Appendix E
Geotechnical Investigation Report

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LIMITED GEOTECHNICAL INVESTIGATION REPORT

VARIOUS CAMPUS UPGRADES
MORNINGSIDE HIGH SCHOOL
10500 YUKON AVENUE SOUTH
INGLEWOOD, CALIFORNIA 90303

PREPARED FOR:
INGLEWOOD UNIFIED SCHOOL DISTRICT
401 SOUTH INGLEWOOD AVENUE
INGLEWOOD, CA 90303

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PROJECT NO. 19-1110

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January 22, 2020
Project No. 19-1110

Inglewood Unified School District
401 S. Inglewood Avenue
Inglewood, CA 90303

Attention: Ms. Kimberly Munoz

**SUBJECT: Limited Geotechnical Investigation Report
Morningside High School Site Upgrades
10500 Yukon Avenue South
Inglewood, CA 90303**

1. INTRODUCTION

This report presents the results of a Limited Geotechnical Investigation performed by Koury Engineering & Testing, Inc. (Koury) for the proposed Morningside High School improvements located at 10500 Yukon Avenue South, Inglewood, California. The study was performed to evaluate the subsurface soil conditions in the area of the proposed improvements in order to provide geotechnical recommendations for design and construction. This report includes our findings and recommendations for the design and construction of the proposed buildings, bleachers and associated improvements.

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. 19-1110, dated November 4, 2019.

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 the Inglewood Unified 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 Morningside High School is bounded by W 104th Street on the north, Yukon Avenue South on the west, W 108th Street and residential homes on the south and Monroe Middle School and commercial facilities on the east. The main access to the site is from Yukon Avenue South near the southwest corner of the campus. The proposed small one-story buildings are located in the southwest portion of the campus, south and northwest of the existing football field. The existing football field will be shifted west and the bleachers will be constructed north and south of the track that will surround the new football field.

The south portion of the campus is presently occupied by an existing football field, basketball courts and portable trailers. Most of the area is landscape except for the track, the basketball court, and the portable trailer area. The proposed expanded parking lot is presently occupied partially by an existing parking lot and landscape and walkway strips. The proposed electrical yard within the northern portion of the site is covered with grass landscape. Immediately east of the future electrical yard, a new baseball field will be constructed following the removal of some portable buildings and tennis courts. There will be a new softball field located northeast of the existing basketball courts and at the east end of the existing football field.

The site slopes gently to the south and southwest, and the ground surface lies at elevations between about 88 and 112 feet (NAVD88). Drainage of the site is generally by sheet flow toward the south.

3. PROPOSED IMPROVEMENTS

Koury understands that the Inglewood Unified School District is planning to add facilities to the Morningside High School consisting of several small one-story buildings, a football field with two sets of bleachers, running track, parking lot, tennis courts, and baseball and softball fields. Based on the site plan provided to us, there will be a new student visitor parking lot at the northwest corner of the campus supporting approximately 154 parking spaces. The proposed electrical yard to be located within the northern portion of the site east Building H and west of the new baseball field will measure about 30 by 65 feet in plan and may have masonry walls with metal deck roofing.

Within the southwest corner of the campus, it is proposed to construct a new football field and an 8-lane track and new bleachers will be located on the north and south sides of the running track. The site plan shows also three small one-story buildings on the northwest side of the playfield area; these buildings are likely to contain bathrooms, locker rooms, storage rooms or concessions. The smallest building covers an area of about 700 square feet and the largest building covers an area of about 3,600 square feet. The third building appears to cover an area of about 1,900 square feet. The site plan also indicates that two rows of 3 side-by-side tennis courts are planned east of the football field and south of the future softball field. The basketball courts will be relocated west partially between the basketball courts and the east end of the new track.

The associated improvements at the site may consist of low walls, fire access road, new utility lines, sidewalks, fencing, light poles, scoreboard and various hardscape areas. Architectural and structural design details for the new buildings were not provided. Koury assumed maximum building column load of about 25 kips and maximum wall loads not exceeding 2 kips per lineal foot.

4. FIELD EXPLORATION

The field exploration program consisted of drilling ten soil test borings (B-1 through B-10) on January 2, 2020 using truck-mounted hollow-stem auger drilling equipment. The borings were drilled to depths ranging between about 6½ and 41½ below the existing ground surface.

The locations of the borings are shown on the Boring Location Map, Figure A-2, presented in Appendix A. Standard penetration test samples, California ring samples and bulk samples were obtained from the borings 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 a maximum of 18 inches into the soils.

5. LABORATORY TESTING

Laboratory tests, including moisture content, dry unit weight, #200 sieve wash, direct shear, pocket penetrometer, consolidation and expansion index were performed to aid in the classification of the materials encountered and to evaluate their engineering properties. Sulfates, chlorides, resistivity, and pH tests (corrosivity tests) were also performed on one sample. The results of pertinent laboratory tests are presented on the boring logs in Appendix B, and/or in Appendix C.

6. SOIL CONDITIONS

The subsurface soil profile consists of fill underlain by alluvial deposits. The fill depth was found to range between about 2 and 5 feet at the boring locations. Deeper fill may be encountered at utility locations, or at other locations between and beyond the borings. Except for Boring B-5 where 6 inches of asphalt was encountered, the asphalt pavement thickness at the boring locations consisted of about 3½ to 5 inches of asphalt concrete. The asphalt was found to be underlain by 2 to 3 inches of aggregate base except for Boring B-4 that indicated 6 inches of base.

Except for Boring B-2 drilled in the southwest portion of the campus in relatively close vicinity of Yukon Avenue South where clayey sand was encountered, the fill materials encountered in the borings consisted generally of firm to stiff sandy lean clay. The fill encountered were generally moist to very moist.

With a few exceptions, the underlying alluvium consists predominantly of interbedded sandy lean clay with clayey sand and silty sand. Interbeds of poorly graded sand were encountered in Borings B-6 and B-9 at depths exceeding 13 feet below the ground surface. The sandy clay alluvial soils are generally stiff to very stiff, and the sand are medium dense to dense.

With a few exceptions, the alluvial soils encountered are generally moist to very moist. Except for one sample of Boring B-9 at a depth of 21 feet where a moisture content of 31 percent was measured, the moisture contents of the sandy clay range from about 12½ to 21 percent with an average of about 14 percent, and the moisture contents of the sand range from about 2 to 14½ percent with an average of about 10 percent. The degree of saturation of the clay is generally high (about 90 to nearly 100 percent).

Our #200 sieve wash tests indicated that the fines contents of sand generally range from about 4 to 48 percent with an average of about 31½ percent. The fines contents of the clay vary from about 50 to 90 percent with an average of about 64½ percent. The dry unit weights of the clay tested range from about 100 to 127 pcf with an average of about 119½ pcf. The dry unit weights of sand range from about 108 to 127 pcf with an average of about 119 pcf.

The consolidation tests did not indicate significant collapse upon addition of water. Despite the high degree of saturation, however, the consolidation test from a depth of 9 feet indicated about 0.3% swell strain upon addition of water under a pressure of 3200 psf. The clay soils are generally overconsolidated and are considered moderately compressible. The rebound (unloading) curve of the consolidation tests indicate that some of the clay tested have potential for expansion. The direct shear test performed on a clayey sand sample indicated peak and ultimate friction angles of about 32 and 31 degrees and peak and ultimate cohesion of about 472 and 52 psf, respectively.

Pocket penetrometer test results indicate unconfined compression strengths of the tested soils ranging between about 1.5 and 4.5 tsf with an average of about 4.4 tsf. An expansion index test on a sandy lean clay of Boring B-1 at 0 to 2 feet yielded a value of 14 while a test on Boring B-4 on a sample near the surface yielded a value of 22.

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 site lies at approximately elevations 88 to 112 feet (NAVD88). Groundwater was not encountered in the borings drilled for this study. The Seismic Hazard Zone Report for the Inglewood 7.5-Minute Quadrangle, Los Angeles County CA, Seismic Hazard Zone Report 029, Department of Conservation, Division of Mines and Geology indicates that the historic high groundwater is at least 50 feet below ground surface (see Figure A-5 for the Historic High Groundwater Map).

8. SITE GEOLOGY

The site is located within the Los Angeles physiographic basin. The Los Angeles basin is bounded on the north by the Santa Monica and San Gabriel Mountains, on the east and southeast by the Santa Ana Mountains and the San Joaquin Hills, and on the west and south by the Pacific Ocean. The Los Angeles basin represents a down-warped block of basement rock overlain by approximately 31,000 feet of sediment.

The Los Angeles physiographic basin is part of the Peninsular Ranges Geomorphic Province. The Peninsular Ranges extend north to the San Gabriel Mountains and south into Mexico to the tip of Baja California. The Peninsular Ranges Province is characterized by alluviated basins, elevated erosion surfaces, and northwest-trending mountain ranges bounded by northwest trending faults.

The Geology Map of the Long Beach Quadrangle indicates that the subsurface conditions at the site consist of old alluvial floodplain deposits consisting of sand, silt and clay (see Figure A-3 for the Geology Map). The subsurface soil profile encountered in the borings consists of fill underlain by alluvial deposits.

9. OIL WELL

The State of California Department of Conservation, Division of Oil, Gas and Geothermal Resources indicates that Morningside High School is located about ½ mile south of the Potrero Oil/Gas Field and ½ mile west of the Howard Townsite Oil/Gas Field. The nearest dry hole is located about 1,000 feet north of the site and the nearest idle hole is about ½ mile northwest of the site. The nearest active well is located about 1½ miles southeast of the site (See Figure A-9, in Appendix A).

During our subsurface exploration, we did not observe oil-field derived hazardous or toxic materials within the borings drilled to the maximum depth of 41½ feet. No hazardous materials associated with oil fields are anticipated at the building sites.

10. SEISMIC CONSIDERATIONS

10.1. General

Morningside High 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 potentially active fault as any fault which has been active during the Quaternary Period (approximately the last 2,000,000 years). 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 on 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. Probably the most important fault line to the site from a seismic shaking standpoint is the northwest trending Alquist-Priolo Newport-Inglewood Fault, located approximately 1,000 feet northeast of the site. The nearest Alquist-Priolo active segment of the Palos Verdes Fault is located approximately 13 miles south of the site and the Whittier Fault Zone nearest active segment is located about 19 miles east of the site. The Redondo Canyon Fault is located about 9½ miles southwest of the site, the Santa Monica Fault Zone is located about 10 miles northwest of the site and the Hollywood Fault Zone is located about 11 miles north of the site (see Figure A-6, Appendix A). The Puente Hills (LA) Fault is located about 5 miles to the northeast, the Puente Hills (Santa Fe Springs) Fault is about 11 miles to the southeast, and the Compton Fault is about 8¼ miles to the southwest.

Based on the information available at this time, it is our opinion that a Mw7.3 earthquake may occur on the Palos Verdes Fault, a Mw6.9 earthquake may occur on the Whittier Fault, a Mw7.2 earthquake may occur on the Newport-Inglewood Fault, a Mw6.2 earthquake may occur on the Redondo Canyon

Fault, a Mw7.0 earthquake may occur on the Santa Monica Fault, a Mw6.6 earthquake may occur on the Hollywood Fault, a Mw6.9 earthquake may occur on the Puente Hills (LA) Fault segment, a Mw6.6 earthquake may occur on the Puente Hills (Santa Fe Springs) segment, and a Mw6.9 earthquake may occur on the Compton Fault. 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 the Newport-Inglewood Fault, near field effects from strong ground motion associated with a large earthquake along this fault 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.

According to the EQSEARCH program, within a search radius of 60 miles, about 63 earthquakes of magnitude 5 or greater have been recorded up to the year 2000. Within that same period, there are records of 11 earthquakes of magnitude 6 or greater, 5 earthquakes of magnitude 6.5 or greater and 3 earthquakes of magnitude 7 or greater within the same search area. The largest earthquake from the site was reported to have occurred in 1827 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 0.22g. 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 Hazard Zone on the State of California Seismic Hazard Zones Map (Figure A-4 in Appendix A). No evidence for landsliding was observed on or in the immediate vicinity of the site at the time of our field exploration. Based on topographic conditions, landsliding is not considered a potential hazard at the site.

10.3. 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-4 in Appendix A) indicates that the site is not located in a liquefaction susceptibility zone. Due to the absence of shallow groundwater, the presence of clayey soils and some medium dense to dense sands, it is our opinion that the potential for liquefaction is remote. However; the potential for dry seismic settlement was evaluated.

For seismic dry settlement evaluation, we obtained an earthquake magnitude of Mw6.35 from a seismic-hazard deaggregation using the USGS Unified Hazard Tool. Our analysis also utilized a site acceleration of 0.898g (PGA_M) obtained from ASCE 7-16 Seismic Design Ground Motion Analysis. The seismic settlement calculations were performed for the deepest borings (B-6 and B-7) using the SPT and equivalent California sampler blow count data. The SPT tests were performed with an automatic hammer and unlined SPT samplers. The California sampler blow counts were multiplied by a factor of 0.65 to obtain the equivalent SPT blow counts.

Using the LiquifyPro software, we calculated seismic settlements on the order of ½ to ¾ inch (see result of calculations in Appendix C). Considering the recommendations in Section 7.66 of the SCEC Guidelines for Implementation of SP 117 and our total seismic settlement calculations, it is our opinion that a differential settlement on the order of ¼ inch in 40 feet may be considered for the design seismic event.

10.4. Tsunamis and Seiches

The site is located at an average elevation ranging from approximately 88 to 112 feet and 5½ miles away from the coastline. There is no mapped major reservoir in the immediate vicinity and upslope of the site. Therefore, tsunamis and seiches are not considered potential hazards.

11. FLOODING

The project site lies within an area of minimal flood hazard as shown on the FEMA Flood Map #06037C1780G, effective date December 21, 2018 (Figure A-7, Appendix A). Based on the County of Los Angeles GIS, the site is located within a 500-year flood zone; however, the site is not reported as being located in a dam inundation zone. Flooding is not considered a high potential hazard to the site.

12. COLLAPSIBLE SOILS

Soils prone to collapse are generally young and deposited by flash floods and wind. The onsite soils have been mapped as older alluvium and the soils at shallow depth have moisture contents that are near or above optimum, which aid in mitigating collapse potential. Our laboratory tests did not indicate significant collapse. Therefore, the potential for collapse is considered low. Overerexcavation and recompaction, and appropriate drainage are recommended to mitigate the potential for hydrocollapse.

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 clay soils and high moisture content soils.

The following sections contain preliminary geotechnical recommendations for the design and construction of the proposed improvements and include our recommendations and discussions about grading, bearing capacity, settlement, flatwork, slabs-on-grade, temporary excavations, and utility trenches.

13.2. Grading

13.2.1. Building Pads

The thickness of undocumented fill encountered at the boring locations range from about 2 to 5 feet. We recommend removing all undocumented fill within the proposed building pads and structure areas. The exact thickness of undocumented fill should be verified at the time of grading.

Any existing pavement, foundation, vegetation, organic material, abandoned underground utilities and other debris should be removed from the proposed building pad and structure areas. Additional recommendations for overexcavation are presented below.

Within building pad areas, we recommend complete overexcavation of the existing fill and the subgrade to at least 3½ feet below existing grades and 2 feet below footings, whichever is deeper. Where feasible, the overexcavation should extend laterally at least 5 feet beyond the building perimeter.

Following subgrade approval by the Geotechnical Engineer, the bottom of the removal excavation should be scarified to a depth of 8 inches, moisture conditioned to at least 125 percent of optimum and recompacted to 90% relative compaction for clay soils and moisture conditioned within 2½ percent of optimum and recompacted to 92% relative compaction for sand as determined by ASTM D1557. However, if the subgrade is firm and consists of undisturbed clay alluvium and the moisture content is at least 125 percent of optimum, the scarification should not be performed, and measures should be taken to prevent subgrade disturbance. The subgrade should be proof rolled with heavy construction equipment to determine its firmness, as needed.

All fill placed within the building pads should be compacted to at least 95% relative compaction at a moisture content within 2½ percent of optimum for sand/granular soils and to at least 90% relative compaction at a moisture content of at least 125 percent of optimum for sandy clay soils unless approved otherwise by the Geotechnical Consultant at the time of construction. The shallow on site sandy clay soils can generally be used as backfill; however, if encountered, any medium to high plastic onsite clay soils should not be used within 2 feet of footings or building slabs. All fill should be deemed as “failing” and unsuitable if the moisture content is less than the recommended value unless determined otherwise by the Geotechnical Engineer at the time of construction.

13.2.2. Exterior Flatwork, Sidewalk 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 overexcavating 18 inches of subgrade material and placing at least 18 inches of new engineered fill for the subgrade of all new non-structural flatwork and pavement. Prior to backfill placement, the subgrade should be scarified to a depth of 8 inches, moisture conditioned and recompacted to 90% relative compaction.

We further recommend the placement of at least 12 inches of non-expansive granular material below all new pedestrian concrete flatwork. It is critical that the upper 12 inches of clay soils below the non-expansive soils be thoroughly moisture conditioned to 125 percent of optimum except for asphalt pavement where it should be at least 115 percent of optimum.

Except for pavement areas, all fill outside the structure areas should be compacted to at least 90% relative compaction at moisture content within 2½ percent of optimum for sand and other granular material and at least 125 percent of optimum for clay soils except as indicated otherwise by the Geotechnical Engineer. Below pavement areas, all clay soils should be compacted to at least 90 percent relative compaction and all granular material to 95 percent relative compaction.

13.2.3. 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 density as determined by ASTM D1557 Method of Soil Compaction for clay soils and 95 percent relative compaction for sand and other granular soils, unless specified otherwise.

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 3 inches may not be placed in the fill without approval of the Geotechnical Consultant or his representative, and in a manner specified for each occurrence. There should not be any concentrations of particle sizes of 2 inches or greater; proper mixing should be performed. If encountered, oversize materials should be disposed outside the structural fill and flatwork areas at the locations designated by the District representative.

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 operation to obtain uniformity of material and moisture.

6. When the moisture content of the fill material is lower than the specified value or is too low to obtain adequate compaction, water should be added and thoroughly dispersed until the soil has a moisture within 2½ percent of optimum moisture content for sand material and 125 percent of optimum for clayey soils unless indicated otherwise in this report and/or 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 should not be constructed at gradients steeper than 2:1(H: V).

It should be noted that some of the clay soils have a high in-situ degree of saturation and moisture contents above optimum and outside the compactable moisture range. These soils are subject to disturbance and "pumping", specially under heavy rubber tire equipment. The contractor will have to select appropriate excavation and compaction equipment to avoid disturbing the high moisture content subgrade soils or soils with high degree of saturation and to be able to compact the fill to the project specifications above relatively soft subgrade. Any scarified clay soils must be compacted to at least 90 percent relative compaction as determined by ASTM D1557.

We recommend that all excavated clay soils be pre-mixed and moisture conditioned outside the fill area prior to reuse as fill. Where the soil consists of sandy clay (50 to 70% fines) and severe "pumping" conditions develop during compaction, the moisture conditioning requirement may be revised at the discretion of the Geotechnical Engineer. Pre-soaking or aeration will be required if the compaction moisture does not meet the above requirements.

13.3. Fill Materials

13.3.1. Onsite Materials

The onsite shallow clayey sand and sandy clay with low expansion potential are deemed suitable to be re-used as engineered fill, provided they are properly processed, moisture conditioned, and free from deleterious material prior to fill placement except as indicated. Some import non-expansive material should also be anticipated for backfilling purpose. The imported materials being used for backfilling should have a low expansion potential (EI less than 20) and should comply with the specifications of this report.

The onsite sandy clay soils may be used under asphalt pavement and concrete roadway areas provided they are properly moisture conditioned; however, they should not be used as backfill within the upper one foot below concrete pedestrian flatwork areas unless further testing at the time of construction indicates an expansion index less than about 20. Unless indicated otherwise, the shallow sandy clay soils are considered suitable for backfilling purpose where no concrete flatwork is anticipated provided, they are free of deleterious and oversize materials and are properly processed and moisture conditioned.

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 10 to 15 percent for existing soils excavated and recompacted to 90 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 from the prior use of the site and removal of topsoil.

13.3.2. Import

Import materials should contain sufficient fines (binder material) 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 40% passing the # 200 sieve. A bulk sample of potential import material, weighing at least 35 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.

13.4. Temporary Excavations

Temporary excavations adjacent to un-surcharged areas are anticipated to be stable vertically to a depth up to 5 feet in fill and alluvium. For deeper excavations up to a depth of 8 feet, we recommend a gradient no steeper than ¾:1 (H:V) for unsurcharged excavations unless shoring is used.

The tops of slopes should be barricaded to prevent vehicles and storage loads within 6 feet of the tops of the slopes, or within a distance equal to at least the height of the slope, whichever is greater. 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 2 feet away from the footing edge. Where there is insufficient space to slope back an excavation, shoring may be required. All regulations of State and Federal OSHA should be followed.

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.

13.5. Floor Slabs

13.5.1. General

The grading recommendations for the new building pads are provided in Section 13.2.1. The building floor slabs-on-grade, as a minimum, should have a thickness of 5 inches and should contain as a minimum No. 4 bars spaced a maximum of 16 inches on centers, in both directions or as recommended otherwise by the Structural Engineer. The Structural Engineer should ultimately determine the size and spacing of the reinforcement to be used. We recommend a concrete strength of at least 4000 psi unless determined otherwise by the Structural Engineer.

13.6.2 Moisture Sensitive Floor Covering

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 slab to reduce excess vapor transmission through the slab.

The function of the recommended 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 at least 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.

A capillary break below the slab may be used at the discretion of the Project Architect. If used, the capillary break should consist of at least 4 inches of free draining gravel or coarse sand, with no more than 2 percent passing the ASTM No. 200 sieve, and should be placed below the vapor retarder. The gradation for the free draining 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).

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 the building slab in accordance with the latest version of the applicable codes. The placement of sand above the vapor retarder is the purview of the Structural Engineer.

13.6. Seismic Coefficients

Under the Earthquake Design Regulations of Chapter 16A, Section 1613A of the 2019 CBC, and based on the mapped values, the coefficients and factors presented in Table 1 were calculated using ASCE 7-16 and the USGS map parameters (Figure A-8, Response Spectrum).

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	1.895
Mapped Acceleration Parameter for 1.0 Second, S_1	0.667
Adjusted Maximum Spectral Response Parameter for Short Period (0.2 Second), S_{MS}	1.895
Adjusted Maximum Spectral Response Parameter for 1.0 Second Period, S_{M1}	*1.134
Design Spectral Response Acceleration Parameter, S_{DS}	1.263
Design Spectral Response Acceleration Parameter, S_{D1}	*0.756
Peak Ground Acceleration (PGA_M)	0.898

Project Site Coordinates: Longitude: W118.333090º Latitude: N33.929175º (WGS84)
*Based on F_v of 1.7. See Section 11.4.8 of ASCE 7-16 for calculation requirements

The site class is determined in accordance with ASCE 7 Chapter 20 using either 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 and 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 count test results presented on the boring logs indicate that the requirements for Class D are met.

13.7. Shallow Foundations

General: For the purpose of preparing this report, we assumed that the proposed building structures will impose maximum column loads of about 25 kips and wall loads less than 2 kips per lineal foot. The recommendations for preparation of the subgrade underlying the footings are provided in the "Earthwork" Section of this report. The Structural Engineer should design foundations in accordance with the requirements of the applicable building code.

Footings should have a minimum width of 2 feet for isolated footings and 18 inches for continuous footings. The bottom of building footings should be located at least 24 inches below the lowest adjacent finish grade, and reinforcement should consist of a minimum of two No. 5 bars, top and bottom or equivalent as determined by the Structural Engineer.

The proposed building structures may be supported on isolated and/or strip footings designed using a net allowable bearing pressure of 2,000 pounds per square foot (psf) for footings supported on at least 2 feet of engineered fill as indicated in the grading section of this report and embedded at least 24 inches below the lowest adjacent grade. This bearing pressure may be increased by 250 psf for each additional foot of depth and 200 psf for each additional foot of width to a maximum of 3,200 psf. A one-third increase in the bearing value may be used when considering wind or seismic loads. In the event of new footings located within one footing width of an existing footing, we recommend reducing the bearing pressure of the new footing by 30 percent.

Minor footings may be required for low height exterior landscape walls (4 feet or less in height), or other small ancillary structures. These footings should be supported on at least 2 feet of new engineered fill and should be embedded at least 18 inches. A vertical bearing pressure of 1,500 psf may be used for these footings.

Lateral Resistance of Footings: 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 0.30 may be used between the footings, floor slabs, and the supporting soils comprised of engineered fill. Where visqueen is used below floor slabs, the friction coefficient 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. 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 first foot 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.

Estimated Settlement of Footings: 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 will be on the order of ¾ inch for the anticipated maximum structural load. The maximum static differential settlement, over a horizontal distance of 40 feet, is anticipated to be on the order of ½ inch for similarly loaded footings. The differential settlement during the design seismic event is anticipated to be on the order of ¼ inch in 40 feet.

13.8. Retaining Walls

We have assumed that retaining walls, if needed, will have heights in the range of 1½ to 6 feet. Design earth pressures for retaining walls depend primarily on the allowable wall movement, wall inclination, type of backfill materials, backfill slopes, surcharges, and drainage. The earth pressures provided assume that non-expansive soil backfill will be used and a drainage system will be installed behind the walls so that external water pressure will not develop. 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 cantilever level-backfilled walls, under static conditions, should be designed to resist a hydrostatic pressure equal to that developed by a fluid with a density of 95 pcf for the full height of the wall.

Determination of whether the active or at-rest condition is appropriate for design will depend on the flexibility of the wall. Walls that are free to rotate at least 0.002 radians (deflection at the top of the wall of at least 0.002 x H, where H is the unbalanced wall height) may be designed for the active condition. Walls that are not capable of this movement should be assumed rigid and designed for at-rest conditions. The recommended static active and at-rest earth pressures are provided in the following table.

Table 2 - Earth Pressures for Retaining Walls, Import Sand Backfill

Wall Movement	Backfill Condition	Equivalent Fluid Pressure
Free to Deflect	Level	40
Restrained	Level	65

The above lateral earth pressures do not include the effects of surcharge (e.g., traffic, footings, sloping ground) or compaction-induced wall pressures. Any surcharge (live, including traffic, dead load, or slope) located within a 1:1 plane drawn upward from the base of the excavation should be added to the lateral earth pressures. The lateral contribution 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. The onsite clay soils should not be used as backfill for the walls unless the soil expansion is considered in the design due to an increase of lateral pressure.

Walls should be waterproofed using appropriate membranes, and properly drained or designed to resist hydrostatic pressures. The waterproofing membrane should be covered with a protection board or equivalent to prevent perforation during backfilling.

Except for the upper 2 feet, the backfill immediately behind retaining walls (minimum horizontal distance of 12 inches measured perpendicular to the wall) should consist of free-draining ¾-inch crushed rock wrapped with filter fabric. The upper 1½ feet of cover backfill should consist of relatively impervious onsite material. A 4-inch diameter perforated PVC pipe, placed perforations down at the bottom of the crushed rock layer, leading to a suitable gravity outlet, should be installed at the base of the walls. As an alternative to extending the crushed rock to within 1½ feet of the ground surface for the wall drain, geocomposite panel drains may be used. With wall drain panels, the 4-inch diameter perforated pipe located at the heel of the wall/footing should be surrounded with one cubic foot of ¾-inch crushed rock wrapped with filter fabric; the pipe invert should be supported on about 1½ inches of crushed rock. All drainage should be directed to the street in non-erosive devices.

In the event of a large earthquake, the lateral earth pressure on walls may be significant. When combining both static and seismic lateral earth pressures, a decreased factor of safety may be used in the design of retaining walls when checking for sliding and overturning stability. For cantilever walls, we have calculated the seismic increment of lateral pressure using the Mononobe-Okabe equation assuming the seismic coefficient to be 0.42 of the peak acceleration (PGA_M). We suggest using a dynamic earth pressure increment of 38 psf/ft for cantilever yielding walls with level backfill assuming the walls will not exceed 8 feet in height. The seismic pressure should be taken as a regular triangular distribution (not inverted). The point of application of the dynamic thrust may be taken at 0.37H above the toe of the wall, where H is the retained height. The Structural

Engineer should determine if a seismic increment of lateral earth pressure is applicable based on wall heights and allowable wall movements.

13.9. 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. A minimum of 4-inch thick bedding material should be placed below the bottom of the utility lines, on a firm and unyielding subgrade. 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 materials or imported granular materials and mechanically compacted to at least 90% of the maximum dry density of the soils.

For utility trenches within the building areas, the backfill should be compacted to the minimum required relative compaction indicated under the "Grading" section of this report. The backfill material should be observed, tested and approved by the Geotechnical Consultant. The trench bedding materials should be placed in accordance with Section 306-6 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 (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.

13.10. Drainage

Foundation, slab, flatwork, and pavement performance depend greatly on proper drainage within and along the boundary of the development. Perimeter grades around the buildings should be sloped in a manner allowing water to drain away from the structures and not pond next to the foundations. Roof downdrains should be connected to underground pipes carrying water away from the structure areas or have extenders so water does not drain and pond next to the structures. Per the 2019 CBC, landscape areas within 10 feet of structures should slope away at gradients of at least 5 percent. Paved areas within 10 feet of structures should slope away at gradients of at least 2 percent. Proper drainage is recommended for all surfaces to reduce the risk of settlement due to hydroconsolidation and heave due to soil expansion.

We recommend minimizing the size and number of planters adjacent to the building and using drought resistant planting. Any planter located within 8 feet of the building should have a solid bottom and a drain outlet to the storm drain. To reduce the potential for overwatering, irrigation should be performed under the management of experienced landscape architects, and not under the control of a landscape contractor.

13.11. Asphalt Concrete (AC) Pavement

The required pavement structural sections depend on the expected wheel loads, volume of traffic, and subgrade soils. Based on soil classification and our experience with R-value correlations with fines contents and plasticity index, an R-value of 10 may be used for pavement design for the clayey subgrade soils. The R-value should be confirmed with additional tests, if necessary, at the time of construction. The following pavement sections are recommended based on assumed traffic indices of 4, 5, 5.5 and 6. We recommend a traffic index of at least 6 for driveways where trucks, including trash trucks and fire trucks will have access. The project Civil Engineer should determine the traffic index to be used for different areas of the site.

Table 3 – Alternative Pavement Sections for Vehicular Traffic

Traffic Index	Asphalt Thickness (Inches)	Base Course (CAB) Thickness (Inches)
4	3.0	6.5
5	3.0	9.5
5.5	3.5	10.5
6	4.0	11.0

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 and asphalt concrete 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 and provide documentation to that effect.

The subgrade underlying the pavement areas should be overexcavated 18 inches below the proposed base course layer. Prior to fill placement, the exposed surface should be scarified to a minimum depth of 8 inches, moisture conditioned within two percent of optimum moisture content for sand and to at least 115 percent of optimum moisture content for clay, and compacted to at least 90% of the maximum dry density obtained per ASTM D1557. The upper 12 inches of subgrade soils should be compacted to 95% relative compaction if sandy soils are present and to 90% for clayey soils. 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 be laterally extended a minimum of 2 feet beyond the perimeters and edges of parking areas, roadways and curbs. Any abandoned footing and/or underground concrete structure within the work limit should be removed entirely and the excavation should be backfilled to grade.

In order to increase pavement performance and extend the pavement life, concrete curbs and gutters could be deepened to extend below the base course material and be seated in the compacted subgrade. Priority should be given to areas where heavier traffic is anticipated and where irrigation may be greater. The intent of deepening the curbs and gutters is to form a "cut-off" wall to reduce the amount of water flow through the base course material from adjacent landscaped areas. Subgrade soils, which become soaked as a result of water flowing through base course material, can reduce the life of the pavement and cause heaving of the pavement. Where feasible, the curbs should be deepened to an elevation at least 6 inches below the bottom level of the proposed base course section.

13.12. 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 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 2012). The aggregate base course should be compacted to at least 95% of the maximum dry density of that material. Crushed Miscellaneous Base (CMB), as defined by Section 200-2.4 of the Greenbook, 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 of 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. The recommended 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	6.0	6.0
Heavy Duty	7.5	6.0

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

- 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.
- Where the pavement does not abut against a curb or gutter, an 8-inch thickened edge should be constructed.
- Pavement section design assumes that proper maintenance such as sealing, and repair of localized distress will be performed on a periodic basis.

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 6 feet unless slabs thicker than 4 inches are used. The pavement sections should be reinforced with No. 3 rebars spaced no further than 18 inches on centers each way to reduce the potential for shrinkage cracking. A thickened edge is recommended at the exterior edge of the flatwork adjacent to landscape subject to irrigation. The thickened edges should be a minimum of 12 inches deep and 8 inches wide and reinforced with two No. 3 bars at the top and bottom. In addition to the thickened edges, the walkway should be underlain by at least 12 inches of non-expansive soils and the upper 12 inches of the clay subgrade below the non-expansive soils should have a moisture content of at least 125 percent of optimum prior to placement of the non-expansive soils. The concrete strength for pedestrian walkways should be at least 2,500 psi unless determined otherwise by the Structural Engineer.

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 vehicle traffic, the pavers should be underlain by one inch of sand and at least 10 inches of aggregate base (CAB). For heavy duty traffic area, we recommend increasing the aggregate base thickness to 16 inches. A separation/reinforcing fabric should be placed on the prepared subgrade prior to placement of the aggregate base.

13.13. Hardscape Play Courts

The overexcavation and subgrade preparation for the play courts such as tennis courts and basketball courts should be as indicated for the for the pavement sections (minimum 18 inches of overexcavation and moisture conditioning to 125% of optimum). If conventional concrete slabs are used, we recommend 5-inch thick slab reinforced with #4 bars at 16 inches on center unless indicated otherwise by the slab designer. The concrete strength should be at least 3500 psi and the slabs should be underlain by at least 4 inches of aggregate base. Any portions of the slab adjacent to irrigated planter should have a reinforced thickened concrete edge extending 18 inches below the ground surface.

13.14. Grading of the Football Field

Any existing pavement, foundations, vegetation, abandoned underground utilities and other debris should be removed from the areas to be graded. All excavation should be backfilled with engineered fill. All topsoil should also be removed. Following removal of topsoil, we recommend overexcavating an additional 12 inches of material. Following geotechnical approval, the bottom of the removal excavations should be scarified to a depth of 10 inches, moisture conditioned within 2 percent of optimum for sand, above optimum for silt and to 125 percent of optimum for clay, and recompacted to at least 90% relative compaction as determined by ASTM D1557. All fill placed should be compacted to at least 90% relative compaction at moisture contents as indicated above.

Depending upon the time of the year the grading occurs and the irrigation of the lawn during the prior months, localized wet soils should be anticipated. These wet soils can induce "pumping conditions" and prevent compaction of the proposed backfill. It should be anticipated that the wet soils will require additional overexcavation and bridging with crushed rock, dryer soils, and with the use of geosynthetic. Landscape irrigation should be discontinued at least 4 weeks before grading to allow some drying at the surface.

It is recommended to slope the subgrade to at least one percent toward the internal drains. The thickness of the drain stone/rock above the subgrade can be adjusted as necessary to obtain the desired synthetic turf grade. The subgrade should be proof rolled using heavy rubber tire equipment or a loaded truck under the observation of the Geotechnical Engineer. Any unsuitable soft spots should be remediated as recommended by the Geotechnical Engineer.

Liner for Synthetic Turf Field

It should be noted that the upper soils within the football field area are relatively impervious and subject to develop "pumping" conditions once they become soaked, thus resulting in unstable subgrade. It is, therefore, recommended that an impervious liner overlain by a drainage system be installed for supporting the synthetic turf. Prior to installation of the membrane, it is recommended to finish rolling the subgrade with a large smooth drum roller (at least 4 by 4 drums). The polyethylene/HDPE impervious liner should be highly flexible, resisting to abrasion, to puncture, and to the anticipated heat of the field. The grab tensile strength (ASTM D7004) and the puncture resistance (ASTM D4833) should be at least 300 pounds and 150 pounds, respectively, unless indicated otherwise by the field designer.

Drainage System

The drainage system is anticipated to consist of aggregate stone, flat drain collectors and perforated piping as designed by the landscape architect and/or the civil engineer. The drainage pipes are normally designed so that the main drain (s) are placed along the side of the field and lateral piping is installed at an angle across the slope of the field. The lateral drains are normally 15 to 20 feet apart. Drainage pipes should maintain a consistent slope to the outlet of at least 0.5 percent. Geotextile fabric is recommended as a barrier between unstable subsoil and the gravel drainage blanket. At least one inch of drain gravel is normally placed below round perforated piping, which should be installed with the perforation holes facing down.

All permeable stone should consist of clean, virgin crushed stone free of shale, soft limestone and sandstone, clay and organic material and any other deleterious material. For field perimeter drain, the drain gravel protected with filter fabric may consist of ¾-inch crushed rock conforming to the Greenbook Specifications or other aggregate as selected by the landscape architect/civil engineer. The crushed rock should be protected by a geotextile such as Mirafi 140N or approved equivalent and the fill cap above the geotextile should be at least 12 inches thick.

The stone mix must have enough fines to be stable but not so much as to reduce drainage excessively. Some construction system calls for a simple structural stone (free draining finishing stone) layer laid directly over the free draining crushed base stone. The sand equivalent of the base stone should be at least 75 and the percentage of wear at 500 revolutions (ASTM C131) should not exceed 40 unless approved otherwise by the designer. The crushed base stone should be at least 6 inches thick and should consist of modified Caltrans Class 2 permeable aggregate with the recommended following grading or approved equivalent by the Field Designer.

Sieve Size	Percent Passing
1”	100
¾”	90-100
½”	50-90
3/8	35-70
No. 4	25-40
No. 8	18-30
#30	6-15
#50	0-7
#100	0-4
#200	0-1

The base stone may fragment at every stage of the construction process and result in higher fine content than determined at the plant due to loading, unloading, spreading and compacting. The stone should be spread in such a way that it is immediately level with the field design gradient. It is important to avoid modifying the grade once the stone is laid out. Stockpile stones tend to separate by size during handling (the finer stones tend to move to the bottom). Preferably, stones should not be mix on site to avoid separation and the optimum moisture should be maintained to provide better cohesion and reduce segregation.

It is recommended to compact the stone base between 92 and 95 percent relative compaction as determined by ASTM D1557 unless determined otherwise by the Field Designer. Over compaction should be avoided since it will cause additional breakdown of the stone particles and may reduce the hydraulic conductivity/permeability. The contractor should establish circulation pattern in the field to avoid concentration of traffic that may result in over compaction/breakdown of the stone in localized areas. The Synthetic Turf Council’s suggests a minimum flow rate through the permeable base of 28 inches per hour. At the discretion of the Field Designer, the flow rate of the base stone should be measured in the field at the time of construction.

Running Track

Within the track area, we recommend overexcavating 18 inches of subgrade material below the pavement section. The overexcavation should extend at least 2 feet outside the track, where feasible. Prior to backfilling, the subgrade should be scarified to a depth of 8 inches, moisture conditioned and recompacted to 90 percent relative compaction. The upper 12 inches of subgrade should be compacted to 92% relative compaction. The excavated material may be used as fill except for high plastic clay soils, if encountered.

The pavement section should consist of 4 inches of asphalt concrete underlain by 6 inches of Crushed Aggregate Base (CAB) as defined by Section 200-2.2 of the Standard Specifications for Public Works Construction ("Greenbook"). A material with shock absorbing properties may be placed over the asphalt concrete. Proper drainage adjacent to the track is essential to prevent soil movement due to water penetration below the ground surface. The curbs should extend 16 inches below the finish grade to serve as a barrier for water migration below the track unless indicated otherwise by the track designer.

14. SOIL EXPANSIVITY

The subsurface soils encountered at shallow depths within the building pad area ranges range from sandy lean clay to clayey sand. These types of material generally have a moderate susceptibility to expansion when facing seasonal cycles of saturation/desiccation. Expansion index testing on shallow samples of Borings B-1 and B-4 indicated values of 14 and 22, respectively, indicating low expansion potential, however, medium expansion potential soils may be present within the proposed improvement areas. As such, the recommendations provided in this report regarding drainage, moisture content during compaction, presoaking, the use of sand blankets and other pertinent recommendations for site improvements should be incorporated into the design and construction.

15. SOIL CORROSIVITY

The corrosion potential of the onsite materials to steel and buried concrete was preliminarily evaluated. Laboratory testing was performed on one 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-4	2 – 3	8060	6.8	60	15

These tests are only an indicator of soil corrosivity for the sample tested. Other soils found on site may be more, less, or of a similar corrosive nature. Imported fill materials should be tested to confirm that their corrosion potential is not significantly more severe than those noted. 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. 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.

17. 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 updated 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.

REGISTERED PROFESSIONAL ENGINEER
JACQUES B. ROY
NO. 2077
Ex. Date 9/30/21
Geotechnical
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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
Seismic Hazard Zones Map – Figure A-4
Historic High Groundwater Map – Figure A-5
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Flood Map – Figure A-7
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Oil and Gas Map – Figure A-9

Appendix B: Field Exploratory Boring Logs

Borings B-1 through B-10

Appendix C: Laboratory Test Results and Calculations

Appendix D: Historical Earthquake Data

EQSEARCH Results

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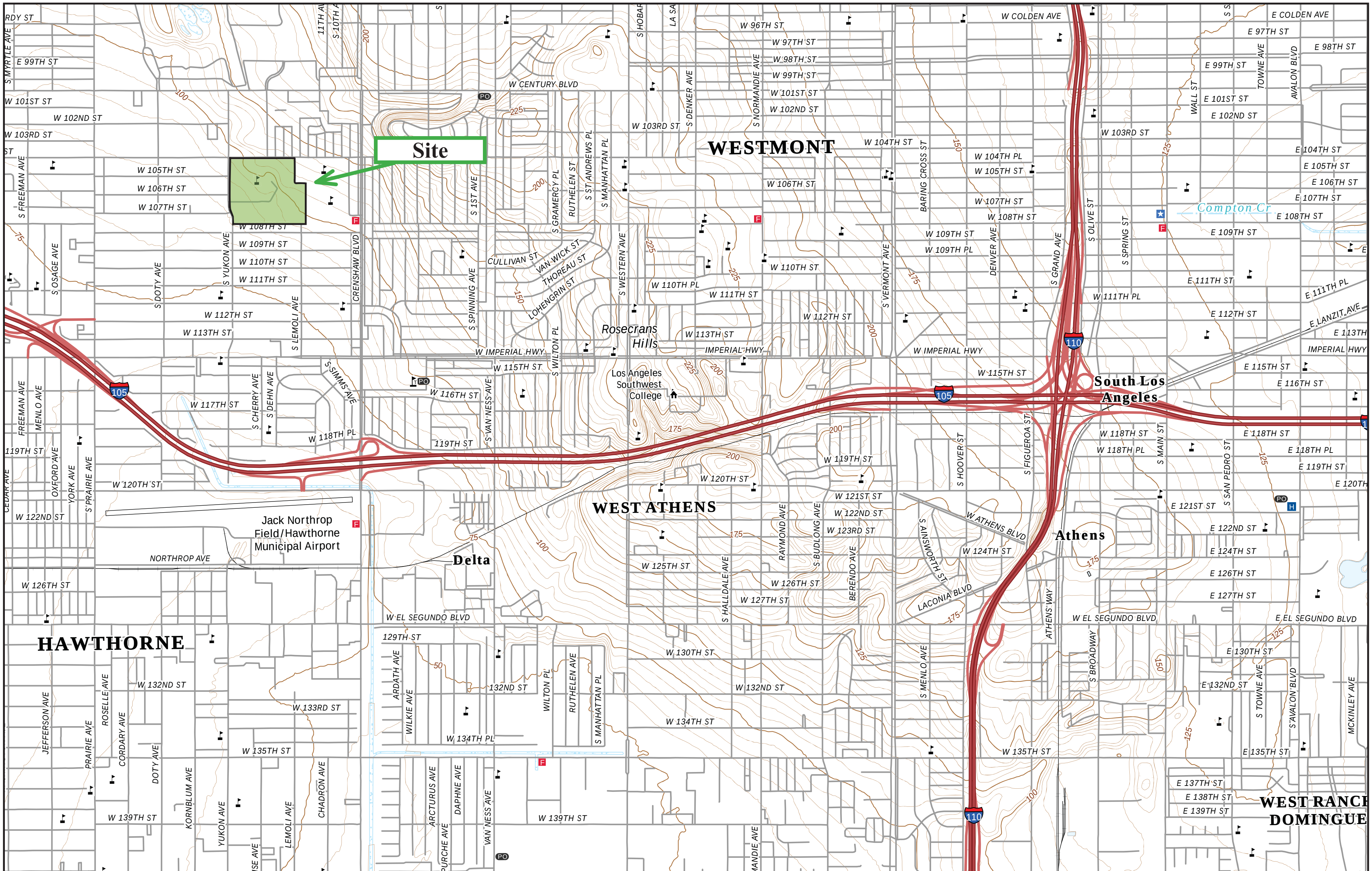
APPENDIX A

Maps and Plans



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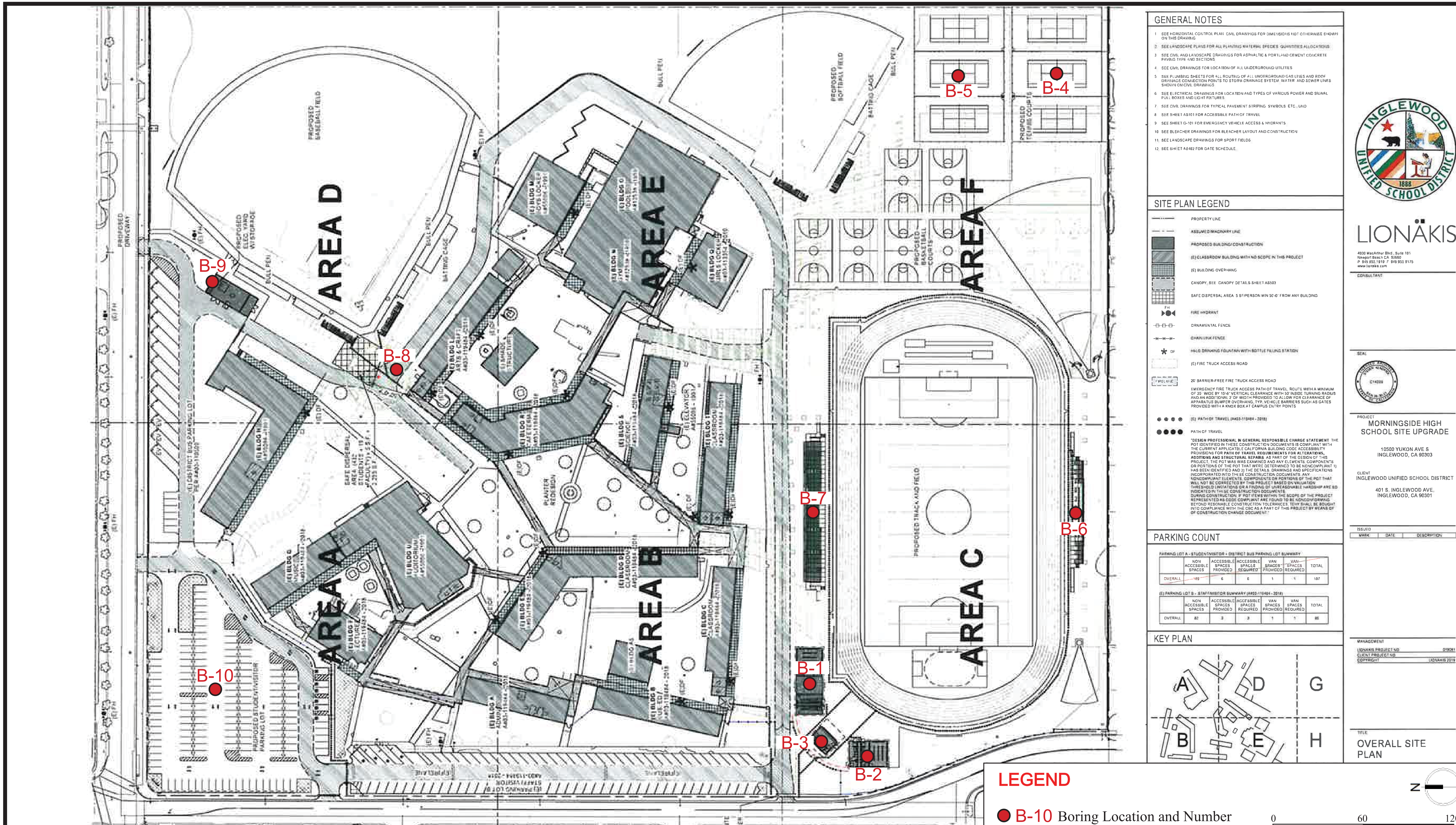
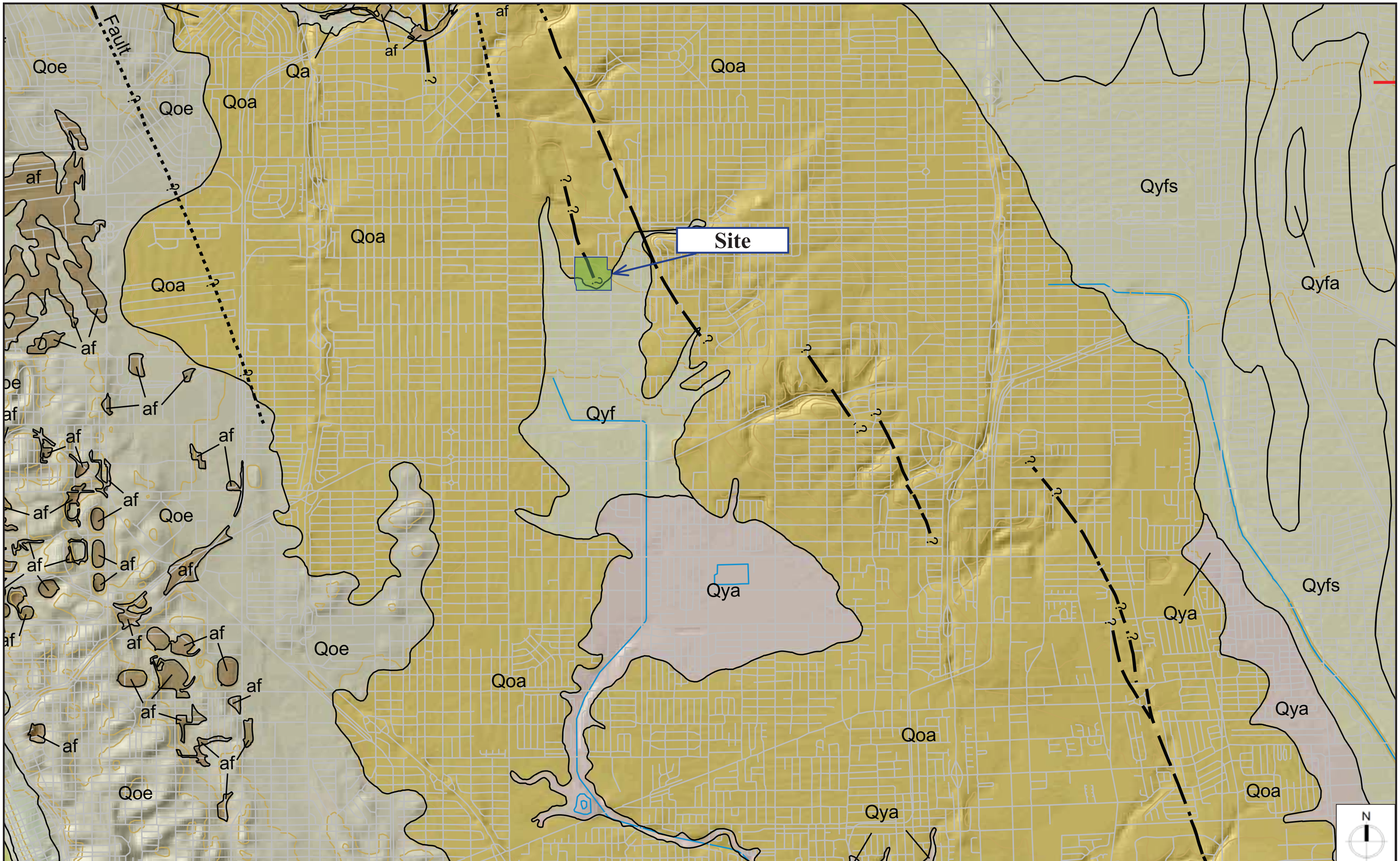
Reference: USGS Topographic Map, Inglewood Quadrangle, California-Los Angeles County, 7.5 Minutes Series, 2018 - Contour Interval 5 ft.

KOURY PASSION INTEGRITY QUALITY	Project Name: Morningside High School	Project No.: 19-1110 Date: January 2020	Drawing Title: Vicinity Map	Figure: A-1



KOURY
PASSION INTEGRITY QUALITY

Project Name: Morningside High School	**Project No.:**	19-1110	**Drawing Title:** Boring Location Map	**Figure:** A-2
	Date:	January 2020		



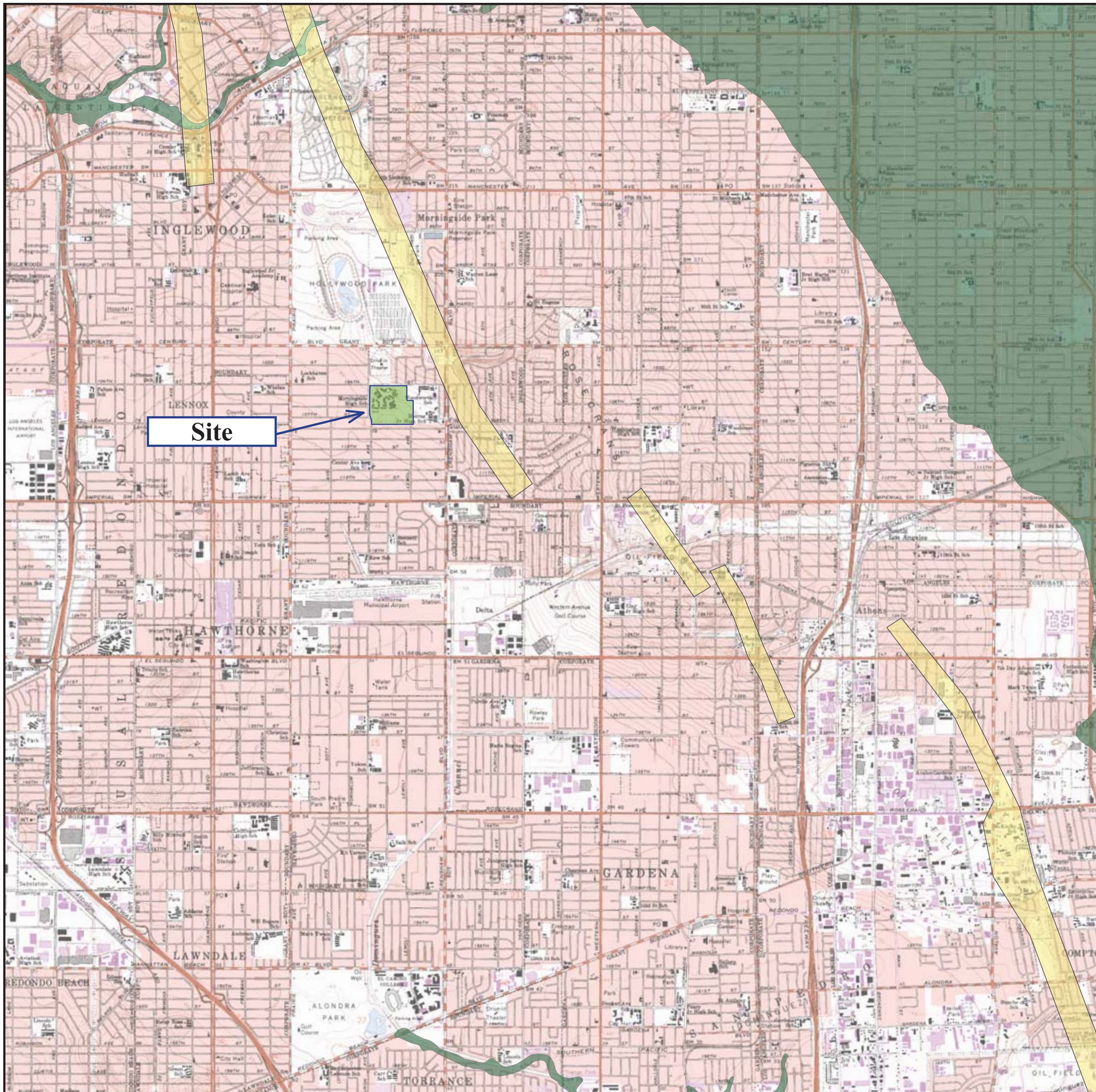
Legend

Qyf	Young Alluvial Fan & Valley Deposits
Qoa	Old Alluvial Flood Plain Deposits

0 1 2 Mile

Reference: Geologic Map of the Long Beach 30' x 60' Quadrangle, Southern California, Version 1.0 - 2003

KOURY PASSION INTEGRITY QUALITY	Project Name: Morningside High School	Project No.: 19-1110 Date: January 2020	Drawing Title: Geology Map	Figure: A-3



Earthquake Zones of Required Investigation
Inglewood Quadrangle

California Geological Survey

This Map Shows Both Alquist-Priolo Earthquake Fault Zones And Seismic Hazard Zones Issued For The Inglewood Quadrangle

This map shows the location of Alquist-Priolo (AP) Earthquake Fault Zones and Seismic Hazard Zones, collectively referred to here as Earthquake Zones of Required Investigation. The Geographic Information System (GIS) digital files of these regulatory zones released by the California Geological Survey (CGS) are the "Official Maps." GIS files are available at the CGS website http://maps.conservation.ca.gov/cgs/informationwarehouse/. These zones will assist cities and counties in fulfilling their responsibilities for protecting the public from the effects of surface fault rupture and earthquake-triggered ground failure as required by the AP Earthquake Fault Zoning Act (Public Resources Code Sections 2621-2630) and the Seismic Hazards Mapping Act (Public Resources Code Sections 2690-2699.6). For information regarding the general approach and recommended methods for preparing these zones, see CGS Special Publication 42, *Earthquake Fault Zones, a Guide for Government Agencies, Property Owners/Developers, and Geoscience Practitioners for Assessing Fault Rupture Hazards in California*, Appendix C, and CGS Special Publication 118, *Recommended Criteria for Delineating Seismic Hazard Zones in California*.

For information regarding the scope and recommended methods to be used in conducting required site investigations refer to CGS Special Publication 42, and CGS Special Publication 117A, *Guidelines for Evaluating and Mitigating Seismic Hazards in California*. For a general description of the AP and Seismic Hazards Mapping acts, the zonation programs, and related information, please refer to the website at www.conservation.ca.gov/cgs/.

MAP EXPLANATION

EARTHQUAKE FAULT ZONES

Earthquake Fault Zones
Zone boundaries are delineated by straight-line segments; the boundaries define the zone encompassing active faults that constitute a potential hazard to structures from surface faulting or fault creep such that avoidance as described in Public Resources Code Section 2621.5(a) would be required.

Active Fault Traces
Faults considered to have been active during Holocene time and to have potential for surface rupture: Solid Line in Black or Red where Accurately Located; Long Dash in Black or Solid Line in Purple where Approximately Located; Short Dash in Black or Solid Line in Orange where Inferred; Dotted Line in Black or Solid Line in Rose where Concealed; Query (?) indicates additional uncertainty. Evidence of historic offset indicated by year of earthquake-associated event or C for displacement caused by fault creep.

SEISMIC HAZARD ZONES

Liquefaction Zones
Areas where historical occurrence of liquefaction, or local geological, geotechnical and ground water conditions indicate a potential for permanent ground displacements such that mitigation as defined in Public Resources Code Section 2693(c) would be required.

Earthquake-Induced Landslide Zones
Areas where previous occurrence of landslide movement, or local topographic, geological, geotechnical and subsurface water conditions indicate a potential for permanent ground displacements such that mitigation as defined in Public Resources Code Section 2693(c) would be required.

OVERLAPPING EARTHQUAKE FAULT AND SEISMIC HAZARD ZONES

Overlap of Earthquake Fault Zone and Liquefaction Zone
Areas that are covered by both Earthquake Fault Zone and Liquefaction Zone.

Overlap of Earthquake Fault Zone and Earthquake-Induced Landslide Zone
Areas that are covered by both Earthquake Fault Zone and Earthquake-Induced Landslide Zone.

Note: Mitigation methods differ for each zone – AP Act only allows avoidance; Seismic Hazard Mapping Act allows mitigation by engineering/geotechnical design as well as avoidance.

ADDITIONAL INFORMATION

For additional information on the zones of required investigation presented on this map, the data and methodology used to prepare them, and additional references consulted, please refer to the following:

The Northern Newport-Inglewood Fault Zone, in the Long Beach, Inglewood, Hollywood, and Beverly Hills Quadrangles, Ventura County, California
California Geological Survey, Fault Evaluation Report FER-173,
http://gmw.conservation.ca.gov/SHP/EZRIM/Reports/FER/173/

For more information on the Alquist-Priolo Earthquake Fault Zoning Act please refer to:
http://www.conservation.ca.gov/cgs/rghm/ap/Pages/main.aspx

Seismic Hazard Zone Report for the Inglewood 7.5-minute Quadrangle, Los Angeles County, California
California Geological Survey, Seismic Hazard Zone Report 027,
http://gmw.conservation.ca.gov/SHP/EZRIM/Reports/SHZR/SHZR_027_Inglewood.pdf

For more information on the Seismic Hazards Mapping Act please refer to:
http://www.conservation.ca.gov/cgs/shzp/Pages/SHMPpgminfo.aspx

Click the link below to learn how to take greater advantage of the GeoPDF format of this map after downloading.
http://gmw.conservation.ca.gov/SHP/EZRIM/Docs/TerragoUserGuide.pdf

INGLEWOOD QUADRANGLE

EARTHQUAKE FAULT ZONES	SEISMIC HAZARD ZONES
Delineated in compliance with Chapter 7.5 Division 2 of the California Public Resources Code (*Alquist-Priolo Earthquake Fault Zoning Act*)	Delineated in compliance with Chapter 7.8 Division 2 of the California Public Resources Code (*Seismic Hazards Mapping Act*)
REVISED OFFICIAL MAP	**OFFICIAL MAP**
Released: July 1, 1986	Released: March 25, 1999
James F. Davis, STATE GEOLOGIST	James F. Davis, STATE GEOLOGIST

0 0.5 1 Mile

N

KOURY PASSION INTEGRITY QUALITY	Project Name: Morningside High School	Project No.: 19-1110 Date: January 2020	Drawing Title: Seismic Hazard Zones Map	Figure: A-4

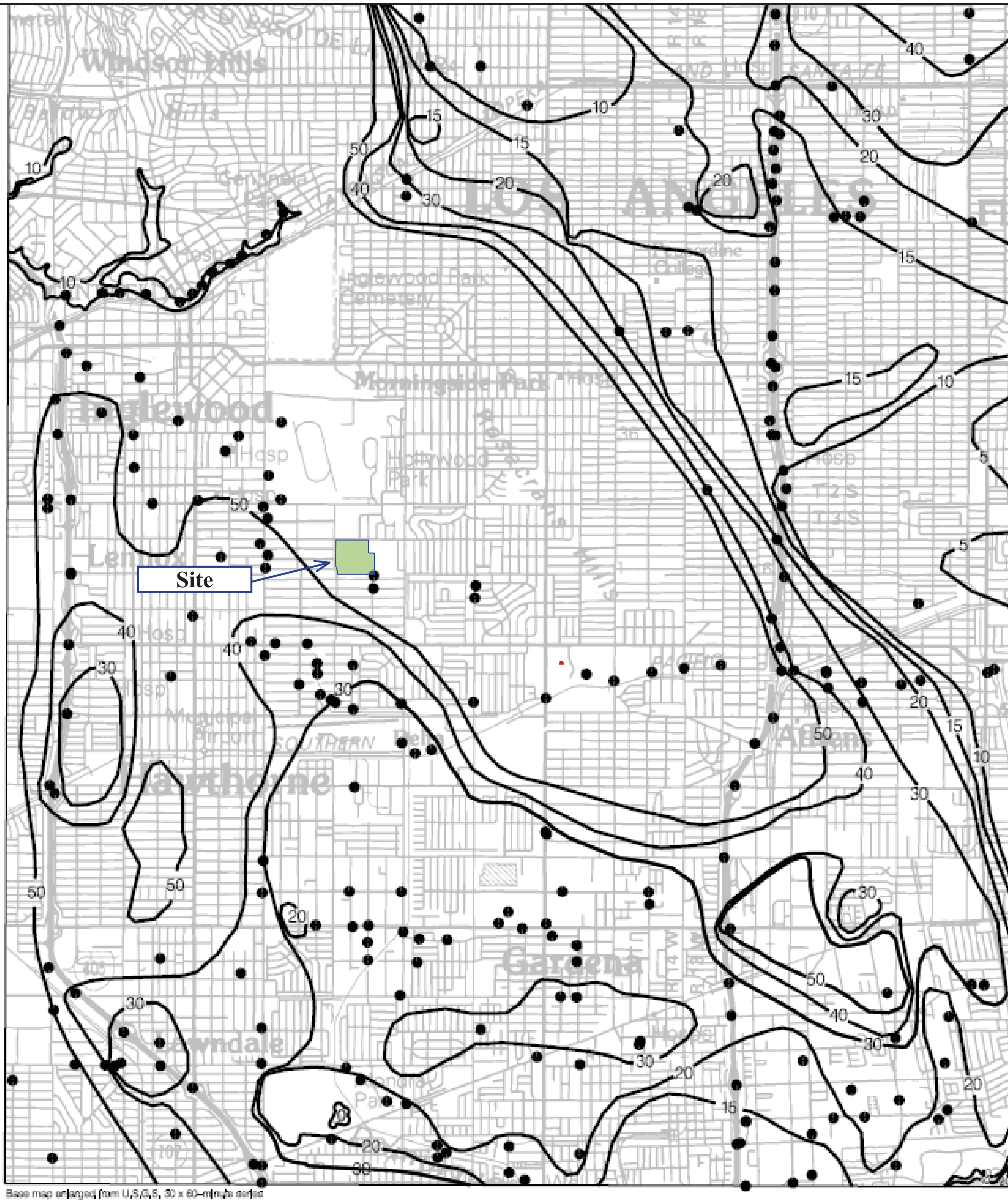
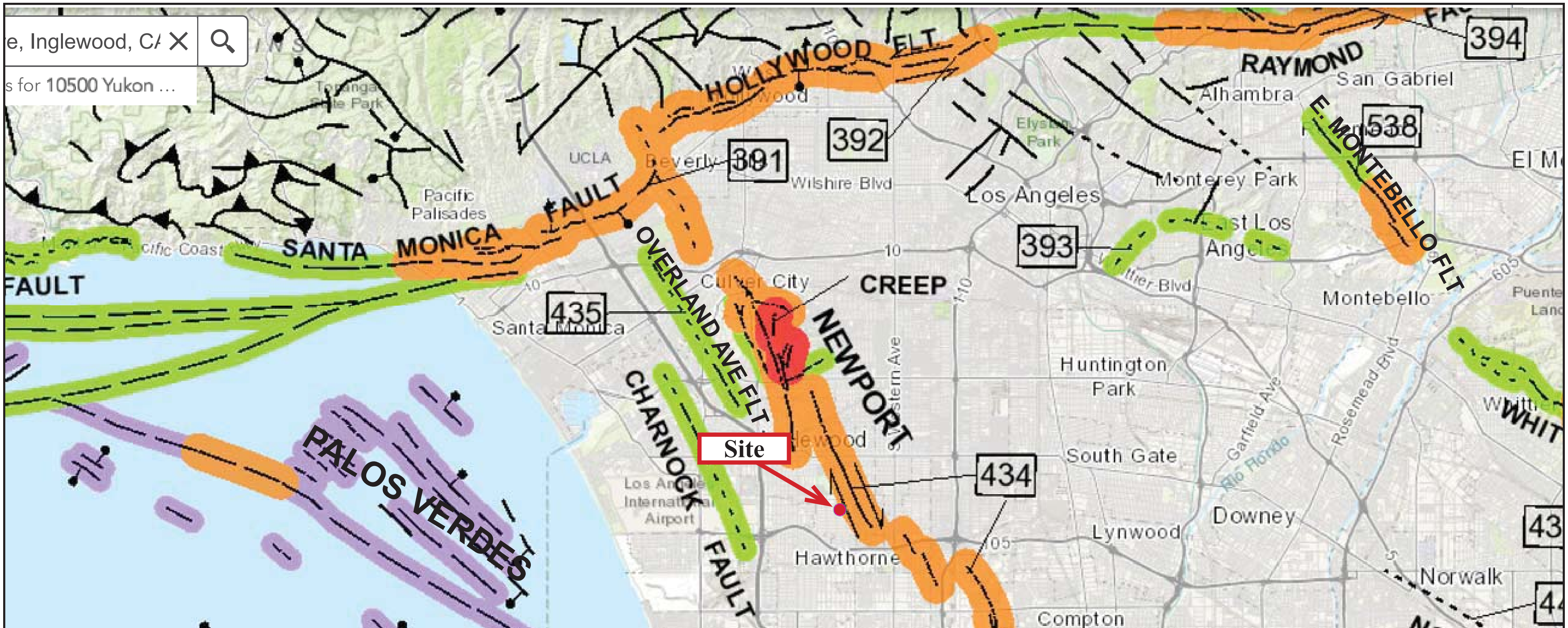


Plate 1.2 Historically Highest Ground Water Contours and Borehole Log Data Locations, Inglewood Quadrangle.

● Borehole Site

30 Depth to ground water in feet

KOURY	Project Name: Morningside High School	Project No.: 19-1110 Date: January 2020	Drawing Title: Historic High Groundwater Map	Figure: A-5



Geologic Time Scale			Years Before Present (Approx.)	Fault Symbol	Recency of Movement	DESCRIPTION ON LAND	DESCRIPTION OFFSHORE
Quaternary	Late Quaternary	Historic				Displacement during historic time (e.g. San Andreas fault 1906). Includes areas of known fault creep.	
		Holocene	200			Displacement during Holocene time.	Fault offsets seafloor sediments or strata of Holocene age.
		Pleistocene	11,700			Faults showing evidence of displacement during late Quaternary time.	Fault cuts strata of Late Pleistocene age.
	Early Quaternary		700,000			Undivided Quaternary faults - most faults in this category show evidence of displacement during the last 1,600,000 years; possible exceptions are faults which displace rocks of undifferentiated Plio-Pleistocene age.	Fault cuts strata of Quaternary age.
Pre-Quaternary			1,600,000* – 4.5 billion (Age of Earth)			Faults without recognized Quaternary displacement or showing evidence of no displacement during Quaternary time. Not necessarily inactive.	Fault cuts strata of Pliocene or older age.

* Quaternary now recognized as extending to 2.6 Ma (Walker and Geissman, 2009). Quaternary faults in this map were established using the previous 1.6 Ma criterion.

Reference: Fault Activity Map of California (2010) - California Geological Survey
Department of Conservation Web Site @ http://maps.conservation.ca.gov/cgs/fam/ - See Figure A-6a, for explanation

0 4 Mile

N

KOURY PASSION INTEGRITY QUALITY	Project Name: Morningside High School	Project No.: 19-1110 Date: January 2020	Drawing Title: Fault Map	Figure: A-6

National Flood Hazard Layer FIRMette

FEMA

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, Ar of 1% annual chance flood with avera depth less than one foot or with drain areas of less than one square mile Zor
- 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

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

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

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

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

N

The pin displayed on the map is an approxim point selected by the user and does not repre 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 6/21/2019 at 12:21:48 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.

0 500’

KOURY PASSION INTEGRITY QUALITY	Project Name: Morningside High School	Project No.: 19-1110 Date: January 2020	Drawing Title: Flood Map	Figure: A-7

U.S. Seismic Design Maps

OSHPD

Morningside High School

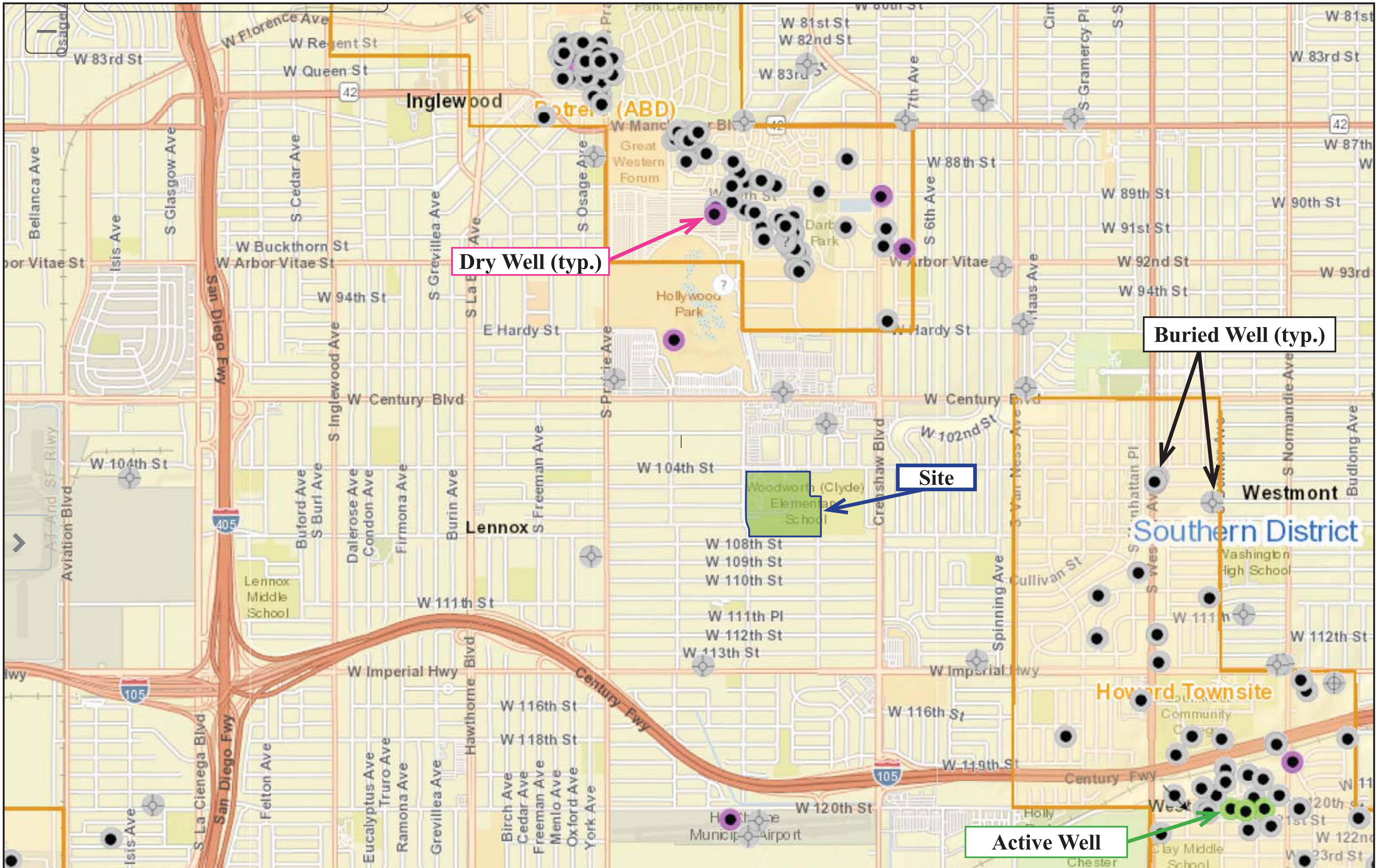
Latitude, Longitude: 33.941487, -118.333090

Date	10/28/2019, 4:04:21 PM
Design Code Reference Document	ASCE7-16
Risk Category	III
Site Class	D - Stiff Soil

Type	Value	Description
S_S	1.895	MCE_R ground motion. (for 0.2 second period)
S_1	0.667	MCE_R ground motion. (for 1.0s period)
S_{MS}	1.895	Site-modified spectral acceleration value
S_{M1}	null- See Section 11.4.8	Site-modified spectral acceleration value
S_{DS}	1.263	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	D	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	0.819	MCE_G peak ground acceleration
F_{PGA}	1.1	Site amplification factor at PGA
PGA_M	0.898	Site modified peak ground acceleration
T_L	8	Long-period transition period in seconds
SsRT	1.895	Probabilistic risk-targeted ground motion. (0.2 second)
SsUH	2.1	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration
SsD	2.461	Factored deterministic acceleration value. (0.2 second)
S1RT	0.667	Probabilistic risk-targeted ground motion. (1.0 second)
S1UH	0.74	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration.
S1D	0.832	Factored deterministic acceleration value. (1.0 second)
PGAd	0.996	Factored deterministic acceleration value. (Peak Ground Acceleration)
C_{RS}	0.902	Mapped value of the risk coefficient at short periods
C_{R1}	0.901	Mapped value of the risk coefficient at a period of 1 s

KOURY PASSION INTEGRITY QUALITY	Project Name: Morningside High School	Project No.: 19-1110 Date: January 2020	Drawing Title: Response Spectrum	Figure: A-8



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

0 — 1 Mile

N

Project Name: Morningside High School	Project No.: 19-1110 Date: January 2020	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. : 19-1110
Project Name : Morningside High School Site Upgrades
Boring No. : B-1
Sheet : 1 **of :** 1
Drilling Method : Hollow Stem 6" Auger
Sampling Method : Bulk - CD - SPT
Ground Elevation:
Hammer Weight : 140 lbs **Drop Height :** 30"
Drilling Co. : Cal Pac Drilling
Location : See Figure A-2
Date Drilled : 1/02/20

Sample No.	Moisture Content (%)	Dry Unit Weight (pcf)	Blows per 6"	Depth (ft)	Soil Type (USCS)	Description	Additional Tests
				0	AC/AB	5" asphalt concrete over 3" of aggregate base	
1	15.0					**FILL: Sandy Lean CLAY;** trace of gravel, stiff, moist to very moist, dark brown	EI = 14
2	12.4		4 5 7		CL	**ALLUVIUM:** **Sandy Lean CLAY;** stiff to very stiff, moist, yellowish brown	#200 Wash Fines = 54% PP = 3.5-4.5 tsf
3	12.4	124	4 7 11	5			PP = 4.5 tsf
4	14.0		4 8 10		SM	**Silty SAND;** fine to medium, medium dense, dark brown with grayish brown	#200 Wash Fines = 26% PP = 4.5 tsf
5	15.1	121	9 16 26	10	CL	**Sandy Lean CLAY;** very stiff, moist to very moist, dark brown with grayish brown inclusions	#200 Wash Fines = 70% PP = 4.5 tsf
6	13.6		5 8 15	15			#200 Wash Fines = 50% PP = 4.5 tsf
7	13.9	122	5 10 25	20		layers of clayey sand	
						End of Boring @ 21' 6" No groundwater encountered	
				25			
				30			
				35			
				40			

Groundwater ▽ Bulk ☒ CD ■ SPT ☒

Boring Log

KOURY
PASSION INTEGRITY QUALITY

Project No. : 19-1110
Project Name : Morningside High School Site Upgrades
Drilling Method : Hollow Stem 6" Auger
Sampling Method : Bulk - CD - SPT
Hammer Weight : 140 lbs **Drop Height :** 30"
Location : See Figure A-2

Boring No. : B-2
Sheet : 1 **of :** 1
Ground Elevation:
Drilling Co. : Cal Pac Drilling
Date Drilled : 1/02/20

Sample No.	Moisture Content (%)	Dry Unit Weight (pcf)	Blows per 6"	Depth (ft)	Soil Type (USCS)	Description	Additional Tests
				0	AC/AB	5" of asphalt concrete over 3" of aggregate base	
1	11.7				SC	**FILL:** **Clayey SAND;** fine to medium, medium dense, moist, dark brown	
2	11.0	119	2 2 3				#200 Wash Fines = 43% PP = 3.0-4.0 tsf
3	12.1		3 5 7	5	CL	**ALLUVIUM:** **Sandy Lean CLAY;** lenses and layers of clayey sand, stiff, moist to very moist, dark brown	PP = 4.5 tsf
4	14.9	123	7 13 21				#200 Wash Fines = 70% PP = 4.5 tsf
5	17.9		4 7 12	10			#200 Wash Fines = 74% PP = 4.5 tsf
6	18.6	116	4 15 29	15			PP = 4.5 tsf
7	15.7		4 8 14	20			#200 Wash Fines = 74% PP = 4.0-4.5 tsf
				25		End of Boring @ 21' 6" No groundwater encountered	
				30			
				35			
				40			

Groundwater ▽ Bulk ☒ CD ■ SPT ☒

Boring Log

KOURY — PASSION INTEGRITY QUALITY

Project No. : 19-1110
Project Name : Morningside High School Site Upgrades
Boring No. : B-3
Sheet : 1 **of :** 1
Drilling Method : Hollow Stem 6" Auger
Sampling Method : Bulk - CD - SPT
Ground Elevation:
Hammer Weight : 140 lbs **Drop Height :** 30"
Drilling Co. : Cal Pac Drilling
Location : See Figure A-2
Date Drilled : 1/02/20

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 2" of aggregate base	
1	13.1				Bulk			**FILL:** **Sandy Lean CLAY;** firm to stiff, moist, dark brown	
2	12.6		4 7 9		SPT			**ALLUVIUM:** **Sandy Lean CLAY;** stiff to very stiff, moist moist to very moist, dark brown	#200 Wash Fines = 62% PP = 4.0-4.5 tsf
3	15.4	120	7 10 14	5	CD			lenses of clayey sand	#200 Wash Fines = 57% PP = 4.0-4.5 tsf Consolidation
4	13.2		4 7 12		SPT				#200 Wash Fines = 73% PP = 4.5 tsf
5	15.8	118	5 12 23	10	CD		CL		PP = 4.5 tsf Consolidation
6	14.3		5 10 16	15	SPT				#200 Wash Fines = 59% PP = 4.5 tsf
7	21.0	110	6 12 25	20	CD			lenses of clayey sand	#200 Wash Fines = 74% PP = 4.5 tsf
								End of Boring @ 21' 6" No groundwater encountered	
				25					
				30					
				35					
				40					

Groundwater ▽ Bulk ☒ CD ■ SPT ☒

Boring Log

KOURY
PASSION INTEGRITY QUALITY

Project No. : 19-1110
Project Name : Morningside High School Site Upgrades
Boring No. : B-4
Sheet : 1 of : 1
Drilling Method : Hollow Stem 6" Auger
Sampling Method : Bulk - CD - SPT
Ground Elevation:
Hammer Weight : 140 lbs   Drop Height : 30"
Drilling Co. : Cal Pac Drilling
Location : See Figure A-2
Date Drilled : 1/02/20

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	3.5" of asphalt concrete over 6" of aggregate base	
1	16.2				Bulk		CL	**FILL:** **Sandy Lean CLAY;** firm, moist to very moist, very dark brown	#200 Wash Fines = 70% EI = 22 Corrosivity
2	15.8		1 1 2		SPT		CL		
3	15.0	120	5 11 20	5	CD		CL	**ALLUVIUM:** **Sandy Lean CLAY;** firm to stiff, moist to very moist, dark brown	#200 Wash Fines = 70% PP = 4.5 tsf Direct Shear
				10				End of Boring @ 6' 6" No groundwater encountered	
				15					
				20					
				25					
				30					
				35					
				40					

Groundwater   Bulk   CD   SPT

Boring Log

KOURY
PASSION INTEGRITY QUALITY

Project No. : 19-1110
Project Name : Morningside High School Site Upgrades
Drilling Method : Hollow Stem 6" Auger
Sampling Method : Bulk - CD - SPT
Hammer Weight : 140 lbs Drop Height : 30"
Location : See Figure A-2

Boring No. : B-5
Sheet : 1 of : 1
Ground Elevation:
Drilling Co. : Cal Pac Drilling
Date Drilled : 1/02/20

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	6" of asphalt concrete	
1	15.4				Bulk			**FILL: Sandy Lean CLAY;** firm to stiff, moist to very moist, very dark brown	#200 Wash Fines = 63% PP = 1.5 tsf
2	13.7		3 6 10		SPT		CL	**ALLUVIUM:** **Sandy Lean CLAY;** very stiff, moist, dark brown	
3	12.9	127	11 21 37	5	CD				#200 Wash Fines = 50% PP = 4.5 tsf
								End of Boring @ 6' 6" No groundwater encountered	

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

Groundwater Bulk CD SPT

Boring Log

KOURY — PASSION INTEGRITY QUALITY

Project No. : 19-1110
Project Name : Morningside High School Site Upgrades
Boring No. : B-6
Sheet : 1 **of :** 1
Drilling Method : Hollow Stem 6" Auger
Sampling Method : Bulk - CD - SPT
Ground Elevation:
Hammer Weight : 140 lbs **Drop Height :** 30"
Drilling Co. : Cal Pac Drilling
Location : See Figure A-2
Date Drilled : 1/02/20

Sample No.	Moisture Content (%)	Dry Unit Weight (pcf)	Blows per 6"	Depth (ft)	Sample Location	Soil Type (USCS)	Description	Additional Tests
1	6.3			0	Bulk	CL	Grass over topsoil **FILL: Sandy Lean CLAY;** stiff, moist, dark brown	#200 Wash Fines = 52%
2	4.2	116	10 12 16		CD	SC	**Clayey SAND;** fine to medium, medium dense, moist, brown	#200 Wash Fines = 32% PP = 4.5 tsf
3	9.6		3 6 6	5	SPT	CL	**ALLUVIUM:** **Sandy Lean CLAY;** stiff to very stiff, moist to very moist, dark brown	PP = 4.5 tsf
4	14.8	121	9 15 28		CD			#200 Wash Fines = 72% PP = 4.5 tsf
5	12.7		5 13 15	10	SPT		layers of clayey sand	#200 Wash Fines = 51% PP = 4.5 tsf
6	14.8	122	6 21 36	15	CD			#200 Wash Fines = 53% PP = 4.5 tsf
7	13.0		5 9 14	20	SPT	SC	**Clayey SAND;** fine to medium, medium dense, moist, dark brown	#200 Wash Fines = 35% PP = 4.5 tsf
8	4.4	108	5 12 24	25	CD	SP-SM	**Poorly Graded SAND with SILT;** fine to medium, medium dense to dense, slightly moist, yellowish brown	
				30			End of Boring @ 26' 6" No groundwater encountered	
				35				
				40				

Groundwater ▽ Bulk ☒ CD ■ SPT ☒

Boring Log

KOURY
PASSION INTEGRITY QUALITY

Project No. : 19-1110
Project Name : Morningside High School Site Upgrades

Boring No. : B-7
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. : Cal Pac Drilling
Date Drilled : 1/02/20

Sample No.	Moisture Content (%)	Dry Unit Weight (pcf)	Blows per 6"	Depth (ft)	Sample Location	Soil Type (USCS)	Description	Additional Tests
1	8.9			0	Bulk	CL	Grass over topsoil **FILL: Sandy Lean CLAY;** stiff, moist, brown	
2	8.6		13 17 17		SPT	CL	**ALLUVIUM:** **Sandy Lean CLAY;** very stiff to hard, moist, dark brown	#200 Wash Fines = 68% PP = 4.5 tsf
3	10.3	126	13 25 43	5	CD	CL		PP = 4.5 tsf
4	15.1		7 11 19		SPT	CL	**Lean CLAY with SAND;** very stiff to hard, moist to very moist, dark brown	#200 Wash Fines = 83% PP = 4.5 tsf
5	16.5	118	9 24 50	10	CD	CL		
6	15.2		8 13 22	15	SPT	CL	**Sandy Lean CLAY;** hard, moist to very moist, dark brown	#200 Wash Fines = 71% PP = 4.5 tsf
7	3.7	119	11 21 43	20	CD	SM	**Silty SAND;** fine to medium, medium dense, slightly moist, brown	#200 Wash Fines = 14% PP = 4.5 tsf
8	8.9		7 9 15	25	SPT	SM		#200 Wash Fines = 35%
9	10.7	113	8 15 29	30	CD	SM	grayish brown	#200 Wash Fines = 43%
10	2.4		10 15 19	35	SPT	SP-SM	**Poorly Graded SAND with SILT;** fine, dense, slightly moist, pale brown	#200 Wash Fines = 8%
				40		SC	**Clayey SAND**; layers of sandy clay, fine to medium, medium dense to dense, moist, grayish brown with dark brown	

Groundwater Bulk CD SPT

Boring Log

KOURY
PASSION INTEGRITY QUALITY

Project No. : 19-1110
Project Name : Morningside High School Site Upgrades
Boring No. : B-7
Sheet : 2 **of :** 2
Drilling Method : Hollow Stem 6" Auger
Sampling Method : Bulk - CD - SPT
Ground Elevation:
Hammer Weight : 140 lbs **Drop Height :** 30"
Drilling Co. : Cal Pac Drilling
Location : See Figure A-2
Date Drilled : 1/02/20

Sample No.	Moisture Content (%)	Dry Unit Weight (pcf)	Blows per 6"	Depth (ft)	Sample Location	Graphic Log	Soil Type (USCS)	Description	Additional Tests
11	11.5	124	9 22 36	40	CD		SC	**Clayey SAND;** layers of sandy clay, fine to medium, medium dense to dense, moist, grayish brown with dark brown	#200 Wash Fines = 48%
								End of Boring @ 41' 6" No groundwater encountered	
				45					
				50					
				55					
				60					
				65					
				70					
				75					
				80					

Groundwater Bulk CD SPT

Boring Log

KOURY
PASSION INTEGRITY QUALITY

Project No. : 19-1110
Project Name : Morningside High School Site Upgrades
Drilling Method : Hollow Stem 6" Auger
Sampling Method : Bulk - CD - SPT
Hammer Weight : 140 lbs Drop Height : 30"
Location : See Figure A-2

Boring No. : B-8
Sheet : 1 of : 1
Ground Elevation:
Drilling Co. : Cal Pac Drilling
Date Drilled : 1/02/20

Sample No.	Moisture Content (%)	Dry Unit Weight (pcf)	Blows per 6"	Depth (ft)	Sample Location	Soil Type (USCS)	Description	Additional Tests
1	17.5			0	Bulk	CL	Grass over topsoil **FILL: Sandy Lean CLAY;** stiff, moist to very moist, dark brown	#200 Wash Fines = 63%
2	9.8		2 3 8		SPT	CL	**ALLUVIUM:** **Sandy Lean CLAY;** stiff to very stiff, moist, brown to dark brown	PP = 3.0-4.5 tsf
3	11.7	127	7 21 33	5	CD	SC	**Clayey SAND;** fine to medium, medium dense, moist, dark brown	#200 Wash Fines = 33% PP = 4.5 tsf
4	12.2		8 12 25		SPT	SC		#200 Wash PP = 4.0-4.5 tsf
5	13.7	121	11 16 31	10	CD	SC		#200 Wash Fines = 48% PP = 4.5 tsf
6	15.4		6 10 16	15	SPT	CL	**Sandy Lean CLAY;** stiff to very stiff, moist to very moist, dark brown	PP = 4.5 tsf
							End of Boring @ 16' 6" No groundwater encountered	

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

Groundwater ▽   Bulk ⊠   CD ■   SPT ⊠

Boring Log

KOURY — PASSION INTEGRITY QUALITY

Project No. : 19-1110
Project Name : Morningside High School Site Upgrades
Boring No. : B-9
Sheet : 1 **of :** 1
Drilling Method : Hollow Stem 6" Auger
Sampling Method : Bulk - CD - SPT
Ground Elevation:
Hammer Weight : 140 lbs **Drop Height :** 30"
Drilling Co. : Cal Pac Drilling
Location : See Figure A-2
Date Drilled : 1/02/20

Sample No.	Moisture Content (%)	Dry Unit Weight (pcf)	Blows per 6"	Depth (ft)	Sample Location	Graphic Log	Soil Type (USCS)	Description	Additional Tests
1	15.5			0	Bulk		CL	Grass over topsoil **FILL: Sandy Lean CLAY;** stiff, moist to very moist, dark brown	#200 Wash Fines = 53%
2	13.5	121	2 6 11		CD		SC	**ALLUVIUM:** **Clayey SAND;** layers of sandy clay, medium dense, moist to very moist, dark brown	PP = 4.0-4.5 tsf
3	14.5		6 9 13	5	SPT				#200 Wash Fines = 46% PP = 4.5 tsf
4	11.4	124	6 11 18	10	CD				#200 Wash Fines = 27%
5	6.0		7 9 12	15	SPT		SP	**Poorly Graded SAND;** fine to medium, medium dense, moist, dark yellowish brown	#200 Wash Fines = 4%
6	31.0	100	7 11 21	20	CD		CL	**Lean CLAY;** lenses of clayey sand, stiff to very stiff, moist to very moist, olive brown	#200 Wash Fines = 90% PP = 1.5-2.0 tsf
								End of Boring @ 21' 6" No groundwater encountered	
				25					
				30					
				35					
				40					

Groundwater ▽ Bulk ☒ CD ■ SPT ☒

Boring Log

KOURY
PASSION INTEGRITY QUALITY

Project No. : 19-1110
Project Name : Morningside High School Site Upgrades
Drilling Method : Hollow Stem 6" Auger
Sampling Method : Bulk - CD - SPT
Hammer Weight : 140 lbs   Drop Height : 30"
Location : See Figure A-2

Boring No. : B-10
Sheet : 1 of : 1
Ground Elevation:
Drilling Co. : Cal Pac Drilling
Date Drilled : 1/02/20

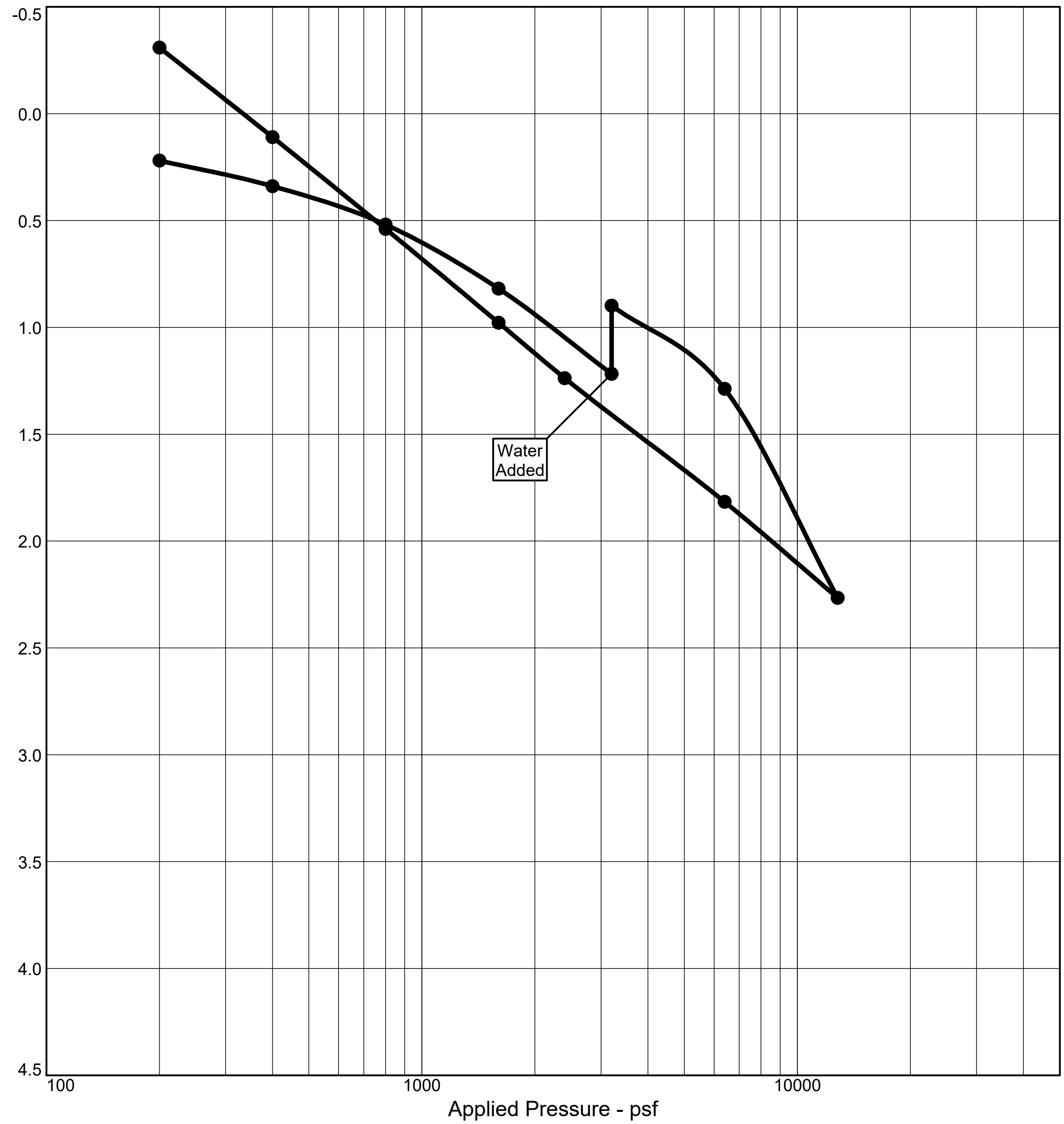
Sample No.	Moisture Content (%)	Dry Unit Weight (pcf)	Blows per 6"	Depth (ft)	Sample Location	Graphic Log	Soil Type (USCS)	Description	Additional Tests
				0			AC/AB	3.5" of asphalt concrete over 2" of aggregate base	
1					Bulk			**FILL: Sandy Lean CLAY;** firm to stiff, moist to very moist, very dark brown	
2	12.0	127	3 6 15		CD		CL	**ALLUVIUM:** **Sandy Lean CLAY;** stiff, moist, lenses of clayey sand, dark brown	#200 Wash Fines = 55% PP = 4.5 tsf
3	11.9		4 7 8	5	SPT				#200 Wash Fines = 50% PP = 4.5 tsf
								End of Boring @ 6' 6" No groundwater encountered	
				10					
				15					
				20					
				25					
				30					
				35					
				40					

Groundwater   Bulk   CD   SPT

APPENDIX C

Laboratory Test Results & Calculations

CONSOLIDATION TEST REPORT



Natural Saturation	Natural Moisture	Dry Dens. (pcf)	LL	PI	Sp. Gr.	Overburden (psf)	P_c (psf)	C_c	C_r	Initial Void Ratio
97.5 %	15.3 %	116.9			2.65		6197	0.05	0.02	0.415

MATERIAL DESCRIPTION	USCS	AASHTO
Observed as: Dark Yellowish Brown Sandy Clay	Observed as: CL	

Project No. 19-1110 **Client:**

Project: Morningside HS

Location: B3 @ 9' **Sample Number:** 2020-005 Series

Koury Engineering & Testing, Inc.

Chino, CA

Remarks:
Lab #6349.

Figure

These results are for the exclusive use of the client for whom they were obtained. They apply only to the samples tested and are not indicitive of apparently identical samples.

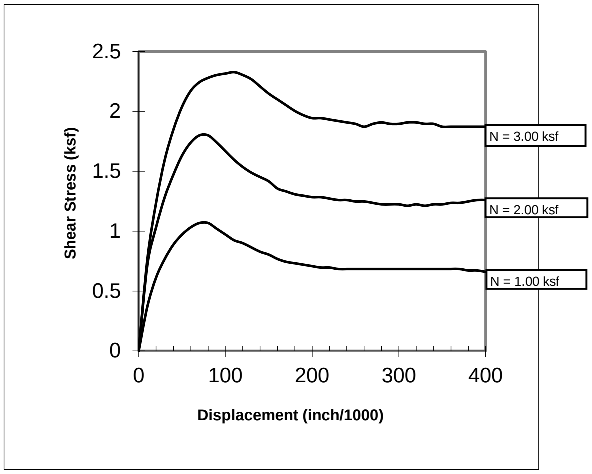
Tested By: Mathew F. Perry **Checked By:**

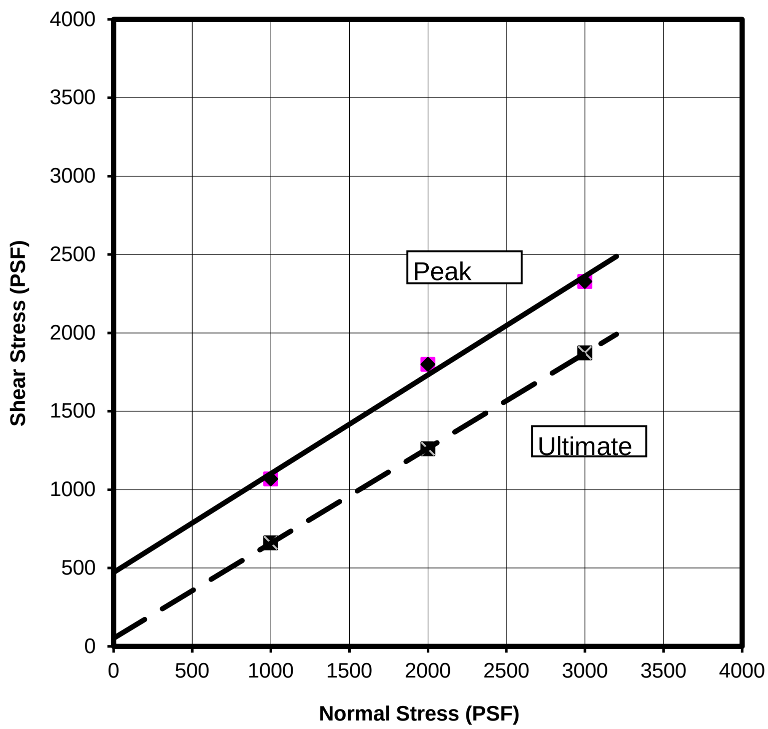
Direct Shear Test Report

Sample Identification	Sample Description	Sample Test State
B1 @ 6'	Brown to Dark Yellowish Brown Clayey Sand	Saturated-Consolidated

Peak:	**Phi (Degrees)**	**32.2**	(Avg.Dry Dens. = 120.5 pcf) (Avg.Moist. = 13.6 %)
	Cohesion (PSF)	**472.0**	
Ultimate:	**Phi (Degrees)**	**31.2**	
	Cohesion (PSF)	**52.0**	

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



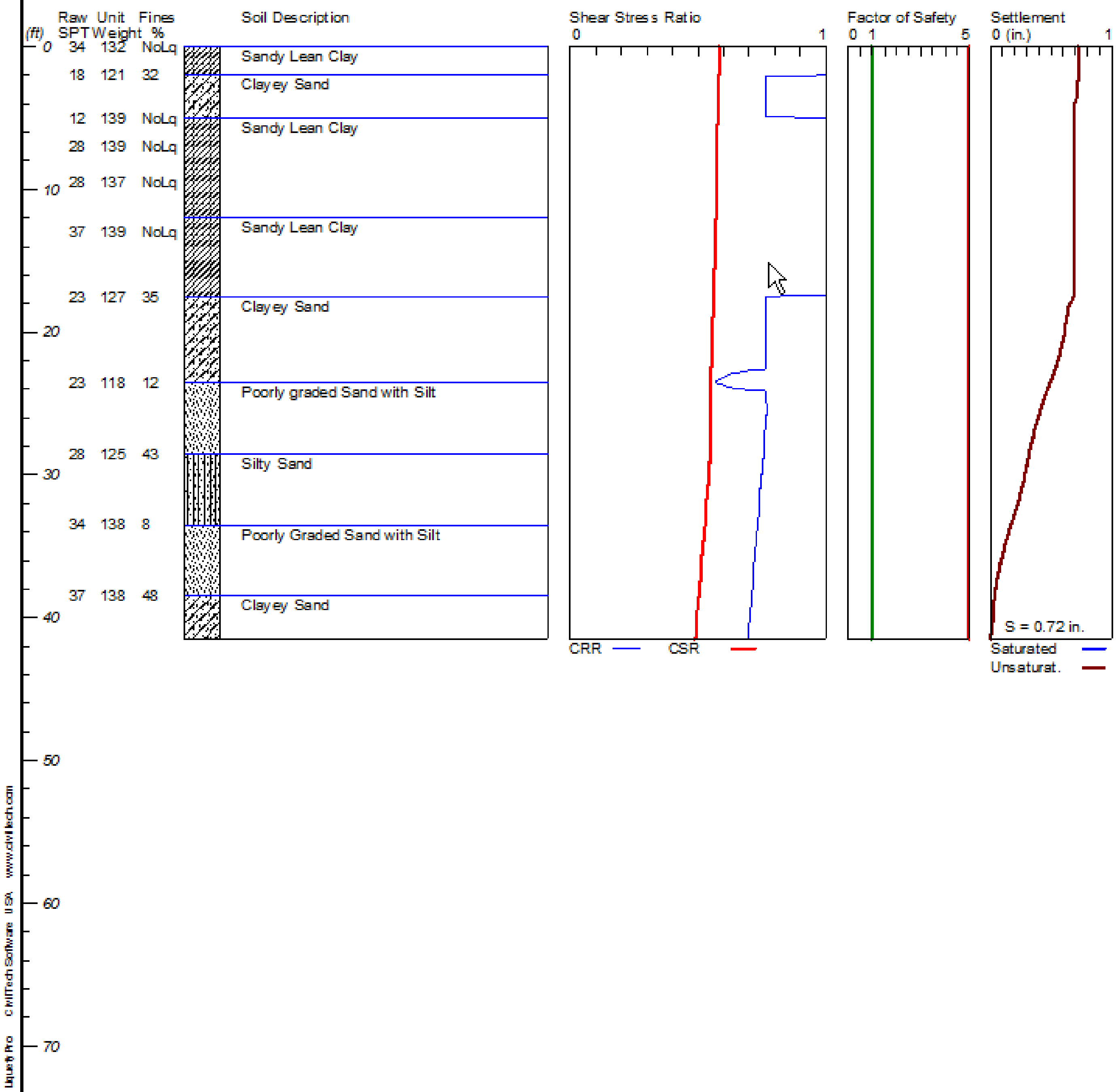


KOURY PASSION INTEGRITY QUALITY	**Project Name:** **Morningside HS**	**Project No.: 19-1110** **Date: 1/10/20**	**Lab #** **6337**

DRY SEISMIC SETTLEMENT

Morningside HS

Hole No.=B-6 *Water Depth=60 ft* *Magnitude=6.35* *Acceleration=0.898g*

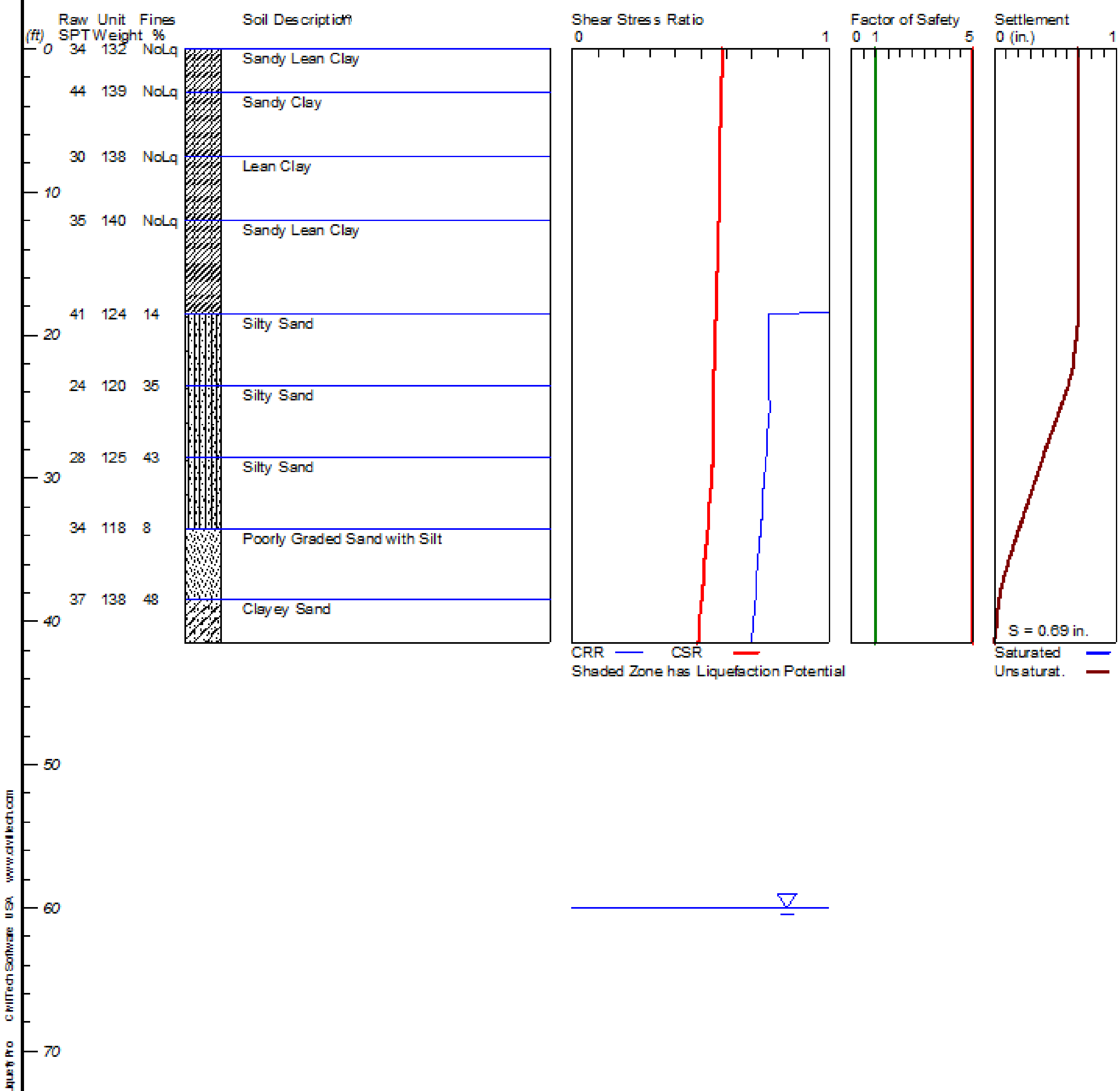


DRY SEISMIC SETTLEMENT

Morningside HS

Hole No.=B-7   Water Depth=60 ft

Magnitude=6.35
Acceleration=0.898g



APPENDIX D

Historical Earthquake Data

```
                    ************************
                    *                      *
                    *    E Q S E A R C H   *
                    *                      *
                    *     Version 3.00     *
                    *                      *
                    ************************

                          ESTIMATION OF
                     PEAK ACCELERATION FROM
                 CALIFORNIA EARTHQUAKE CATALOGS


JOB NUMBER: 19-1110
                                                     DATE: 01-21-2020

JOB NAME: Morningside HS

EARTHQUAKE-CATALOG-FILE NAME: ALLQUAKE.DAT


MAGNITUDE RANGE:
   MINIMUM MAGNITUDE:  5.00
   MAXIMUM MAGNITUDE:  9.00

SITE COORDINATES:
   SITE LATITUDE:  33.9415
   SITE LONGITUDE:  118.3331

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	118.3000	09/03/1905	540 0.0	0.0	5.30	0.222	IX	4.5(7.2)
T-A	34.0000	118.2500	09/23/1827	0 0 0.0	0.0	5.00	0.131	VIII	6.2(10.0)
T-A	34.0000	118.2500	01/10/1856	0 0 0.0	0.0	5.00	0.131	VIII	6.2(10.0)
T-A	34.0000	118.2500	03/26/1860	0 0 0.0	0.0	5.00	0.131	VIII	6.2(10.0)
DMG	33.8500	118.2670	03/11/1933	1425 0.0	0.0	5.00	0.113	VII	7.4(11.8)
DMG	34.0000	118.5000	08/04/1927	1224 0.0	0.0	5.00	0.078	VII	10.4(16.7)
MGI	34.0000	118.5000	11/19/1918	2018 0.0	0.0	5.00	0.078	VII	10.4(16.7)
MGI	34.0800	118.2600	07/16/1920	18 8 0.0	0.0	5.00	0.077	VII	10.4(16.8)
DMG	33.7830	118.2500	11/14/1941	84136.3	0.0	5.40	0.092	VII	11.9(19.2)
DMG	33.7830	118.1330	10/02/1933	91017.6	0.0	5.40	0.064	VI	15.9(25.5)
PAS	34.0730	118.0980	10/04/1987	105938.2	8.2	5.30	0.057	VI	16.2(26.1)
PAS	34.0610	118.0790	10/01/1987	144220.0	9.5	5.90	0.089	VII	16.7(26.9)
PAS	33.9190	118.6270	01/19/1989	65328.8	11.9	5.00	0.042	VI	16.9(27.2)
DMG	33.9500	118.6320	08/31/1930	04036.0	0.0	5.20	0.049	VI	17.1(27.6)
MGI	34.1000	118.1000	07/11/1855	415 0.0	0.0	6.30	0.116	VII	17.2(27.8)
MGI	34.0000	118.0000	12/25/1903	1745 0.0	0.0	5.00	0.035	V	19.5(31.4)
DMG	33.7500	118.0830	03/11/1933	2 9 0.0	0.0	5.00	0.035	V	19.5(31.4)

DMG	33.7500	118.0830	03/11/1933	323 0.0	0.0	5.00	0.035	V	19.5(31.4)
DMG	33.7500	118.0830	03/11/1933	910 0.0	0.0	5.10	0.038	V	19.5(31.4)
DMG	33.7500	118.0830	03/13/1933	131828.0	0.0	5.30	0.045	VI	19.5(31.4)
DMG	33.7500	118.0830	03/11/1933	230 0.0	0.0	5.10	0.038	V	19.5(31.4)
PAS	33.9440	118.6810	01/01/1979	231438.9	11.3	5.00	0.034	V	19.9(32.1)
GSP	34.2310	118.4750	03/20/1994	212012.3	13.0	5.30	0.039	V	21.6(34.7)
GSP	34.2130	118.5370	01/17/1994	123055.4	18.0	6.70	0.113	VII	22.1(35.5)
DMG	33.7000	118.0670	03/11/1933	51022.0	0.0	5.10	0.031	V	22.6(36.4)
DMG	33.7000	118.0670	03/11/1933	85457.0	0.0	5.10	0.031	V	22.6(36.4)
DMG	33.6830	118.0500	03/11/1933	658 3.0	0.0	5.50	0.039	V	24.1(38.8)
DMG	34.3080	118.4540	02/09/1971	144346.7	6.2	5.20	0.027	V	26.2(42.2)
GSB	34.3010	118.5650	01/17/1994	204602.4	9.0	5.20	0.025	V	28.1(45.3)
GSP	34.3050	118.5790	01/29/1994	112036.0	1.0	5.10	0.022	IV	28.8(46.3)
DMG	33.6170	118.0170	03/14/1933	19 150.0	0.0	5.10	0.022	IV	28.8(46.4)
DMG	34.3000	118.6000	04/04/1893	1940 0.0	0.0	6.00	0.045	VI	29.1(46.8)
GSP	34.2620	118.0020	06/28/1991	144354.5	11.0	5.40	0.028	V	29.1(46.9)
DMG	34.2000	117.9000	08/28/1889	215 0.0	0.0	5.50	0.028	V	30.5(49.1)
DMG	33.6170	117.9670	03/11/1933	154 7.8	0.0	6.30	0.053	VI	30.7(49.4)
DMG	33.5750	117.9830	03/11/1933	518 4.0	0.0	5.20	0.020	IV	32.3(52.0)
DMG	34.4110	118.4010	02/09/1971	14 244.0	8.0	5.80	0.033	V	32.6(52.5)
DMG	34.4110	118.4010	02/09/1971	141028.0	8.0	5.30	0.022	IV	32.6(52.5)
DMG	34.4110	118.4010	02/09/1971	14 1 8.0	8.0	5.80	0.033	V	32.6(52.5)
DMG	34.4110	118.4010	02/09/1971	14 041.8	8.4	6.40	0.053	VI	32.6(52.5)
GSP	34.3260	118.6980	01/17/1994	233330.7	9.0	5.60	0.027	V	33.8(54.3)
GSP	34.3780	118.6180	01/19/1994	211144.9	11.0	5.10	0.017	IV	34.2(55.1)
GSP	34.3690	118.6720	04/26/1997	103730.7	16.0	5.10	0.017	IV	35.3(56.8)
GSP	34.3770	118.6980	01/18/1994	004308.9	11.0	5.20	0.017	IV	36.6(58.9)
GSP	34.3940	118.6690	06/26/1995	084028.9	13.0	5.00	0.015	IV	36.7(59.0)
GSB	34.3790	118.7110	01/19/1994	210928.6	14.0	5.50	0.021	IV	37.1(59.7)
DMG	34.0000	119.0000	09/24/1827	4 0 0.0	0.0	7.00	0.067	VI	38.4(61.8)
MGI	34.0000	119.0000	12/14/1912	0 0 0.0	0.0	5.70	0.024	V	38.4(61.8)
GSP	34.1400	117.7000	02/28/1990	234336.6	5.0	5.20	0.016	IV	38.7(62.3)
DMG	34.5190	118.1980	08/23/1952	10 9 7.1	13.1	5.00	0.013	III	40.6(65.3)
DMG	34.0650	119.0350	02/21/1973	144557.3	8.0	5.90	0.026	V	41.1(66.1)
MGI	33.8000	117.6000	04/22/1918	2115 0.0	0.0	5.00	0.012	III	43.1(69.4)
MGI	34.0000	117.5000	12/16/1858	10 0 0.0	0.0	7.00	0.049	VI	47.9(77.0)

EARTHQUAKE SEARCH RESULTS

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]

```
PAS |33.6710|119.1110|09/04/1981|155050.3|  5.0| 5.30| 0.013 | III| 48.4( 77.8)
DMG |34.3000|117.6000|07/30/1894| 512 0.0|  0.0| 6.00| 0.022 | IV | 48.7( 78.3)
DMG |34.3700|117.6500|12/08/1812|15 0 0.0|  0.0| 7.00| 0.048 | VI | 49.0( 78.8)
DMG |33.6990|117.5110|05/31/1938| 83455.4| 10.0| 5.50| 0.014 | IV | 50.0( 80.5)
DMG |34.2700|117.5400|09/12/1970|143053.0|  8.0| 5.40| 0.013 | III| 50.7( 81.6)
DMG |34.3000|117.5000|07/22/1899|2032 0.0|  0.0| 6.50| 0.028 |  V | 53.7( 86.4)
DMG |33.7000|117.4000|05/15/1910|1547 0.0|  0.0| 6.00| 0.018 | IV | 56.1( 90.2)
DMG |33.7000|117.4000|04/11/1910| 757 0.0|  0.0| 5.00| 0.008 | III| 56.1( 90.2)
DMG |33.7000|117.4000|05/13/1910| 620 0.0|  0.0| 5.00| 0.008 | III| 56.1( 90.2)
DMG |34.2000|117.4000|07/22/1899| 046 0.0|  0.0| 5.50| 0.012 | III| 56.3( 90.6)

******************************************************************************
-END OF SEARCH-   63 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 4.5 MILES (7.2 km) AWAY.

LARGEST EARTHQUAKE MAGNITUDE FOUND IN THE SEARCH RADIUS: 7.0

LARGEST EARTHQUAKE SITE ACCELERATION FROM THIS SEARCH: 0.222 g

COEFFICIENTS FOR GUTENBERG & RICHTER RECURRENCE RELATION:
  a-value=  1.226
  b-value=  0.395
  beta-value=  0.909


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TABLE OF MAGNITUDES AND EXCEEDANCES:
------------------------------------
```

Earthquake Magnitude	Number of Times Exceeded	Cumulative No. / Year
4.0	63	0.31343
4.5	63	0.31343
5.0	63	0.31343
5.5	22	0.10945
6.0	11	0.05473
6.5	5	0.02488
7.0	3	0.01493

KOURY
PASSION INTEGRITY QUALITY

THE KOURY DIFFERENCE

We are a key member of the construction team while safeguarding the public. We improve operational logistics and provide superior quality control through the continuing development of our engineering staff and technical expertise, utilization of classroom training and field supervisors, thus defining the industry standard.

We increase market share and become the largest through superior management practices and planning.

MAIN OFFICE

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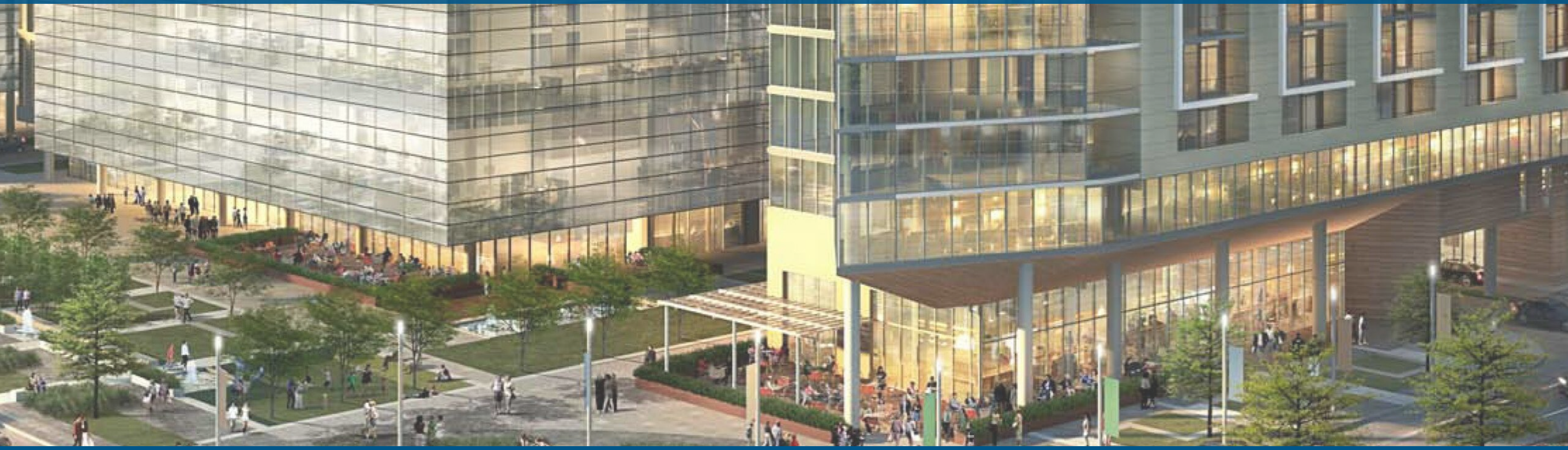
17800 S.MAIN STREET
SUITE 302
GARDENA, CA 90248
PHONE 310-851-8685
FAX 310-851-8692



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