake Data

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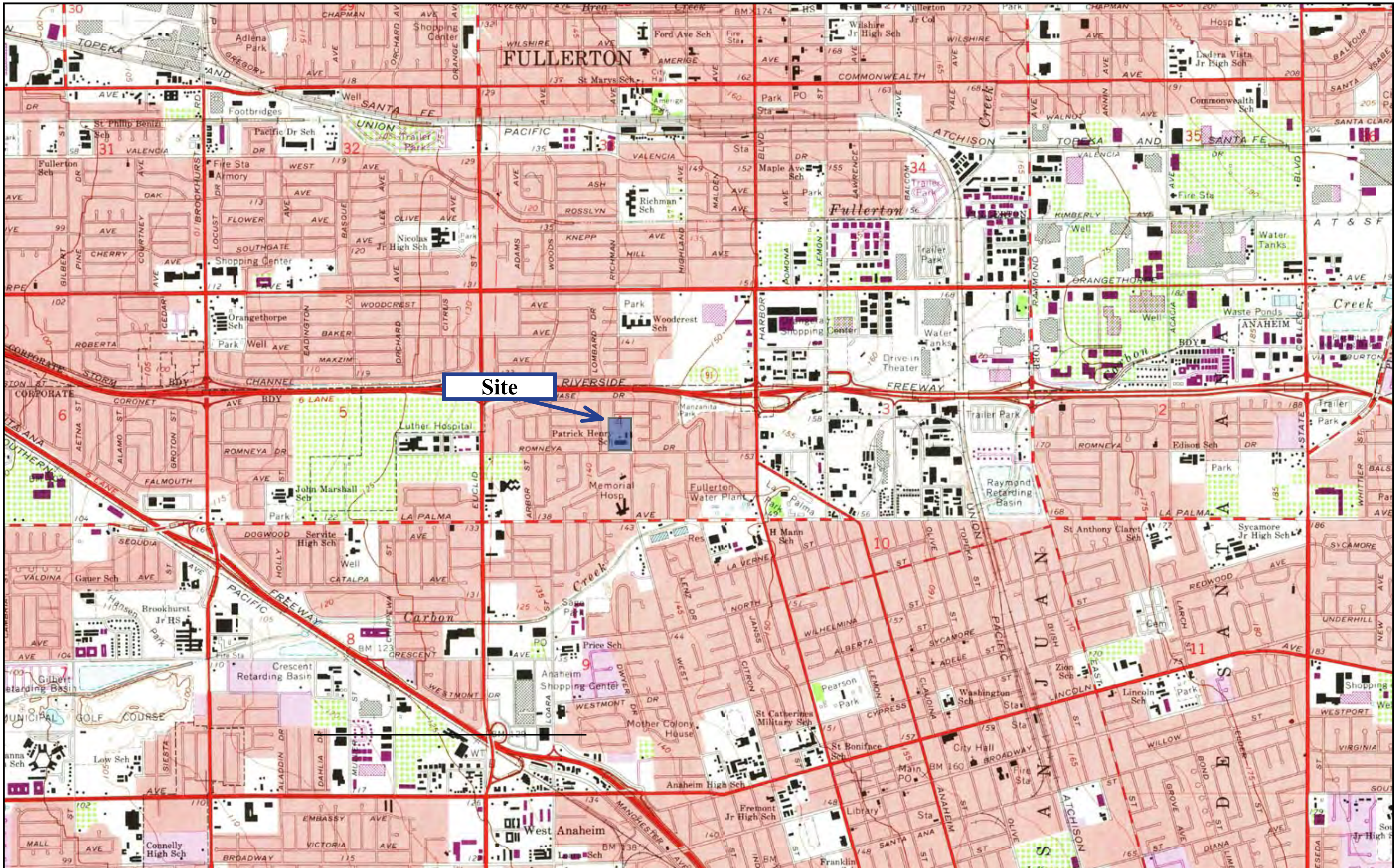
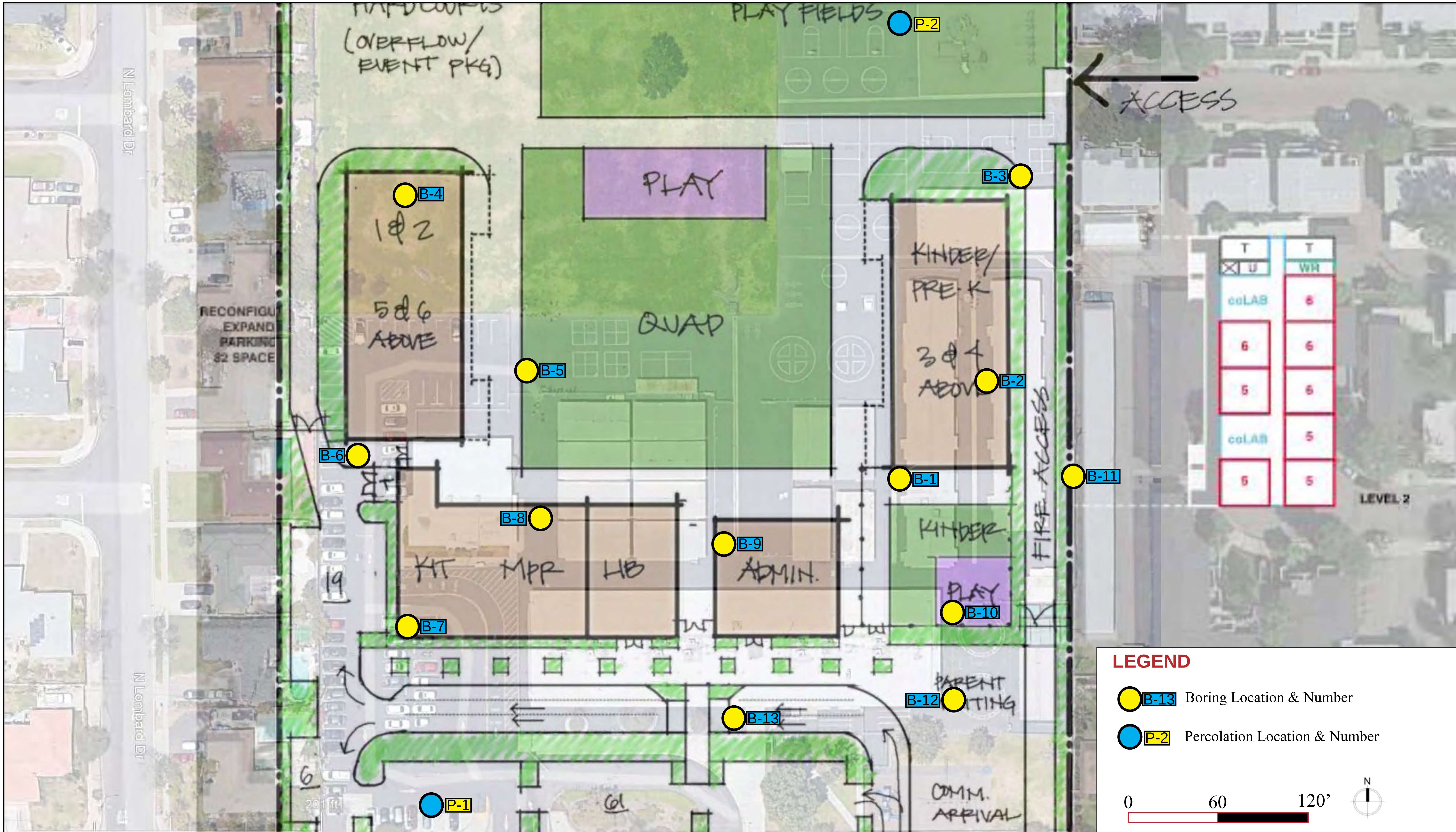
APPENDIX A

Maps and Plans



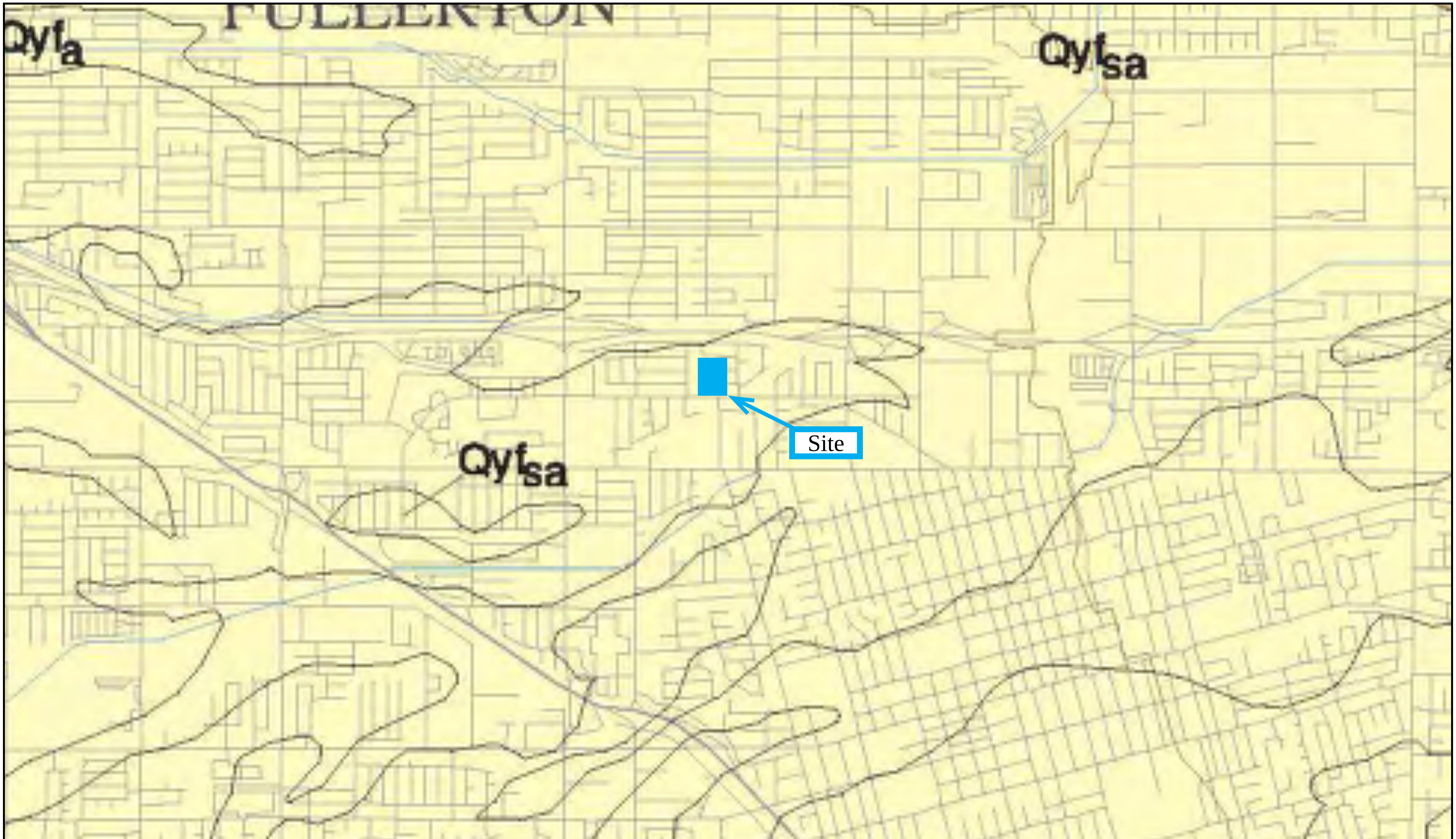
Reference: USGS Topographic Map, Anaheim Quadrangle, California- Orange County, 7.5 Minute Series, 1965 - Contour Interval 5 ft. Scale 1:24,000.

KOURY PASSION INTEGRITY QUALITY	Project Name: **Patrick Henry Elementary School Modernization**	Project No.: **22-0161** Date: **August 2022**	Drawing Title: **Vicinity Map**	Figure: **A-1**



KOURY PASSION INTEGRITY QUALITY	Project Name: Patrick Henry Elementary School Modernization	Project No.: 22-0161 Date: August 2022	Drawing Title: Boring Location Map	Figure: A-2

0 60 120'



LEGEND

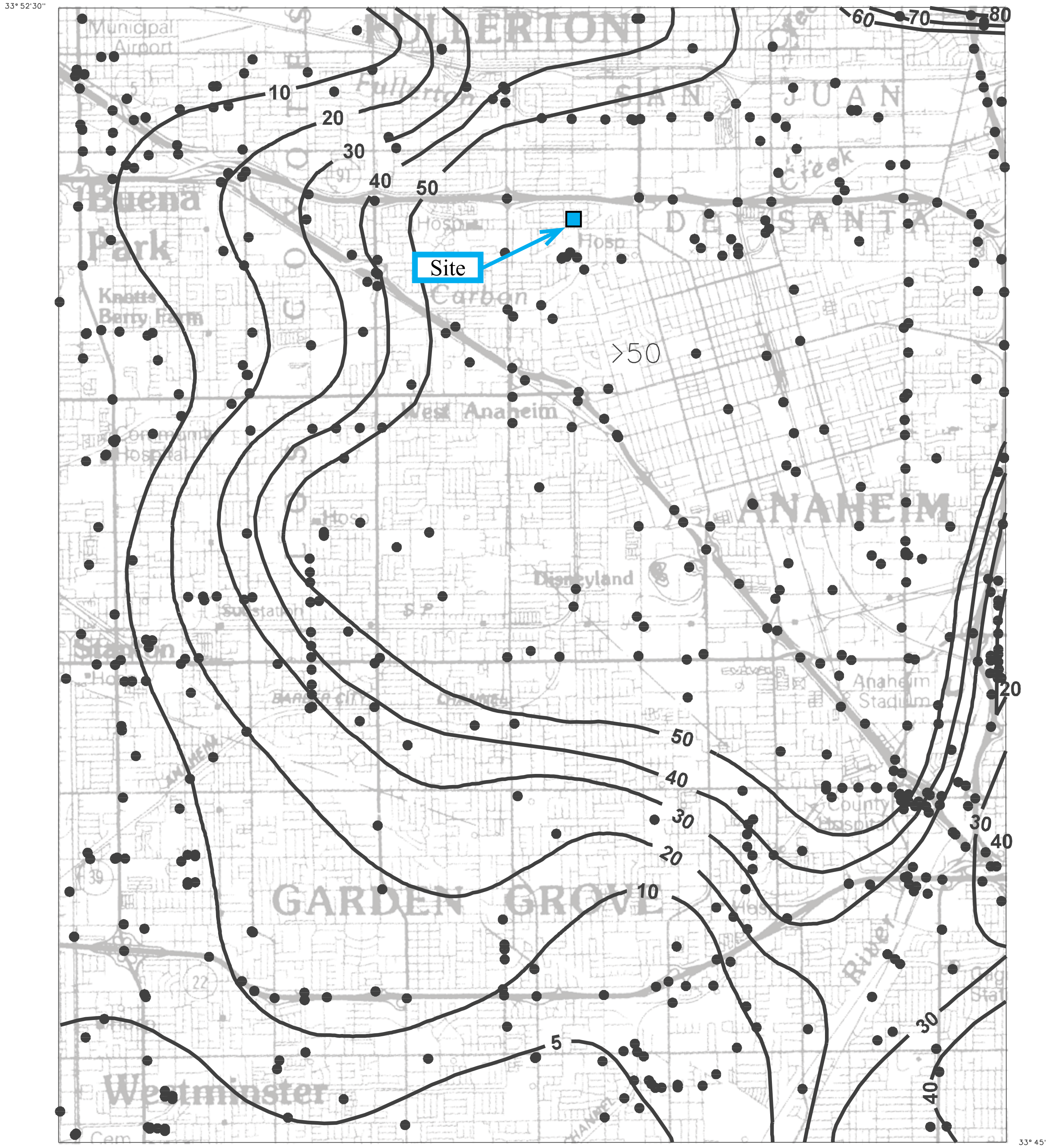
Qyfa Young alluvial fan and valley deposits, sand (Holocene & Late Pleistocene); a= arenaceous

Qyfsa Young alluvial fan and valley deposits, sand and silt (Holocene & Late Pleistocene); sa= silty sand with gravel and clay

0 0.5 1 Mile

Reference: USGS Geologic Map of the San Bernardino and Santa Ana 30' x 60' Quadrangle, California, 2006

	Project Name:	Project No.:	Drawing Title:	Figure:
KOURY PASSION INTEGRITY QUALITY	Patrick Henry Elementary School Modernization	22-0161 Date: August 2022	Geology Map	A-3



Base map enlarged from U.S.G.S. 30 x 60–minute series

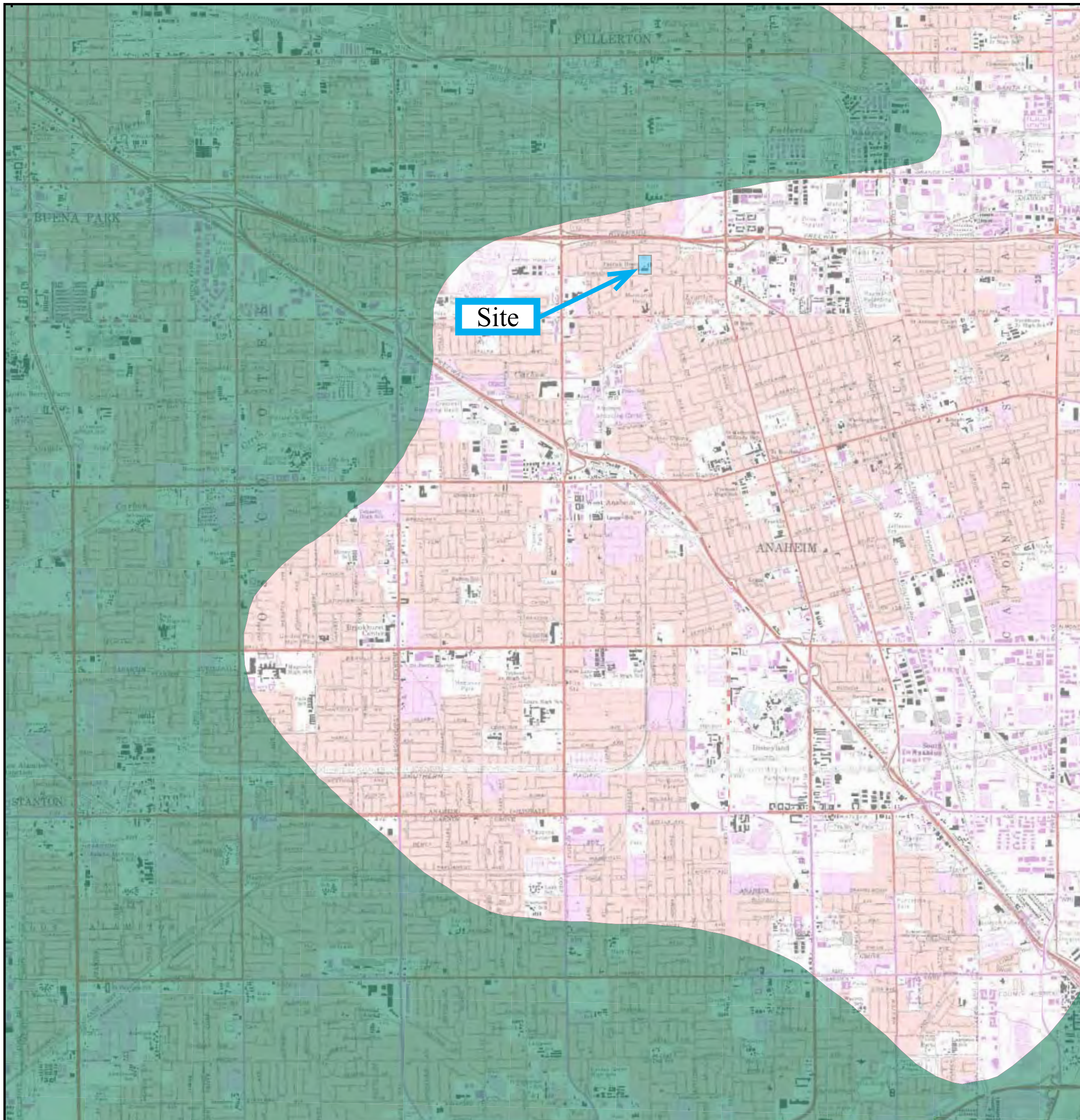
● Borehole Site

30 Depth to ground water in feet

ONE MILE
SCALE

Plate 1.2 Historically Highest Ground Water Contours and Borehole Log Data Locations, Anaheim 7.5–minute Quadrangle.

KOURY PASSION INTEGRITY QUALITY	Project Name: Patrick Henry Elementary School Modernization	Project No.: 22-0161 Date: August 2022	Drawing Title: Historic High Groundwater Map	Figure: A-4



Earthquake Zones of Required Investigation Anaheim Quadrangle

California Geological Survey

This Map Shows Seismic Hazard Zones
Alquist-Priolo Earthquake Fault Zones Have Not Been Prepared
For The Anaheim Quadrangle

MAP EXPLANATION

ADDITIONAL INFORMATION

ANAHEIM QUADRANGLE

SEISMIC HAZARD ZONES

Delineated in compliance with
Chapter 7.8 Division 2 of the California Public Resources Code
(Seismic Hazards Mapping Act)

OFFICIAL MAP

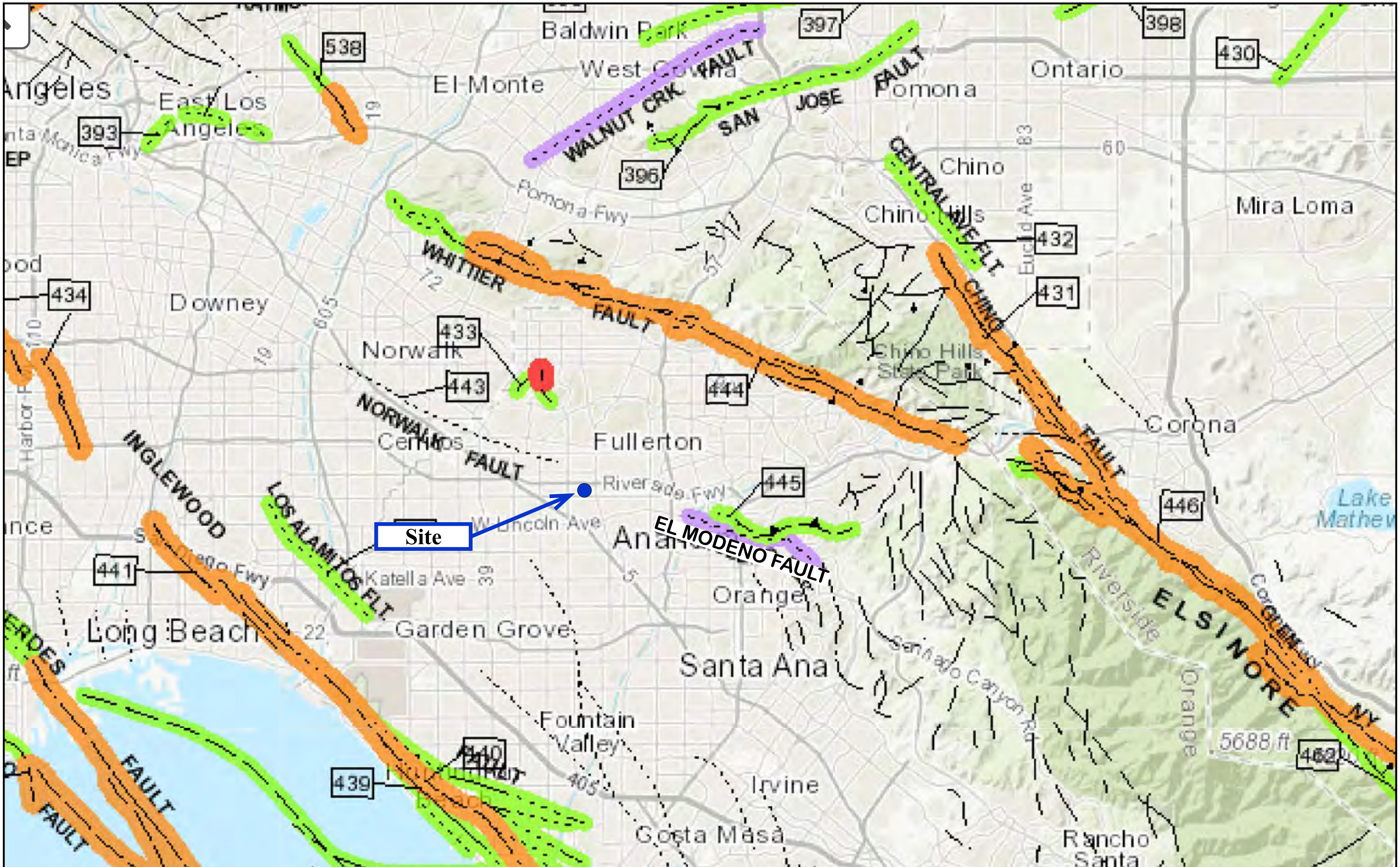
Released: April 15, 1998

STATE GEOLOGIST

0 1 Mile 2 Miles

N

KOURY PASSION INTEGRITY QUALITY	Project Name: **Patrick Henry Elementary School Modernization**	Project No.: **22-0161** Date: **August 2022**	Drawing Title: **Seismic Hazard Zones Map**	Figure: **A-5**



Reference: Fault Activity Map of California (2015) - California Geological Survey
Web Site @ http://maps.conservation.ca.gov/cgs/fam/ - See Figure 6a for explanation

0 5 Miles

N

	Project Name:	Project No.: 22-0161	Drawing Title:	Figure:
KOURY PASSION INTEGRITY QUALITY	Patrick Henry Elementary School Modernization	Date: August 2022	Fault Map	A-6

EXPLANATION

Fault traces on land are indicated by solid lines where well located, by dashed lines where approximately located or inferred, and by dotted lines where concealed by younger rocks or by lakes or bays. Fault traces are queried where continuation or existence is uncertain. Concealed faults in the Great Valley are based on maps of selected subsurface horizons, so locations shown are approximate and may indicate structural trend only. All offshore faults based on seismic reflection profile records are shown as solid lines where well defined, dashed where inferred, queried where uncertain.

FAULT CLASSIFICATION COLOR CODE (Indicating Recency of Movement)

Fault along which historic (last 200 years) displacement has occurred and is associated with one or more of the following:

(a) a recorded earthquake with surface rupture. (Also included are some well-defined surface breaks caused by ground shaking during earthquakes, e.g. extensive ground breakage, not on the White Wolf fault, caused by the Arvin-Tehachapi earthquake of 1952). The date of the associated earthquake is indicated. Where repeated surface ruptures on the same fault have occurred, only the date of the latest movement may be indicated, especially if earlier reports are not well documented as to location of ground breaks.
(b) fault creep slippage - slow ground displacement usually without accompanying earthquakes.
(c) displaced survey lines.

1906 ▶ ◀ 1906

A triangle to the right or left of the date indicates termination point of observed surface displacement. Solid red triangle indicates known location of rupture termination point. Open black triangle indicates uncertain or estimated location of rupture termination point.

1838 ▷ ◁ 1838

▶ 1951 ◀

Date bracketed by triangles indicates local fault break.

1992

No triangle by date indicates an intermediate point along fault break.

CREEP

Fault that exhibits fault creep slippage. Hachures indicate linear extent of fault creep. Annotation (creep with leader) indicates representative locations where fault creep has been observed and recorded.

1969

1968 ■ ■ 1968

Square on fault indicates where fault creep slippage has occured that has been triggered by an earthquake on some other fault. Date of causative earthquake indicated. Squares to right and left of date indicate termi- nal points between which triggered creep slippage has occurred (creep either continuous or intermittent between these end points).

Holocene fault displacement (during past 11,700 years) without historic record. Geomorphic evidence for Holocene faulting includes sag ponds, scarps showing little erosion, or the following features in Holocene age deposits: offset stream courses, linear scarps, shutter ridges, and triangular faceted spurs. Recency of faulting offshore is based on the interpreted age of the youngest strata displaced by faulting.

Late Quaternary fault displacement (during past 700,000 years). Geomorphic evidence similar to that described for Holocene faults except features are less distinct. Faulting may be younger, but lack of younger overlying deposits precludes more accurate age classification.

Quaternary fault (age undifferentiated). Most faults of this category show evidence of displacement some- time during the past 1.6 million years; possible exceptions are faults which displace rocks of undifferenti- ated Plio-Pleistocene age. Unnumbered Quaternary faults were based on Fault Map of California, 1975. See Bulletin 201, Appendix D for source data.

Pre-Quaternary fault (older that 1.6 million years) or fault without recognized Quaternary displacement. Some faults are shown in this category because the source of mapping used was of reconnaissnce nature, or was not done with the object of dating fault displacements.

ADDITIONAL FAULT SYMBOLS

Bar and ball on downthrown side (relative or apparent).

Arrows along fault indicate relative or apparent direction of lateral movement.

Arrow on fault indicates direction of dip.

Low angle fault (barbs on upper plate). Fault surface generally dips less than 45° but locally may have been subsequently steepened. On offshore faults, barbs simply indicate a reverse fault regardless of steepness of dip

OTHER SYMBOLS

491

Numbers refer to annotations listed in the appendices of the accompanying report. Annotations include fault name, age of fault displacement, and pertinent references including Earthquake Fault Zone maps where a fault has been zoned by the Alquist-Priolo Earthquake Fault Zoning Act. This Act requires the State Geolo- gist to delineate zones to encompass faults with Holocene displacement.

Structural discontinuity (offshore) separating differing Neogene structural domains. May indicate disconti- nuities between basement rocks.

Brawley Seismic Zone, a linear zone of seismicity locally up to 10 km wide associated with the releasing step between the Imperial and San Andreas faults

KOURY PASSION INTEGRITY QUALITY	Project Name: Patrick Henry Elementary School Modernization	Project No.: 22-0161 Date: August 2022	Drawing Title: Fault Map Legend	Figure: A-6a

National Flood Hazard Layer FIRMette

FEMA

117°56'19"W 33°51'20"N

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

1:6,000

117°55'42"W 33°50'50"N

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

Legend

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

SPECIAL FLOOD HAZARD AREAS
- Without Base Flood Elevation (BFE) *Zone A, V, A99*
- With BFE or Depth *Zone AE, AO, AH, VE, AR*
- Regulatory Floodway

OTHER AREAS OF FLOOD HAZARD
- 0.2% Annual Chance Flood Hazard, Areas of 1% annual chance flood with average depth less than one foot or with drainage areas of less than one square mile *Zone X*
- Future Conditions 1% Annual Chance Flood Hazard *Zone X*
- Area with Reduced Flood Risk due to Levee. See Notes. *Zone X*
- Area with Flood Risk due to Levee *Zone D*

OTHER AREAS
- NO SCREEN Area of Minimal Flood Hazard *Zone X*
- Effective LOMRs
- Area of Undetermined Flood Hazard *Zone D*

GENERAL STRUCTURES
- Channel, Culvert, or Storm Sewer
- Levee, Dike, or Floodwall

OTHER FEATURES
- 20.2 / 17.5 Cross Sections with 1% Annual Chance Water Surface Elevation
- Coastal Transect
- 513 Base Flood Elevation Line (BFE)
- Limit of Study
- Jurisdiction Boundary
- Coastal Transect Baseline
- Profile Baseline
- Hydrographic Feature

MAP PANELS
- Digital Data Available
- No Digital Data Available
- Unmapped

The pin displayed on the map is an approximate point selected by the user and does not represent an authoritative property location.

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

The flood hazard information is derived directly from the authoritative NFHL web services provided by FEMA. This map was exported on 8/10/2022 at 12:53 PM and does not reflect changes or amendments subsequent to this date and time. The NFHL and effective information may change or become superseded by new data over time.

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

KOURY PASSION INTEGRITY QUALITY	Project Name: Patrick Henry Elementary School Modernization	Project No.: 22-0161 Date: August 2022	Drawing Title: Flood Map	Figure: A-7

22-0161 Henry ES

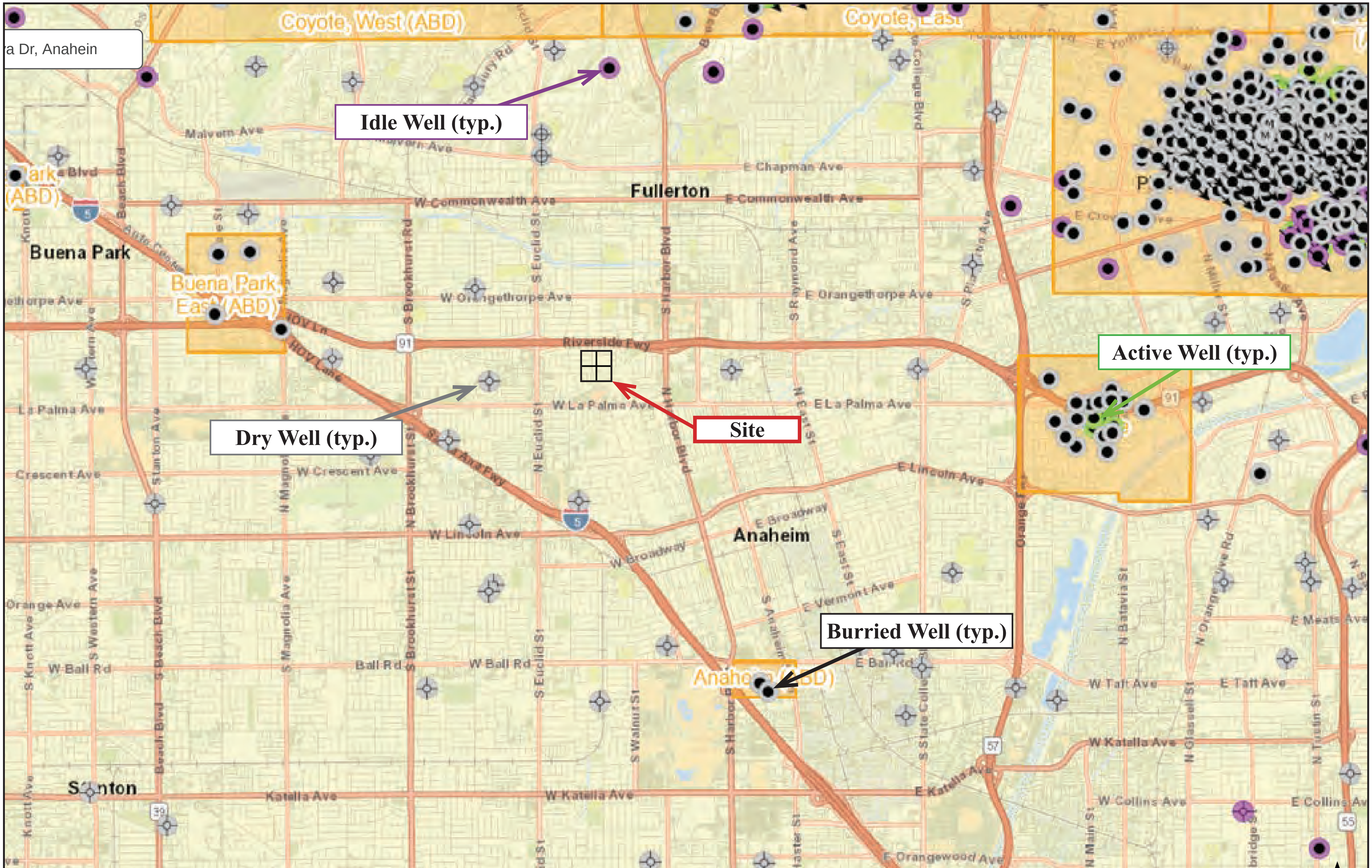
Latitude, Longitude: 33.851286, -117.932929

Date	3/11/2022, 10:36:44 AM
Design Code Reference Document	ASCE7-16
Risk Category	III
Site Class	D - Stiff Soil

Type	Value	Description
S_S	1.529	MCE_R ground motion. (for 0.2 second period)
S_1	0.539	MCE_R ground motion. (for 1.0s period)
S_{MS}	1.529	Site-modified spectral acceleration value
S_{M1}	null -See Section 11.4.8	Site-modified spectral acceleration value
S_{DS}	1.02	Numeric seismic design value at 0.2 second SA
S_{D1}	null -See Section 11.4.8	Numeric seismic design value at 1.0 second SA

Type	Value	Description
SDC	null -See Section 11.4.8	Seismic design category
F_a	1	Site amplification factor at 0.2 second
F_v	null -See Section 11.4.8	Site amplification factor at 1.0 second
PGA	0.651	MCE_G peak ground acceleration
F_{PGA}	1.1	Site amplification factor at PGA
PGA_M	0.716	Site modified peak ground acceleration
T_L	8	Long-period transition period in seconds
SsRT	1.529	Probabilistic risk-targeted ground motion. (0.2 second)
SsUH	1.676	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration
SsD	2.381	Factored deterministic acceleration value. (0.2 second)
S1RT	0.539	Probabilistic risk-targeted ground motion. (1.0 second)
S1UH	0.59	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration.
S1D	0.797	Factored deterministic acceleration value. (1.0 second)
PGAd	0.961	Factored deterministic acceleration value. (Peak Ground Acceleration)
C_{RS}	0.912	Mapped value of the risk coefficient at short periods
C_{R1}	0.914	Mapped value of the risk coefficient at a period of 1 s

KOURY PASSION INTEGRITY QUALITY	Project Name: Patrick Henry Elementary School Modernization	Project No.: 22-0161 Date: August 2022	Drawing Title: Seismic Parameters	Figure: A-8



Reference: California Department of Conservation, Division of Oil, Gas & Thermal Resources Well Finder (DOGGR)

0 1 2 Mile

N

KOURY PASSION INTEGRITY QUALITY	Project Name: Patrick Henry Elementary School Modernization	Project No.: 22-0161 Date: Augus 2022	Drawing Title: Oil & Gas Map	Figure: A-9

APPENDIX B

Field Exploratory Boring Logs

KEY TO LOGS

SOILS CLASSIFICATION

MAJOR DIVISIONS			GRAPHIC LOG	USCS SYMBOL	TYPICAL NAMES
COARSE GRAINED SOILS MORE THAN 50% OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE	GRAVELS MORE THAN 50% OF COARSE FRACTION IS LARGER THAN NO. 4 SIEVE	CLEAN GRAVELS LESS THAN 5% FINES		GW	WELL-GRADED GRAVELS, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES
				GP	POORLY-GRADED GRAVELS, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES
		GRAVELS WITH FINES MORE THAN 12% FINES		GM	SILTY GRAVELS, GRAVEL-SAND-SILT MIXTURES
				GC	CLAYEY GRAVELS, GRAVEL-SAND-CLAY MIXTURES
	SANDS 50% OR MORE OF COARSE FRACTION IS SMALLER THAN NO. 4 SIEVE	CLEAN SANDS LESS THAN 5% FINES		SW	WELL-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
				SP	POORLY-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
		SANDS WITH FINES MORE THAN 12% FINES		SM	SILTY SANDS, SAND-SILT MIXTURES
				SC	CLAYEY SANDS, SAND-CLAY MIXTURES
FINE GRAINED SOILS 50% OR MORE OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE	SILTS AND CLAYS LIQUID LIMIT IS LESS THAN 50			ML	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS OR CLAYEY SILTS WITH SLIGHT PLASTICITY
				CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
				OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS LIQUID LIMIT IS 50 OR MORE			MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SANDY OR GRAVELLY ELASTIC SILTS
				CH	INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS
				OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
HIGHLY ORGANIC SOILS				PT	PEAT AND OTHER HIGHLY ORGANIC SOILS

GRAIN SIZES

SILT AND CLAY	SAND			GRAVEL		COBBLES	BOULDERS
	FINE	MEDIUM	COARSE	FINE	COARSE		
	#200	#40	#10	#4	3/4"	3"	12"

SIEVE SIZES

KEY TO LOGS (continued)

SPT/CD BLOW COUNTS VS. CONSISTENCY/DENSITY					
FINE-GRAINED SOILS (SILTS, CLAYS, etc.)			GRANULAR SOILS (SANDS, GRAVELS, etc.)		
CONSISTENCY	*BLOWS/FOOT		RELATIVE DENSITY	*BLOWS/FOOT	
	SPT	CD		SPT	CD
SOFT	0-4	0-4	VERY LOOSE	0-4	0-8
FIRM	5-8	5-9	LOOSE	5-10	9-18
STIFF	9-15	10-18	MEDIUM DENSE	11-30	19-54
VERY STIFF	16-30	19-39	DENSE	31-50	55-90
HARD	over 30	over 39	VERY DENSE	over 50	over 90

* CONVERSION BETWEEN CALIFORNIA DRIVE SAMPLERS (CD) AND STANDARD PENETRATION TEST (SPT) BLOW COUNT HAS BEEN CALCULATED USING "FOUNDATION ENGINEERING HAND BOOK" BY H.Y. FANG. (**VALUES ARE FOR 140 Lbs HAMMER WEIGHT ONLY**)

DESCRIPTIVE ADJECTIVE VS. PERCENTAGE	
DESCRIPTIVE ADJECTIVE	PERCENTAGE REQUIREMENT
TRACE	1 - 10%
LITTLE	10 - 20%
SOME	20 - 35%
AND	35 - 50%

*THE FOLLOWING "DESCRIPTIVE TERMINOLOGY/ RANGES OF MOISTURE CONTENTS" HAVE BEEN USED FOR MOISTURE CLASSIFICATION IN THE LOGS.

APPROXIMATE MOISTURE CONTENT DEFINITION	
DEFINITION	DESCRIPTION
DRY	Dry to the touch; no observable moisture
SLIGHTLY MOIST	Some moisture but still a dry appearance
MOIST	Damp, but no visible water
VERY MOIST	Enough moisture to wet the hands
WET	Almost saturated; visible free water

Boring Log

KOURY
PASSION INTEGRITY QUALITY

Project No. : 22-0161
Project Name : Patrick Henry Elementary School

Boring No. : B-1
Sheet : 1 **of :** 1

Drilling Method : Hollow Stem 6" Auger
Sampling Method : Bulk - CD - SPT
Hammer Weight : 140 lbs **Drop Height :** 30"
Location : See Figure A-2

Ground Elevation:
Drilling Co. : One Way Drilling
Date Drilled : 7/18/2022

Sample No.	Moisture Content (%)	Dry Unit Weight (pcf)	Blows per 6"	Depth (ft)	Soil Type (USCS)	Description	Additional Tests
				0	AC/AB	3" of asphalt concrete over 5" of aggregate base	
1					SM	**FILL:** **Silty SAND;** fine to medium, medium dense, moist, light brown	
2	13.2	90	3 5 7		SM		Consolidation Direct Shear
3	7.0		4 5 7	5	SM	**ALLUVIUM:** **Silty SAND;** layers of poorly graded sand with silt**,** fine to medium, medium dense, slightly moist to moist, light brown	
4	5.8	108	8 8 8	10	SM		
5	9.0		4 7 11	15	SM		
6	2.8	115	5 13 20	20	SP-SM	**Poorly Graded SAND with SILT;** fine to medium, medium dense, slightly moist, light brown	
7			10 19 35	25	SP-SM		

End of Boring @ 26' 6"
No groundwater encountered

Unit weight based on correlations

Depth scale: 0, 5, 10, 15, 20, 25, 30, 35, 40

Bulk ☒ CD ■ SPT ☒

Boring Log

KOURY
PASSION INTEGRITY QUALITY

Project No. : 22-0161	**Boring No. :** B-2
Project Name : Patrick Henry Elementary School	**Sheet :** 1 **of :** 2
Drilling Method : Hollow Stem 6" Auger	
Sampling Method : Bulk - CD - SPT	**Ground Elevation:**
Hammer Weight : 140 lbs **Drop Height :** 30"	**Drilling Co. :** One Way Drilling
Location : See Figure A-2	**Date Drilled :** 7/18/2022

Depth (ft)	Soil Type (USCS)	Description
0	AC/AB	4" of asphalt concrete over 4" of aggregate base
	SP-SM	**FILL: Poorly Graded SAND with SILT;** fine to medium, trace of gravel, medium dense, slightly moist, light brown
5–15	SP-SM	**ALLUVIUM:** **Poorly Graded SAND with SILT;** fine to medium, trace of gravel, medium dense, slightly moist to moist, light brown
	CL	**Sandy Lean CLAY**
20–25	SP-SM	**Poorly Graded SAND with SILT;** fine to medium, medium dense, slightly moist, brown
30–40	SM	**Silty SAND;** fine to medium, medium dense, moist, brown

Sample No.	Moisture Content (%)	Dry Unit Weight (pcf)	Blows per 6"	Additional Tests
1				
2	3.2		2 3 4	
3	5.9	107	5 7 10	
4	6.6		2 5 8	#200 Wash Fines = 20%
5	5.4	107	3 6 10	
6	15.7		2 2 5	#200 Wash Fines = 50%
7	3.9		7 12 27	
8	3.5		7 9 14	#200 Wash Fines = 9%
9	13.6		9 12 18	
10	13.9		6 9 13	#200 Wash Fines = 36%

Bulk CD SPT

Boring Log

KOURY — PASSION INTEGRITY QUALITY

Project No. : 22-0161
Project Name : Patrick Henry Elementary School

Boring No. : B-2
Sheet : 2 **of :** 2

Drilling Method : Hollow Stem 6" Auger
Sampling Method : Bulk - CD - SPT
Hammer Weight : 140 lbs **Drop Height :** 30"
Location : See Figure A-2

Ground Elevation:
Drilling Co. : One Way Drilling
Date Drilled : 7/18/2022

Sample No.	Moisture Content (%)	Dry Unit Weight (pcf)	Blows per 6"	Depth (ft)	Sample Location	Graphic Log	Soil Type (USCS)	Description	Additional Tests
11	5.0		11 15 23	40	SPT		SP-SM	**Poorly Graded SAND with SILT**; fine to medium, medium dense, slightly moist to moist, light brown	
12	7.8		10 16 25	45	CD		SM	**Silty SAND;** fine to medium, layers of poorly graded sand with silt, medium dense to dense, moist, brown	
13	9.6		9 15 23	50	SPT				
				55				End of Boring @ 51' 6" No groundwater encountered Unit weight based on correlations	
				60					
				65					
				70					
				75					
				80					

Groundwater ▽ Bulk ☒ CD ■ SPT ☒

Boring Log

KOURY PASSION INTEGRITY QUALITY

Project No. : 22-0161
Project Name : Patrick Henry Elementary School

Boring No. : B-3
Sheet : 1 **of :** 1

Drilling Method : Hollow Stem 6" Auger
Sampling Method : Bulk - CD - SPT
Hammer Weight : 140 lbs **Drop Height :** 30"
Location : See Figure A-2

Ground Elevation:
Drilling Co. : One Way Drilling
Date Drilled : 7/18/2022

Sample No.	Moisture Content (%)	Dry Unit Weight (pcf)	Blows per 6"	Depth (ft)	Sample Location	Soil Type (USCS)	Description	Additional Tests
				0			3" of asphalt concrete over 5" of aggregate base	
1					Bulk	SM	**FILL:** **Silty SAND;** fine to medium**,** slightly moist, loose to medium dense, light brown	
2	1.8		4 5 7		SPT			
3	1.8	112	6 8 11	5	CD	SP-SM	**ALLUVIUM:** **Poorly Graded SAND with SILT;** fine to medium, trace gravel medium dense, dry to slightly moist, light brown	
4	11.2		5 6 6		SPT	SM	**Silty SAND;** layers of sandy silt, fine to medium, medium dense, moist, light brown to olive brown	Consolidation
5	6.2	104	7 9 13	10	CD			
6	11.1		4 7 11	15	SPT			
7	9.0		5 8 10	20	SPT			
8			6 11 15	25	CD			
				30			End of Boring @ 26' 6" No groundwater encountered Unit weight based on correlations	
				35				
				40				

Bulk ☒ CD ■ SPT ☒

Boring Log

KOURY PASSION INTEGRITY QUALITY

Project No. : 22-0161
Project Name : Patrick Henry Elementary School

Boring No. : B-4
Sheet : 1 **of :** 1

Drilling Method : Hollow Stem 6" Auger
Sampling Method : Bulk - CD - SPT
Hammer Weight : 140 lbs **Drop Height :** 30"
Location : See Figure A-2

Ground Elevation:
Drilling Co. : One Way Drilling
Date Drilled : 7/18/2022

Sample No.	Moisture Content (%)	Dry Unit Weight (pcf)	Blows per 6"	Depth (ft)	Soil Type (USCS)	Description	Additional Tests
				0		Grass over topsoil **FILL:** **Silty SAND**; fine to medium, loose, light brown	
1							
					SM	**ALLUVIUM:** **Silty SAND;** layers of poorly graded sand with silt; fine to medium, loose to medium dense, moist, light brown	
2	14.8		3 4 4	5			
3	8.6	102	4 6 8				
4	18.1		3 4 8	10		layers of sandy silt, medium dense, moist	
5	18.5		7 9 11	15	ML	**ML: Sandy SILT;** pockets of sand, stiff, moist	
6	4.9		8 11 19	20	SP-SM	**Poorly Graded SAND with SILT;** fine to medium, medium dense, slightly moist to moist, brown	
7	12.3		9 12 19	25	SM	**Silty SAND;** fine to medium, medium dense, moist, light olive brown	
8	16.2		9 10 14	30		layers of sandy silt, moist, dense	
						End of Boring at 31' 6" No groundwater encountered Unit weight based on correlations	
				35			
				40			

Bulk ☒ CD ■ SPT ☒

Boring Log

KOURY — PASSION INTEGRITY QUALITY

Project No. : 22-0161
Project Name : Patrick Henry Elementary School

Boring No. : B-5
Sheet : 1 **of :** 1

Drilling Method : Hollow Stem 6" Auger
Sampling Method : Bulk - CD - SPT
Hammer Weight : 140 lbs **Drop Height :** 30"
Location : See Figure A-2

Ground Elevation:
Drilling Co. : One Way Drilling
Date Drilled : 7/18/2022

Sample No.	Moisture Content (%)	Dry Unit Weight (pcf)	Blows per 6"	Depth (ft)	Sample Location
1				0–5	Bulk
2	10.5		2 3 3	5	SPT
3	7.1	106	5 7 10		CD
4	17.0		3 4 6	10	SPT
5	21.0		6 7 7	15	SPT
6	4.0		11 13 29	20	CD
7	4.6		7 12 18	25	SPT

Soil Type (USCS)	Description	Additional Tests
SM	Grass over topsoil **FILL:** **Silty SAND;** fine to medium, loose to medium dense, slihgtly moist, light yellowish brown	Corrosivity
SM	**ALLUVIUM:** **Silty SAND;** fine to medium, loose to medium dense, moist, light brown to light olive brown	#200 Wash Fines = 26%
ML	**Sandy SILT;** stiff, moist, light olive brown	
SP-SM	**Poorly Graded SAND with SILT;** fine to medium, medium dense to dense, slightly moist to moist, olive brown	
	End of Boring @ 26' 6" No groundwater encountered Unit weight based on correlations	

Depth (ft): 0, 5, 10, 15, 20, 25, 30, 35, 40

Bulk ☒ CD ■ SPT ☒

Boring Log

KOURY — PASSION INTEGRITY QUALITY

Project No. : 22-0161
Project Name : Patrick Henry Elementary School

Boring No. : B-6
Sheet : 1 **of :** 1

Drilling Method : Hollow Stem 6" Auger
Sampling Method : Bulk - CD - SPT
Hammer Weight : 140 lbs **Drop Height :** 30"
Location : See Figure A-2

Ground Elevation:
Drilling Co. : One Way Drilling
Date Drilled : 7/18/2022

Sample No.	Moisture Content (%)	Dry Unit Weight (pcf)	Blows per 6"	Depth (ft)	Sample Location	Soil Type (USCS)	Description	Additional Tests
				0		AC/AB	3" of asphalt concrete over 9" of aggregate base	
1					Bulk	SP-SM	**FILL:** **Poorly Graded SAND with SILT;** fine to medium, medium dense, trace of gravel, slightly moist, light brown	
2	1.3	114	3 7 11		CD			
3	0.8		5 5 8	5	SPT		**ALLUVIUM:** **Poorly Graded SAND with SILT;** fine to medium, medium dense, dry, brown	
4	8.5	102	4 5 9		CD	SM	**Silty SAND;** fine to medium, medium dense, moist, brown	
5	6.6		4 5 8	10	SPT			
6	23.2		9 8 8	15	CD	ML	**Sandy SILT;** layers of fine silty sand, stiff, moist, olive brown	
7	2.5		6 9 15	20	SPT	SP-SM	**Poorly Graded SAND with SILT;** fine to medium, medium dense, slightly moist, olive brown	
8	3.1		7 12 22	25	CD			
							End of Boring @ 26' 6" No groundwater encountered Unit weight based on correlations	
				30				
				35				
				40				

Bulk ☒ CD ■ SPT ☒

Boring Log

KOURY — PASSION INTEGRITY QUALITY

Project No. : 22-0161
Project Name : Patrick Henry Elementary School

Boring No. : B-7
Sheet : 1 **of :** 2

Drilling Method : Hollow Stem 6" Auger
Sampling Method : Bulk - CD - SPT
Hammer Weight : 140 lbs **Drop Height :** 30"
Location : See Figure A-2
Ground Elevation:
Drilling Co. : One Way Drilling
Date Drilled : 07/18/2022

Sample No.	Moisture Content (%)	Dry Unit Weight (pcf)	Blows per 6"	Depth (ft)	Soil Type (USCS)	Description	Additional Tests
				0	AC/AB	4" of asphalt concrete over 4" of aggregate base	
					SM	**FILL: Silty SAND;** fine to medium, medium dense, slightly moist, brown	
1						**Poorly Graded SAND with SILT;** fine to medium, brown	
2	2.5	110	5 6 9	5	SP-SM	**ALLUVIUM:** **Poorly Graded SAND with SILT;** fine to medium, trace of gravel, medium dense, slightly moist, brown	
3	7.9		3 4 5			**Silty SAND**; fine to medium, medium dense, moist, brown	#200 Wash Fines = 20%
4	9.6	108	6 8 14	10			
5	21.9		3 5 8	15	ML	**Sandy SILT;** stiff, moist, light brown to light olive brown	#200 Wash Fines = 67%
6	14.5		7 11 16	20	SM	**Silty SAND;** fine to medium, medium dense, moist, brown	#200 Wash Fines = 29%
7	3.4		9 12 14	25	SP-SM	**Poorly Graded SAND with SILT;** fine to medium, medium dense, slightly moist, brown	
8	13.7		8 12 19	30	SM	**Silty SAND;** fine to medium, medium dense, moist, brown	
9	15.9		8 9 9	35	ML	**Sandy SILT;** layers of silty sand, stiff, moist, light brown with light olive brown	#200 Wash Fines = 52%
				40			

Bulk ☒ CD ■ SPT ☒

Boring Log

KOURY — PASSION INTEGRITY QUALITY

Project No. : 22-0161
Project Name : Patrick Henry Elementary School

Boring No. : B-7
Sheet : 2 **of :** 2

Drilling Method : Hollow Stem 6" Auger
Sampling Method : Bulk - CD - SPT
Hammer Weight : 140 lbs **Drop Height :** 30"
Location : See Figure A-2

Ground Elevation:
Drilling Co. : One Way Drilling
Date Drilled : 07/18/2022

Sample No.	Moisture Content (%)	Dry Unit Weight (pcf)	Blows per 6"	Depth (ft)	Sample Location	Graphic Log	Soil Type (USCS)	Description	Additional Tests
10	20.4		6 11 16	40	SPT		SM	**Silty SAND;** fine to medium, medium dense to dense, moist, olive brown	
11			8 17 29	45	CD			layers of sandy clay	
12	21.9		10 12 25	50	SPT				
				55				End of Boring @ 51' 6" No groundwater encountered Unit weight based on correlations	
				60					
				65					
				70					
				75					
				80					

Groundwater Bulk CD SPT

Boring Log

KOURY — PASSION INTEGRITY QUALITY

Project No. : 22-0161
Project Name : Patrick Henry Elementary School

Boring No. : B-8
Sheet : 1 **of :** 1

Drilling Method : Hollow Stem 6" Auger
Sampling Method : Bulk - CD - SPT
Hammer Weight : 140 lbs **Drop Height :** 30"
Location : See Figure A-2

Ground Elevation:
Drilling Co. : One Way Drilling
Date Drilled : 07/19/2022

Sample No.	Moisture Content (%)	Dry Unit Weight (pcf)	Blows per 6"	Depth (ft)	Sample Location	Soil Type (USCS)	Description	Additional Tests
				0		SM	4" of asphalt concrete over 6" of aggregate base **FILL:** **Silty SAND;** fine to medium, medium dense, moist, brown	
1					Bulk			
2	2.3		5 11 12		CD	SP-SM	**Poorly Graded SAND with SILT;** brown	
				5		SP-SM	**ALLUVIUM:** **Poorly Graded SAND with SILT;** fine to medium, medium dense, slightly moist, very light brown	
3	2.7		4 5 7		SPT			
4	8.8	102	4 6 9		CD	SM	**Silty SAND;** fine to medium, medium dense, moist, brown to light brown	Consolidation
5	6.4		5 5 7	10	SPT			
6	10.3		13 9 13	15	CD			
						SP-SM	**Poorly Graded SAND with SILT;** fine to medium, medium dense, slightly moist, brown	
7	4.1		12 16 21	20	SPT			
				25			End of Boring at 21' 6" No groundwater encountered	
				30				
				35				
				40				

Bulk ☒ CD ■ SPT ☒

Boring Log

Project No. : 22-0161
Project Name : Patrick Henry Elementary School

Boring No. : B-9
Sheet : 1 **of :** 1

Drilling Method : Hollow Stem 6" Auger
Sampling Method : Bulk - CD - SPT
Hammer Weight : 140 lbs **Drop Height :** 30"
Location : See Figure A-2

Ground Elevation:
Drilling Co. : One Way Drilling
Date Drilled : 07/19/2022

Sample No.	Moisture Content (%)	Dry Unit Weight (pcf)	Blows per 6"	Depth (ft)	Soil Type (USCS)	Description	Additional Tests
				0	AC/AB	2 1/2" of asphalt concrete over 6" of aggregate base	
1					SM	**FILL: Silty SAND;** fine to medium, medium dense, moist, brown	
2	4.1		6 6 8				
					SP-SM	**ALLUVIUM: Poorly Graded SAND with SILT;** fine to medium, medium dense, slightly moist to moist, tan brown	
3	2.5	113	8 9 12	5		trace of gravel **Poorly Graded SAND with SILT;** layers of silty sand, moist, light brown to tan brown	
4	6.7		3 4 7				
5	5.6		4 6 12	10		fine to coarse sand layers	
					SM	**Silty SAND;** fine to medium, medium dense, moist, light brown to brown	
6	9.2		5 7 9	15			
						End of Boring @ 16' 6" No groundwater encountered Unit weight based on correlations	
				20			
				25			
				30			
				35			
				40			

Bulk CD SPT

Boring Log

KOURY — PASSION INTEGRITY QUALITY

Project No. : 20-2094
Project Name : Patrick Henry Elementary School

Boring No. : B-10
Sheet : 1 **of :** 1

Drilling Method : Hollow Stem 6" Auger
Sampling Method : Bulk - CD - SPT
Hammer Weight : 140 lbs **Drop Height :** 30"
Location : See Figure A-2

Ground Elevation:
Drilling Co. : One Way Drilling
Date Drilled : 07/19/2022

Sample No.	Moisture Content (%)	Dry Unit Weight (pcf)	Blows per 6"	Depth (ft)	Sample Location	Graphic Log	Soil Type (USCS)	Description	Additional Tests
				0			AC/AB	3" of asphalt concrete over 6" of aggregate base	
1					Bulk		SM	**FILL:** **Silty SAND;** fine to medium, medium dense, moist, brown	
2	1.8	112	5 7 11	5	CD		SP-SM	**ALLUVIUM: Poorly Graded SAND with SILT;** fine to medium, medium dense, slightly moist, tan brown	
3	19.8		3 5 5		SPT		SM	**Silty SAND;** fine to medium, layers of sandy silt, medium dense, moist, brown	
4	5.5		7 9 13	10	CD		SP-SM	**Poorly Graded SAND with SILT;** fine to medium, medium dense, slightly moist, tan brown	
5	15.2		4 6 6	15	SPT		SM	**Silty SAND**; fine to medium, medium dense, moist, brown	
6			9 11 22	20	CD		SP-SM	**Poorly Graded SAND with SILT;** fine to medium, medium dense, slightly moist, brown	
				25				End of Boring at 26' 6" No groundwater encountered Unit weight based on correlations	
				30					
				35					
				40					

Bulk ☒ CD ■ SPT ☒

Boring Log

KOURY — PASSION INTEGRITY QUALITY

Project No. : 22-0161
Project Name : Patrick Henry Elementary School

Boring No. : B-11
Sheet : 1 **of :** 1

Drilling Method : Hollow Stem 6" Auger
Sampling Method : Bulk - CD - SPT
Hammer Weight : 140 lbs **Drop Height :** 30"
Location : See Figure A-2

Ground Elevation:
Drilling Co. : One Way Drilling
Date Drilled : 07/19/2022

Sample No.	Moisture Content (%)	Dry Unit Weight (pcf)	Blows per 6"	Depth (ft)	Sample Location	Graphic Log	Soil Type (USCS)	Description	Additional Tests
				0			AC/AB	5" of asphalt concrete over 5" of aggregate base	
								FILL: **Poorly Graded SAND with SILT;** fine to medium, tan brown	
1				5	Bulk			**ALLUVIUM:** **Poorly Graded SAND with SILT**; fine to medium, medium dense, slightly moist, tan brown	
				10				End of Boring @ 6' 6" No groundwater encountered Unit weight based on correlations	
				15					
				20					
				25					
				30					
				35					
				40					

Bulk ☒ CD ■ SPT ☒

Boring Log

KOURY
PASSION INTEGRITY QUALITY

Project No. : 22-0161
Project Name : Patrick Henry Elementary School

Boring No. : B-12
Sheet : 1 **of :** 1

Drilling Method : Hollow Stem 6" Auger
Sampling Method : Bulk - CD - SPT
Hammer Weight : 140 lbs **Drop Height :** 30"
Location : See Figure A-2

Ground Elevation:
Drilling Co. : One Way Drilling
Date Drilled : 07/19/2022

Sample No.	Moisture Content (%)	Dry Unit Weight (pcf)	Blows per 6"	Depth (ft)	Sample Location	Graphic Log	Soil Type (USCS)	Description	Additional Tests
				0			AC/AB	2.5" of asphalt concrete over 5" of aggregate base	
1					Bulk			**FILL:** **Poorly Graded SAND with SILT;** fine to medium, tan brown	
2	1.7		4 4 7	5	SPT		SP-SM	**ALLUVIUM:** **Poorly Graded SAND with SILT**; fine to medium, medium dense, slightly moist, tan brown	
3			2 4 6		SPT				
4	15.1	101	5 8 14	10	CD		SM	**Silty SAND;** layers of sandy clay, medium dense, moist, brown	
5	8.6		5 7 9	15	SPT				
								End of Boring @ 16' 6" No groundwater encountered Unit weight based on correlations	
				20					
				25					
				30					
				35					
				40					

Bulk ☒ CD ■ SPT ☒

Boring Log

KOURY — PASSION INTEGRITY QUALITY

Project No. : 22-0161
Project Name : Patrick Henry Elementary School

Boring No. : B-13
Sheet : 1 **of :** 1

Drilling Method : Hollow Stem 6" Auger
Sampling Method : Bulk - CD - SPT
Hammer Weight : 140 lbs **Drop Height :** 30"
Location : See Figure A-2

Ground Elevation:
Drilling Co. : One Way Drilling
Date Drilled : 07/19/2022

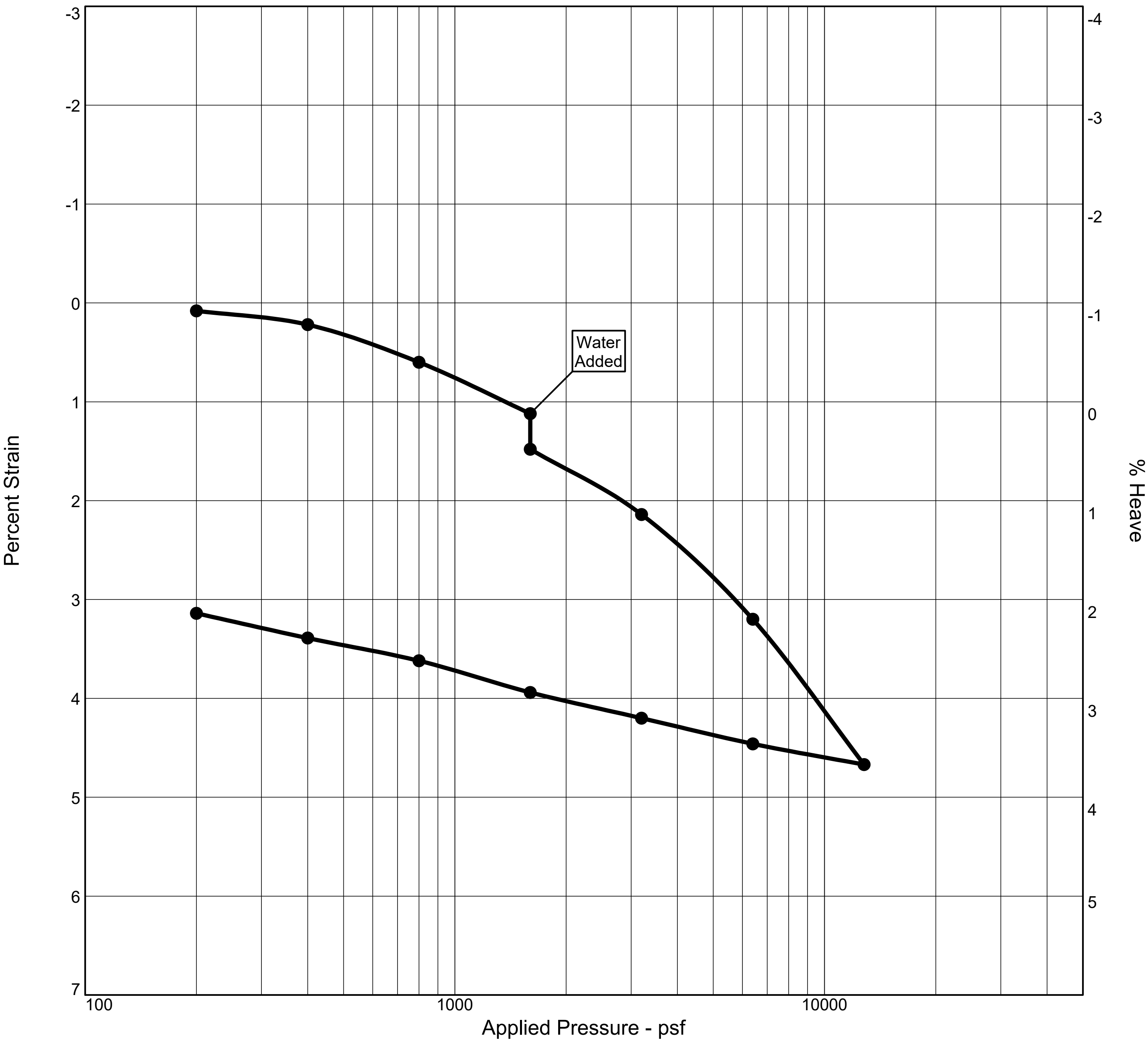
Sample No.	Moisture Content (%)	Dry Unit Weight (pcf)	Blows per 6"	Depth (ft)	Sample Location	Graphic Log	Soil Type (USCS)	Description	Additional Tests
				0			AC/AB	3" of asphalt concrete over 5" of aggregate base	
1					Bulk			**FILL: Silty SAND;** fine to medium, medium dense, moist, light brown	
2	6.9		4 3 4		SPT		SM	**ALLUVIUM:** **Silty SAND**; medium dense, slightly moist to moist, light brown	
3	6.0	101	2 3 6	5	CD				
4	4.8		2 8 4		SPT		SP-SM	**Poorly Graded SAND with SILT;** fine to medium, layers of silty sand, slightly moist, light brown	
5	4.8		6 8 12	10	CD				
				15				End of Boring @ 11' 6" No groundwater encountered Unit weight based on correlations	
				20					
				25					
				30					
				35					
				40					

Bulk ☒ CD ■ SPT ☒

APPENDIX C

Laboratory Test Results and Calculations

CONSOLIDATION TEST REPORT



Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	Overburden (psf)	P_c (psf)	C_c	C_s	Swell Press. (psf)	Heave %	e_o
Sat.	Moist.											
40.7 %	13.2 %	89.7			2.7		4299	0.09	0.02		-0.4	0.878

MATERIAL DESCRIPTION	USCS	AASHTO
Observed as: SM, fine sand brown	Observed as: SM	

Project No. 22-0161 **Client:**

Project: Patrick Henry ES Modernization

Location: B1 @ 2' **Sample Number:** 2022-530 Series

Koury Engineering & Testing, Inc.

Chino, CA

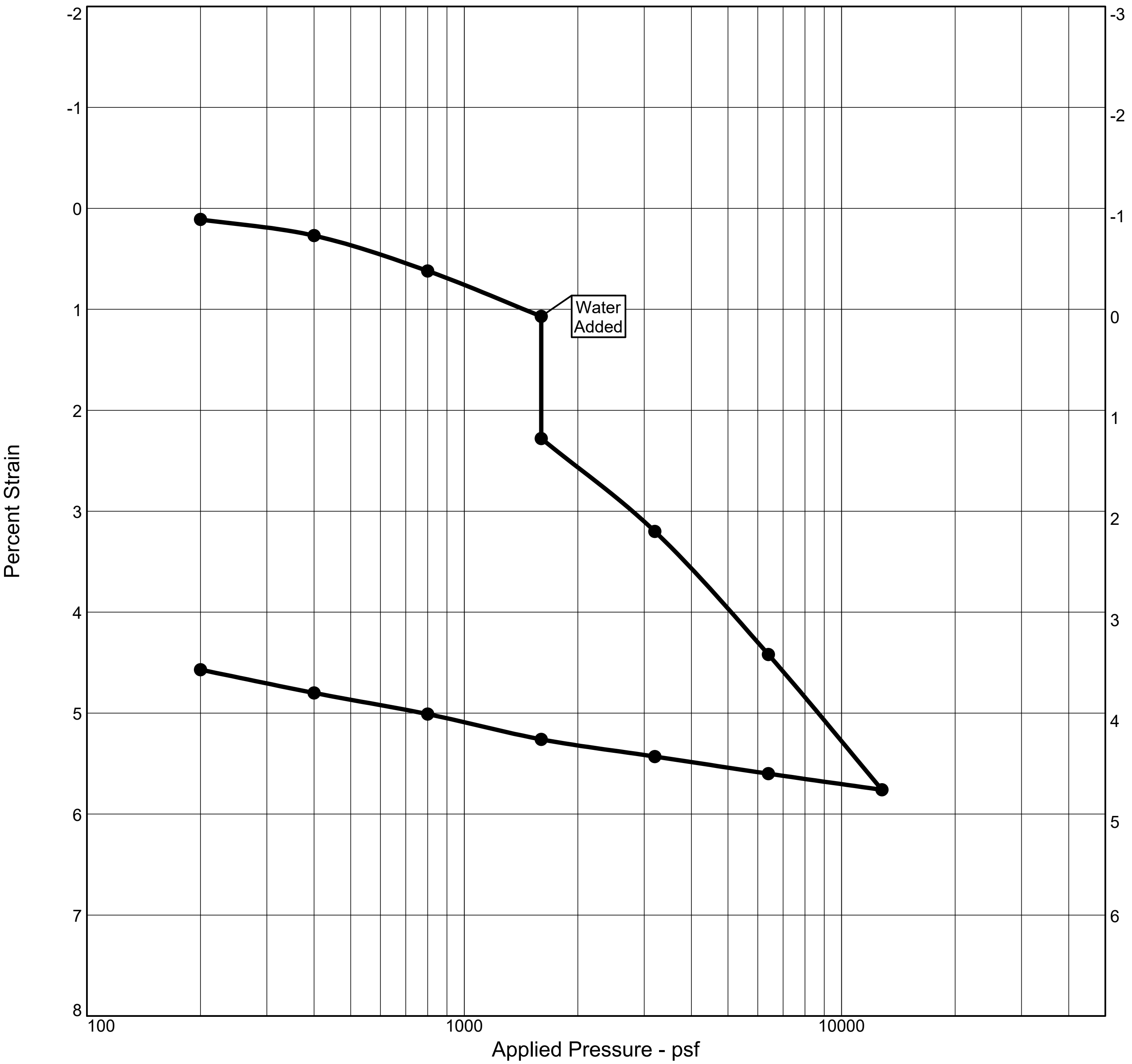
Remarks:

Lab #8414.

Figure

Tested By: Kevin Beath **Checked By:**

CONSOLIDATION TEST REPORT



Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	Overburden (psf)	P_c (psf)	C_c	C_s	Swell Press. (psf)	Heave %	e_o
Sat.	Moist.											
27.2 %	6.2 %	104.2			2.7		3576	0.07	0.01		-1.2	0.618

MATERIAL DESCRIPTION	USCS	AASHTO
Observed as: SM, fine sand, trace gravel, brown	Observed as: SM	

Project No. 22-0161 **Client:**

Project: Patrick Henry ES Modernization

Location: B3 @ 10' **Sample Number:** 2022-530 Series

Koury Engineering & Testing, Inc.

Chino, CA

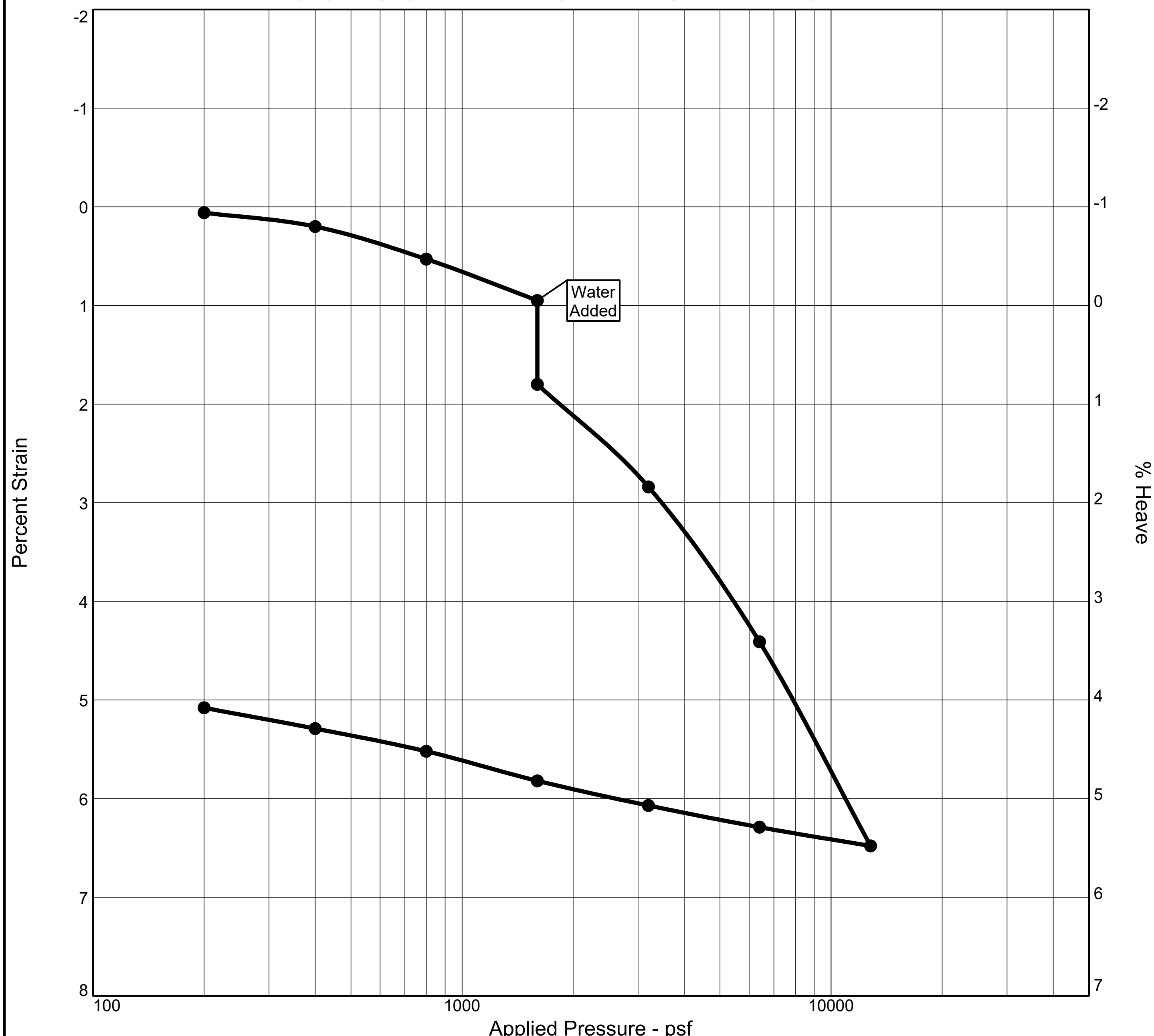
Remarks:

Lab #8414.

Figure

Tested By: Kevin Beath **Checked By:**

CONSOLIDATION TEST REPORT



Natural Sat.	Natural Moist.	Dry Dens. (pcf)	LL	PI	Sp. Gr.	Overburden (psf)	P_c (psf)	C_c	C_s	Swell Press. (psf)	Heave %	e_o
36.1 %	8.8 %	101.5			2.7		4169	0.12	0.01		-0.9	0.661

MATERIAL DESCRIPTION	USCS	AASHTO
Observed as: SM, fine sand, brown	Observed as: SM	

Project No. 22-0161 **Client:**

Project: Patrick Henry ES Modernization

Location: B8 @ 8' **Sample Number:** 2022-530 Series

Koury Engineering & Testing, Inc.

Chino, CA

Remarks:

Lab #8414.

Figure

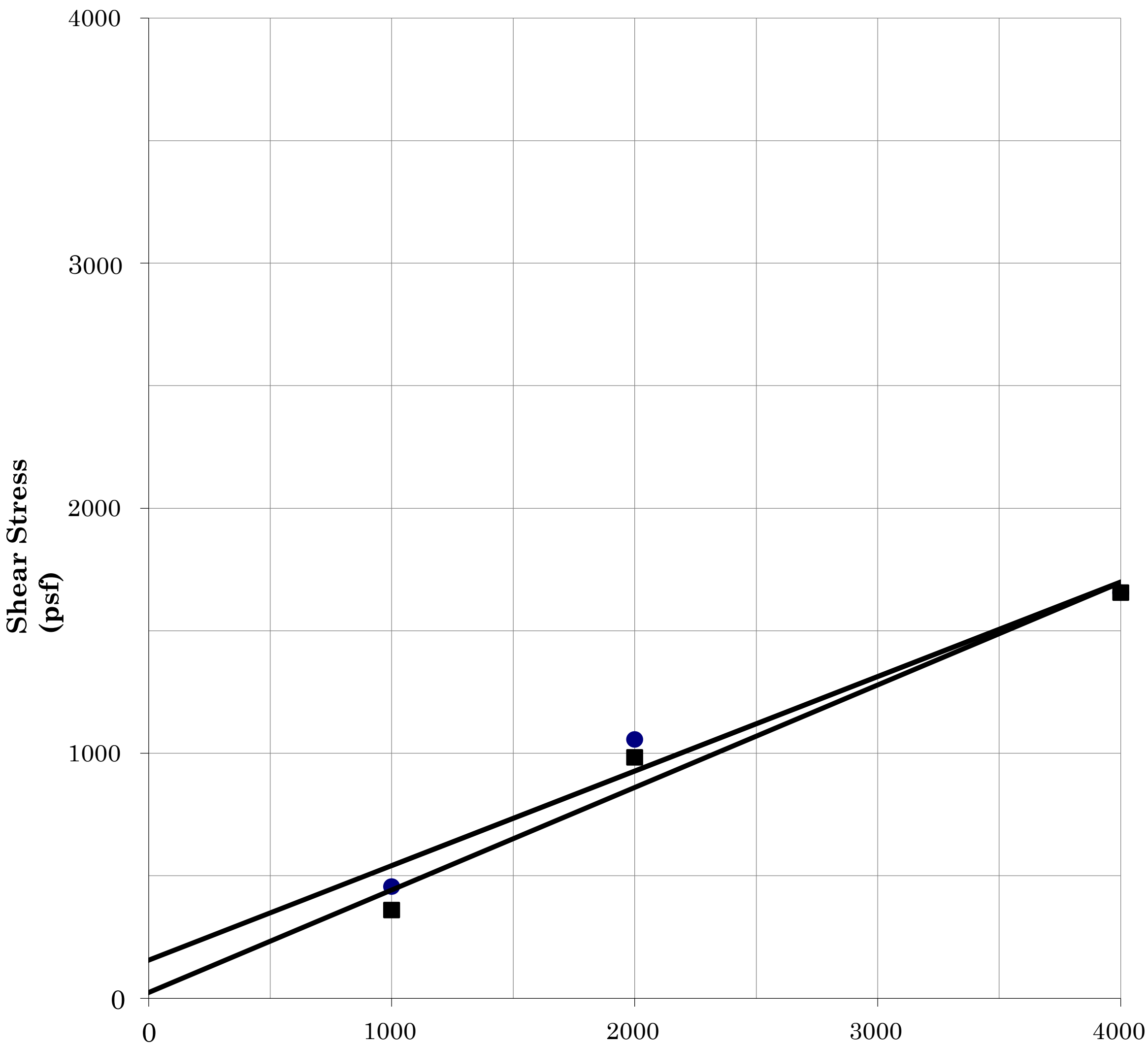
Tested By: Kevin Beath **Checked By:**

Direct Shear Test Report

Sample Identification	Sample Description	Sample Test State
B1 @ 2'	Silty Fine sand, brown (SM)	Saturated-Consolidated

Peak:	Phi (Degrees)	21.0	(Avg.Dry Dens. = 92.1 pcf) (Avg.Moist. = 17.2%)
	Cohesion (PSF)	156.0	
Ultimate:	Phi (Degrees)	23.0	
	Cohesion (PSF)	24.0	

☑ **Relatively Undisturbed**
☐ **Remolded**



	Project Name:	Project No.: 22-0161	Lab #
KOURY PASSION INTEGRITY QUALITY	Patrick Henry ES Modernization	Date: 8/05/22	8415

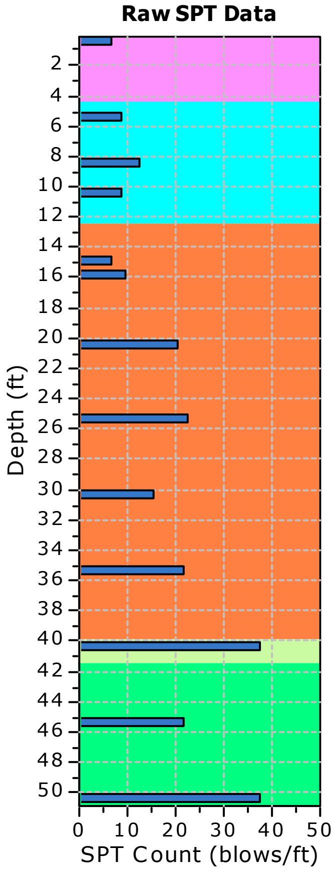
SPT BASED LIQUEFACTION ANALYSIS REPORT

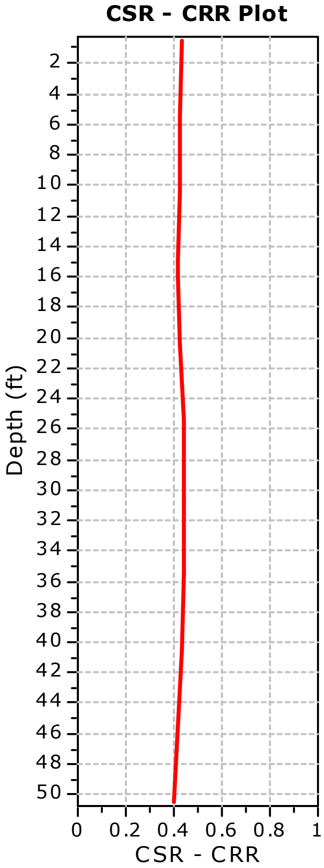
Project title : 22-0161 Patrick Henry E.S. **SPT Name: B-2**

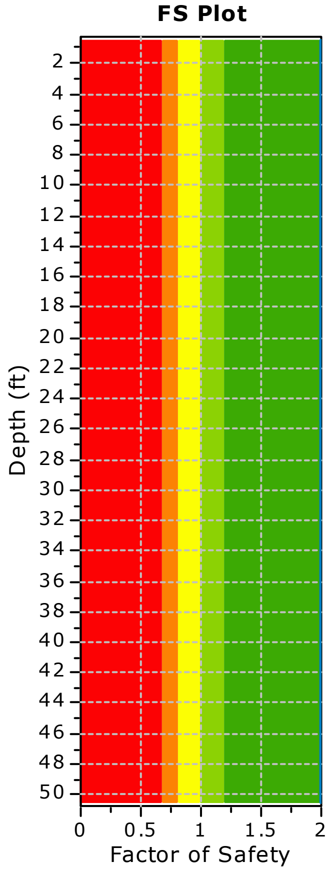
Location :

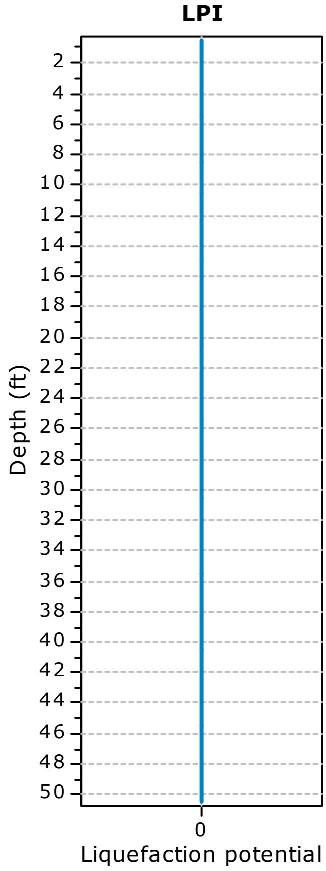
:: Input parameters and analysis properties ::

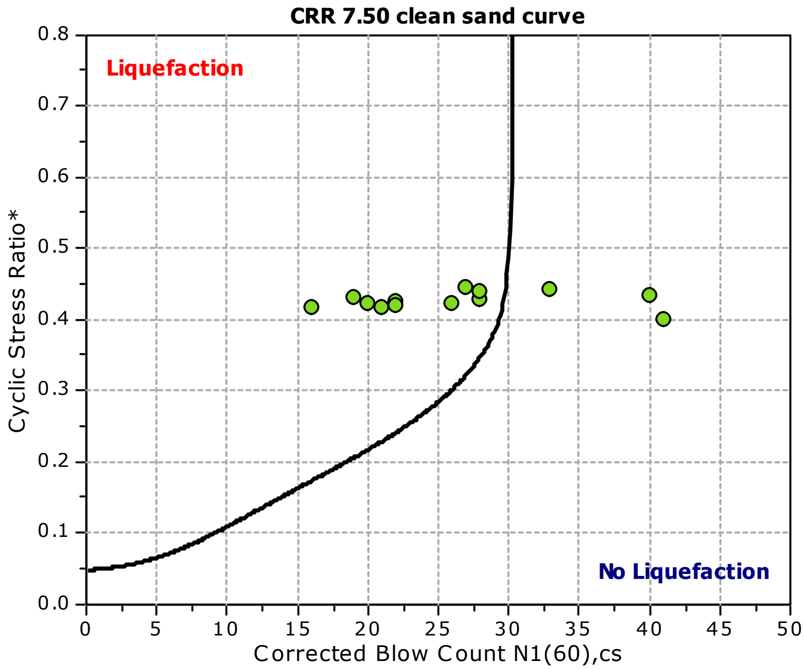
Analysis method:	NCEER 1998	G.W.T. (in-situ):	60.00 ft
Fines correction method:	NCEER 1998	G.W.T. (earthq.):	60.00 ft
Sampling method:	Sampler wo liners	Earthquake magnitude M_w:	7.30
Borehole diameter:	65mm to 115mm	Peak ground acceleration:	0.71 g
Rod length:	3.30 ft	Eq. external load:	0.00 tsf
Hammer energy ratio:	1.25		











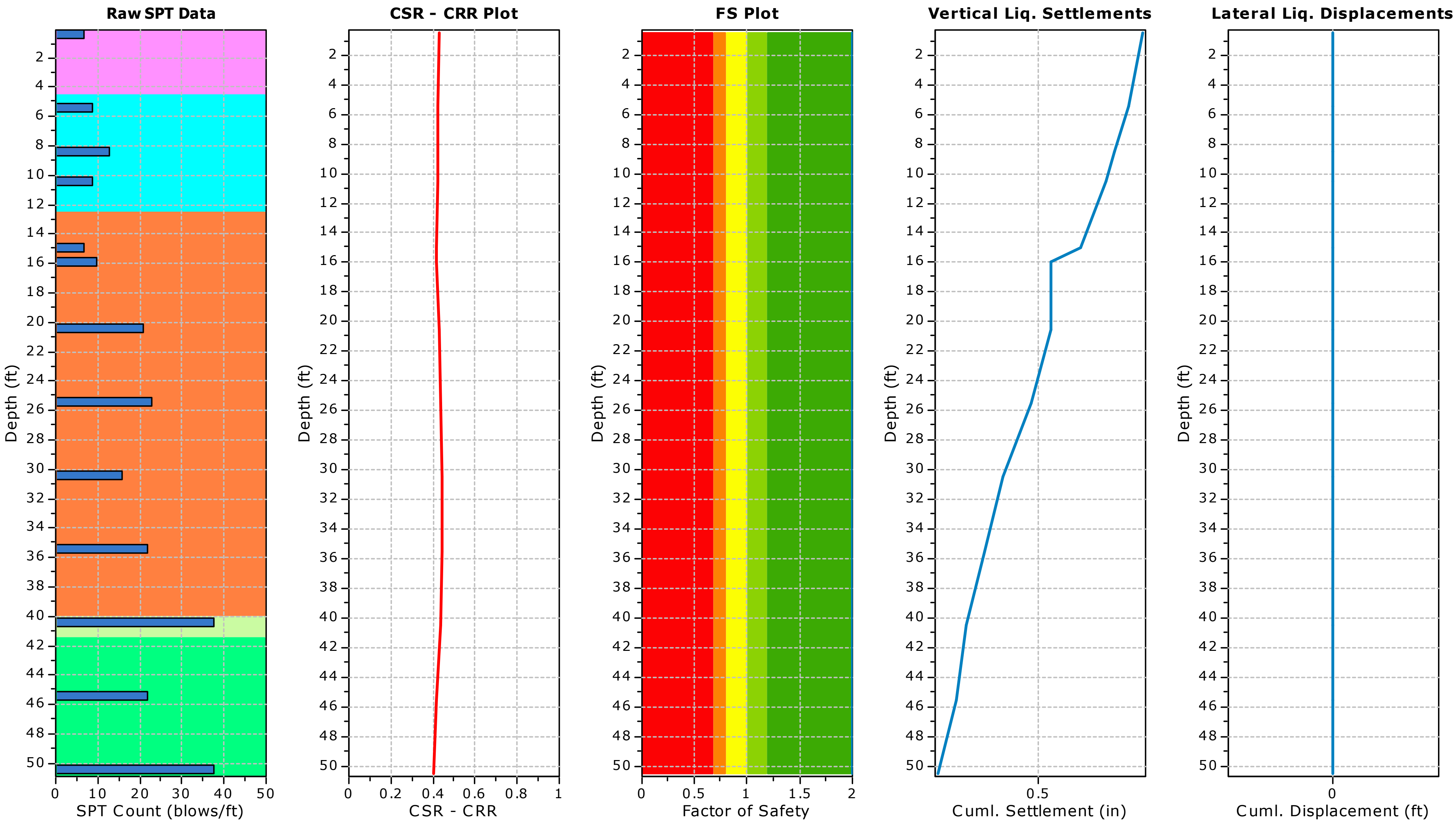
F.S. color scheme

- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

LPI color scheme

- Very high risk
- High risk
- Low risk

:: Overall Liquefaction Assessment Analysis Plots ::



:: Field input data ::

Test Depth (ft)	SPT Field Value (blows)	Fines Content (%)	Unit Weight (pcf)	Infl. Thickness (ft)	Can Liquefy
0.50	7	25.00	125.00	3.50	Yes
5.50	9	20.00	125.00	3.50	Yes
8.50	13	20.00	125.00	2.50	Yes
10.50	9	20.00	125.00	4.00	Yes
15.00	7	48.00	125.00	2.50	Yes
16.00	10	51.00	125.00	2.00	No
20.50	21	9.00	125.00	5.00	Yes
25.50	23	9.00	125.00	7.00	Yes
30.50	16	36.00	125.00	4.00	Yes
35.50	22	36.00	125.00	6.00	Yes
40.50	38	12.00	125.00	5.00	Yes
45.50	22	12.00	125.00	3.00	Yes
50.50	38	25.00	125.00	3.50	Yes

Abbreviations

Depth: Depth at which test was performed (ft)
SPT Field Value: Number of blows per foot
Fines Content: Fines content at test depth (%)
Unit Weight: Unit weight at test depth (pcf)
Infl. Thickness: Thickness of the soil layer to be considered in settlements analysis (ft)
Can Liquefy: User defined switch for excluding/including test depth from the analysis procedure

:: Cyclic Resistance Ratio (CRR) calculation data ::

Depth (ft)	SPT Field Value	Unit Weight (pcf)	σ_v (tsf)	u_o (tsf)	σ'_{vo} (tsf)	C_N	C_E	C_B	C_R	C_S	$(N_1)_{60}$	Fines Content (%)	α	β	$(N_1)_{60cs}$	$CRR_{7.5}$
0.50	7	125.00	0.03	0.00	0.03	1.70	1.25	1.00	0.75	1.20	13	25.00	4.29	1.11	19	4.000
5.50	9	125.00	0.34	0.00	0.34	1.70	1.25	1.00	0.75	1.20	17	20.00	3.61	1.08	22	4.000
8.50	13	125.00	0.53	0.00	0.53	1.41	1.25	1.00	0.75	1.20	21	20.00	3.61	1.08	26	4.000
10.50	9	125.00	0.66	0.00	0.66	1.27	1.25	1.00	0.85	1.20	15	20.00	3.61	1.08	20	4.000
15.00	7	125.00	0.94	0.00	0.94	1.06	1.25	1.00	0.85	1.20	9	48.00	5.00	1.20	16	4.000
16.00	10	125.00	1.00	0.00	1.00	1.03	1.25	1.00	0.85	1.20	13	51.00	5.00	1.20	21	4.000
20.50	21	125.00	1.28	0.00	1.28	0.91	1.25	1.00	0.95	1.20	27	9.00	0.56	1.02	28	4.000
25.50	23	125.00	1.59	0.00	1.59	0.81	1.25	1.00	0.95	1.20	27	9.00	0.56	1.02	28	4.000
30.50	16	125.00	1.91	0.00	1.91	0.75	1.25	1.00	1.00	1.20	18	36.00	5.00	1.20	27	4.000
35.50	22	125.00	2.22	0.00	2.22	0.69	1.25	1.00	1.00	1.20	23	36.00	5.00	1.20	33	4.000
40.50	38	125.00	2.53	0.00	2.53	0.65	1.25	1.00	1.00	1.20	37	12.00	1.55	1.03	40	4.000
45.50	22	125.00	2.84	0.00	2.84	0.61	1.25	1.00	1.00	1.20	20	12.00	1.55	1.03	22	4.000
50.50	38	125.00	3.16	0.00	3.16	0.58	1.25	1.00	1.00	1.20	33	25.00	4.29	1.11	41	4.000

Abbreviations

σ_v: Total stress during SPT test (tsf)
u_o: Water pore pressure during SPT test (tsf)
σ'_{vo}: Effective overburden pressure during SPT test (tsf)
C_N: Overburden corretion factor
C_E: Energy correction factor
C_B: Borehole diameter correction factor
C_R: Rod length correction factor
C_S: Liner correction factor
$N_{1(60)}$: Corrected N_{SPT} to a 60% energy ratio
α, β: Clean sand equivalent clean sand formula coefficients
$N_{1(60)cs}$: Corected $N_{1(60)}$ value for fines content
$CRR_{7.5}$: Cyclic resistance ratio for M=7.5

:: Cyclic Stress Ratio calculation (CSR fully adjusted and normalized) ::

Depth (ft)	Unit Weight (pcf)	$\sigma_{v,eq}$ (tsf)	$u_{o,eq}$ (tsf)	$\sigma'_{vo,eq}$ (tsf)	r_d	α	CSR	MSF	$CSR_{eq,M=7.5}$	K_{sigma}	CSR^*	FS
0.50	125.00	0.03	0.00	0.03	1.00	1.00	0.462	1.07	0.431	1.00	0.431	2.000
5.50	125.00	0.34	0.00	0.34	0.99	1.00	0.456	1.07	0.426	1.00	0.426	2.000
8.50	125.00	0.53	0.00	0.53	0.98	1.00	0.453	1.07	0.423	1.00	0.423	2.000
10.50	125.00	0.66	0.00	0.66	0.98	1.00	0.451	1.07	0.421	1.00	0.421	2.000
15.00	125.00	0.94	0.00	0.94	0.97	1.00	0.447	1.07	0.417	1.00	0.417	2.000
16.00	125.00	1.00	0.00	1.00	0.97	1.00	0.446	1.07	0.416	1.00	0.416	2.000
20.50	125.00	1.28	0.00	1.28	0.96	1.00	0.441	1.07	0.412	0.96	0.428	2.000
25.50	125.00	1.59	0.00	1.59	0.94	1.00	0.434	1.07	0.405	0.92	0.440	2.000
30.50	125.00	1.91	0.00	1.91	0.92	1.00	0.424	1.07	0.395	0.89	0.445	2.000
35.50	125.00	2.22	0.00	2.22	0.89	1.00	0.409	1.07	0.382	0.86	0.443	2.000
40.50	125.00	2.53	0.00	2.53	0.85	1.00	0.391	1.07	0.365	0.84	0.434	2.000
45.50	125.00	2.84	0.00	2.84	0.80	1.00	0.369	1.07	0.344	0.82	0.419	2.000
50.50	125.00	3.16	0.00	3.16	0.75	1.00	0.345	1.07	0.322	0.80	0.401	2.000

Abbreviations

$\sigma_{v,eq}$:	Total overburden pressure at test point, during earthquake (tsf)
$u_{o,eq}$:	Water pressure at test point, during earthquake (tsf)
$\sigma'_{vo,eq}$:	Effective overburden pressure, during earthquake (tsf)
r_d :	Nonlinear shear mass factor
α:	Improvement factor due to stone columns
CSR :	Cyclic Stress Ratio (adjusted for improvement)
MSF :	Magnitude Scaling Factor
$CSR_{eq,M=7.5}$:	CSR adjusted for M=7.5
K_{sigma}:	Effective overburden stress factor
CSR^*:	CSR fully adjusted (user FS applied)***
FS:	Calculated factor of safety against soil liquefaction

*** User FS: 1.00

:: Liquefaction potential according to Iwasaki ::

Depth (ft)	FS	F	wz	Thickness (ft)	I_L
0.50	2.000	0.00	9.92	5.00	0.00
5.50	2.000	0.00	9.16	5.00	0.00
8.50	2.000	0.00	8.70	3.00	0.00
10.50	2.000	0.00	8.40	2.00	0.00
15.00	2.000	0.00	7.71	4.50	0.00
16.00	2.000	0.00	7.56	1.00	0.00
20.50	2.000	0.00	6.88	4.50	0.00
25.50	2.000	0.00	6.11	5.00	0.00
30.50	2.000	0.00	5.35	5.00	0.00
35.50	2.000	0.00	4.59	5.00	0.00
40.50	2.000	0.00	3.83	5.00	0.00
45.50	2.000	0.00	3.07	5.00	0.00
50.50	2.000	0.00	2.30	5.00	0.00

Overall potential I_L : 0.00

I_L = 0.00 - No liquefaction
I_L between 0.00 and 5 - Liquefaction not probable
I_L between 5 and 15 - Liquefaction probable
I_L > 15 - Liquefaction certain

:: Vertical settlements estimation for dry sands ::

Depth (ft)	$(N_1)_{60}$	τ_{av}	p	G_{max} (tsf)	α	b	γ	ε_{15}	N_c	ε_{Nc} (%)	Δh (ft)	ΔS (in)
0.50	13	0.01	0.02	172.59	0.13	51200.00	0.00	0.00	13.34	0.08	3.50	0.063
5.50	17	0.16	0.23	601.09	0.14	12146.03	0.00	0.00	13.34	0.08	3.50	0.070
8.50	21	0.24	0.36	790.05	0.14	9354.07	0.00	0.00	13.34	0.06	2.50	0.039
10.50	15	0.30	0.44	804.56	0.15	8240.21	0.00	0.00	13.34	0.12	4.00	0.120
15.00	9	0.42	0.63	892.70	0.16	6652.69	0.00	0.00	13.34	0.23	2.50	0.140
16.00	13	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	2.00	0.000
20.50	27	0.57	0.86	1257.62	0.17	5515.69	0.00	0.00	13.34	0.07	5.00	0.089
25.50	27	0.69	1.07	1402.62	0.19	4838.69	0.00	0.00	13.34	0.08	7.00	0.134
30.50	18	0.81	1.28	1515.50	0.20	4345.82	0.00	0.00	13.34	0.09	4.00	0.085
35.50	23	0.91	1.49	1748.12	0.21	3967.48	0.00	0.00	13.34	0.06	6.00	0.085
40.50	37	0.99	1.70	1990.82	0.22	3665.89	0.00	0.00	13.34	0.04	5.00	0.048
45.50	20	1.05	1.91	1728.88	0.23	3418.58	0.00	0.00	13.34	0.12	3.00	0.086
50.50	33	1.09	2.11	2241.43	0.25	3211.27	0.00	0.00	13.34	0.03	3.50	0.029

Cumulative settlemetns: 0.987

Abbreviations

τ_{av}: Average cyclic shear stress
p: Average stress
G_{max}: Maximum shear modulus (tsf)
α, b: Shear strain formula variables
γ: Average shear strain
ε_{15}: Volumetric strain after 15 cycles
N_c: Number of cycles
ε_{Nc}: Volumetric strain for number of cycles N_c (%)
Δh: Thickness of soil layer (in)
ΔS: Settlement of soil layer (in)

:: Lateral displacements estimation for saturated sands ::

Depth (ft)	$(N_1)_{60}$	D_r (%)	γ_{max} (%)	d_z (ft)	LDI	LD (ft)
0.50	13	50.48	0.00	3.50	0.000	0.00
5.50	17	57.72	0.00	3.50	0.000	0.00
8.50	21	64.16	0.00	2.50	0.000	0.00
10.50	15	54.22	0.00	4.00	0.000	0.00
15.00	9	42.00	0.00	2.50	0.000	0.00
16.00	13	50.48	0.00	2.00	0.000	0.00
20.50	27	72.75	0.00	5.00	0.000	0.00
25.50	27	72.75	0.00	7.00	0.000	0.00
30.50	18	59.40	0.00	4.00	0.000	0.00
35.50	23	67.14	0.00	6.00	0.000	0.00
40.50	37	85.16	0.00	5.00	0.000	0.00
45.50	20	62.61	0.00	3.00	0.000	0.00
50.50	33	80.42	0.00	3.50	0.000	0.00

Cumulative lateral displacements: 0.00

Abbreviations

D_r: Relative density (%)
γ_{max}: Maximum amplitude of cyclic shear strain (%)
d_z: Soil layer thickness (ft)
LDI: Lateral displacement index (ft)
LD: Actual estimated displacement (ft)

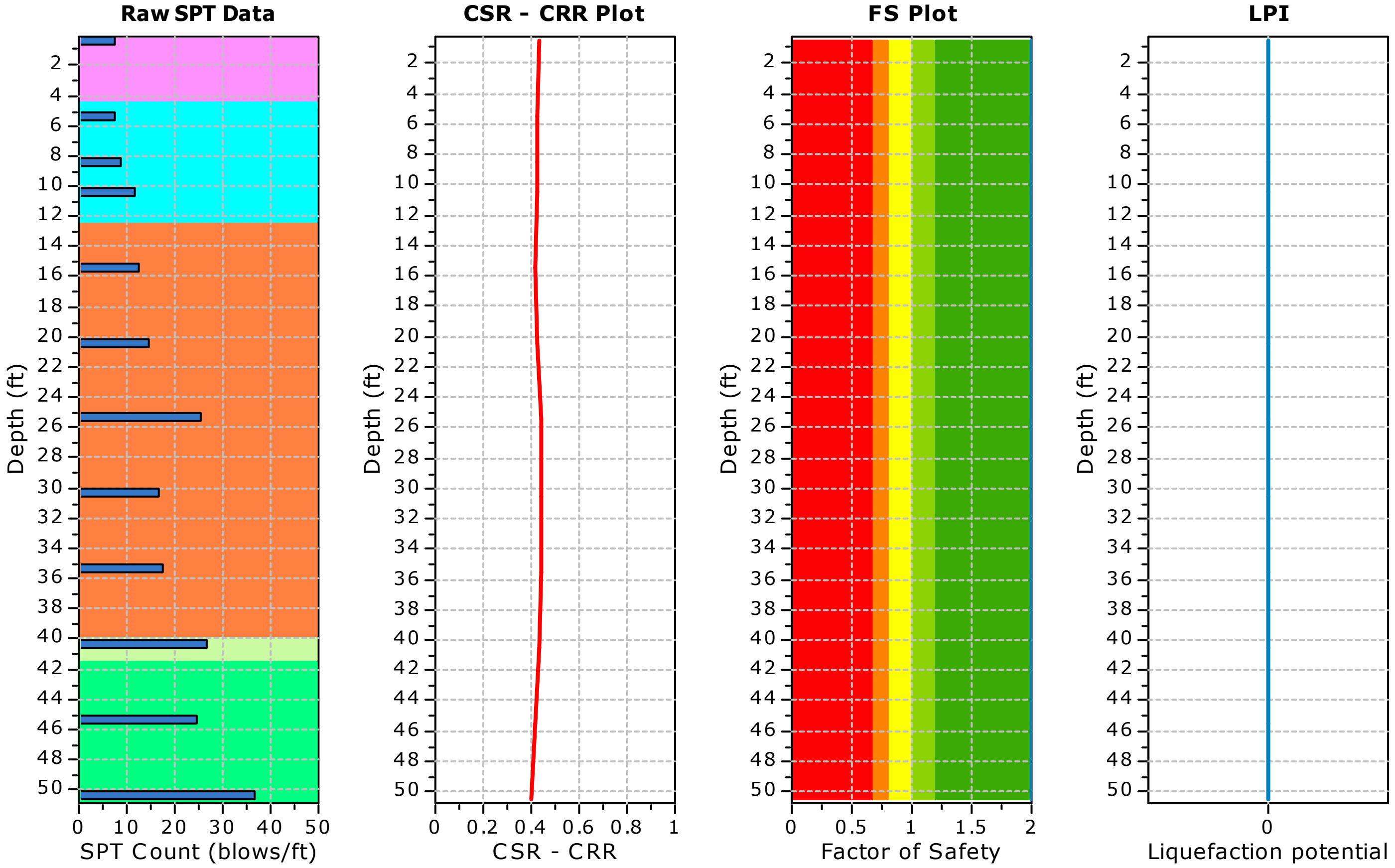
SPT BASED LIQUEFACTION ANALYSIS REPORT

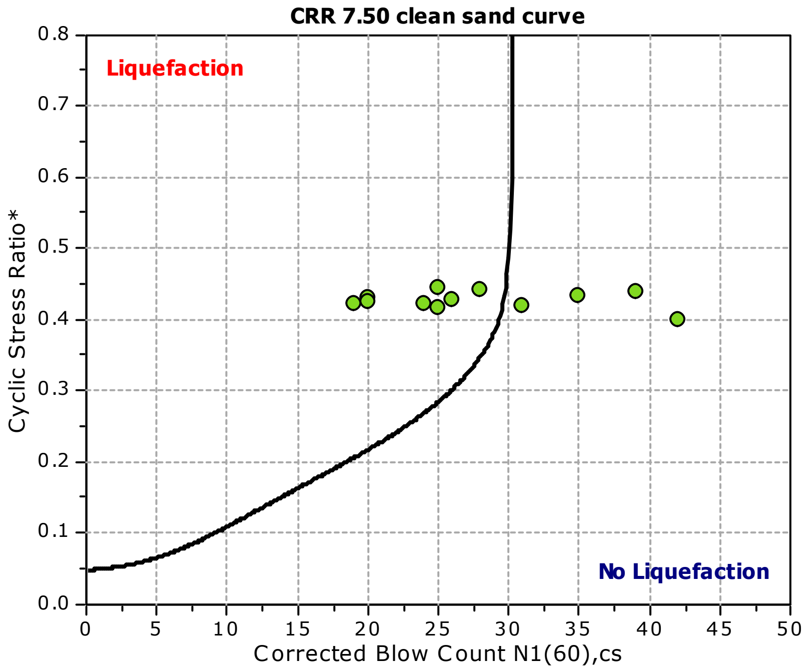
Project title : 22-0161 Patrick Henry E.S. **SPT Name: B-7**

Location :

:: Input parameters and analysis properties ::

Analysis method:	NCEER 1998	G.W.T. (in-situ):	60.00 ft
Fines correction method:	NCEER 1998	G.W.T. (earthq.):	60.00 ft
Sampling method:	Sampler wo liners	Earthquake magnitude M_w:	7.30
Borehole diameter:	65mm to 115mm	Peak ground acceleration:	0.71 g
Rod length:	3.30 ft	Eq. external load:	0.00 tsf
Hammer energy ratio:	1.25		





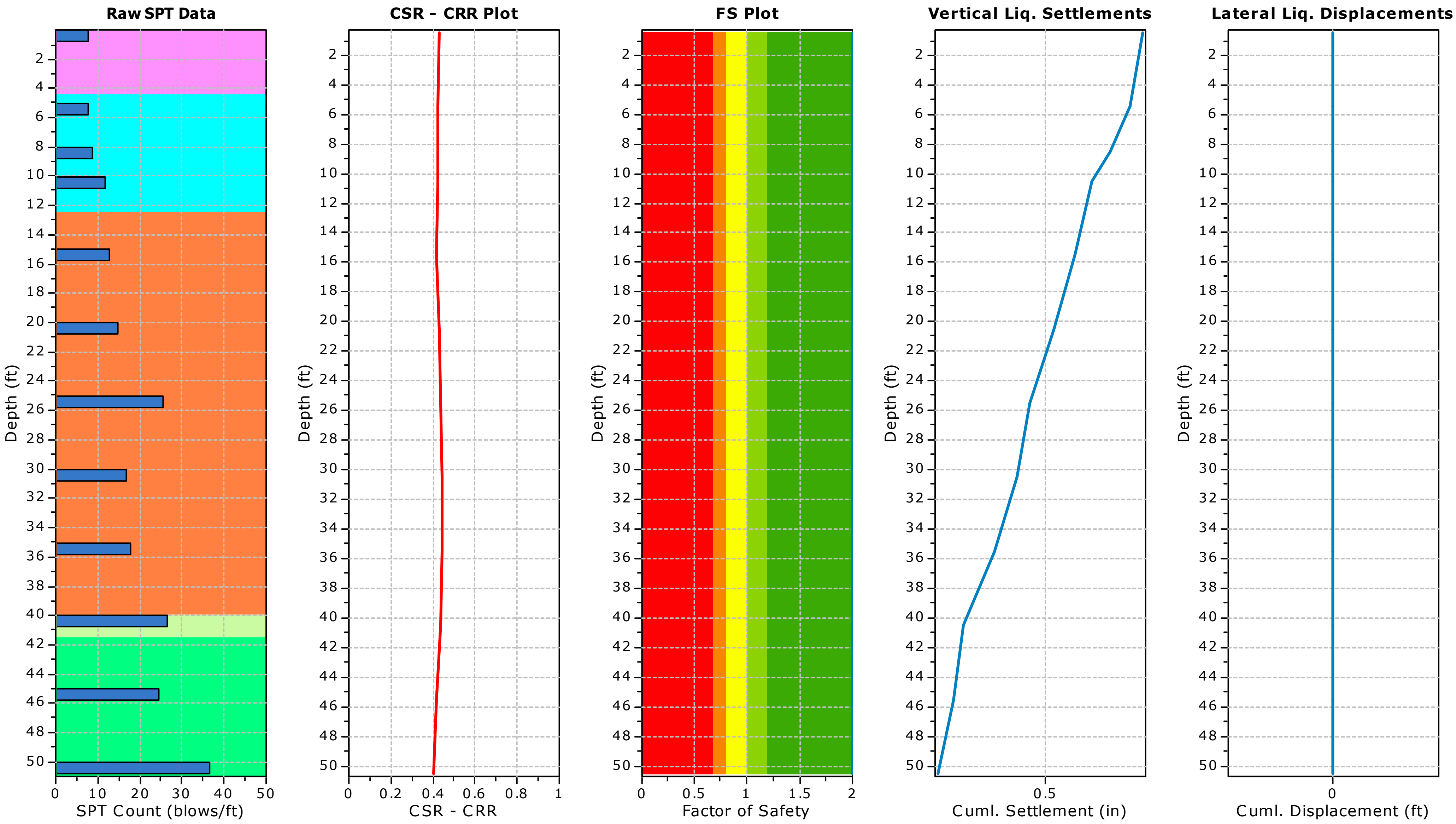
F.S. color scheme

- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

LPI color scheme

- Very high risk
- High risk
- Low risk

:: Overall Liquefaction Assessment Analysis Plots ::



:: Field input data ::

Test Depth (ft)	SPT Field Value (blows)	Fines Content (%)	Unit Weight (pcf)	Infl. Thickness (ft)	Can Liquefy
0.50	8	20.00	125.00	3.50	Yes
5.50	8	20.00	125.00	3.50	Yes
8.50	9	20.00	125.00	2.50	Yes
10.50	12	20.00	125.00	4.00	Yes
15.50	13	67.00	125.00	4.50	Yes
20.50	15	29.00	125.00	5.00	Yes
25.50	26	29.00	125.00	6.00	Yes
30.50	17	25.00	125.00	4.00	Yes
35.50	18	52.00	125.00	7.00	Yes
40.50	27	30.00	125.00	3.50	Yes
45.50	25	30.00	125.00	4.50	Yes
50.50	37	30.00	125.00	3.50	Yes

Abbreviations

Depth:	Depth at which test was performed (ft)
SPT Field Value:	Number of blows per foot
Fines Content:	Fines content at test depth (%)
Unit Weight:	Unit weight at test depth (pcf)
Infl. Thickness:	Thickness of the soil layer to be considered in settlements analysis (ft)
Can Liquefy:	User defined switch for excluding/including test depth from the analysis procedure

:: Cyclic Resistance Ratio (CRR) calculation data ::

Depth (ft)	SPT Field Value	Unit Weight (pcf)	σ_v (tsf)	u_o (tsf)	σ'_{vo} (tsf)	C_N	C_E	C_B	C_R	C_S	$(N_1)_{60}$	Fines Content (%)	α	β	$(N_1)_{60cs}$	$CRR_{7.5}$
0.50	8	125.00	0.03	0.00	0.03	1.70	1.25	1.00	0.75	1.20	15	20.00	3.61	1.08	20	4.000
5.50	8	125.00	0.34	0.00	0.34	1.70	1.25	1.00	0.75	1.20	15	20.00	3.61	1.08	20	4.000
8.50	9	125.00	0.53	0.00	0.53	1.41	1.25	1.00	0.75	1.20	14	20.00	3.61	1.08	19	4.000
10.50	12	125.00	0.66	0.00	0.66	1.27	1.25	1.00	0.85	1.20	19	20.00	3.61	1.08	24	4.000
15.50	13	125.00	0.97	0.00	0.97	1.05	1.25	1.00	0.85	1.20	17	67.00	5.00	1.20	25	4.000
20.50	15	125.00	1.28	0.00	1.28	0.91	1.25	1.00	0.95	1.20	19	29.00	4.64	1.15	26	4.000
25.50	26	125.00	1.59	0.00	1.59	0.81	1.25	1.00	0.95	1.20	30	29.00	4.64	1.15	39	4.000
30.50	17	125.00	1.91	0.00	1.91	0.75	1.25	1.00	1.00	1.20	19	25.00	4.29	1.11	25	4.000
35.50	18	125.00	2.22	0.00	2.22	0.69	1.25	1.00	1.00	1.20	19	52.00	5.00	1.20	28	4.000
40.50	27	125.00	2.53	0.00	2.53	0.65	1.25	1.00	1.00	1.20	26	30.00	4.71	1.15	35	4.000
45.50	25	125.00	2.84	0.00	2.84	0.61	1.25	1.00	1.00	1.20	23	30.00	4.71	1.15	31	4.000
50.50	37	125.00	3.16	0.00	3.16	0.58	1.25	1.00	1.00	1.20	32	30.00	4.71	1.15	42	4.000

Abbreviations

σ_v:	Total stress during SPT test (tsf)
u_o:	Water pore pressure during SPT test (tsf)
σ'_{vo}:	Effective overburden pressure during SPT test (tsf)
C_N:	Overburden corretion factor
C_E:	Energy correction factor
C_B:	Borehole diameter correction factor
C_R:	Rod length correction factor
C_S:	Liner correction factor
$N_{1(60)}$:	Corrected N_{SPT} to a 60% energy ratio
α, β:	Clean sand equivalent clean sand formula coefficients
$N_{1(60)cs}$:	Corected $N_{1(60)}$ value for fines content
$CRR_{7.5}$:	Cyclic resistance ratio for M=7.5

:: Cyclic Stress Ratio calculation (CSR fully adjusted and normalized) ::

Depth (ft)	Unit Weight (pcf)	$\sigma_{v,eq}$ (tsf)	$u_{o,eq}$ (tsf)	$\sigma'_{vo,eq}$ (tsf)	r_d	α	CSR	MSF	$CSR_{eq,M=7.5}$	K_{sigma}	CSR^*	FS	
0.50	125.00	0.03	0.00	0.03	1.00	1.00	0.462	1.07	0.431	1.00	0.431	2.000	●
5.50	125.00	0.34	0.00	0.34	0.99	1.00	0.456	1.07	0.426	1.00	0.426	2.000	●
8.50	125.00	0.53	0.00	0.53	0.98	1.00	0.453	1.07	0.423	1.00	0.423	2.000	●
10.50	125.00	0.66	0.00	0.66	0.98	1.00	0.451	1.07	0.421	1.00	0.421	2.000	●
15.50	125.00	0.97	0.00	0.97	0.97	1.00	0.446	1.07	0.417	1.00	0.417	2.000	●
20.50	125.00	1.28	0.00	1.28	0.96	1.00	0.441	1.07	0.412	0.96	0.428	2.000	●
25.50	125.00	1.59	0.00	1.59	0.94	1.00	0.434	1.07	0.405	0.92	0.440	2.000	●
30.50	125.00	1.91	0.00	1.91	0.92	1.00	0.424	1.07	0.395	0.89	0.445	2.000	●
35.50	125.00	2.22	0.00	2.22	0.89	1.00	0.409	1.07	0.382	0.86	0.443	2.000	●
40.50	125.00	2.53	0.00	2.53	0.85	1.00	0.391	1.07	0.365	0.84	0.434	2.000	●
45.50	125.00	2.84	0.00	2.84	0.80	1.00	0.369	1.07	0.344	0.82	0.419	2.000	●
50.50	125.00	3.16	0.00	3.16	0.75	1.00	0.345	1.07	0.322	0.80	0.401	2.000	●

Abbreviations

$\sigma_{v,eq}$:	Total overburden pressure at test point, during earthquake (tsf)
$u_{o,eq}$:	Water pressure at test point, during earthquake (tsf)
$\sigma'_{vo,eq}$:	Effective overburden pressure, during earthquake (tsf)
r_d :	Nonlinear shear mass factor
α:	Improvement factor due to stone columns
CSR :	Cyclic Stress Ratio (adjusted for improvement)
MSF :	Magnitude Scaling Factor
$CSR_{eq,M=7.5}$:	CSR adjusted for M=7.5
K_{sigma}:	Effective overburden stress factor
CSR^*:	CSR fully adjusted (user FS applied)***
FS:	Calculated factor of safety against soil liquefaction

*** User FS: 1.00

:: Liquefaction potential according to Iwasaki ::

Depth (ft)	FS	F	wz	Thickness (ft)	I_L
0.50	2.000	0.00	9.92	5.00	0.00
5.50	2.000	0.00	9.16	5.00	0.00
8.50	2.000	0.00	8.70	3.00	0.00
10.50	2.000	0.00	8.40	2.00	0.00
15.50	2.000	0.00	7.64	5.00	0.00
20.50	2.000	0.00	6.88	5.00	0.00
25.50	2.000	0.00	6.11	5.00	0.00
30.50	2.000	0.00	5.35	5.00	0.00
35.50	2.000	0.00	4.59	5.00	0.00
40.50	2.000	0.00	3.83	5.00	0.00
45.50	2.000	0.00	3.07	5.00	0.00
50.50	2.000	0.00	2.30	5.00	0.00

Overall potential I_L : 0.00

I_L = 0.00 - No liquefaction
I_L between 0.00 and 5 - Liquefaction not probable
I_L between 5 and 15 - Liquefaction probable
I_L > 15 - Liquefaction certain

:: Vertical settlements estimation for dry sands ::

Depth (ft)	$(N_1)_{60}$	τ_{av}	p	G_{max} (tsf)	α	b	γ	ε_{15}	N_c	ε_{Nc} (%)	Δh (ft)	ΔS (in)
0.50	15	0.01	0.02	175.57	0.13	51200.00	0.00	0.00	13.34	0.07	3.50	0.055
5.50	15	0.16	0.23	582.29	0.14	12146.03	0.00	0.00	13.34	0.10	3.50	0.087
8.50	14	0.24	0.36	711.62	0.14	9354.07	0.00	0.00	13.34	0.13	2.50	0.079
10.50	19	0.30	0.44	854.97	0.15	8240.21	0.00	0.00	13.34	0.08	4.00	0.079
15.50	17	0.43	0.65	1053.01	0.16	6523.08	0.00	0.00	13.34	0.09	4.50	0.093
20.50	19	0.57	0.86	1226.93	0.17	5515.69	0.00	0.00	13.34	0.09	5.00	0.104
25.50	30	0.69	1.07	1566.43	0.19	4838.69	0.00	0.00	13.34	0.04	6.00	0.059
30.50	19	0.81	1.28	1477.12	0.20	4345.82	0.00	0.00	13.34	0.10	4.00	0.100
35.50	19	0.91	1.49	1654.95	0.21	3967.48	0.00	0.00	13.34	0.08	7.00	0.138
40.50	26	0.99	1.70	1904.15	0.22	3665.89	0.00	0.00	13.34	0.05	3.50	0.043
45.50	23	1.05	1.91	1938.26	0.23	3418.58	0.00	0.00	13.34	0.06	4.50	0.066
50.50	32	1.09	2.11	2259.51	0.25	3211.27	0.00	0.00	13.34	0.03	3.50	0.027

Cumulative settlemetns: 0.931

Abbreviations

τ_{av}: Average cyclic shear stress
p: Average stress
G_{max}: Maximum shear modulus (tsf)
α, b: Shear strain formula variables
γ: Average shear strain
ε_{15}: Volumetric strain after 15 cycles
N_c: Number of cycles
ε_{Nc}: Volumetric strain for number of cycles N_c (%)
Δh: Thickness of soil layer (in)
ΔS: Settlement of soil layer (in)

:: Lateral displacements estimation for saturated sands ::

Depth (ft)	$(N_1)_{60}$	D_r (%)	γ_{max} (%)	d_z (ft)	LDI	LD (ft)
0.50	15	54.22	0.00	3.50	0.000	0.00
5.50	15	54.22	0.00	3.50	0.000	0.00
8.50	14	52.38	0.00	2.50	0.000	0.00
10.50	19	61.02	0.00	4.00	0.000	0.00
15.50	17	57.72	0.00	4.50	0.000	0.00
20.50	19	61.02	0.00	5.00	0.000	0.00
25.50	30	76.68	0.00	6.00	0.000	0.00
30.50	19	61.02	0.00	4.00	0.000	0.00
35.50	19	61.02	0.00	7.00	0.000	0.00
40.50	26	71.39	0.00	3.50	0.000	0.00
45.50	23	67.14	0.00	4.50	0.000	0.00
50.50	32	79.20	0.00	3.50	0.000	0.00

Cumulative lateral displacements: 0.00

Abbreviations

D_r: Relative density (%)
γ_{max}: Maximum amplitude of cyclic shear strain (%)
d_z: Soil layer thickness (ft)
LDI: Lateral displacement index (ft)
LD: Actual estimated displacement (ft)

Percolation Testing

Job Name: Patrick Henry Elementary School

Job No.: 22-0161

Test Location: 55 feet N of W Romneya Dr CL & 88 feet E of Eastern PL. See Figure A-2

Water Table Depth (ft): > 50 **Relatively Impervious Layer Depth (ft):** >20

Test Date: 7/19/2022

Test No.: P-1

Depth of Hole (db): 66 in

Diameter of Hole (D): 8.75 in

Test Performer: ABB

Pre-Test Readings

Trial No.	Start Time (hr:min)	Stop Time (hr:min)	Time Interval (hr:min)	Initial Water Depth (in)	Final Water Depth (in)	Water Level Change (in)	Water Level Change >6"
1	12:43	13:08	0:25	36	66	30	Yes
2	13:12	13:37	0:25	36	66	30	Yes

If both yes, run test for an additional hour, reading at 10 minute interval

If no, run test for an additional 6 hours, reading at 30 minute interval

Trial No.	Time of Testing			Water Level Measurement		Water Level Calculations				Percolation & Infiltration Rate Calculations		
	Initial Time T_1 (hr:min)	Final Time T_2 (hr:min)	Time Interval $\Delta T = T_2 - T_1$ (hr:min)	Initial Depth to Water d_1 (in)	Final Depth to Water d_2 (in)	Initial Height of Water Column $d_{H1} = d_b - d_1$ (in)	Final Height of Water Column $d_{H2} = d_b - d_2$ (in)	Drop in Height $\Delta d_H = d_{H1} - d_{H2}$ (in)	Average Height of Water Column $d_{avg} = (d_{H1}+d_{H2})/2$ (in)	Measured Percolation $K_i = \Delta d_H / \Delta T$ (in/hr)	Reduction Factor	Calculated Infiltration Rate = (60ΔdHD/2)/(ΔT(D/2+2davg) (in/hr)
1	12:08	12:18	0:10	24	53 6/16	42	12 10/16	29 6/16	27 5/16	176.40	13.5	13.09
2	12:20	12:30	0:10	24	52 3/16	42	13 13/16	28 3/16	27 14/16	169.20	13.8	12.30
3	12:31	12:41	0:10	24	51 10/16	42	14 6/16	27 10/16	28 3/16	165.60	13.9	11.92
4	12:43	12:53	0:10	24	51	42	15	27	28 8/16	162.00	14.0	11.55
5	12:55	13:05	0:10	24	50 6/16	42	15 10/16	26 6/16	28 13/16	158.40	14.2	11.18
6	13:08	13:18	0:10	24	50 6/16	42	15 10/16	26 6/16	28 13/16	158.40	14.2	11.18
7	13:20	13:30	0:10	24	49 13/16	42	16 3/16	25 13/16	29 2/16	154.80	14.3	10.82
8	13:32	13:42	0:10	24	48 10/16	42	17 6/16	24 10/16	29 11/16	147.60	14.6	10.13
9	13:44	13:54	0:10	24	48	42	18	24	30	144.00	14.7	9.79
10	13:56	14:06	0:10	24	48	42	18	24	30	144.00	14.7	9.79

Note:

1. Infiltration Rate, It = (60ΔdHD/2)/(ΔT(D/2+2davg)

2. Long Term Infiltration Rate = Short Term Infiltration Rate / Correction Factor for Test Limitations

Correction Factor Range Normally used to account for Long Term Moderate Siltation, Test Scale Limitations and Other Factors = 3 to 12

Reference: Riverside County - Low Impact Development BMP Design Handbook Appendix A, dated 9/2011

Lowest Infiltration Rate = 9.79 in/hr

Adjusted Infiltration Rate = 3.26 in/hr

Percolation Testing

Job Name: Patrick Henry Elementary School

Job No.: 22-0161

Test No.: P-2

Test Location: 570 feet N of W Romneya Dr CL & 298 feet E of Eastern PL. See Figure A-2

Depth of Hole (db): 60 in

Water Table Depth (ft): > 50 **Relatively Impervious Layer Depth (ft):** >20

Diameter of Hole (D): 8.75 in

Test Date: 7/19/2022

Test Performer: ABB

Pre-Test Readings

Trial No.	Start Time (hr:min)	Stop Time (hr:min)	Time Interval (hr:min)	Initial Water Depth (in)	Final Water Depth (in)	Water Level Change (in)	Water Level Change >6"
1	13:00	13:25	0:25	24	>60	>60	Yes
2	13:33	13:58	0:25	24	>60	>60	Yes

If both yes, run test for an additional hour, reading at 10 minute interval

If no, run test for an additional 6 hours, reading at 30 minute interval

Trial No.	Time of Testing			Water Level Measurement		Water Level Calculations				Percolation & Infiltration Rate Calculations		
	Initial Time	Final Time	Time Interval	Initial Depth to Water	Final Depth to Water	Initial Height of Water Column	Final Height of Water Column	Drop in Height	Average Height of Water Column	Measured Percolation	Reduction Factor	Calculated Infiltration Rate =
	T_1	T_2	$\Delta T = T_2 - T_1$	d_1	d_2	$d_{H1} = d_b - d_1$	$d_{H2} = d_b - d_2$	$\Delta d_H = d_{H1} - d_{H2}$	$d_{avg} = (d_{H1}+d_{H2})/2$	$K_i = \Delta d_H / \Delta T$		(60ΔdHD/2)/(ΔT(D/2+2davg)
	(hr:min)	(hr:min)	(hr:min)	(in)	(in)	(in)	(in)	(in)	(in)	(in/hr)		(in/hr)
1	14:35	14:40	0:05	36	52 3/16	24	7 13/16	16 3/16	15 14/16	194.40	8.3	23.51
2	14:42	14:47	0:05	36	50 6/16	24	9 10/16	14 6/16	16 13/16	172.80	8.7	19.91
3	14:49	14:54	0:05	36	49 13/16	24	10 3/16	13 13/16	17 2/16	165.60	8.8	18.78
4	15:02	15:07	0:05	36	49 3/16	24	10 13/16	13 3/16	17 6/16	158.40	9.0	17.69
5	15:10	15:15	0:05	36	48	24	12	12	18	144.00	9.2	15.60
6	15:17	15:22	0:05	36	48	24	12	12	18	144.00	9.2	15.60
7	15:25	15:30	0:05	36	48	24	12	12	18	144.00	9.2	15.60
8	15:32	15:37	0:05	36	48	24	12	12	18	144.00	9.2	15.60
9	15:40	15:45	0:05	36	47 6/16	24	12 10/16	11 6/16	18 5/16	136.80	9.4	14.61
10	15:47	15:52	0:05	36	46 13/16	24	13 3/16	10 13/16	18 10/16	129.60	9.5	13.64

Note:

1. Infiltration Rate, It = (60ΔdHD/2)/(ΔT(D/2+2davg)
2. Long Term Infiltration Rate = Short Term Infiltration Rate / Correction Factor for Test Limitations

Correction Factor Range Normally used to account for Long Term Moderate Siltation, Test Scale Limitations and Other Factors = 3 to 12

Reference: Riverside County - Low Impact Development BMP Design Handbook Appendix A, dated 9/2011

Lowest Infiltration Rate = 13.64 in/hr

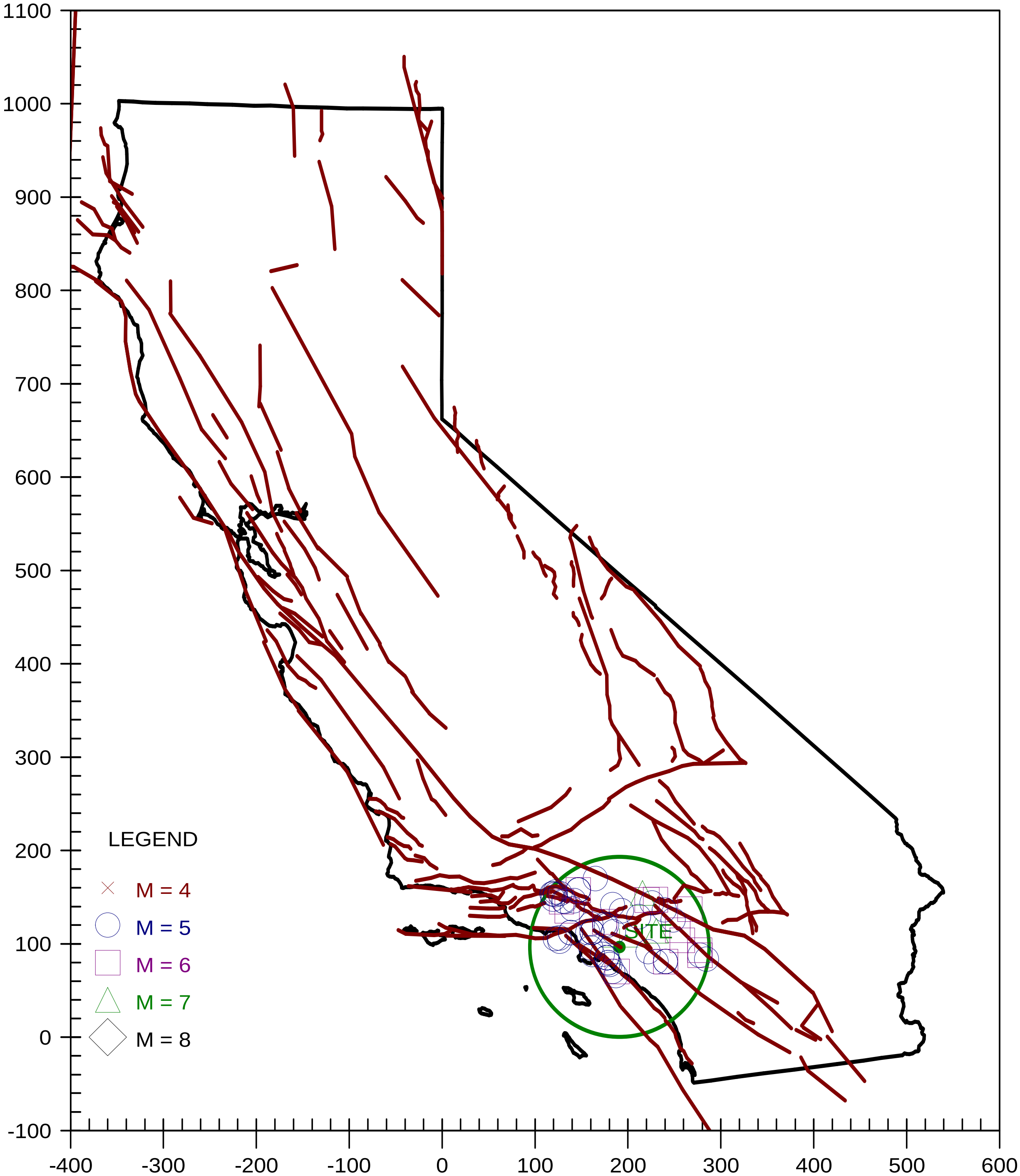
Adjusted Infiltration Rate = 4.55 in/hr

APPENDIX D

Historical Earthquake Data

EARTHQUAKE EPICENTER MAP

Patrick Henry ES



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ESTIMATION OF
PEAK ACCELERATION FROM
CALIFORNIA EARTHQUAKE CATALOGS

JOB NUMBER: 22-0161

DATE: 07-25-2022

JOB NAME: Patrick Henry ES

EARTHQUAKE-CATALOG-FILE NAME: ALLQUAKE.DAT

MAGNITUDE RANGE:
MINIMUM MAGNITUDE: 5.00
MAXIMUM MAGNITUDE: 9.00

SITE COORDINATES:
SITE LATITUDE: 33.8513
SITE LONGITUDE: 117.9329

SEARCH DATES:
START DATE: 1800
END DATE: 2000

SEARCH RADIUS:
60.0 mi
96.6 km

ATTENUATION RELATION: 14) Campbell & Bozorgnia (1997 Rev.) - Alluvium
UNCERTAINTY (M=Median, S=Sigma): M Number of Sigmas: 0.0
ASSUMED SOURCE TYPE: DS [SS=Strike-slip, DS=Reverse-slip, BT=Blind-thrust]
SCOND: 0 Depth Source: A
Basement Depth: 5.00 km Campbell SSR: 0 Campbell SHR: 0
COMPUTE PEAK HORIZONTAL ACCELERATION

MINIMUM DEPTH VALUE (km): 3.0

EARTHQUAKE SEARCH RESULTS

Page 1

FILE CODE	LAT. NORTH	LONG. WEST	DATE	TIME (UTC) H M Sec	DEPTH (km)	QUAKE MAG.	SITE ACC. g	SITE MM INT.	APPROX. DISTANCE mi [km]
MGI	34.0000	118.0000	12/25/1903	1745 0.0	0.0	5.00	0.073	VII	11.0(17.6)
DMG	33.7500	118.0830	03/11/1933	230 0.0	0.0	5.10	0.078	VII	11.1(17.8)
DMG	33.7500	118.0830	03/11/1933	910 0.0	0.0	5.10	0.078	VII	11.1(17.8)
DMG	33.7500	118.0830	03/13/1933	131828.0	0.0	5.30	0.093	VII	11.1(17.8)
DMG	33.7500	118.0830	03/11/1933	323 0.0	0.0	5.00	0.072	VI	11.1(17.8)
DMG	33.7500	118.0830	03/11/1933	2 9 0.0	0.0	5.00	0.072	VI	11.1(17.8)
DMG	33.7830	118.1330	10/02/1933	91017.6	0.0	5.40	0.088	VII	12.4(20.0)
DMG	33.7000	118.0670	03/11/1933	51022.0	0.0	5.10	0.065	VI	13.0(20.9)
DMG	33.7000	118.0670	03/11/1933	85457.0	0.0	5.10	0.065	VI	13.0(20.9)
DMG	33.6830	118.0500	03/11/1933	658 3.0	0.0	5.50	0.086	VII	13.4(21.6)
DMG	33.6170	117.9670	03/11/1933	154 7.8	0.0	6.30	0.125	VII	16.3(26.2)
PAS	34.0610	118.0790	10/01/1987	144220.0	9.5	5.90	0.089	VII	16.7(26.9)
DMG	33.6170	118.0170	03/14/1933	19 150.0	0.0	5.10	0.046	VI	16.9(27.2)
PAS	34.0730	118.0980	10/04/1987	105938.2	8.2	5.30	0.050	VI	18.0(28.9)
DMG	33.7830	118.2500	11/14/1941	84136.3	0.0	5.40	0.051	VI	18.8(30.2)
DMG	33.8500	118.2670	03/11/1933	1425 0.0	0.0	5.00	0.036	V	19.2(30.8)
DMG	33.5750	117.9830	03/11/1933	518 4.0	0.0	5.20	0.042	VI	19.3(31.0)
MGI	33.8000	117.6000	04/22/1918	2115 0.0	0.0	5.00	0.035	V	19.4(31.2)
MGI	34.1000	118.1000	07/11/1855	415 0.0	0.0	6.30	0.098	VII	19.7(31.6)
T-A	34.0000	118.2500	09/23/1827	0 0 0.0	0.0	5.00	0.032	V	20.9(33.6)
T-A	34.0000	118.2500	03/26/1860	0 0 0.0	0.0	5.00	0.032	V	20.9(33.6)
T-A	34.0000	118.2500	01/10/1856	0 0 0.0	0.0	5.00	0.032	V	20.9(33.6)
MGI	34.0000	118.3000	09/03/1905	540 0.0	0.0	5.30	0.035	V	23.4(37.7)
GSP	34.1400	117.7000	02/28/1990	234336.6	5.0	5.20	0.031	V	24.0(38.6)
DMG	34.2000	117.9000	08/28/1889	215 0.0	0.0	5.50	0.039	V	24.1(38.9)
MGI	34.0800	118.2600	07/16/1920	18 8 0.0	0.0	5.00	0.026	V	24.5(39.4)
DMG	33.6990	117.5110	05/31/1938	83455.4	10.0	5.50	0.035	V	26.4(42.5)
MGI	34.0000	117.5000	12/16/1858	10 0 0.0	0.0	7.00	0.109	VII	26.8(43.2)
GSP	34.2620	118.0020	06/28/1991	144354.5	11.0	5.40	0.029	V	28.6(46.1)
DMG	33.7000	117.4000	04/11/1910	757 0.0	0.0	5.00	0.017	IV	32.3(52.0)
DMG	33.7000	117.4000	05/13/1910	620 0.0	0.0	5.00	0.017	IV	32.3(52.0)
DMG	33.7000	117.4000	05/15/1910	1547 0.0	0.0	6.00	0.039	V	32.3(52.0)
DMG	34.0000	118.5000	08/04/1927	1224 0.0	0.0	5.00	0.016	IV	34.1(54.8)
MGI	34.0000	118.5000	11/19/1918	2018 0.0	0.0	5.00	0.016	IV	34.1(54.8)
DMG	34.3000	117.6000	07/30/1894	512 0.0	0.0	6.00	0.033	V	36.4(58.5)
DMG	34.2700	117.5400	09/12/1970	143053.0	8.0	5.40	0.020	IV	36.6(58.9)
DMG	34.2000	117.4000	07/22/1899	046 0.0	0.0	5.50	0.020	IV	38.8(62.5)
DMG	34.3700	117.6500	12/08/1812	15 0 0.0	0.0	7.00	0.065	VI	39.3(63.2)
DMG	34.3000	117.5000	07/22/1899	2032 0.0	0.0	6.50	0.043	VI	39.7(63.8)
PAS	33.9190	118.6270	01/19/1989	65328.8	11.9	5.00	0.013	III	40.1(64.5)
MGI	34.1000	117.3000	07/15/1905	2041 0.0	0.0	5.30	0.016	IV	40.1(64.5)
DMG	34.0000	117.2500	07/23/1923	73026.0	0.0	6.25	0.035	V	40.4(65.1)
GSP	34.2310	118.4750	03/20/1994	212012.3	13.0	5.30	0.016	IV	40.6(65.3)
DMG	33.9500	118.6320	08/31/1930	04036.0	0.0	5.20	0.015	IV	40.6(65.4)
DMG	33.9000	117.2000	12/19/1880	0 0 0.0	0.0	6.00	0.027	V	42.1(67.8)
GSP	34.2130	118.5370	01/17/1994	123055.4	18.0	6.70	0.046	VI	42.6(68.6)
PAS	33.9440	118.6810	01/01/1979	231438.9	11.3	5.00	0.011	III	43.3(69.8)
DMG	34.3080	118.4540	02/09/1971	144346.7	6.2	5.20	0.013	III	43.4(69.8)
DMG	34.4110	118.4010	02/09/1971	14 1 8.0	8.0	5.80	0.020	IV	47.0(75.6)
DMG	34.4110	118.4010	02/09/1971	141028.0	8.0	5.30	0.013	III	47.0(75.6)
DMG	34.4110	118.4010	02/09/1971	14 041.8	8.4	6.40	0.032	V	47.0(75.6)
DMG	34.4110	118.4010	02/09/1971	14 244.0	8.0	5.80	0.020	IV	47.0(75.6)
GSB	34.3010	118.5650	01/17/1994	204602.4	9.0	5.20	0.012	III	47.6(76.7)

EARTHQUAKE SEARCH RESULTS

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FILE CODE	LAT. NORTH	LONG. WEST	DATE	TIME (UTC) H M Sec	DEPTH (km)	QUAKE MAG.	SITE ACC. g	SITE MM INT.	APPROX. DISTANCE mi [km]
GSP	34.3050	118.5790	01/29/1994	112036.0	1.0	5.10	0.011	III	48.4(77.9)
DMG	34.5190	118.1980	08/23/1952	10 9 7.1	13.1	5.00	0.010	III	48.5(78.1)
DMG	34.3000	118.6000	04/04/1893	1940 0.0	0.0	6.00	0.022	IV	49.1(79.1)
DMG	34.2000	117.1000	09/20/1907	154 0.0	0.0	6.00	0.019	IV	53.4(85.9)
GSP	34.3780	118.6180	01/19/1994	211144.9	11.0	5.10	0.009	III	53.4(86.0)
DMG	33.8000	117.0000	12/25/1899	1225 0.0	0.0	6.40	0.026	V	53.6(86.3)
DMG	33.7500	117.0000	04/21/1918	223225.0	0.0	6.80	0.036	V	54.0(86.9)
DMG	33.7500	117.0000	06/06/1918	2232 0.0	0.0	5.00	0.008	III	54.0(86.9)

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GSP |34.3260|118.6980|01/17/1994|233330.7|  9.0| 5.60| 0.013 | III| 54.7( 88.0)
GSP |34.3690|118.6720|04/26/1997|103730.7| 16.0| 5.10| 0.009 | III| 55.3( 89.1)
GSP |34.3940|118.6690|06/26/1995|084028.9| 13.0| 5.00| 0.008 | II | 56.3( 90.7)
GSP |34.3770|118.6980|01/18/1994|004308.9| 11.0| 5.20| 0.009 | III| 56.8( 91.5)
GSB |34.3790|118.7110|01/19/1994|210928.6| 14.0| 5.50| 0.011 | III| 57.5( 92.5)
DMG |33.7100|116.9250|09/23/1963|144152.6| 16.5| 5.00| 0.007 | II | 58.7( 94.4)

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-END OF SEARCH-   67 EARTHQUAKES FOUND WITHIN THE SPECIFIED SEARCH AREA.

TIME PERIOD OF SEARCH:   1800  TO  2000

LENGTH OF SEARCH TIME:   201  years

THE EARTHQUAKE CLOSEST TO THE SITE IS ABOUT 11.0 MILES (17.6 km) AWAY.

LARGEST EARTHQUAKE MAGNITUDE FOUND IN THE SEARCH RADIUS: 7.0

LARGEST EARTHQUAKE SITE ACCELERATION FROM THIS SEARCH: 0.125 g

COEFFICIENTS FOR GUTENBERG & RICHTER RECURRENCE RELATION:
  a-value=  1.050
  b-value=  0.349
  beta-value=  0.804


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TABLE OF MAGNITUDES AND EXCEEDANCES:
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Earthquake Magnitude	Number of Times Exceeded	Cumulative No. / Year
4.0	67	0.33333
4.5	67	0.33333
5.0	67	0.33333
5.5	24	0.11940
6.0	15	0.07463
6.5	5	0.02488
7.0	2	0.00995

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