Geotechnical Report

Calexico Transit Center
SWC 3rd and Heffernan Avenue
Calexico, California

Prepared for:

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April 20, 2021

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Geotechnical Report
Calexico Transit Center
SWC 3rd Street and Heffernan Avenue
Calexico, CA
LCI Report No. LE21050

Dear Mr. Chan:

This geotechnical report is provided for design and construction of the proposed Calexico Transit Center located at the southwest corner of 3rd Street and Heffernan Avenue in Calexico, California. Our geotechnical exploration was conducted in response to your request for our services. The enclosed report describes our soil engineering site evaluation and presents our professional opinions regarding geotechnical conditions at the site to be considered in the design and construction of the project.

Based on the geotechnical conditions encountered at the points of exploration, the project site appears suitable for the proposed construction provided the professional opinions contained in this report are considered in the design and construction of this project.

We appreciate the opportunity to provide our findings and professional opinions regarding geotechnical conditions at the site. Please provide our office with a set of the foundation plans and civil plans for review to insure that the geotechnical site constraints have been included in the design documents. If you have any questions or comments regarding our findings, please call our office at (760) 370-3000.

Respectfully Submitted,
Landmark Consultants, Inc.

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EXECUTIVE SUMMARY

This executive summary presents ***selected*** elements of our findings and professional opinions. This summary ***may not*** present all details needed for the proper application of our findings and professional opinions. Our findings, professional opinions, and application options are ***best related through reading the full report***, and are best evaluated with the active participation of the engineer of record who developed them. The findings of this study are summarized below:

- Clay soils (CL) of medium to high expansion (EI = 75-110) predominate the near surface soils at the project site.

- Foundation designs for slab-on-grade structures should mitigate expansive soil conditions by either the removal and replacement of the upper 3.0 feet of clay soils with non-expansive soil or design of foundations to resist expansive forces, such as flat plate structural mats, grade-beam stiffened floor slabs, or post-tensioned floor slabs. A combination of the methods described above may also be used.

- Design soil bearing pressure = 1,500 psf (clays) with standard increases allowed by the California Building Code. Footings placed over 18 inches of granular soil may be designed for 2,500 psf. Differential movement of 1.0 to 1.5 inches can be expected for slab on grade foundations placed on clay soils.

- The risk of liquefaction induced settlement is low. There is a very low risk of ground rupture and/or sand boil formation should liquefaction occur.

- The native soils are aggressive to concrete and steel. Concrete mixes for concrete placed in contact with native soils shall have a maximum water cement ratio of 0.45 and a minimum compressive strength of 4,500 psi (minimum of 6 sacks Type V cement per cubic yard). All concrete should be thoroughly vibrated to remove rock pockets and minimize air voids.

- All reinforcing bars, anchor bolts and hold down bolts shall have a minimum concrete cover of 3.0 inches unless epoxy coated (ASTM D3963/A934). Hold-down straps at the foundation perimeter and pressurized water lines below or within the foundations are not allowed.

- Pavement structural sections should be designed for clay subgrade soils (R-Value = 5) and an appropriate Traffic Index (TI) selected by the civil designer.

Section 1
INTRODUCTION

1.1 Project Description

This report presents the findings of our geotechnical exploration and soil testing for the proposed Calexico Transit Center located at the southwest corner of 3rd Street and Heffernan Avenue in Calexico, California (See Vicinity Map, Plate A-1). The proposed development will consist of a commuter bus transfer facility with parking areas, a single story building with terrace, shade structures, and a stormwater basin. A site plan for the proposed development was provided by Psomas.

The structure is planned to consist of slabs-on-grade foundations and wood-frame construction. Footing loads at exterior bearing walls are estimated at 0.5 to 2 kips per lineal foot. Column loads are estimated to range from 5 to 20 kips. If structural loads exceed those stated above, we should be notified so we may evaluate their impact on foundation settlement and bearing capacity. Site development will include building pad preparation, underground utility installation including trench backfill, concrete foundation construction, street and parking lot construction, and concrete hardscape.

1.2 Purpose and Scope of Work

The purpose of this geotechnical study was to investigate the subsurface soil at selected locations within the site for evaluation of physical/engineering properties and liquefaction potential during seismic events. Professional opinions were developed from field and laboratory test data and are provided in this report regarding geotechnical conditions at this site and the effect on design and construction. The scope of our services consisted of the following:

- Field exploration and in-situ testing of the site soils at selected locations and depths.
- Laboratory testing for physical and/or chemical properties of selected samples.
- Review of the available literature and publications pertaining to local geology, faulting, and seismicity.
- Engineering analysis and evaluation of the data collected.
- Preparation of this report presenting our findings and professional opinions regarding the geotechnical aspects of project design and construction.

This report addresses the following geotechnical parameters:

- Subsurface soil and groundwater conditions
- Site geology, regional faulting and seismicity, near source factors, and site seismic accelerations
- Liquefaction potential and its mitigation
- Expansive soil and methods of mitigation
- Aggressive soil conditions to metals and concrete

Professional opinions with regard to the above parameters are provided for the following:

- Site grading and earthwork
- Building pad and foundation subgrade preparation
- Allowable soil bearing pressures and estimated settlements
- Concrete slabs-on-grade
- Excavation conditions and buried utility installations
- Mitigation of the potential effects of salt concentrations in native soil to concrete mixes and steel reinforcement
- Seismic design parameters
- Pavement structural sections

Our scope of work for this report did not include an evaluation of the site for the presence of environmentally hazardous materials or conditions, or landscape suitability of the soil.

1.3 Authorization

Ms. Kimberly Wender of Psomas provided authorization by email to proceed with our work on April 6, 2021. We conducted our work in general accordance with our written proposal dated January 18, 2021.

Section 2
METHODS OF INVESTIGATION

2.1 Field Exploration

Subsurface exploration was performed on March 4, 2021 using 2R Drilling of Chino, California to advance four (4) borings to depths of 5 to 51.5 feet below existing ground surface. The borings were advanced with a truck-mounted, CME 75 drill rig using 8-inch diameter, hollow-stem, continuous-flight augers. The approximate boring locations were established in the field and plotted on the site map by sighting to discernible site features. The boring locations are shown on the Site and Exploration Plan (Plate A-2).

A professional engineer observed the drilling operations and maintained logs of the soil encountered with sampling depths. Soils were classified during drilling according to the Unified Soil Classification System using the visual-manual procedure in accordance with ASTM D2488. Relatively undisturbed and bulk samples of the subsurface materials were obtained at selected intervals. The relatively undisturbed soil samples were retrieved using a 2-inch outside diameter (OD) split-spoon sampler or a 3-inch OD Modified California Split-Barrel (ring) sampler lined with 6-inch stainless-steel sleeves. In addition, Standard Penetration Tests (SPT) were performed in accordance with ASTM D1586 and ASTM D6066. The samples were obtained by driving the samplers ahead of the auger tip at selected depths using a 140-pound CME automatic hammer with a 30-inch drop. The number of blows required to drive the samplers the last 12 inches of an 18-inch drive depth into the soil is recorded on the boring logs as "blows per foot". Blow counts (N values) reported on the boring logs represent the field blow counts. No corrections have been applied to the blow counts shown on the boring logs for effects of overburden pressure, automatic hammer drive energy, drill rod lengths, liners, and sampler diameter. Pocket penetrometer readings were also obtained to evaluate the stiffness of cohesive soils retrieved from sampler barrels.

After logging and sampling the soil, the exploratory borings were backfilled with the excavated material. The backfill was loosely placed and was not compacted to the requirements specified for engineered fill. The existing asphalt surfaces were repaired with asphalt cold patch.

The subsurface logs are presented on Plates B-1 through B-4 in Appendix B. A key to the log symbols is presented on Plate B-5. The stratification lines shown on the subsurface logs represent the approximate boundaries between the various strata. However, the transition from one stratum to another may be gradual over some range of depth.

2.2 Laboratory Testing

Laboratory tests were conducted on selected bulk (auger cuttings) and relatively undisturbed soil samples obtained from the soil borings to aid in classification and evaluation of selected engineering properties of the site soils. The tests were conducted in general conformance to the procedures of the American Society for Testing and Materials (ASTM) or other standardized methods as referenced below. The laboratory testing program consisted of the following tests:

- Plasticity Index (ASTM D4318)
- Particle Size Analyses (ASTM D6913/D7928)
- Unit Dry Densities (ASTM D2937)
- Moisture Contents (ASTM D2216)
- Moisture-Density Relationship (ASTM D1557)
- R Value (CAL 301)
- Chemical Analyses (soluble sulfates & chlorides, pH, and resistivity) (Caltrans Methods)

The laboratory test results are presented on the subsurface logs (Appendix B) and in Appendix C.

Engineering parameters of soil strength, compressibility and relative density utilized for developing design criteria provided within this report were obtained from the field and laboratory testing program.

Section 3
DISCUSSION

3.1 Site Conditions

The project site is rectangular in plan view, elongated in the east-west direction. The project site has a vacant building (masonry and brick) located in the eastern portion of the site and paved and unpaved parking lots on the western portion. A small retail business (Precio Loco) is currently occupying the southeast corner of the project site. The subject site is bounded by 3rd Street on the north, Heffernan Avenue to the east, and Rockwood Avenue to the west. An alley forms the southern boundary of the subject site and separates the site from the commercial (retail) businesses to the south. Overhead power lines run along the north and south sides of the project site with several pole mounted electrical transformers located on power poles along the southern margin of the subject site (north side of the alley).

The project site is located within a commercial (retail) area of downtown Calexico, California. Adjacent properties are flat-lying and are approximately at the same elevation with this site and consist of various commercial businesses to the south across the alley, east, west and north. The project site lies at an elevation of approximately 5 feet above mean sea level (MSL) (El. 1005 local datum) in the Imperial Valley region of the California low desert. The surrounding properties lie on terrain which is flat (planar), part of a large agricultural valley, which was previously an ancient lake bed covered with fresh water to an elevation of 43± feet above MSL. Annual rainfall in this arid region is less than 3 inches per year with four months of average summertime temperatures above 100 ^{o}F. Winter temperatures are mild, seldom reaching freezing.

3.2 Geologic Setting

The project site is located in the Salton Trough region of the Colorado Desert physiographic province of southeastern California. The Salton Trough is a topographic and geologic structural depression resulting extending from the San Gorgonio Pass to the Gulf of California (Norris & Webb, 1990). The Salton Trough is bounded on the northeast by the San Andreas fault and Chocolate Mountains and the southwest by the Peninsular Range and faults of the San Jacinto Fault Zone. The Salton Trough represents the northward extension of the Gulf of California, containing both marine and non-marine sediments deposited since the Miocene Epoch (Morton, 1977).

Tectonic activity that formed the trough continues at a high rate as evidenced by deformed young sedimentary deposits and high levels of seismicity. Figure 1 shows the location of the site in relation to regional faults and physiographic features.

The Imperial Valley is directly underlain by lacustrine deposits, which consist of interbedded lenticular and tabular silt, sand, and clay. The Late Pleistocene to Holocene (present) lake deposits are probably less than 100 feet thick and derived from periodic flooding of the Colorado River which intermittently formed a fresh water lake (Lake Cahuilla). Older deposits consist of Miocene to Pleistocene non-marine and marine sediments deposited during intrusions of the Gulf of California. Basement rock consisting of Mesozoic granite and Paleozoic metamorphic rocks are estimated to exist at depths between 15,000 - 20,000 feet.

3.3 Subsurface Soil

The UC Davis California Soil Resource Lab "SoilWeb Earth" computer application (UC Davis, 2021) for Google Earth indicates that surficial deposits at the project site consist predominantly of silty clay loams overlying fine sands of the Imperial soil group (see Plate A-3). These loams are formed in sediment and alluvium of mixed origin (Colorado River overflows and fresh-water lake-bed sediments).

The subsurface soils encountered during the field exploration conducted on March 4, 2021 consist of interbedded silts and clays. A silty sand layer was encountered at 15 to 20 feet below ground surface (bgs). About 5 inches of asphaltic concrete was found at the paved area. **Hydrocarbon odors were noted in soil samples taken below a depth of 30 feet.** The subsurface logs (Plates B-1 through B-4) depict the stratigraphic relationships of the subsurface soil encountered at the boring locations. Variations in subsurface stratigraphy may occur between the points of exploration. The stratification lines shown on the subsurface log represent the approximate boundaries between the various strata. However, the transition from one stratum to another may be gradual over some range of depth.

The native surface clays (to 15 feet bgs) likely exhibit moderate to swell potential (Expansion Index, EI = 75-110) when correlated to Plasticity Index tests (ASTM D4318) performed on the native soils. The clay is expansive when wetted and can shrink with moisture loss (drying).

Development of building foundations and concrete flatwork should include provisions for mitigating potential swelling forces and reduction in soil strength, which can occur from saturation of the soil.

Causes for soil saturation include landscape irrigation, broken utility lines, or capillary rise in moisture upon sealing the ground surface to evaporation. Moisture losses can occur with lack of landscape watering, close proximity of structures to downslopes and root system moisture extraction from deep rooted shrubs and trees placed near the foundations. The design structural engineer (foundations) should consider the effects of non-uniform moisture conditions around the entire foundation when selecting design criteria for the foundations.

Typical measures used for similar projects to remediate expansive soil include:

- Replacement of expansive silts/clays (3.0 feet) with non-expansive sands or silts.
- Moisture conditioning subgrade soils to a minimum of 5% above optimum moisture (ASTM D1557) within the drying zone of surface soils.
- Capping silt/clay soil with a non-expansive sand layer of sufficient thickness (3.0 feet minimum) to reduce the effects of soil shrink/swell.
- Design of foundations that are resistant to shrink/swell forces of clay soil.
- A combination of the methods described above

3.4 Groundwater

Groundwater was encountered in Boring B-1 at about 49 feet at the time of exploration, but may rise with time to approximately 30 feet below ground surface at this site. There is uncertainty in the accuracy of short-term water level measurements, particularly in fine-grained soil. Groundwater levels may fluctuate with precipitation, irrigation of adjacent properties, site landscape watering, drainage, and site grading. The referenced groundwater level should not be interpreted to represent an accurate or permanent condition. Our work scope did not include a groundwater surface mounding study resulting from applied landscape water.

3.5 Faulting

The project site is located in the seismically active Imperial Valley of southern California with numerous mapped faults of the San Andreas Fault System traversing the region. The San Andreas Fault System is comprised of the San Andreas, San Jacinto, and Elsinore Fault Zones in southern California. The Imperial fault represents a transition from the more continuous San Andreas fault to a more nearly echelon pattern characteristic of the faults under the Gulf of California (USGS, 1990). We have performed a computer-aided search of known faults or seismic zones that lie within a 37.5 mile radius of the project site (Table 1).
A fault map illustrating known active faults relative to the site is presented on Figure 1, *Regional Fault Map*. Figure 2 shows the project site in relation to local faults. The criterion for fault classification adopted by the California Geological Survey defines Earthquake Fault Zones along Holocene-active or pre-Holocene faults (CGS, 2019b). Earthquake Fault Zones are regulatory zones that address the hazard of surface fault rupture.

A Holocene-active fault is one that has ruptured during Holocene time (within the last 11,700 years). A pre-Holocene fault is a fault that has not ruptured in the last 11,700 years. Pre-Holocene faults may still be capable of surface rupture in the future, but are not regulated by the Alquist-Priolo Act (AP). Review of the current Earthquake Fault Zone maps (CGS, 2019a) indicates that the nearest zoned fault is the Imperial fault located approximately 6.8 miles east of the project site.

3.6 General Ground Motion Analysis

The project site is considered likely to be subjected to moderate to strong ground motion from earthquakes in the region. Ground motions are dependent primarily on the earthquake magnitude and distance to the seismogenic (rupture) zone. Acceleration magnitudes also are dependent upon attenuation by rock and soil deposits, direction of rupture and type of fault; therefore, ground motions may vary considerably in the same general area.

<u>2019 CBC General Ground Motion Parameters:</u> The California Building Code (CBC) requires that a site-specific ground motion hazard analysis be performed in accordance with ASCE 7-16 Section 11.4.8 for structures on Site Class D and E sites with S_1 greater than or equal to 0.2 and Site Class E sites with S_s greater than or equal to 1.0.

This project site has been classified as Site Class D and has a S_1 value of 0.6, which would require a site-specific ground motion hazard analysis. However, ASCE 7-16 Section 11.4.8 provides three exceptions which permit the use of conservative values of design parameters for certain conditions for Site Class D and E sites in lieu of a site specific hazard analysis.

The exceptions are:

- Exception 1: Structures on Site Class E sites with S_s greater than or equal to 1.0, provided the site coefficient F_a is taken as equal to that of Site Class C.
- Exception 2: Structures on Site Class D sites with S_1 greater than or equal to 0.2, provided the value of the seismic response coefficient C_s is determined by Equations 12.8-2 for values of $T \leq 1.5T_S$ and taken as equal to 1.5 times the value computed in accordance with either Equation 12.8-3 for $T_L \geq T > 1.5T_S$ or Equation 12.8-4 for $T > T_L$.
- Exception 3: Structures on Site Class E sites with S_1 greater than or equal to 0.2, provided that T is less than or equal to T_S and the equivalent static force procedure is used for design.

The project design engineer should confirm that an exception applies to the project. If none of the exceptions apply, our office should be consulted to perform a site-specific ground motion hazard analysis.

The 2019 CBC general ground motion parameters are based on the Risk-Targeted Maximum Considered Earthquake (MCE_R). The Structural Engineers Association of California (SEAOC) and Office of Statewide Health Planning and Development (OSHPD) Seismic Design Maps Web Application (SEAOC, 2021) was used to obtain the site coefficients and adjusted maximum considered earthquake spectral response acceleration parameters.

Design spectral response acceleration parameters are defined as the earthquake ground motions that are two-thirds (2/3) of the corresponding MCE_R ground motions. The Maximum Considered Earthquake Geometric Mean (MCE_G) peak ground acceleration adjusted for soil site class effects (PGA_M) value to be used for liquefaction and seismic settlement analysis in accordance with 2019 CBC Section 1803.5.12 ($PGA_M = F_{PGA}$*PGA) is estimated at 0.62g for the project site. **Design earthquake ground motion parameters are provided in Table 2.**

3.7 Seismic and Other Hazards

- **Groundshaking.** The primary seismic hazard at the project site is the potential for strong groundshaking during earthquakes along the Imperial, Cerro Prieto, and Laguna Salada faults.
- **Surface Rupture.** The California Geological Survey (2019b) has established Earthquake Fault Zones in accordance with the 1972 Alquist-Priolo Earthquake Fault Zone Act. The Earthquake Fault Zones consists of boundary zones surrounding well defined, active faults or fault segments. The project site does not lie within an A-P Earthquake Fault Zone; therefore, surface fault rupture is considered to be low at the project site.
- **Liquefaction and lateral spreading.** Liquefaction is a potential design consideration because of underlying saturated sandy substrata. Although the Imperial Valley has not yet been evaluated for seismic hazards by the California Geological Survey seismic hazards zonation program, liquefaction is well documented in the Imperial Valley after strong seismic events (McCrink, et al, 2011 and Rymer et al, 2011). Liquefaction is unlikely to be a potential hazard at the site due to the lack of saturated granular soil (clay soils predominate).

<u>Other Potential Geologic Hazards.</u>

- **Landsliding.** The hazard of landsliding is unlikely due to the regional planar topography. No ancient landslides are shown on geologic maps, aerial photographs and topographic maps of the region and no indications of landslides were observed during our site investigation.
- **Volcanic hazards.** The site is not located proximal to any known volcanically active area and the risk of volcanic hazards is considered low.
- **Tsunamis and seiches.** Tsunamis are giant ocean waves created by strong underwater seismic events, asteroid impact, or large landslides. Seiches are large waves generated in enclosed bodies of water in response to strong ground shaking. The site is not located near any large bodies of water, so the threat of tsunami, seiches, or other seismically-induced flooding is considered unlikely.
- **Flooding.** Based on our review of FEMA (2008) FIRM Panel 06025C2100C which encompasses the project site, the project site is located in Flood Zone X, an area determined to be outside the 0.2% annual chance (500-year) floodplain.
- **Collapsible soils.** Collapsible soil generally consists of dry, loose, low-density material that have the potential collapse and compact (decrease in volume) when subjected to the addition of water or excessive loading. Soils found to be most susceptible to collapse include loess (fine grained wind-blown soils), young alluvium fan deposits in semi-arid to arid climates, debris flow deposits and residual soil deposits.

Due to the cohesive nature of the subsurface soils, the potential for hydro-collapse of the subsurface soils at this project site is considered very low.

- **Expansive soils.** In general, much of the near surface soils in the Imperial Valley consist of silty clays and clays which are moderate to highly expansive. The expansive soil conditions are discussed in more detail in Section 3.3.

3.8 Liquefaction

Liquefaction occurs when granular soils below the water table are subjected to vibratory motions, such as those produced by earthquakes. With strong ground shaking, the pore water pressure increases as the soil tends to reduce in volume. If the increase in pore water pressure is sufficient to reduce the vertical effective stress (suspending the soil particles in water), the soil strength decreases and the soil behaves as a liquid (similar to quicksand). Liquefaction can produce excessive settlement, ground rupture, lateral spreading, or failure of shallow bearing foundations.

Four conditions are generally required for liquefaction to occur:

(1) the soil must be saturated (relatively shallow groundwater);
(2) the soil must be loosely packed (low to medium relative density);
(3) the soil must be relatively cohesionless (not clayey); and
(4) groundshaking of sufficient intensity must occur to function as a trigger mechanism.

All of these conditions exist to some degree at this site.

The clay soil encountered at the points of exploration at the project site is not considered to be susceptible to liquefaction due to the high fines content and cohesive nature of the soil deposits.

<u>Methods of Analysis:</u> The liquefaction potential at the project site was evaluated using the 1997 NCEER Liquefaction Workshop and the Idriss and Boulanger (2008) methods. The 1997 NCEER methods utilize direct SPT blow counts from site exploration and earthquake magnitude/PGA estimates from the seismic hazard analysis. The resistance to liquefaction is plotted on a chart of cyclic shear stress ratio (CSR) versus a corrected blow count $N_{1(60)}$. The analysis was performed using a PGA_M value of 0.62g was used in the analysis with a 30-foot groundwater depth and a threshold factor of safety (FS) of 1.3.

The fines content of the liquefiable sands and silts increases their liquefaction resistance in that more ground motion cycles are required to fully develop the increased pore pressures. Prior to calculating the settlements, the field SPT blow counts were corrected to account for the type of hammer, borehole diameter, overburden pressure and rod length $N_{1(60)}$ in accordance with Idriss and Boulanger (2008). The corrected blow counts were then converted to equivalent clean sand blow counts ($N_{1(60)cs}$).

Liquefaction Induced Settlements: ***Based on empirical relationships, liquefaction induced settlements are not anticipated to occur at this site.***

Mitigation: Mitigation for liquefaction induced settlement is not required at this project site.

Section 4
DESIGN CRITERIA

4.1 Site Preparation

Preconstruction Meeting: A preconstruction conference should be held at the site prior to the beginning of grading operations with, as a minimum, the owner's representative, grading contractor and geotechnical engineer in attendance.

Clearing and Grubbing: All surface improvements, buildings and foundations, debris or vegetation on the site at the time of construction should be removed from the construction area. Root balls should be completely excavated. Organic strippings should be stockpiled and not used as engineered fill. All trash, construction debris, concrete slabs, old pavement, landfill, contaminated soil, and buried obstructions such as old foundations and utility lines exposed during rough grading should be traced to the limits of the foreign material by the grading contractor and removed under our supervision. Buildings currently exist on the eastern half of this site and previously have existed on the western portion of the site. If construction debris, undocumented fill or loose soils are encountered, additional removals will be required. ***Any excavations resulting from site clearing should be sloped to a bowl shape to the lowest depth of disturbance and backfilled under the observation of the geotechnical engineer's representative.***

Mass Grading (Non-Building Pad Areas): Prior to placing any import fills within the site, the native surface soil should be graded level. Subsequent to placing fill, the surface 12 inches of soil ***in areas planned for fill soil placement*** should be removed, the exposed surface uniformly moisture conditioned to a depth of 8 inches by discing and wetting to a minimum of optimum plus 4% and recompacted to minimum of 90% of ASTM D1557 maximum density. Onsite native clays placed as engineered fill should be uniformly moisture conditioned by discing and wetting or drying to optimum plus 4% and compacted in 6 inch maximum lifts to minimum 90% relative compaction. Clods shall be reduced by discing to a maximum dimension of 1.0 inch prior to being placed as fill.

Building Pad Preparation for Foundations Placed on Native Clay Soils: The existing soils within the building pad/foundation areas should be overexcavated to a minimum depth of 48 inches below the existing natural surface grade or 24 inches below the deepest footing (whichever is deeper) and should extend at least five (5) feet beyond all exterior wall/column lines (including concreted areas adjacent to the building). Exposed subgrade should be scarified to a depth of 8 inches, uniformly moisture conditioned to 5 to 10% above optimum moisture content and recompacted to 85 to 90% of the maximum density determined in accordance with ASTM D1557 methods.

The native soil is suitable for use as engineered fill provided it is free from concentrations of organic matter or other deleterious material. The fill soil should be uniformly moisture conditioned by discing and watering to the limits specified above, placed in maximum 8-inch lifts (loose), and compacted to the limits specified above. Clay soil should not be overcompacted because highly compacted soil will result in increased swelling.

Imported fill soil (for foundations designed for expansive soil conditions) should have a Plasticity Index less than 25 and sulfates (SO_4) less than 1,000 ppm.

Building Pad Preparation for Foundations Placed on Imported Non-expansive Soil: If foundation designs are to be utilized which do not include provisions for expansive soil, an engineered building support pad consisting of 3.0 feet of imported non-expansive soil should be used. The existing soils within the building pad/foundation areas should be overexcavated to a minimum depth of 36 inches below the existing natural surface grade or 18 inches below the deepest footing (whichever is deeper) and should extend at least five (5) feet beyond all exterior wall/column lines (including concreted areas adjacent to the building). The imported non-expansive fill material shall be placed in maximum 8-inch lifts (loose), compacted to a minimum of 90% of ASTM D1557 maximum density at 2% below to 4% above optimum moisture, should be placed below the bottom of the slab. The imported non-expansive soils should be placed over a minimum of 12 inches of uniformly moisture conditioned native clay soil (5-10% above optimum moisture content) which has been compacted to 85-90% of ASTM D1557 maximum dry density.

The imported soils should meet the USCS classifications of ML (non-plastic), SM, SP-SM, or SW-SM with a maximum rock size of 3 inches and no less than 5% passing the No. 200 sieve. The geotechnical engineer should approve imported fill soil sources before hauling material to the site. Imported fill should be placed in lifts no greater than 8 inches in loose thickness and compacted to a minimum of 90% of ASTM D1557 maximum dry density at optimum moisture ±2%.

Sidewalk and Concrete Hardscape Areas: In areas other than the building pad which are to receive sidewalks or area concrete slabs, the upper 12 inches should be removed and replaced with granular fill compacted to a minimum of 90% of ASTM D1557 maximum density. The exposed native soil scarified to 8 inches, moisture conditioned to a minimum of 5% over optimum, and recompacted to 85-90% of ASTM D1557 maximum density.

Moisture Control and Drainage: Adequate site drainage is essential to future performance of the project. Infiltration of excess irrigation water and stormwaters can adversely affect the performance of the subsurface soil at the site. Positive drainage should be maintained away from all structures (5% for 10 feet minimum across unpaved areas) to prevent ponding and subsequent saturation of the native clay soil.

Observation and Density Testing: All site preparation and fill placement should be continuously observed and tested by a representative of a qualified geotechnical engineering firm. Full-time observation services during the excavation and scarification process is necessary to detect undesirable materials or conditions and soft areas that may be encountered in the construction area. The geotechnical firm that provides observation and testing during construction shall assume the responsibility of "*geotechnical engineer of record*" and, as such, shall perform additional tests and investigation as necessary to satisfy themselves as to the site conditions and the geotechnical parameters for site development.

Auxiliary Structures Foundation Preparation: Auxiliary structures such as free standing or retaining walls should have footings extended to a minimum of 30 inches below grade. The existing soil beneath the structure foundation prepared in the manner described for the building pad except the preparation needed only to extend 18 inches below and beyond the footing.

4.2 Foundations and Settlements

Expansive Soil Engineered Building Pad: For foundations placed on an engineered building pad consisting of native clay soils, shallow spread or continuous footings are suitable to support the building provided they are structurally tied with grade-beams to continuous perimeter wall footings to resist differential movement associated with expansive soils and potential soil liquefaction at depth. A minimum of 12 inches of compacted fill should exist beneath the footings.

Continuous wall footings should have a minimum depth of 24 inches and minimum width of 12 inches. Spread footings should have a minimum dimension of 24 inches and should be structurally tied to perimeter footings or grade beams. Concrete reinforcement and sizing for all footings should be provided by the structural engineer.

The foundations may be designed using an allowable soil bearing pressure of 1,500 psf for compacted native clay soil. The allowable soil pressure may be increased by 20% for each foot of embedment depth of the footings in excess of 18 inches and by one-third for short term loads induced by winds or seismic events. The maximum allowable soil pressure at increased embedment depths shall not exceed 3,000 psf (clays).

Resistance to horizontal loads will be developed by passive earth pressure on the sides of footings and frictional resistance developed along the bases of footings and concrete slabs. Passive resistance to lateral earth pressure may be calculated using an equivalent fluid pressure of 250 pcf to resist lateral loadings. The top one foot of embedment should not be considered in computing passive resistance unless the adjacent area is confined by a slab or pavement. An allowable friction coefficient of 0.25 may also be used at the base of the footings to resist lateral loading.

Flat plate structural mats, grade-beam reinforced foundations, or post tensioned reinforced foundations may be used to mitigate expansive soil heave and/or liquefaction related movement.

- <u>Flat Plate Structural Mats</u>: Flat plate structural mats may be used to mitigate expansive soils at the project site. The structural mat shall have a double mat of steel (minimum No. 4's @ 12 inches O.C. each way – top and bottom) and a minimum thickness of 10 inches. Mat edges shall have a minimum edge footing of 12 inches width and 24 inches depth (below the building pad surface). Mats may be designed by CBC Chapter 18, Section 1808.6.2 methods (*WRI/CRSI Design of Slab-on-Ground Foundations*).

  Structural mats may be designed for a modulus of subgrade reaction (Ks) of 50 pci when placed on compacted clay or a subgrade modulus of 300 pci when placed on 3.0 feet of granular fill. Mats shall overlay 2 inches of sand and a 10-mil polyethylene vapor retarder. The building support pad shall be moisture conditioned and recompacted as specified in Section 4.1 of this report.

- Grade-beam Reinforced Foundations: Specific soil data for structures with grade-beam reinforced foundations placed on the native clays are presented below in accordance with the design method given in CBC Chapter 18 Section 1808.6.2 (*WRI/CRSI Design of Slab-on-Ground Foundations*):

  Weighted Plasticity Index (PI) = 33
  Slope Coefficient (C_s) = 1.0
  Strength Coefficient (C_o) = 0.8
  Climatic Rating (C_w) = 15
  Effective PI = 26
  Maximum Grade-beam Spacing = 19 feet

  Exterior footings shall be founded a minimum of 24 inches below the surface of the building support pad on a layer of properly prepared and compacted native soil as described in Section 4.1. Interior footings shall have a minimum embedment depth of 12 inches.

Non-expansive Soil Engineered Building Pad: Shallow spread or continuous conventional footings are suitable to support the building. Exterior footings shall be founded a minimum of 18 inches below the surface of the building support pad when supported on a non-expansive granular fill as described in Section 4.1. Interior footings shall have a minimum embedment depth of 12 inches.

The foundations may be designed using an allowable soil bearing pressure of 2,500 psf when foundations are supported on imported granular soils (extending a minimum of 1.5 feet below footings). Column footings with greater than 50 kip loading shall have 3 feet of engineered fill below and beyond (laterally) the footing. The allowable soil pressure may be increased by 20% for each foot of embedment depth of the footings in excess of 18 inches and by one-third for short term loads induced by winds or seismic events. The maximum allowable soil pressure at increased embedment depths shall not exceed 3,000 psf.

Resistance to horizontal loads will be developed by passive earth pressure on the sides of footings and frictional resistance developed along the bases of footings and concrete slabs. Passive resistance to lateral earth pressure may be calculated using an equivalent fluid pressure of 300 pcf to resist lateral loadings. The top one foot of embedment should not be considered in computing passive resistance unless the adjacent area is confined by a slab or pavement. An allowable friction coefficient of 0.35 may also be used at the base of the footings to resist lateral loading.

Settlements: Foundation movement under the estimated static (non-seismic) loadings and static site conditions are estimated to not exceed 1 inch with differential movement of about two-thirds of total movement for the loading assumptions stated above when the subgrade preparation guidelines given above are followed. Seismically induced liquefaction settlement is not expected to occur at this project site.

4.3 Slabs-On-Grade

Structural Concrete: Structural concrete slabs are those slabs (foundations) that underlie structures or patio covers (shades). These slabs that are placed over native clay soil should be designed in accordance with Chapter 18 of the 2019 CBC and shall be a minimum of 5 inches thick due to expansive soil conditions. Concrete floor slabs shall be monolithically placed with the footings (no cold joints) unless placed on 2.5 feet of granular fill.

American Concrete Institute (ACI) guidelines (ACI 302.1R-04 Chapter 3, Section 3.2.3) provide recommendations regarding the use of moisture barriers beneath concrete slabs. The concrete floor slabs should be underlain by a 10-mil polyethylene vapor retarder that works as a capillary break to reduce moisture migration into the slab section. All laps and seams should be overlapped 6-inches or as recommended by the manufacturer. The vapor retarder should be protected from puncture. The joints and penetrations should be sealed with the manufacturer's recommended adhesive, pressure-sensitive tape, or both. The vapor retarder should extend a minimum of 12 inches into the footing excavations. The vapor retarder should be covered by 4 inches of clean sand (Sand Equivalent SE>30) unless placed on 3.0 feet of granular fill, in which case, the vapor retarder may lie directly on the granular fill with 2 inches of clean sand cover.

Placing sand over the vapor retarder may increase moisture transmission through the slab, because it provides a reservoir for bleed water from the concrete to collect. The sand placed over the vapor retarder may also move and mound prior to concrete placement, resulting in an irregular slab thickness. For areas with moisture sensitive flooring materials, ACI recommends that concrete slabs be placed without a sand cover directly over the vapor retarder, provided that the concrete mix uses a low-water cement ratio and concrete curing methods are employed to compensate for release of bleed water through the top of the slab. The vapor retarder should have a minimum thickness of 15-mil (Stego-Wrap or equivalent).

Structural concrete slab reinforcement should consist of chaired rebar slab reinforcement (minimum of No. 3 bars at 16-inch centers, both horizontal directions) placed at slab mid-height to resist potential swell forces and cracking. Slab thickness and steel reinforcement are minimums only and should be verified by the structural engineer/designer knowing the actual project loadings. All steel components of the foundation system should be protected from corrosion by maintaining a 3-inch minimum concrete cover of densely consolidated concrete at footings (by use of a vibrator).

The construction joint between the foundation and any mowstrips/sidewalks placed adjacent to foundations should be sealed with a polyurethane based non-hardening sealant to prevent moisture migration between the joint. Epoxy coated embedded steel components (ASTM D3963/A934) or permanent waterproofing membranes placed at the exterior footing sidewall may also be used to mitigate the corrosion potential of concrete placed in contact with native soil.

Control joints should be provided in all concrete slabs-on-grade at a maximum spacing (in feet) of 2 to 3 times the slab thickness (in inches) as recommended by American Concrete Institute (ACI) guidelines. All joints should form approximately square patterns to reduce randomly oriented contraction cracks. Contraction joints in the slabs should be tooled at the time of the pour or sawcut (¼ of slab depth) within 6 to 8 hours of concrete placement. Construction (cold) joints in foundations and area flatwork should either be thickened butt-joints with dowels or a thickened keyed-joint designed to resist vertical deflection at the joint. All joints in flatwork should be sealed to prevent moisture, vermin, or foreign material intrusion. Precautions should be taken to prevent curling of slabs in this arid desert region (refer to ACI guidelines).

Non-structural Concrete: All non-structural independent flatwork (sidewalks, hardscape and uncovered patios) should be placed on a minimum of 12 inches of concrete sand or aggregate base compacted to a minimum of 90% of ASTM D1557 maximum density. The flatwork shall be dowelled to the perimeter foundations where adjacent to the building to prevent separation and sloped 2% (sidewalks) or 1 to 2% (patios) away from the building. Patio slabs with shade structures shall have a perimeter footing (18-inch embedment depth) and shall have interior grade beams (12-inch minimum embedment depth) at 15 feet on center. Planters that trap water between sidewalks and foundations are not allowed.

Flatwork which contains steel reinforcing (except wire mesh) should be underlain by a 10-mil (minimum) polyethylene separation sheet and at least a 2-inch sand cover. All flatwork should be jointed in square patterns and at irregularities in shape at a maximum spacing of 8 feet or the least width of the sidewalk.

4.4 Shade Structure Foundations

Shallow spread footings or individual concrete short drilled piers are suitable to support the shade canopy structures.

Spread Footings: Spread footings may be used to support the shade canopy structures. The spread footing foundation shall be founded on a layer of properly prepared and compacted soil as described above. Spread footings should have a minimum horizontal dimension of 36 inches. Concrete reinforcement and sizing for all footings should be provided by the structural engineer.

Resistance to horizontal loads will be developed by passive earth pressure on the sides of footings and frictional resistance developed along the bases of footings. Passive resistance to lateral earth pressure may be calculated using an equivalent fluid pressure of 250 pcf to resist lateral loadings. The top one foot of embedment should not be considered in computing passive resistance unless the adjacent area is confined by a slab or pavement. An allowable friction coefficient of 0.25 (clay) may also be used at the base of the footings to resist lateral loading. Native clay soils unit weight may be about 125 pcf for saturated unit weight. A modulus of subgrade reaction (Ks) of 150 pci may be used.

Drilled pier foundation: Individual short piers should be adequate to support the shade canopy structure. Non-constrained and constrained design parameters are provided below.

Non-constrained: Embedment depth for short piers to resist lateral loads where no-constraint is provided at ground surface may be designed using the following formula per 2019 CBC Section 1807.3.2.1:

$$d = 0.5A\,[1 + (1+4.36h/A)^{1/2}] \qquad \text{(Equation 18-1)}$$

where:

$A = 2.34P/S_1 b$
b = Pier diameter in feet
d = Embedment depth in feet (but not over 12 feet for purpose of computing lateral pressure)
h = Distance in feet from ground surface to point of application of "P"
P = Applied lateral force in pounds
S_1 = Allowable lateral soil bearing pressure (basic value of 100 psf/f (see 2019 CBC Table 1806.2). Isolated piers that are not adversely affected by a 0.5 inch motion at the ground surface due to short-term lateral loads are permitted to be designed using lateral soil bearing pressures equal to two times the basic soil bearing value.

The short pier foundations may be designed using an allowable soil bearing pressure of 1,500 psf for the native soils and a cohesion of 130 psf for the native clay soil. The cohesion value shall be multiplied by the contact area, as limited by Section 1806.3 of the 2019 CBC. Uplift capacity may be determined by using ⅔ of the cohesion value.

The short pier foundations may be designed using an allowable soil bearing pressure of 1,500 psf for the native soils and a coefficient of friction of 0.25 for the native sand soils. The coefficient of friction may be multiplied by the dead load, as limited by Section 1806.3 of the 2019 CBC.

The uplift capacity may be may be defined as the sum of the frictional resistance of the soils against the concrete pile plus the weight of the pile as follows:

$$P_{all} = (K_{HT}{*}P_o{*}\mathrm{Tan}\,\delta{*}\pi{*}D{*}H)/FS + W_p,$$

Incorporating the soil conditions at the site and applying a Safety Factor of 3 it may be expressed as,

$$P_{all} = 16DH^2 + Wp$$

where:

P_{all} = Allowable Uplift Capacity in pounds
D = Diameter of the pile in feet
H = Depth of embedment below ground surface in feet (to a maximum of 14 feet)
W_p = Weight of the pile in pounds

Constrained: The following formula (2019 CBC Section 1807.3.2.2) shall be used to determine the depth of embedment required to resist lateral loads where lateral constrain is provided at the ground surface, such as by rigid floor or pavement.

$$d = \sqrt{(4.25Ph / S_3b)} \text{ or alternatively, } d = \sqrt{(4.25Mg / S_3b)}$$

where:

b = Pier diameter in feet.
d = Embedment depth in feet (but not over 12 feet for purpose of computing lateral pressure).
h = Distance in feet from ground surface to point of application of "P".
P = Applied lateral force in pounds.
S_3 = Allowable lateral soil bearing pressure (basic value of 100 psf, see 2019 CBC Table 1806.2) based on a depth equal to the depth of embedment in psf. This value may be doubled where ½ inch deflection at ground surface is allowed due to short-term lateral loads.
Mg = Moment in the post at grade in ft-lb.

The vertical and uplift load capacities may be determined as noted for the unconstrained case.

4.4 Concrete Mixes and Corrosivity

Selected chemical analyses for corrosivity were conducted on bulk samples of the near surface soil from the project site (Plate C-3). The native soils were found to have S0 (low levels of sulfate ion concentration (363 to 790 ppm). Sulfate ions in high concentrations can attack the cementitious material in concrete, causing weakening of the cement matrix and eventual deterioration by raveling. The following table provides American Concrete Institute (ACI) recommended cement types, water-cement ratio and minimum compressive strengths for concrete in contact with soils:

Concrete Mix Design Criteria due to Soluble Sulfate Exposure

Sulfate Exposure Class	Water-soluble Sulfate (SO_4) in soil, ppm	Cement Type	Maximum Water-Cement Ratio by weight	Minimum Strength f'c (psi)
S0	0-1,000	–	–	–
S1	1,000-2,000	II	0.50	4,000
S2	2,000-20,000	V	0.45	4,500
S3	Over 20,000	V (plus Pozzolon)	0.45	4,500

Note: From ACI 318-14 Table 19.3.1.1 and Table 19.3.2.1

However, in consideration of general corrosive environment in the vicinity, a minimum of 4,000 psi concrete of Type V Portland Cement with a maximum water-cement ration of 0.50 (by weight) should be placed in contact with native soil on this project (sitework including sidewalks, hardscape, and foundations).

The native soil has moderate levels of chloride ion concentration (280 to 460 ppm). Chloride ions can cause corrosion of reinforcing steel, anchor bolts and other buried metallic conduits. Resistivity determinations on the soil indicate very severe potential for metal loss because of electrochemical corrosion processes. Mitigation of the corrosion of steel can be achieved by using steel pipes coated with epoxy corrosion inhibitors, asphaltic and epoxy coatings, cathodic protection or by encapsulating the portion of the pipe lying above groundwater with a minimum of 3 inches of densely consolidated concrete. ***No metallic water pipes or conduits should be placed below foundations.***

Foundation designs shall provide a minimum concrete cover of three (3) inches around steel reinforcing or embedded components (anchor bolts, etc.) exposed to native soil or landscape water (to 18 inches above grade). If the 3-inch concrete edge distance cannot be achieved, all embedded steel components (anchor bolts, etc.) shall be epoxy coated for corrosion protection (in accordance with ASTM D3963/A934) or a corrosion inhibitor and a permanent waterproofing membrane shall be placed along the exterior face of the exterior footings. ***Hold-down straps should not be used at foundation edges due to corrosion of metal at its protrusion from the slab edge.*** Additionally, the concrete should be thoroughly vibrated at footings during placement to decrease the permeability of the concrete.

Exterior foundation faces exposed to native soils (without adjacent mowstrips, sidewalks, or patios) should be coated with a permanent waterproofing membrane to prevent salt migration into concrete.

Copper water piping (except for trap primers) should not be placed under floor slabs. All copper piping within 18 inches of ground surface shall be sleeved or wrapped with two layers of 10 mil plumbers tape or sleeved with PVC piping to prevent contact with soil. The trap primer pipe shall be completely encapsulated in a PVC sleeve and Type K copper should be utilized if polyethylene tubing cannot be used. Pressurized waterlines are not allowed under the floor slab. Fire protection piping (risers) should be placed outside of the building foundation.

Landmark does not practice corrosion engineering. We recommend that a qualified corrosion engineer evaluate the corrosion potential on metal construction materials and concrete at the site to obtain final design recommendations.

4.5 Excavations

All site excavations should conform to CalOSHA requirements for Type B soil. The contractor is solely responsible for the safety of workers entering trenches. Temporary excavations with depths of 4 feet or less may be cut nearly vertical for short duration. Excavations deeper than 4 feet will require shoring or slope inclinations in conformance to CAL/OSHA regulations for Type B soil. Surcharge loads of stockpiled soil or construction materials should be set back from the top of the slope a minimum distance equal to the height of the slope.

All permanent slopes should not be steeper than 3:1 to reduce wind and rain erosion. Protected slopes with ground cover may be as steep as 2:1. However, maintenance with motorized equipment may not be possible at this inclination.

4.6 Utility Trench Backfill

<u>Utility Trench Backfill:</u> Prior to placement of utility bedding, the exposed subgrade at the bottom of trench excavations should be examined for soft, loose, or unstable soil. Loose materials at trench bottoms resulting from excavation disturbance should be removed to firm material. If extensive soft or unstable areas are encountered, these areas should be over-excavated to a depth of at least 2 feet or to a firm base and be replaced with additional bedding material.

<u>Backfill Materials:</u> Pipe zone backfill (i.e., material beneath and in the immediate vicinity of the pipe) should consist of a 4 to 8 inch bed of ⅜-inch crushed rock, sand/cement slurry (3 sack cement factor), and/or crusher fines (sand) extending to a minimum of 12 inches above the top of pipe. If crushed rock is used for pipe zone backfill for utilities, the crushed rock material should be completed surrounded by a non-woven filter fabric such as Mirafi 140N or equivalent. The filter fabric shall cover the trench bottom, sidewalls and over the top of the crushed rock. The filter fabric is recommended to inhibit the migration of fine material into void spaces in the crushed rock which may create the potential for sinkholes or depressions to develop at the ground surface.

Pipe bedding should be in accordance with pipe manufacturer's recommendations. Recommendations provided above for pipe zone backfill are minimum requirements only. More stringent material specifications may be required to fulfill local codes and/or bedding requirements for specific types of pipes. On-site soil free of debris, vegetation, and other deleterious matter may be suitable for use as utility trench backfill above pipezone, but may be difficult to uniformly maintain at specified moistures and compact to the specified densities. Native backfill should only be placed and compacted after encapsulating buried pipes with suitable bedding and pipe envelope material.

Compaction Criteria: Mechanical compaction is recommended; ponding or jetting should not be allowed, especially in areas supporting structural loads or beneath concrete slabs supported-on-grade, pavements, or other improvements. All trench backfill should be placed and compacted in accordance with recommendations provided above for engineered fill.
The pipe zone material (crusher fines, sand) shall be compacted to a minimum of 95% of ASTM D1557 maximum density. Pipe deflection should be checked to not exceed 2% of pipe diameter. Native clay/silt soils may be used to backfill the remainder of the trench. Soils used for trench backfill shall be placed in maximum 6 inch lifts (loose), compacted to a minimum of 90% of ASTM D1557 maximum density at a minimum of 4% above optimum moisture.

Imported granular material is acceptable for backfill of utility trenches. Granular trench backfill used in building pad areas should be plugged with a solid (no clods or voids) 2-foot width of native clay soils at each end of the building foundation to prevent landscape water migration into the trench below the building.

Backfill soil of utility trenches within paved areas should be uniformly moisture conditioned to a minimum of 4% above optimum moisture, placed in layers not more than 6 inches in thickness and mechanically compacted to a minimum of 90% of the ASTM D1557 maximum dry density, except that the top 12 inches shall be compacted to 95% (if granular trench backfill).

4.8 Seismic Design

This site is located in the seismically active southern California area and the site structures are subject to strong ground shaking due to potential fault movements along the Imperial, Cerro Prieto, and Laguna Salada faults. Engineered design and earthquake-resistant construction are the common solutions to increase safety and development of seismic areas. Designs should comply with the latest edition of the CBC for Site Class D using the seismic coefficients given in Section 3.6 and Table 2 of this report.

4.9 Pavements

Pavements should be designed according to the 2020 Caltrans Highway Design Manual or other acceptable methods. Traffic indices were not provided by the project engineer or owner; therefore, we have provided structural sections for several traffic indices for comparative evaluation. The public agency or design engineer should decide the appropriate traffic index for the site. Maintenance of proper drainage is necessary to prolong the service life of the pavements.

Based on the current Caltrans method, an R-value of 5 for the subgrade soil and assumed traffic indices, the following table provides our estimates for asphaltic concrete (AC) and Portland Cement Concrete (PCC) pavement sections.

Pavement Structural Sections

R-Value of Subgrade Soil - 5 Design Method - Caltrans 2020

Traffic Index	Flexible Pavements		Rigid (PCC) Pavements	
	Asphaltic Concrete Thickness (in.)	Aggregate Base Thickness (in.)	Concrete Thickness (in.)	Aggregate Base Thickness (in.)
4.0	3.0	6.5	5.0	6.0
5.0	3.0	10.0	5.5	6.0
6.0	4.0	11.5	6.0	8.0
6.5	4.0	14.0	7.0	8.0
8.0	5.0	17.5	8.0	11.0

Notes:

1) Asphaltic concrete shall be Caltrans, Type A HMA (Hot Mix Asphalt), ¾ inch maximum (½ inch maximum for parking areas), with PG70-10 asphalt concrete, compacted to a minimum of 95% of the Hveem density (CAL 308) or a minimum of 92% of the Maximum Theoretical Density (ASTM D2041).
2) Aggregate base shall conform to Caltrans Class 2 (¾ in. maximum), compacted to a minimum of 95% of ASTM D1557 maximum dry density.
3) Place pavements on 12 inches of moisture conditioned (minimum 4% above optimum if clays) native clay soil compacted to a minimum of 90% (95% if sand subgrade) of the maximum dry density determined by ASTM D1557. Prewetting of subgrade soils (to 3.5 feet) may be required depending on moisture of subgrade at time of aggregate base placement.
4) Portland cement concrete for pavements should have Type V cement, a minimum compressive strength of 4,500 psi at 28 days, and a maximum water-cement ratio of 0.45.
5) Typical Street Classifications (Imperial County).

Parking Areas:	TI = 4.0
Cul-de-Sacs:	TI = 5.0
Local Streets:	TI = 6.0
Minor Collectors:	TI = 6.5 (Heavier Traffic Areas)
Major Collectors:	TI = 8.0

Section 5
LIMITATIONS AND ADDITIONAL SERVICES

5.1 Limitations

The findings and professional opinions within this report are based on current information regarding the proposed Calexico Transit Center located at the southwest corner of 3rd Street and Heffernan Avenue in Calexico, California. The conclusions and professional opinions of this report are invalid if:

- Structural loads change from those stated or the structures are relocated.
- The Additional Services section of this report is not followed.
- This report is used for adjacent or other property.
- Changes of grade or groundwater occur between the issuance of this report and construction other than those anticipated in this report.
- Any other change that materially alters the project from that proposed at the time this report was prepared.

This report was prepared according to the generally accepted *geotechnical engineering standards of practice* that existed in Imperial County at the time the report was prepared. No express or implied warranties are made in connection with our services.

Findings and professional opinions in this report are based on selected points of field exploration, geologic literature, limited laboratory testing, and our understanding of the proposed project. Our analysis of data and professional opinions presented herein are based on the assumption that soil conditions do not vary significantly from those found at specific exploratory locations. Variations in soil conditions can exist between and beyond the exploration points or groundwater elevations may change. The nature and extend of such variations may not become evident until, during or after construction. If variations are detected, we should immediately be notified as these conditions may require additional studies, consultation, and possible design revisions.

Environmental or hazardous materials evaluations were not performed by Landmark for this project. Landmark will assume no responsibility or liability whatsoever for any claim, damage, or injury which results from pre-existing hazardous materials being encountered or present on the project site, or from the discovery of such hazardous materials.

The client has responsibility to see that all parties to the project including designer, contractor, and subcontractor are made aware of this entire report within a reasonable time from its issuance. This report should be considered invalid for periods after two years from the date of report issuance without a review of the validity of the findings and professional opinions by our firm, because of potential changes in the Geotechnical Engineering Standards of Practice. This report is based upon government regulations in effect at the time of preparation of this report. Future changes or modifications to these regulations may require modification of this report. Land or facility use, on and off-site conditions, regulations, design criteria, procedures, or other factors may change over time, which may require additional work. Any party other than the client who wishes to use this report shall notify Landmark of such intended use. Based on the intended use of the report, Landmark may require that additional work be performed and that an updated report be issued. Non-compliance with any of these requirements by the client or anyone else will release Landmark from any liability resulting from the use of this report by any unauthorized party and client agrees to defend, indemnify, and hold Landmark harmless from any claim or liability associated with such unauthorized use or non-compliance.

This report contains information that may be useful in the preparation of contract specifications. However, the report is not worded is such a manner that we recommend its use as a construction specification document without proper modification. The use of information contained in this report for bidding purposes should be done at the contractor's option and risk.

5.2 Plan Review

Landmark Consultants, Inc. should be retained during development of design and construction documents to check that the geotechnical professional opinions are appropriate for the proposed project and that the geotechnical professional opinions are properly interpreted and incorporated into the documents. Landmark should have the opportunity to review the final design plans and specifications for the project prior to the issuance of such for bidding.

Governmental agencies may require review of the plans by the geotechnical engineer of record for compliance to the geotechnical report.

5.3 Additional Services

We recommend that Landmark Consultant be retained to provide the tests and observations services during construction. *The geotechnical engineering firm providing such tests and observations shall become the geotechnical engineer of record and assume responsibility for the project.*

Landmark Consultants, Inc. professional opinions for this site are, to a high degree, dependent upon appropriate quality control of subgrade preparation, fill placement, and foundation construction. Accordingly, the findings and professional opinions in this report are made contingent upon the opportunity for Landmark Consultants to observe grading operations and foundation excavations for the proposed construction.

If parties other than Landmark Consultants, Inc. are engaged to provide observation and testing services during construction, such parties must be notified that they will be required to assume complete responsibility as the geotechnical engineer of record for the geotechnical phase of the project by concurring with the professional opinions in this report and/or by providing alternative professional guidance.

Additional information concerning the scope and cost of these services can be obtained from our office.

Section 6
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TABLES

Table 1
Summary of Characteristics of Closest Known Active Faults

Fault Name	Approximate Distance (miles)	Approximate Distance (km)	Maximum Moment Magnitude (Mw)	Fault Length (km)	Slip Rate (mm/yr)
Imperial	6.8	10.9	7	62 ± 6	20 ± 5
Rico *	10.7	17.1			
Unnamed 2*	10.7	17.1			
Brawley *	11.2	18.0			
Cerro Prieto *	11.4	18.3			
Borrego (Mexico)*	12.2	19.5			
Superstition Hills	12.4	19.9	6.6	23 ± 2	4 ± 2
Laguna Salada	13.7	21.9	7	67 ± 7	3.5 ± 1.5
Pescadores (Mexico)*	14.5	23.1			
Unnamed 1*	15.0	23.9			
Cucapah (Mexico)*	15.2	24.3			
Yuha*	16.0	25.6			
Superstition Mountain	18.7	29.9	6.6	24 ± 2	5 ± 3
Shell Beds	20.3	32.5			
Yuha Well *	21.0	33.6			
Vista de Anza*	23.0	36.8			
Painted Gorge Wash*	27.5	44.0			
Ocotillo*	28.4	45.4			
Elsinore - Coyote Mountain	32.2	51.5	6.8	39 ± 4	4 ± 2
Elmore Ranch	32.3	51.7	6.6	29 ± 3	1 ± 0.5
Algodones *	34.2	54.7			
San Jacinto - Borrego	37.5	60.0	6.6	29 ± 3	4 ± 2

* Note: Faults not included in CGS database.

Table 2
2019 California Building Code (CBC) and ASCE 7-16 Seismic Parameters

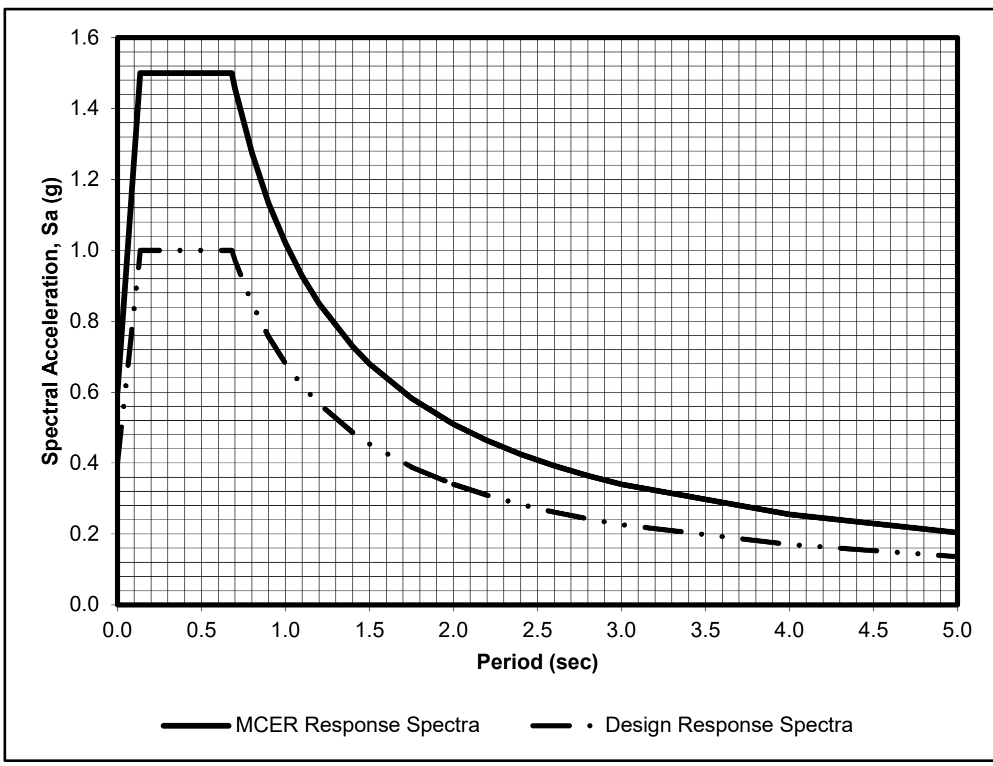
		ASCE 7-16 Reference
Soil Site Class:	**D**	Table 20.3-1
Latitude:	32.6675 N	
Longitude:	-115.4956 W	
Risk Category:	II	
Seismic Design Category:	D	

Maximum Considered Earthquake (MCE) Ground Motion

Mapped MCE_R Short Period Spectral Response	$\mathbf{S_s}$	1.500 g	ASCE Figure 22-1	
Mapped MCE_R 1 second Spectral Response	$\mathbf{S_1}$	0.600 g	ASCE Figure 22-2	
Short Period (0.2 s) Site Coefficient	$\mathbf{F_a}$	1.00	ASCE Table 11.4-1	
Long Period (1.0 s) Site Coefficient	$\mathbf{F_v}$	1.70	ASCE Table 11.4-2	
MCE_R Spectral Response Acceleration Parameter (0.2 s)	$\mathbf{S_{MS}}$	1.500 g	$= Fa * S_s$	ASCE Equation 11.4-1
MCE_R Spectral Response Acceleration Parameter (1.0 s)	$\mathbf{S_{M1}}$	1.020 g	$= Fv * S_1$	ASCE Equation 11.4-2

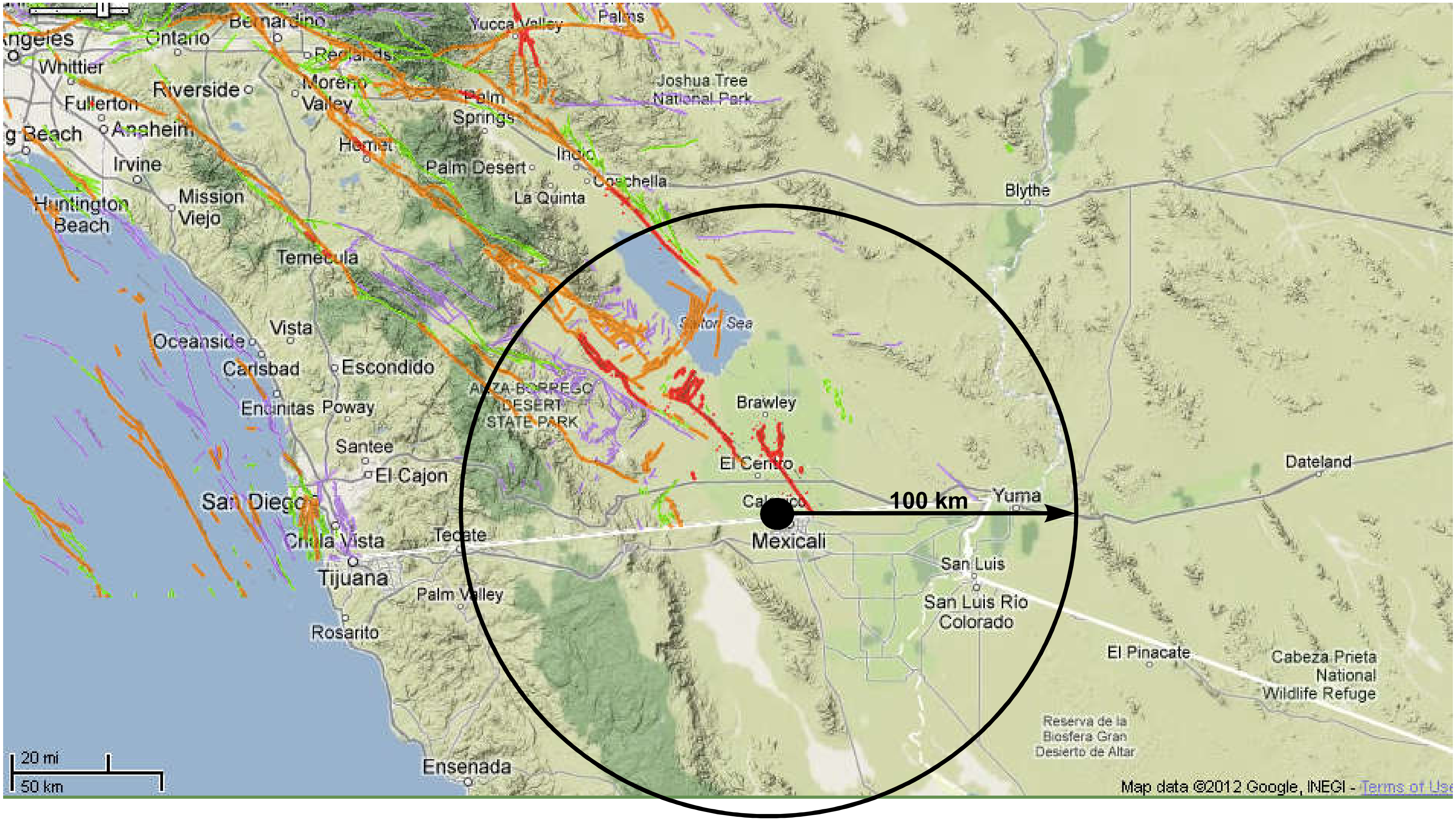
Design Earthquake Ground Motion

Design Spectral Response Acceleration Parameter (0.2 s)	$\mathbf{S_{DS}}$	1.000 g	$= 2/3*S_{MS}$	ASCE Equation 11.4-3
Design Spectral Response Acceleration Parameter (1.0 s)	$\mathbf{S_{D1}}$	0.680 g	$= 2/3*S_{M1}$	ASCE Equation 11.4-4
Risk Coefficient at Short Periods (less than 0.2 s)	$\mathbf{C_{RS}}$	0.945		ASCE Figure 22-17
Risk Coefficient at Long Periods (greater than 1.0 s)	$\mathbf{C_{R1}}$	0.921		ASCE Figure 22-18
	$\mathbf{T_L}$	8.00 sec		ASCE Figure 22-12
	$\mathbf{T_O}$	0.14 sec	$=0.2*S_{D1}/S_{DS}$	
	$\mathbf{T_S}$	0.68 sec	$=S_{D1}/S_{DS}$	
Peak Ground Acceleration	$\mathbf{PGA_M}$	0.62 g		ASCE Equation 11.8-1



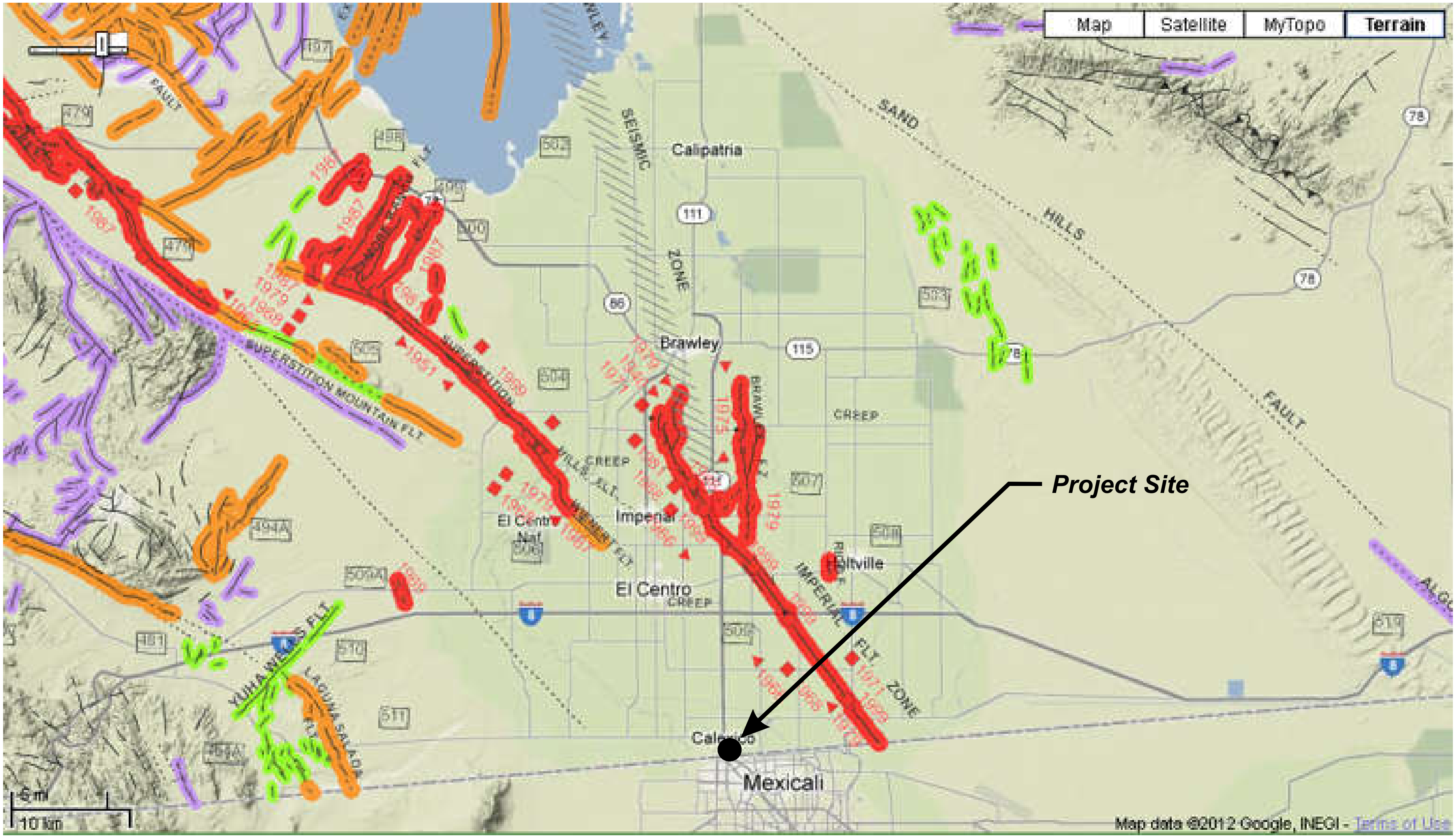
Period T (sec)	Sa (g)	MCE_R Sa (g)
0.00	0.40	0.60
0.14	1.00	1.50
0.68	1.00	1.50
0.70	0.97	1.46
0.80	0.85	1.28
0.90	0.76	1.13
1.00	0.68	1.02
1.10	0.62	0.93
1.20	0.57	0.85
1.20	0.57	0.85
1.40	0.49	0.73
1.50	0.45	0.68
1.75	0.39	0.58
2.00	0.34	0.51
2.20	0.31	0.46
2.40	0.28	0.43
2.60	0.26	0.39
2.80	0.24	0.36
3.00	0.23	0.34
4.00	0.17	0.26
5.00	0.14	0.20

FIGURES



Source: California Geological Survey 2010 Fault Activity Map of California
http://www.quake.ca.gov/gmaps/FAM/faultactivitymap.html#

LANDMARK Geo-Engineers and Geologists **Project No.: LE21050**	**Regional Fault Map**	**Figure 1**



Source: California Geological Survey 2010 Fault Activity Map of California
http://www.quake.ca.gov/gmaps/FAM/faultactivitymap.html#

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Map of Local Faults

Figure 2

EXPLANATION

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

FAULT CLASSIFICATION COLOR CODE
(Indicating Recency of Movement)

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

(a) a recorded earthquake with surface rupture. (Also included are some well-defined surface breaks caused by ground shaking during earthquakes, e.g. extensive ground breakage, not on the White Wolf fault, caused by the Arvin-Tehachapi earthquake of 1952). The date of the associated earthquake is indicated. Where repeated surface ruptures on the same fault have occurred, only the date of the latest movement may be indicated, especially if earlier reports are not well documented as to location of ground breaks.

(b) fault creep slippage - slow ground displacement usually without accompanying earthquakes.

(c) displaced survey lines.

1906 ▶ ◀ 1906
1838 ▷ ◁ 1838

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

▶ 1951 ◀

Date bracketed by triangles indicates local fault break.

1992

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

CREEP

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

1969
1968 ■ ■ 1968

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

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

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

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

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

ADDITIONAL FAULT SYMBOLS

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

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

Arrow on fault indicates direction of dip.

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

OTHER SYMBOLS

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

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

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

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

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

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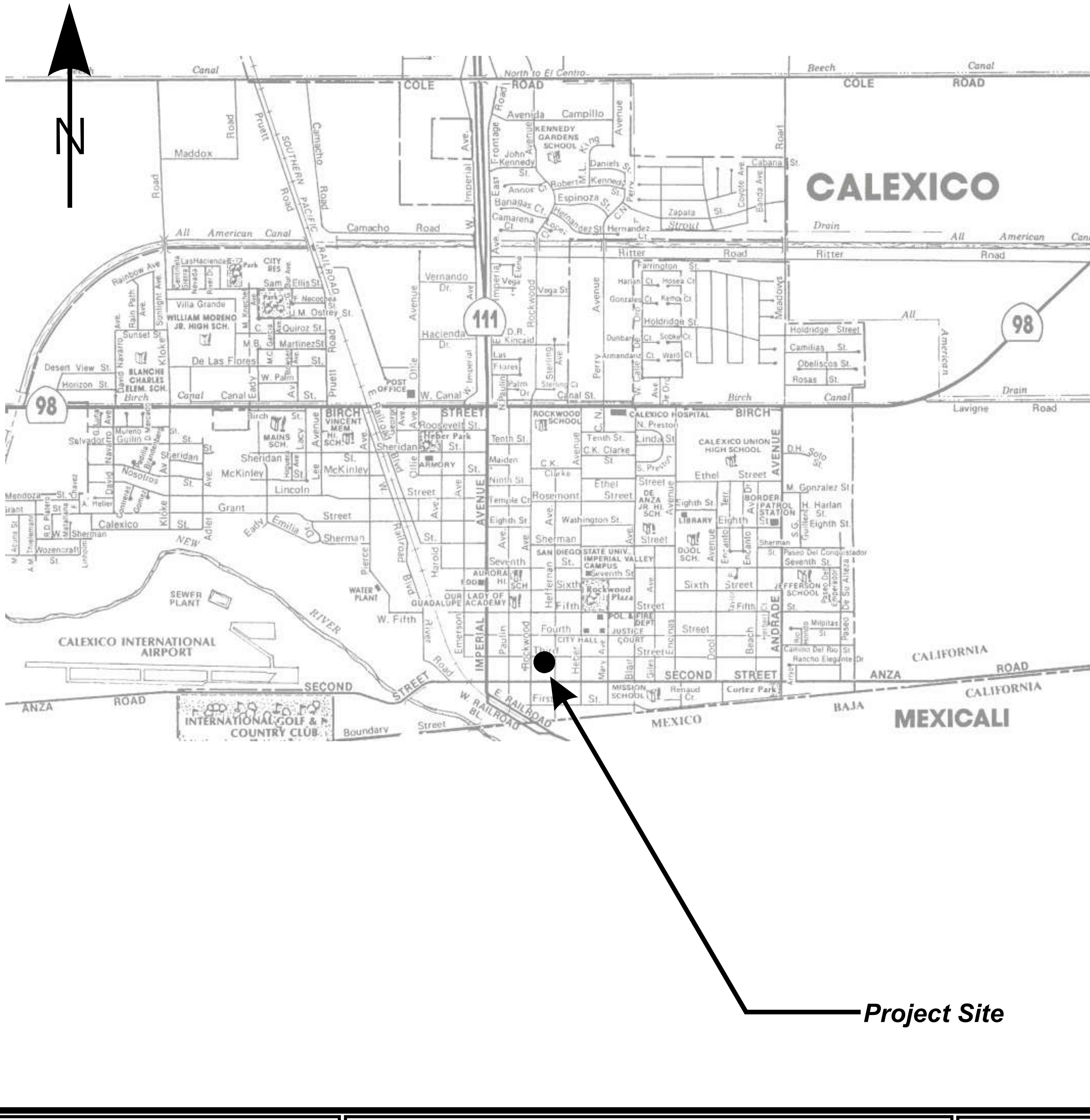
Fault Map Legend

Figure 3

APPENDIX A



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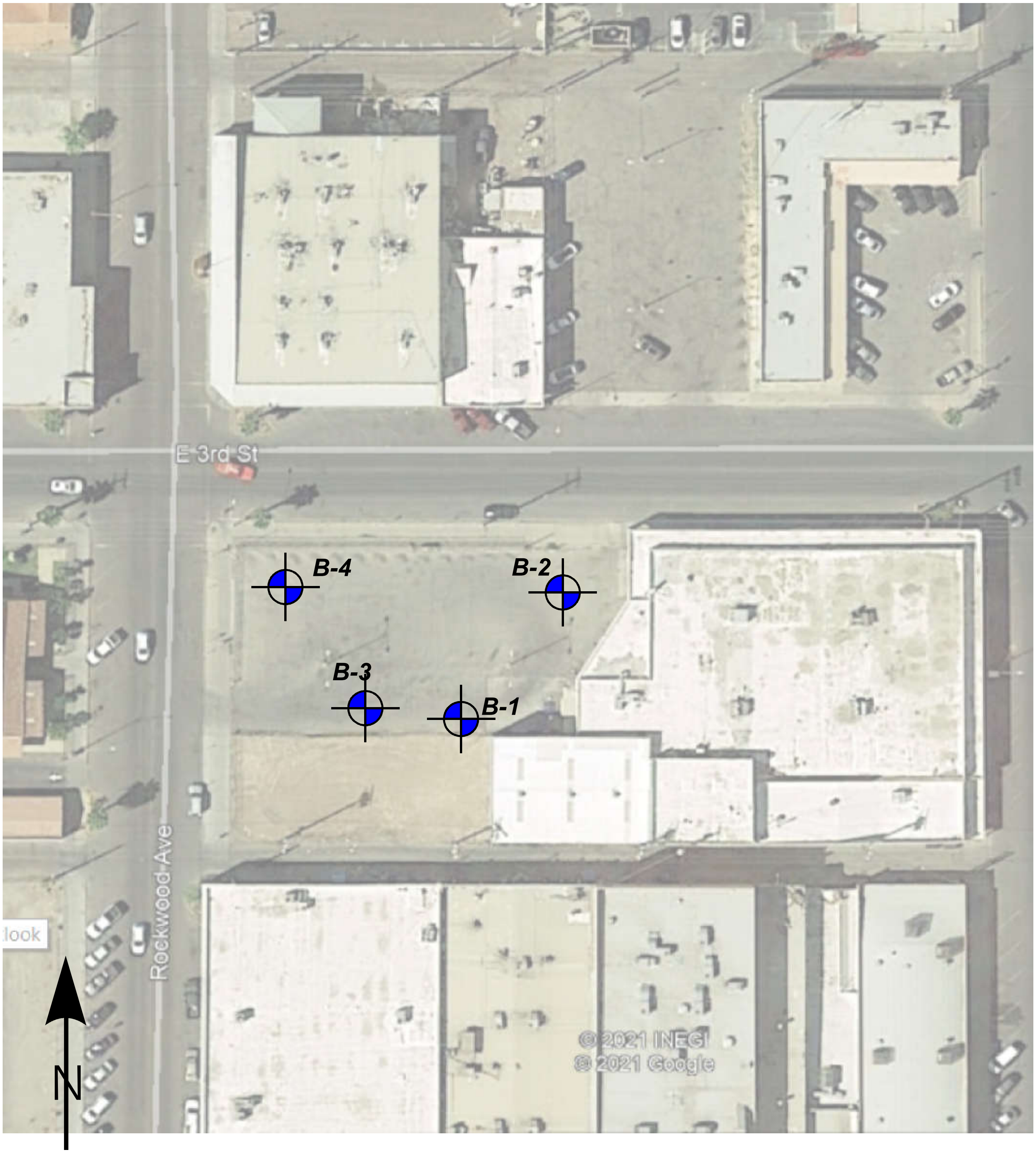
Vicinity Map

Plate A-1



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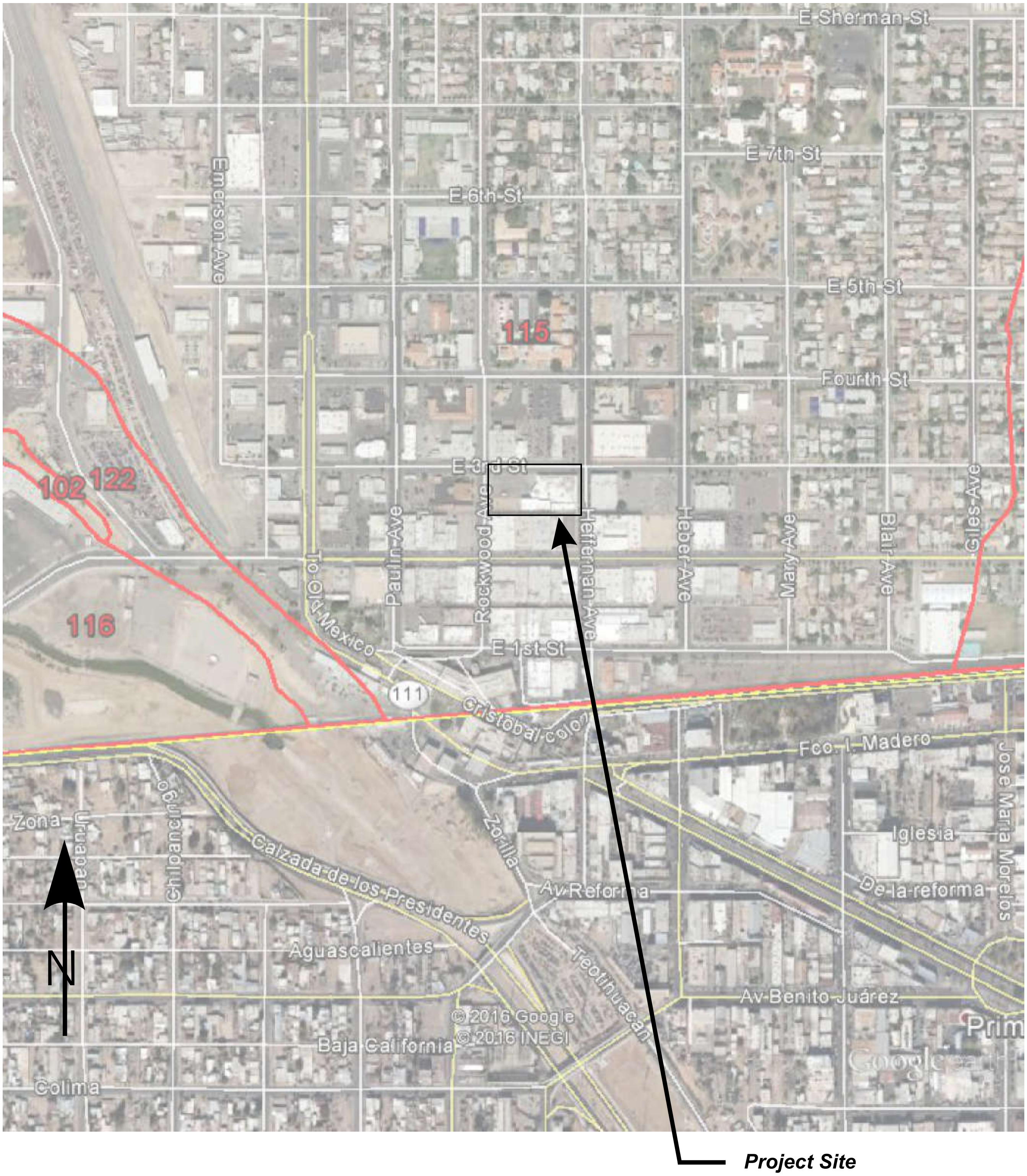
Site and Exploration Plan

Plate
A-2



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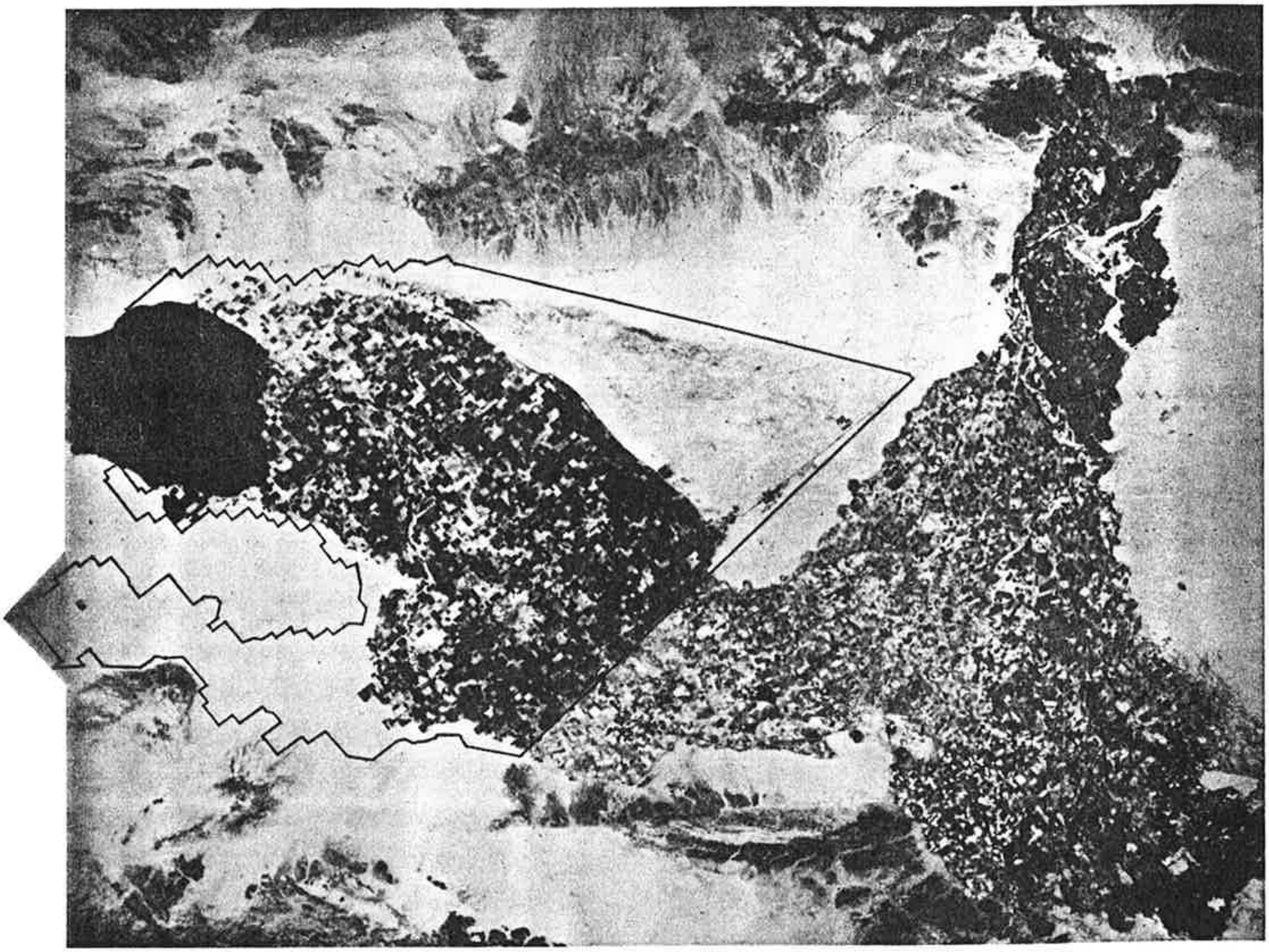
Soil Survey Map

Plate
A-3

Soil Survey of

IMPERIAL COUNTY CALIFORNIA IMPERIAL VALLEY AREA



United States Department of Agriculture Soil Conservation Service

in cooperation with

University of California Agricultural Experiment Station

and

Imperial Irrigation District

TABLE 11.--ENGINEERING INDEX PROPERTIES

[The symbol > means more than. Absence of an entry indicates that data were not estimated]

Soil name and map symbol	Depth	USDA texture	Classification: Unified	Classification: AASHTO	Fragments > 3 inches	Percentage passing sieve number-- 4	10	40	200	Liquid limit	Plasticity index
	In				Pct					Pct	
100---------------- Antho	0-13	Loamy fine sand	SM	A-2	0	100	100	75-85	10-30	---	NP
	13-60	Sandy loam, fine sandy loam.	SM	A-2, A-4	0	90-100	75-95	50-60	15-40	---	NP
101*:											
Antho-------------	0-8	Loamy fine sand	SM	A-2	0	100	100	75-85	10-30	---	NP
	8-60	Sandy loam, fine sandy loam.	SM	A-2, A-4	0	90-100	75-95	50-60	15-40	---	NP
Superstition------	0-6	Fine sand-------	SM	A-2	0	100	95-100	70-85	15-25	---	NP
	6-60	Loamy fine sand, fine sand, sand.	SM	A-2	0	100	95-100	70-85	15-25	---	NP
102*. Badland											
103---------------- Carsitas	0-10	Gravelly sand---	SP, SP-SM	A-1, A-2	0-5	60-90	50-85	30-55	0-10	---	NP
	10-60	Gravelly sand, gravelly coarse sand, sand.	SP, SP-SM	A-1	0-5	60-90	50-85	25-50	0-10	---	NP
104* Fluvaquents											
105---------------- Glenbar	0-13	Clay loam-------	CL	A-6	0	100	100	90-100	70-95	35-45	15-30
	13-60	Clay loam, silty clay loam.	CL	A-6	0	100	100	90-100	70-95	35-45	15-30
106---------------- Glenbar	0-13	Clay loam-------	CL	A-6, A-7	0	100	100	90-100	70-95	35-45	15-25
	13-60	Clay loam, silty clay loam.	CL	A-6, A-7	0	100	100	90-100	70-95	35-45	15-25
107*--------------- Glenbar	0-13	Loam-----------	ML, CL-ML, CL	A-4	0	100	100	100	70-80	20-30	NP-10
	13-60	Clay loam, silty clay loam.	CL	A-6, A-7	0	100	100	95-100	75-95	35-45	15-30
108---------------- Holtville	0-14	Loam-----------	ML	A-4	0	100	100	85-100	55-95	25-35	NP-10
	14-22	Clay, silty clay	CL, CH	A-7	0	100	100	95-100	85-95	40-65	20-35
	22-60	Silt loam, very fine sandy loam.	ML	A-4	0	100	100	95-100	65-85	25-35	NP-10
109---------------- Holtville	0-17	Silty clay------	CL, CH	A-7	0	100	100	95-100	85-95	40-65	20-35
	17-24	Clay, silty clay	CL, CH	A-7	0	100	100	95-100	85-95	40-65	20-35
	24-35	Silt loam, very fine sandy loam.	ML	A-4	0	100	100	95-100	65-85	25-35	NP-10
	35-60	Loamy very fine sand, loamy fine sand.	SM, ML	A-2, A-4	0	100	100	75-100	20-55	---	NP
110---------------- Holtville	0-17	Silty clay------	CH, CL	A-7	0	100	100	95-100	85-95	40-65	20-35
	17-24	Clay, silty clay	CH, CL	A-7	0	100	100	95-100	85-95	40-65	20-35
	24-35	Silt loam, very fine sandy loam.	ML	A-4	0	100	100	95-100	55-85	25-35	NP-10
	35-60	Loamy very fine sand, loamy fine sand.	SM, ML	A-2, A-4	0	100	100	75-100	20-55	---	NP

See footnote at end of table.

TABLE 11.--ENGINEERING INDEX PROPERTIES--Continued

Soil name and map symbol	Depth	USDA texture	Classification: Unified	Classification: AASHTO	Frag- ments > 3 inches	Percentage passing sieve number--: 4	10	40	200	Liquid limit	Plas- ticity index
	In				Pct					Pct	
111*:											
Holtville---------	0-10	Silty clay loam	CL, CH	A-7	0	100	100	95-100	85-95	40-65	20-35
	10-22	Clay, silty clay	CL, CH	A-7	0	100	100	95-100	85-95	40-65	20-35
	22-60	Silt loam, very fine sandy loam.	ML	A-4	0	100	100	95-100	65-85	25-35	NP-10
Imperial----------	0-12	Silty clay loam	CL	A-7	0	100	100	100	85-95	40-50	10-20
	12-60	Silty clay loam, silty clay, clay.	CH	A-7	0	100	100	100	85-95	50-70	25-45
112---------------- Imperial	0-12	Silty clay------	CH	A-7	0	100	100	100	85-95	50-70	25-45
	12-60	Silty clay loam, silty clay, clay.	CH	A-7	0	100	100	100	85-95	50-70	25-45
113---------------- Imperial	0-12	Silty clay------	CH	A-7	0	100	100	100	85-95	50-70	25-45
	12-60	Silty clay, clay, silty clay loam.	CH	A-7	0	100	100	100	85-95	50-70	25-45
114---------------- Imperial	0-12	Silty clay------	CH	A-7	0	100	100	100	85-95	50-70	25-45
	12-60	Silty clay loam, silty clay, clay.	CH	A-7	0	100	100	100	85-95	50-70	25-45
115*:											
Imperial----------	0-12	Silty clay loam	CL	A-7	0	100	100	100	85-95	40-50	10-20
	12-60	Silty clay loam, silty clay, clay.	CH	A-7	0	100	100	100	85-95	50-70	25-45
Glenbar-----------	0-13	Silty clay loam	CL	A-6, A-7	0	100	100	90-100	70-95	35-45	15-25
	13-60	Clay loam, silty clay loam.	CL	A-6, A-7	0	100	100	90-100	70-95	35-45	15-25
116*:											
Imperial----------	0-13	Silty clay loam	CL	A-7	0	100	100	100	85-95	40-50	10-20
	13-60	Silty clay loam, silty clay, clay.	CH	A-7	0	100	100	100	85-95	50-70	25-45
Glenbar-----------	0-13	Silty clay loam	CL	A-6, A-7	0	100	100	90-100	70-95	35-45	15-25
	13-60	Clay loam, silty clay loam.	CL	A-6	0	100	100	90-100	70-95	35-45	15-30
117, 118----------- Indio	0-12	Loam------------	ML	A-4	0	95-100	95-100	85-100	75-90	20-30	NP-5
	12-72	Stratified loamy very fine sand to silt loam.	ML	A-4	0	95-100	95-100	85-100	75-90	20-30	NP-5
119*:											
Indio-------------	0-12	Loam------------	ML	A-4	0	95-100	95-100	85-100	75-90	20-30	NP-5
	12-72	Stratified loamy very fine sand to silt loam.	ML	A-4	0	95-100	95-100	85-100	75-90	20-30	NP-5
Vint--------------	0-10	Loamy fine sand	SM	A-2	0	95-100	95-100	70-80	25-35	---	NP
	10-60	Loamy sand, loamy fine sand.	SM	A-2	0	95-100	95-100	70-80	20-30	---	NP
120*--------------- Laveen	0-12	Loam------------	ML, CL-ML	A-4	0	100	95-100	75-85	55-65	20-30	NP-10
	12-60	Loam, very fine sandy loam.	ML, CL-ML	A-4	0	95-100	85-95	70-80	55-65	15-25	NP-10

See footnote at end of table.

TABLE 11.--ENGINEERING INDEX PROPERTIES--Continued

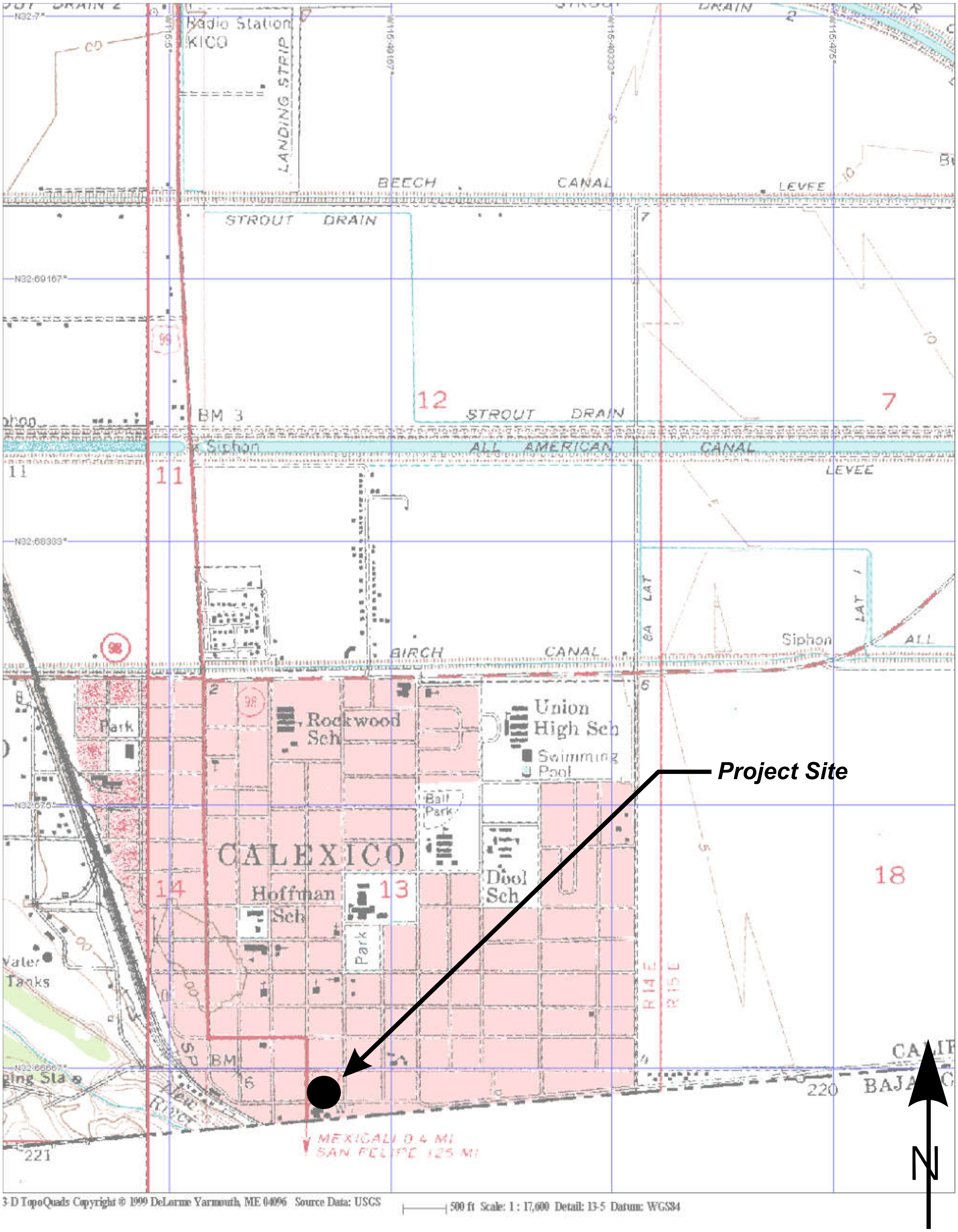
Soil name and map symbol	Depth	USDA texture	Classification		Frag- ments > 3 inches	Percentage passing sieve number--				Liquid limit	Plas- ticity index
			Unified	AASHTO		4	10	40	200		
	In				Pct					Pct	
121----------------- Meloland	0-12	Fine sand-------	SM, SP-SM	A-2, A-3	0	95-100	90-100	75-100	5-30	---	NP
	12-26	Stratified loamy fine sand to silt loam.	ML	A-4	0	100	100	90-100	50-65	25-35	NP-10
	26-71	Clay, silty clay, silty clay loam.	CL, CH	A-7	0	100	100	95-100	85-95	40-65	20-40
122----------------- Meloland	0-12	Very fine sandy loam.	ML	A-4	0	95-100	95-100	95-100	55-85	25-35	NP-10
	12-26	Stratified loamy fine sand to silt loam.	ML	A-4	0	100	100	90-100	50-70	25-35	NP-10
	26-71	Clay, silty clay, silty clay loam.	CH, CL	A-7	0	100	100	95-100	85-95	40-65	20-40
123*:											
Meloland----------	0-12	Loam------------	ML	A-4	0	95-100	95-100	95-100	55-85	25-35	NP-10
	12-26	Stratified loamy fine sand to silt loam.	ML	A-4	0	100	100	90-100	50-70	25-35	NP-10
	26-38	Clay, silty clay, silty clay loam.	CH, CL	A-7	0	100	100	95-100	85-95	40-65	20-40
	38-60	Stratified silt loam to loamy fine sand.	SM, ML	A-4	0	100	100	75-100	35-55	25-35	NP-10
Holtville---------	0-12	Loam------------	ML	A-4	0	100	100	85-100	55-95	25-35	NP-10
	12-24	Clay, silty clay	CH, CL	A-7	0	100	100	95-100	85-95	40-65	20-35
	24-36	Silt loam, very fine sandy loam.	ML	A-4	0	100	100	95-100	55-85	25-35	NP-10
	36-60	Loamy very fine sand, loamy fine sand.	SM, ML	A-2, A-4	0	100	100	75-100	20-55	---	NP
124, 125----------- Niland	0-23	Gravelly sand---	SM, SP-SM	A-2, A-3	0	90-100	70-95	50-65	5-25	---	NP
	23-60	Silty clay, clay, clay loam.	CL, CH	A-7	0	100	100	85-100	80-95	40-65	20-40
126----------------- Niland	0-23	Fine sand-------	SM, SP-SM	A-2, A-3	0	90-100	90-100	50-65	5-25	---	NP
	23-60	Silty clay------	CL, CH	A-7	0	100	100	85-100	80-95	40-65	20-40
127----------------- Niland	0-23	Loamy fine sand	SM	A-2	0	90-100	90-100	50-65	15-30	---	NP
	23-60	Silty clay------	CL, CH	A-7	0	100	100	85-100	80-95	40-65	20-40
128*:											
Niland------------	0-23	Gravelly sand---	SM, SP-SM	A-2, A-3	0	90-100	70-95	50-65	5-25	---	NP
	23-60	Silty clay, clay, clay loam.	CL, CH	A-7	0	100	100	85-100	80-100	40-65	20-40
Imperial----------	0-12	Silty clay------	CH	A-7	0	100	100	100	85-95	50-70	25-45
	12-60	Silty clay loam, silty clay, clay.	CH	A-7	0	100	100	100	85-95	50-70	25-45
129*: Pits											
130, 131----------- Rositas	0-27	Sand------------	SP-SM	A-3, A-1, A-2	0	100	80-100	40-70	5-15	---	NP
	27-60	Sand, fine sand, loamy sand.	SM, SP-SM	A-3, A-2, A-1	0	100	80-100	40-85	5-30	---	NP

See footnote at end of table.

TABLE 11.--ENGINEERING INDEX PROPERTIES--Continued

Soil name and map symbol	Depth	USDA texture	Classification		Fragments > 3 inches	Percentage passing sieve number--				Liquid limit	Plasticity index
			Unified	AASHTO		4	10	40	200		
	In				Pct					Pct	
132, 133, 134, 135-Rositas	0-9	Fine sand	SM	A-3, A-2	0	100	80-100	50-80	10-25	---	NP
	9-60	Sand, fine sand, loamy sand.	SM, SP-SM	A-3, A-2, A-1	0	100	80-100	40-85	5-30	---	NP
136 Rositas	0-4	Loamy fine sand	SM	A-1, A-2	0	100	80-100	40-85	10-35	---	NP
	4-60	Sand, fine sand, loamy sand.	SM, SP-SM	A-3, A-2, A-1	0	100	80-100	40-85	5-30	---	NP
137 Rositas	0-12	Silt loam	ML	A-4	0	100	100	90-100	70-90	20-30	NP-5
	12-60	Sand, fine sand, loamy sand.	SM, SP-SM	A-3, A-2, A-1	0	100	80-100	40-85	5-30	---	NP
138*:											
Rositas	0-4	Loamy fine sand	SM	A-1, A-2	0	100	80-100	40-85	10-35	---	NP
	4-60	Sand, fine sand, loamy sand.	SM, SP-SM	A-3, A-2, A-1	0	100	80-100	40-85	5-30	---	NP
Superstition	0-6	Loamy fine sand	SM	A-2	0	100	95-100	70-85	15-25	---	NP
	6-60	Loamy fine sand, fine sand, sand.	SM	A-2	0	100	95-100	70-85	15-25	---	NP
139 Superstition	0-6	Loamy fine sand	SM	A-2	0	100	95-100	70-85	15-25	---	NP
	6-60	Loamy fine sand, fine sand, sand.	SM	A-2	0	100	95-100	70-85	15-25	---	NP
140*: Torriorthents											
Rock outcrop											
141*: Torriorthents											
Orthids											
142 Vint	0-10	Loamy very fine sand.	SM, ML	A-4	0	100	100	85-95	40-65	15-25	NP-5
	10-60	Loamy fine sand	SM	A-2	0	95-100	95-100	70-80	20-30	---	NP
143 Vint	0-12	Fine sandy loam	ML, CL-ML, SM, SM-SC	A-4	0	100	100	75-85	45-55	15-25	NP-5
	12-60	Loamy sand, loamy fine sand.	SM	A-2	0	95-100	95-100	70-80	20-30	---	NP
144*:											
Vint	0-10	Very fine sandy loam.	SM, ML	A-4	0	100	100	85-95	40-65	15-25	NP-5
	10-40	Loamy fine sand	SM	A-2	0	95-100	95-100	70-80	20-30	---	NP
	40-60	Silty clay	CL, CH	A-7	0	100	100	95-100	85-95	40-65	20-35
Indio	0-12	Very fine sandy loam.	ML	A-4	0	95-100	95-100	85-100	75-90	20-30	NP-5
	12-40	Stratified loamy very fine sand to silt loam.	ML	A-4	0	95-100	95-100	85-100	75-90	20-30	NP-5
	40-72	Silty clay	CL, CH	A-7	0	100	100	95-100	85-95	40-65	20-35

* See description of the map unit for composition and behavior characteristics of the map unit.

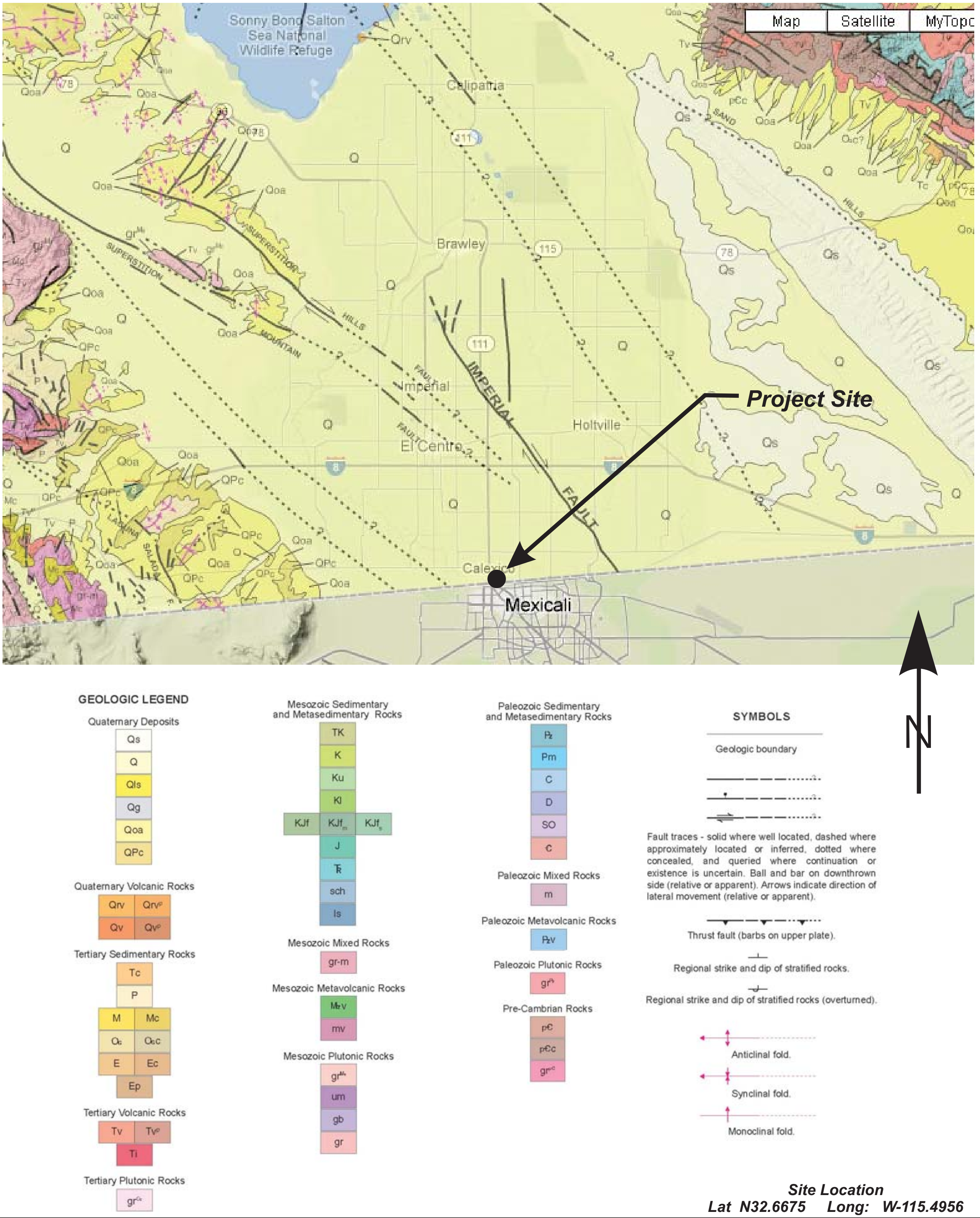


LANDMARK
Geo-Engineers and Geologists

Project No.: LE21050

Topographic Map

Plate A-4



Site Location
Lat N32.6675 Long: W-115.4956

LANDMARK
Geo-Engineers and Geologists

Project No.: LE21050

Regional Geologic Map

Plate A-5

APPENDIX B

LOG OF BORING No. B-1

SHEET 1 OF 1

DEPTH	BLOW COUNT	POCKET PEN. (tsf)	DESCRIPTION OF MATERIAL	DRY DENSITY (pcf)	MOISTURE CONTENT (% dry wt.)	OTHER TESTS
0–16			SILTY CLAY (CL): Light brown, moist, hard, medium plasticity with some very fine grain sands.			
5	73	4.5+		113.0	17.2	
10	61	4.5+				LL=43% PI=25%
15	57					
16–21			SILTY SAND (SM): Tan, moist, medium dense, fine and very fine grained			
20	17					
21–24		3.5	SILTY CLAY (CL): Brown, moist, very stiff, medium to high plasticity			
25	9	0.75	SILT (ML): Light brown, very moist, loose			Passing #200 = 90.6%
30	10	1.5	SILTY CLAY (CL): Brown, very moist, stiff to very stiff, medium to high plasticity, hydrocarbon odors			
35	12	2.5				
40	10	2.0			25.8	LL=36% PI=27%
45	9	1.0	CLAYEY SILT/SILTY CLAY (ML/CL): Light brown, wet, loose/firm, hydrocarbon odor		28.9	LL=34% PI=23%
50	17		CLAY (CL-CH): Brown, saturated, very stiff, high plasticity			

Groundwater measured at 49 feet at time of drilling. This is not considered the stabilized groundwater depth as groundwater may rise to a level higher than that measured in borehole.

DATE DRILLED: 3/4/21
LOGGED BY: P. LaBrucherie
SURFACE ELEVATION: Approximately 5'

TOTAL DEPTH: 51.5 Feet
TYPE OF BIT: Hollow Stem Auger
HAMMER WT.: 140 lbs.

DEPTH TO WATER: ~30 ft.
DIAMETER: 8 in.
DROP: 30 in.

PROJECT NO. LE21050

LANDMARK Geo-Engineers and Geologists

PLATE B-1

LOG OF BORING NO. B-2

SHEET 1 OF 1

DEPTH	SAMPLE	USCS CLASS.	BLOW COUNT	POCKET PEN. (tsf)	DESCRIPTION OF MATERIAL	DRY DENSITY (pcf)	MOISTURE CONTENT (% dry wt.)	OTHER TESTS
					4” AC over 5” of recycled AC base			
					CLAY (CL-CH): Brown, very moist, stiff, high plasticity.			
5								
10								
15								
20								
25								
30								

FIELD / LABORATORY

DATE DRILLED: 3/4/21
TOTAL DEPTH: 5 Feet
DEPTH TO WATER: NA
LOGGED BY: P. LaBrucherie
TYPE OF BIT: 8” Auger
DIAMETER: 8”
SURFACE ELEVATION: Approximately 5’
HAMMER WT.: N/A
DROP: N/A

PROJECT No. LE21050

LANDMARK Geo-Engineers and Geologists

PLATE B-2

LOG OF BORING NO. B-3

SHEET 1 OF 1

DEPTH	SAMPLE	USCS CLASS.	BLOW COUNT	POCKET PEN. (tsf)	DESCRIPTION OF MATERIAL	DRY DENSITY (pcf)	MOISTURE CONTENT (% dry wt.)	OTHER TESTS
					5” AC over 3” of recycled AC and concrete base			
5					CLAY/SILTY CLAY (CL-CH): Brown, very moist, stiff, medium to high plasticity.			
10								
15								
20								
25								
30								

FIELD / LABORATORY

DATE DRILLED:	3/4/21	TOTAL DEPTH:	5 Feet	DEPTH TO WATER:	NA
LOGGED BY:	P. LaBrucherie	TYPE OF BIT:	8” Auger	DIAMETER:	8”
SURFACE ELEVATION:	Approximately 5’	HAMMER WT.:	N/A	DROP:	N/A

PROJECT No. LE21050

LANDMARK Geo-Engineers and Geologists

PLATE B-3

DEPTH	FIELD: SAMPLE	FIELD: USCS CLASS.	FIELD: BLOW COUNT	FIELD: POCKET PEN. (tsf)	LOG OF BORING NO. B-4 SHEET 1 OF 1 DESCRIPTION OF MATERIAL	LABORATORY: DRY DENSITY (pcf)	LABORATORY: MOISTURE CONTENT (% dry wt.)	LABORATORY: OTHER TESTS
					4” AC over 3” of recycled concrete base			
					CLAY (CH): Brown, very moist, stiff, high plasticity.			
5								
10								
15								
20								
25								
30								

DATE DRILLED: 3/4/21 TOTAL DEPTH: 5 Feet DEPTH TO WATER: NA

LOGGED BY: P. LaBrucherie TYPE OF BIT: 8” Auger DIAMETER: 8”

SURFACE ELEVATION: Approximately 5’ HAMMER WT.: N/A DROP: N/A

PROJECT No. LE21050

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PLATE B-4

DEFINITION OF TERMS

PRIMARY DIVISIONS			SYMBOLS	SECONDARY DIVISIONS
Coarse grained soils More than half of material is larger that No. 200 sieve	Gravels	Clean gravels (less than 5% fines)	GW	Well graded gravels, gravel-sand mixtures, little or no fines
	More than half of coarse fraction is larger than No. 4 sieve		GP	Poorly graded gravels, or gravel-sand mixtures, little or no fines
		Gravel with fines	GM	Silty gravels, gravel-sand-silt mixtures, non-plastic fines
			GC	Clayey gravels, gravel-sand-clay mixtures, plastic fines
	Sands	Clean sands (less than 5% fines)	SW	Well graded sands, gravelly sands, little or no fines
	More than half of coarse fraction is smaller than No. 4 sieve		SP	Poorly graded sands or gravelly sands, little or no fines
		Sands with fines	SM	Silty sands, sand-silt mixtures, non-plastic fines
			SC	Clayey sands, sand-clay mixtures, plastic fines
Fine grained soils More than half of material is smaller than No. 200 sieve	Silts and clays		ML	Inorganic silts, clayey silts with slight plasticity
	Liquid limit is less than 50%		CL	Inorganic clays of low to medium plasticity, gravely, sandy, or lean clays
			OL	Organic silts and organic clays of low plasticity
	Silts and clays		MH	Inorganic silts, micaceous or diatomaceous silty soils, elastic silts
	Liquid limit is more than 50%		CH	Inorganic clays of high plasticity, fat clays
			OH	Organic clays of medium to high plasticity, organic silts
Highly organic soils			PT	Peat and other highly organic soils

GRAIN SIZES

Silts and Clays	Sand			Gravel		Cobbles	Boulders
	Fine	Medium	Coarse	Fine	Coarse		

200 40 10 4 3/4" 3" 12"

US Standard Series Sieve — Clear Square Openings

Sands, Gravels, etc.	Blows/ft. *
Very Loose	0-4
Loose	4-10
Medium Dense	10-30
Dense	30-50
Very Dense	Over 50

Clays & Plastic Silts	Strength **	Blows/ft. *
Very Soft	0-0.25	0-2
Soft	0.25-0.5	2-4
Firm	0.5-1.0	4-8
Stiff	1.0-2.0	8-16
Very Stiff	2.0-4.0	16-32
Hard	Over 4.0	Over 32

* Number of blows of 140 lb. hammer falling 30 inches to drive a 2 inch O.D. (1 3/8 in. I.D.) split spoon (ASTM D1586).

** Unconfined compressive strength in tons/s.f. as determined by laboratory testing or approximated by the Standard Penetration Test (ASTM D1586), Pocket Penetrometer, Torvane, or visual observation.

Type of Samples:

Ring Sample — Standard Penetration Test — Shelby Tube — Bulk (Bag) Sample

Drilling Notes:

1. Sampling and Blow Counts
   - Ring Sampler - Number of blows per foot of a 140 lb. hammer falling 30 inches.
   - Standard Penetration Test - Number of blows per foot.
   - Shelby Tube - Three (3) inch nominal diameter tube hydraulically pushed.
2. P. P. = Pocket Penetrometer (tons/s.f.).
3. NR = No recovery.
4. GWT = Ground Water Table observed @ specified time.

LANDMARK Geo-Engineers and Geologists

Project No. LE21050

Key to Logs

Plate B-5

APPENDIX C

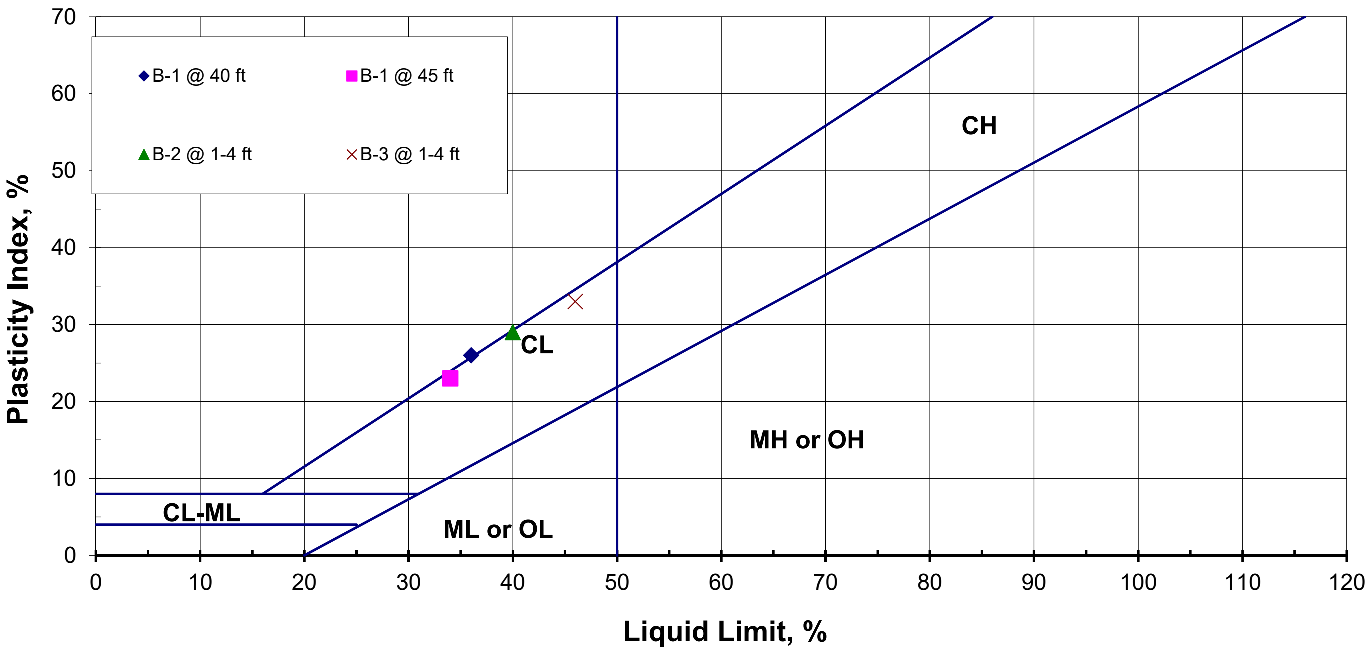
LANDMARK CONSULTANTS, INC.

CLIENT: Psomas
PROJECT: Transit Center - Calexico, CA
JOB No.: LE21050
DATE: 04/12/21

ATTERBERG LIMITS (ASTM D4318)

Sample Location	Sample Depth (ft)	Liquid Limit (LL)	Plastic Limit (PL)	Plasticity Index (PI)	USCS Classification
B-1	40	36	10	26	CL
B-1	45	34	11	23	CL
B-2	1-4	40	11	29	CL
B-3	1-4	46	13	33	CL

PLASTICITY CHART

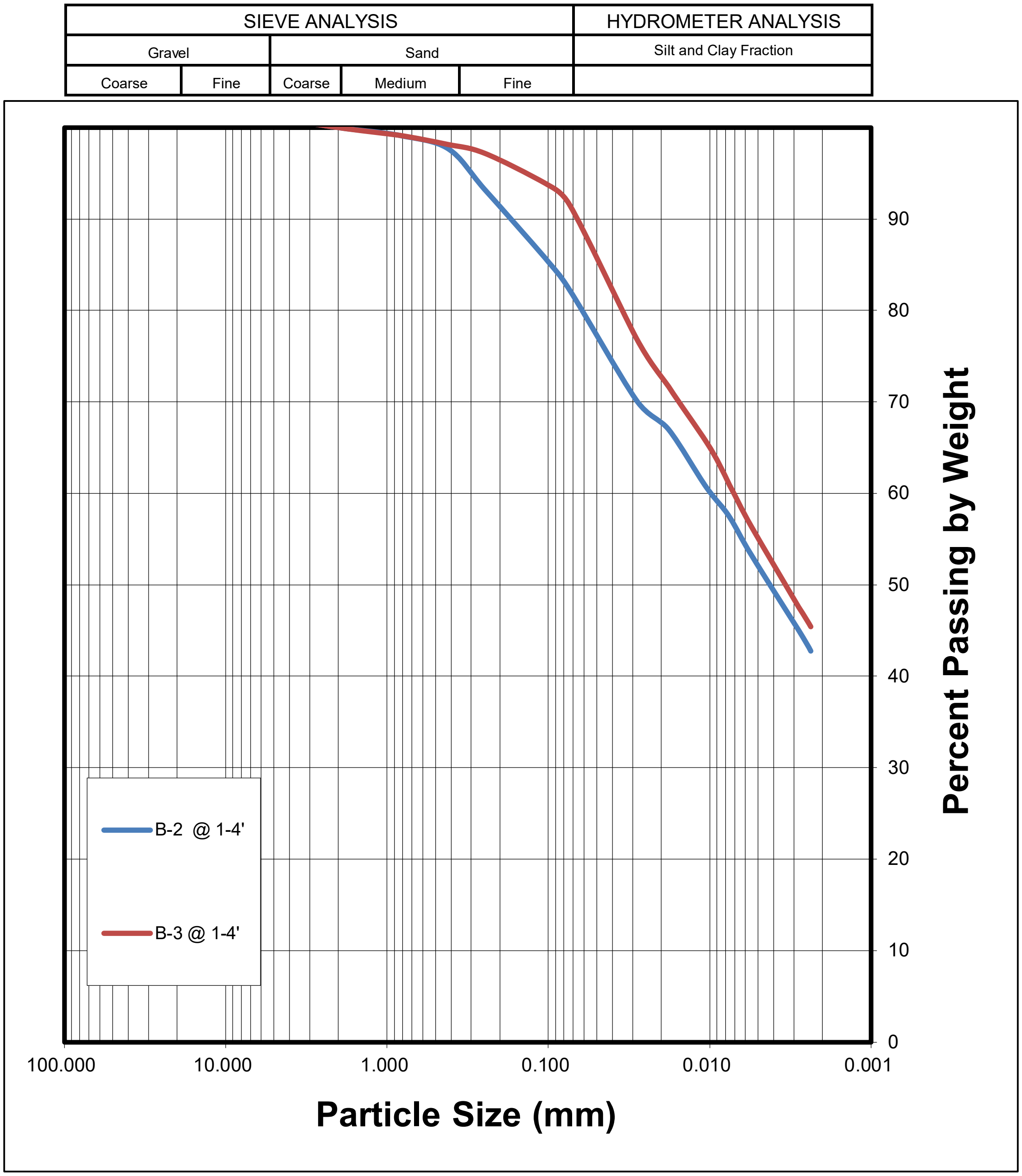


LANDMARK Geo-Engineers and Geologists

Project No.: LE21050

Atterberg Limits Test Results

Plate C-1



LANDMARK Geo-Engineers and Geologists

Project No.: LE21050

Grain Size Analysis

Plate C-2

LANDMARK CONSULTANTS, INC.

CLIENT: Psomas
PROJECT: Calexico Transit Center
JOB No.: LE21050
DATE: 04/12/21

CHEMICAL ANALYSIS

Boring:	B-2	B-4	**Caltrans**
Sample Depth, ft:	1-4	1-4	**Method**
pH:	8.5	8.6	**643**
Electrical Conductivity (mmhos):	--	--	**424**
Resistivity (ohm-cm):	540	560	**643**
Chloride (Cl), ppm:	460	280	**422**
Sulfate (SO4), ppm:	790	363	**417**

General Guidelines for Soil Corrosivity

Material Affected	Chemical Agent	Range of Values	Degree of Corrosivity
Concrete	Soluble Sulfates (ppm)	0 - 1,000	Low
		1,000 - 2,000	Moderate
		2,000 - 20,000	Severe
		> 20,000	Very Severe
Normal Grade Steel	Soluble Chlorides (ppm)	0 - 200	Low
		200 - 700	Moderate
		700 - 1,500	Severe
		> 1,500	Very Severe
Normal Grade Steel	Resistivity (ohm-cm)	1 - 1,000	Very Severe
		1,000 - 2,000	Severe
		2,000 - 10,000	Moderate
		> 10,000	Low

LANDMARK
Geo-Engineers and Geologists

Project No.: LE21050

Selected Chemical Test Results

Plate C-3

LANDMARK CONSULTANTS, INC.

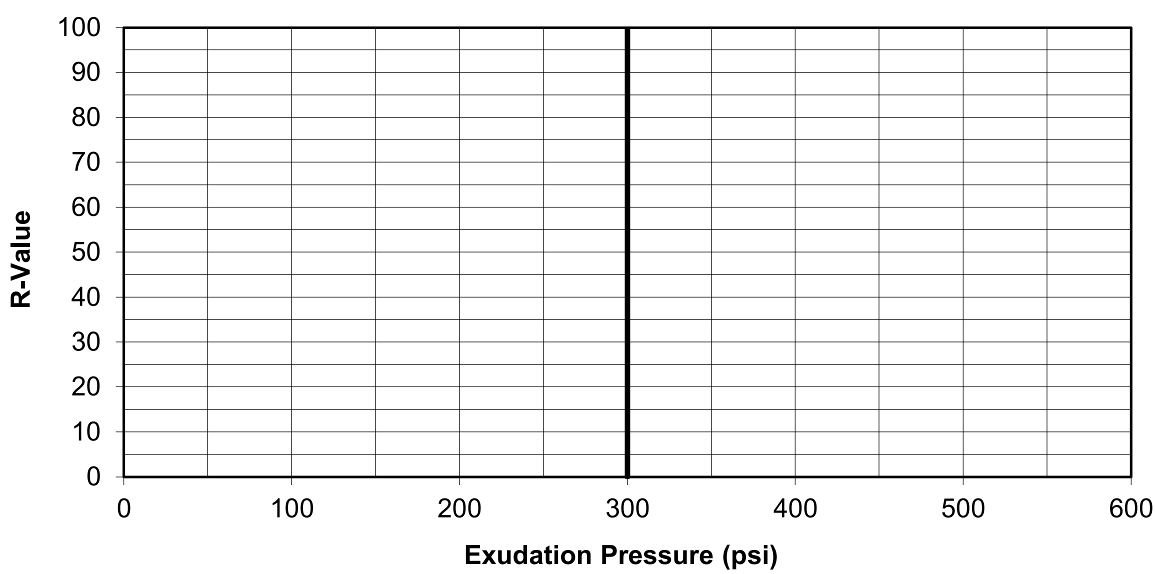
Client: Psomas
Project: Calexico Transit Station
Project No.: LE21050
Date: 4/12/2021 **Lab No.:** EC21-130

R-Value By Exudation Pressure (ASTM D2844/CAL 301)

Description: Clay/Silty Clay (CL-CH)
Sample Location: B-3
Sample Depth: 1-4 ft.

Sample	A	B	C
Moisture Content, %:	25.0%	23.0%	21.0%
Dry Density, pcf:	105.8	106.3	108.2
Compaction foot pressure, psi:	70	80	90
Specimen Height, in.:	2.52	2.47	2.52
Stabilometer, Ph @ 1000 lb:	---	---	---
Stabilometer, Ph @ 2000 lb:	---	---	---
Displacement:	---	---	---
Expantion pressure, psf:	39	61	122
Exudation pressure, psi:	227	393	512
Equilibrum R Value:	---	---	---

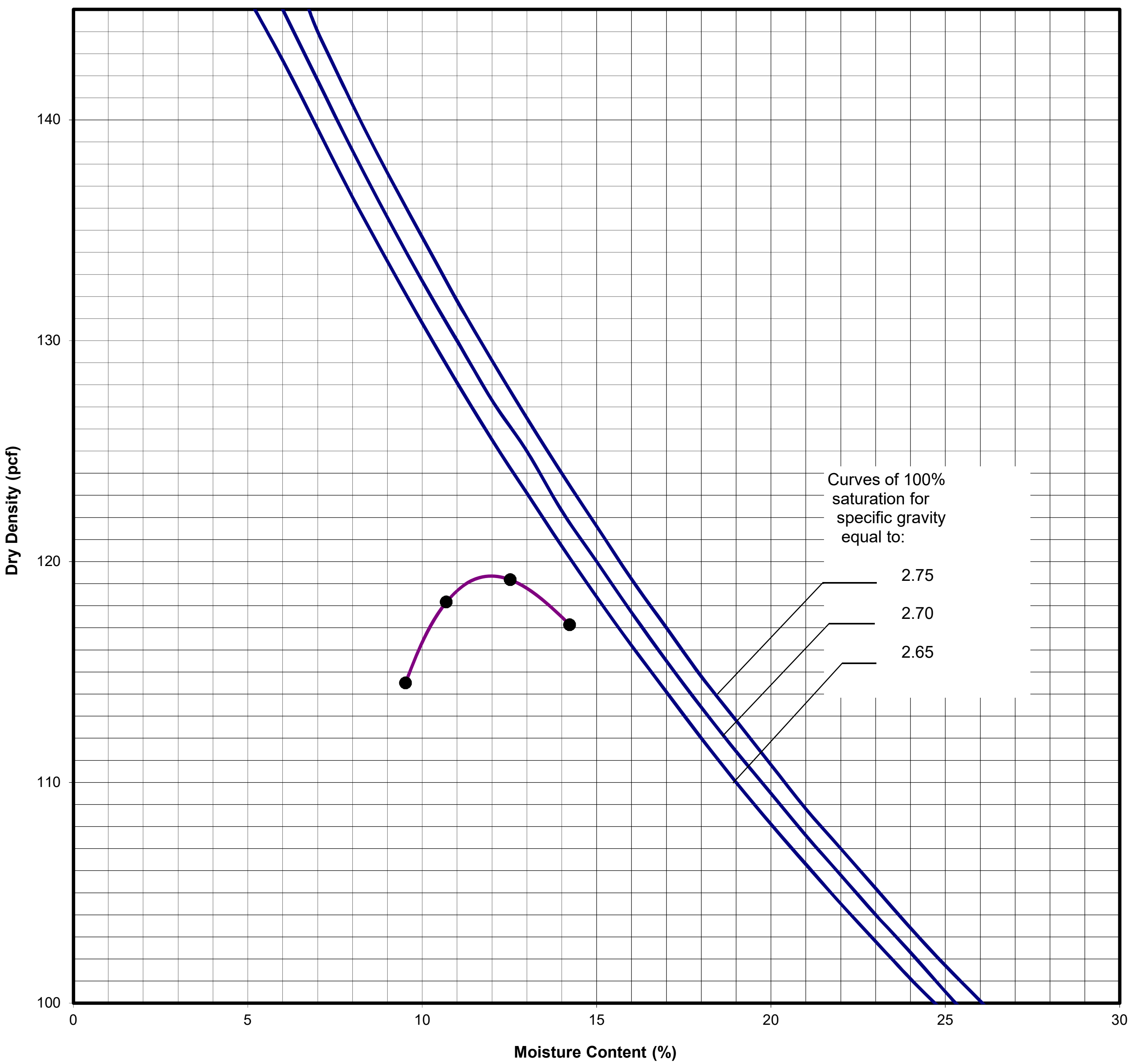
R-Value Less than 5



LANDMARK Geo-Engineers and Geologists **Project No.: LE21050**	**R-Value Test**	**Plate** **C-4**

Client: Psomas
Project: CLX Transit Center - Calexico, CA
Project No.: LE21050
Date: 3/12/2021
Lab. No.: EC2021-129

Soil Description: Silty Clay (CL)
Sample Location: B-2 @ 1-4'
Test Method: ASTM D-1557-A
Maximum Dry Density (pcf): **119.3**
Optimum Moisture Content (%): **12.0**



Project No.: LE21050

Moisture Density Relationship

Plate C-5

APPENDIX D

Liquefaction Evaluation and Settlement Calculation

Project Name: **Calexico Transit Center - Calexico, CA**
Project No.: **LE21050**
Location: **B-1**

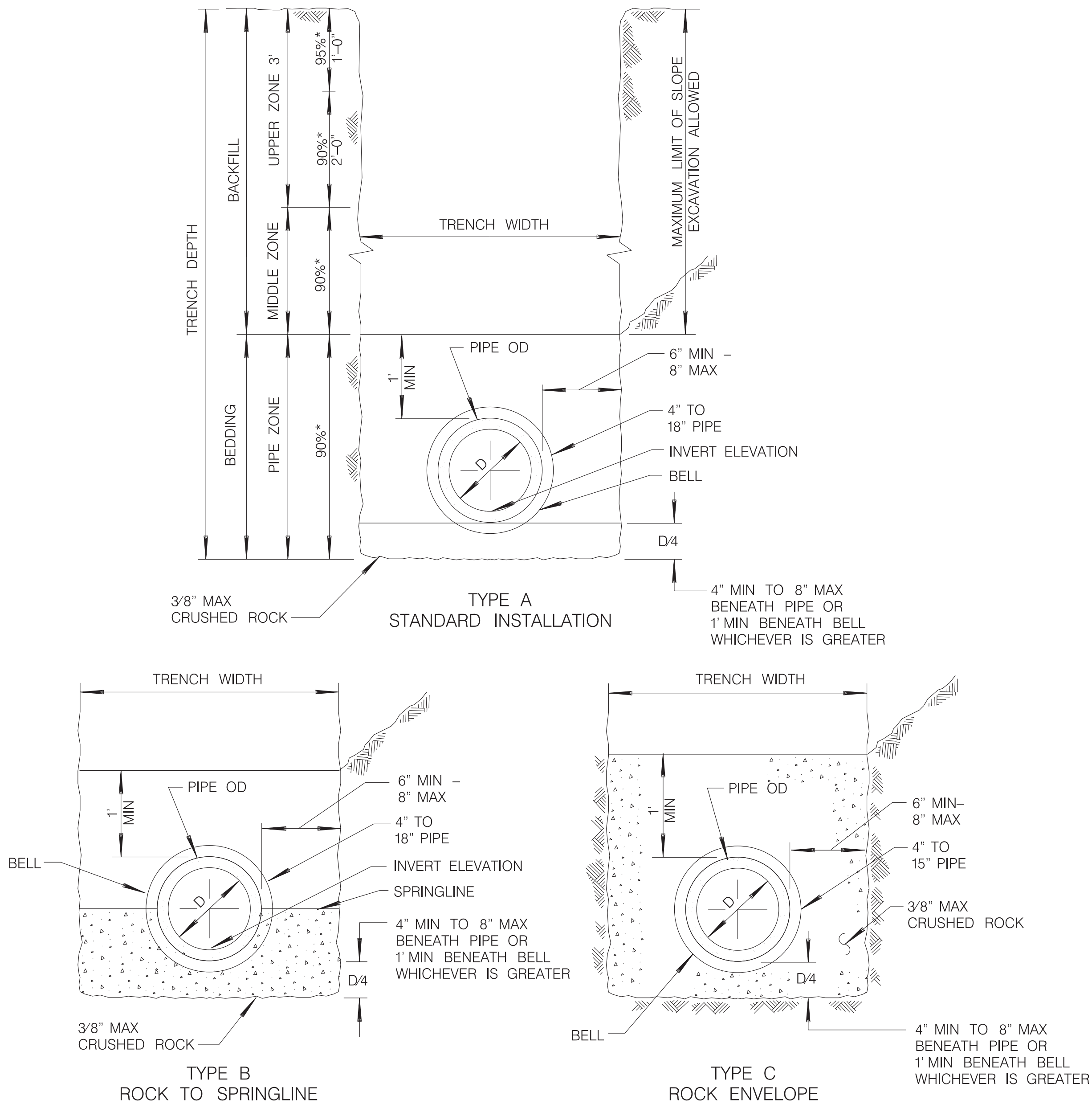
Maximum Credible Earthquake	7		**Borehole Diameter**	8	in.
Design Ground Motion	0.62	g	**Rod Length**	3	ft.
Total Unit Weight,	110	pcf	**Rod Length**	0.91	m.
Water Unit Weight,	62.4	pcf	**Liners**	N	
Depth to Groundwater	30	ft	**K aging**	1	
Depth to Groundwater	9.15	m		3	
Hammer Effenciency	85		**Percentile of Liquefaction**	84	
Required Factor of Safety	1.3				

Boring Data						Sampling Corrections							Corrected	Fines	Compute Deterministic Vertical Strain				Individual Layer
Depth		Blow Counts		Liquefiable		Sampler	SPT	Energy	Borehole	Rod	Liner	Overburden	SPT	Content					Subsidence
(ft)	(m)	SPT	Mod. Cal.	Soil (0 / 1)	σ_v' (kPa)	Diameter	N_m	C_E	C_B	C_R	C_L	C_N	$(N_1)_{60}$	%	$(N1)_{60,Cs}^{site}$	CRR(N^{site})	CSR^{site}	FS_L^{site}	(inches)
5	1.52		73	0	26.33	1	73	1.42	1.15	0.75	1.0	1.05	94	95	99.54	10.00		10.00	0.00
10	3.05		61	0	52.67	1	61	1.42	1.15	0.80	1.0	1.05	84	95	89.32	10.00		10.00	0.00
15	4.57		57	0	79.00	1	57	1.42	1.15	0.85	1.0	1.02	81	95	86.30	10.00		10.00	0.00
20	6.10	17		1	105.34	1	17	1.42	1.15	0.95	1.0	0.98	26	30	31.28	0.50		10.00	0.00
25	7.62	9		1	131.67	1	9	1.42	1.15	0.95	1.0	0.87	12	91	17.68	0.16		10.00	0.00
30	9.14	10		0	158.00	1	10	1.42	1.15	1.00	1.0	0.80	13	95	18.51	0.16		10.00	0.00
35	10.67	12		0	169.40	1	12	1.42	1.15	1.00	1.0	0.78	15	95	20.74	0.19	0.37	0.51	0.00
40	12.19	10		0	180.79	1	10	1.42	1.15	1.00	1.0	0.74	12	95	17.58	0.16	0.39	0.40	0.00
45	13.72	9		0	192.19	1	9	1.42	1.15	1.00	1.0	0.71	10	90	15.92	0.14	0.40	0.35	0.00
50	15.24	17		0	203.59	1	17	1.42	1.15	1.00	1.0	0.74	20	95	25.92	0.27	0.40	0.68	0.00

Based on *Proceeding of the NCEER Workshop on Evaluation of Liquefaction Resistance of Soils* , Technical Report NCEER-97-0022, December 31, 1997.
Sampling Corrections from Idriss and Boulanger (2010)

Total Settlement (in.) **0.00**

APPENDIX E



NOTES

1. FOR TRENCH RESURFACING IN IMPROVED STREETS, SEE STANDARD DRAWINGS SDG–107 AND SDG–108.
2. (*) INDICATES MINIMUM RELATIVE COMPACTION.
3. MINIMUM DEPTH OF COVER FROM THE TOP OF PIPE TO FINISH GRADE FOR PVC SDR 35 SEWER MAIN SHALL BE 5'. FOR SHALLOWER DEPTH, SPECIAL DESIGN IS REQUIRED. SEE SDS–101.
4. SEE TYPE A INSTALLATION FOR DETAILS NOT SHOWN FOR TYPES B AND C.
5. FOR PIPE SIZE ENCASEMENT LARGER THAN 15", MAXIMUM SIDE WALL CLEARANCE SHALL BE 12" OR AS SHOWN ON THE PLANS.
6. 6" METAL TAPE SHALL BE INSTALLED ABOVE PIPE 4" BELOW TRENCH CAP AND 12" BELOW FINISH GRADE IN UNIMPROVED STREETS.
7. 1' SAND CUSHION OR A 6" MINIMUM SAND CUSHION WITH 1" NEOPRENE PAD SHALL BE PLACED FOR CROSSINGS UTILITIES WHEN VERTICAL CLEARANCE IS 1' OR LESS. THE NEOPRENE PAD SHALL BE PLACED ON THE MOST FRAGILE UTILITY.

From: City of San Diego Standard Drawing SDS-110 (2016)

LANDMARK
Geo-Engineers and Geologists

Project No.: LE21050

Pipe Bedding and Trench Backfill Recommendations

Plate E-1