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GEOTECHNICAL AND GEOLOGICAL ENGINEERING INVESTIGATION REPORT

NEW CHILD DEVELOPMENT CENTER
SOUTHWEST CORNER OF S 10th AVE. AND W 104th ST.
INGLEWOOD, CALIFORNIA 90303

PREPARED FOR:
INGLEWOOD UNIFIED SCHOOL DISTRICT
401 SOUTH INGLEWOOD AVENUE
INGLEWOOD, CA 90301

PREPARED BY:
KOURY ENGINEERING & TESTING, INC.
5711 SCHAEFER AVENUE
CHINO, CALIFORNIA 91710

PROJECT NO. 24-2369

JUNE 6, 2024

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June 6, 2024
Project No. 24-2369

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

Attention: Ms. Stephanie Pulcifer, Design & Construction Manager
Cordoba Corporation

SUBJECT: Geotechnical and Geological Engineering Investigation
New Child Development Center
Southwest Corner of S 10th Ave. and W 104th St.
Inglewood, CA 90303

1. INTRODUCTION

This report presents the results of a Geotechnical and Geological Engineering Investigation performed by Koury Engineering & Testing, Inc. (Koury) for the proposed new Child Development Center to be located northeast of the Morningside High School campus, specifically at the southwest corner of South 10th Avenue and West 104th Street, City of 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 classroom buildings, multipurpose building, administration building, 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. 24-2369, dated March 26, 2024.

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 proposed New Child Development Center will occupy the site of Woodworth (Clyde) Elementary School, which is in the process of moving. The site, which is located about 220 feet west of Crenshaw Boulevard, is bounded by W 104th Street on the north, S 10th Avenue on the east, Monroe Middle School on the south, and Morningside High School on the west. The main access to the site will be from S 10th Avenue via a looped access road. See Figures A-1 and A-2, Appendix A for the Site Vicinity Map and Boring Location Map. The project site is currently occupied with modular buildings, a concrete building, grass playground areas, and associated parking and driveways. A small parking lot, and few small shade structures exist adjacent to S 10th Avenue.

The north end of the site is roughly at elevation 126 feet (NAVD88). The site slopes gently to the south, and the ground surface lies at elevations between about 126 and 116 feet (NAVD88). Drainage of the site is generally by sheet flow toward the south, to low areas of the site, and to local storm drains and to the streets.

3. PROPOSED IMPROVEMENTS

Based on the Schematic Site Plan received (DSK Architects, 2024), the proposed development will include a new playground within the northern portion of the site, multiple classroom buildings south of the playground, a multipurpose building, an administration building, and a parking lot at the south end of the site. Based on review of the architectural building plans received (Silver Creek Modular Inc., 2023) we understand the proposed building structures will consist of modular buildings with raised floors above crawl spaces, supported on perimeter wall footings and interior pad footings. Other site improvements will include new hardscaping and landscaping, playgrounds, a trash enclosure, underground utilities, and low-lying retaining walls. There will be paved walkways and shallow trench work associated with the construction.

Architectural and structural design details (Coversheet-Building Data; Silver Creek Modular Inc., 2023), for the new modular building indicates the proposed footing schedules were based on maximum soil bearing pressures of 1000 psf and 1500 psf for wood and concrete footings, respectively.

4. FIELD EXPLORATION

The field exploration program consisted of drilling seven soil test borings (B-1 through B-7) and performing two percolation tests (P-1 & P-2). Borings B-1 through B-6 were drilled on May 6, 2024, utilizing a truck mounted drill rig equipped with 9-inch diameter hollow-stem augers, owned and operated by OneWay Drilling. Boring B-7 was drilled by Koury's field geologist on May 7, 2024, utilizing a 3-inch diameter hand auger. The borings were drilled to depths ranging from about 20 to 46½ feet below the existing ground surface.

The locations of the borings and percolation tests are shown on the Boring Location Map, Figure A-2, presented in Appendix A. The drill rig was equipped with a 140-lbs automatic hammer to drive the samplers a maximum of 18 inches into the soils (Borings B-1 through B-6), while for the hand auger boring (B-7), the California ring sampler was driven using a 35 lb hammer, falling 15 inches.

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 percolation testing procedure is described in Section 16 of this report.

5. LABORATORY TESTING

Laboratory tests, including moisture content, dry unit weight, laboratory estimation of fine contents, 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. Sulfate, chloride, 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 from about 2 to 5 feet at the boring locations. Deeper fill may be encountered at utility locations, or at other locations between and beyond the borings. At the boring locations, in landscape areas, the fill was found to be overlain by topsoil with a mantle of grass, and in other areas, a pavement consisting of about 2 to 4 inches of asphalt concrete over approximately 4 to 12 inches of aggregate base was noted.

The fill materials encountered in the borings consist generally of firm to stiff sandy lean clay. The fill encountered was generally moist at the time of drilling.

Underlying the fill, alluvial soil consisting of sandy lean clay, clayey sand, silty sand and poorly graded sand with silt, was noted to the maximum depth explored. The alluvium in the upper 5 to 9 feet of the subsurface consists predominantly of sandy lean clay, varying in thickness from 3 to 5 feet in the seven soil boring drilled. Below the upper lean clay layer, soils were generally coarser grained, consisting of clayey sand, silty sand, and poorly graded sand with silt to the maximum depth explored (46½ feet).

The lean clay soils within the alluvium are generally medium stiff to very stiff, and the sands are medium dense to dense within the upper 20 feet and with a few exceptions dense to very dense below 20 feet. The alluvial soils are generally moist. The moisture contents of fine-grained soils (mostly clay) range from about 12 to 16 percent with an average of about 13½ percent. The moisture contents of the clayey sand and silty sand soils range from about 6 to 16 percent with an average of about 11½ percent. The poorly graded sand (SP and SP-SM) generally have moisture contents ranging from about 2 to 9½ percent with an average of about 5½ percent (not presented on the boring logs).

Based on laboratory estimates, the boring samples indicated that fines contents for the clayey sand and silty sand generally range from about 13 to 49 percent with an average of about 30 percent. The fines contents of the clay and silt soils (mostly clay) vary from about 50 to 59 percent with an average of about 52 percent. The dry unit weights of the clay tested range from about 111 to 125 pcf with an average of about 120 pcf. The dry unit weights of silty sand and clayey sand range from about 106 to 117 pcf with an average of about 110½ pcf.

The consolidation tests did not indicate significant collapse upon addition of water (0.3% max). The two consolidation tests did not show signs of swelling 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 has potential for soil heave.

The direct shear test on a sandy clay sample indicated peak and ultimate friction angles of about 24 and 22 degrees, respectively, with corresponding apparent cohesion of 468 and 277 psf.

Pocket penetrometer test results indicate unconfined compression strengths of the tested soils ranging between about 2.2 and 4.5 tsf with an average of about 3.8 tsf. An expansion index test performed on a sandy clay sample indicated a value on the order of 42, which falls in the range of low to medium.

With a few exceptions, for the upper 16 feet of the subsurface profile, the standard penetration test blow counts and equivalent blow counts from the modified California sampler indicate blow counts for the sand ranging from about 15 to 34 with an average of about 22. These blow counts generally indicate medium dense sand. With one exception, for the same profile depth, the blow count for the clay range from about 7 to 33 with an average of approximately 19, which generally indicates the presence of clay with consistency ranging from firm to very stiff, with a very stiff average.

At a depth of 20 feet or greater, with a few exceptions, the sampling blow counts generally exceed 30, which confirmed the presence of dense to very dense soils below that depth.

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 portions of the site subject to the proposed improvements lie at approximately elevations 116 to 126 feet (NAVD88). Groundwater was not encountered in the borings drilled for this study and during prior subsurface explorations for the adjacent sites (Koury, 2022 & 2020). 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-4 for the Historic High Groundwater Map).

Environmental groundwater monitoring for a site located about 3800 feet to the northwest indicated a historic high groundwater elevation of about 22 feet. This groundwater elevation corresponds roughly to depths on the order of 92 to 104 feet below ground surface at the proposed Child Development Center site. Other than localized perched water that could be encountered at shallow depth below the ground surface, no groundwater is anticipated during the proposed construction.

8. SITE GEOLOGY

The site is located within the south portion of Los Angeles physiographic basin. The south portion of Los Angeles physiographic basin is part of the Peninsular Ranges Geomorphic Province. The Peninsular Ranges are bounded on north by the Transverse Ranges and extend 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 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 Geologic Map of the Long Beach Quadrangle indicates that the subsurface conditions at the site consist of young alluvial fan and valley deposits consisting of sand, silt and clay (see Figure

A-3 for the Regional Geologic Map). The subsurface soil profile encountered in the borings consists of fill underlain by alluvial deposits which is consistent with the regional geology.

9. OIL WELL

The State of California Department of Conservation, Division of Oil, Gas and Geothermal Resources indicates that the site 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 46½ feet. No hazardous materials associated with oil fields are anticipated in the subsurface at the child development center site.

10. SEISMIC CONSIDERATIONS

10.1. General

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

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

The project site is not located within a currently designated State of California Earthquake Fault Zone for surface fault rupture. Therefore, a project site-specific fault hazard evaluation in accordance with the A-P Earthquake Fault Zoning Act is not required per State Regulations. According to CGS publications, the nearest fault zone to the project site with mappable surface projections is the onshore segment of the Potrero Fault Zone located approximately 600 feet to the east. Based on CGS personnel that we have contacted in the past, there is no new fault studies that have been performed in the immediate vicinity of the site. However, fault studies have been performed for some of the developments north of the site.

We reviewed two studies performed by GeoSoils Consultants, one study by CFS Geotechnical Consultants, one study by the J. Byer Group, one study by Geo-Systems, and an overall assessment study performed by Geosyntec in 2019. Geotechnical reports by GeoSoils Consultants, Inc. included a review of multiple independent fault studies located to the north of the subject site. These studies were focused along and to the west of the Potrero Fault branch of the Newport-Inglewood Fault. The studies included literature research, trench excavation, logging, and preparation of graphic illustrations. The seismic trenching within these areas did not indicate fault traces outside of the Potrero Fault Zone. One of these studies was conducted approximately 1000 feet north of the subject site near the western edge of the mapped Potrero Fault and through the inferred Townsite Fault. The study concluded that this area was outside any area of previous ground displacement and the primary fault in the immediate area was located east of the study.

The study performed by Geo-Systems did not encounter the Townsite Fault. However, Geo-Systems inferred that the Townsite Fault should be located about 400 feet east of their study. The Byer Group did not observe faulting in their study in 1998. Geosyntec reviewed 42 separate fault investigations north of the site and projected the inferred Townsite Fault as indicated on figures 3 and 4 of their report (figures attached). The same fault projection was transferred to Koury Figure A-5A. Based on the latest information gathered, the Townsite Fault does not project through the school site and there is no other presently known active fault projecting through the proposed Child Care Development Center site. The current available geologic reports do not support evidence of faulting with potential surface rupture projecting through or towards the site along the Townsite Fault. After review of the above referenced reports, due to the absence of faulting evidence in the exploratory trench studies reviewed, the location of these studies along the general trend of the

Townsite fault line near the subject site, and the lack of recent faulting evidence in the near surface soils, the potential for future surface displacement at the site is considered low. There is no current trenching data that support the existence of the queried fault located immediately north of the proposed Child Care Development Center site as shown on the Geologic Map of the Long Beach 30'X60' Quadrangle (2003).

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 630 feet northeast of the site. The Alquist-Priolo Palos Verdes Connected Fault is located approximately 8.4 miles south of the site and the Whittier-Elsinore Fault Zone is located about 16½ miles east of the site. The Santa Monica Connected Fault Zone is located about 7½ miles northwest of the site and the Puente Hills (LA Segment) Fault is located about 5 miles to the northeast (see Figure A-6, Appendix A).

Based on the information available at this time, according to CGS, there is a potential for an Mw7.5 earthquake on the Newport Inglewood Fault Zone, a Mw7.7 earthquake may occur on the Palos Verdes Fault, a Mw7.8 earthquake may occur on the Whittier-Elsinore connected Fault and a Mw7.1 earthquake may occur on the Puente Hills (LA) Fault segment. 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. The potential for fault rupture at the site is considered low to moderate.

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-5 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-5 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 Mw6.36 from a seismic-hazard deaggregation using the USGS Unified Hazard Tool. However, due to the proximity of the Newport Inglewood Fault, we utilized an earthquake magnitude of 7.5. Our analysis also utilized a site acceleration of 0.98g (PGA_M) obtained from ASCE 7-16 Seismic Design Ground Motion Analysis. The seismic settlement calculations were performed for two relatively deep borings (B-1 & B-2) using the SPT and equivalent California sampler blow count data. The California sampler blow counts were multiplied by a factor of 0.65 to obtain the equivalent SPT blow counts. The SPT tests were performed with an automatic hammer and unlined SPT samplers with an inner diameter of 1.5 inches. We used a hammer energy factor of 1.25 (Ce=1.25), a borehole diameter factor of 1.0 (Cb=1), and a sampling method factor of 1.1 (Cs=1.1) in our analyses.

Using the LiquifyPro software, we calculated total seismic dry sand settlements on the order of 1 to 1½ inches (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 ¾ 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 116 to 126 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 young 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 significant 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 previously placed fill (undocumented) in the upper 2 to 5 feet, soil disturbance that will occur during the demolition and removal of existing building structural foundations, parking lots and other facilities, and the presence of clay soils at shallow depth with expansion potential.

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

We understand the proposed building structures will be modular buildings with raised floors supported by perimeter wall and interior pad footings.

The thickness of undocumented fill encountered at the boring locations are on the order of 2 to 5 feet. Deeper fill is expected at utility locations and below the existing facilities. 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 pavements, building or other structure foundations, 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.

For building pad areas, we recommend complete overexcavation of existing fill and the subgrade soils to minimum depths of 3 feet below existing grades and 1½ feet below footings, whichever is deeper.

Where feasible, the overexcavation should extend laterally at least 5 feet beyond the building and structure perimeter.

Following subgrade approval by the Geotechnical Engineer, the bottom of the removal excavations 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 above optimum and recompacted to 92% relative compaction for sand as determined by ASTM D1557. However, if the subgrade is stiff and consists of undisturbed clay alluvium and the moisture content is at least 125 percent of optimum, the scarification should not be performed unless indicated otherwise, and measures should be taken to prevent subgrade disturbance. The subgrade should be proofrolled with heavy construction equipment to determine its firmness, as needed.

All fill placed within the building pad should be compacted to at least 90% relative compaction at a moisture content of at least 125 percent of optimum for sandy clay soils and, at least 95% relative compaction at a moisture content within 2½ percent of optimum for sand/granular soils unless approved otherwise by the Geotechnical Consultant at the time of construction. 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 area, 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 to 125% of optimum for clay and above optimum for sand, and recompacted to 90% relative compaction.

Except for pavement areas, all fill outside the structure areas should be compacted to at least 90% relative compaction at moisture content above 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 School 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

Except as otherwise indicated, the onsite shallow clayey sand and sandy lean clay with low expansion potential are deemed suitable to be re-used as engineered fill, provided they are free from deleterious material, and properly processed and moisture conditioned prior to fill placement. If relatively expansive material (EI > 45) are encountered during grading, these materials should not be placed below foundations but may be used below asphalt pavement and in landscape areas unless indicated otherwise by the Geotechnical Engineer at the time of construction. Some non-expansive import 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.

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 6 to 13 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 prior use of the site and removal of topsoil.

13.3.2. Import

Import materials if required, 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 following recommendations are provided, if slab on grade foundations are proposed for any of the new building structures:

The grading recommendations for the new building pad are provided in Section 13.2.1. The building floor slab-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 2022 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 (Figures A-8 and A-8a, for Site Class D and Class C, respectively).

Table 1 – Seismic Coefficients and Factors

Site Class (CBC 2022 – 1613A.2.2)	D	C
Seismic Design Category based on Occupancy Category III (CBC 2022-1604A.5 &1613A.2.5)	D	D
Mapped Acceleration Parameter for Short Period (0.2 Sec.), S_S	1.894	1.894
Mapped Acceleration Parameter for 1.0 Second, S_1	0.667	0.667
Adjusted Maximum Spectral Response Parameter for Short Period (0.2 Second), S_{MS}	1.894	2.273
Adjusted Maximum Spectral Response Parameter for 1.0 Second Period, S_{M1}	*1.134	0.934
Design Spectral Response Acceleration Parameter, S_{DS}	1.263	1.515
Design Spectral Response Acceleration Parameter, S_{D1}	*0.756	0.622
Peak Ground Acceleration (PGA_M)	0.90	0.962
Period (T_0/T_S)	$^+$.120/.599	$^+$.082/.411
Earthquake Design Magnitude	7.5	7.5

Project Site Coordinates: Longitude: W -118.327645º Latitude: N 33.941155º (WGS84)
$^+$Based on Fv of 1.71 for period calculation. *See Section 11.4.8 Supplement 3, ASCE 7-16 for exception to site-specific ground motion study.

No site-specific ground motion study is required per ASCE Standard 7-16, Supplement 3, Section 11.4.8 Item 1, where the value of the parameter S_{M1} determined by Equation 11.4-2 is increased by 50% for all applications of S_{M1} in this Standard. The resulting value of the parameter S_{D1} determined by Equation 11.4-4 should be used for all applications of S_{D1}.

The site class can be determined in accordance with ASCE 7 Chapter 20 using shear wave velocity, SPT blow count or undrained shear strength. The SPT blow counts presented on the boring logs indicate the site Class can be either C or D depending upon the location on site (borderline classification). We are, therefore, presenting the seismic coefficients and factors for both site classes. In our opinion, the site class should be selected based on the discretion of the Structural Engineer; however, our suggestion is to use the most conservative of ground motion design parameters.

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 30 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.

Architectural/structural design, foundation plans and details for the new modular buildings indicate the proposed footing schedule (Silver Creek Modular Inc., 2023; Sheet numbers 1.01, 1.11, 1.50, 2.01, and 2.11) utilizes wall footings (end wall and side wall) of width varying from 12 to 20 inches, and square pad footings for interior floor support having widths ranging from 3 to 5 feet. The proposed footings have embedment depths of 12 inches below the adjacent pad grade. The design plans (Silver Creek Modular Inc., 2023; Cover sheet), indicate that the proposed footing schedules are based on maximum soil bearing pressure of 1500 psf for concrete footings. In our opinion, the assumed allowable bearing pressures can be used for the proposed footing dimensions; however, the embedment depth should be at least 18 inches below the lowest adjacent grade and the footings should be supported on at least 18 inches of engineered fill.

All other building structures founded on conventional slab on grade foundation, 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 18 inches 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.

In general, footings should have a minimum width of 2 feet for isolated footings and 18 inches for continuous footings. The bottom of non-portable 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.

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 0.3 may be used between the footings, floor slabs on grade, and the supporting soils comprised of engineered fill. Where visqueen is used below floor slabs or footings, 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 (allowable). A one-third increase in the passive value may be used for wind or seismic loads. The frictional resistance and the passive resistance of the soils may be combined provided that the passive resistance is reduced by one third. We recommend that the 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 ¾ 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 an equivalent 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.

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 flatwork 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 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 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 except where flatter gradients are required for ADA compliance. 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 was used for preliminary 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, 6 and 6.5. 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
6.5	4.5	12.0

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 above optimum moisture content for sand and to at least 120 percent of optimum moisture content for clay (less moisture than for non-pavement area) 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 upper 12 inches of the clay subgrade 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. Lateral Resistance for Pole and Post Foundations

As a typical foundation, cast-in-place drilled piers (shafts) are usually used to support the axial and lateral loads of this kind of structure. Lateral loads on the foundation shaft for the proposed poles may be resisted by the passive resistance utilized by the surrounding soils. When the ground surface is level, the passive resistance may be assumed to be equal to the pressure developed by a fluid with a density of 200 pcf, with the zero point 1.5 feet deep below the ground surface and to a maximum value 2,000 psf. These values apply to the design of the poles when they are adversely affected by 0.5 inch of lateral movement at ground level. These lateral resistance values should not be multiplied by 2 as addressed in section 1806A.3.4 of CBC 2022.

If the embedment depth is obtained in accordance to the Pole Formula provided in CBC 2022 (Equation 18A-1), the allowable lateral soil-bearing pressure based on a depth of one-third of the embedment depth (S1 in Section 1807A.3.2.1 of CBC 2022) may be calculated according to the aforementioned lateral resistance values. The S1 may be obtained from the equation; S1= 85 (d-1.5), where d is the embedment depth. If movement of 0.5 inch or greater at the ground surface is allowed, the equivalent fluid pressure may be multiplied by 2.

13.14. Lunch Shelter Support

At the time of preparation of this report, the location of the potential lunch shelter was unknown. The lunch shelter structure, if needed, may be supported on cast-in-drilled-hole (CIDH) concrete pile (caisson foundations) or on pad footings. We have assumed the piles will have a diameter of 18 inches or greater. If the spacing of the posts/piles exceeds 8 pile diameters, no reduction factor for lateral load resistance efficiency will be required. The downward or upward capacity of cast-in-drilled-hole (CIDH) concrete pile foundations is based on the friction resistance between the pile shaft and surrounding soils. For vertical downward capacity calculations, the upper 2 feet of soil should be neglected, and a shaft friction resistance of 325 psf may be used. For uplift resistance, a skin friction of 160 psf may be used along with the weight of the pile shaft. For vertical downward resistance, the pile weight may be taken as end bearing.

14. SOIL EXPANSIVITY

The subsurface soils encountered at shallow depths within the building pad areas consist predominantly of sandy lean clay. The sandy lean clay is expected to have a low to medium expansion potential when facing seasonal cycles of saturation/desiccation. Past expansion index testing on shallow samples obtained from adjacent sites yielded values of 14, 22, and 94 (Koury, 2022 and 2020). The expansion index test performed for this study indicated a value on the order of 42. As such, the recommendations provided in this report regarding surface drainage and moisture conditioning during site grading 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. 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 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-3	1 –5	12,600	8.8	44	11

Based on the minimum resistivity results from the soil tested, some of the near-surface site soils should be considered low to moderately corrosive towards buried ferrous metals. The concentrations of soluble sulfates indicate that the potential of sulfate attack on concrete in contact with the onsite soils is “negligible” based on ACI 318 Table 4.3.1. Cement Type II may be used in the concrete. Maximum water-cement ratios are not specified for the sulfate concentrations; however, the Structural Engineer should select concrete with appropriate strength. The exposures to site class in accordance ACI 318-14, Table 19.3.1.1 are F_0, S_0, W_0, and C_1.

Further interpretation of the corrosivity test results, including the resistivity value, and providing corrosion design and construction recommendations are the purview of corrosion specialists/consultants.

16. PERCOLATION TESTING

At the time of our field exploration, the type, location, and depth of the proposed Best Management Practices (BMPs) had not been determined. Normally, percolation testing is performed about one foot below the invert of the BMPs. The drilling for the percolation holes and percolation testing were performed on sunny days. No rain had occurred for several days prior to percolation testing. The percolation testing locations were selected based on potential areas where infiltration might be considered. The depth of the holes for percolation testing was about 13½ feet below the ground surface. The test borings (B-1 and B-7) were drilled in the vicinity of Percolations P-1 and P-2, respectively. The boring logs may be used to identify the soil stratification encountered within the percolation borings (see boring logs in Appendix B).

Koury performed the tests in general conformance with the boring percolation test procedure of the County of Los Angeles as defined in the Low Impact Development BMP Design Handbook dated 9/20/14, and the Administrative Manual of the County of Los Angeles Public Works, Geotechnical & Materials Engineering Division, County Document GS200.1 dated 6/30/21. The percolation test procedure consisted of drilling to the test depth 9- and 6-inch diameter boreholes for P-1 and P-2. respectively, and placing a 2-inch layer of filter gravel at the bottom of the holes. We also placed a 3-inch diameter perforated pipe in the hole and filter rock to prevent caving.

The percolation holes were presoaked per the test method. Following presoaking, percolation testing began by filling the lower portion of the percolation hole with water and measuring the dropin water level. Based on the rate of water dissipation observed during presoaking, a 30-minute measuring interval was selected. The water column heights were generally in the range of slightly less than 1 foot at the end of the 30 minute intervals to 2½ feet at the beginning of the interval. Using fixed reference points, we measured the water level drop for the time interval selected and refilled each time once the time interval was achieved. Refilling with water was repeated several times until consistent results were noted. The stabilized rate of drop for the last 3 readings was less than 10 percent. Table 6 summarizes the results of the falling head percolation tests. Detailed test data for the percolation tests are presented in Appendix C.

Table 6– Summary of Falling Head Percolation Testing

Test Number	Depth (ft)	Short Term Infiltration *(in/hour)	Adjusted Long Term Infiltration (in/hour)
P-1	13.5	2.3	1.7
P-2	13.5	0.5	0.4

*No correction factor applied

The percolation testing data was converted to infiltration rates as presented in Appendix C. The uncorrected field test data indicated short-term infiltration rates on the order of about 1.7 to 2.3 inches per hour.

The in-situ field percolation tests performed provide short-term infiltration rates, which apply mainly to the initiation of the infiltration process due to the short time of the test (minutes to hours instead of days) and the amount of water used. Where appropriate the short-term infiltration rates should be converted to long-term infiltration rates using a reduction factor ranging from 3 to 9 depending upon the degree of infiltrate quality, maintenance access and frequency, site variability, subsurface stratigraphy variation, hydraulic gradient, and other factors. The small-scale percolation testing cannot model the complexity of the effect of interbedded layers of different soil composition, and our test results should be considered index values of infiltration rates.

We have applied a correction factor of 4.5 to determine the long-term infiltration rates presented in Table 6. The correction factor was selected per the test method to account for the boring percolation testing method, subsurface soil variations, and long-term maintenance. The correction factor calculations and assumptions are indicated on the percolation test data sheet in Appendix C. For long-term maintenance, we used a correction factor of 1 assuming excellent maintenance. For the test method and the subsurface variability, we have assumed correction factors of 2 and 1.5, respectively. The design professional may re-adjust the composite correction factor, if deemed appropriate; however, a reduction factor that is too low may affect the longevity of the BMPs.

Subsurface Soil Consideration for Infiltration: The closest boring (B-1), which corresponds to Percolation Test P-1, indicates the presence of silty sand with low fine contents within the percolation zone. However, the silty sand is underlain by additional silty sand below the percolation zone; however, this lower sand contains interbeds of sandy clay, which is not conducive to infiltration. Because the soils become denser with depth, deeper infiltration is unlikely to yield significantly greater infiltration rates.

Boring B-7 drilled in the vicinity of Percolation Test P-2 show medium dense poorly graded fine to medium grained sand with silt within the percolation zone, which explains a slightly better percolation than measured in P-1 due to the presence of poorly graded sand, but still on the low side.

Design Consideration: Based on the soil borings and the results of percolation testing, it is our opinion that the site soils are generally slightly conducive to infiltration. Based on the results of percolation testing, localized areas may allow low infiltration volume due to localized semi-pervious soil layers. The infiltration should be performed at depths of at least 14 feet and into poorly graded sand with silt material or relatively clean silty sand (less than 18 percent fines).

The site soils generally have a low expansion potential at the depth recommended for infiltration and the effects of infiltration on soil expansion should be low. There is potential for settlement of the ground surface due to hydroconsolidation, and some maintenance should be anticipated in the vicinity of the infiltration facilities. The infiltration facilities should be designed to overflow to the storm drain in the event the drainage capacity is exceeded or in case of future failure to infiltrate sufficiently.

Because the horizontal hydraulic conductivity is generally greater than the vertical hydraulic conductivity, which could affect nearby improvements, the infiltration facilities should be located at least 10 feet away from property lines and 25 feet away from any existing or proposed buildings or structures. Utility pipelines should be located well outside the infiltration facilities or special measures should be taken to prevent water from entering the bedding and shading materials placed around utilities.

No infiltration facility should be designed to infiltrate water into fill material except if coarsegrained clean uniform sand and filter gravel are used as fill. Any construction method should prevent compaction of the area where infiltration is proposed. Any processing and compaction of the onsite soils may reduce the infiltration by factors ranging between about 2 and 10 times. We recommend that the lower 18 inches of the excavations for infiltration facilities be performed using an excavator; no rubber tire equipment should be allowed at the bottom of the excavations. No disturbance to the bottom of the excavations should be allowed. If silt or clay material is encountered at the bottom of the excavations, it should be removed and replaced with coarse clean sand or filter gravel. The filter rock placed at the bottom of the excavation should be vibrated in place and ample water should be used during the vibration/compaction process. The proposed infiltration design system should be reviewed by the Geotechnical Consultant prior to construction.

17. OBSERVATION AND TESTING

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

18. CLOSURE

The findings and recommendations presented in this report were based on the results of our field and laboratory investigations, combined with professional engineering experience and judgment. The report was prepared in accordance with generally accepted engineering principles and practice. We make no other warranty, either expressed or implied. Subsurface variations between borings should be anticipated. Koury should be notified if subsurface conditions are encountered, which differ from those described in this report since 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 9/30/25
Geotechnical
STATE OF CALIFORNIA

CERTIFIED ENGINEERING GEOLOGIST
Shaofu Chen
2688
STATE OF CALIFORNIA

Jacques Roy
Jacques B. Roy P.E. G.E.
Principal Geotechnical Engineer

Shaofu Chen, C.E.G. 2688
Principal Geologist

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1. Addressee (a pdf copy via e-mail)
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APPENDICES

Appendix A: Maps and Plans

Vicinity Map – Figure A-1
Boring Location Map – Figure A-2
Regional Geologic Map – Figure A-3
Historically Highest Groundwater Map – Figure A-4
Seismic Hazard Zones Map – Figure A-5
Fault Map – Figure A-6
Flood Map – Figure A-7
Seismic Parameters – Figures A-8 & A-8a
Oil and Gas Map – Figure A-9

Appendix B: Field Exploratory Boring Logs

Borings B-1 through B-7

Appendix C: Laboratory Test Results and Calculations

Appendix D: Historical Earthquake Data

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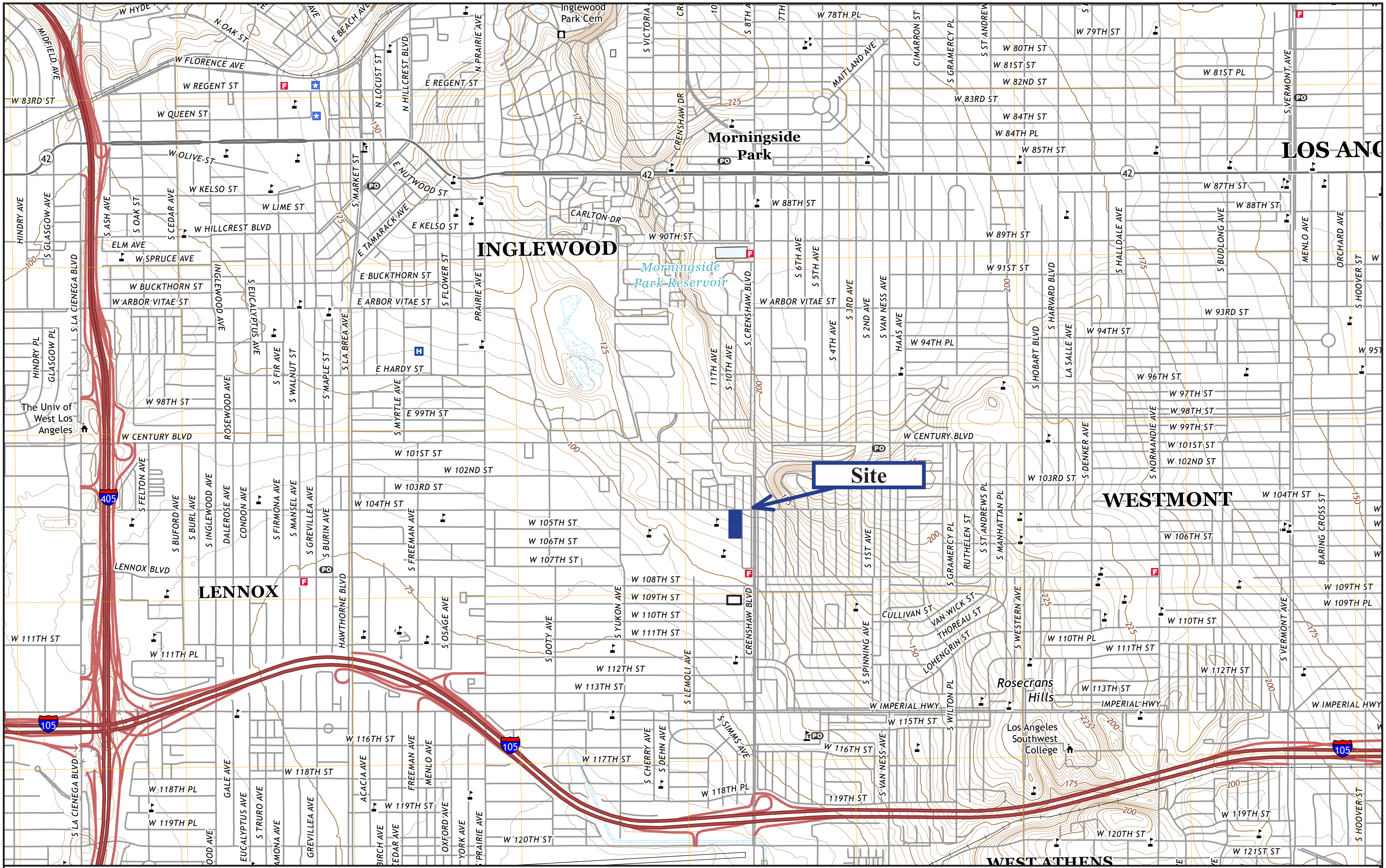
APPENDIX A

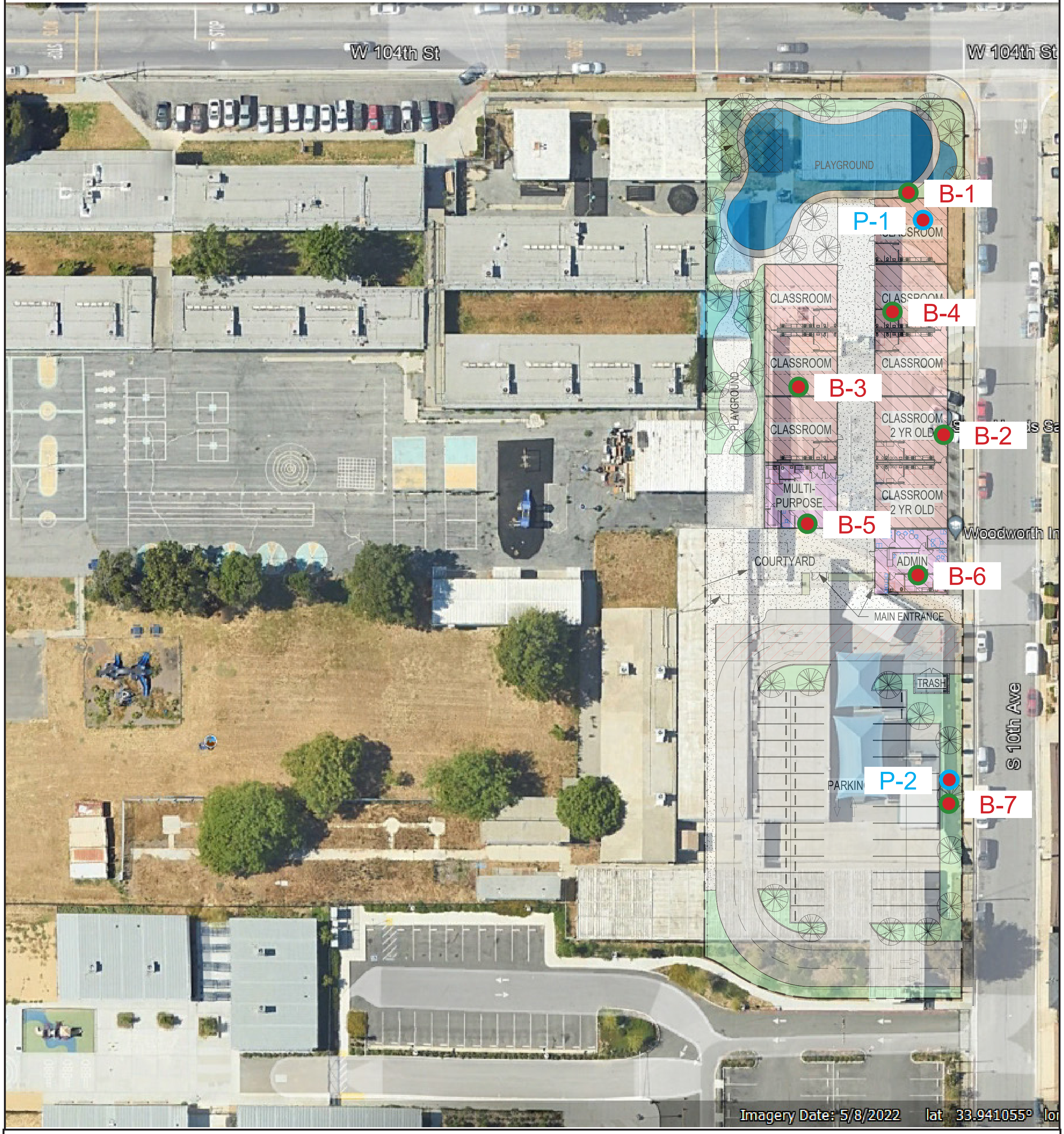
Maps and Plans



Reference: USGS Topographic Map, Inglewood Quadrangle, California-Los Angeles County, 7.5 Minute Series, 2018 - Contour Interval 5 ft. Scale 1:24,000.

KOURY
PASSION INTEGRITY QUALITY

Project Name:	Project No.:	Drawing Title:	Figure:
New Child Development Center Campus	24-2369 Date: June 2024	Vicinity Map	A-1



LEGEND

● B-7 Approximate Boring Location & Number.

● P-2 Approximate Percolation Location & Number.

N

0 50 100 ft

Reference: Google Earth Imagery Dated 10/08/21, overlayed with Plan Sheet SD-Site, drafted by DSK Architects dated 02/06/24

KOURY PASSION INTEGRITY QUALITY

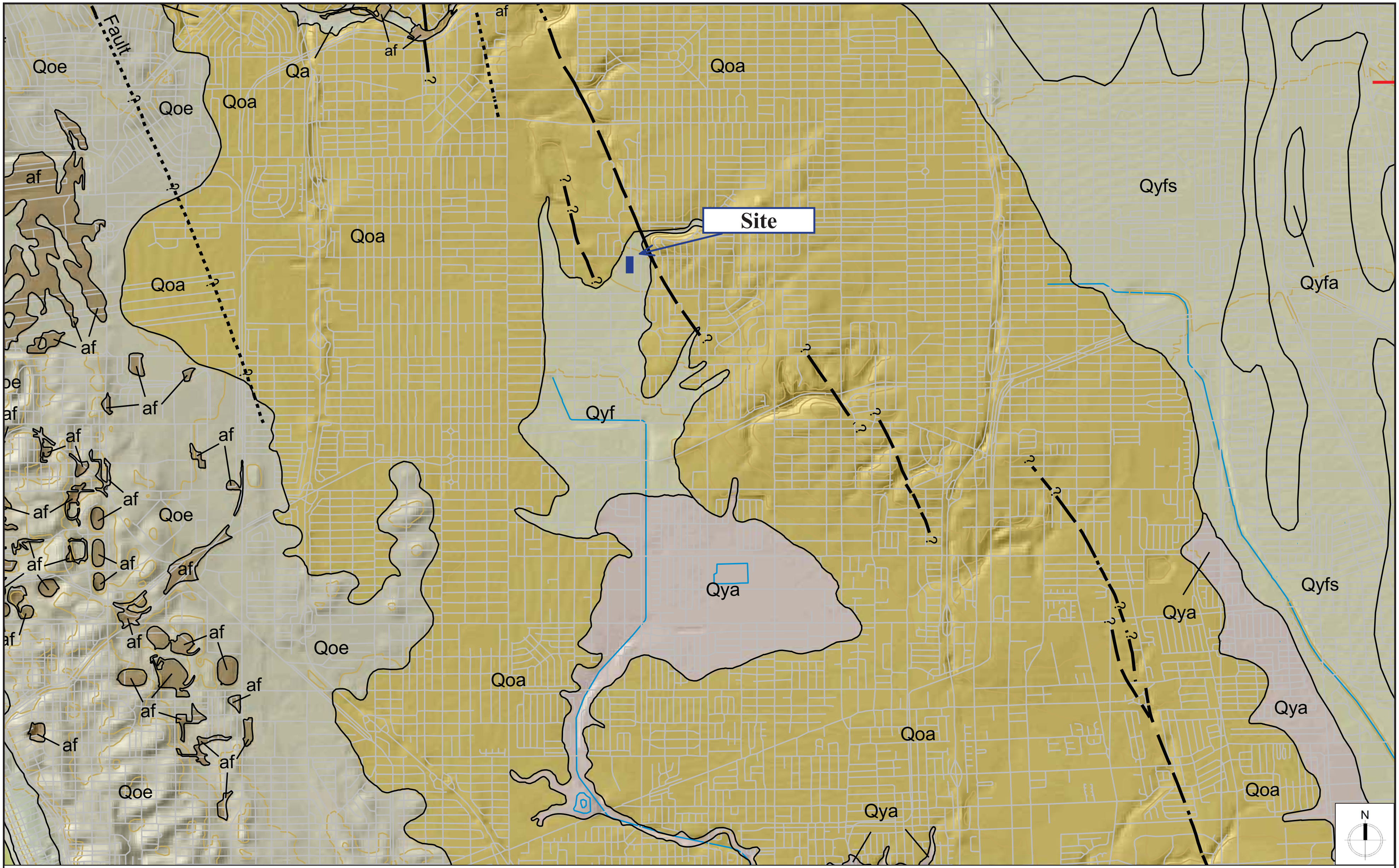
Project Name: New Child Development Center Campus

Project No.: 24-2369

Date: June 2024

Drawing Title: Boring Location Map

Figure: A-2



Legend

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

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

KOURY
PASSION INTEGRITY QUALITY

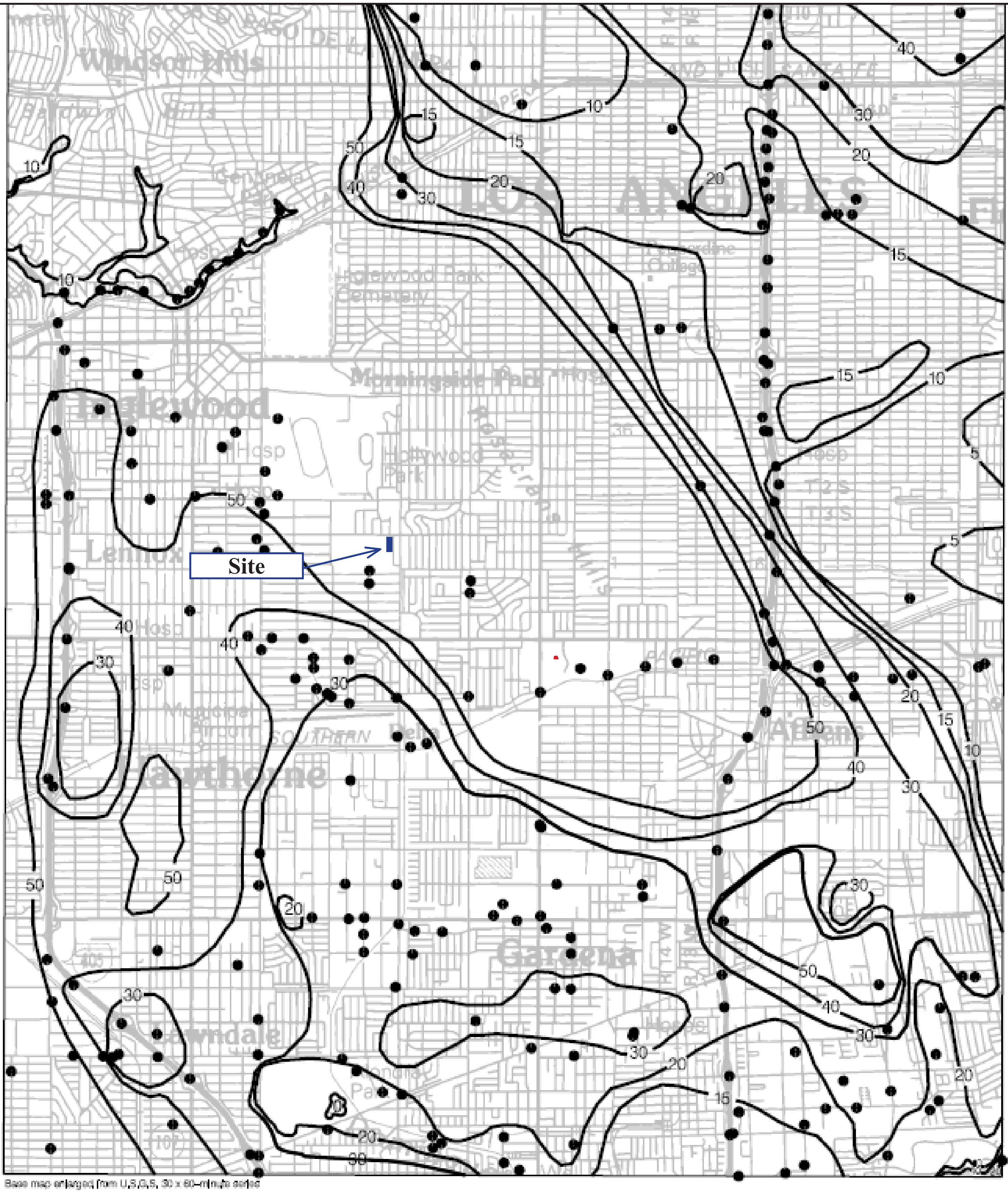
Project Name: New Child Development Center Campus

Project No.: 24-2369

Date: June 2024

Drawing Title: Regional Geologic Map

Figure: A-3



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

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

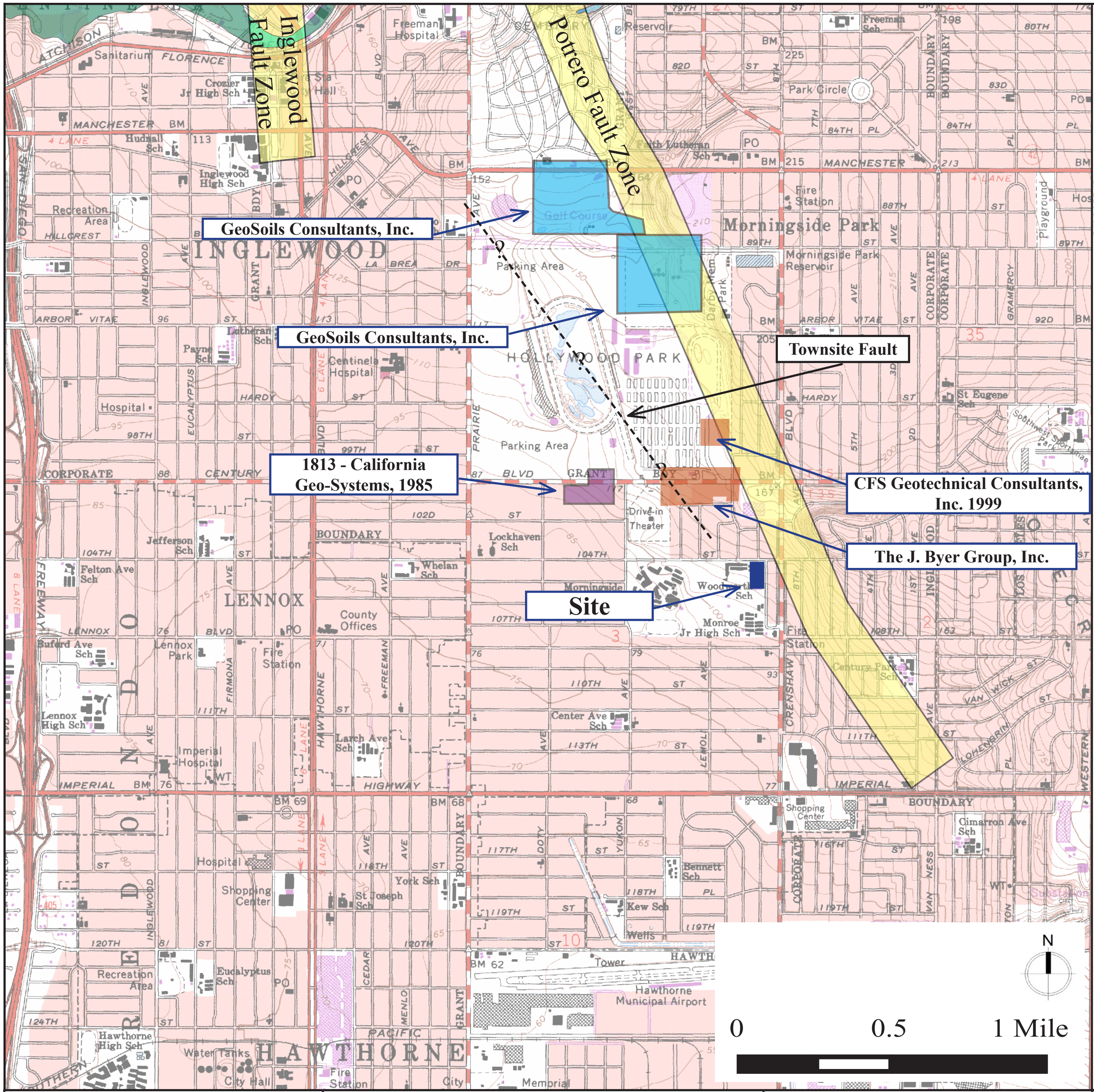
N

0 1 Mile

● Borehole Site

30 Depth to ground water in feet

KOURY PASSION INTEGRITY QUALITY	Project Name **New Child Development Center Campus**	Project No.: **24-2369** Date: **June 2024**	Drawing Title: **Historically Highest Groundwater Map**	Figure: **A-4**



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
STATE GEOLOGIST	STATE GEOLOGIST

The J. Byer Group, Inc. — Location and Firm Conducting Fault Studies

Approximate location or inferred fault line

KOURY
PASSION INTEGRITY QUALITY

Project Name: New Child Development Center Campus

Project No.: 24-2369

Date: June 2024

Drawing Title: Seismic Hazard Zones Map

Figure: A-5

🟨 Fault Zone

╲ Quaternary Fault Line

References: Earthquake Zones of Required Investigation . ArcGIS web application. (n.d.). Retrieved March 20, 2023, from https://maps.conservation.ca.gov/cgs/EQZApp/app/ & Quaternary faults (2010). City of Los Angeles Hub. (n.d.). Retrieved March 20, 2023, from https://geohub.lacity.org/datasets/quaternary-faults-2010/explore

0 400 feet

KOURY PASSION INTEGRITY QUALITY	Project Name: New Child Development Center Campus	Project No.: 24-2369 Date: June 2024	Drawing Title: Fault Map	Figure: A-6

National Flood Hazard Layer FIRMette

FEMA

Legend

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

SPECIAL FLOOD HAZARD AREAS	🟦	Without Base Flood Elevation (BFE) *Zone A, V, A99*
		With BFE or Depth *Zone AE, AO, AH, VE, AR*
	▨	Regulatory Floodway
OTHER AREAS OF FLOOD HAZARD	🟧	0.2% Annual Chance Flood Hazard, Areas of 1% annual chance flood with average depth less than one foot or with drainage areas of less than one square mile *Zone X*
	▨	Future Conditions 1% Annual Chance Flood Hazard *Zone X*
	▨	Area with Reduced Flood Risk due to Levee. See Notes. *Zone X*
	▨	Area with Flood Risk due to Levee *Zone D*
OTHER AREAS	NO SCREEN	Area of Minimal Flood Hazard *Zone X*
	▭	Effective LOMRs
	▭	Area of Undetermined Flood Hazard *Zone D*
GENERAL STRUCTURES	— – — –	Channel, Culvert, or Storm Sewer
	IIIIIII	Levee, Dike, or Floodwall
OTHER FEATURES	B 20.2 17.5	Cross Sections with 1% Annual Chance Water Surface Elevation
	8 — — —	Coastal Transect
	~~ 513 ~~~~	Base Flood Elevation Line (BFE)
	——	Limit of Study
	——	Jurisdiction Boundary
	--- ---	Coastal Transect Baseline
	— ——	Profile Baseline
	——	Hydrographic Feature
MAP PANELS	☐	Digital Data Available
	☐	No Digital Data Available
	☒	Unmapped

N

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

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

The flood hazard information is derived directly from the authoritative NFHL web services provided by FEMA. This map was exported on 5/29/2024 at 4:35 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.

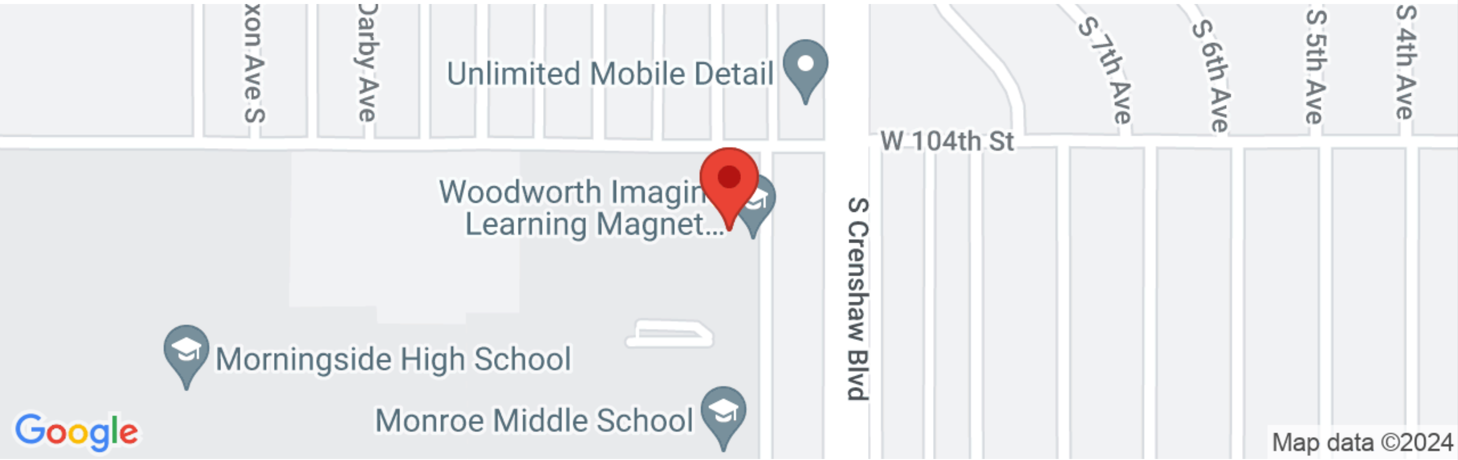
This map image is void if the one or more of the following map elements do not appear: basemap imagery, flood zone labels, legend, scale bar, map creation date, community identifiers, FIRM panel number, and FIRM effective date. Map images for unmapped and unmodernized areas cannot be used for

KOURY PASSION INTEGRITY QUALITY	**Project Name:** New Child Development Center Campus	**Project No.:** 24-2369 **Date:** June 2024	**Drawing Title:** Flood Map	**Figure:** A-7

OSHPD

Inglewood USD - New Child Development Center Campus

Latitude, Longitude: 33.941155, -118.327645



Date	5/29/2024, 1:42:09 PM
Design Code Reference Document	ASCE7-16
Risk Category	III
Site Class	D - Stiff Soil

Type	Value	Description
S_S	1.894	MCE_R ground motion. (for 0.2 second period)
S_1	0.667	MCE_R ground motion. (for 1.0s period)
S_{MS}	1.894	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

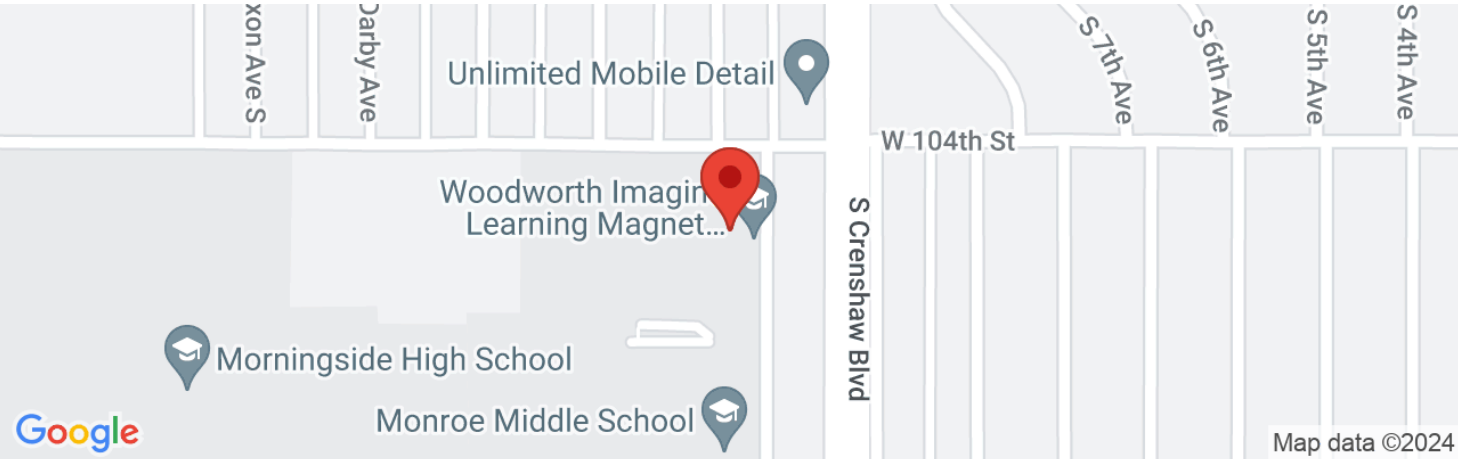
Type	Value	Description
SDC	null -See Section 11.4.8	Seismic design category
F_a	1	Site amplification factor at 0.2 second
F_v	null -See Section 11.4.8	Site amplification factor at 1.0 second
PGA	0.818	MCE_G peak ground acceleration
F_{PGA}	1.1	Site amplification factor at PGA
PGA_M	0.9	Site modified peak ground acceleration
T_L	8	Long-period transition period in seconds
SsRT	1.894	Probabilistic risk-targeted ground motion. (0.2 second)
SsUH	2.099	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)
PGA_{UH}	0.818	Uniform-hazard (2% probability of exceedance in 50 years) 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
C_V	1.479	Vertical coefficient

KOURY PASSION INTEGRITY QUALITY	Project Name: New Child Development Center Campus	Project No.: 24-2369 Date: June 2024	Drawing Title: Seismic Parameters Site Class D	Figure: A-8

OSHPD

Inglewood USD - New Child Development Center Campus

Latitude, Longitude: 33.941155, -118.327645

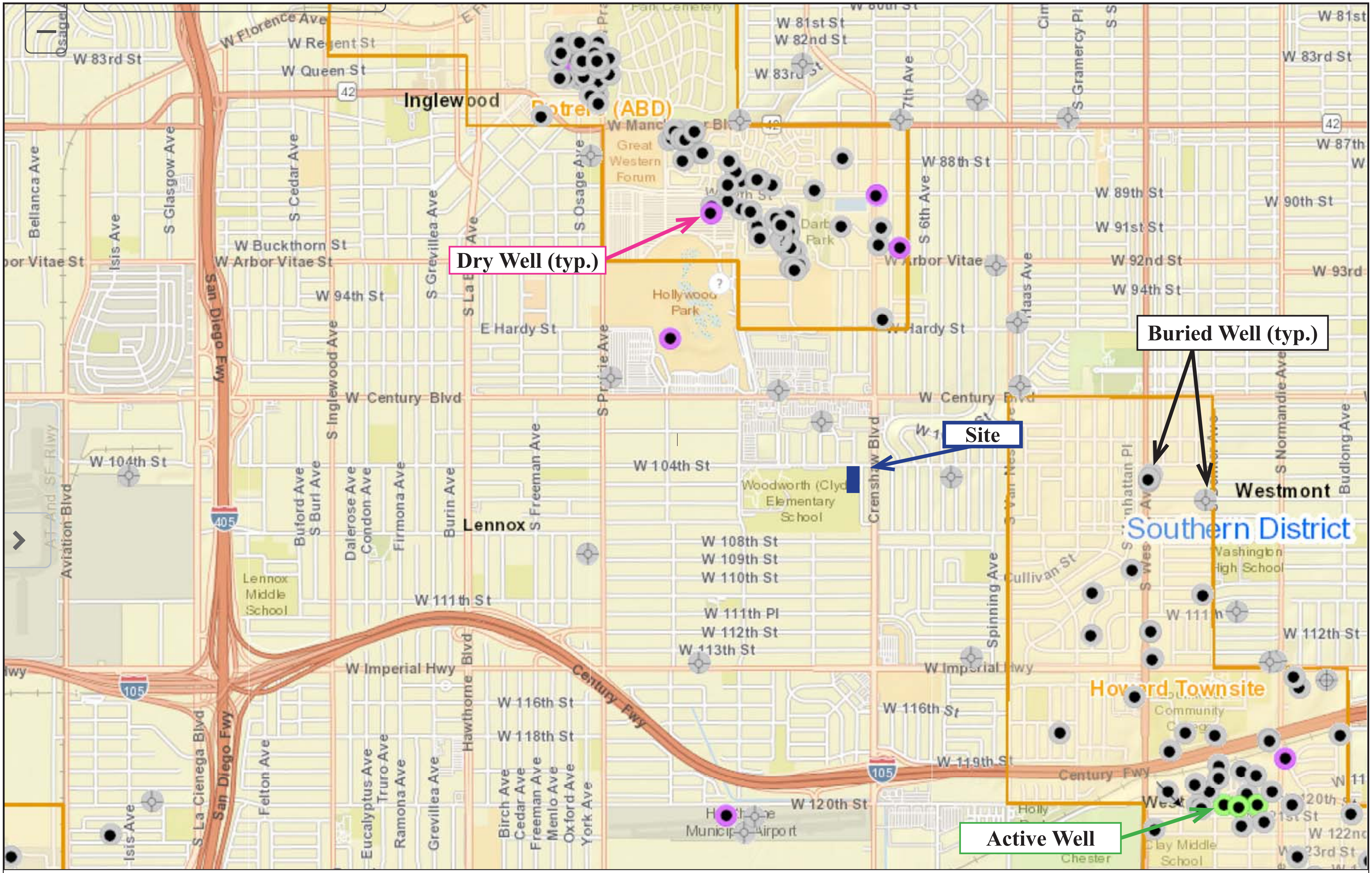


Date	5/31/2024, 5:36:00 PM
Design Code Reference Document	ASCE7-16
Risk Category	III
Site Class	C - Very Dense Soil and Soft Rock

Type	Value	Description
S_S	1.894	MCE_R ground motion. (for 0.2 second period)
S_1	0.667	MCE_R ground motion. (for 1.0s period)
S_{MS}	2.273	Site-modified spectral acceleration value
S_{M1}	0.934	Site-modified spectral acceleration value
S_{DS}	1.515	Numeric seismic design value at 0.2 second SA
S_{D1}	0.622	Numeric seismic design value at 1.0 second SA

Type	Value	Description
SDC	D	Seismic design category
F_a	1.2	Site amplification factor at 0.2 second
F_v	1.4	Site amplification factor at 1.0 second
PGA	0.818	MCE_G peak ground acceleration
F_{PGA}	1.2	Site amplification factor at PGA
PGA_M	0.982	Site modified peak ground acceleration
T_L	8	Long-period transition period in seconds
SsRT	1.894	Probabilistic risk-targeted ground motion. (0.2 second)
SsUH	2.099	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)
PGA_{UH}	0.818	Uniform-hazard (2% probability of exceedance in 50 years) 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
C_V	1.279	Vertical coefficient

KOURY PASSION INTEGRITY QUALITY	Project Name: **New Child Development Center Campus**	Project No.: **24-2369** Date: **June 2024**	Drawing Title: **Seismic Parameters Site Class C**	Figure: **A-8a**



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

0 1 Mile

N

Project Name:	Project No.: 24-2369	Drawing Title:	Figure:
New Child Development Center Campus	Date: June 2024	Oil & Gas Map	A-9

KOURY PASSION INTEGRITY QUALITY

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

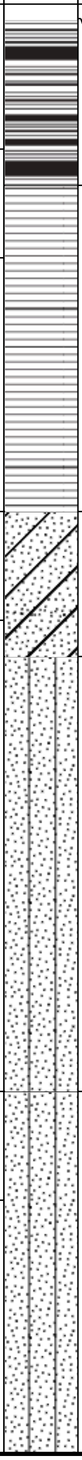
Koury Engineering

14280 Euclid Avenue, Chino, California, 91710

Phone: 909 606 6111

Boring No.: B-1

Latitude	: 33.941623	**Drill Supplier**	: OneWay Drilling	**Job Number**	: 24-2369	**Sheet**	: 1 OF 3
Longitude	: -118.327526	**Driller Company**	: OneWay Drilling	**Client**	: Inglewood Unified School District		
Elevation	: Not Surveyed	**Logged By**	: Albert Buffet	**Project**	: Child Care Development Center		
Total Depth	: 46.5 ft	**Date**	: 05/06/2024	**Location**	: 104th and 110th St, Los Angeles		



Depth (ft)	Samples Bulk	 SPT Sample	 Mod Cal Sample	Blows per 6" (SPT)	Blows per 6" (ModCal)	Soil Origin	Classification Code	Material Description	Testing % Fines	 Moisture %	 Dry Density (pcf)	 Other
0						Non-Soil	GRASS	Grass over topsoil				
0.2	X					Fill	CL	SANDY LEAN CLAY (CL) : fine grained sand, firm to stiff, moist, dark yellowish brown, pockets of very moist clay .				
2			■		7, 11, 18, (N = 18.85)							
2.5						Alluvium	CL	SANDY LEAN CLAY (CL) : very stiff, fine grained sand, low plasticity, moist, dark yellowish brown.	50	12.1	125	PP=4.5tsf
5		⧗		7, 9, 16, (N = 25)					50	13.2		PP=4.5tsf
7			■		9, 16, 21, (N = 24.05)	Alluvium	SC	CLAYEY SAND (SC) : fine to medium grained, medium dense, moist, trace fine sized gravel, dark yellowish brown, pockets of sandy clay.	30	10.7	125	
9						Alluvium	SM	SILTY SAND (SM) : fine to medium grained, dense, moist, light yellowish brown, layers of clayey sand.				
10		⧗		10, 14, 16, (N = 30)					13	6.2		
15			■		7, 15, 21, (N = 23.4)	Alluvium	SM	SILTY SAND (SM) : fine grained, medium dense to dense, moist, light yellowish brown and dark yellowish brown, layers of sandy lean clay.				

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Boring No.: B-1

Latitude : 33.941623
Longitude : -118.327526
Elevation : Not Surveyed
Total Depth : 46.5 ft

Drill Supplier : OneWay Drilling
Driller Company : OneWay Drilling
Logged By : Albert Buffet
Date : 05/06/2024

Job Number : 24-2369
Client : Inglewood Unified School District
Project : Child Care Development Center
Location : 104th and 110th St, Los Angeles

Sheet : 2 OF 3

Depth (ft)	Samples			Blows per 6" (SPT)	Blows per 6" (ModCal)	Graphic Log	Soil Origin	Classification Code	Material Description	Testing			
	Bulk	SPT Sample	Mod Cal Sample							% Fines	Moisture %	Dry Density (pcf)	Other
20		⧗		7, 15, 18, (N = 33)			Alluvium	SM	SILTY SAND (SM) : fine to medium grained, medium dense to dense, moist, dark yellowish brown.				
24							Alluvium	SC	CLAYEY SAND (SC) : fine grained, medium dense to dense, moist, dark yellowish brown, some cementation.				
25			■		16, 25, 25, (N = 32.5)								
30		⧗		7, 22, 27, (N = 49)			Alluvium	SC	CLAYEY SAND (SC) : fine grained, dense, moist, yellowish brown and dark olive brown, layers of sandy silt and sandy clay.				
35			■		20, 50/4in, (N = 97.5)								

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Boring No.: B-1

Latitude	: 33.941623	**Drill Supplier**	: OneWay Drilling	**Job Number**	: 24-2369	**Sheet**	: 3 OF 3		
Longitude	: -118.327526	**Driller Company**	: OneWay Drilling	**Client**	: Inglewood Unified School District				
Elevation	: Not Surveyed	**Logged By**	: Albert Buffet	**Project**	: Child Care Development Center				
Total Depth	: 46.5 ft	**Date**	: 05/06/2024	**Location**	: 104th and 110th St, Los Angeles				

Depth (ft)	Samples			Blows per 6" (SPT)	Blows per 6" (ModCal)	Graphic Log	Soil Origin	Classification Code	Material Description	Testing			
	Bulk	SPT Sample	Mod Cal Sample							% Fines	Moisture %	Dry Density (pcf)	Other
		⧗		18, 40, 50/5in, (N = 120)		▨	Alluvium	SC	CLAYEY SAND (SC) : fine grained, dense, moist, yellowish brown and dark olive brown, layers of sandy silt and sandy clay.				
41						▨							
42						▨							
43						▨							
44						▨							
45		⧗		12, 30, 50, (N = 80)		▨							
46						▨							
									B-1 Terminated at 46.5ft (No Groundwater Encountered)				
47													
48													
49													
50													
51													
52													
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Phone: 909 606 6111

Boring No.: B-2

Latitude	: 33.941303	**Drill Supplier**	: OneWay Drilling	**Job Number**	: 24-2369
Longitude	: -118.327433	**Driller Company**	: OneWay Drilling	**Client**	: Inglewood Unified School District
Elevation	: Not Surveyed	**Logged By**	: Albert Buffet	**Project**	: Child Care Development Center
Total Depth	: 31 ft	**Date**	: 05/06/2024	**Location**	: 104th and 110th St, Los Angeles

Sheet : 1 OF 2

Depth (ft)	Samples			Blows per 6" (SPT)	Blows per 6" (ModCal)	Graphic Log	Soil Origin	Classification Code	Material Description	Testing			
	Bulk	SPT Sample	Mod Cal Sample							% Fines	Moisture %	Dry Density (pcf)	Other
0							Non-Soil	PAV	3 inches of AC over 12 inches of AB				
1	☒												
1.3							Fill	CL	SANDY LEAN CLAY (CL) : fine to medium grained sand, firm to stiff, moist, dark yellowish brown.				
2			■		7, 7, 11, (N = 11.7)					51	13.7	121	PP=2.2tsf
2.5							Alluvium	CL	SANDY LEAN CLAY (CL) : medium stiff to stiff, fine to medium grained sand, low plasticity, moist, dark yellowish brown, pockets of very moist clay and pockets of clayey sand .				
3.5													Direct Shear
5		⧗		7, 9, 10, (N = 19)			Alluvium	SC	CLAYEY SAND (SC) : fine to medium grained, medium dense, moist, dark yellowish brown, layers of sandy clay.	41	13.4		
6.5							Alluvium	SC	CLAYEY SAND (SC) : fine to medium grained, medium dense to dense, moist, dark yellowish brown, pockets of silty sand.				
7			■		9, 10, 15, (N = 16.25)					24	12.9	117	
9							Alluvium	SM	SILTY SAND (SM) : fine to coarse grained, medium dense, moist, light yellowish brown, layers of clayey sand .				
10		⧗		7, 10, 14, (N = 24)						13	10.0		
12							Alluvium	SP-SM	POORLY GRADED SAND WITH SILT (SP-SM) : fine to medium grained, dense to very dense, moist, light yellowish brown.				
15			■		20, 50, (N = 65)								

Boring No.: B-2

Latitude : 33.941303
Longitude : -118.327433
Elevation : Not Surveyed
Total Depth : 31 ft

Drill Supplier : OneWay Drilling
Driller Company : OneWay Drilling
Logged By : Albert Buffet
Date : 05/06/2024

Job Number : 24-2369
Client : Inglewood Unified School District
Project : Child Care Development Center
Location : 104th and 110th St, Los Angeles

Sheet : 2 OF 2

Depth (ft)	Samples			Blows per 6" (SPT)	Blows per 6" (ModCal)	Graphic Log	Soil Origin	Classification Code	Material Description	Testing			
	Bulk	SPT Sample	Mod Cal Sample							% Fines	Moisture %	Dry Density (pcf)	Other
		⧗		11, 21, 27, (N = 48)			Alluvium	SP-SM	POORLY GRADED SAND WITH SILT (SP-SM) : fine to medium grained, dense to very dense, moist, light yellowish brown.				
21													
22													
23													
24													
25			■		16, 50, (N = 65)								
26													
27													
28													
29							Alluvium	SP	POORLY GRADED SAND (SP) : fine to medium grained, dense to very dense, moist, light yellowish brown.				
30		⧗		18, 50/5in, (N = 120)									
31									**B-2 Terminated at 31ft (No Groundwater Encountered.)**				
32													
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Phone: 909 606 6111

Boring No.: B-3

Latitude : 33.941320
Longitude : -118.327719
Elevation : Not Surveyed
Total Depth : 26 ft

Drill Supplier : OneWay Drilling
Driller Company : OneWay Drilling
Logged By : Albert Buffet
Date : 05/06/2024

Job Number : 24-2369
Client : Inglewood Unified School District
Project : Child Care Development Center
Location : 104th and 110th St, Los Angeles

Sheet : 1 OF 2

Depth (ft)	Samples: Bulk	Samples: SPT Sample	Samples: Mod Cal Sample	Blows per 6" (SPT)	Blows per 6" (ModCal)	Graphic Log	Soil Origin	Classification Code	Material Description	Testing: % Fines	Testing: Moisture %	Testing: Dry Density (pcf)	Testing: Other
0.5							Non-Soil	PAV	2 inches of AC over 4 inches of AB				
3.5	✕	⧗		2, 3, 4, (N = 7)			Fill	CL	SANDY LEAN CLAY (CL) : fine to medium grained sand, stiff, moist, grayish brown, pockets of very moist clay .	62	15.2		PP=2.5-4.5ts
6	✕		■		13, 27, 38, (N = 42.25)		Alluvium	CL	SANDY LEAN CLAY (CL) : stiff to very stiff, fine to medium grained sand, low plasticity, moist, dark yellowish brown.	58	15.9	120	Expansion Index C-Series Testing PP= 3.5-4.5tsf
9		⧗		7, 10, 13, (N = 23)			Alluvium	SC	CLAYEY SAND (SC) : fine grained, medium dense, moist, dark yellowish brown, pockets of sandy clay.	34	12.5		
12			■		6, 12, 25, (N = 24.05)		Alluvium	SM	SILTY SAND (SM) : fine to medium grained, fine to medium grained sand, medium dense to dense, moist, dark yellowish brown, pockets of sandy clay .	13	8.9	106	
17.5		⧗		9, 15, 19, (N = 34)			Alluvium	SP-SM	POORLY GRADED SAND WITH SILT (SP-SM) : fine to medium grained, dense, moist, light yellowish brown.				
							Alluvium	SP-SM	POORLY GRADED SAND WITH SILT (SP-SM) : fine to coarse grained, dense to very dense, moist, light yellowish brown and dark yellowish brown, layers of silty sand .				

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Boring No.: B-3

Latitude	: 33.941320	**Drill Supplier**	: OneWay Drilling	**Job Number**	: 24-2369
Longitude	: -118.327719	**Driller Company**	: OneWay Drilling	**Client**	: Inglewood Unified School District
Elevation	: Not Surveyed	**Logged By**	: Albert Buffet	**Project**	: Child Care Development Center
Total Depth	: 26 ft	**Date**	: 05/06/2024	**Location**	: 104th and 110th St, Los Angeles

Sheet : 2 OF 2

Depth (ft)	Samples			Blows per 6" (SPT)	Blows per 6" (ModCal)	Graphic Log	Soil Origin	Classification Code	Material Description	Testing			
	Bulk	SPT Sample	Mod Cal Sample							% Fines	Moisture %	Dry Density (pcf)	Other
20–21			■		20, 50, (N = 65)	▨	Alluvium	SP-SM	POORLY GRADED SAND WITH SILT (SP-SM) : fine to coarse grained, dense to very dense, moist, light yellowish brown and dark yellowish brown, layers of silty sand .				
24						▨	Alluvium	SM	SILTY SAND (SM) : fine to coarse grained, dense to very dense, moist, trace fine sized gravel, light yellowish brown.				
25–26		⧗		22, 50, (N = 100)		▨							
26									**B-3 Terminated at 26ft (No Groundwater Encountered)**				

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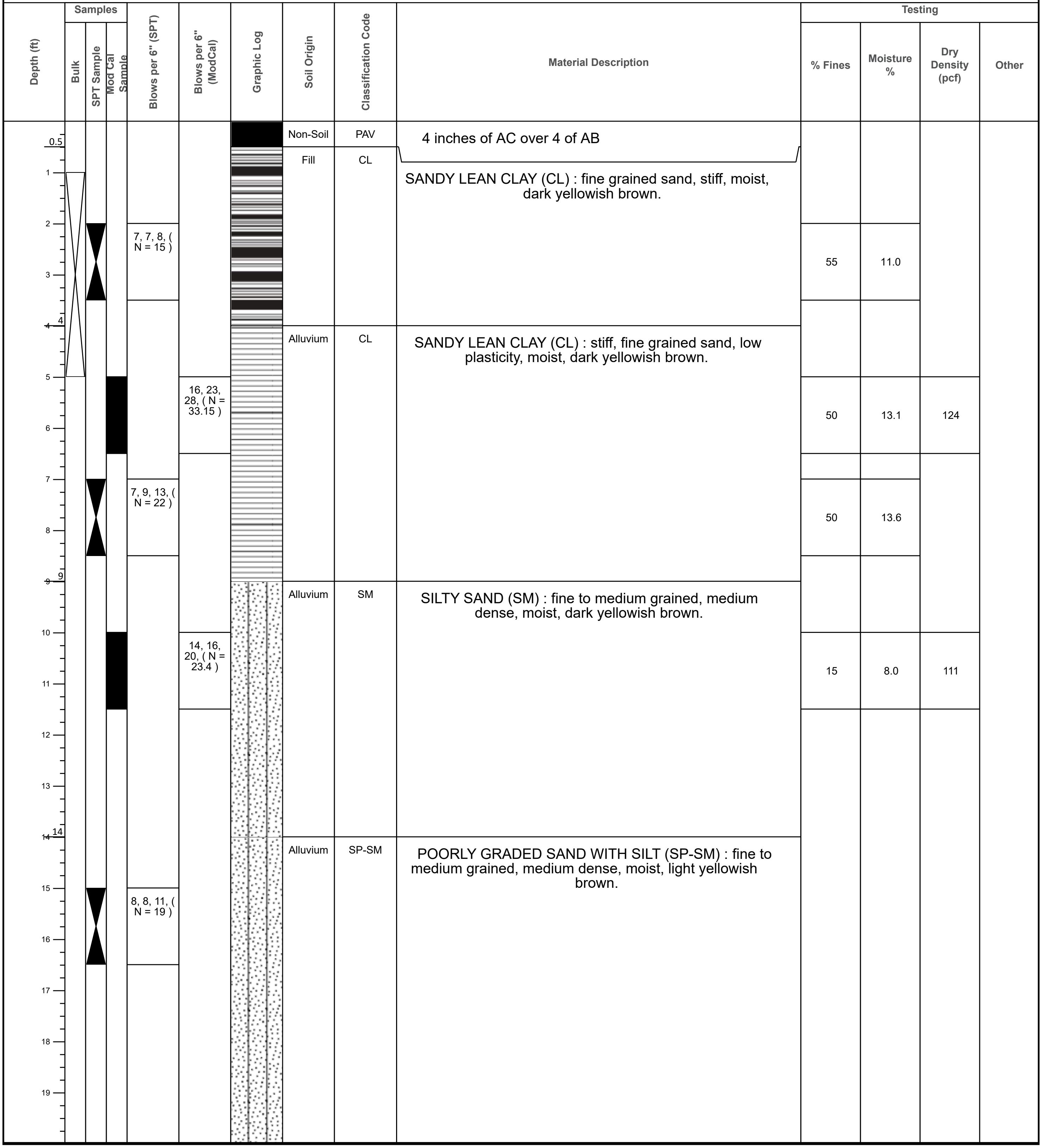
Boring No.: B-4

Latitude	: 33.941428	**Drill Supplier**	: OneWay Drilling	**Job Number**	: 24-2369
Longitude	: -118.327530	**Driller Company**	: OneWay Drilling	**Client**	: Inglewood Unified School District
Elevation	: Not Surveyed	**Logged By**	: Albert Buffet	**Project**	: Child Care Development Center
Total Depth	: 26.5 ft	**Date**	: 05/06/2024	**Location**	: 104th and 110th St, Los Angeles

Sheet : 1 OF 2



Depth (ft)	Samples: Bulk	Samples: SPT Sample	Samples: Mod Cal Sample	Blows per 6" (SPT)	Blows per 6" (ModCal)	Soil Origin	Classification Code	Material Description	Testing: % Fines	Testing: Moisture %	Testing: Dry Density (pcf)	Testing: Other
0 – 0.5						Non-Soil	PAV	4 inches of AC over 4 of AB				
0.5						Fill	CL	SANDY LEAN CLAY (CL) : fine grained sand, stiff, moist, dark yellowish brown.				
1 – 5	☒											
2 – 3.5		⧗		7, 7, 8, (N = 15)					55	11.0		
4						Alluvium	CL	SANDY LEAN CLAY (CL) : stiff, fine grained sand, low plasticity, moist, dark yellowish brown.				
5 – 6.5			■		16, 23, 28, (N = 33.15)				50	13.1	124	
7 – 8.5		⧗		7, 9, 13, (N = 22)					50	13.6		
9						Alluvium	SM	SILTY SAND (SM) : fine to medium grained, medium dense, moist, dark yellowish brown.				
10 – 11.5			■		14, 16, 20, (N = 23.4)				15	8.0	111	
14						Alluvium	SP-SM	POORLY GRADED SAND WITH SILT (SP-SM) : fine to medium grained, medium dense, moist, light yellowish brown.				
15 – 16.5		⧗		8, 8, 11, (N = 19)								

Boring No.: B-4

Latitude : 33.941428
Longitude : -118.327530
Elevation : Not Surveyed
Total Depth : 26.5 ft

Drill Supplier : OneWay Drilling
Driller Company : OneWay Drilling
Logged By : Albert Buffet
Date : 05/06/2024

Job Number : 24-2369
Client : Inglewood Unified School District
Project : Child Care Development Center
Location : 104th and 110th St, Los Angeles

Sheet : 2 OF 2

Depth (ft)	Samples			Blows per 6" (SPT)	Blows per 6" (ModCal)	Graphic Log	Soil Origin	Classification Code	Material Description	Testing			
	Bulk	SPT Sample	Mod Cal Sample							% Fines	Moisture %	Dry Density (pcf)	Other
20			■		17, 20, 22, (N = 27.3)	▨	Alluvium	SM	SILTY SAND (SM) : fine to medium grained, medium dense, moist, trace fine sized gravel, dark yellowish brown.				
24						▨	Alluvium	SP-SM	POORLY GRADED SAND WITH SILT (SP-SM) : fine to coarse grained, dense, moist, light yellowish brown.				
25		⧗		12, 15, 26, (N = 41)		▨							
26.5									**B-4 Terminated at 26.5ft (No Groundwater Encountered.)**				

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Boring No.: B-5

Latitude	: 33.941115	**Drill Supplier**	: OneWay Drilling	**Job Number**	: 24-2369
Longitude	: -118.327675	**Driller Company**	: OneWay Drilling	**Client**	: Inglewood Unified School District
Elevation	: Not Surveyed	**Logged By**	: Albert Buffet	**Project**	: Child Care Development Center
Total Depth	: 21.5 ft	**Date**	: 05/06/2024	**Location**	: 104th and 110th St, Los Angeles

Sheet : 1 OF 2

Depth (ft)	Samples: Bulk	Samples: SPT Sample	Samples: Mod Cal Sample	Blows per 6" (SPT)	Blows per 6" (ModCal)	Graphic Log	Soil Origin	Classification Code	Material Description	Testing: % Fines	Testing: Moisture %	Testing: Dry Density (pcf)	Testing: Other
0							Non-Soil	PAV	4 inches of AC over 6 of AB				
0.84							Fill	CL	SANDY LEAN CLAY (CL) : fine grained sand, stiff, moist, dark yellowish brown.				
1	☒												
2			■		5, 6, 11, (N = 11.05)					59	16.0	114	PP=4.5tsf Consolidatio
3							Alluvium	CL	SANDY LEAN CLAY (CL) : stiff, fine to medium grained sand, low plasticity, moist, dark yellowish brown.				
5		⧗		5, 11, 15, (N = 26)						51	15.1		PP=4.5tsf
7			■		5, 9, 15, (N = 15.6)		Alluvium	SC	CLAYEY SAND (SC) : fine to medium grained, medium dense, moist, dark yellowish brown, pockets of very moist clayey sand .	30	14.4	118	
9							Alluvium	SM	SILTY SAND (SM) : fine to medium grained, dense, moist, light yellowish brown.				
10		⧗		7, 9, 13, (N = 22)						13	10.1		
12							Alluvium	SP-SM	POORLY GRADED SAND WITH SILT (SP-SM) : fine to medium grained, dense to very dense, moist, light yellowish brown.				
15			■		20, 50, (N = 65)								
18							Alluvium	SP-SM	POORLY GRADED SAND WITH SILT (SP-SM) : fine to medium grained, dense, moist, light yellowish brown and dark yellowish brown, layers of silty sand .				

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Phone: 909 606 6111

Boring No.: B-5

Latitude	**: 33.941115**	**Drill Supplier**	**: OneWay Drilling**	**Job Number**	**: 24-2369**	**Sheet**	**: 2 OF 2**
Longitude	**: -118.327675**	**Driller Company**	**: OneWay Drilling**	**Client**	**: Inglewood Unified School District**		
Elevation	**: Not Surveyed**	**Logged By**	**: Albert Buffet**	**Project**	**: Child Care Development Center**		
Total Depth	**: 21.5 ft**	**Date**	**: 05/06/2024**	**Location**	**: 104th and 110th St, Los Angeles**		

Depth (ft)	Samples			Blows per 6" (SPT)	Blows per 6" (ModCal)	Graphic Log	Soil Origin	Classification Code	Material Description	Testing			
	Bulk	SPT Sample	Mod Cal Sample							% Fines	Moisture %	Dry Density (pcf)	Other
21		⧗		11, 16, 23, (N = 39)		▨	Alluvium	SP-SM	POORLY GRADED SAND WITH SILT (SP-SM) : fine to medium grained, dense, moist, light yellowish brown and dark yellowish brown, layers of silty sand .				
22 23 24 25 26 27 28 29 30 31 32 33 34 35 36 37 38 39									**B-5 Terminated at 21.5ft (No Groundwater Encountered)**				

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Boring No.: B-6

Latitude	: 33.941020	**Drill Supplier**	: OneWay Drilling	**Job Number**	: 24-2369	**Sheet**	: 1 OF 2
Longitude	: -118.327443	**Driller Company**	: OneWay Drilling	**Client**	: Inglewood Unified School District		
Elevation	: Not Surveyed	**Logged By**	: Albert Buffet	**Project**	: Child Care Development Center		
Total Depth	: 31.5 ft	**Date**	: 05/06/2024	**Location**	: 104th and 110th St, Los Angeles		

Depth (ft)	Samples Bulk	 SPT Sample	 Mod Cal Sample	Blows per 6" (SPT)	Blows per 6" (ModCal)	Graphic Log	Soil Origin	Classification Code	Material Description	Testing % Fines	 Moisture %	 Dry Density (pcf)	 Other
0–0.84							Non-Soil	PAV	4 inches of AC over 6 inches of AB				
0.84–2.5							Fill	CL	SANDY LEAN CLAY (CL) : fine grained sand, stiff, moist, dark yellowish brown.				
1–5	☒												
2–3.5		⧗		3, 6, 8, (N = 14)						50	11.8		
2.5–7.5							Alluvium	CL	SANDY LEAN CLAY (CL) : stiff to very stiff, fine grained sand, low plasticity, moist, dark yellowish brown.				
5–6.5			■		8, 16, 24, (N = 26)					50	13.7	122	Consolidatio
7–8.5		⧗		7, 8, 8, (N = 16)						23	12.5		
7.5–12							Alluvium	SC	CLAYEY SAND (SC) : fine to medium grained, medium dense to dense, moist, dark yellowish brown.				
10–11.5			■		25, 25, 26, (N = 33.15)					13	10.8	103	
12–							Alluvium	SP-SM	POORLY GRADED SAND WITH SILT (SP-SM) : fine to medium grained, medium dense, moist, light yellowish brown.				
15–16.5		⧗		8, 8, 13, (N = 21)									

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Phone: 909 606 6111

Boring No.: B-6

Latitude	: 33.941020	**Drill Supplier**	: OneWay Drilling	**Job Number**	: 24-2369
Longitude	: -118.327443	**Driller Company**	: OneWay Drilling	**Client**	: Inglewood Unified School District
Elevation	: Not Surveyed	**Logged By**	: Albert Buffet	**Project**	: Child Care Development Center
Total Depth	: 31.5 ft	**Date**	: 05/06/2024	**Location**	: 104th and 110th St, Los Angeles

Sheet : 2 OF 2

Depth (ft)	Samples: Bulk	Samples: SPT Sample	Samples: Mod Cal Sample	Blows per 6" (SPT)	Blows per 6" (ModCal)	Graphic Log	Soil Origin	Classification Code	Material Description	Testing: % Fines	Testing: Moisture %	Testing: Dry Density (pcf)	Testing: Other
20–21.5			■		21, 50, (N = 65)		Alluvium	SP-SM	POORLY GRADED SAND WITH SILT (SP-SM) : fine to medium grained, medium dense, moist, light yellowish brown.				
25–26.5		⧗		8, 21, 26, (N = 47)									
29							Alluvium	SP	POORLY GRADED SAND (SP) : fine to medium grained, dense, moist to slightly moist, light yellowish brown.				
30–31.5		⧗		9, 18, 32, (N = 50)									

Depth scale: 21, 22, 23, 24, 25, 26, 27, 28, 29, 30, 31, 32, 33, 34, 35, 36, 37, 38, 39

B-6 Terminated at 31.5ft (No Groundwater Encountered)

Koury Engineering

14280 Euclid Avenue, Chino, California, 91710

Phone: 909 606 6111

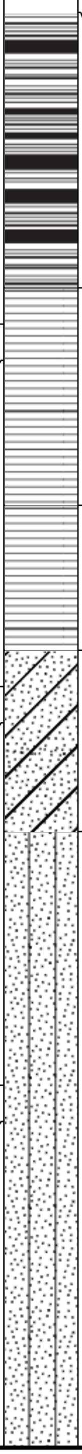
Boring No.: B-7

Latitude : 33.940711
Longitude : -118.327424
Elevation : Not Surveyed
Total Depth : 20 ft

Drill Supplier : Koury Engineering & Testing, Inc.
Driller Company : Koury Engineering & Testing, Inc.
Logged By : Albert Buffet
Date : 05/07/2024

Job Number : 24-2369
Client : Inglewood Unified School District
Project : Child Care Development Center
Location : 104th and 110th St, Los Angeles

Sheet : 1 OF 1



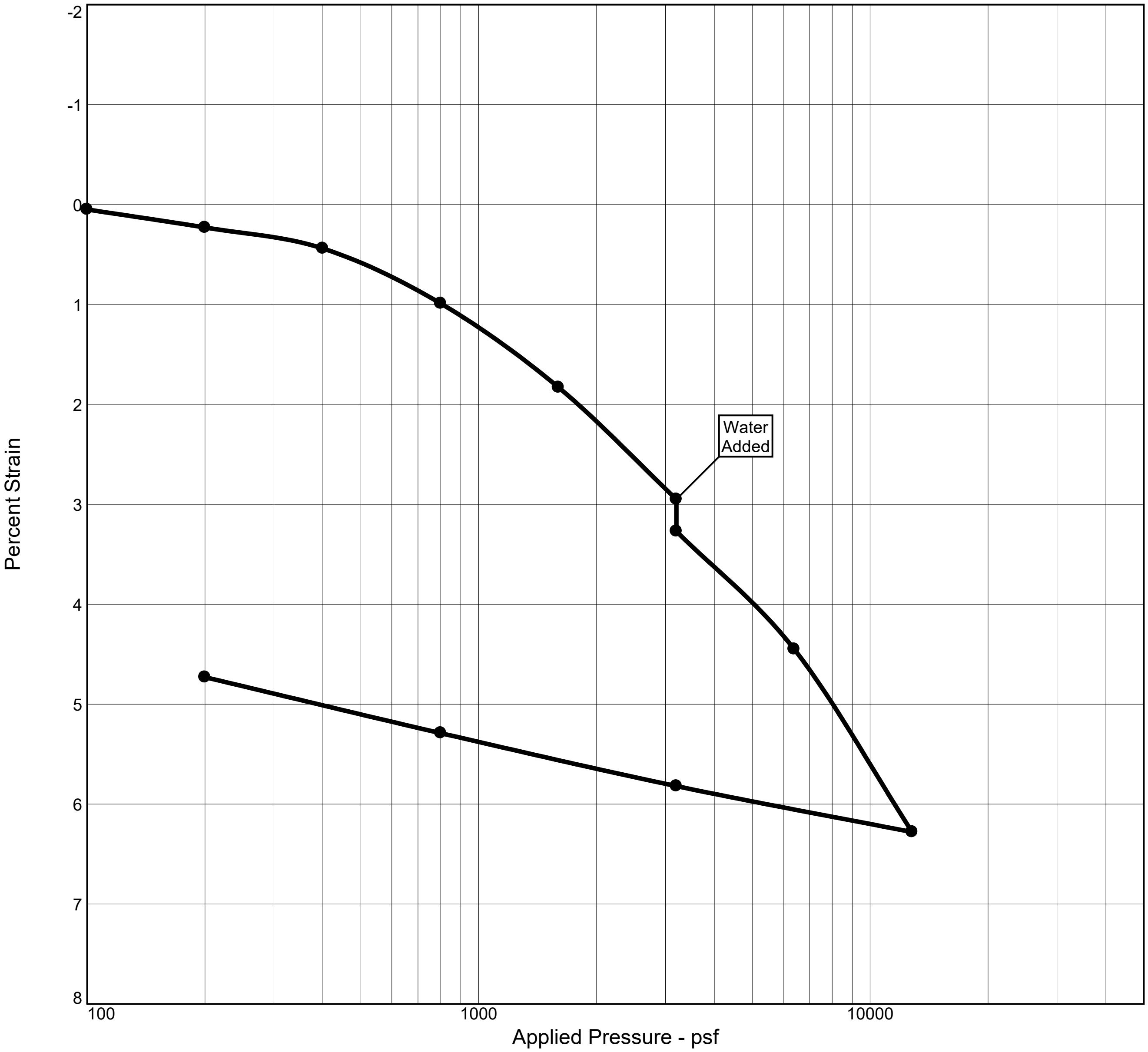
Depth (ft)	Blows per 6" (ModCal)	Soil Origin	Classification Code	Material Description	% Fines	Moisture %	Dry Density (pcf)	Other
0.2		Non-Soil	GRASS	Grass over topsoil				
		Fill	CL	SANDY LEAN CLAY (CL) : fine grained sand, firm to stiff, moist, dark yellowish brown.				
4	20, (N = 26)	Alluvium	CL	LEAN CLAY (CL) : medium stiff to stiff, low plasticity, moist, dark yellowish brown, pockets of very moist clay .	53	12.5	122	PP=3.5-4.2ts
7		Alluvium	CL	SANDY LEAN CLAY (CL) : medium stiff, fine to medium grained sand, low plasticity, moist, dark yellowish brown.				
9	32, (N = 41.6)	Alluvium	SC	CLAYEY SAND (SC) : fine to medium grained, medium dense, moist, dark yellowish brown.	37	13.2	109	PP=2.8-4.0ts
11.5	55, (N = 71.5)	Alluvium	SP-SM	POORLY GRADED SAND WITH SILT (SP-SM) : fine to medium grained, medium dense, moist, light yellowish brown.				

B-7 Terminated at 20ft (No Groundwater Encountered)

APPENDIX C

Laboratory Test Results & Calculations

CONSOLIDATION TEST REPORT



Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	Overburden (psf)	P_c (psf)	C_c	C_s	Swell Press. (psf)	Clpse. %	e_o
Sat.	Moist.											
101.4 %	15.4 %	119.4			2.7	375	6120	0.09	0.01		0.3	0.411

MATERIAL DESCRIPTION	USCS	AASHTO
Sandy Lean Clay	CL	

Project No. 24-2369 **Client:**

Project: New Child Development Center Campus

Location: B-5 @ 3'

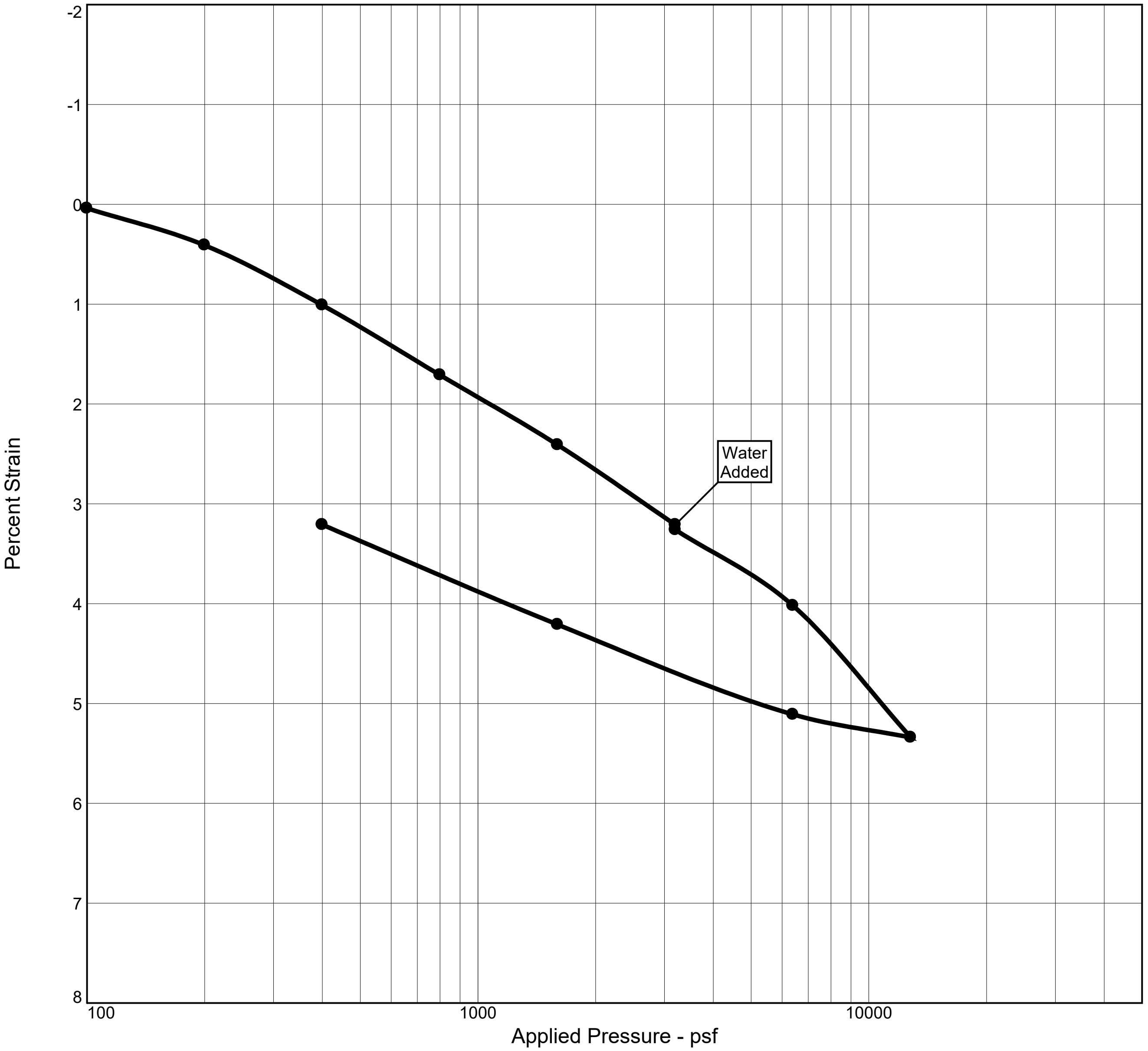
Remarks:

Koury Engineering & Testing, Inc.

Chino, CA

Figure

CONSOLIDATION TEST REPORT



Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	Overburden (psf)	P_c (psf)	C_c	C_s	Swell Press. (psf)	Clpse. %	e_o
Sat.	Moist.											
94.4 %	15.0 %	118.0			2.7	810	6410	0.06	0.02		0.0	0.429

MATERIAL DESCRIPTION	USCS	AASHTO
Sandy Lean Clay		

Project No. 24-2369 **Client:**

Project: New Child Development Center Campus

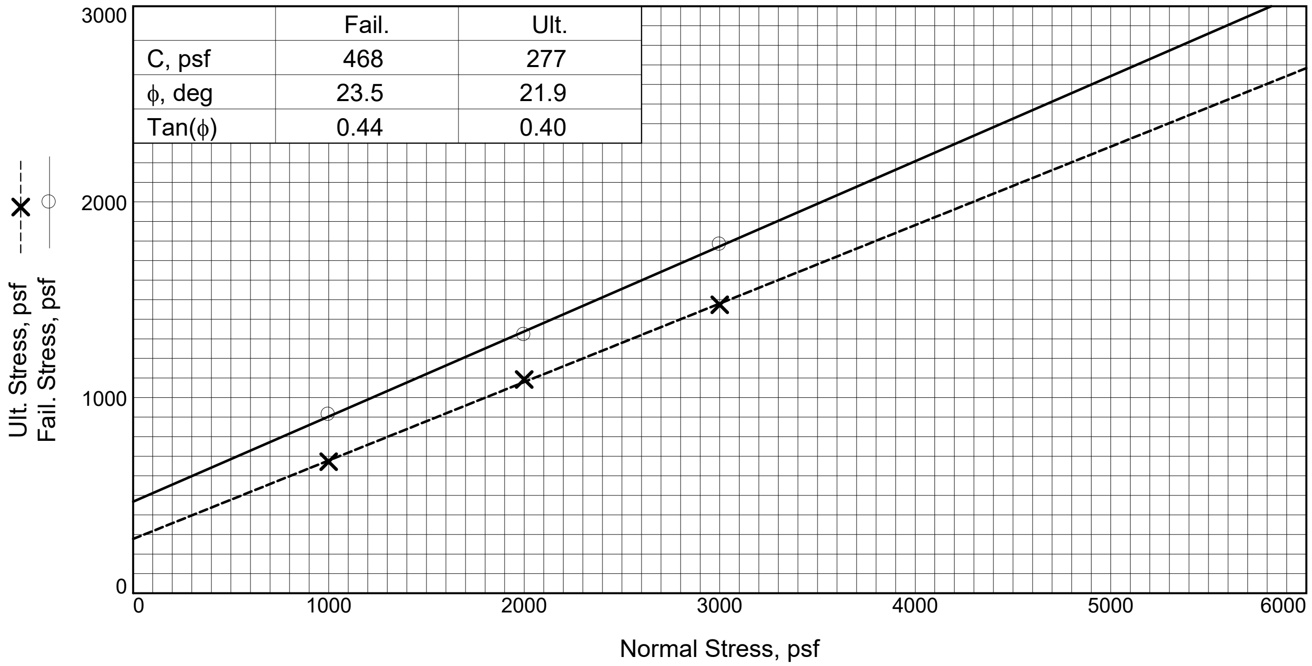
Location: B-5 @ 6'-

Remarks:

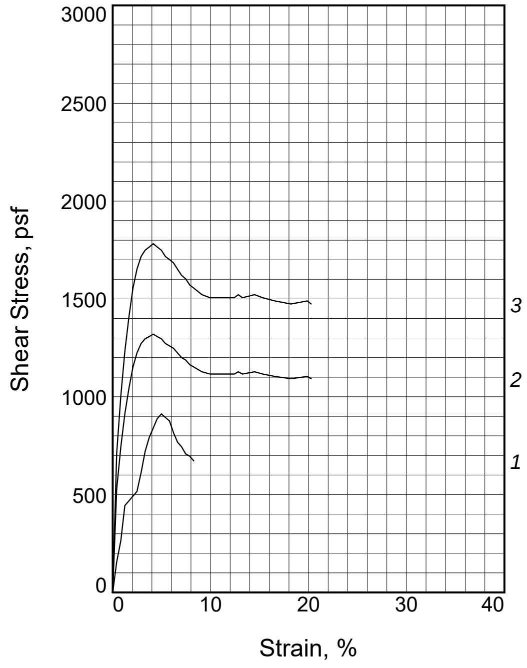
Koury Engineering & Testing, Inc.

Chino, CA

Figure



	Fail.	Ult.
C, psf	468	277
ϕ, deg	23.5	21.9
Tan(ϕ)	0.44	0.40



	Sample No.	1	2	3
Initial	Water Content, %	15.6	15.1	14.7
	Dry Density, pcf	116.8	116.6	118.0
	Saturation, %	95.4	91.8	88.1
	Void Ratio	0.4428	0.4451	0.4593
	Diameter, in.	2.42	2.42	2.42
	Height, in.	1.00	1.00	1.00
At Test	Water Content, %	16.0	14.7	16.7
	Dry Density, pcf	116.8	116.6	118.0
	Saturation, %	97.3	89.3	100.0
	Void Ratio	0.4428	0.4451	0.4593
	Diameter, in.	2.42	2.42	2.42
	Height, in.	1.00	1.00	1.00
	Normal Stress, psf	1000	2000	3000
	Fail. Stress, psf	912	1320	1782
	Strain, %	5.0	4.1	4.1
	Ult. Stress, psf	672	1092	1474
	Strain, %	8.3	20.3	20.3
	Strain rate, in./min.	0.001	0.001	0.001

Sample Type:

Description: Sandy Lean Clay

Specific Gravity= 2.7

Remarks:

Figure ________

Client:

Project: New Child Development Center Campus

Location: B-2 @ 3'

Proj. No.: 24-2369 **Date Sampled:**

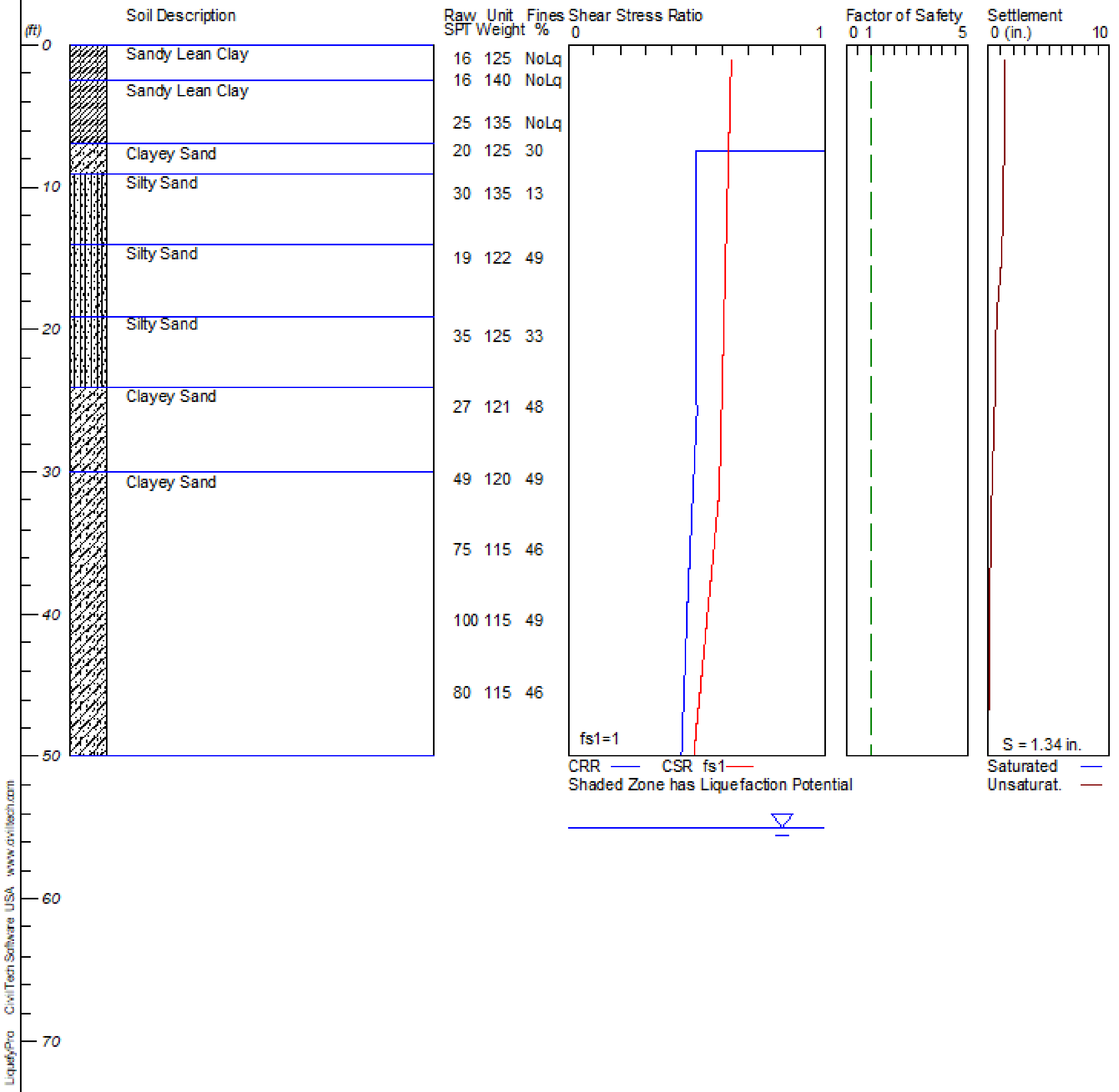
DIRECT SHEAR TEST REPORT
Koury Engineering & Testing, Inc.
Chino, CA

DRY SEISMIC SETTLEMENT

Child Care Development Center

Hole No.=B-1 Water Depth=55 ft

Magnitude=7.5
Acceleration=0.98g



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                                    LIQUEFACTION ANALYSIS SUMMARY
                                   Copyright by CivilTech Software
                                         www.civiltech.com
  *********************************************************************************************************
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           Licensed to ,            6/6/2024   12:51:56 PM

           Input File Name: P:\2024\24-2369 Cordoba - Morningside High School SI\Soils Folder\Calculations\Seismic Settlement\B-1
Seismic Settlement.liq
           Title:  Child Care Development Center
           Subtitle:  24-2369

           Surface Elev.=
           Hole No.=B-1
           Depth of Hole= 50.00 ft
           Water Table during Earthquake= 55.00 ft
           Water Table during In-Situ Testing= 55.00 ft
           Max. Acceleration= 0.98 g
           Earthquake Magnitude= 7.50

 Input Data:
           Surface Elev.=
           Hole No.=B-1
           Depth of Hole=50.00 ft
           Water Table during Earthquake= 55.00 ft
           Water Table during In-Situ Testing= 55.00 ft
           Max. Acceleration=0.98 g
           Earthquake Magnitude=7.50
           No-Liquefiable Soils:   CL, OL are Non-Liq. Soil

           1. SPT or BPT Calculation.
           2. Settlement Analysis Method: Tokimatsu/Seed
           3. Fines Correction for Liquefaction: Idriss/Seed
           4. Fine Correction for Settlement: During Liquefaction*
           5. Settlement Calculation in: All zones*
           6. Hammer Energy Ratio,                        Ce = 1.25
           7. Borehole Diameter,                           Cb= 1.0
           8. Sampling Method,                            Cs= 1.2
           9. User request factor of safety (apply to CSR) ,   User= 1
             Plot one CSR curve (fs1=1)
           10. Use Curve Smoothing: No
           * Recommended Options

           In-Situ Test Data:
           Depth     SPT       gamma     Fines
           ft                  pcf       %
           _____________________________________
           1.00      16.00     125.00    NoLiq
           2.50      16.00     140.00    NoLiq
           5.50      25.00     135.00    NoLiq
           7.50      20.00     125.00    30.00
           10.50     30.00     135.00    13.00
           15.00     19.00     122.00    49.00
           20.50     35.00     125.00    33.00
           25.50     27.00     121.00    48.00
           30.50     49.00     120.00    49.00
```

```
35.50     75.00     115.00    46.00
40.50     100.00    115.00    49.00
45.50     80.00     115.00    46.00
___________________________________________

Output Results:
          Settlement of Saturated Sands=0.00 in.
          Settlement of Unsaturated Sands=1.34 in.
          Total Settlement of Saturated and Unsaturated Sands=1.34 in.
          Differential Settlement=0.671 to 0.886 in.

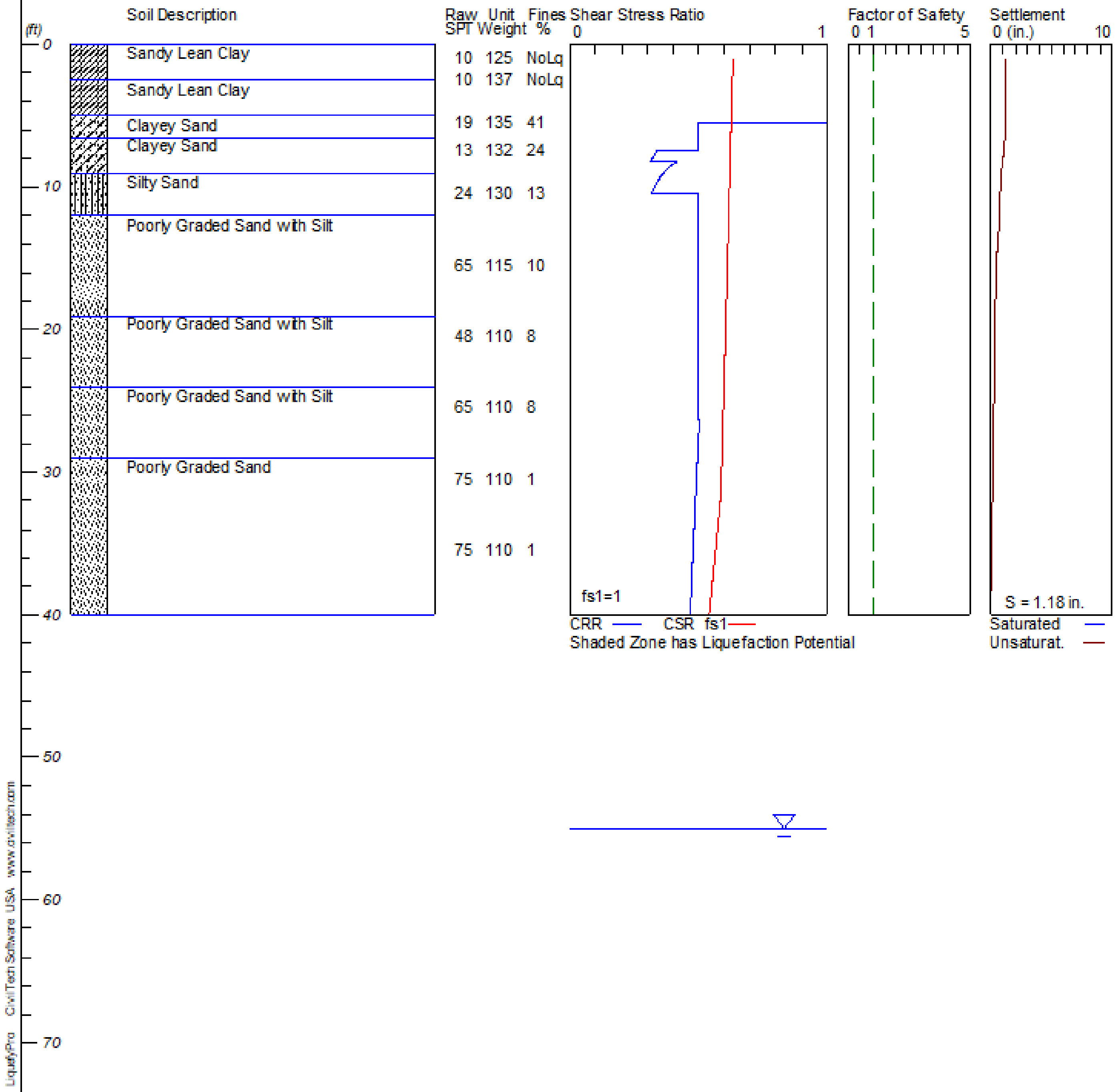
          Depth     CRRm      CSRfs     F.S.      S_sat.    S_dry     S_all
          ft                                      in.       in.       in.
          __________________________________________________________________
          1.00      2.00      0.64      5.00      0.00      1.34      1.34
          1.05      2.00      0.64      5.00      0.00      1.34      1.34
          1.10      2.00      0.64      5.00      0.00      1.34      1.34
          1.15      2.00      0.64      5.00      0.00      1.34      1.34
          1.20      2.00      0.64      5.00      0.00      1.34      1.34
          1.25      2.00      0.64      5.00      0.00      1.34      1.34
          1.30      2.00      0.64      5.00      0.00      1.34      1.34
          1.35      2.00      0.63      5.00      0.00      1.34      1.34
          1.40      2.00      0.63      5.00      0.00      1.34      1.34
          1.45      2.00      0.63      5.00      0.00      1.34      1.34
          1.50      2.00      0.63      5.00      0.00      1.34      1.34
          1.55      2.00      0.63      5.00      0.00      1.34      1.34
          1.60      2.00      0.63      5.00      0.00      1.34      1.34
          1.65      2.00      0.63      5.00      0.00      1.34      1.34
          1.70      2.00      0.63      5.00      0.00      1.34      1.34
          1.75      2.00      0.63      5.00      0.00      1.34      1.34
          1.80      2.00      0.63      5.00      0.00      1.34      1.34
          1.85      2.00      0.63      5.00      0.00      1.34      1.34
          1.90      2.00      0.63      5.00      0.00      1.34      1.34
          1.95      2.00      0.63      5.00      0.00      1.34      1.34
          2.00      2.00      0.63      5.00      0.00      1.34      1.34
          2.05      2.00      0.63      5.00      0.00      1.34      1.34
          2.10      2.00      0.63      5.00      0.00      1.34      1.34
          2.15      2.00      0.63      5.00      0.00      1.34      1.34
          2.20      2.00      0.63      5.00      0.00      1.34      1.34
          2.25      2.00      0.63      5.00      0.00      1.34      1.34
          2.30      2.00      0.63      5.00      0.00      1.34      1.34
          2.35      2.00      0.63      5.00      0.00      1.34      1.34
          2.40      2.00      0.63      5.00      0.00      1.34      1.34
          2.45      2.00      0.63      5.00      0.00      1.34      1.34
          2.50      2.00      0.63      5.00      0.00      1.34      1.34
          2.55      2.00      0.63      5.00      0.00      1.34      1.34
          2.60      2.00      0.63      5.00      0.00      1.34      1.34
          2.65      2.00      0.63      5.00      0.00      1.34      1.34
          2.70      2.00      0.63      5.00      0.00      1.34      1.34
          2.75      2.00      0.63      5.00      0.00      1.34      1.34
          2.80      2.00      0.63      5.00      0.00      1.34      1.34
          2.85      2.00      0.63      5.00      0.00      1.34      1.34
          2.90      2.00      0.63      5.00      0.00      1.34      1.34
          2.95      2.00      0.63      5.00      0.00      1.34      1.34
          3.00      2.00      0.63      5.00      0.00      1.34      1.34
          3.05      2.00      0.63      5.00      0.00      1.34      1.34
          3.10      2.00      0.63      5.00      0.00      1.34      1.34
```

DRY SEISMIC SETTLEMENT

Child Care Development Center

Hole No.=B-2 Water Depth=55 ft

Magnitude=7.5
Acceleration=0.98g



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                              LIQUEFACTION ANALYSIS SUMMARY
                              Copyright by CivilTech Software
                                    www.civiltech.com
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          Licensed to ,            6/6/2024   12:55:29 PM

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Seismic Settlement.liq
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          Subtitle:  24-2369

          Surface Elev.=
          Hole No.=B-2
          Depth of Hole= 40.00 ft
          Water Table during Earthquake= 55.00 ft
          Water Table during In-Situ Testing= 55.00 ft
          Max. Acceleration= 0.98 g
          Earthquake Magnitude= 7.50

 Input Data:
          Surface Elev.=
          Hole No.=B-2
          Depth of Hole=40.00 ft
          Water Table during Earthquake= 55.00 ft
          Water Table during In-Situ Testing= 55.00 ft
          Max. Acceleration=0.98 g
          Earthquake Magnitude=7.50
          No-Liquefiable Soils:   CL, OL are Non-Liq. Soil

          1. SPT or BPT Calculation.
          2. Settlement Analysis Method: Tokimatsu/Seed
          3. Fines Correction for Liquefaction: Idriss/Seed
          4. Fine Correction for Settlement: During Liquefaction*
          5. Settlement Calculation in: All zones*
          6. Hammer Energy Ratio,                        Ce = 1.25
          7. Borehole Diameter,                          Cb= 1.0
          8. Sampling Method,                            Cs= 1.2
          9. User request factor of safety (apply to CSR) ,   User= 1
            Plot one CSR curve (fs1=1)
          10. Use Curve Smoothing: No
          * Recommended Options

          In-Situ Test Data:
          Depth     SPT       gamma    Fines
          ft                  pcf      %
          ____________________________________
          1.00      10.00     125.00   NoLiq
          2.50      10.00     137.00   NoLiq
          5.50      19.00     135.00   41.00
          7.50      13.00     132.00   24.00
          10.50     24.00     130.00   13.00
          15.50     65.00     115.00   10.00
          20.50     48.00     110.00   8.00
          25.50     65.00     110.00   8.00
          30.50     75.00     110.00   1.00
```

```
        35.50    75.00    110.00   1.00
        _________________________________________

Output Results:
        Settlement of Saturated Sands=0.00 in.
        Settlement of Unsaturated Sands=1.18 in.
        Total Settlement of Saturated and Unsaturated Sands=1.18 in.
        Differential Settlement=0.588 to 0.776 in.

        Depth    CRRm     CSRfs    F.S.     S_sat.   S_dry    S_all
        ft                                  in.      in.      in.
        _______________________________________________________________
        1.00     2.00     0.64     5.00     0.00     1.18     1.18
        1.05     2.00     0.64     5.00     0.00     1.18     1.18
        1.10     2.00     0.64     5.00     0.00     1.18     1.18
        1.15     2.00     0.64     5.00     0.00     1.18     1.18
        1.20     2.00     0.64     5.00     0.00     1.18     1.18
        1.25     2.00     0.64     5.00     0.00     1.18     1.18
        1.30     2.00     0.64     5.00     0.00     1.18     1.18
        1.35     2.00     0.63     5.00     0.00     1.18     1.18
        1.40     2.00     0.63     5.00     0.00     1.18     1.18
        1.45     2.00     0.63     5.00     0.00     1.18     1.18
        1.50     2.00     0.63     5.00     0.00     1.18     1.18
        1.55     2.00     0.63     5.00     0.00     1.18     1.18
        1.60     2.00     0.63     5.00     0.00     1.18     1.18
        1.65     2.00     0.63     5.00     0.00     1.18     1.18
        1.70     2.00     0.63     5.00     0.00     1.18     1.18
        1.75     2.00     0.63     5.00     0.00     1.18     1.18
        1.80     2.00     0.63     5.00     0.00     1.18     1.18
        1.85     2.00     0.63     5.00     0.00     1.18     1.18
        1.90     2.00     0.63     5.00     0.00     1.18     1.18
        1.95     2.00     0.63     5.00     0.00     1.18     1.18
        2.00     2.00     0.63     5.00     0.00     1.18     1.18
        2.05     2.00     0.63     5.00     0.00     1.18     1.18
        2.10     2.00     0.63     5.00     0.00     1.18     1.18
        2.15     2.00     0.63     5.00     0.00     1.18     1.18
        2.20     2.00     0.63     5.00     0.00     1.18     1.18
        2.25     2.00     0.63     5.00     0.00     1.18     1.18
        2.30     2.00     0.63     5.00     0.00     1.18     1.18
        2.35     2.00     0.63     5.00     0.00     1.18     1.18
        2.40     2.00     0.63     5.00     0.00     1.18     1.18
        2.45     2.00     0.63     5.00     0.00     1.18     1.18
        2.50     2.00     0.63     5.00     0.00     1.18     1.18
        2.55     2.00     0.63     5.00     0.00     1.18     1.18
        2.60     2.00     0.63     5.00     0.00     1.18     1.18
        2.65     2.00     0.63     5.00     0.00     1.18     1.18
        2.70     2.00     0.63     5.00     0.00     1.18     1.18
        2.75     2.00     0.63     5.00     0.00     1.18     1.18
        2.80     2.00     0.63     5.00     0.00     1.18     1.18
        2.85     2.00     0.63     5.00     0.00     1.18     1.18
        2.90     2.00     0.63     5.00     0.00     1.18     1.18
        2.95     2.00     0.63     5.00     0.00     1.18     1.18
        3.00     2.00     0.63     5.00     0.00     1.18     1.18
        3.05     2.00     0.63     5.00     0.00     1.18     1.18
        3.10     2.00     0.63     5.00     0.00     1.18     1.18
        3.15     2.00     0.63     5.00     0.00     1.18     1.18
        3.20     2.00     0.63     5.00     0.00     1.18     1.18
```

Percolation Testing

KOURY
PASSION INTEGRITY QUALITY

Job Name: Child Development Center Campus

Job No.: 24-2369

Test No.: P-1

Test Location: Approx. 85' south of W. 104th St. CL & Approx. 52' west of S. 10th Ave. CL (See Figure A-2)

Depth of Boring (d_b): 162 in

Water Table Depth (Perched) (ft): > 50 ft **Relatively Impervious Layer Depth (ft):** 0-11 ft

Diameter of Boring (D): 9 in

Test Date: 5/7/2024

Test Performer: ABB

Trial No.	Initial Time T_1 (min)	Final Time T_2 (min)	Time Interval $\Delta T = T_2 - T_1$ (min)	Initial Depth to Water d_1 (in)	Final Depth to Water d_2 (in)	Initial Height of Water Column $d_{H1} = d_b - d_1$ (in)	Final Height of Water Column $d_{H2} = d_b - d_2$ (in)	Average Height of Water Column $d_{avg} = (d_{H1}+d_{H2})/2$ (in)	Drop in Height $\Delta d_H = d_{H1} - d_{H2}$ (in)	Boring Wet Surface Area (ft^2)	Volume of Water Drop (ft^3)	Calculated Flow Rate (ft^3/min)	Calculated Flow Rate (ft^3/hr)	Infltration Rate (in/hr)
1	0.0	30.0	30.0	142 6/8	152 3/8	19 2/8	9 5/8	14 3/8	9 5/8	2.33	0.35	0.01	0.71	3.65
2	0.0	30.0	30.0	144	151 6/8	18	10 2/8	14 1/8	7 6/8	2.44	0.29	0.01	0.57	2.82
3	0.0	30.0	30.0	144	151 6/8	18	10 2/8	14 1/8	7 6/8	2.46	0.28	0.01	0.57	2.76
4	0.0	30.0	30.0	144	151 2/8	18	10 6/8	14 3/8	7 2/8	2.56	0.27	0.01	0.53	2.48
5	0.0	30.0	30.0	144	151 2/8	18	10 6/8	14 3/8	7 2/8	2.56	0.27	0.01	0.53	2.48
6	0.0	30.0	30.0	144	151 2/8	18	10 6/8	14 3/8	7 2/8	2.56	0.27	0.01	0.53	2.48
7	0.0	30.0	30.0	144	151 2/8	18	10 6/8	14 3/8	7 2/8	2.56	0.27	0.01	0.53	2.48
8	0.0	30.0	30.0	144	151 2/8	18	10 6/8	14 3/8	7 2/8	2.56	0.27	0.01	0.53	2.48
9	0.0	30.0	30.0	144	151	18	11	14 4/8	7	2.61	0.26	0.01	0.51	2.36
10	0.0	30.0	30.0	144	150 6/8	18	11 2/8	14 5/8	6 6/8	2.64	0.25	0.01	0.50	2.28

Notes:

During Testing:

Average Infiltration Rate (in/hr)=	2.63
Median Infiltration Rate (in/hr)=	2.48
Lowest Infiltration Rate (in/hr)=	2.28
Average Water Depth (ft) =	1.20
Median Water Depth (ft) =	1.20
Total Time of Test Conducted (hr) =	5.00

$RF = RF_t + RF_v + RF_s = 4.5$

$RF_t = 2$

$RF_v = 1.5$

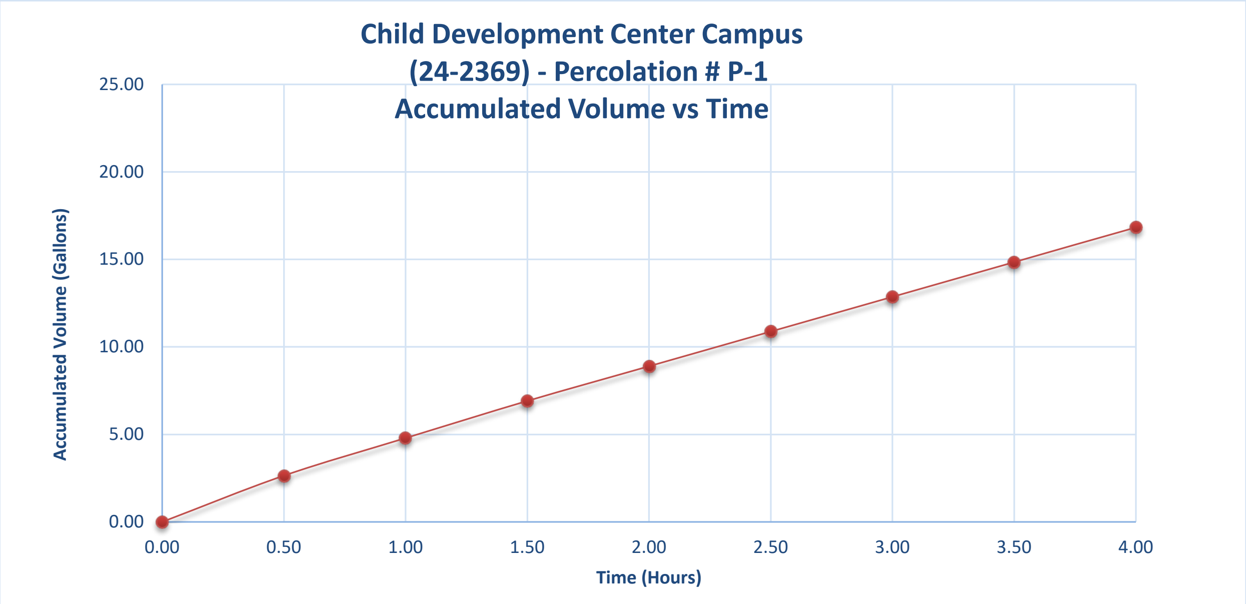
$RF_s = 1$

Reduction Factor for Boring Percolation Testing RFt, Site Variability, Number of Tests & Borings RFv, and Long Term Maintenance RFs

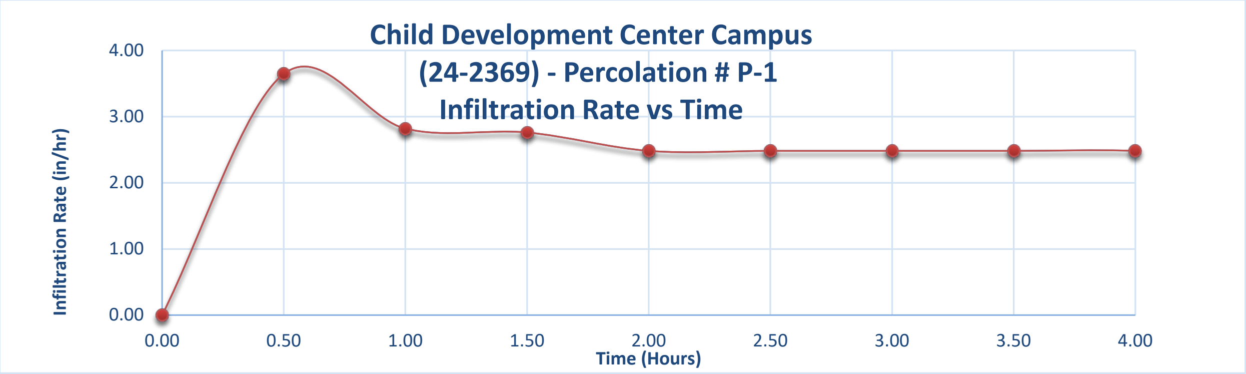
Short Term Infiltration Rate (in/hr) = **2.3**

Long Term Infiltration Rate (in/hr) = **0.5**

Reference: Administrative Manual - Los Angeles County Public works, Geotechnical & Materials Engineering Division, GS200.1 dated 06/30/21



Child Development Center Campus
(24-2369) - Percolation # P-1
Accumulated Volume vs Time



Child Development Center Campus
(24-2369) - Percolation # P-1
Infiltration Rate vs Time

Percolation Testing

Job Name: Child Development Center Campus

Job No.: 24-2369

Test No.: P-2

Test Location: Approx. 420' south of W. 104th St. CL & Approx. 33' west of S. 10th Ave. CL (See Figure A-2)

Depth of Boring (d_b): 162 in

Water Table Depth (Perched) (ft): > 50ft

Relatively Impervious Layer Depth (ft): 0-11 ft

Diameter of Boring (D): 6 in

Test Date: 5/7/2024

Test Performer: ABB

Trial No.	Initial Time T_1 (min)	Final Time T_2 (min)	Time Interval $\Delta T = T_2 - T_1$ (min)	Initial Depth to Water d_1 (in)	Final Depth to Water d_2 (in)	Initial Height of Water Column $d_{H1} = d_b - d_1$ (in)	Final Height of Water Column $d_{H2} = d_b - d_2$ (in)	Average Height of Water Column $d_{avg} = (d_{H1}+d_{H2})/2$ (in)	Drop in Height $\Delta d_H = d_{H1} - d_{H2}$ (in)	Boring Wet Surface Area (ft^2)	Volume of Water Drop (ft^3)	Calculated Flow Rate (ft^3/min)	Calculated Flow Rate (ft^3/hr)	Infltration Rate (in/hr)
1	0.0	30.0	30.0	132	149 3/8	30	12 5/8	21 2/8	17 3/8	1.85	0.28	0.01	0.57	3.70
2	0.0	30.0	30.0	132	145 2/8	30	16 6/8	23 3/8	13 2/8	2.40	0.22	0.01	0.43	2.16
3	0.0	30.0	30.0	132	144 5/8	30	17 3/8	23 6/8	12 5/8	2.47	0.21	0.01	0.41	2.00
4	0.0	30.0	30.0	132	144 5/8	30	17 3/8	23 6/8	12 5/8	2.47	0.21	0.01	0.41	2.00
5	0.0	30.0	30.0	132	144	30	18	24	12	2.55	0.20	0.01	0.39	1.85
6	0.0	30.0	30.0	132	144	30	18	24	12	2.55	0.20	0.01	0.39	1.85
7	0.0	30.0	30.0	132	144	30	18	24	12	2.55	0.20	0.01	0.39	1.85
8	0.0	30.0	30.0	132	143 3/8	30	18 5/8	24 2/8	11 3/8	2.63	0.19	0.01	0.37	1.70
9	0.0	30.0	30.0	132	143 3/8	30	18 5/8	24 2/8	11 3/8	2.63	0.19	0.01	0.37	1.70

Notes:

During Testing:

Average Infiltration Rate (in/hr)= 2.09

Median Infiltration Rate (in/hr)= 1.85

Lowest Infiltration Rate (in/hr)= 1.70

Average Water Depth (ft) = 1.97

Median Water Depth (ft) = 2.00

Total Time of Test Conducted (hr) = 4.50

RF= $RF_t+RF_v+RF_s$ = 4.5

RF_t= 2

RF_v= 1.5

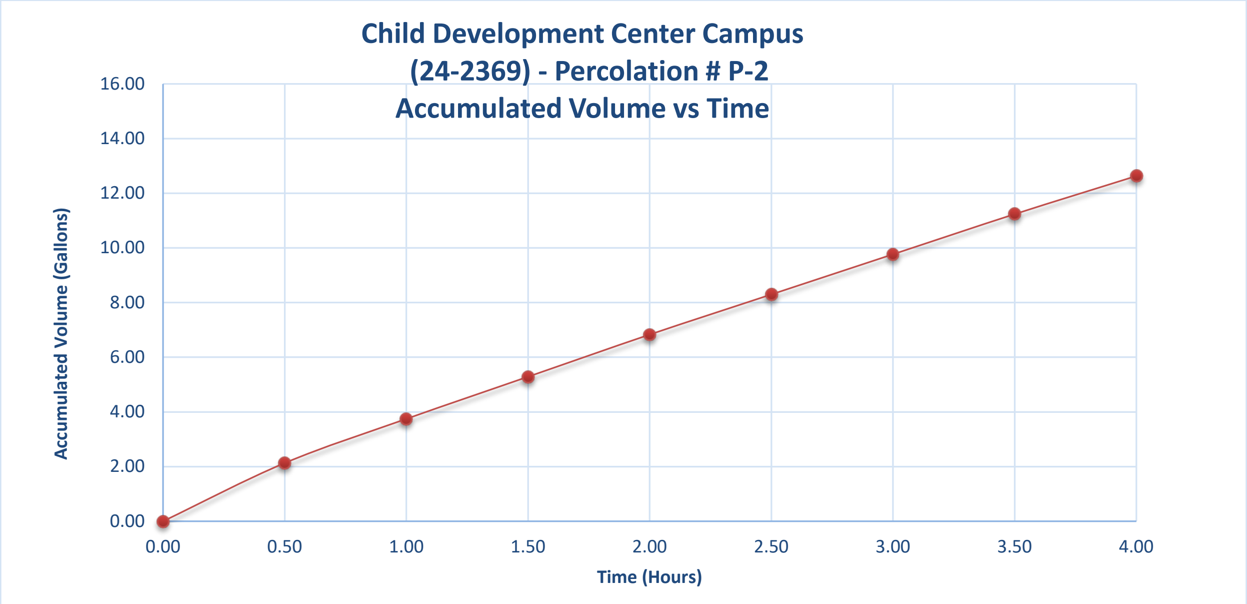
RF_s= 1

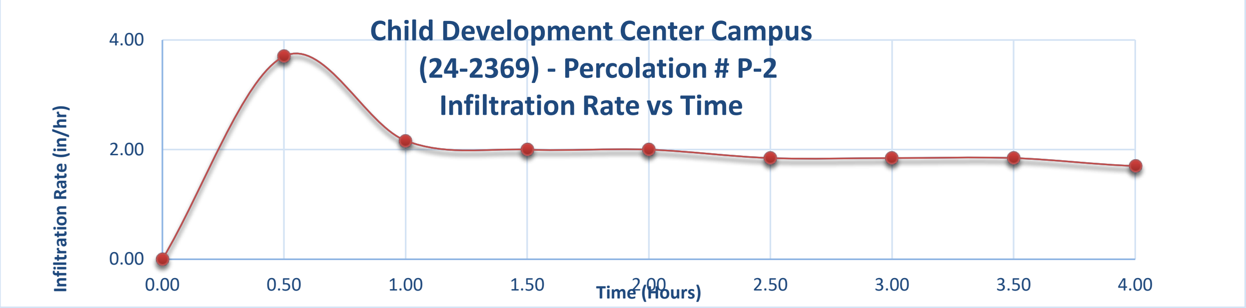
Reduction Factor for Boring Percolation Testing RFt, Site Variability, Number of Tests & Borings RFv, and Long Term Maintenance RFs

Short Term Infiltration Rate (in/hr) = 1.7

Long Term Infiltration Rate (in/hr) = 0.4

Reference: Administrative Manual - Los Angeles County Public works, Geotechnical & Materials Engineering Division, GS200.1 dated 06/30/21


Child Development Center Campus
(24-2369) - Percolation # P-2
Accumulated Volume vs Time
Accumulated Volume (Gallons)
16.00
14.00
12.00
10.00
8.00
6.00
4.00
2.00
0.00
0.00
0.50
1.00
1.50
2.00
2.50
3.00
3.50
4.00
Time (Hours)

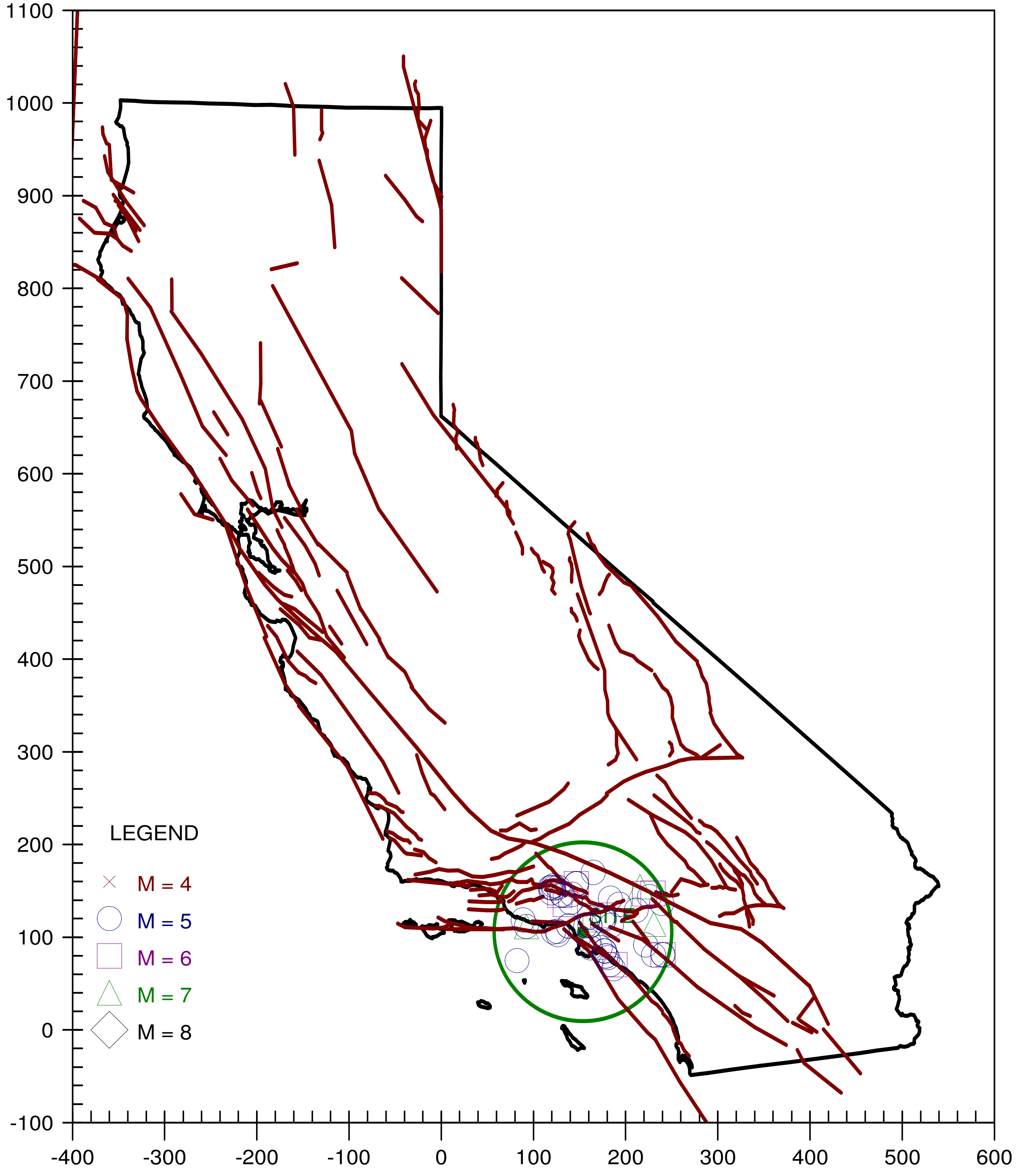

Child Development Center Campus
(24-2369) - Percolation # P-2
Infiltration Rate vs Time
Infiltration Rate (in/hr)
4.00
2.00
0.00
0.00
0.50
1.00
1.50
2.00
2.50
3.00
3.50
4.00
Time (Hours)

APPENDIX D

Historical Earthquake Data

EARTHQUAKE EPICENTER MAP

Child Care Development Center



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

                                 ESTIMATION OF
                            PEAK ACCELERATION FROM
                        CALIFORNIA EARTHQUAKE CATALOGS


JOB NUMBER: 24-2369
                                                     DATE: 06-06-2024

JOB NAME: Child Care Development Center

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
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                            -------------------------
                            EARTHQUAKE SEARCH RESULTS
                            -------------------------

Page  1
-------------------------------------------------------------------------------
    |       |        |          |  TIME  |     |     | SITE  |SITE|  APPROX.
FILE|  LAT. |  LONG. |   DATE   |  (UTC) |DEPTH|QUAKE|  ACC. | MM |  DISTANCE
CODE| NORTH |  WEST  |          | H M Sec| (km)| MAG.|   g   |INT.|  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)
```

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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
                              -------------------------

Page  2
------------------------------------------------------------------------------
    |       |        |          |  TIME  |     |     | SITE  |SITE|  APPROX.
FILE|  LAT. |  LONG. |   DATE   |  (UTC) |DEPTH|QUAKE|  ACC. | MM |  DISTANCE
CODE| NORTH |  WEST  |          | H M Sec| (km)| MAG.|   g   |INT.|  mi   [km]
----+-------+--------+----------+--------+-----+-----+-------+----+------------
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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


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

  Earthquake | Number of Times | Cumulative
   Magnitude |    Exceeded     | 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
```

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FAX 909-606-6555

BRANCH OFFICE

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

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