# APPENDIX E1

# GEOTECHNICAL REPORT

# GEOTECHNICAL INVESTIGATION
# PROPOSED ALLEN INDUSTRIAL FACILITY
309 WEST ALLEN AVENUE
SAN DIMAS, CALIFORNIA

-Prepared By-

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Project No. 444-21106
22-01-001

CEG Construction, Inc.
7901 Crossway Drive
Pico Rivera, California 90660-4449

Subject: Geotechnical Investigation

Project: Proposed Allen Industrial Facility
309 West Allen Avenue
San Dimas, California

Sladden Engineering is pleased to present the results of our geotechnical investigation performed for the new industrial/warehouse building proposed for the subject property located at309 West Allen Avenue in the City of San Dimas, California. Our services were completed in accordance with our proposal for geotechnical engineering services dated September 28, 2021 and your authorization to proceed with the work. The purpose of our investigation was to explore the subsurface conditions at the site to provide recommendations for foundation design and site preparation. Evaluation of environmental issues and hazardous wastes was not included within the scope of services provided.

The opinions, recommendations and design criteria presented in this report are based on our field exploration program, laboratory testing and engineering analyses. Based on the results of our investigation, it is our professional opinion that the proposed project is feasible from a geotechnical perspective provided that the recommendations presented in this report are implemented into design and carried out through construction.

We appreciate the opportunity to provide service to you on this project. If you have any questions regarding this report, please contact the undersigned.

Respectfully submitted,
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GEOTECHNICAL INVESTIGATION
PROPOSED ALLEN INDUSTRIAL FACILITY
309 WEST ALLEN AVENUE
SAN DIMAS, CALIFORNIA

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## INTRODUCTION

This report presents the results of the geotechnical investigation performed for the new Allen Industrial facility proposed for the subject property located 309 West Allen Avenue in the City of San Dimas, California. The site is located at approximately 34.1181 degrees North latitude and -117.8120 degrees West longitude. The approximate location of the site is indicated on the Site Location Map (Figure 1).

Our investigation was conducted in order to evaluate the engineering properties of the subsurface materials, to evaluate their *in-situ* characteristics, and to provide engineering recommendations and design criteria for site preparation, foundation design and the design of various site improvements. This study also includes a review of published and unpublished geotechnical and geological literature regarding seismicity at and near the subject site.

## PROJECT DESCRIPTION

Based on the provided site plan (O.C. Design & Engineering, 2021), it is our understanding that the proposed project will consist of constructing a 2-unit industrial/warehouse building on the subject property. Unit-1 occupies a total of 26,193 square feet (sf) and consists of warehouse and mezzanine level office space. Unit-2 occupies a total of 37,556 sf and consists of warehouse and mezzanine level office space. The project also includes paved parking areas and truck loading docks. In addition, we expect that the project will include exterior concrete flatwork, underground utilities, landscape areas and various other improvements. For our analyses, we expect that the proposed industrial buildings will be of reinforced concrete tilt-up construction supported on conventional shallow spread footings and concrete slabs-on-grade.

We anticipate that grading will be limited to minor cuts and fills to accomplish the desired pad elevations and provide adequate gradients for site drainage. This does not include the removal and re-compaction of foundation bearing soil within the building envelope. Upon completion of precise grading plans, Sladden should be retained in order to ensure that the recommendations presented within in this report are incorporated into the design of the proposed project.

Structural foundation loads were not available at the time of this report. Based on our experience with relatively lightweight concrete tilt-up structures, we expect that isolated column loads will be less than 80 kips and continuous wall loads will be less than 5.0 kips per linear foot. If these assumed loads vary significantly from the actual loads, we should be consulted to verify the applicability of the recommendations provided.

## SCOPE OF SERVICES

The purpose of our investigation was to determine specific engineering characteristics of the surface and near surface soil in order to develop foundation design criteria and recommendations for site preparation. Specifically, our site characterization consisted of the following tasks:

- Site reconnaissance to assess the existing surface conditions on and adjacent to the site.
- The excavation of six (6) exploratory boreholes to practical auger refusal depths between approximately 4 and 17 feet bgs in order to characterize the subsurface soil conditions. Representative samples of the soil were classified in the field and retained for laboratory testing and engineering analyses.
- The performance of laboratory testing on selected samples to evaluate their engineering characteristics.
- The review of geologic literature with respect to potential geologic hazards.
- The performance of engineering analyses to develop recommendations for foundation design and site preparation.
- The preparation of this report summarizing our work at the site.

## SITE CONDITIONS

The site is located 309 West Allen Avenue in the City of San Dimas, California. The property occupies approximately 2.58 acres. At the time of our investigation, the property was occupied by multiple single-family residences. The project site is relatively level with minimal surface gradients and near the elevation of the adjacent properties and roadways. The property is bounded on the north and west by commercial/industrial facilities, on the east by North Cataract Avenue and on the south by West Allen Avenue.

According to the USGS 7.5′ San Dimas Quadrangle map (2012) and Google Earth (2022), the site is at an approximate elevation of 965 feet above mean sea level (MSL).

No ponding water or surface seeps were observed at or near the site during our site investigation conducted on December 1, 2022.. Site drainage appears to be controlled via sheet flow, surface infiltration and city maintained stormwater drains.

## GEOLOGIC SETTING

The project site is located in the northern portion of the Peninsular Ranges Physiographic Province of California. The Peninsular Ranges are mountainous areas that extend from the western edge of the continental borderland to the Salton Trough and from the Transverse Ranges Physiographic Province in the north to the tip of Baja California in the south. The province is characterized by elongated, northwest-southeast trending mountain ranges and valleys and is truncated at its northern margin by the east-west grain of the Transverse Ranges.

The site has been mapped by Rogers (1965) to be immediately underlain by alluvium (Qal). The geologic setting for the site and site vicinity is presented on the Regional Geologic Map (Figure 2).

## SUBSURFACE CONDITIONS

The subsurface conditions at the site were investigated by drilling six (6) exploratory boreholes to practical refusal depths between approximately 4 and 17 feet bgs in order to evaluate the subsurface soil conditions. The approximate locations of the boreholes are illustrated on the Borehole Location Photograph (Figure 3). The boreholes were advanced using a truck-mounted Mobile B-61 drill-rig equipped with 8-inch outside diameter (O.D.) hollow stem augers. A representative of Sladden was on-site to log the materials encountered and retrieve samples for laboratory testing and engineering analysis.

During our field investigation, artificial fill soil was encountered to a depth of approximately two (2) feet bgs. The artificial fill soil consisted of yellowish brown, fine-to coarse-grained, dry silty sand (SM) with scattered gravel and cobbles. Underlying the artificial fill soil and extending to maximum depths explored, native alluvial sediment was encountered. The native soil consists primarily of gravelly sand (SP/SW) and silty sand (SM). Generally, the native earth materials appeared yellowish brown in in-situ color, dry and fine- to coarse-grained with scattered cobbles and boulders. The presence of boulders and cobbles resulted in generally shallow practical auger refusal depths within our bores.

Groundwater was not encountered at the maximum explored depth of 17 feet bgs during our field investigation. Based upon the depth to groundwater in the project vicinity, it is our opinion that groundwater should not be a factor in the design or construction of the proposed project.

## SEISMICITY AND FAULTING

The southwestern United States is a tectonically active and structurally complex region, dominated by northwest trending dextral faults. The faults of the region are often part of complex fault systems, composed of numerous subparallel faults that splay or step from the main fault traces. Strong seismic shaking could be produced by any of these faults during the design life of the proposed project.

We consider the most significant geologic hazard to the project to be the potential for moderate to strong seismic shaking that is likely to occur during the design life of the project. The proposed project is located in the highly seismic Southern California region within the influence of several fault systems that are considered to be active or potentially active. An active fault is defined by the State of California as a "sufficiently active and well defined fault" that has exhibited surface displacement within the Holocene epoch (about the last 11,000 years). A potentially active fault is defined by the State as a fault with a history of movement within Pleistocene time (between 11,000 and 1.6 million years ago).

Table 1 lists the closest known potentially active faults that was generated in part using the EZ FRISK computer program (Fugro, 2020), as modified using the fault parameters from The Revised 2002 California Probabilistic Seismic Hazard Maps (Cao et al, 2003) and the Fault and Fold Database of the United States (USGS, 2006). This table does not identify the probability of reactivation or the on-site effects from earthquakes occurring on any faults in the region.

**TABLE 1**
**CLOSEST KNOWN ACTIVE FAULTS**

| Fault Name | Distance (Km) | Maximum Event |
|---|---|---|
| Sierra Madre | < 2.0 | 7.2 |
| San Jose | 2.2 | 6.4 |
| Cucamonga | 7.3 | 6.9 |
| Chino-Central Avenue | 11.3 | 6.7 |
| Clamshell – Sawpit | 13.3 | 6.5 |
| Elysian Park Thrust | 16.4 | 6.7 |
| Raymond | 18.6 | 6.5 |
| Whittier | 20.9 | 6.8 |
| Verdugo | 27.2 | 6.9 |

## SITE SPECIFIC GROUND MOTION PARAMETERS

Sladden has reviewed the 2019 California Building Code (CBC) and ASCE7-16 and developed site specific ground motion parameters for the subject site.The project Seismic Design Maps and site-specific ground motion parameters are summarized in the following table and included within Appendix C. The project Structural Engineer should verify that all design parameters provided are applicable for the subject project.

## TABLE 2
## GROUND MOTION PARAMETERS

| | |
|---|---|
| Latitude / Longitude | 34.1181/ -117.8120 |
| Risk Category | II |
| Site Class | D |
| Seismic Design Category | D |
| Code Reference Documents | ASCE 7-16; Chapter 11 & 21 |

| **Description** | **Type** | **Map Based** | **Site-Specific** |
|---|---|---|---|
| MCE$_R$ Ground Motion (0.2 second period) | $S_S$ | 1.691 | --- |
| MCE$_R$ Ground Motion (1.0 second period) | $S_1$ | 0.637 | --- |
| Site-Modified Spectral Acceleration Value | $S_{MS}$ | 1.691 | **2.099** |
| Site-Modified Spectral Acceleration Value | $S_{M1}$ | null | **1.662** |
| Numeric Seismic Design Value at 0.2 second SA | $S_{DS}$ | 1.128 | **1.400** |
| Numeric Seismic Design Value at 1.0 second SA | $S_{D1}$ | null | **1.108** |
| Site Amplification Factor at 0.2 second | $F_a$ | 1.000 | **1.0** |
| Site Amplification Factor at 1.0 second | $F_v$ | null | **2.5** |
| Site Peak Ground Acceleration | $PGA_M$ | 0.792 | **0.777** |

## GEOLOGIC HAZARDS

The subject site is located in an active seismic zone and will likely experience strong seismic shaking during the design life of the proposed project. In general, the intensity of ground shaking will depend on several factors including; the distance to the earthquake focus, the earthquake magnitude, the response characteristics of the underlying materials, and the quality and type of construction. Geologic hazards and their relationship to the site are discussed below.

I. Surface Rupture. Surface rupture is expected to occur along preexisting, known active fault traces. However, surface rupture could potentially splay or step from known active faults or rupture along unidentified traces. Based on review of CGS (2022), Jennings (1994) and Dibblee (2002) known faults are not mapped on the site. In addition, no signs of active surface faulting were observed during our review of non-stereo digitized photographs of the site and site vicinity (Google, 2022). Finally, no signs of active surface rupture or secondary seismic effects (lateral spreading, lurching etc.) were identified on-site during our field investigation. Therefore, it is our opinion that risks associated with primary surface ground rupture should be considered "low".

II. Ground Shaking. The site has been subjected to past ground shaking by faults that traverse through the region. Strong seismic shaking from nearby active faults is expected to produce strong seismic shaking during the design life of the proposed project. A site-specific approach determined the peak ground acceleration (PGAm) at the site to be 0.777g.

III. Liquefaction. Liquefaction is the process in which loose, saturated granular soil loses strength as a result of cyclic loading. The strength loss is a result of a decrease in granular sand volume and a positive increase in pore pressures. Generally, liquefaction can occur if all of the following conditions apply; liquefaction-susceptible soil, groundwater within a depth of 50 feet or less, and strong seismic shaking.

The site is currently located within a designated liquefaction zone. Based on the anticipated depth to groundwater within the site vicinity and presence of dense sediment underlying the subject site, it is our opinion that risks from liquefaction hazards are considered "negligible".

IV. Tsunamis and Seiches. Because the site is situated at an elevated inland location and is not immediately adjacent to any impounded bodies of water, risk associated with tsunamis and seiches is considered "negligible".

V. Slope Failure, Landslides, Rock Falls. The site is situated on relatively level ground and is not immediately adjacent to any slopes or hillsides that could be potentially susceptible to slope instability. No signs of slope instability in the form of landslides, rock falls, earthflows or slumps were observed at or near the subject site during our investigation. As such, risks associated with slope instability should be considered "negligible".

VI. Expansive Soil. Expansion Index testing of select samples was performed in order to evaluate the expansive potential of the materials underlying the site. Based the results of our laboratory testing (EI = 1), the materials underlying the site are considered to have a "very low" expansion potential. Expansion potential should be revaluated after grading.

VII. Flooding and Erosion. No signs of flooding or erosion were observed during our field investigation. However, risks associated with flooding and erosion should be evaluated and mitigated by the project design Civil Engineer.

## CONCLUSIONS

Based on the results of our investigation, it is our professional opinion that the project should be feasible from a geotechnical perspective provided that the recommendations provided in this report are incorporated into design and carried out through construction. The main geotechnical concerns in the design and construction of the proposed project are the presence of existing building elements, and utilities, the presence of artificial fill soil and the potentially compressible condition of the near surface native.

Because of the presence of artificial fill soil and the somewhat soft and compressible condition of the near surface native soil, remedial grading including overexcavation and recompaction is recommended for the proposed building and foundation areas. We recommend that remedial grading within the proposed building areas include over-excavation and/or re-compaction of the artificial fill and primary foundation bearing soil. Specific recommendations for site preparation are presented in the Earthwork and Grading section of this report.

Groundwater was not encountered within a maximum explored depth of approximately 17 feet bgs during our field investigation. The presence of groundwater should not be a factor in design or during the construction of the proposed project.

Caving did occur to varying degrees within each of our exploratory bores and the surface soil may be susceptible to caving within deeper excavations. All excavations should be constructed in accordance with the normal CalOSHA excavation criteria. Based on our observations of the materials encountered, we anticipate that the subsoil will conform to that described by CalOSHA as Type B or C. Soil conditions should be verified in the field by a "Competent person" employed by the Contractor.

The following recommendations present more detailed design criteria that have been developed based on our field investigation and laboratory testing.

## EARTHWORK AND GRADING

All earthwork including excavation, backfill and preparation of the surface soil, should be performed in accordance with the geotechnical recommendations presented in this report and portions of the local regulatory requirements, as applicable. All earth work should be performed under the observation and testing of a qualified soil engineer. The following geotechnical engineering recommendations for the proposed project are based on observations from the field investigation program, laboratory testing and geotechnical engineering analyses.

a. Stripping. Areas to be graded should be cleared of the existing vegetation and trees, any previous structural or foundation elements, any remaining surface improvements and underground utilities. All areas scheduled to receive fill should be cleared of surface improvements, artificial fill and any unsuitable matter. The unsuitable materials should be removed off-site. Existing fill soil should be removed in its entirety and replaced as engineering fill soil. Voids left by obstructions should be properly backfilled in accordance with the compaction recommendations of this report.

b. Preparation of Building Areas. In order to achieve a firm and uniform bearing conditions, we recommend over-excavation and re-compaction throughout the building areas. All artificial fill soil and native low density near surface soil should be removed to a depth of at least 4 feet below existing grade or 3 feet below the bottom of the footings, whichever is deeper. Remedial grading should extend laterally a minimum of five feet beyond the building foundations. The soil exposed by over-excavation should be scarified, moisture conditioned to near optimum moisture content, and compacted to at least 90 percent relative compaction. Some additional stabilization efforts may be necessary where wet native soil is encountered.

c. Compaction. Soil to be used as engineered fill should be free of organic material, debris, and other deleterious substances, and should not contain irreducible matter greater than three inches in maximum dimension. All fill materials should be placed in thin lifts, not exceeding six inches in a loose condition at near optimum moisture content. Some drying of wet native soil encountered during over-excavation may be necessary prior to use as engineered fill material. If import fill is required, the material should be of a low to non-expansive nature and should meet the following criteria:

| | |
|---|---|
| Plastic Index | Less than 12 |
| Liquid Limit | Less than 35 |
| Percent Soil Passing #200 Sieve | Between 15% and 35% |
| Maximum Aggregate Size | 3 inches |

The subgrade soil and all fill material should be compacted with acceptable compaction equipment to at least 90 percent relative compaction. The bottom of the exposed subgrade should be observed by a representative of Sladden Engineering prior to fill placement. Compaction testing should be performed in order to verify proper compaction. Table 2 provides a summary of the excavation and compaction recommendations.

**TABLE 3**
**SUMMARY OF RECOMMENDATIONS**

| | |
|---|---|
| *Remedial Grading | Over-excavation and re-compaction within the building envelope and extending laterally for 5 feet beyond the building limits and to a minimum of 4 feet below existing grade or 3 feet below the bottom of the footings, whichever is deeper. |
| Native / Import Engineered Fill | Place in thin lifts not exceeding 6 inches in a loose condition, compact to a minimum of 90 percent relative compaction. |
| Asphalt Concrete Sections | Compact the top 12 inches to at least 95 percent compaction within 2 percent of optimum moisture content. |

*Actual depth may vary and should be determined by a representative of Sladden Engineering in the field during construction.

d. Shrinkage and Subsidence**.** Volumetric shrinkage of the material that is excavated and replaced as controlled compacted fill should be anticipated. We estimate that this shrinkage could vary from 10 to 15 percent. Subsidence of the surfaces that are scarified and compacted should be between 1 and 2 tenths of a foot. This will vary depending upon the type of equipment used, the moisture content of the soil at the time of grading and the actual degree of compaction attained.

## FOUNDATIONS: CONVENTIONAL SHALLOW SPREAD FOOTINGS

The proposed industrial building may be supported upon conventional shallow spread footings. Exterior footings should extend at least 18 inches beneath lowest adjacent grade and interior footings should extend at least 12 inches below slab subgrade. Isolated square or rectangular footings at least 2 feet square and continuous footings at least 12 inches wide may be designed using allowable bearing pressures of 2000 and 1800 pounds per square foot, respectively. The allowable bearing pressure may be increased by approximately 250 psf for each additional 1 foot of width and 250 psf for each additional 6 inches of depth, if desired. The maximum allowable bearing pressure should be limited to 3000 psf unless confirmed by Sladden Engineering subsequent to performing specific settlement calculations. The allowable bearing pressures are for dead and frequently applied live loads and may be increased by 1/3 to resist wind, seismic or other transient loading. All footings should be reinforced in accordance with the project structural engineer's recommendations.

Based on the allowable bearing pressures recommended above the total static settlement of conventional shallow spread footings is anticipated to be less than one inch, provided that foundation preparation conforms to the recommendations provided in this report. Differential static settlement is anticipated to be approximately one-half the total static settlement for similarly loaded footings spaced approximately 50 feet apart.

Resistance to lateral loads may be provided by a combination of friction acting at the base of the slabs or foundations and passive earth pressure along the sides of the foundations. A coefficient of friction of 0.45 between soil and concrete may be used for dead load forces only. A passive earth pressure of 300 pounds per square foot, per foot of depth, may be used for the sides of footings that are placed against properly compacted native soil. Passive earth pressure should be ignored within the upper 1 foot except where confined.

All footing excavations should be observed by a representative of the project geotechnical consultant to verify adequate embedment depths prior to placement of forms, steel reinforcement or concrete. The excavations should be trimmed neat, level and square. All loose, disturbed, sloughed or moisture-softened soils and/or any construction debris should be removed prior to concrete placement. Excavated soil generated from footing and/or utility trenches should not be stockpiled within the building envelope or in areas of exterior concrete flatwork.

## SLABS-ON-GRADE

In order to reduce the risk of heave, cracking and settlement, concrete slabs-on-grade must be placed on properly compacted fill as outlined in the previous sections. The slab subgrades should remain near optimum moisture content and should not be permitted to dry prior to concrete placement. All slab subgrades should be firm and unyielding. Disturbed soil should be removed and then replaced and compacted to a minimum of 90 percent relative compaction.

Slab thickness and reinforcement should be determined by the structural engineer. All slab reinforcement should be supported on concrete chairs to ensure that reinforcement is placed at slab mid-height. Considering the expected uses, we recommend a minimum slab thickness of 6.0 inches within warehouse areas and 4.0 inches within office areas along with minimum reinforcement of #3 bars at 24 inches on center in both directions in office areas and #4 bars at 24 inches on center in both directions in warehouse areas.

Slabs with moisture sensitive surfaces should be underlain with a moisture vapor barrier consisting of a polyvinyl chloride membrane such as 10-mil Visqueen. All laps within the membrane should be sealed and at least 2 inches of clean sand should be placed over the membrane to promote uniform curing of the concrete and to limit damage. To reduce the potential for punctures, the membrane should be placed on a pad surface that has been graded smooth without any sharp protrusions. If a smooth surface can not be achieved by grading, consideration should be given to placing a thin leveling course of sand across the pad surface prior to placement of the membrane.

## RETAINING WALLS

Cantilever retaining walls may be designed using “active” pressures. Active pressures may be estimated using an equivalent fluid weight of 35 pcf for gently sloping (less than 3H:1V) native backfill soil acting in a triangular pressure distribution with free-draining backfill conditions. “At Rest” pressures should be utilized for restrained walls. At rest pressures may be estimated using an equivalent fluid weight of 55 pcf for native backfill soil with level free-draining backfill conditions.

We recommend that a back drain system be provided behind all retaining walls or that the walls be designed for full hydrostatic pressures. The back drains should consist of a heavy walled, four inch diameter, perforated pipe sloped to drain to outlets by gravity, and of clean, free-draining, three-quarter to one and one-half inch crushed rock or gravel. The crushed rock or gravel should extend to within one foot of the surface. The upper one foot should be backfilled with compacted, fine-grained soil to exclude surface water. A Mirafi 140N (or equivalent) filter cloth should be placed between the on-site native material and the drain rock.

## ON-SITE PAVEMENT DESIGN

Asphalt concrete pavements should be designed in accordance with the Caltrans Highway Design Manual based on R-Value and Traffic Index. The R-Value of the near surface soil was determined to be 73 by exudation pressure and 69 by expansion pressure. On-site soil and any imported soil should be tested for R-Value after grading prior to establishing final pavement design sections. For preliminary pavement design, Traffic Indices (TI) of 5.0 and 6.5 were used for the light duty and heavy duty pavements, respectively. We assumed Asphalt Concrete (AC) over Class II Aggregate Base (AB). The preliminary flexible pavement layer thickness is as follows:

**TABLE 4**
**RECOMMENDED ASPHALT PAVEMENT SECTION LAYER THICKNESS**

| Pavement Material | Recommended Thickness | |
|---|---|---|
| | TI = 5.0 | TI = 6.5 |
| Asphalt Concrete Surface Course | 3.0 inches | 4.0 inches |
| Class II Aggregate Base Course | 4.0 inches | 6.0 inches |
| Compacted Subgrade Soil | 12.0 inches | 12.0 inches |

Asphalt concrete and Class II aggregate base should conform to the latest edition of the Standard Specifications for Public Works Construction (“Greenbook”) or CalTrans Standard Specifications. The aggregate base course should be compacted to at least 95 percent of the maximum dry density as determined by ASTM Method D 1557.

We expect that concrete pavement may also be considered for on-site pavement areas. A concrete pavement section of 6.0 inches of Portland Cement Concrete (PCC) on native subgrade soil should be adequate for the on-site concrete pavement limited to automobile and light truck traffic. In areas where heavy truck traffic is expected, the concrete pavement section should be increased to 8.0 inches of PCC compact native soil. Properly spaced and constructed control joints including expansion joints and contraction joints should be incorporated into concrete pavement design to accommodate temperature and shrinkage related cracking. Joint spacing and joint patterns should be established based upon Portland Cement Association (PCA) and American Concrete Institute (ACI) guidelines.

## CORROSION SERIES

The soluble sulfate concentrations of the surface soil were determined to be 20 parts per million (ppm). The soil is considered to have a “negligible” corrosion potential with respect to concrete. The use of Type V cement and special sulfate resistant concrete mixes should not be required. The soluble sulfate content of the surface soil should be reevaluated after grading and appropriate concrete mix designs should be established based upon post-grading test results.

The pH level of the surface soil was 5.8. Based on soluble chloride concentration testing (30 ppm), the soil is considered to have a “negligible” corrosion potential with respect to normal grade steel. The minimum resistivity of the surface soil was found to be 17,000 ohm-cm, which suggests that the site soil is considered to have a “negligible” corrosion potential with respect to ferrous metal installations. A corrosion expert should be consulted regarding appropriate corrosion protection measures for corrosion sensitive installations.

## UTILITY TRENCH BACKFILL

All utility trench backfill should be compacted to a minimum of 90 percent relative compaction. Trench backfill materials should be placed in thin lifts no greater than six inches in a loose condition, moisture conditioned (or air-dried) as necessary to achieve near optimum moisture content and then mechanically compacted to a minimum of 90 percent relative compaction. A representative of the project geotechnical consultant should test the backfill to verify adequate compaction.

## EXTERIOR CONCRETE FLATWORK

To minimize cracking of concrete flatwork the subgrade soil below concrete flatwork areas should be compacted to a minimum of 90 percent relative compaction. A representative of the project geotechnical consultant should observe and verify the density and moisture content of the soil.

## DRAINAGE

All final grades should be provided with positive gradients away from foundations to provide rapid removal of surface water runoff to an adequate discharge point. No water should be allowed to be pond on or immediately adjacent to foundation elements. In order to reduce water infiltration into the subgrade soil, surface water should be directed away from building foundations to an adequate discharge point. Subgrade drainage should be evaluated upon completion of the precise grading plans and in the field during grading.

## LIMITATIONS

The findings and recommendations presented in this report are based upon an interpolation of the soil conditions between the exploratory boring locations and extrapolation of these conditions throughout the proposed building area. Should conditions encountered during grading appear different than those indicated in this report, this office should be notified.

The use of this report by other parties or for other projects is not authorized. The recommendations of this report are contingent upon monitoring of the grading operation by a representative of Sladden Engineering. All recommendations are considered to be tentative pending our review of the grading operation and additional testing, if indicated. If others are employed to perform any soil testing, this office should be notified prior to such testing in order to coordinate any required site visits by our representative and to assure indemnification of Sladden Engineering.

We recommend that a pre-job conference be held on the site prior to the initiation of site grading. The purpose of this meeting will be to assure a complete understanding of the recommendations presented in this report as they apply to the actual grading performed.

## ADDITIONAL SERVICES

Once completed, final project plans and specifications should be reviewed by use prior to construction to confirm that the full intent of the recommendations presented herein have been applied to design and construction. Following review of plans and specifications, observation should be performed by the Soil Engineer during construction to document that foundation elements are founded on/or penetrate into the recommended soil, and that suitable backfill soil is placed upon competent materials and properly compacted at the recommended moisture content.

Tests and observations should be performed during grading by the Soil Engineer or his representative in order to verify that the grading is being performed in accordance with the project specifications. Field density testing shall be performed in accordance with acceptable ASTM test methods. The minimum acceptable degree of compaction should be 90 percent for subgrade soils and 95 percent for Class II aggregate base as obtained by the ASTM Test Method D1557. Where testing indicates insufficient density, additional compactive effort shall be applied until retesting indicates satisfactory compaction.

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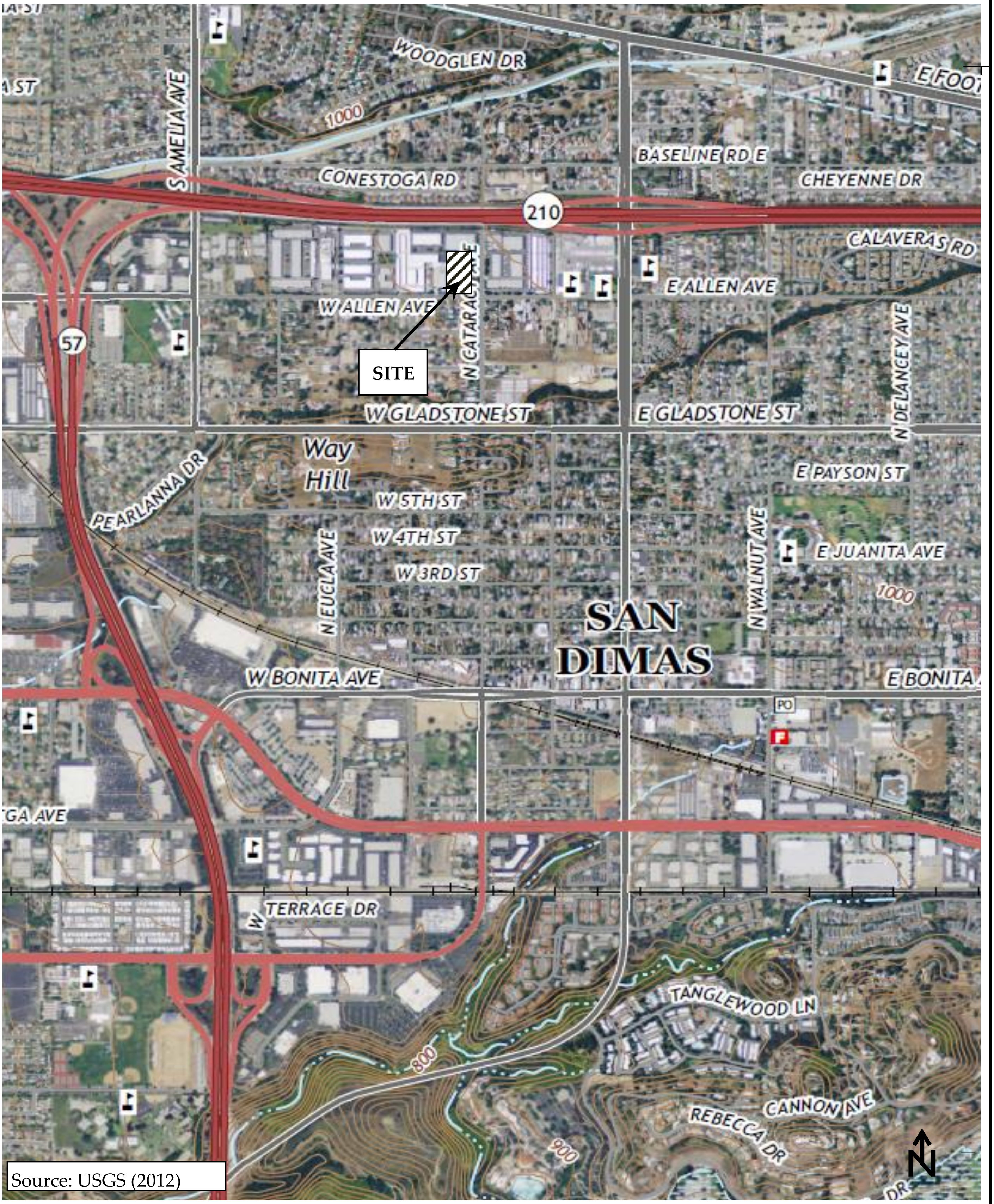
## FIGURES

SITE LOCATION MAP
REGIONAL GEOLOGIC MAP
BOREHOLE LOCATION PHOTOGRAPH
SITE PLAN



Source: USGS (2012)

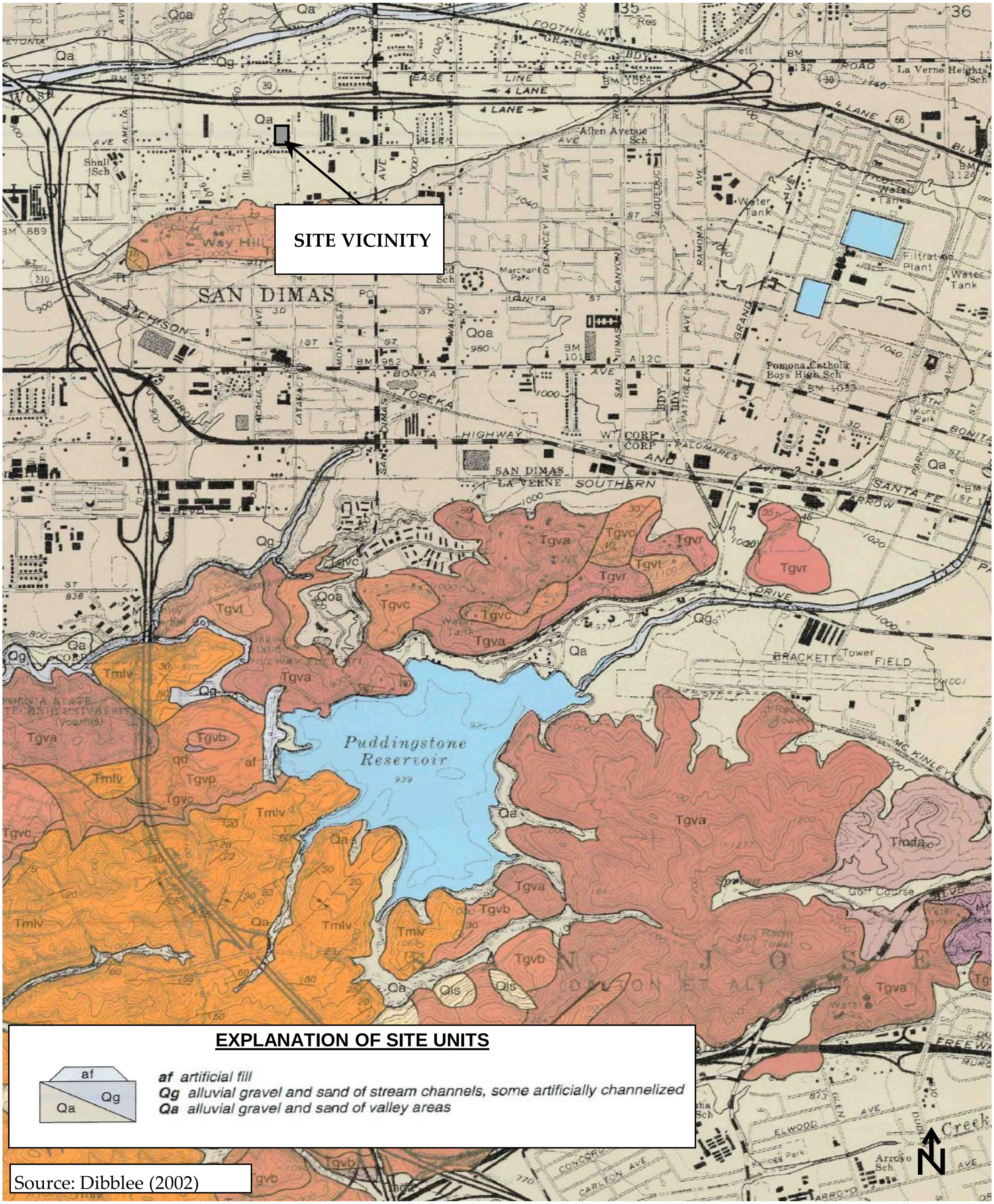
**Sladden Engineering**

# SITE LOCATION MAP

| | |
|---|---|
| Project Number: | 444-21106 |
| Report Number: | 22-01-001 |
| Date: | January 14, 2022 |

**FIGURE**

1



## EXPLANATION OF SITE UNITS

| | |
|---|---|
| af | ***af*** *artificial fill* |
| Qg | ***Qg*** *alluvial gravel and sand of stream channels, some artificially channelized* |
| Qa | ***Qa*** *alluvial gravel and sand of valley areas* |

Source: Dibblee (2002)

**Sladden Engineering**

# REGIONAL GEOLOGIC MAP

| | |
|---|---|
| Project Number: | 444-21106 |
| Report Number: | 22-01-001 |
| Date: | January 14, 2022 |

**FIGURE**

2

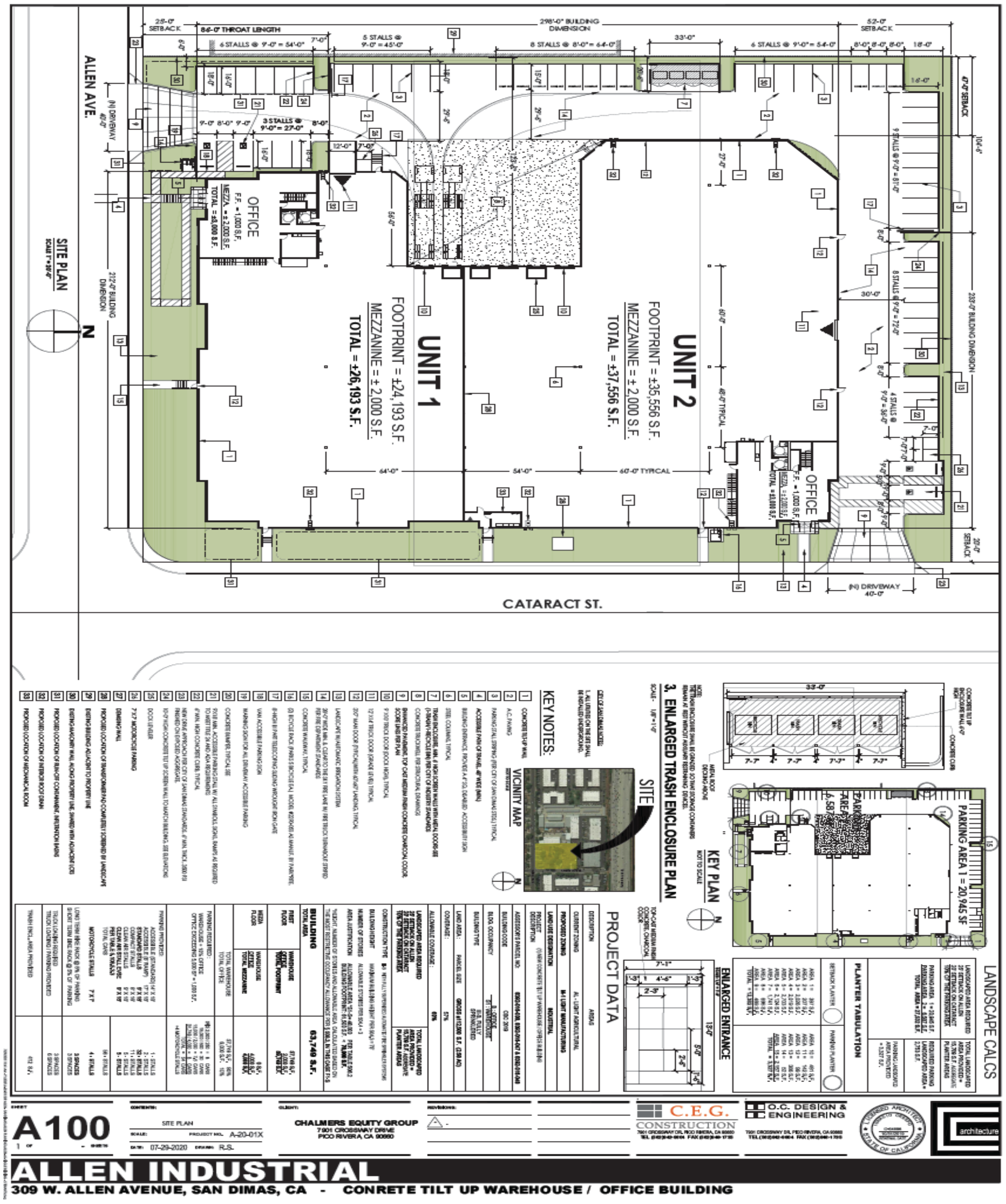
Source: Google Earth (2022)

Sladden Engineering

# BOREHOLE LOCATION PHOTOGRAPH

Project Number: 444-21106
Report Number: 22-01-001
Date: January 14, 2022

FIGURE
3



**Sladden Engineering**

# SITE PLAN

| | |
|---|---|
| Project Number: | 444-21106 |
| Report Number: | 22-01-001 |
| Date: | January 14, 2022 |

**FIGURE**

4

# APPENDIX A

## FIELD EXPLORATION

# APPENDIX A

# FIELD EXPLORATION

For our field investigation six (6) exploratory bores were excavated on December 1, 2021 utilizing a truck mounted hollow stem auger rig (Mobile B-61). Continuous logs of the materials encountered were made by a representative of Sladden Engineering. Materials encountered in the boreholes were classified in accordance with the Unified Soil Classification System which is presented in this appendix.

Representative undisturbed samples were obtained within our bores by driving a thin-walled steel penetration sampler (California split spoon sampler) or a Standard Penetration Test (SPT) sampler with a 140 pound automatic-trip hammer dropping approximately 30 inches (ASTM D1586). The number of blows required to drive the samplers 18 inches was recorded in 6-inch increments and blowcounts are indicated on the boring logs.

The California samplers are 3.0 inches in diameter, carrying brass sample rings having inner diameters of 2.5 inches. The standard penetration samplers are 2.0 inches in diameter with an inner diameter of 1.5 inches. Undisturbed samples were removed from the sampler and placed in moisture sealed containers in order to preserve the natural soil moisture content. Bulk samples were obtained from the excavation spoils and samples were then transported to our laboratory for further observations and testing.

**SLADDEN ENGINEERING**

# BORE LOG

| | | | |
|---|---|---|---|
| Drill Rig: | Mobil B-61 | Date Drilled: | 12/1/2021 |
| Elevation: | 965 Ft (MSL) | Boring No: | BH-1 |

| Sample | Blow Counts | Bulk Sample | Expansion Index | % Minus #200 | % Moisture | Dry Density | Depth (Feet) | Graphic Lithology | Description |
|---|---|---|---|---|---|---|---|---|---|
| | | | | | | | 2 | ▨ | Silty Sand (SM); yellowish brown, dry, fine-to-coarse grained with gravel and cobbles (Fill). |
| ☒ | 8/9/25 | | | | | | 4 | ▨ | Gravelly Sand (SW); yellowish brown, dry, medium dense, fine-to-coarse grained with cobbles (Qa). |
| | | | | | | | 6 | | Practical Auger Refusal at ~ 4.0 Feet bgs.<br>No Bedrock Encountered.<br>No Groundwater or Seepage Encountered. |
| | | | | | | | 8 | | |
| | | | | | | | 10 | | |
| | | | | | | | 12 | | |
| | | | | | | | 14 | | |
| | | | | | | | 16 | | |
| | | | | | | | 18 | | |
| | | | | | | | 20 | | |
| | | | | | | | 22 | | |
| | | | | | | | 24 | | |
| | | | | | | | 26 | | |
| | | | | | | | 28 | | |
| | | | | | | | 30 | | |
| | | | | | | | 32 | | |
| | | | | | | | 34 | | |
| | | | | | | | 36 | | |
| | | | | | | | 38 | | |
| | | | | | | | 40 | | |
| | | | | | | | 42 | | |
| | | | | | | | 44 | | |
| | | | | | | | 46 | | |
| | | | | | | | 48 | | |
| | | | | | | | 50 | | |

Completion Notes:

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309 WEST ALLEN AVENUE, SAN DIMAS

Project No: 444-21106
Report No: 22-01-001

Page 1

**SLADDEN ENGINEERING**

## BORE LOG

| | | | |
|---|---|---|---|
| Drill Rig: | Mobil B-61 | Date Drilled: | 12/1/2021 |
| Elevation: | 965 Ft (MSL) | Boring No: | BH-2/P-1 |

| Sample | Blow Counts | Bulk Sample | Expansion Index | % Minus #200 | % Moisture | Dry Density | Depth (Feet) | Graphic Lithology | Description |
|---|---|---|---|---|---|---|---|---|---|
| | | | | | | | 2 | ▨ | Silty Sand (SM); yellowish brown, dry, fine-to-coarse grained with gravel and cobbles (Fill). |
| | | | | | | | 4 | ▨ | Gravelly Sand (SW); yellowish brown, dry, medium dense, fine-to-coarse grained with cobbles (Qa). |
| ■ | 25/17/16 | | | 7.6 | 1.4 | | 6 | | Terminated at ~ 5.0 Feet bgs.<br>No Bedrock Encountered.<br>No Groundwater or Seepage Encountered.<br>Borehole Cased with Perforated Pipe for Percolation Testing. |
| | | | | | | | 8 | | |
| | | | | | | | 10 | | |
| | | | | | | | 12 | | |
| | | | | | | | 14 | | |
| | | | | | | | 16 | | |
| | | | | | | | 18 | | |
| | | | | | | | 20 | | |
| | | | | | | | 22 | | |
| | | | | | | | 24 | | |
| | | | | | | | 26 | | |
| | | | | | | | 28 | | |
| | | | | | | | 30 | | |
| | | | | | | | 32 | | |
| | | | | | | | 34 | | |
| | | | | | | | 36 | | |
| | | | | | | | 38 | | |
| | | | | | | | 40 | | |
| | | | | | | | 42 | | |
| | | | | | | | 44 | | |
| | | | | | | | 46 | | |
| | | | | | | | 48 | | |
| | | | | | | | 50 | | |

Completion Notes:

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309 WEST ALLEN AVENUE, SAN DIMAS

Project No: 444-21106
Report No: 22-01-001

**SLADDEN ENGINEERING**

# BORE LOG

| Drill Rig: | Mobil B-61 | Date Drilled: | 12/1/2021 |
|---|---|---|---|
| Elevation: | 965 Ft (MSL) | Boring No: | BH-3/P-2 |

| Sample | Blow Counts | Bulk Sample | Expansion Index | % Minus #200 | % Moisture | Dry Density | Depth (Feet) | Graphic Lithology | Description |
|---|---|---|---|---|---|---|---|---|---|
| | | | | | | | 2 | ▨ | Silty Sand (SM); yellowish brown, dry, fine-to-coarse grained with gravel and cobbles (Fill). |
| ■ | 9/14/15 | | | 4.7 | 1.4 | 118.7 | 4 | ░ | Gravelly Sand (SW); yellowish brown, dry, medium dense, fine-to-coarse grained with cobbles (Qa). |
| ■ | 18/19/22 | | | 8.1 | 1.7 | 127.7 | 6<br>8 | ░ | Gravelly Sand (SW); yellowish brown, dry, medium dense, fine-to-coarse grained with cobbles (Qa). |
| ■ | 14/16/17 | | | 8.3 | 1.4 | | 10 | ░ | Gravelly Sand (SW); yellowish brown, dry, dense, fine-to-coarse grained with cobbles (Qa). |
| | | | | | | | 12<br>14<br>16<br>18<br>20<br>22<br>24<br>26<br>28<br>30<br>32<br>34<br>36<br>38<br>40<br>42<br>44<br>46<br>48<br>50 | | Practical Auger Refusal at ~ 10.0 Feet bgs.<br>No Bedrock Encountered.<br>No Groundwater or Seepage Encountered.<br>Borehole Cased with Perforated Pipe for Percolation Testing. |

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Project No: 444-21106
Report No: 22-01-001

SLADDEN ENGINEERING

# BORE LOG

| | | | |
|---|---|---|---|
| Drill Rig: | Mobil B-61 | Date Drilled: | 12/1/2021 |
| Elevation: | 965 Ft (MSL) | Boring No: | BH-4 |

| Sample | Blow Counts | Bulk Sample | Expansion Index | % Minus #200 | % Moisture | Dry Density | Depth (Feet) | Graphic Lithology | Description |
|---|---|---|---|---|---|---|---|---|---|
| | | | | | | | | ▒ | Silty Sand (SM); yellowish brown, dry, fine-to-coarse grained with gravel and cobbles (Fill). |
| | | | | | | | 2 | ▒ | |
| | | | | | | | 4 | ▒ | Gravelly Sand (SW); yellowish brown, dry, fine-to-coarse grained with cobbles (Qa). |
| ■ | 12/12/12 | | | 4.0 | 2.2 | 123.1 | 6 | ▒ | Gravelly Sand (SW); yellowish brown, dry, medium dense, fine-to-coarse grained with cobbles (Qa). |
| | | | | | | | 8 | ▒ | |
| ▮ | 12/25/21 | | | 4.7 | 2.9 | | 10 | ▒ | Gravelly Sand (SW); yellowish brown, dry, dense, fine-to-coarse grained with cobbles (Qa). |
| | | | | | | | 12 | ▒ | |
| | | | | | | | 14 | | Practical Auger Refusal at ~ 13.0 Feet bgs.<br>No Bedrock Encountered.<br>No Groundwater or Seepage Encountered. |
| | | | | | | | 16 | | |
| | | | | | | | 18 | | |
| | | | | | | | 20 | | |
| | | | | | | | 22 | | |
| | | | | | | | 24 | | |
| | | | | | | | 26 | | |
| | | | | | | | 28 | | |
| | | | | | | | 30 | | |
| | | | | | | | 32 | | |
| | | | | | | | 34 | | |
| | | | | | | | 36 | | |
| | | | | | | | 38 | | |
| | | | | | | | 40 | | |
| | | | | | | | 42 | | |
| | | | | | | | 44 | | |
| | | | | | | | 46 | | |
| | | | | | | | 48 | | |
| | | | | | | | 50 | | |

Completion Notes:

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309 WEST ALLEN AVENUE, SAN DIMAS

Project No: 444-21106
Report No: 22-01-001

**SLADDEN ENGINEERING**

# BORE LOG

| | | | |
|---|---|---|---|
| Drill Rig: | Mobil B-61 | Date Drilled: | 12/1/2021 |
| Elevation: | 965 Ft (MSL) | Boring No: | BH-5 |

| Sample | Blow Counts | Bulk Sample | Expansion Index | % Minus #200 | % Moisture | Dry Density | Depth (Feet) | Graphic Lithology | Description |
|---|---|---|---|---|---|---|---|---|---|
| | | | | | | | | ▨ | Silty Sand (SM); yellowish brown, dry, fine-to-coarse grained with gravel and cobbles (Fill). |
| | | | | | | | 2 | ▨ | |
| | | | | | | | | ▨ | Gravelly Sand (SW); yellowish brown, dry, fine-to-coarse grained with cobbles (Qa). |
| | | | | | | | 4 | ▨ | |
| ▣ | 8/9/9 | | | 3.8 | 1.1 | | 6 | ▨ | Gravelly Sand (SW); yellowish brown, dry, medium dense, fine-to-coarse grained with cobbles (Qa). |
| | | | | | | | 8 | ▨ | |
| ■ | 50-6" | | | 3.7 | 1.2 | | 10 | ▨ | Gravelly Sand (SW); yellowish brown, dry, very dense, fine-to-coarse grained with cobbles (Qa). |
| | | | | | | | 12 | ▨ | |
| | | | | | | | 14 | ▨ | |
| | | | | | | | 16 | | Practical Auger Refusal at ~ 14.0 Feet bgs.<br>No Bedrock Encountered.<br>No Groundwater or Seepage Encountered. |
| | | | | | | | 18 | | |
| | | | | | | | 20 | | |
| | | | | | | | 22 | | |
| | | | | | | | 24 | | |
| | | | | | | | 26 | | |
| | | | | | | | 28 | | |
| | | | | | | | 30 | | |
| | | | | | | | 32 | | |
| | | | | | | | 34 | | |
| | | | | | | | 36 | | |
| | | | | | | | 38 | | |
| | | | | | | | 40 | | |
| | | | | | | | 42 | | |
| | | | | | | | 44 | | |
| | | | | | | | 46 | | |
| | | | | | | | 48 | | |
| | | | | | | | 50 | | |

Completion Notes:

PROPOSED ALLEN INDUSTRIAL FACILITY
309 WEST ALLEN AVENUE, SAN DIMAS

Project No: 444-21106
Report No: 22-01-001

**SLADDEN ENGINEERING**

# BORE LOG

| | | | |
|---|---|---|---|
| Drill Rig: | Mobil B-61 | Date Drilled: | 12/1/2021 |
| Elevation: | 965 Ft (MSL) | Boring No: | BH-6 |

| Sample | Blow Counts | Bulk Sample | Expansion Index | % Minus #200 | % Moisture | Dry Density | Depth (Feet) | Graphic Lithology | Description |
|---|---|---|---|---|---|---|---|---|---|
| | | | | | | | 2 | ░ | Silty Sand (SM); yellowish brown, dry, fine-to-coarse grained with gravel and cobbles (Fill). |
| ■ | 13/13/17 | 1 | 1 | 22.4 | 3.8 | 105.9 | 4 | ░ | Gravelly Sand (SW); yellowish brown, dry, medium dense, fine-to-coarse grained with cobbles (Qa). |
| ■ | 17/35/27 | | | 5.8 | 2.1 | 127.9 | 6<br>8 | ░ | Gravelly Sand (SW); yellowish brown, dry, dense, fine-to-coarse grained with cobbles (Qa). |
| ▮ | 12/18/20 | | | 8.4 | 2.3 | | 10<br>12<br>14 | ░ | Gravelly Sand (SW); yellowish brown, dry, dense, fine-to-coarse grained with cobbles (Qa). |
| ■ | 16/17/23 | | | 4.4 | 2.1 | 123.1 | 16 | ░ | Gravelly Sand (SW); yellowish brown, dry, dense, fine-to-coarse grained with cobbles (Qa). |
| | | | | | | | 18<br>20<br>22<br>24<br>26<br>28<br>30<br>32<br>34<br>36<br>38<br>40<br>42<br>44<br>46<br>48<br>50 | | Practical Auger Refusal at ~ 17.0 Feet bgs.<br>No Bedrock Encountered.<br>No Groundwater or Seepage Encountered. |

Completion Notes:

PROPOSED ALLEN INDUSTRIAL FACILITY
309 WEST ALLEN AVENUE, SAN DIMAS

Project No: 444-21106
Report No: 22-01-001

# APPENDIX B

## LABORATORY TESTING

# APPENDIX B

## LABORATORY TESTING

Representative bulk and relatively undisturbed soil samples were obtained in the field and returned to our laboratory for additional observations and testing. Laboratory testing was generally performed in two phases. The first phase consisted of testing in order to determine the compaction of the existing natural soil and the general engineering classifications of the soils underlying the site. This testing was performed in order to estimate the engineering characteristics of the soil and to serve as a basis for selecting samples for the second phase of testing. The second phase consisted of soil mechanics testing. This testing including consolidation, shear strength and expansion testing was performed in order to provide a means of developing specific design recommendations based on the mechanical properties of the soil.

## CLASSIFICATION AND COMPACTION TESTING

**Unit Weight and Moisture Content Determinations:** Each undisturbed sample was weighed and measured in order to determine its unit weight. A small portion of each sample was then subjected to testing in order to determine its moisture content. This was used in order to determine the dry density of the soil in its natural condition. The results of this testing are shown on the Bore Logs.

**Maximum Density-Optimum Moisture Determinations:** Representative soil types were selected for maximum density determinations. This testing was performed in accordance with the ASTM Standard D1557, Test Method A. The results of testing are presented graphically in this appendix. The maximum densities are compared to the field densities of the soil in order to determine the existing relative compaction to the soil.

**Classification Testing:** Soil samples were selected for classification testing. This testing consists of mechanical grain size analyses. This provides information for developing classifications for the soil in accordance with the Unified Soil Classification System which is presented in the preceding appendix. This classification system categorizes the soil into groups having similar engineering characteristics. The results of this testing is very useful in detecting variations in the soils and in selecting samples for further testing.

## SOIL MECHANIC'S TESTING

**Expansion Testing:** One (1) bulk sample was selected for Expansion testing. Expansion testing was performed in accordance with the UBC Standard 18-2. This testing consists of remolding 4-inch diameter by 1-inch thick test specimens to a moisture content and dry density corresponding to approximately 50 percent saturation. The samples are subjected to a surcharge of 144 pounds per square foot and allowed to reach equilibrium. At that point the specimens are inundated with distilled water. The linear expansion is then measured until complete.

**Direct Shear Testing:** One (1) sample was selected for Direct Shear testing. This test measures the shear strength of the soil under various normal pressures and is used to develop parameters for foundation design and lateral design. Tests were performed using a recompacted test specimen that was saturated prior to tests. Tests were performed using a strain controlled test apparatus with normal pressures ranging from 800 to 2300 pounds per square foot.

**Consolidation Testing:** Two (2) relatively undisturbed samples were selected for consolidation testing. For this test, a one-inch thick test specimen was subjected to vertical loads varying from 575 psf to 11520 psf applied progressively. The consolidation at each load increment was recorded prior to placement of each subsequent load. The specimens were saturated at 575 psf or 720 psf load increment.

**Corrosion Series Testing:** The soluble sulfate concentrations of the surface soil were determined in accordance with California Test Method Number (CA) 417. The pH and Minimum Resistivity were determined in accordance with CA 643. The soluble chloride concentrations were determined in accordance with CA 422.

**R-Value Testing:** One (1) representative bulk sample was selected for R-Value testing. The R-Value test measures the response of compacted subgrade soil to a vertically applied load. The R-Value tests and traffic indices are used for determining pavement design.

**Sladden Engineering**
450 Egan Avenue, Beaumont CA 92223 (951) 845-7743 Fax (951) 845-8863

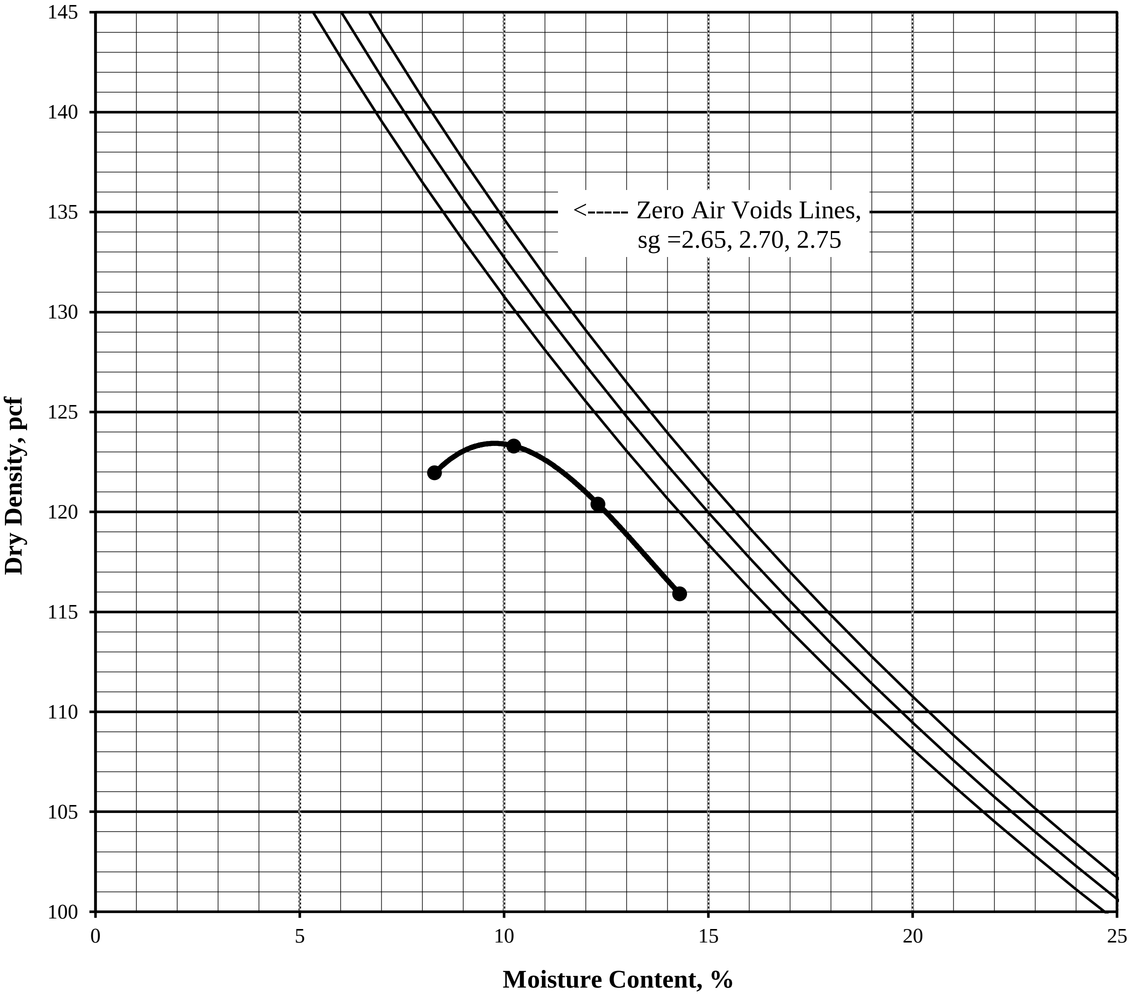
# Maximum Density/Optimum Moisture

ASTM D698/D1557

| | | |
|---|---|---|
| Project Number: | 444-21106 | December 16, 2021 |
| Project Name: | 309 West Allen Avenue | |
| Lab ID Number: | LN6-21629 | ASTM D-1557 A |
| Sample Location: | BH-6 Bulk 2 @ 0-5' | Rammer Type: Machine |
| Description: | Dark Brown Silty Sand w/Gravel (SM) | |

**Maximum Density:** **132 pcf**
**Optimum Moisture:** **8%**
Corrected for Oversize (ASTM D4718)

| Sieve Size | % Retained |
|---|---|
| 3/4" | |
| 3/8" | 20.6 |
| #4 | 25.3 |



# Expansion Index

ASTM D 4829

Job Number: 444-21106

December 16, 2021

Job Name: 309 West Allen Avenue

Lab ID Number: LN6-21629

Sample ID: BH-6  Bulk 2 @ 0-5'

Soil Description: Dark Brown Silty Sand w/Gravel (SM)

| | |
|---|---|
| Wt of Soil + Ring: | 579.7 |
| Weight of Ring: | 194.8 |
| Wt of Wet Soil: | 384.9 |
| Percent Moisture: | 8.7% |
| Sample Height, in | 0.95 |
| Wet Density, pcf: | 123.2 |
| Dry Denstiy, pcf: | 113.3 |

| | |
|---|---|
| **% Saturation:** | 48.2 |

**Expansion** **Rack # 2**

| | | |
|---|---|---|
| Date/Time | 12/13 | 3:20 PM |
| Initial Reading | 0.0000 | |
| Final Reading | 0.0010 | |

**Expansion Index** **1**

(Final - Initial) x 1000

## **Direct Shear** ASTM D 3080-04

(modified for unconsolidated condition)

Job Number: 444-21106

Job Name 309 West Allen Avenue

Lab ID No. LN6-21629

Sample ID BH-6 Bulk 2 @ 0-5'

Classification Dark Brown Silty Sand w/Gravel (SM)

Sample Type Remolded @ 90% of Maximum Density

December 16, 2021

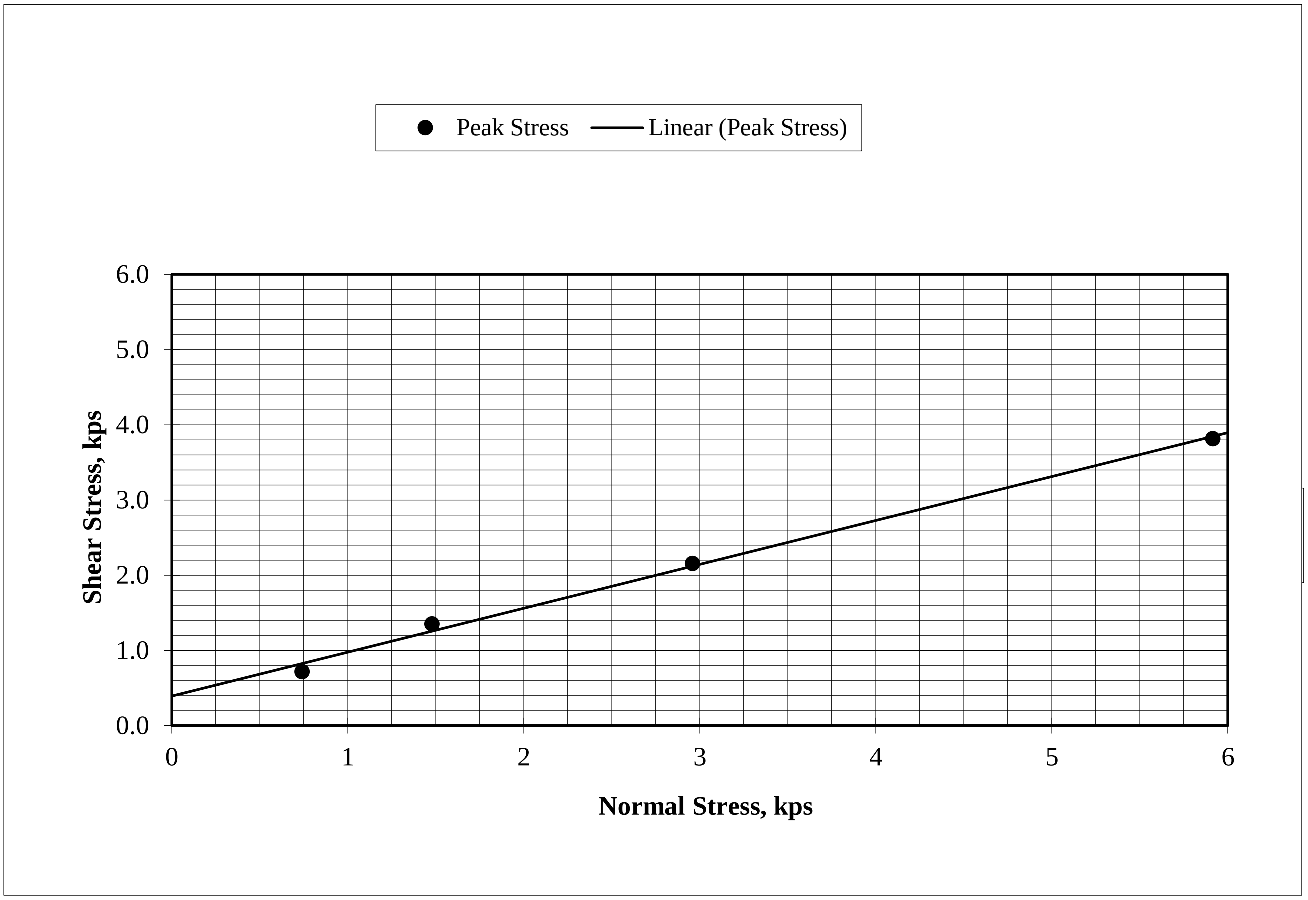
Initial Dry Density: 111.5 pcf

Initial Mosture Content: 10.0 %

Peak Friction Angle (Ø): 30°

Cohesion (c): 390 psf

| Test Results | 1 | 2 | 3 | 4 | Average |
|---|---|---|---|---|---|
| Moisture Content, % | 16.6 | 16.6 | 16.6 | 16.6 | 16.6 |
| Saturation, % | 87.6 | 87.6 | 87.6 | 87.6 | 87.6 |
| Normal Stress, kps | 0.739 | 1.479 | 2.958 | 5.916 | |
| Peak Stress, kps | 0.719 | 1.352 | 2.158 | 3.815 | |



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# Gradation

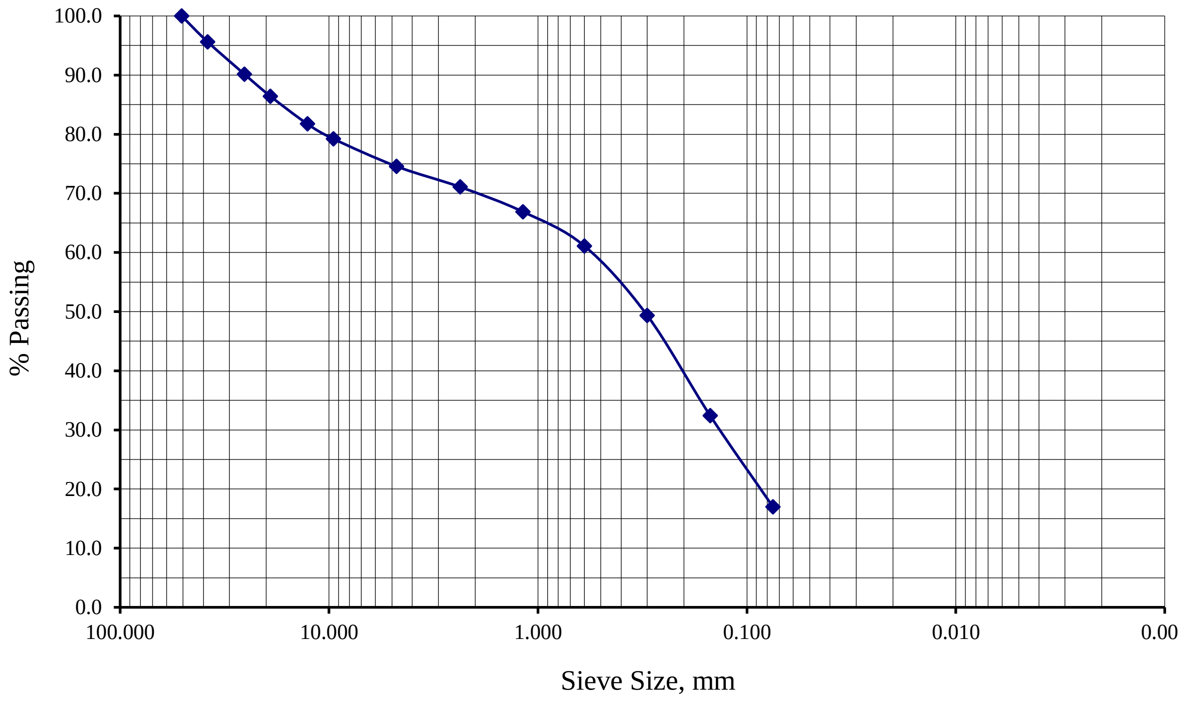
ASTM C117 & C136

Project Number: 444-21106
Project Name: 309 West Allen Avenue
Lab ID Number: LN6-21629
Sample ID: BH-6 Bulk 2 @ 0-5'

December 16, 2021

Soil Classification: SM

| Sieve Size, in | Sieve Size, mm | Percent Passing |
|---|---|---|
| 2" | 50.8 | 100.0 |
| 1 1/2" | 38.1 | 95.6 |
| 1" | 25.4 | 90.1 |
| 3/4" | 19.1 | 86.4 |
| 1/2" | 12.7 | 81.7 |
| 3/8" | 9.53 | 79.2 |
| #4 | 4.75 | 74.6 |
| #8 | 2.36 | 71.1 |
| #16 | 1.18 | 66.9 |
| #30 | 0.60 | 61.1 |
| #50 | 0.30 | 49.4 |
| #100 | 0.15 | 32.4 |
| #200 | 0.075 | 17.0 |



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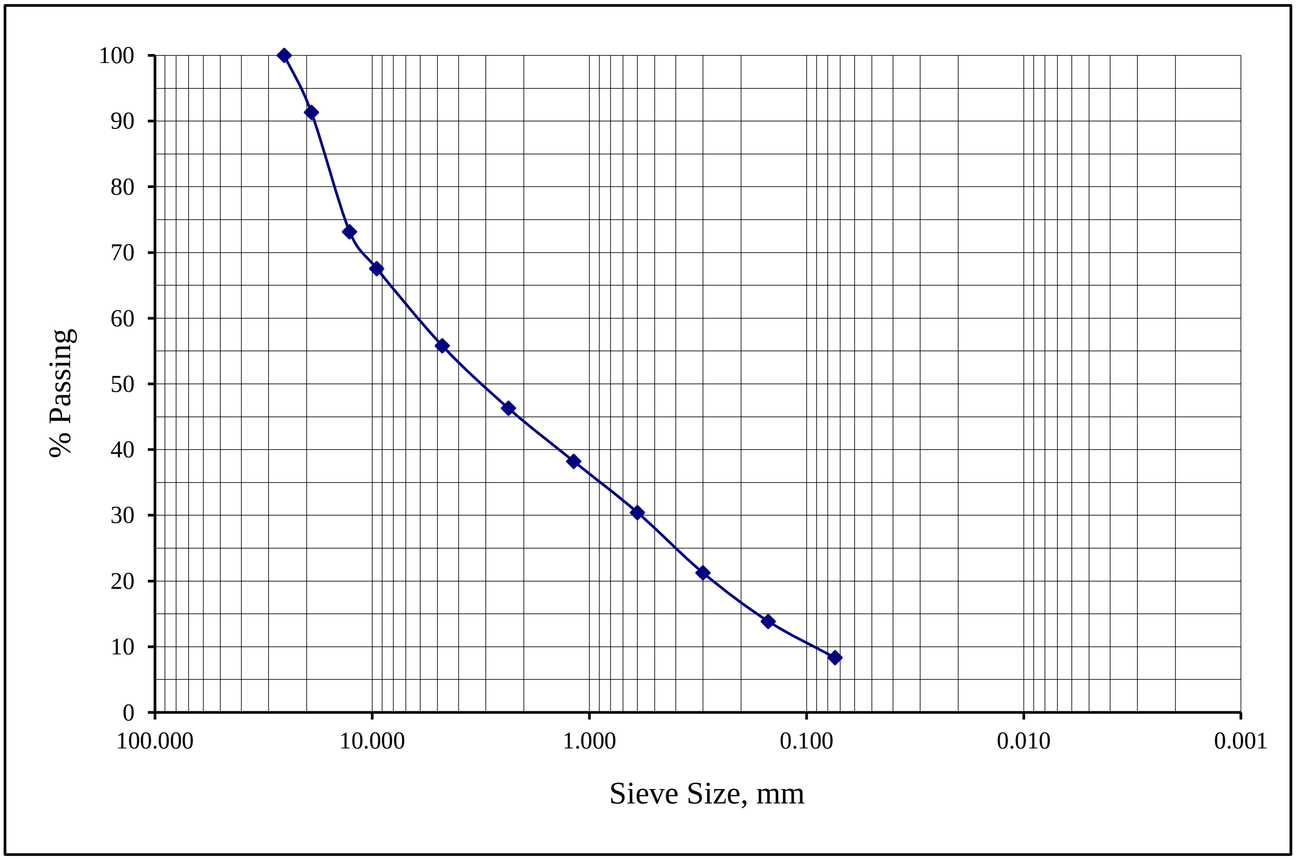
# Gradation

ASTM C117 & C136

| | | | |
|---|---|---|---|
| Project Number: | 444-21106 | | December 16, 2021 |
| Project Name: | 309 West Allen Avenue | | |
| Lab ID Number: | LN6-21629 | | |
| Sample ID: | BH-3  S-3 @ 10' | Soil Classification: | SP-SM |

| Sieve Size, in | Sieve Size, mm | Percent Passing |
|---|---|---|
| 1" | 25.4 | 100.0 |
| 3/4" | 19.1 | 91.3 |
| 1/2" | 12.7 | 73.2 |
| 3/8" | 9.53 | 67.6 |
| #4 | 4.75 | 55.8 |
| #8 | 2.36 | 46.3 |
| #16 | 1.18 | 38.2 |
| #30 | 0.60 | 30.4 |
| #50 | 0.30 | 21.3 |
| #100 | 0.15 | 13.9 |
| #200 | 0.074 | 8.3 |

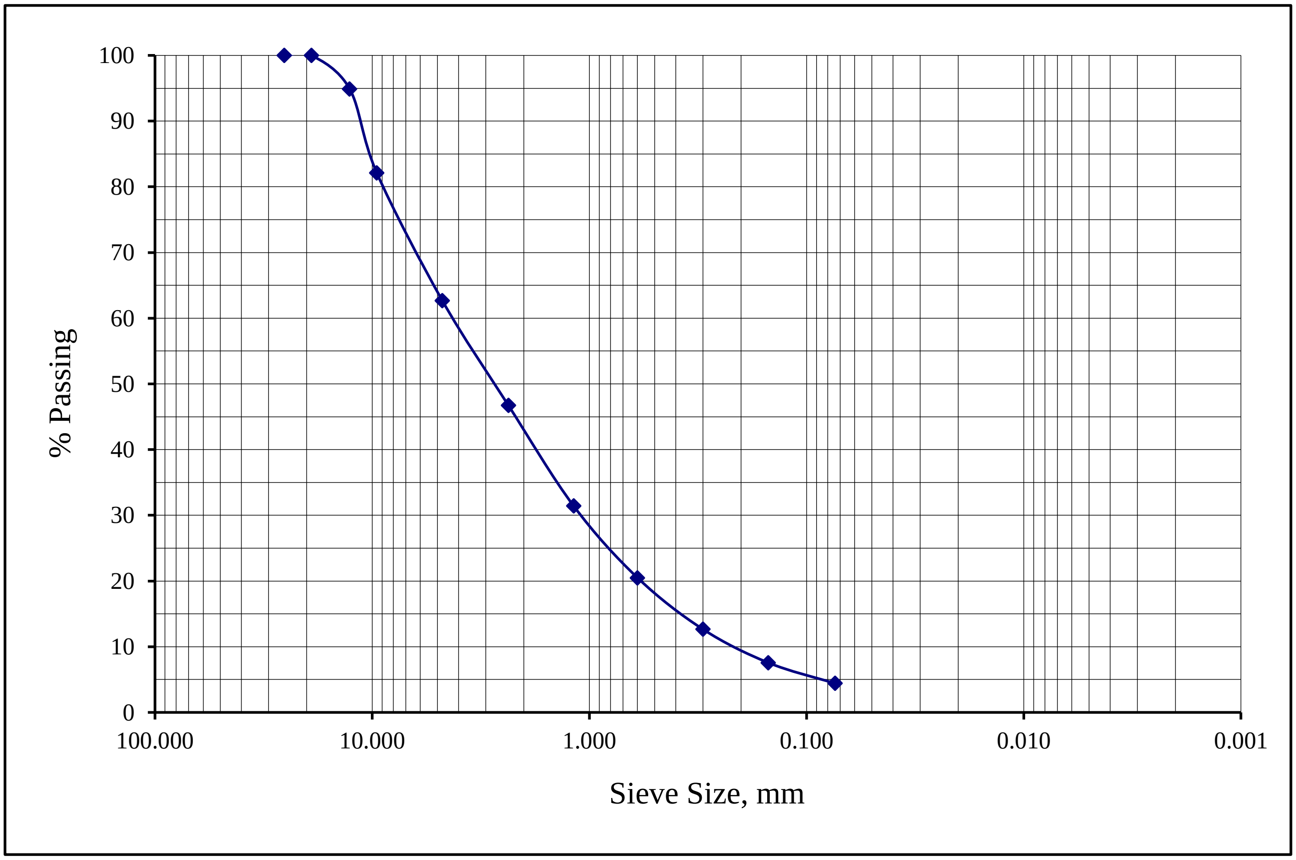


## Gradation

ASTM C117 & C136

Project Number: 444-21106 December 16, 2021
Project Name: 309 West Allen Avenue
Lab ID Number: LN6-21629
Sample ID: BH-6  R-4 @ 15' Soil Classification: SW

| Sieve Size, in | Sieve Size, mm | Percent Passing |
|---|---|---|
| 1" | 25.4 | 100.0 |
| 3/4" | 19.1 | 100.0 |
| 1/2" | 12.7 | 94.9 |
| 3/8" | 9.53 | 82.1 |
| #4 | 4.75 | 62.7 |
| #8 | 2.36 | 46.7 |
| #16 | 1.18 | 31.4 |
| #30 | 0.60 | 20.5 |
| #50 | 0.30 | 12.7 |
| #100 | 0.15 | 7.6 |
| #200 | 0.074 | 4.4 |



**Sladden Engineering**

450 Egan Avenue, Beaumont, CA 92223 (951) 845-7743 Fax (951) 845-8863

## RESISTANCE '*R*' VALUE AND EXPANSION PRESSURE

**CTM 301**

December 16, 2021

Project Number: 444-21106

Project Name: 309 West Allen Avenue

Lab ID Number: LN6-21629

Sample ID: BH-6 Bulk 2 @ 0-5'

Sample Description: Dark Brown Silty Sand with Gravel (SM)

Specified Traffic Index: 5.0

Dry Density @ 300 psi Exudation Pressure: 119.8-pcf

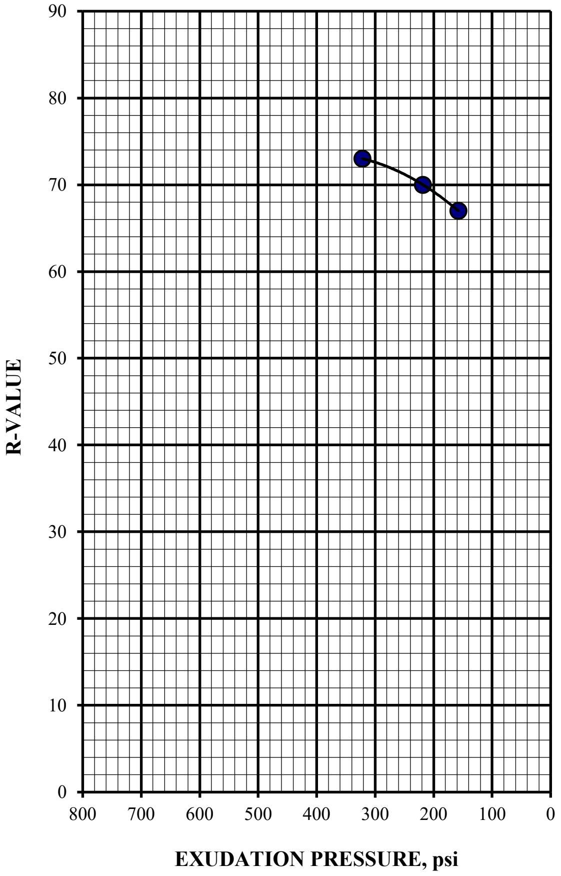
%Moisture @ 300 psi Exudation Pressure: 11.2%

R-Value - Exudation Pressure: 73

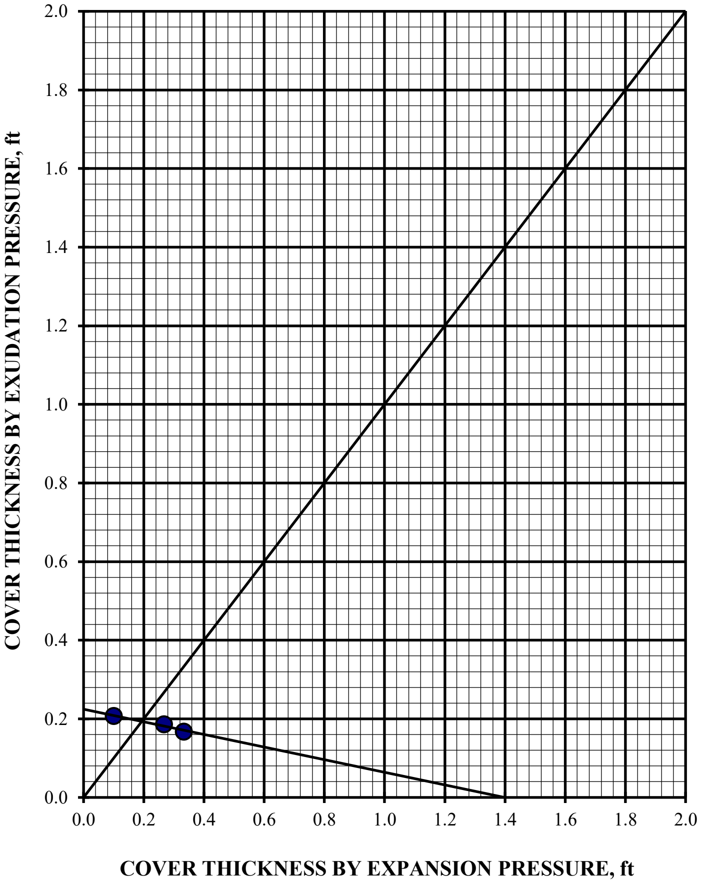
R-Value - Expansion Pressure: 69

**R-Value @ Equilibrium: 69**

### EXUDATION PRESSURE CHART



### EXPANSION PRESSURE CHART



# APPENDIX C

SEISMIC DESIGN MAP AND REPORT
SEISMIC HAZARD ANALYSIS (SHA)

## Latitude, Longitude: 34.1181, -117.8120

| | |
|---|---|
| **Date** | 1/13/2022, 9:06:01 AM |
| **Design Code Reference Document** | ASCE7-16 |
| **Risk Category** | II |
| **Site Class** | D - Stiff Soil |

| Type | Value | Description |
|---|---|---|
| $S_S$ | 1.691 | $MCE_R$ ground motion. (for 0.2 second period) |
| $S_1$ | 0.637 | $MCE_R$ ground motion. (for 1.0s period) |
| $S_{MS}$ | 1.691 | Site-modified spectral acceleration value |
| $S_{M1}$ | null -See Section 11.4.8 | Site-modified spectral acceleration value |
| $S_{DS}$ | 1.128 | 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 | 0.72 | $MCE_G$ peak ground acceleration |
| $F_{PGA}$ | 1.1 | Site amplification factor at PGA |
| $PGA_M$ | 0.792 | Site modified peak ground acceleration |
| $T_L$ | 8 | Long-period transition period in seconds |
| SsRT | 1.691 | Probabilistic risk-targeted ground motion. (0.2 second) |
| SsUH | 1.834 | Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration |
| SsD | 2.138 | Factored deterministic acceleration value. (0.2 second) |
| S1RT | 0.637 | Probabilistic risk-targeted ground motion. (1.0 second) |
| S1UH | 0.698 | Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration. |
| S1D | 0.768 | Factored deterministic acceleration value. (1.0 second) |
| PGAd | 0.883 | Factored deterministic acceleration value. (Peak Ground Acceleration) |
| $C_{RS}$ | 0.922 | Mapped value of the risk coefficient at short periods |
| $C_{R1}$ | 0.912 | Mapped value of the risk coefficient at a period of 1 s |

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**Project:** 309 W. Allen Ave., San Dimas
**Project Number:** 444-21106
**Client:** CEG
**Site Lat/Long:** 34.1181/ -117.8120
**Controlling Seismic Source:** San Jose

| REFERENCE | NOTATION | VALUE |
|---|---|---|
| Site Class | C, D, D default, or E | D measured |
| Site Class D - Table 11.4-1 | $F_a$ | 1.0 |
| Site Class D - 21.3(ii) | $F_v$ | 2.5 |
| $0.2*(S_{D1}/S_{DS})$ | $T_0$ | 0.128 |
| $S_{D1}/S_{DS}$ | $T_S$ | 0.640 |
| Fundamental Period (12.8.2) | T | Period |
| Seismic Design Maps or Fig 22-14 | $T_L$ | 8 |
| Equation 11.4-4 - $2/3*S_{M1}$ | $S_{D1}$ | 0.7219* |
| Equation 11.4-2 - $F_V*S_1$ | $S_{M1}$ | 1.0829* |

| REFERENCE | NOTATION | VALUE |
|---|---|---|
| Fv (Table 11.4-2)[Used for General Spectrum] | $F_v$ | 1.7 |
| Design Maps | $S_s$ | 1.691 |
| Design Maps | $S_1$ | 0.637 |
| Equation 11.4-1 - $F_A*S_S$ | $S_{MS}$ | 1.691* |
| Equation 11.4-3 - $2/3*S_{MS}$ | $S_{DS}$ | 1.127* |
| Design Maps | PGA | 0.72 |
| Table 11.8-1 | $F_{PGA}$ | 1.1 |
| Equation 11.8-1 - $F_{PGA}$*PGA | $PGA_M$ | 0.792* |
| Section 21.5.3 | 80% of $PGA_M$ | 0.634 |
| Design Maps | $C_{RS}$ | 0.922 |
| Design Maps | $C_{R1}$ | 0.912 |

## RISK COEFFICIENT

|  |  |  |
|---|---|---|
| Cr - At Perods <=0.2, Cr=$C_{RS}$ | $C_{RS}$ | 0.922 |
| Cr - At Periods >=1.0, Cr=$C_{R1}$ | $C_{R1}$ | 0.912 |

Cr - At Periods between 0.2 and 1.0 use trendline formula to complete

| Period | Cr |
|---|---|
| 0.200 | 0.922 |
| 0.300 | 0.921 |
| 0.400 | 0.920 |
| 0.500 | 0.918 |
| 0.600 | 0.917 |
| 0.680 | 0.916 |
| 1.000 | 0.912 |

* **Code based design value**. See accompanying data for Site Specific Design values.

Mapped values from https://seismicmaps.org/

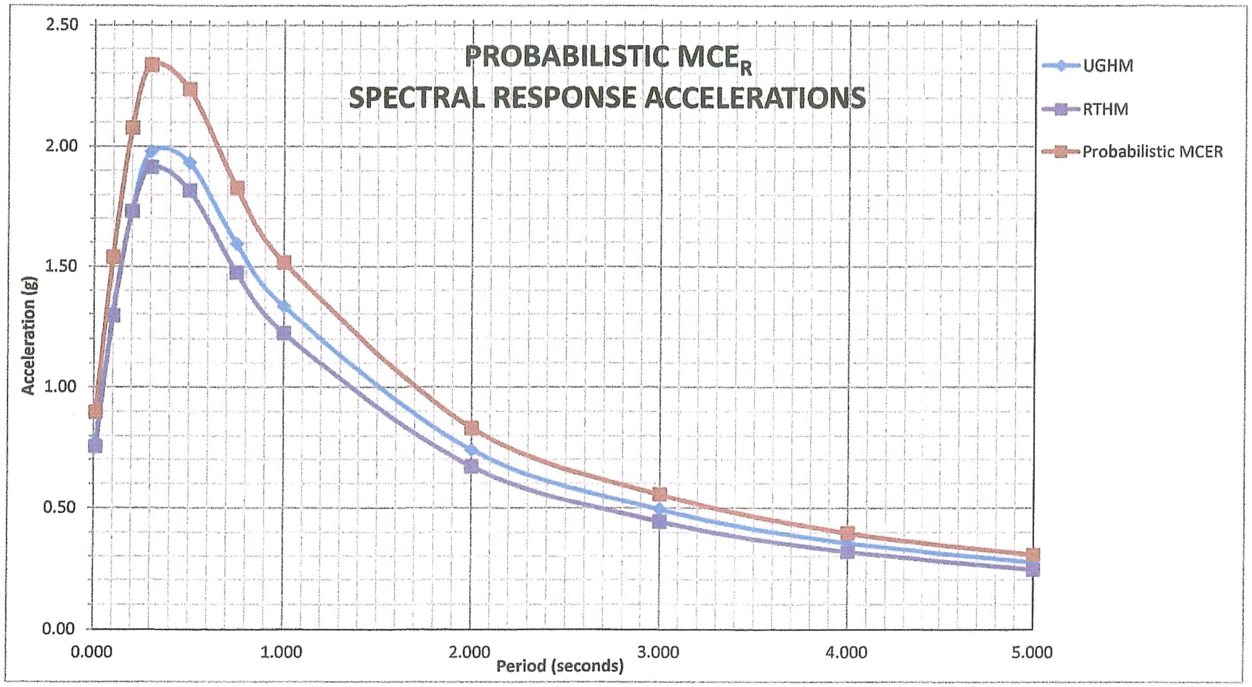
**Sladden Engineering**

## PROBABILISTIC SPECTRA[1]
## 2% in 50 year Exceedence

Project No: 444-21106

| Period | UGHM | RTHM | Max Directional Scale Factor[2] | Probabilistic MCE |
|---|---|---|---|---|
| 0.010 | 0.777 | 0.753 | 1.19 | 0.896 |
| 0.100 | 1.309 | 1.292 | 1.19 | 1.537 |
| 0.200 | 1.738 | 1.730 | 1.20 | 2.076 |
| 0.300 | 1.977 | 1.912 | 1.22 | 2.333 |
| 0.500 | 1.931 | 1.817 | 1.23 | 2.235 |
| 0.750 | 1.591 | 1.474 | 1.24 | 1.828 |
| 1.000 | 1.331 | 1.222 | 1.24 | 1.515 |
| 2.000 | 0.739 | 0.670 | 1.24 | 0.831 |
| 3.000 | 0.492 | 0.443 | 1.25 | 0.554 |
| 4.000 | 0.354 | 0.317 | 1.25 | 0.396 |
| 5.000 | 0.272 | 0.242 | 1.26 | 0.305 |

|  |  |
|---|---|
| Probabilistic PGA: | 0.777 |
| Is Probabilistic $Sa_{(max)}<1.2F_a$? | NO |



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[1] Data Sources:
https://earthquake.usgs.gov/hazards/interactive/
https://earthquake.usgs.gov/designmaps/rtgm/

[2] Shahi-Baker RotD100/RotD50 Factors (2014)

# DETERMINISTIC SPECTRUM

**Largest Amplitudes of Ground Motions Considering All Sources Calculated using Weighted Mean of Attenuation Equations[1]**

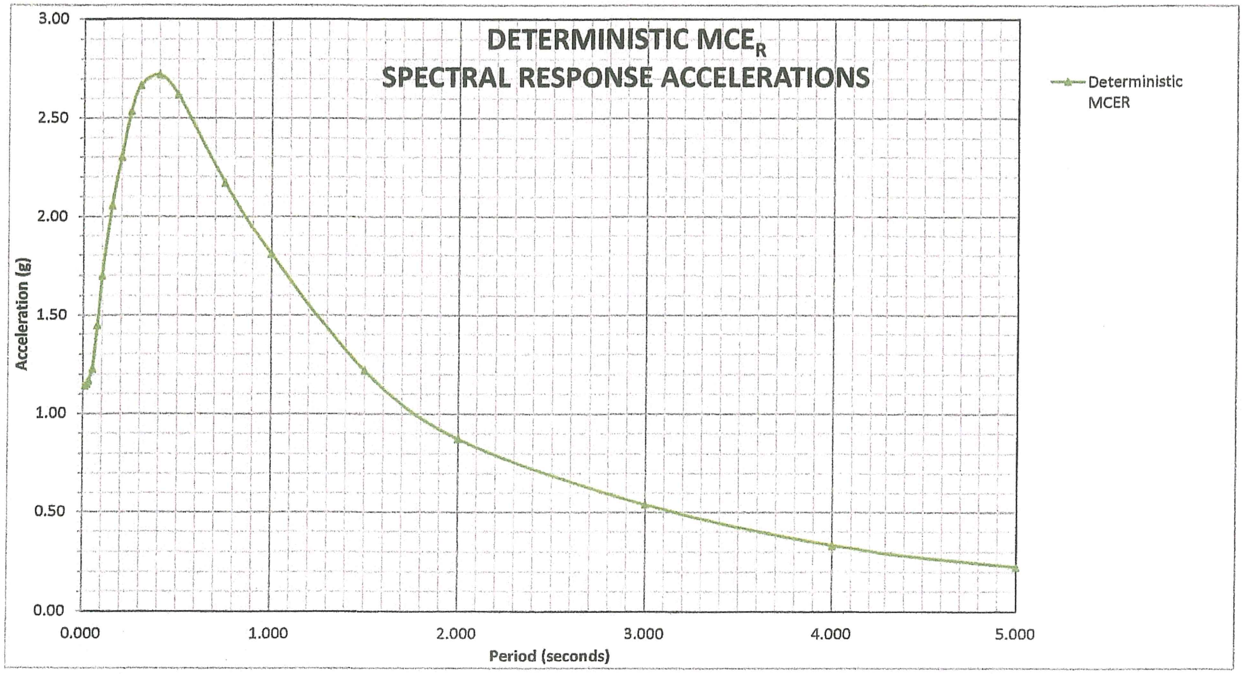
**Controlling Source: San Jose**

Is Probabilistic $Sa_{(max)}$<1.2Fa? NO

Project No: 444-21106

| Period | Deterministic PSa Median + 1.σ for 5% Damping | Max Directional Scale Factor[2] | Deterministic MCE | Section 21.2.2 Scaling Factor Applied |
|---|---|---|---|---|
| 0.010 | 0.959 | 1.19 | 1.141 | 1.141 |
| 0.020 | 0.968 | 1.19 | 1.152 | 1.152 |
| 0.030 | 0.981 | 1.19 | 1.168 | 1.168 |
| 0.050 | 1.031 | 1.19 | 1.227 | 1.227 |
| 0.075 | 1.216 | 1.19 | 1.447 | 1.447 |
| 0.100 | 1.428 | 1.19 | 1.699 | 1.699 |
| 0.150 | 1.711 | 1.20 | 2.053 | 2.053 |
| 0.200 | 1.916 | 1.20 | 2.299 | 2.299 |
| 0.250 | 2.094 | 1.21 | 2.533 | 2.533 |
| 0.300 | 2.186 | 1.22 | 2.667 | 2.667 |
| 0.400 | 2.213 | 1.23 | 2.722 | 2.722 |
| 0.500 | 2,131 | 1.23 | 2.621 | 2.621 |
| 0.750 | 1.750 | 1.24 | 2.170 | 2.170 |
| 1.000 | 1.456 | 1.24 | 1.805 | 1.805 |
| 1.500 | 0.982 | 1.24 | 1.218 | 1.218 |
| 2.000 | 0.705 | 1.24 | 0.874 | 0.874 |
| 3.000 | 0.431 | 1.25 | 0.539 | 0.539 |
| 4.000 | 0.265 | 1.25 | 0.332 | 0.332 |
| 5.000 | 0.179 | 1.26 | 0.225 | 0.225 |

| | |
|---|---|
| Is Determinstic $Sa_{(max)}$<1.5*Fa? | NO |
| Section 21.2.2 Scaling Factor: | N/A |
| Deterministic PGA: | 0.959 |
| Is Deterministic PGA >=$F_{PGA}$*0.5? | YES |



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[1] NGAWest 2 GMPE worksheet and Uniform California Earthquake Rupture Forecast, Version 3 (UCERF3) - Time Dependent Model

[2] Shahi-Baker RotD100/RotD50 Factors (2014)

## SITE SPECIFIC SPECTRA

| Period | Probabilistic MCE | Deterministic MCE | Site-Specific MCE | Design Response Spectrum (Sa) |
|---|---|---|---|---|
| 0.010 | 0.896 | 1.141 | 0.896 | 0.597 |
| 0.100 | 1.537 | 1.699 | 1.537 | 1.025 |
| 0.200 | 2.076 | 2.299 | 2.076 | 1.384 |
| 0.300 | 2.333 | 2.667 | 2.333 | 1.555 |
| 0.500 | 2.235 | 2.621 | 2.235 | 1.490 |
| 0.750 | 1.828 | 2.170 | 1.828 | 1.219 |
| 1.000 | 1.515 | 1.805 | 1.515 | 1.010 |
| 2.000 | 0.831 | 0.874 | 0.831 | 0.554 |
| 3.000 | 0.554 | 0.539 | 0.539 | 0.359 |
| 4.000 | 0.396 | 0.332 | 0.332 | 0.221 |
| 5.000 | 0.305 | 0.225 | 0.225 | 0.150 |

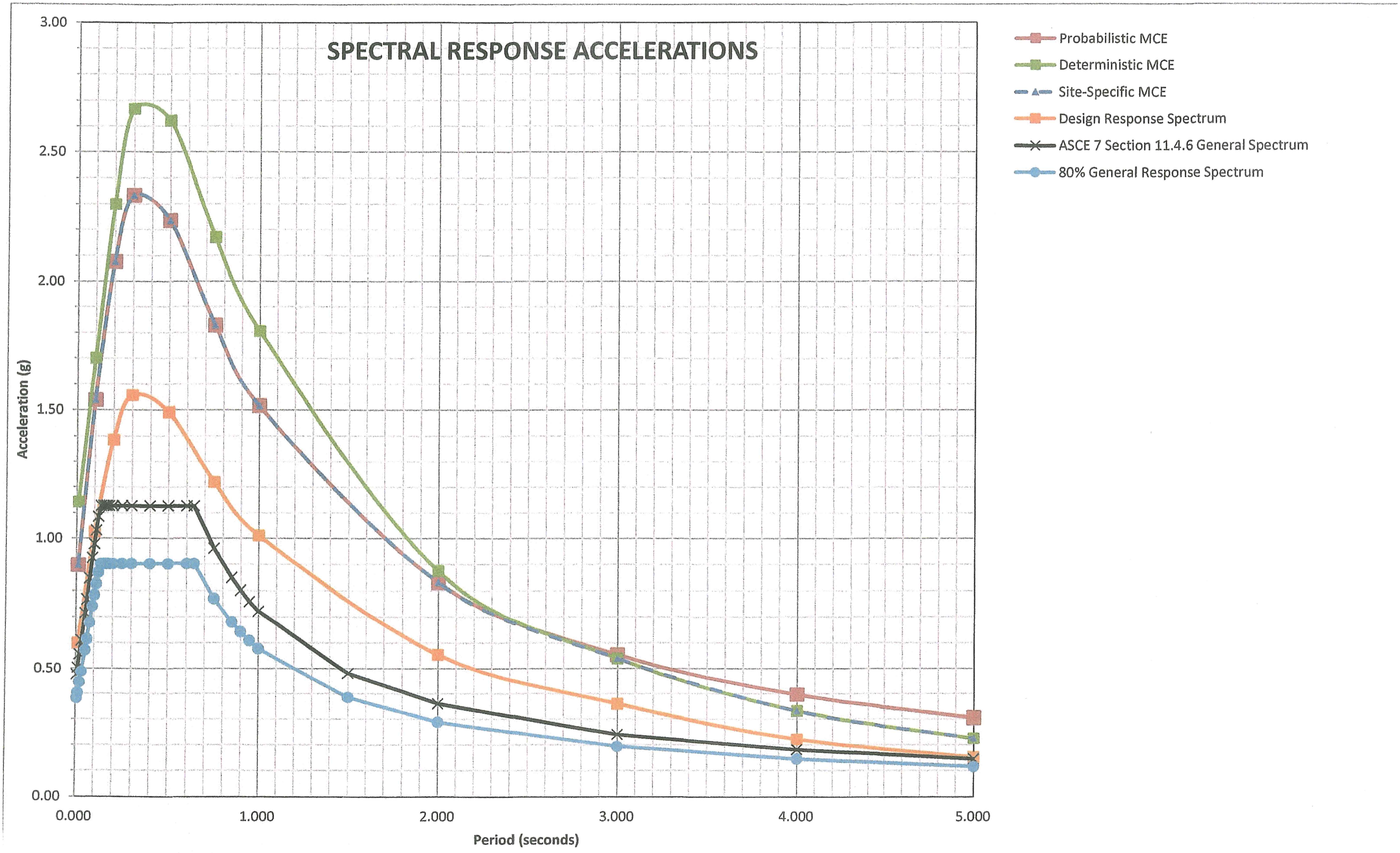
**ASCE 7-16: Section 21.4**
**Site Specific**

| | Calculated Value | Design Value |
|---|---|---|
| **SDS:** | 1.400 | 1.400 |
| **SD1:** | 1.108 | 1.108 |
| **SMS:** | 2.099 | 2.099 |
| **SM1:** | 1.662 | 1.662 |
| **Site Specific PGAm:** | 0.777 | 0.777 |
| **Site Class:** | D measured | |

| | |
|---|---|
| **Seismic Design Category - Short*** | D |
| **Seismic Design Category - 1s*** | D |

* Risk Categories I, II, or III

| Period | ASCE 7 SECTION 11.4.6 General Spectrum | 80% General Response Spectrum |
|---|---|---|
| 0.005 | 0.477 | 0.382 |
| 0.010 | 0.504 | 0.403 |
| 0.020 | 0.557 | 0.445 |
| 0.030 | 0.609 | 0.487 |
| 0.050 | 0.715 | 0.572 |
| 0.060 | 0.768 | 0.614 |
| 0.075 | 0.847 | 0.678 |
| 0.090 | 0.926 | 0.741 |
| 0.100 | 0.979 | 0.783 |
| 0.110 | 1.032 | 0.825 |
| 0.120 | 1.085 | 0.868 |
| 0.136 | 1.127 | 0.902 |
| 0.150 | 1.127 | 0.902 |
| 0.160 | 1.127 | 0.902 |
| 0.170 | 1.127 | 0.902 |
| 0.180 | 1.127 | 0.902 |
| 0.200 | 1.127 | 0.902 |
| 0.250 | 1.127 | 0.902 |
| 0.300 | 1.127 | 0.902 |
| 0.400 | 1.127 | 0.902 |
| 0.500 | 1.127 | 0.902 |
| 0.600 | 1.127 | 0.902 |
| 0.640 | 1.127 | 0.902 |
| 0.750 | 0.963 | 0.770 |
| 0.850 | 0.849 | 0.679 |
| 0.900 | 0.802 | 0.642 |
| 0.950 | 0.760 | 0.608 |
| 1.000 | 0.722 | 0.578 |
| 1.500 | 0.481 | 0.385 |
| 2.000 | 0.361 | 0.289 |
| 3.000 | 0.241 | 0.193 |
| 4.000 | 0.180 | 0.144 |
| 5.000 | 0.144 | 0.116 |


SPECTRAL RESPONSE ACCELERATIONS
Acceleration (g)
3.00
2.50
2.00
1.50
1.00
0.50
0.00
0.000
1.000
2.000
3.000
4.000
5.000
Period (seconds)
Probabilistic MCE
Deterministic MCE
Site-Specific MCE
Design Response Spectrum
ASCE 7 Section 11.4.6 General Spectrum
80% General Response Spectrum