

**LYNWOOD UNIFIED SCHOOL DISTRICT
PROCUREMENT DEPARTMENT
11321 BULLIS AVENUE
LYNWOOD, CA 90262**

**BID NO. 10MOT26
FOR**

**BID NO. 10MOT26 Installation of Playground & Shade Structure at Will Rogers
Elementary School**

**ADDENDUM NO. 1
DATE: 3/30/2026**

This Addendum forms a part of the Contract Documents and modifies the original Bidding Documents. Acknowledge receipt of this Addendum in space provided on the Bid Form. Failure to acknowledge may subject Bidder to disqualification.

1. Technical Specifications Section:

1A) ADD: GPI – Geotech Report Dated June 10th 2025 (Total 26 pages attached)

1B) ADD: Underground Utility Job Summary Report (Total 11 pages)

2. Bid Document Section:

2A) Add – DSA Bulletin 24-05 – Contractor to provide site safety plan at the time Notice to Award. (Total page 1)

June 10, 2025

Lynwood Unified School District
11321 Bullis Road
Lynwood, California 90262

Attention: Dr. Gretchen Janson
Assistant Superintendent of Business Services

Subject: Geotechnical Investigation Report
Proposed Playground Canopy Shade Structure
Will Rogers Elementary School
11220 Duncan Avenue, Lynwood, California
GPI Project No. 3319.I

Dear Dr. Janson:

In accordance with our proposal dated May 7, 2025, this letter report provides our geotechnical recommendations for the proposed playground canopy shade structure at the subject school site. Geotechnical Professionals Inc. (GPI) conducted this geotechnical investigation in accordance with Purchase Order PO1-250000003628, and our Agreement for On-Going Architectural/Engineering Services dated September 6, 2023.

PROJECT DESCRIPTION

We understand Lynwood Unified School District plans to install one 20 feet by 30 feet HIP shade canopy as part of the planned improvements to the playground at the subject site. The canopy structure is anticipated to be supported on 5.75 feet square spread footings extending approximately 3 feet below grade or 2.5 feet diameter drilled piers extending approximately 8 to 9 feet below existing grade. Other associated improvements include playground equipment and concrete pedestrian hardscape.

Significant cuts and fills for grade changes are not anticipated. The site location is shown on the Site Location Map, Figure 1. The proposed canopy locations are shown on the Site Plan (Proposed), Figure 2. The current site conditions, including our recent and prior exploration locations, are shown on the Site Plan (Existing), Figure 3.

Our scope of work was performed in compliance with the California Geologic Survey (CGS) Note 48 for geologic seismic hazard requirements and the 2022 California Building Code (CBC).

BACKGROUND

GPI previously performed geotechnical investigations for the existing solar power shade/carport canopies at the school site and for other playground pavement improvements at the campus. GPI issued our findings and recommendations for these projects in our reports dated February 1, 2013 (GPI, 2013a) and May 16, 2013 (GPI, 2013b). The locations of our explorations performed as part of the referenced investigations are shown in Figure 3. The explorations and laboratory tests from our prior investigations at the site are available in the referenced reports.

The site is located within a State of California designated Seismic Hazard Zone for Liquefaction and Historical High Groundwater is reported to be 8 feet below ground surface. Our prior study at the site concluded that there is a potential for liquefaction to occur at the site.

The California Division of the State Architect (DSA) provides their Geohazard Report Requirements for the 2022 CBC in DSA IR A-4 (revised May 2, 2025). Section 3.4 of DSA IR A-4 states that fabric shade structures in a mapped geological hazard zone do not require a geohazard report if the shade structures is an *“Individual fabric shade structures 1,600 SF or less of regular shape (e.g., HIP, flat, gable, pyramid) supported on all corners (three minimum) with maximum column grid (bay) spacing of 40 feet.”*

Although the subject site is located within a geologic hazard zone for liquefaction as mapped by the State, a geohazard report is not required for the planned playground canopy because the planned fabric canopy is less than 1600 SF in size and will be supported on the four corners.

SCOPE OF WORK

Our scope of work included a review of prior investigations, field investigation, laboratory testing, engineering analysis, and preparation of this report.

Our field investigation consisted of two exploratory hand auger borings (Borings B-1 and B-2) advanced to depths of approximately 5 to 16 feet below the existing grades. Descriptions of the field procedures and logs of the borings are presented in Appendix A. A copy of the log of CPT 1 from our prior study (GPI, 2013a) is included in Appendix A. Logs of other prior borings and CPTs at the site are available in the referenced reports (GPI, 2013a and 2013b).

Our laboratory testing program included evaluations of in-place moisture content, fines content, shear strength (direct shear), consolidation, and corrosivity. Laboratory test procedures and results are presented in Appendix B. Soil corrosivity testing was performed by Project X Corrosion Engineers (Project X) under subcontract to GPI. Their test results are presented at the end of Appendix B.

Engineering evaluations were performed to provide geotechnical recommendations for foundation design and construction, earthwork criteria, and support of hardscape. The results of our evaluations are presented in the remainder of this report.

SITE CONDITIONS

The existing school campus is located at the southeast corner of Duncan Avenue and Beechwood Avenue in Lynwood, California. The proposed canopy and associated improvements are located within the Early Childhood Playground just east of Building D as shown in Figure 2.

Surface Conditions

Site topography of the subject school and surrounding area is relatively flat, with a slight downgrade to the south. Ground surface elevations at the subject playground, based on internet sources (i.e., Google Earth), range from approximately +84 to +83 feet above mean sea level.

The existing asphalt pavement section at Borings B-1 and B-2 consisted of approximately 5 and 4 inches of asphalt concrete pavement over subgrade soils.

Subsurface Conditions

Our current field investigations disclosed a subsurface profile consisting of shallow undocumented fill soils over natural soils. Detailed descriptions of the conditions encountered in our current explorations are shown on the Log of Borings in Appendix A.

Undocumented fill soils were encountered to approximate depths of 5 feet in the borings. Boring B-2 was terminated at approximately 5 feet due to encountering a subsurface obstruction. An alternate location, approximately 3 feet to the northwest, also encountered an obstruction at approximately 5 feet below grade and the boring was terminated due to refusal.

The fill soils at the boring locations consisted of moist to very moist sandy silt and clayey sand. Documentation on the placement and compaction of the fill was not available. The fill materials appeared to be moderately well compacted.

The underlying natural materials consisted of moist to very moist alternating layers of clayey silt, silty sand, sandy silt, and silty clay to the depth explored, approximately 16 feet below existing grade. The natural soils encountered are similar to those encountered in prior explorations by GPI on the school campus. Prior explorations encountered natural soils that consisted of medium dense silty sands and stiff to very stiff silts and clays within the 50-foot depth explored (GPI, 2013a). The upper natural soils exhibit moderate strength and compressibility characteristics.

Groundwater and Caving

Data published by the State of California indicates that historical high groundwater depth in the site vicinity is approximately 8 feet below existing grades. Ground water was not encountered in our current explorations. Caving was not encountered in our relatively small diameter explorations.

CONCLUSIONS AND RECOMMENDATIONS

OVERVIEW

Based on the results of our evaluation, it is our opinion that from a geotechnical viewpoint it is feasible to develop the site as proposed. The most significant geotechnical issues that will affect the design and construction of the improvements are as follows:

- The proposed shade structure may be supported on shallow spread foundations or drilled pier foundations, as planned. Shallow spread foundations should be underlain by properly compacted fill following overexcavation and recompaction of existing fill or extended into the underlying natural soils. Drilled pier foundations should extend through the existing fill into the native soils.
- The subject school site is located in a Seismic Hazard Zone for liquefaction as mapped by the State of California (CGS, South Gate Quadrangle). Based on our analyses, we computed potential total seismic-induced liquefaction settlement of approximately 3 inches, with differential seismic settlement across a 40-foot span of approximately 1 inch or less. As such, the potential for liquefaction settlements to negatively impact the proposed canopy structures is considered to be low.
- A subsurface obstruction was encountered at a depth of 5 feet below existing grade at Boring B-2. The extent of this subsurface obstruction should be evaluated and foundations should be located and designed not to surcharge the obstruction.
- The near-surface soils (within 5 feet of ground surface) are generally moist to very moist, with moisture contents ranging from approximately 11 to 17 percent. Optimum moisture contents for the site soils are anticipated to be about 11 to 14 percent. Mixing and some moisture conditioning will be required to achieve suitable moisture contents where earthwork is planned. If wet soils are encountered, subgrade stabilization may be required prior to placing fill.
- Corrosivity testing performed by Project X on a sample provided from one of our current borings indicates negligible levels of soluble sulfate with respect to concrete. The soils are also considered to be corrosive to buried ferrous metals.

Our recommendations related to the geotechnical aspects of the development of the site are presented in the subsequent sections of this report.

SEISMIC DESIGN

General

We assume the seismic design of the proposed development, where required, will be in accordance with the 2022 CBC. For the 2022 CBC, Site Class D may be used. The seismic code values can be obtained directly from the tables in the building

code using the above Site Class value and pertinent internet websites (<https://seismicmaps.org/>). The Project Structural Engineer should determine the seismic design method. As mapped, the ASCE 7-16 seismic code values are as follows:

$$\begin{array}{lll} S_S = 1.694g & S_{MS} = F_a * S_S = 1.694g & S_{DS} = 2/3 * S_{MS} = 1.129g \\ S_1 = 0.605g & S_{M1} = F_v * S_1 = 1.029g & S_{D1} = 2/3 * S_{M1} = 0.689g \end{array}$$

Due to the limited nature of the proposed improvements and in accordance with our approved scope, we did not perform a site-specific seismic response evaluation.

Strong Ground Motion Potential

Based on published information, the most significant faults in the proximity of the site are the Puente Hills Fault (Los Angeles and Santa Fe Springs Segments) and the Newport Inglewood Fault, which are located about 2.5 miles and 5.1 miles from the subject site, respectively.

During the life of the project, the site will likely be subject to strong ground motions due to earthquakes on nearby faults. Based on the OSHPD website (<https://seismicmaps.org/>), we computed that the site could be subjected to a peak ground acceleration (PGA_M) of 0.80g for a magnitude 7.3 earthquake (Newport Inglewood Fault).

This acceleration has been computed using the mapped Maximum Considered Geometric Mean peak ground acceleration from ASCE 7-16 (ASCE, 2017) and a site coefficient (F_{PGA}) based on Site Class. The predominant earthquake magnitude was determined using a 2-percent probability of exceedance in a 50-year period, or an average return period of 2,475 years. The structural design will need to incorporate measures to mitigate the effects of strong ground motion.

Potential for Ground Rupture

There are no known active faults crossing or projecting through the site. The site is not located in an Alquist-Priolo Earthquake Fault Zone. Therefore, ground rupture due to faulting is considered unlikely at this site. In addition, the proposed improvements are not considered to be habitable and are therefore not subject to the code requirements for site-specific seismic hazard evaluations.

Liquefaction and Seismic Settlement

Liquefaction is a phenomenon in which saturated cohesionless soils undergo a temporary loss of strength during severe ground shaking and acquire a degree of mobility sufficient to permit ground deformation. In extreme cases, the soil particles can become suspended in groundwater, resulting in the soil deposit becoming mobile and fluid-like. Liquefaction is generally considered to occur primarily in loose to medium dense deposits of saturated sandy soils. Thus, three conditions are required for liquefaction to occur: (1) a sandy soil of loose to medium density; (2) saturated conditions; and (3) rapid, large strain, cyclic loading, normally provided by earthquake motions.

According to the published State Seismic Hazard Zones map for the South Gate Quadrangle, the site is located in an area designated by the State Geologist as a "zone of required

investigation" due to the potential for earthquake-induced liquefaction.

Historical high groundwater levels provided by the California Geologic Survey indicate a shallowest historical groundwater table of approximately 8 feet below existing grades (CGS, 1998). We used a groundwater depth of 8 feet in evaluating the potential seismic-induced liquefaction settlement at the subject site.

In accordance with the 2022 CBC, ASEC 7-16, Special Publication 117A (CGS, 2008), and the 2022 revision of CGS Note 48, the ground motion used for this evaluation should be based on the Peak Ground Acceleration (PGA_M), adjusted for Site Class effects, or a site-specific response spectrum. As such, we considered a ground acceleration of 0.80g for a magnitude 7.3 earthquake for our analyses, which was determined based on the mapped values previously detailed.

Based on our analyses, we estimated a potential total and differential seismic-induced liquefaction settlement of approximately 3 inches and 1 inch or less across a 40-foot span, respectively. As such, the potential for liquefaction settlements to negatively impact the proposed canopy structures is considered to be low.

FOUNDATIONS

We understand the proposed canopy shade structure will be supported on 5.75 feet square spread footings extending approximately 3 feet below grade or 2.5 feet diameter drilled piers extending approximately 8 to 9 feet below existing grade. Both are suitable foundation alternatives and should be designed utilizing the geotechnical parameters provided in the following sections.

Shallow spread foundations should be underlain by the native soils or properly compacted fill following overexcavation and recompaction of existing fill. Drilled pier foundations should extend through the existing fill into the native soils.

Spread Footings

The proposed structures may be supported on conventional isolated shallow footings, provided the subsurface soils are prepared in accordance with the recommendations given in this report. Spread footings should be supported on properly compacted fill soils or the native soils.

Bearing Capacity

A static allowable net bearing pressure of up to 2,000 pounds per square foot (psf) may be used for both isolated column footings underlain by compacted fill or the native soils. The actual bearing pressure used may be less, such that economics and structural loads will determine the minimum width for footings as discussed below. The recommended bearing pressures are for dead load-plus-live loads, and may be increased one-third for short-term, transient, wind and seismic loading. The maximum edge pressures induced by eccentric loading or overturning moments should not be allowed to exceed these recommended values.

The following minimum footing widths and embedments are recommended for the corresponding allowable bearing pressure. A minimum footing width of 15 inches and depth of 18 inches should be used even if the actual bearing pressure is less than 1,500 psf.

STATIC BEARING PRESSURE (psf)	MINIMUM FOOTING WIDTH (inches)	MINIMUM FOOTING* EMBEDMENT (inches)
2,000	24	18
1,500	15	18

* Depth to bottom of footing below lowest adjacent finish grade.

Settlement

Under the lightly loaded static conditions typical of minor structures, maximum total settlement is expected to be on the order of ½-inch or less. Maximum differential settlement between similarly loaded adjacent footings is estimated to be on the order of ¼-inch. The above settlement estimates are based on the assumption that the recommended earthwork will be performed and that the footings will be sized in accordance with our recommendations.

Lateral Resistance

Soil resistance to lateral loads will be provided by a combination of frictional resistance between the bottom of footings and underlying soils and by passive soil pressures acting against the embedded sides of the footings. For frictional resistance, a coefficient of friction of 0.30 may be used for design. In addition, an allowable lateral bearing pressure equal to an equivalent fluid weight of 250 pounds per cubic foot may be used, provided the footings are poured tight against the compacted fill or native soils. A one-third increase in the above allowable lateral bearing pressure (but not the frictional resistance) may be taken for short-term wind and seismic loads. The passive pressure provided also assumes a level ground surface extending to a horizontal distance from the wall or footing face at least twice the depth of embedment (defined above). These values may be used in combination without reduction.

Foundation Excavation Observation

Prior to placement of steel and concrete, a representative of GPI should observe and approve foundation excavations. Footing excavations should be moistened immediately prior to concrete placement.

Drilled Pier Foundations

Axial and Lateral Capacity

The proposed shade structure and similar pole-type structures may be supported on drilled pier foundations. The design of such piers is typically governed by uplift and lateral loading conditions. Soil resistance to lateral loads can be provided by the piers. For design by the simplified pole formula presented in Section 1807A.3.2.1 of the 2022 CBC, a unit passive resistance of 250 pounds per square foot per foot (to a maximum of 2,500 pounds per square foot) may be used for the piers with level ground under static loading conditions. Based on our site-specific analyses, we recommend a unit passive resistance of 500 pounds per square

foot per foot (to a maximum of 5,000 pounds per square foot) be used for isolated piers that can tolerate $\frac{1}{2}$ -inch of deflection under short-term loads. We recommend that the upper 1-foot of the subgrade soils be ignored in determining the required depth of embedment to allow for surface disturbance adjacent to the pier.

A pier designed for adequate embedment to resist the anticipated lateral loads will likely have adequate axial capacity to support the anticipated vertical loads. The net allowable vertical compressive capacity can be conservatively calculated based on a unit side friction of 300 pounds per square foot, neglecting end-bearing contribution. For piers installed in areas where undocumented fills were removed in their entirety and replaced with properly compacted, we recommend that the upper 1 foot of the subgrade soils be ignored in determining the required depth of embedment to allow for future surface disturbance adjacent to the pier. For piers installed in areas where undocumented fills remain in place, we recommend the upper 2 feet of the subgrade soils be ignored in determining required depth of embedment for allowable axial capacity.

In determining the allowable uplift capacity for a given pier, we recommend using a unit side friction of 150 pounds per square foot, which is one-half of the compressive capacity previously provided.

Drilled pier foundations will develop their frictional capacity with relatively small deflection (about $\frac{1}{4}$ -inch). The Project Structural Engineer should consider the anticipated static settlement, the previously provided total and different seismic-induced liquefaction settlements, and the requirements of ASCE 7-16 with respect to foundation design.

Drilled Pier Installation and Observation

Pier excavations should be backfilled with concrete promptly after the drilling is completed and not be left open overnight. Drilled pier excavations may encounter some caving, particularly in the sandy deposits. Adjacent piers within 6 diameters on center should be drilled and filled alternately. The potential for caving increases the longer the excavation is left open.

Concrete mix designs should include provisions to minimize shrinkage, which can lead to lower frictional resistance of the pier shaft and reduced allowable capacity. The concrete should be placed with special equipment so that it is not allowed to fall freely more than 5 feet or strike the walls of the excavations. Drilling for piers should not be performed within 5 feet of recently excavated or recently poured piers until the concrete has been allowed to set for at least 6 hours. The piers should be poured in a manner that will not result in concrete flowing into adjacent drilled pier excavations and prevent segregation of aggregate. Drilled pier construction should be performed in accordance with the latest edition of ACI 336.1, "Standard Specifications for the Construction of Drilled Piles."

If caving soil conditions are encountered, casing, slurry, or other methods should be utilized to maintain the pier excavation. Since the drilled piers will be designed to derive resistance from friction only, rigorous cleaning of loose material from the bottom of the excavation prior to placement of steel and concrete is not considered essential. Effort should be made to clean the bottom with the drill rig-mounted equipment.

A representative of GPI should observe pier excavations to confirm and document the depth,

diameter, and embedment in suitable materials. Downhole inspection of drilled pier excavations (by lowering an inspector into the excavation) is not needed if the piers are designed as frictional elements.

EARTHWORK

The earthwork at the project site is anticipated to consist of clearing, excavations, subgrade preparation, and placement and compaction of fill.

Clearing

Prior to grading, performing excavations, or constructing the proposed improvements, the areas to be developed should be cleared of debris and pavements. Buried obstructions, such as footings, abandoned utilities, and tree roots should be removed from areas to be developed. Deleterious material generated during the clearing operation, including organic topsoil, should be removed from the site.

Excavations

Excavations for the planned canopy shade structure will likely be limited to construction of the drilled pier or spread foundations. Additional excavations may include trenching for new utility lines, if planned. Based on the scope of the project, significant cuts and fills for grade changes are not anticipated.

If structures are planned to be supported on shallow spread foundations, the existing undocumented fill should be removed and replaced as properly compacted fill or the footing excavation should extend into the native soils. For planning purposes, removals should extend to a depth of at least 5 feet below existing grades. For minor structures such as light poles or if the shade structure is supported on shallow drilled piers, removals are not required.

For new pavements and hardscape, removals are not anticipated to be required with the exception of areas previously occupied by structures or landscape areas. In these areas, the removals should be performed to a depth of 1 foot below the existing or proposed finished subgrade, whichever is deeper.

Deeper removals than those detailed above may be required where unsuitable subgrade soils are encountered. The actual depths of removals should be determined in the field during grading by a representative of GPI. Existing grades refer to the grades at our exploration locations.

If the corners of the spread foundations are accurately staked by Project Surveyor, the base of overexcavation for shallow foundations does not need to extend beyond the edge of the foundation. Alternatively, the excavation should extend beyond the edge of the foundation to a minimum distance equal to the depth of overexcavation/compaction below the foundations (i.e., a 1:1 projection below the bottom outside edge of footings).

In general, the upper site soils are not considered to be susceptible to caving in shallow excavations. Temporary construction excavations may be made vertically without shoring to a depth of 4 feet below the adjacent grade. For deeper cuts up to 10 feet, the slopes should be properly shored or sloped back to at least 1:1 or flatter. The exposed slope face should be

kept moist (but not saturated) during construction to reduce local sloughing. Some raveling of the sandy deposits should be anticipated at the slope inclinations recommended, especially where dry and more granular soils are exposed.

Surcharge loads should not be permitted within a horizontal distance equal to the height of cut from the top of the excavation or 5 feet from the top of the slopes, whichever is greater, unless the cut is properly shored. Excavations that extend below an imaginary plane inclined at 45 degrees below the edge of the adjacent existing site facilities should be properly shored to maintain support of adjacent elements. Excavations and shoring systems should meet the minimum requirements given in Article 6 of the State of California Occupational Safety and Health Standards.

In general, the excavation can be accomplished by conventional soil excavation equipment such as backhoes, loaders, or dozers.

Subgrade Preparation

Prior to placing fills or construction of new pavements or hardscape, the subgrade soils should be scarified to a depth of 8 inches, moisture conditioned as necessary and compacted to at least 90 percent of the maximum dry density in accordance with ASTM D1557.

Subgrade processing may be omitted where the soils exposed in the subgrade are evaluated by a representative of GPI to have moisture contents significantly in excess of the optimum moisture content. The earthwork subcontractors should review the moisture content information presented in Appendix A.

We anticipate challenges if the earthwork is performed in wet-weather seasons because of the potential need for soil drying prior to compaction. The moisture contents of the near-surface soils encountered in our explorations were, on average, at or above optimum. As such, some moisture conditioning of the soils is anticipated.

If wet soils are encountered, yielding soils can be removed to expose a stable subgrade or subgrade stabilization will likely be required to support compaction equipment. For planning purposes, the stabilization would require the placement of 12 inches of crushed aggregate base (CAB) over a geogrid, such as Tensar BX1100. A thicker section of CAB could be used if the geogrid is omitted.

Material for Fill

In general, soils available from on-site excavations, less debris or organic matter, will be suitable for use in fills. Soils placed within 1-foot of the finished subgrade in concrete pedestrian hardscape areas should be non-expansive (E.I. of 20 or less). Such materials appear to be available onsite within the upper 2 feet below existing grades but may require selective grading. Some moisture conditioning and mixing will likely be required prior to re-using the in-place soils.

Imported fill material, if required, should be predominantly granular (containing no more than 40 percent fines - portion passing No. 200 sieve) and non-expansive (E.I. of 20 or less).

Import or on-site materials used in compacted fills should not contain particles larger than

3 inches in diameter. GPI should be provided with a sample (at least 50 pounds) and notified of the location of soils proposed for import at least 72 hours in advance of importing.

Each proposed import source should be sampled, tested, and accepted for use prior to delivery of the soils to the site. Soils imported prior to acceptance by GPI may be rejected if not suitable. Lynwood Unified School District environmental requirements for import material should also be followed.

Placement and Compaction of Fills

Fill soils should be placed in horizontal lifts, moisture-conditioned, and mechanically compacted to densities equal to at least 90 percent of the maximum dry density, determined in accordance with ASTM D1557. Granular soils within 1 foot of the finished subgrade in asphalt concrete areas and aggregate base materials should be compacted to at least 95 percent of the maximum dry density in accordance with ASTM D1557. The optimum lift thickness will depend on the compaction equipment used and can best be determined in the field. The following uncompacted lift thickness can be used as preliminary guidelines.

Plate compactors	4-6 inches
Small vibratory or static rollers (5-ton±) or track equipment	6-8 inches

The maximum lift thickness should not be greater than 8 inches and each lift should be thoroughly compacted and accepted prior to subsequent lifts.

Fills should be placed at a moisture content of 1 to 3 percent over the optimum moisture content in order to achieve the required compaction. The moisture content of the near-surface soils encountered in the explorations was, on average, at and above optimum. As such, moisture conditioning and mixing will likely be required prior to replacing the soils as properly compacted fill. The soils should not be allowed to dry out prior to covering or additional processing and moisture conditioning may be required.

During backfill of excavations, the fill should be properly benched into the construction slopes as it is placed in lifts.

Trench/Wall Backfill

Utility trench and wall backfill, consisting of the on-site materials or imported sand, should be mechanically compacted in lifts. Lift thickness should not exceed those values given in the "Compacted Fill" section of this report. Jetting or flooding of backfill materials should not be permitted. GPI should observe and test trench and wall backfills as they are placed.

In backfill areas where mechanical compaction of soil backfill is impractical due to space constraints, sand-cement slurry may be substituted for compacted backfill. The slurry should contain 1½ sacks of cement per cubic yard and have a maximum slump of 5 inches. When set, such a mix typically has the consistency of compacted soil.

Observation and Testing

A representative of GPI should observe excavations, subgrade preparation, and fill placement activities. Sufficient in-place field density tests should be performed during fill placement and in-place compaction to evaluate the overall compaction of the soils. Soils that do not meet

minimum compaction requirements should be reworked and tested prior to placement of additional fill.

CONCRETE SLABS AND EXTERIOR HARDSCAPE

Concrete hardscape should be supported on non-expansive, compacted soils as discussed in the Earthwork section. Although not tested, we anticipate the on-site clayey soils generally encountered below depths of 2 feet to have a low to moderate potential for expansion. We recommend that the on-site clays not be placed as properly compacted fill within 1-foot of the finished subgrade in concrete hardscape areas. The on-site soils should not be allowed to dry out or additional processing and moisture conditioning will be required. We recommend hardscape and PCC pavement be reinforced with a minimum of No. 3 rebar at 24 inches on-center in both directions.

CORROSION

Laboratory testing by Project X indicated a soluble sulfate content of 46.6 mg/kg (0.00466 percent by weight). For the 2022 CBC, foundation concrete should conform to the requirements outlined by ACI 318, Section 19.3 for a negligible level of soluble sulfate exposure for soil (Category S0). The chloride content of the recent sample tested was 47.3 mg/kg (0.00473 percent by weight), which is considered to be low (Category C1).

Resistivity testing of a representative sample of the on-site soils in this area indicates that they are corrosive to buried ferrous metals.

The results of the chemical testing are presented in Appendix B. GPI does not practice corrosion protections engineering. We recommend that a corrosion-engineering firm, such as Project X, be consulted if additional corrosion protection recommendations are required.

DRAINAGE

Positive surface gradients should be provided adjacent to structures and the tops of descending slopes to direct surface water run-off and roof drainage away from foundations and slopes toward suitable discharge facilities. Long-term ponding of surface water should not be allowed.

GEOTECHNICAL OBSERVATION AND TESTING

We recommend that a representative of GPI observe the earthwork during construction to confirm that the recommendations provided in our report are applicable during construction.

The earthwork activities include grading, compaction of fills, subgrade preparation, pavement construction and foundation excavations. If conditions are different than expected, we should be afforded the opportunity to provide an alternate recommendation based on the actual conditions encountered.

LIMITATIONS

This report, exploration logs, and other materials resulting from GPI's efforts were prepared exclusively for use by the Lynwood Unified School District and their consultants in designing the proposed development. The report is not intended to be suitable for reuse on extensions or modifications of the project or for use on projects other than the currently proposed development as it may not contain sufficient or appropriate information for such uses.

Soil deposits may vary in type, strength, and many other important properties between points of exploration due to non-uniformity of the geologic formations or to man-made cut and fill operations. While we cannot evaluate the consistency of the properties of materials in areas not explored, the conclusions drawn in this report are based on the assumption that the data obtained by others in the field and laboratory are reasonably representative of field conditions and are conducive to interpolation and extrapolation.

Furthermore, our recommendations were developed with the assumption that a proper level of field observation and construction review will be provided by GPI during grading, excavation, and foundation construction. If construction-phase services are performed by others, they must accept full responsibility for the geotechnical aspects of the project, including this report.

Our investigation and evaluations were performed using generally accepted engineering approaches and principles available at this time and the degree of care and skill ordinarily exercised under similar circumstances by reputable geotechnical engineers practicing in this area. No other representation, either express or implied, is included or intended in our report.

Respectfully submitted,
Geotechnical Professionals Inc.



Justin J. Kempton, G.E.
Principal
(justink@gpi-ca.com)



Cc: Eric Morales, Lynwood Unified School District
Susan Treviso, Lynwood Unified School District

Attachments: References
Appendix A Explorations
Appendix B Laboratory Tests

LIST OF FIGURES

FIGURE NO.

- | | |
|---|----------------------|
| 1 | Site Location Map |
| 2 | Site Plan (Proposed) |
| 3 | Site Plan (Existing) |

APPENDIX A

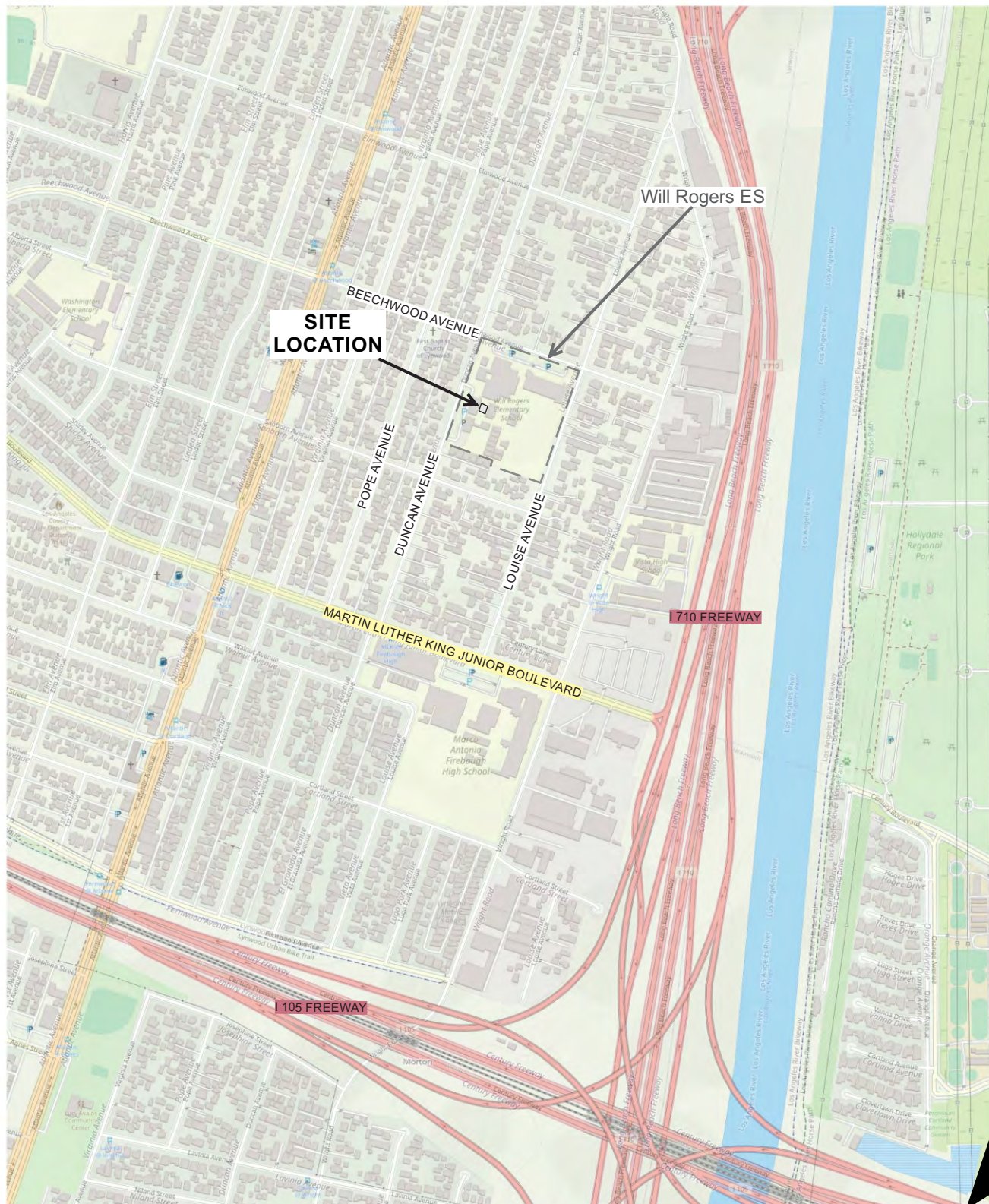
- | | |
|------------|---|
| A-1 to A-2 | Logs of Borings
Log of CPT 1 from (GPI, 2013a) |
|------------|---|

APPENDIX B

- | | |
|-----|--|
| B-1 | Direct Shear Test Results |
| B-2 | Consolidation Test Results
Corrosion Test Results (Project X) |

REFERENCES

- American Society of Civil Engineers (2017), "Minimum Design Loads and Associated Criteria for Buildings and Other Structures," ASCE/SEI 7-16.
- American Society of Civil Engineers (2018), "Minimum Design Loads and Associated Criteria for Buildings and Other Structures, Supplement 1" ASCE/SEI 7-16, effective December 12, 2018.
- American Society of Civil Engineers (2018), "Minimum Design Loads and Associated Criteria for Buildings and Other Structures, Supplement 3" ASCE/SEI 7-16, effective November 5, 2021.
- Bray, J.D. and Sancio, (2006), "*Assessment of the Liquefaction Susceptibility of Fine-Grained Soils*," Journal of Geotechnical and Geoenvironmental Engineering, ASCE, September 2006, pp. 1165-1177.
- California Geological Survey (1998), Seismic Hazard Zone Report for the South Gate 7.5-Minute Quadrangle, Los Angeles County, California," Seismic Hazard Zone Report 034.
- California Geological Survey (1999), Seismic Hazard Zones Map of the South Gate Quadrangle, Map Published March 25, 1999.
- California Office of Statewide Health Planning and Development (OSHPD), Seismic Design Maps Website, <https://seismicmaps.org/>
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0 800 1600 FEET

BASE MAP REPRODUCED FROM CALTOPO © 2025



GEOTECHNICAL
PROFESSIONALS, INC.

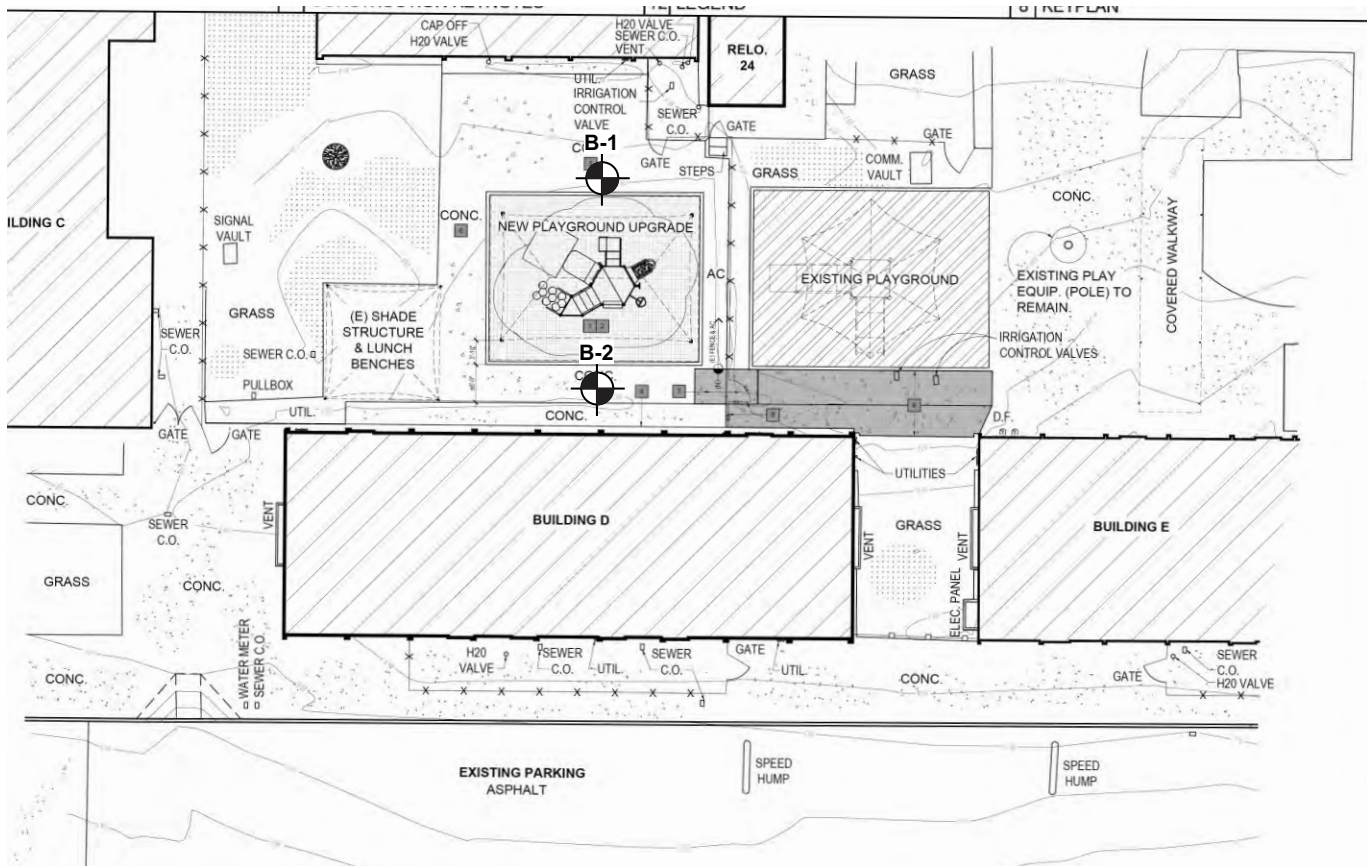
WILL ROGERS ES - PLAYGROUND UPGRADE

GPI PROJECT NO.: 3319.I

SCALE: 1" = 800'

SITE LOCATION MAP

FIGURE 1



EXPLANATION



APPROXIMATE LOCATION AND
NUMBER OF EXPLORATORY BORING



BASE PLAN REPRODUCED FROM ENLARGED CONSTRUCTION
SITE PLAN (A102) PROVIDED BY MSP ARCHITECTS : 3/12/2025



GEOTECHNICAL
PROFESSIONALS, INC.

WILL ROGERS ES - PLAYGROUND UPGRADE

GPI PROPOSAL NO.: 3319.I

SCALE: 1" = 30'

**SITE PLAN
(PROPOSED)**

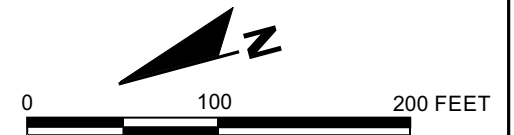
FIGURE 2



EXPLANATION

-  APPROXIMATE LOCATION AND NUMBER OF EXPLORATORY BORING
-  PRIOR GPI BORING AND CPT - 2526.01I (2013)
-  PRIOR GPI BORING - 2577.I (2013)

 PROPOSED CANOPY STRUCTURE



BASE PLAN REPRODUCED FROM GOOGLE EARTH © 2025



WILL ROGERS ES - PLAYGROUND UPGRADE

GPI PROPOSAL NO.: 3319.I

SCALE: 1" = 100'

**SITE PLAN
(EXISTING)**

FIGURE 3

APPENDIX A

EXPLORATIONS

Borings

We investigated the subsurface conditions at the site of the currently proposed canopy structure project by drilling and sampling two exploratory borings, Borings B-1 and B-2. The borings were advanced to depths of approximately 5 and 16 feet below the existing ground surface. The locations of the explorations are shown in Figures 2 and 3.

The borings were drilled using hand auger equipment. The hand auger equipment has an inside diameter of 3 inches. Relatively undisturbed samples were obtained using a brass-ring lined sampler (ASTM D3550). The brass-rings have an inside diameter of 2.42 inches. The ring sample was driven into the soil by a weighted hammer (about 45 pounds) and by hand. The number of blows needed to drive the sampler into the soils a depth of 12 inches was recorded as the penetration resistance and are presented on the hand auger logs. The blow counts were based on variable relative energies due to the sampling method and provide relative comparisons of densities between samples. Disturbed bulk samples were also obtained.

The field explorations for the investigation were performed under the continuous technical supervision of GPI's representative, who visually inspected the site, maintained detailed logs of the borings, classified the soils encountered, and obtained relatively undisturbed samples for examination and laboratory testing. The soils encountered in the borings were classified in the field and through further examination in the laboratory in accordance with the Unified Soils Classification System. Detailed logs of our recent borings are presented in Figures A-1 and A-2 in this appendix.

The boring locations were laid out in the field by measuring from existing site features. Ground surface elevations at the exploration locations were estimated from internet sources (i.e., Google Earth) and should be considered approximate.

Prior Explorations

The locations of prior borings and cone penetrations tests (CPTs) performed by GPI at the school site are shown on Figure 3. The log of CPT 1, which was advanced to a depth of 50 feet, from our prior study (GPI, 2013a) is included in this appendix. Logs of other prior borings and CPTs at the site are available in the referenced reports (GPI, 2013a and 2013b).

PROBE (in)	MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	DESCRIPTION OF SUBSURFACE MATERIALS	ELEVATION (FEET)	
						This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.		
					0	5-Inch AC		
	15.3	106	25	B		Fill: SANDY SILT (ML) grey brown, moist, trace gravel	80	
				D		CLAYEY SAND (SC) brown, very moist to wet, trace gravel		
	17.4	84	20	D	5	Natural: CLAYEY SILT (ML) brown, very moist		
						SILTY SAND (SM) grey, moist to very moist	75	
	17.7	96	23	D	10	SILTY CLAY (CL) grey, moist		
						CLAYEY SILT (ML) grey brown, very moist, trace sand		
	11.2	91	25	D	15	SANDY SILT (ML) grey brown, moist	70	
						Total Depth 15.5 feet		

SAMPLE TYPES

- ☒ C Rock Core
☒ S Standard Split Spoon
☒ D Drive Sample
☒ B Bulk Sample
☒ T Tube Sample

DATE DRILLED:

5-19-25

EQUIPMENT USED:

8" Coring Machine & Hand Auger

GROUNDWATER LEVEL (ft):

Not Encountered



PROJECT NO.: 3319.I

WILL ROGERS ES

LOG OF BORING NO. B-1

FIGURE A-1

					0	4-Inch AC		80
	10.9	114	33	D		Fill: SANDY SILT (ML) grey brown, moist, trace gravel		
						CLAYEY SAND (SC) orange brown, moist to very moist, with gravel		
					5	@ 3 feet, moist to very moist		
						CLAY (CL) brown, moist, with sand and gravel		
						SILT (ML) brown, moist, with sand and gravel		
						Refusal at 5 feet due to concrete		
						Refusal encountered for second time 3 feet to the north west at 5 feet of depth.		

SAMPLE TYPES

- ☒ C Rock Core
☒ S Standard Split Spoon
☒ D Drive Sample
☒ B Bulk Sample
☒ T Tube Sample

DATE DRILLED:

5-19-25

EQUIPMENT USED:

8" Coring Machine & Hand Auger

GROUNDWATER LEVEL (ft):

Not Encountered

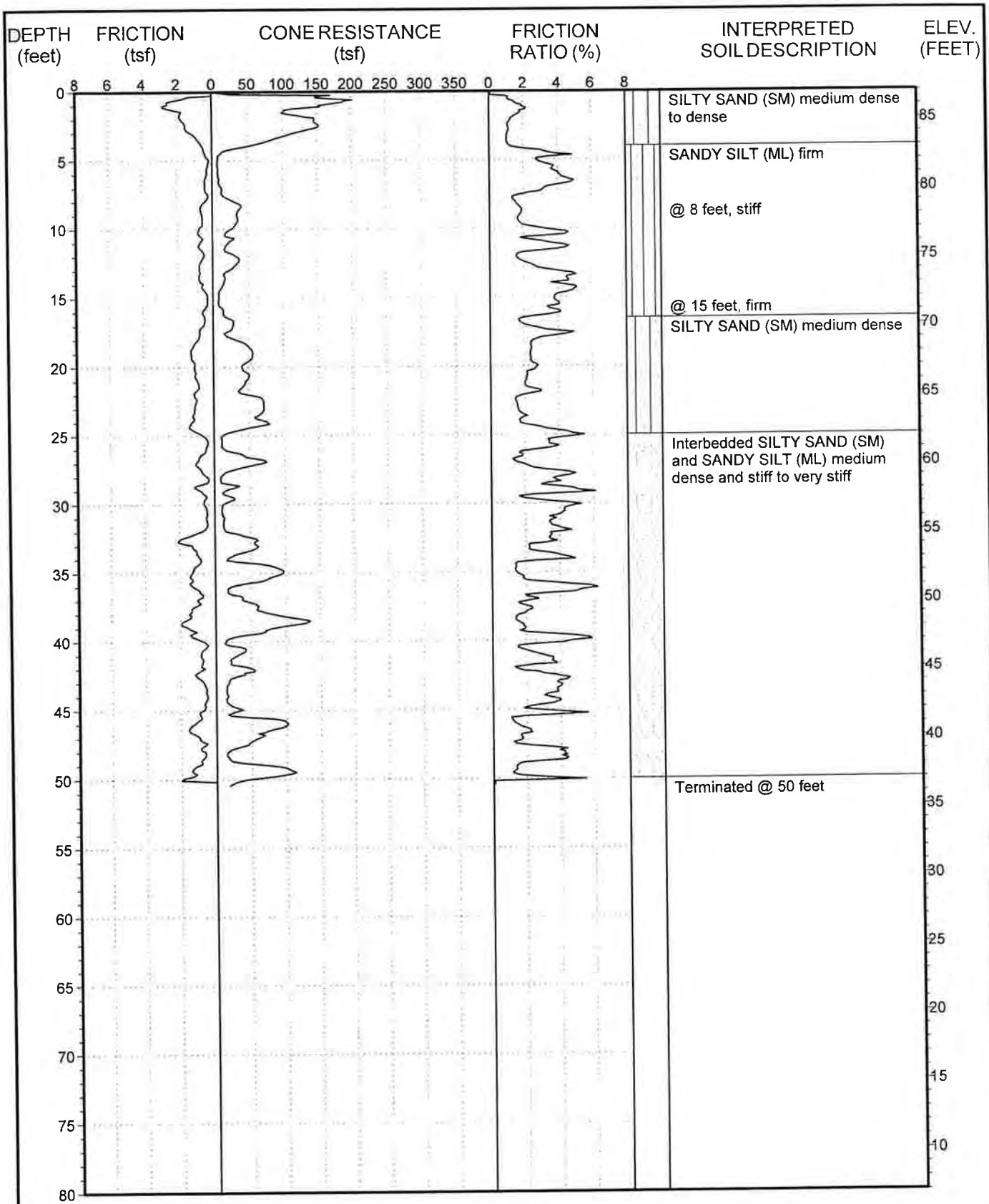


PROJECT NO.: 3319.I

WILL ROGERS ES

LOG OF BORING NO. B-2

FIGURE A-2



Date performed: 1-15-13

This summary applies only at the location of this cone penetration test and at the time of the exploration. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The interpreted soil description is derived from the friction ratio and cone resistance and is a simplification of actual conditions encountered.



PROJECT NO.: 2526.011
WILL ROGERS ES - LYNWOOD

LOG OF CPT NO. C-1

FIGURE A-2

APPENDIX B

LABORATORY TESTS

INTRODUCTION

Representative undisturbed soil samples and bulk samples were carefully packaged in the field and sealed to prevent moisture loss. The samples were then transported to our Cypress office for examination and testing assignments. Laboratory tests were performed on selected representative samples as an aid in classifying the soils and to evaluate the physical properties of the soils affecting foundation design and construction procedures.

Detailed descriptions of the laboratory tests are presented below under the appropriate test headings. Test results are presented in the figures that follow.

MOISTURE CONTENT AND DRY DENSITY

Moisture content and dry density were determined from a number of the samples. The samples were weighed to determine wet weight and then dried in accordance with ASTM D2216. After drying, the weight of each sample was measured, and moisture content was calculated. Moisture content and density values are presented on the logs in Appendix A.

GRAIN SIZE DISTRIBUTION

A soil sample was dried, weighed, soaked in water until individual soil particles were separated, and then washed on the No. 200 sieve. That portion of the material retained on the No. 200 sieve was oven-dried and weighed to determine the percentage of the material passing the No. 200 sieve. The percentages passing the No. 200 sieve is presented below.

BORING NO.	DEPTH (ft)	SOIL DESCRIPTION	PERCENT PASSING No. 200 SIEVE
B-2	3	Clayey Sand (SC)	29

DIRECT SHEAR

Direct shear tests were performed on selected undisturbed samples in accordance with ASTM D3080. The samples were placed in the shear machine, and pre-selected normal loads were applied. The samples were submerged, allowed to consolidate, and then sheared to failure. Shear stress and sample deformation were monitored throughout the test. The results of the direct shear test are presented in Figure B-1.

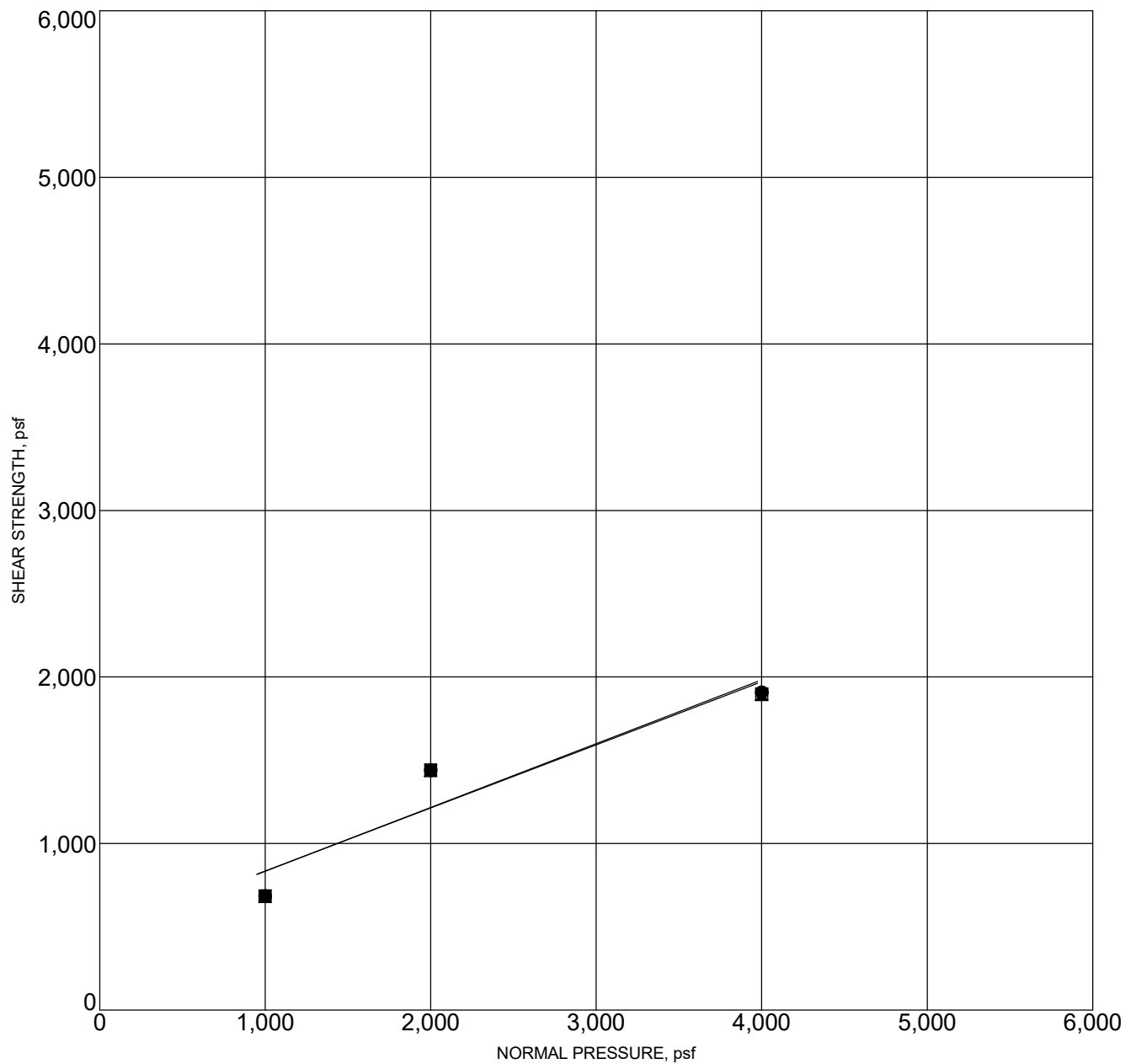
CONSOLIDATION

One-dimensional consolidation testing was performed on an undisturbed sample in accordance with ASTM D2435. After trimming the ends, the sample was placed in the consolidometer and loaded to 0.4 ksf. Thereafter, the sample was incrementally loaded to a maximum load of 6.4 ksf. The sample was inundated at 1.6 ksf. Sample deformation was

measured to 0.0001 inch. Rebound behavior was investigated by unloading the sample back to 0.4 ksf. Results of the consolidation test, in the form of percent consolidation versus log pressure, are presented in Figure B-2.

CORROSIVITY

Soil corrosivity testing was performed by Project X on a selected soil sample provided by GPI. The test results are summarized at the end of this Appendix.



Sample Location		Classification	DD,pcf	MC,%
B-1	5.0	CLAYEY SILT (ML)	84	17.4

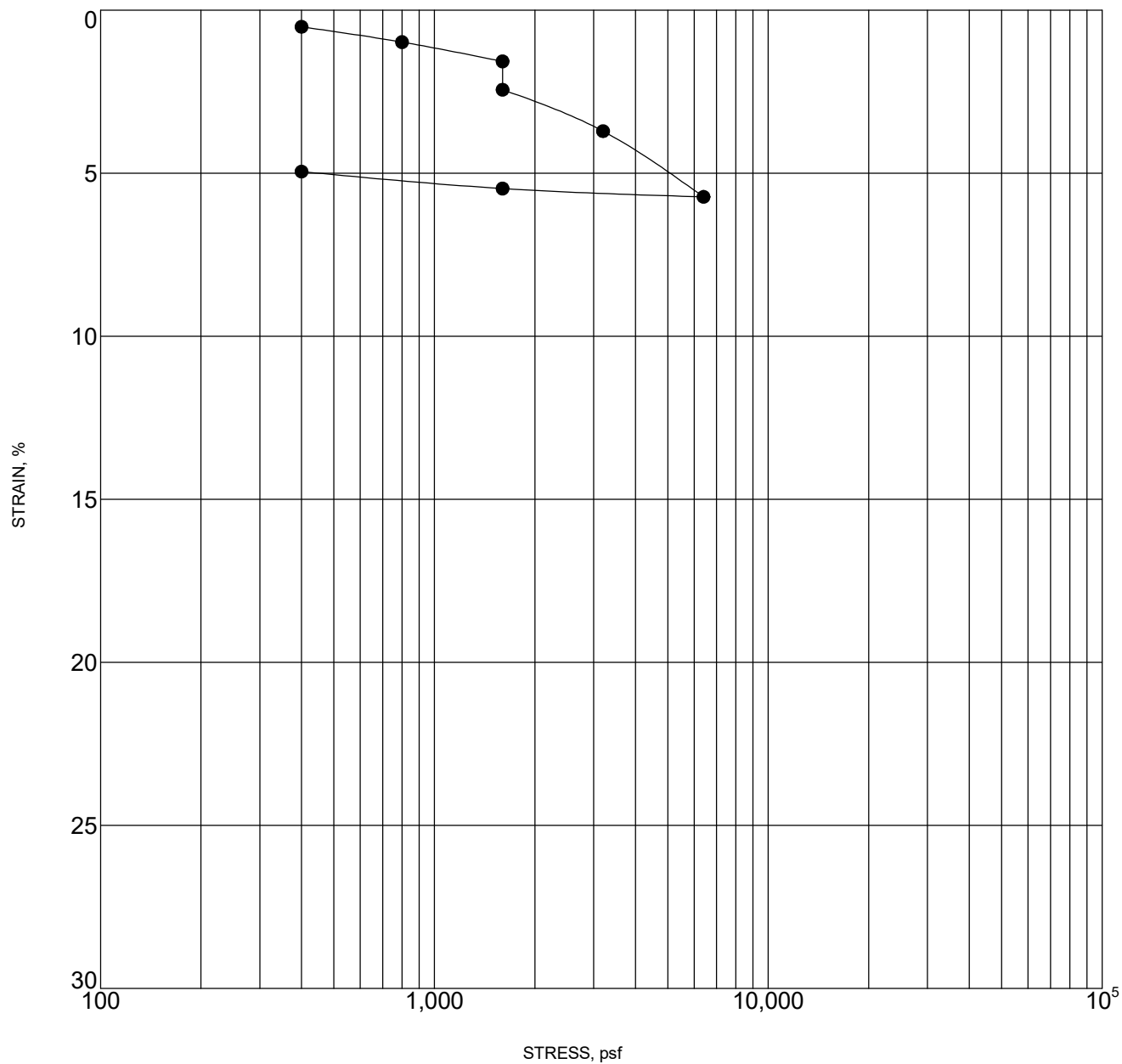
PROJECT: WILL ROGERS ES

PROJECT NO.: 3319.I



DIRECT SHEAR TEST RESULTS

FIGURE B-1



Sample inundated at 1600 psf

Sample Location			Classification	DD,pcf	MC,%
●	B-1	5.0	CLAYEY SILT (ML)	84	17.4

PROJECT: WILL ROGERS ES

PROJECT NO.: 3319.I



CONSOLIDATION TEST RESULTS

FIGURE B-2



Soil Analysis Lab Results

Client: Geotechnical Professionals Inc.

Job Name: Will Rogers ES

Client Job Number: 3319.I

Project X Job Number: S250523D

May 27, 2025

	Method	ASTM D4327	ASTM D4327	ASTM G187	ASTM G51
Bore# / Description	Depth	Sulfates SO ₄ ²⁻	Chlorides Cl ⁻	Resistivity As Rec'd Minimum	pH
	(ft)	(mg/kg)	(wt%)	(mg/kg)	(wt%)
B-1	3.0	46.6	0.0047	47.3	0.0047
				(Ω-cm)	(Ω-cm)
				1,608	1,541
					7.2

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

For AWWA 105C: 0-3mg/kg sulfide = Negative; 3-6mg/kg = trace; >6mg/kg = Positive

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

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

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



JOB SUMMARY REPORT

Order Number:	Work Order #795248	Job Date:	Jun 18, 2025 4:52:00 PM
Customer:	154954 ARCHITECTS MCDONALD SOUTAR & PAZ INC	Billing Address:	ARCHITECTS MCDONALD SOUTAR & PAZ INC 11321 Bullis Road Lynwood CA 90262 United States

JOB DETAILS

Jobsite Location	11220 Duncan Avenue, Lynwood, California 90262
Work Order Number	Work Order #795248
Job Number	
PO Number	PO1-250000003724

GPRS Project Manager: Jorge Colin

Thank you for using GPRS on your project. We appreciate the opportunity to work with you. If you have questions regarding the results of this scanning, please contact the lead GPRS project manager on this project.

EQUIPMENT USED

The following equipment was used on this project:

- **Underground GPR Antenna:** This GPR Antenna uses frequencies ranging from 250 MHz to 450 MHz and is mounted in a stroller frame that rolls over the surface. Data is displayed on a screen and marked in the field in real time. The surface needs to be reasonably smooth and unobstructed to obtain readable scans. Obstructions such as curbs, landscaping, and vegetation will limit the efficacy of GPR. The total effective scan depth can be as much as 8' or more with this antenna but can vary widely depending on the soil conditions and composition. Some soil types, such as clay, may limit maximum depths to 3' or less. As depth increases, targets must be larger to be detected, and non-metallic targets can be challenging to locate. The depths provided should always be treated as estimates as their accuracy can be affected by multiple factors. For more information, please visit: [Link](#)
- **EM Pipe Locator:** Electromagnetic Pipe and Cable Locator. Detects electromagnetic fields. Used to actively trace conductive pipes and tracer wires, or passively detect power and radio signals traveling along conductive pipes and utilities. For more information, please visit: [Link](#)
- **GPS:** This handheld unit offers accuracy down to 4 inches; however, the accuracy achieved will depend on the satellite environment at the time of collection and is not considered survey-grade. Features can be collected as points, lines, or areas and then exported as a KML/KMZ or overlaid on a CAD drawing. For more information, please visit: [Link](#)
- **Traceable Rodder:** The rodder has a copper wire encased in fiberglass. The device is pushed through accessible pipes before placing a current on the wire, and the signal is then traced from the surface. The maximum traceable depth is 10' depending on the soil conditions, and the maximum distance is 200'. The line can be pushed through a pipe with direct access, such as a sewer line at a cleanout or a storm drain catch basin. It may not be able to be pushed through deeper pipes within manholes. GPRS will not access electrical conduits. The signal cannot be located through metallic pipes.



JOB SUMMARY REPORT

WORK PERFORMED

UNDERGROUND UTILITY

Client Provided Drawings	No
Client completed 811 locate request	No
Scope of Work	GPRS conducted a utility locate over an area measuring approximately 2,240 square feet in preparation for the customer's upcoming playground upgrade.
Approximate GPR Effective Depth (ft)	1.5
Utilities Located	- Electric - Communication - Natural Gas - Water - Sanitary Sewer - Irrigation - Unknown
Utilities NOT Located	- Water - Storm Sewer - Sanitary Sewer - Unknown
Details on Non-locatable Utilities	GPRS was not able to rod 2 sewer features located within the scope. There was no visible stone features within the scope. Water features gave weak signal. GPRS was not able to get a trace on a conduit. There was a lot of random anomalies at approximately 1-2 ft but gpra was not able to trace them. No main power picked up in the area.
Limitations Encountered	- Soil conditions not suitable for GPR at time of scanning - RF interference - Other
Limitations Encountered - Other	Limited utility features within scope
Obstructions Encountered	No access into comm/ electric boxes
Marking Medium	- Spray Paint
Results Notes	Findings and their respective Colors: Red: Electrical Lines Blue: Water/Irrigation Orange: Communication Lines Pink: Unknown Lines Yellow: Gas Line Green: Sewer Line Utilities with no trace ,limited trace ,or no features within Scope: Sanitary Lines: limited trace , unable to rod two sewer features Gas Lines: limited trace , possible gas marked out Storm lines: no features visible within scope Water Line: weak signal



JOB SUMMARY REPORT

Methodology:

The scan was conducted utilizing the following methods:

- Direct connections
- Conduit hooping
- traceable rodder
- Passive Sweeps with EM Locator
- GPR grid patterned scans

Precautions:

In light of the identified utilities, it is crucial for the client to know the following limitations associated with locating and steps to take in order to perform safe excavations. Please keep in mind there are some items that may not carry an em signal and are not conductive, such as: plastic pipes, clay pipes, asbestos pipes...etc. If GPRS cannot see these items with GPR and they could not carry a EM signal these items are undetectable. Gprs further advises pot holing all markings, avoiding markings by at least 2' on all sides, using any available as builds, the use of GPRS findings and utilizing 811 to further confirm findings and any other utilities that may not have been detectable.

Further Advice: For a more thorough investigation, it is always advised to have the scan area free of any obstructions prior to getting a scan completed.

It is essential to take all necessary precautions before proceeding with any excavation or alteration to prevent the risks of damage or safety hazards.

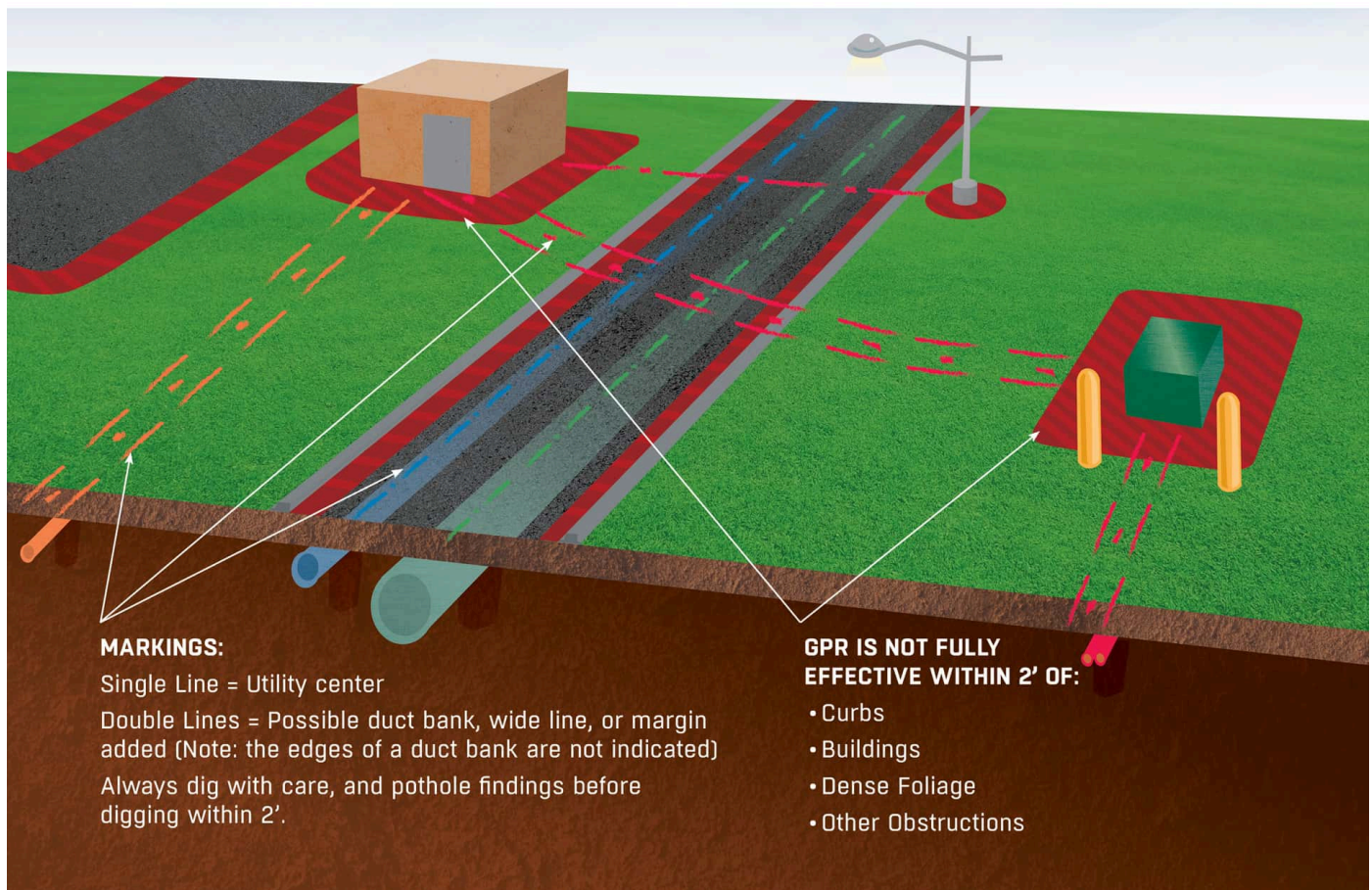


JOB SUMMARY REPORT

SUPPLEMENTAL INFORMATION

COMMON UTILITY LOCATING LIMITATIONS

There are many limitations to locating utilities, due to a variety of factors, with several more common examples illustrated here.





JOB SUMMARY REPORT

JOB SITE IMAGES



Jobsite Photo #1



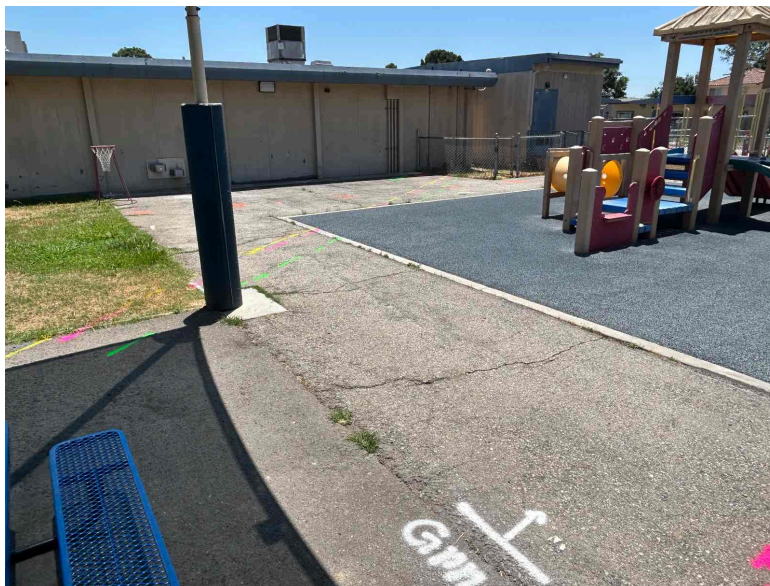
Jobsite Photo #2



JOB SUMMARY REPORT



Jobsite Photo #3



Jobsite Photo #4



JOB SUMMARY REPORT



Jobsite Photo #5



Jobsite Photo #6



JOB SUMMARY REPORT



Jobsite Photo #7



Jobsite Photo #8



JOB SUMMARY REPORT



Jobsite Photo #9



Jobsite Photo #10



JOB SUMMARY REPORT



Jobsite Photo #11



Jobsite Photo #12



JOB SUMMARY REPORT

CONTACT / SIGNATURE INFORMATION

Contact Information

Contact Name	Lieng Sun	Email	lieng.sun@architectsmsp.com
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TERMS & CONDITIONS

<http://www.gprsinc.com/termsandconditions.html>

BULLETIN: FIRE SAFETY DURING CONSTRUCTION AND DEMOLITION

Division of the State Architect (DSA) documents referenced within this publication are available on the [DSA Forms](#) or [DSA Publications](#) webpages.

PURPOSE

A DSA bulletin is a notification to its stakeholders regarding any issue intended to be directed to a broad group of external stakeholders as well as DSA staff. This bulletin provides guidance on the development of, and requirement for, a **site safety plan (SSP)** for demolition projects and or the construction of new buildings, additions or alterations to existing buildings, reconstruction projects, repair, and nonconforming building rehabilitation projects submitted for approval under the 2022 CBC and later editions.

BACKGROUND

California Education Code (EDC) Sections 17280 and 81130 direct the Department of General Services (DGS) to supervise the design and construction any school building to ensure compliance with the standards published in Title 24, California Code of Regulations (CCR).

1. REQUIREMENT

1.1 Section 3302.3 of the California Building Code (CBC) directs compliance with Chapter 33 of the California Fire Code (CFC). Chapter 33 of the CFC outlines requirements for site safety during construction and demolition and directs that the property owner or their authorized agent is responsible for the development and implementation of a written site safety plan establishing a fire prevention program at the project site.

1.2 The school district, in collaboration with the project contractor is responsible for developing an SSP addressing the requirements of CFC Chapter 33. As first responders, DSA recommends that development of the SSP be in consultation and coordination with the local fire authority.

1.3 Beginning February 1, 2025, a completed site safety plan (SSP) is to be submitted to DSA in conjunction with the initial filing of form *DSA 102-IC: Construction Start Notice/Inspection Card Request*, inclusive of completing Section 3 of the form. Failure to include the SSP will result in rejection of the DSA form 102-IC.

1.3.1 Stockpile projects and similar factory-built construction projects do not require a site safety plan be submitted. For projects that include the construction and placement of these factory-built buildings on a site, the site safety plan needs not accompany the DSA 102 IC form when only seeking in-plant inspection cards.

1.4 The SSP is to be a single pdf (separate from the initial DSA 102-IC) with a file name formatted as Origin ID-App #_Site Safety Plan_submission date (i.e., YY-MM-DD).

1.5 DSA will receive and file the fire safety plan in the project file.

2. SITE SAFETY PLAN COMPONENTS

At a minimum, the SSP shall include the following information required in CFC Section 3303.1.1.

1. Name and contact information of site safety director.

BULLETIN: FIRE SAFETY DURING CONSTRUCTION AND DEMOLITION

2. Documentation of training of the site safety director and fire watch personnel.
3. Procedures for reporting emergencies.
4. Fire department vehicle access routes.
5. Locations of fire protection equipment, including portable fire extinguishers, standpipes, fire department connections and fire hydrants.
6. Smoking and cooking policies, designated area to be used where approved, and signage locations in accordance with CFC Section 3305.8.
7. Location and safety considerations for temporary heating equipment.
8. Hot work (welding, roofing, etc.) plan.
9. Plans for control of combustible waste.
10. Locations and methods for storage and use of flammable and combustible liquids and other hazardous materials.
11. Provisions for site security.
12. Changes that affect this plan.
13. Other site- specific information requested by the local fire authority (LFA).

3. PROJECT PLAN SET COORDINATION

Because the SSP is submitted after project approval, project plans shall include the following statement in the Notes block, "All construction and demolition shall be in accordance with Chapter 33 of the CBC and CFC, and the written site safety plan."

REFERENCES:

California Code of Regulations (CCR) Title 24
Part 1: California Administrative Code (CAC), Sections 4-331 and 4-406
Part 2, California Building Code, Chapter 33
Part 9: California Fire Code (CFC), Chapter 33