Geotechnical Report

ICPW Maintenance Building

Brawley, California

Prepared for:

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November 29, 2016

Mr. Jesus Aguilera
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Geotechnical Report
ICPW Maintenance Building
4736 Hwy 111
Brawley, California
LCI Report No. LE16213

Dear Mr. Aguilera:

This geotechnical report is provided for design and construction of the proposed replacement of the existing maintenance building at the Imperial County Public Works (ICPW) Department facility located at 4736 Hwy 111 in northern Brawley, 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.

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 (CH) of high expansion predominate the site. The near surface soils (upper 10 feet) had a hydrocarbon odor. ICPW personnel at the site indicted that an underground fuel storage tank was removed from the site and the excavation backfilled. Undocumented fill may be present at that location, possibly requiring deep removals.

- Foundation designs should mitigate expansive soil conditions by one of the following methods:

  1. Remove and replace upper 3.0 feet of clay soils with non-expansive sands.
  2. Design foundations to resist expansive forces in accordance with the 2013 California Building Code (CBC) Chapter 18, Section 1806. This requires grade-beam stiffened of floor slabs (17.5 feet maximum on center) or post-tensioned floor slabs. Design soil bearing pressure = 1,500 psf. Differential movement of 1.0 to 1.5 inches can be expected for slab on grade foundations placed on clay soils.
  3. A combination of the methods described above.

- The risk of liquefaction induced settlement is very low (estimated settlement of ½ inch at to 49 to 51.5 feet below ground surface. There is a very low risk of ground rupture should liquefaction occur.

- The clay soils are aggressive to concrete and steel. Concrete mixes shall have a maximum water cement ratio of 0.45 and a minimum compressive strength of 4,500 psi (minimum of 6.25 sacks Type V cement per cubic yard).

- 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 are not allowed at the foundation perimeter. No pressurized water lines are allowed below or within the foundations.

- Pavement structural sections should be designed for clay subgrade soils (R-Value = 5).

We did not encounter soil conditions that would preclude development of the proposed project 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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TABLE OF CONTENTS

Page

Appendices

Section 1
INTRODUCTION

1.1 Project Description

This report presents the findings of our geotechnical exploration and soil testing for the proposed replacement of the existing maintenance building at the Imperial County Public Works Department facility located at 4736 Hwy 111 in northern Brawley, California (See Vicinity Map, Plate A-1). The proposed development will consist of a 3,790 square foot pre-engineered metal building with a slab-on-grade foundation. A site plan for the proposed development was provided by the client at the time that this report was prepared.

The structure is planned to consist of a slab-on-grade foundation and steel-frame construction. Footing loads at exterior bearing walls are estimated at 0.5 to 3 kips per lineal foot. Column loads are estimated to range from 5 to 30 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, and concrete foundation construction.

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 expected 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

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

1.3 Authorization

Mr. Jimmy Sanders of Sanders, Inc. provided authorization by written agreement to proceed with our work on November 7, 2016. We conducted our work according to our written proposal dated August 26, 2016.

Section 2
METHODS OF INVESTIGATION

2.1 Field Exploration

Subsurface exploration was performed on November 10, 2016 using 2R Drilling of Ontario, California to advance one (1) boring to a depth of 51.5 feet below existing ground surface. The boring was advanced with a truck-mounted, CME 75 drill rig using 8-inch diameter, hollow-stem, continuous-flight augers. The approximate boring location was established in the field and plotted on the site map by sighting to discernible site features. The boring location is 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 visually classified during drilling according to the Unified Soil Classification System and 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. In addition, Standard Penetration Tests (SPT) were performed in accordance with ASTM D1586. 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 boring was backfilled with the excavated material. The backfill was loosely placed and was not compacted to the requirements specified for engineered fill.

The subsurface log is presented on Plate B-1 in Appendix B. A key to the log symbols is presented on Plate B-2. 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.

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) – used for soil classification and expansive soil design criteria
- Unconfined Compression (ASTM D2166) – used for soil strength estimates.
- Chemical Analyses (soluble sulfates & chlorides, pH, and resistivity) (Caltrans Methods) – used for concrete mix proportions and corrosion protection requirements.

The laboratory test results are presented on the subsurface log (Appendix B) and on Plates C-1 and C-2 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

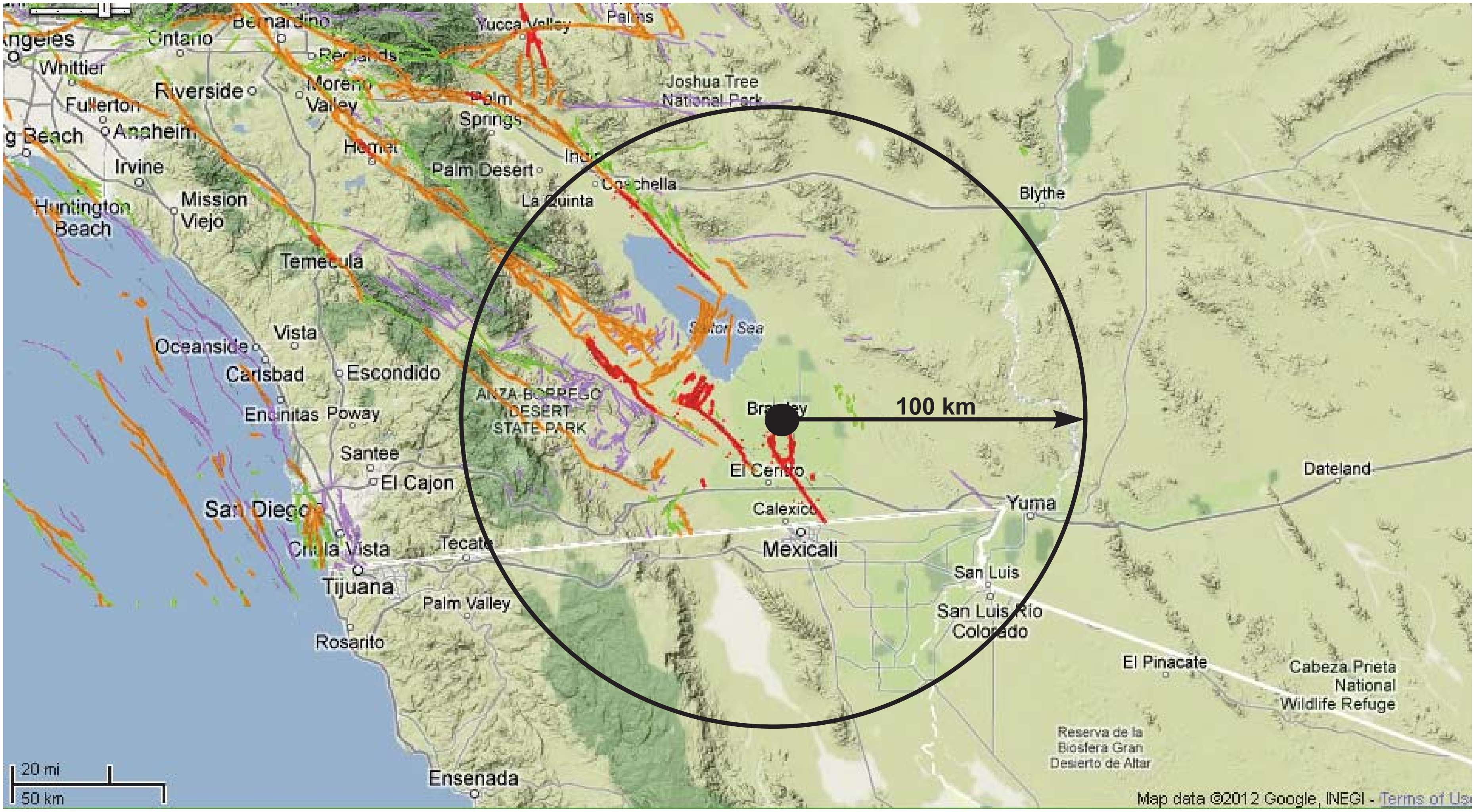
The project site is located on the south side of the Imperial County Department of Public Works maintenance yard located at 4736 Hwy 111 in northern Brawley, California. An existing maintenance building occupies the proposed location of the new building.

Adjacent properties are flat-lying and are approximately at the same elevation with this site. Adjacent properties consist of trucking yards to the south and Rubin Seeds, Inc. to the north and east. State Hwy 111 is located to the west.

The project site lies at an elevation of approximately 130 feet below mean sea level (MSL) (El. 870 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 Imperial Valley portion of the Salton Trough physiographic province. The Salton Trough is a topographic and geologic structural depression resulting from large scale regional faulting. The 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.



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

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Project No.: LE16213

Regional Fault Map

Figure 1

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 U. S. Soil Conservation Service compiled a map of surface soil conditions based on a thirteen-year study from 1962-1975 (Zimmerman, 1981). The Soil Survey maps were published in 1981 and indicate that surficial deposits at the project site and surrounding area consist predominantly of silty clay and silty clay loams of the Imperial-Glenbar 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).

Subsurface soils encountered during the field exploration conducted on November 10, 2016 consist of stiff silty clays to a depth of about 49 feet. A medium dense silty sand was encountered at a depth of 49 to 51.5 feet, the maximum depth of exploration. The subsurface logs (Plate B-1) depict the stratigraphic relationships of the various soil types.

The native surface clays likely exhibit high swell potential (Expansion Index, EI = 110 to 140) when correlated to Plasticity Index tests (ASTM D4318) performed on the native clays. The clay is expansive when wetted and can shrink with moisture loss (drying). Development of building foundations, concrete flatwork, and asphaltic concrete pavements 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 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 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 silt/clay soil.
- A combination of the methods described above

3.4 Groundwater

Groundwater was encountered in the boring at about 36 feet during the time of exploration, but may rise with time to approximately 15 to 20 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.

3.5 Faulting

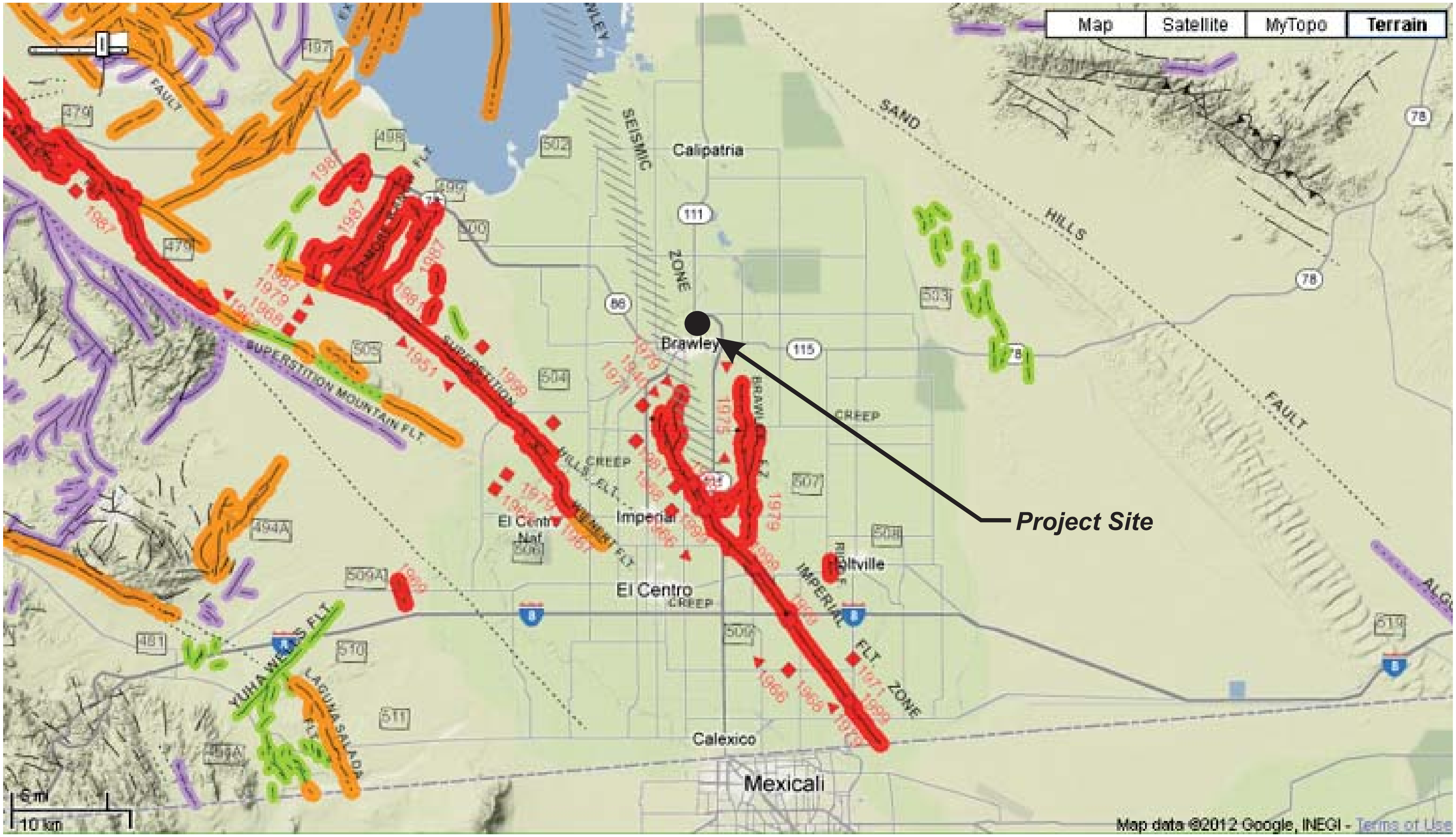
The project site is located in the seismically active Imperial Valley of southern California with numerous mapped faults of the San Andreas Fault System traversing the region. The San Andreas Fault System is comprised of the San Andreas, San Jacinto, and Elsinore Fault Zones in southern California. The 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 62 mile (100 kilometer) 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 active or potentially active faults. An active fault is one that has ruptured during Holocene time (roughly within the last 11,000 years). A fault that has ruptured during the last 1.8 million years (Quaternary time), but has not been proven by direct evidence to have not moved within Holocene time is considered to be potentially active.

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	3.8	6.1	7	62 ± 6	20 ± 5
Brawley *	4.1	6.5			
Superstition Hills	10.3	16.4	6.6	23 ± 2	4 ± 2
Superstition Mountain	12.1	19.3	6.6	24 ± 2	5 ± 3
Rico *	14.5	23.2			
Elmore Ranch	16.8	26.9	6.6	29 ± 3	1 ± 0.5
Unnamed 1*	22.1	35.4			
Unnamed 2*	23.4	37.5			
Painted Gorge Wash*	23.6	37.8			
Yuha*	24.2	38.7			
Yuha Well *	24.2	38.7			
Shell Beds	24.4	39.0			
Vista de Anza*	26.8	42.9			
San Jacinto - Borrego	26.9	43.0	6.6	29 ± 3	4 ± 2
San Andreas - Coachella	27.1	43.4	7.2	96 ± 10	25 ± 5
Hot Springs *	27.1	43.4			
Laguna Salada	28.0	44.8	7	67 ± 7	3.5 ± 1.5
Ocotillo*	30.2	48.4			
Borrego (Mexico)*	31.3	50.1			
Elsinore - Coyote Mountain	32.6	52.2	6.8	39 ± 4	4 ± 2
Cerro Prieto *	34.1	54.6			
Algodones *	35.8	57.3			

* Note: Faults not included in CGS database.



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

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Project No.: LE16213

Map of Local Faults

Figure 2

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

491

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

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

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Project No.: LE16213

Fault Map Legend

Figure 3b

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.

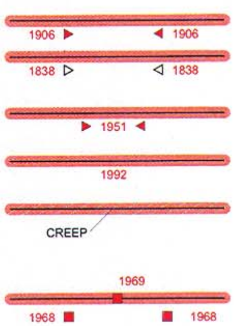
FAULT CLASSIFICATION COLOR CODE
(Indicating Recency of Movement)

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.



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.

Date bracketed by triangles indicates local fault break.

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

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.

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.

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Project No.: LE16213

Fault Map Legend

Figure 3a

A fault that has not moved during Quaternary time is considered to be inactive. Review of the current Alquist-Priolo Earthquake Fault Zone maps (CGS, 2016) indicates that the nearest mapped Earthquake Fault Zone is the Imperial fault located approximately 3.8 miles south of the project site.

The project site lies within the Brawley Seismic Zone (BSZ), a pull-apart basin between the southern terminus of the San Andreas fault and the northern trace of the Imperial fault. The BSZ is composed of numerous cross-cutting high angle normal faults. The BSZ extends northward beyond the termination of the mapped Imperial/Brawley faults to beneath the Salton Sea, where it terminates upon intersecting the San Andreas fault near Bombay Beach. The Brawley Seismic Zone was the source of the 1981 5.9M_W Westmorland earthquake sequence that involved activity on at least seven distinct fault planes within the zone. The faults in the Brawley Seismic Zone are considered to be short enough that earthquakes much larger than 6-6.5M_W are unlikely. The California Geological Survey considers the Brawley Seismic Zone to have a maximum magnitude of 6.4Mw, with a very short 24-year average return interval, and a geologic slip rate of 25 mm/year (CDMG, 1996).

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>CBC General Ground Motion Parameters:</u> The 2013 CBC general ground motion parameters are based on the Risk-Targeted Maximum Considered Earthquake (MCE_R). The U.S. Geological Survey "U.S. Seismic Design Maps Web Application" (USGS, 2016) was used to obtain the site coefficients and adjusted maximum considered earthquake spectral response acceleration parameters. **The site soils have been classified as Site Class D (stiff soil profile).**

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. Design earthquake ground motion parameters are provided in Table 2. **A Risk Category II was determined using Table 1604A.5 and the Seismic Design Category is D since S_1 is less than 0.75g.**

The Maximum Considered Earthquake Geometric Mean (MCE_G) peak ground acceleration (PGA_M) value was determined from the "U.S. Seismic Design Maps Web Application" (USGS, 2016) for liquefaction and seismic settlement analysis in accordance with 2013 CBC Section 1803A.5.12 and CGS Note 48 ($PGA_M = F_{PGA}$*PGA). **A PGA_M value of 0.53g has been determined for the project site.**

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, Brawley, and Superstition Hills faults.
- **Surface Rupture.** The California Geological Survey (2016) 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.** Liquefaction is unlikely to be a potential hazard at the site due to the lack of saturated granular soil (clay soils predominate). The potential for liquefaction at the site is discussed in more detail in Section 3.8.

Other Potential Geologic Hazards.

- **Landsliding.** The hazard of landsliding is unlikely due to the regional planar topography. No ancient landslides are shown on geologic maps of the region and no indications of landslides were observed during our site investigation.
- **Volcanic hazards.** The site is not located in proximity to any known volcanically active area and the risk of volcanic hazards is considered very low.
- **Tsunamis and seiches.** The site is not located near any large bodies of water, so the threat of tsunami, seiches, or other seismically-induced flooding is unlikely.
- **Flooding.** The project site is located in FEMA Flood Zone X, an area determined to be outside the 0.2% annual chance floodplain (FIRM Panel 06025C1375C).

Table 2
2013 California Building Code (CBC) and ASCE 7-10 Seismic Parameters

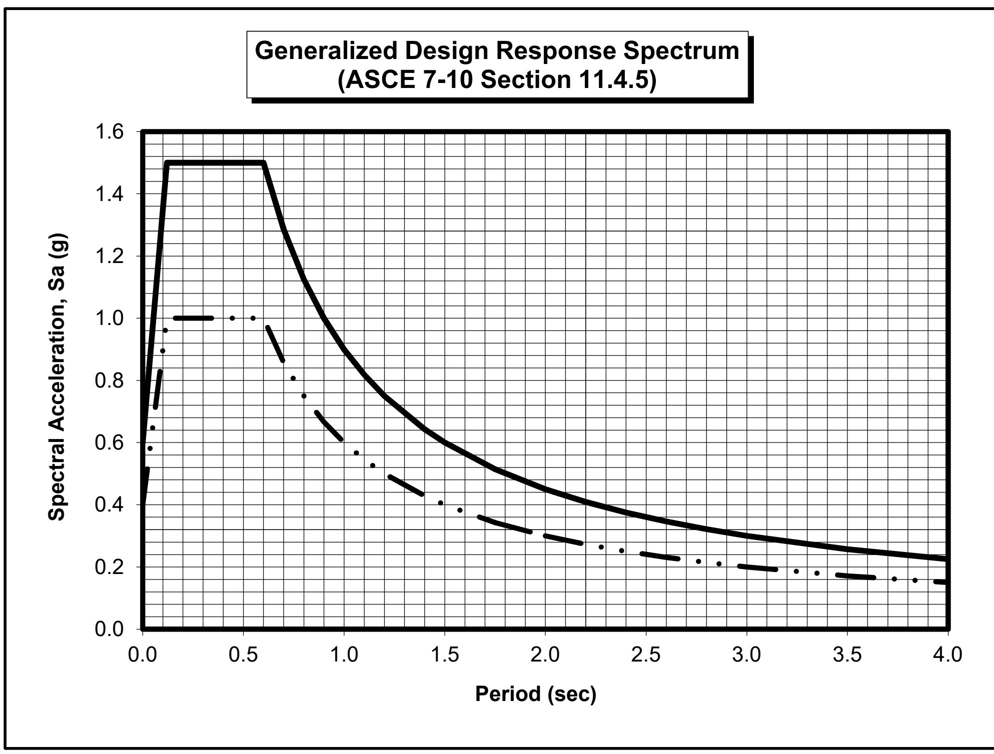
			CBC Reference
Soil Site Class:	**D**		Table 20.3-1
Latitude:	32.9958	N	
Longitude:	-115.5257	W	
Risk Category:	II		
Seismic Design Category:	D		

Maximum Considered Earthquake (MCE) Ground Motion

Mapped MCE_R Short Period Spectral Response	$\mathbf{S_s}$	1.500 g	Figure 1613.3.1(1)	
Mapped MCE_R 1 second Spectral Response	$\mathbf{S_1}$	0.600 g	Figure 1613.3.1(2)	
Short Period (0.2 s) Site Coefficient	$\mathbf{F_a}$	1.00	Table 1613.3.3(1)	
Long Period (1.0 s) Site Coefficient	$\mathbf{F_v}$	1.50	Table 1613.3.3(2)	
MCE_R Spectral Response Acceleration Parameter (0.2 s)	$\mathbf{S_{MS}}$	1.500 g	$= F_a * S_s$	Equation 16-37
MCE_R Spectral Response Acceleration Parameter (1.0 s)	$\mathbf{S_{M1}}$	0.900 g	$= F_v * S_1$	Equation 16-38

Design Earthquake Ground Motion

Design Spectral Response Acceleration Parameter (0.2 s)	$\mathbf{S_{DS}}$	1.000 g	$= 2/3*S_{MS}$	Equation 16-39
Design Spectral Response Acceleration Parameter (1.0 s)	$\mathbf{S_{D1}}$	0.600 g	$= 2/3*S_{M1}$	Equation 16-40
	$\mathbf{T_L}$	8.00 sec		ASCE Figure 22-12
	$\mathbf{T_O}$	0.12 sec	$=0.2*S_{D1}/S_{DS}$	
	$\mathbf{T_S}$	0.60 sec	$=S_{D1}/S_{DS}$	
Peak Ground Acceleration	$\mathbf{PGA_M}$	0.53 g		ASCE Equation 11.8-1



Period T (sec)	Sa (g)	MCE_R Sa (g)
0.00	0.40	0.60
0.12	1.00	1.50
0.60	1.00	1.50
0.70	0.86	1.29
0.80	0.75	1.13
0.90	0.67	1.00
1.00	0.60	0.90
1.10	0.55	0.82
1.20	0.50	0.75
1.20	0.50	0.75
1.40	0.43	0.64
1.50	0.40	0.60
1.75	0.34	0.51
2.00	0.30	0.45
2.20	0.27	0.41
2.40	0.25	0.38
2.60	0.23	0.35
2.80	0.21	0.32
3.00	0.20	0.30
3.50	0.17	0.26
4.00	0.15	0.23

Design Response Spectra
MCE_R Response Spectra

- **Expansive soil.** 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 soil below the water table is subjected to vibratory motions, such as produced by earthquakes. With strong ground shaking, an increase in pore water pressure develops 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.

Methods of Analysis: Liquefaction potential at the project site was evaluated using the 1997 NCEER Liquefaction Workshop methods. The 1997 NCEER methods utilize direct SPT blow counts or CPT cone readings 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 $N_{1(60)}$ or Qc_{1N}. A PGA_M value of 0.53g was used in the analysis with a 20-foot groundwater depth and a threshold factor of safety (FS) of 1.3. The fine content of liquefiable sands and silts increases the liquefaction resistance in that more ground motion cycles are required to fully develop 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 Robertson and Wride (1997). The corrected blow counts were then converted to equivalent clean sand blow counts ($N_{1(60)cs}$).

The soil encountered at the points of exploration included saturated silts and silty sands that could liquefy during a Maximum Considered Earthquake. Liquefaction can occur within a 3-foot thick silty sand layer at a depth of 49 feet below ground surface. The likely triggering mechanism for liquefaction appears to be strong groundshaking associated with the rupture of the Imperial fault.

Liquefaction Induced Settlements: ***Based on empirical relationships, total induced settlements are estimated to be about ½ inch should liquefaction occur.*** The magnitude of potential liquefaction induced differential settlement is estimated at be two-thirds of the total potential settlement in accordance with California Special Publication 117; therefore, there is a potential for ¼ inch of liquefaction induced differential settlement at the project site.

Because of the depth of the liquefiable layer, the 49-foot thick non-liquefiable clay layer may act as a bridge over the liquefiable layer resulting in a fairly uniform ground surface settlement; therefore, wide area subsidence of the soil overburden would be the expected effect of liquefaction rather than bearing capacity failure of the proposed structures.

Mitigation: Based on an estimate of ½ inch of liquefaction induced settlements, no mitigation for liquefaction induced settlements is required at this project site.

Section 4
DESIGN CRITERIA

4.1 Site Preparation

Clearing and Grubbing: All surface improvements, debris or vegetation including grass, trees, and weeds 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, 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. 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.

Building Pad Preparation: The exposed surface soil within the building pad/foundation areas should be removed to 36 inches below the building pad elevation or existing natural surface grade (whichever is lower) extending five 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 not suitable for use as engineered fill because of the hydrocarbon odor present in the soil. The EPA reports "passive" vapor intrusion mitigation methods may be used to prevent entry of chemical vapors in to buildings. Passive methods include installation of vapor barriers beneath the floor slab, passive venting, and sealing of cracks/openings in the floor slab. It is suggested that, as a minimum, a continuously sealed 30-mil vapor retarder be placed at the bottom of the excavation prior to placement of engineered fill. A second vapor retarder should underlie the concrete slab. Consideration should be made for the complete removal of hydrocarbon affected soils within the occupied building areas.

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 granular soil, 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 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%.

In areas other than the building pad which are to receive sidewalks or area concrete slabs, the ground surface should be presaturated to a minimum depth of 24 inches and then scarified to 8 inches, moisture conditioned to a minimum of 5% over optimum, and recompacted to 83-87% of ASTM D1557 maximum density just prior to concrete placement.

Moisture Control and Drainage: If clay soils are used at building pads (without 3.0 feet of granular, non-plastic soil), the moisture condition of the building pad should be maintained during trenching and utility installation until concrete is placed or should be rewetted by use of multiple applications of water with sprinklers before initiating delayed construction. If soil drying is noted in footings, a 2 to 3 inch depth of water may be used in the bottom of footings to restore footing subgrade moisture and reduce potential edge lift.

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 5 feet minimum across unpaved areas) to prevent ponding and subsequent saturation of the native clay soil. Gutters and downspouts should be used as a means to convey water away from foundations. If landscape irrigation is allowed next to the building, drip irrigation systems or lined planter boxes should be used. The subgrade soil around the entire foundation should be maintained in a moist, but not saturated state, and not allowed to dry out. The developer should consider utilizing drip irrigation systems around the entire building perimeter to maintain soil moisture. Drainage should be maintained without ponding. Trees should be set back from foundations a minimum of 20 feet from the foundation.

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 Utility Trench Backfill

Utility Trench Backfill: Trench backfill for utilities should conform to San Diego Regional Standard Drawing S-4 (Appendix D), using either Type A, B or C backfill.

Type A backfill for HDPE pipe (above groundwater) consists of a 4 to 6 inch bed of ¾-inch crushed rock below the pipe and pipezone backfill (to 12" above top of pipe) consisting of crusher fines (sand). Sewer pipes (SDR-35), water mains, and stormdrain pipes of other than HDPE pipe may use crusher fines for bedding. The crusher fines 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 compacted to a minimum of 90% of ASTM D1557 maximum density.

Type B backfill for HDPE pipe (shallow cover) requires 6 inches of ¾-inch crushed rock as bedding and to springline of the pipe. Thereafter, sand/cement slurry (3 sack cement factor) should be used to 12 inches above the top of the pipe. Native clay and silt soils may be used in the remainder of the trench backfill as specified above.

Type C backfill for HDPE pipe (below or partially below groundwater) shall consist of a geotextile filter fabric encapsulating ¾-inch crushed rock. The crushed rock thickness shall be 6 inches below and to the sides of the pipe and shall extend to 12 inches above the top of the pipe. The filter fabric shall cover the trench bottom, sidewalls and over the top of the crushed rock. Native clay and silt soils may be used in the remainder of the trench backfill as specified above.

Type C backfill must be used in wet soils and below groundwater for all buried utility pipelines. Dewatering (by well points) is required to at least 24 inches below the trench bottom prior to excavation. Type A backfill may be used in the case of a dewatered trench condition in clay soils only.

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.

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).

4.3 Foundations and Settlements

Shallow spread 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. 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 or 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 1,500 psf for compacted clay soil and 2,000 psf when foundations are supported on imported sands (extending a minimum of 1.0 feet below footings). 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).

As an alternative to shallow spread foundations, flat plate structural mats or grade-beam reinforced foundations may be used to mitigate expansive soil heave and/or liquefaction related movement.

Flat Plate Structural Mats: 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 1808A.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 (without replacement of the surface clays with 3.0 feet of granular fill or lime treated soil placed over native clays) are presented below in accordance with the design method given in CBC Chapter 18 Section 1808A.6.2 (*WRI/CRSI Design of Slab-on-Ground Foundations*):

Weighted Plasticity Index (PI) = 40
Slope Coefficient (C_s) = 1.0
Strength Coefficient (C_o) = 0.8
Climatic Rating (C_w) = 15
Effective PI = 32
Maximum Grade-beam Spacing = 17.5 feet

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 (300 pcf for imported sands) 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 (0.35 for imported sands) may also be used at the base of the footings to resist lateral loading.

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 of the surrounding land mass and structure may be on the order of ½ inch (total) and ¼ inch (differential).

4.4 Slabs-On-Grade

<u>Structural Concrete:</u> 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 2013 CBC and shall be a minimum of 6 inches thick due to expansive soil conditions and anticipated wheel loads. Concrete floor slabs shall be monolithically placed with the footings (no cold joints) unless placed on 3.0 feet of granular fill or lime treated soil.

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 30-mil polyethylene vapor retarder that works as a capillary break to reduce moisture migration into the slab section and migration of hydrocarbon vapors. All laps and seams should be fully sealed 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 fully line the footing excavations.

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 30-mil (Stego-Wrap or equivalent).

Structural concrete slab reinforcement should consist of chaired rebar slab reinforcement (minimum of No. 4 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 and uncovered housekeeping slabs) shall be a minimum of 4 inches thick and should be placed on a minimum of 2 inches of concrete sand or aggregate base, dowelled to the perimeter foundations where adjacent to the building to prevent separation and sloped 2% (sidewalks) or 1 to 2% (housekeeping slabs) away from the building. Housekeeping 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.

A minimum of 24 inches of moisture conditioned (5% minimum above optimum) and 8 inches of compacted subgrade (85 to 90%) should underlie all independent flatwork. 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.

4.5 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-2). The native soils were found to have severe levels of sulfate ion concentration (5,341 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:

Table 4. Concrete Mix Design Criteria due to Soluble Sulfate Exposure

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

Note: from ACI 318-11 Table 4.2.1

A minimum of 6.25 sacks per cubic yard of concrete (4,500 psi) of Type V Portland Cement with a maximum water/cement ratio of 0.45 (by weight) should be used for concrete placed in contact with native soil on this project (sitework including streets, sidewalks, driveways, patios, and foundations). Admixtures may be required to allow placement of this low water/cement ratio concrete. Thorough concrete consolidation and hard trowel finishes should be used due to the aggressive soil exposure.

The native soil has very severe levels of chloride ion concentration (2,500 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.

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 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.

4.6 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.

4.7 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 Brawley, Superstition Hills, and Imperial 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.

4.8 Pavements

Pavements should be designed according to the 2012 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 estimated 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.

Table 5. Pavement Structural Sections

R-Value of Subgrade Soil - 5 (estimated) Design Method - Caltrans 2012

Traffic Index (assumed)	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 B, ¾ inch maximum (½ inch maximum for parking areas), medium grading with PG70-10 asphalt cement, compacted to a minimum of 95% of the Hveem density (CAL 366).
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) native clay soil compacted to a minimum of 90% of the maximum dry density determined by ASTM D1557.
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
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 replacement of the existing maintenance building at the Imperial County Public Works Department facility located at 4736 Hwy 111 in northern Brawley, 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.

Findings and professional opinions in this report are based on selected points of field exploration, geologic literature, 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. If detected, these conditions may require additional studies, consultation, and possible design revisions.

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.

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.

This report should be considered invalid for periods after two years from the report date 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.

The client has responsibility to see that all parties to the project including, designer, contractor, and subcontractor are made aware of this entire report. The use of information contained in this report for bidding purposes should be done at the contractor's option and risk.

5.2 Additional Services

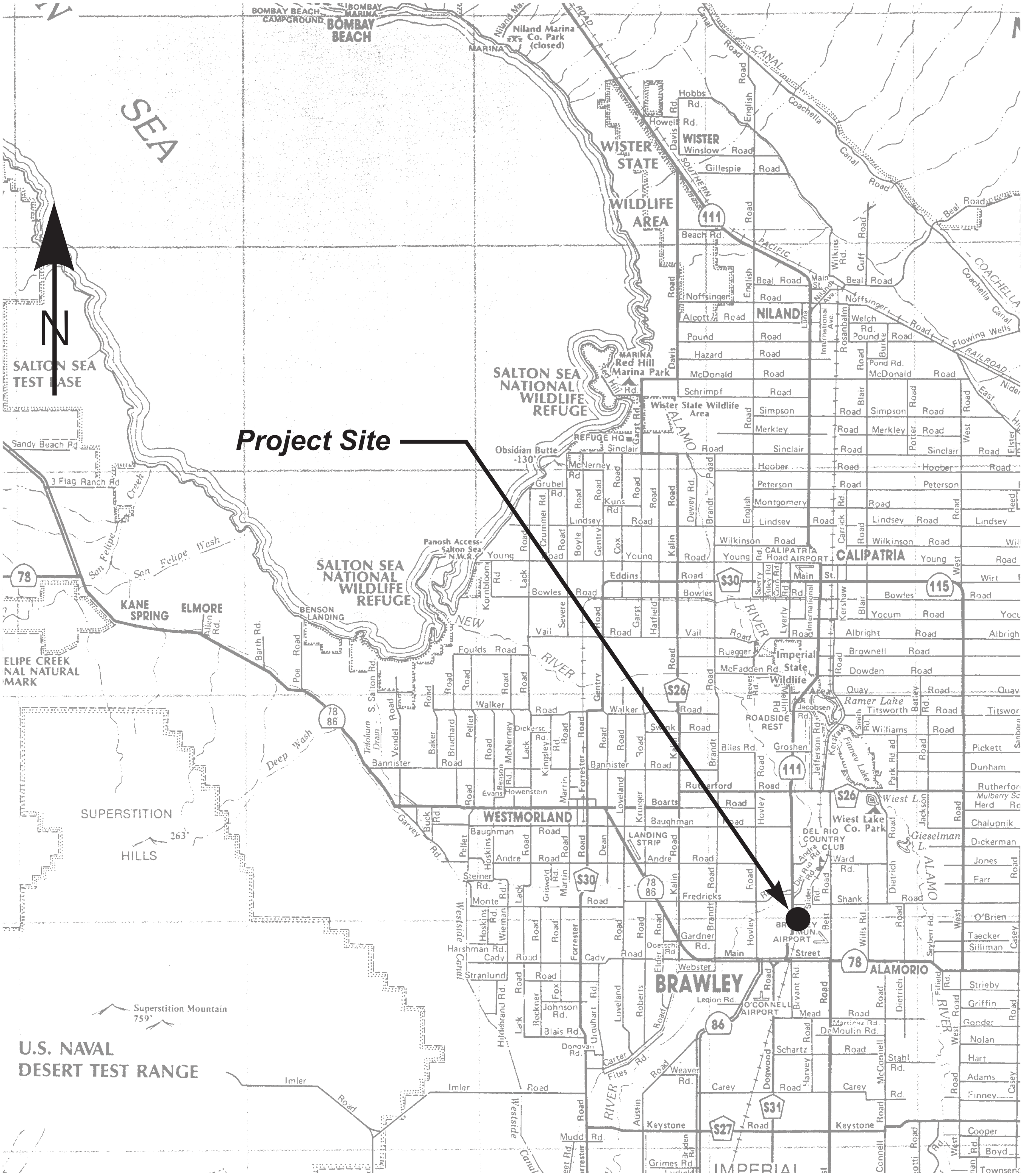
We recommend that a qualified geotechnical consultant be retained to provide the tests and observations services during construction. *The geotechnical engineering firm providing such tests and observations shall become the geotechnical engineer of record and assume responsibility for the project.*

The professional opinions presented in this report are based on the assumption that:

- Consultation 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 Consultants will have the opportunity to review and comment on the plans and specifications for the project prior to the issuance of such for bidding.
- Observation, inspection, and testing by the geotechnical consultant of record during site clearing, grading, excavation, placement of fills, building pad and subgrade preparation, and backfilling of utility trenches.
- Observation of foundation excavations and reinforcing steel before concrete placement.
- Other consultation as necessary during design and construction.

We emphasize our review of the project plans and specifications to check for compatibility with our professional opinions and conclusions. Additional information concerning the scope and cost of these services can be obtained from our office.

APPENDIX A



LANDMARK
Geo-Engineers and Geologists

Project No.: LE16213

Vicinity Map

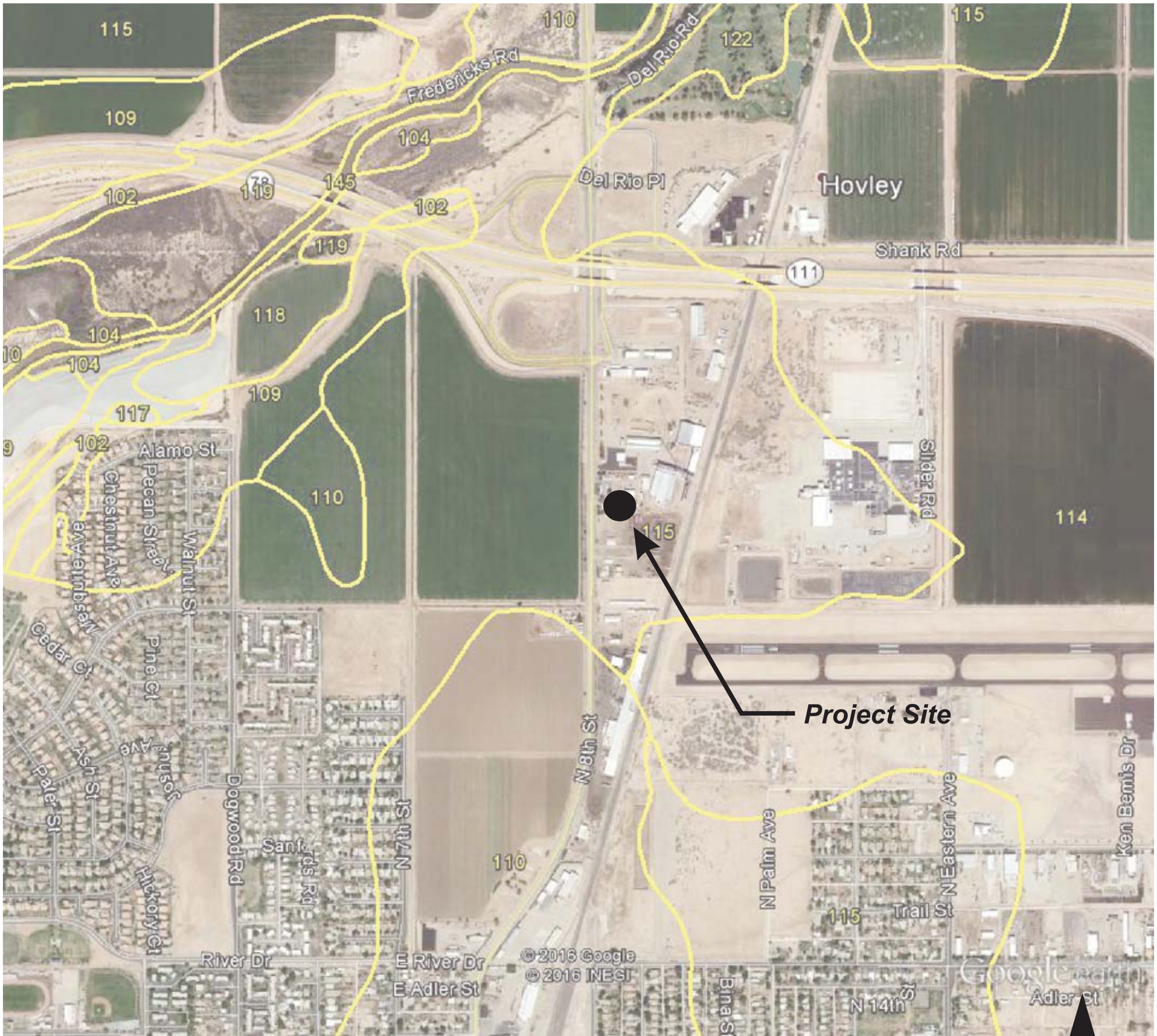
Plate
A-1

LANDMARK
Geo-Engineers and Geologists

Project No.: LE16213

Site and Exploration Plan

Plate
A-2



LANDMARK
Geo-Engineers and Geologists

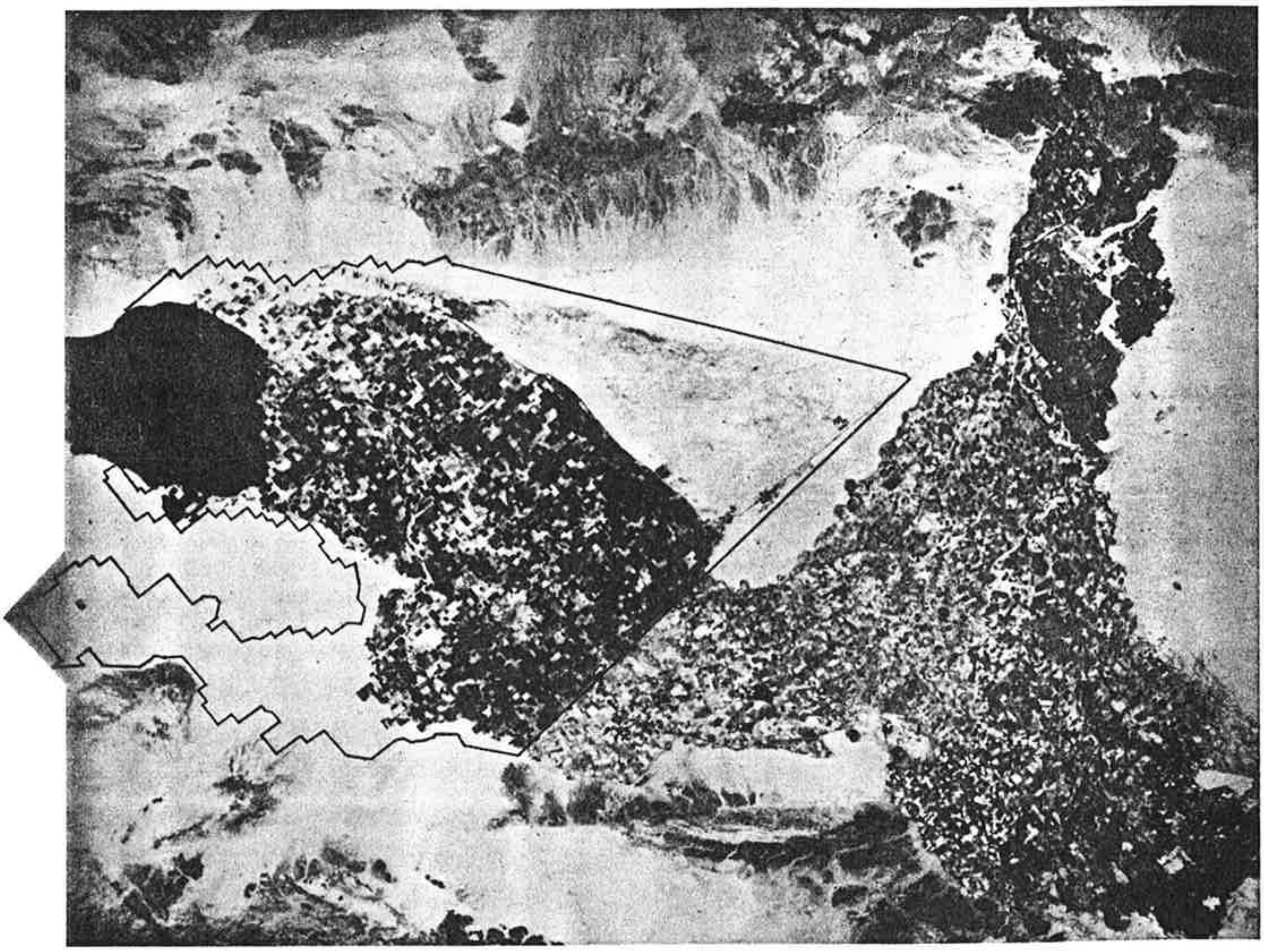
Project No.: LE16213

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

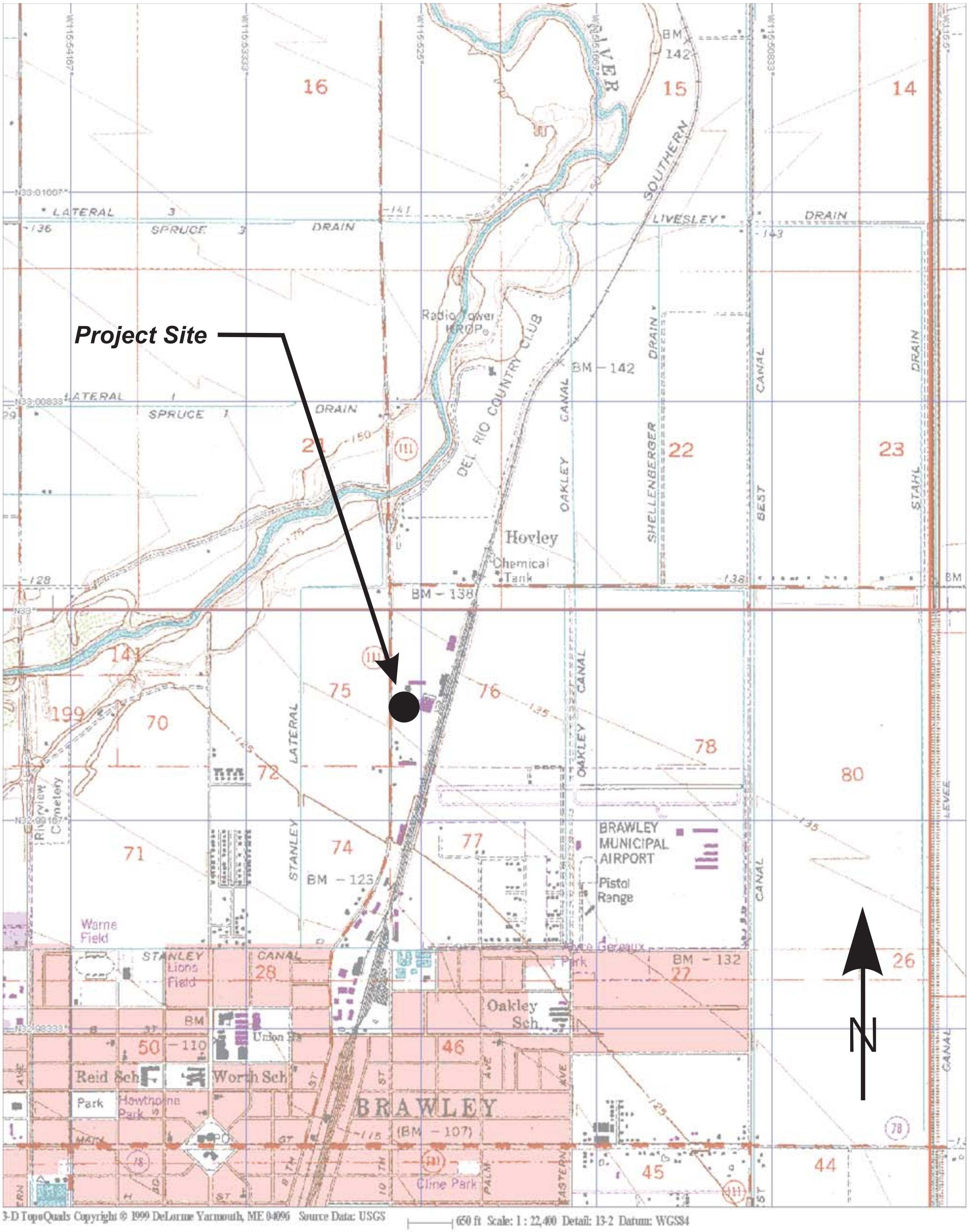
Soil name and map symbol	Depth	USDA texture	Classification		Frag-ments > 3 inches	Percentage passing sieve number--				Liquid limit	Plas-ticity index
			Unified	AASHTO		4	10	40	200		
	In				Pct					Pct	
121----------------- Meloland	0-12	Fine sand-------	SM, SP-SM	A-2, A-3	0	95-100	90-100	75-100	5-30	---	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		Frag- ments > 3 inches	Percentage passing sieve number--				Liquid limit	Plas- ticity 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
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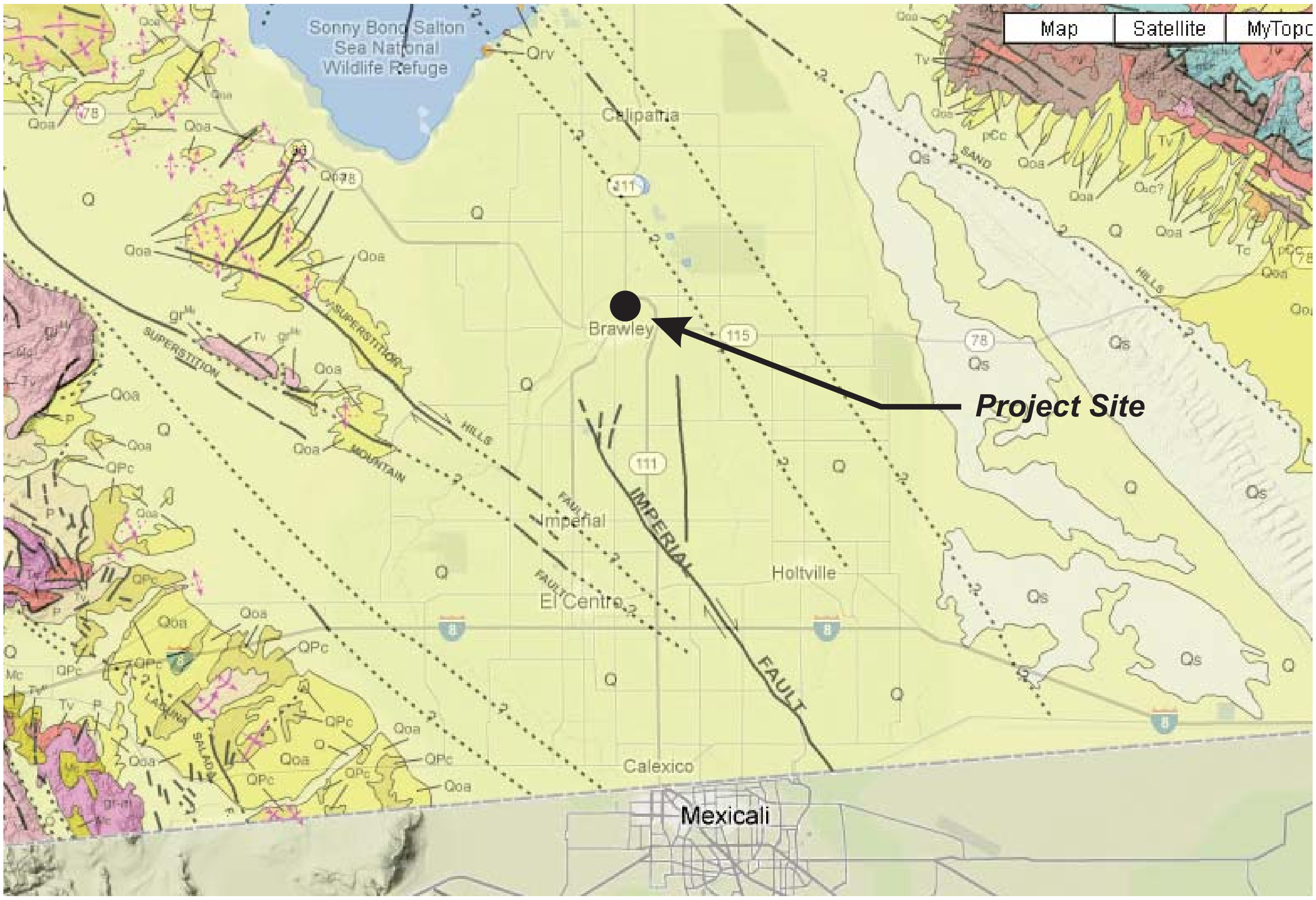
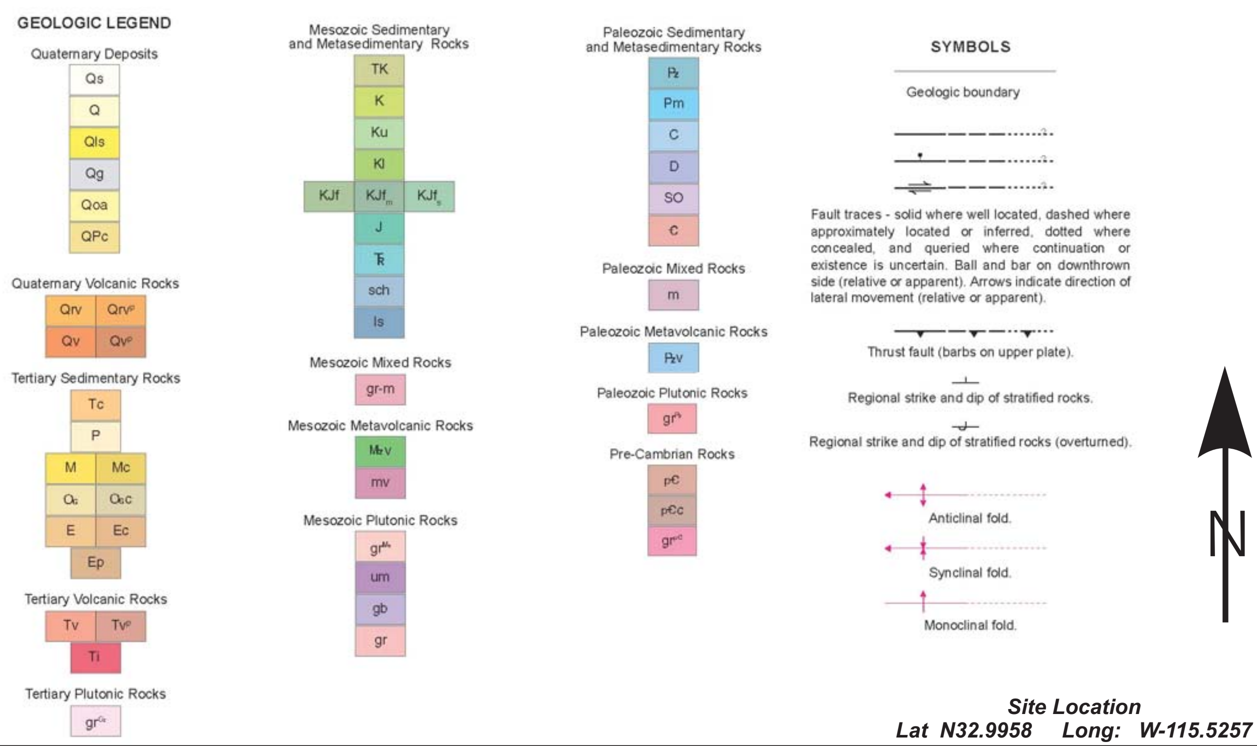
Project No.: LE16213

Topographic Map

Plate A-4





Site Location
Lat N32.9958 Long: W-115.5257

LANDMARK
Geo-Engineers and Geologists

Project No.: LE16213

Regional Geologic Map

Plate A-5

APPENDIX B

LOG OF BORING No. B-1

DEPTH	FIELD: BLOW COUNT	FIELD: POCKET PEN. (tsf)	DESCRIPTION OF MATERIAL	LABORATORY: DRY DENSITY (pcf)	LABORATORY: MOISTURE CONTENT (% dry wt.)	LABORATORY: OTHER TESTS
			2.5 inches of asphaltic			
						LL=60% PI=40%
			CLAY (CH): Brown and black, moist, stiff, hydrocarbon odors			
5	15			97.8	24.8	c=0.77 tsf
10	27					
			SILTY CLAY (CL): Brown, moist, stiff, medium plasticity			
15	20					
20	13	1.5				
25	8	0.25	soft			
30	20	4.0	hard			
35	14	2.5	GW @ 36 ft. at time of drilling			
40	12	1.25				
45	8	0.5				
50	13		SILTY SAND (SM): Lt. brown, saturated, medium dense, fine grained			
55			Total Depth = 51.5' Backfilled with excavated soil			
60						

DATE DRILLED: 11/10/16

LOGGED BY: P. LaBrucherie

SURFACE ELEVATION: Approximately -130'

TOTAL DEPTH: 51.5 Feet

TYPE OF BIT: Hollow Stem Auger

HAMMER WT.: 140 lbs.

DEPTH TO WATER: 36 ft.

DIAMETER: 8 in.

DROP: 30 in.

PROJECT NO. LE15112

LANDMARK Geo-Engineers and Geologists

DEFINITION OF TERMS

PRIMARY DIVISIONS			SYMBOLS	SECONDARY DIVISIONS
Coarse grained soils More than half of material is larger that No. 200 sieve	Gravels: More than half of coarse fraction is larger than No. 4 sieve	Clean gravels (less than 5% fines)	GW	Well graded gravels, gravel-sand mixtures, little or no fines
			GP	Poorly graded gravels, 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: More than half of coarse fraction is smaller than No. 4 sieve	Clean sands (less than 5% fines)	SW	Well graded sands, gravelly sands, little or no fines
			SP	Poorly graded sands 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: Liquid limit is less than 50%		ML	Inorganic silts, clayey silts with slight plasticity
			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: Liquid limit is more than 50%		MH	Inorganic silts, micaceous or diatomaceous silty soils, elastic silts
			CH	Inorganic clays of high plasticity, fat clays
			OH	Organic clays of medium to high plasticity, organic silts
Highly organic soils			PT	Peat and other highly organic soils

GRAIN SIZES

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. LE16213

Key to Logs

Plate B-2

APPENDIX C

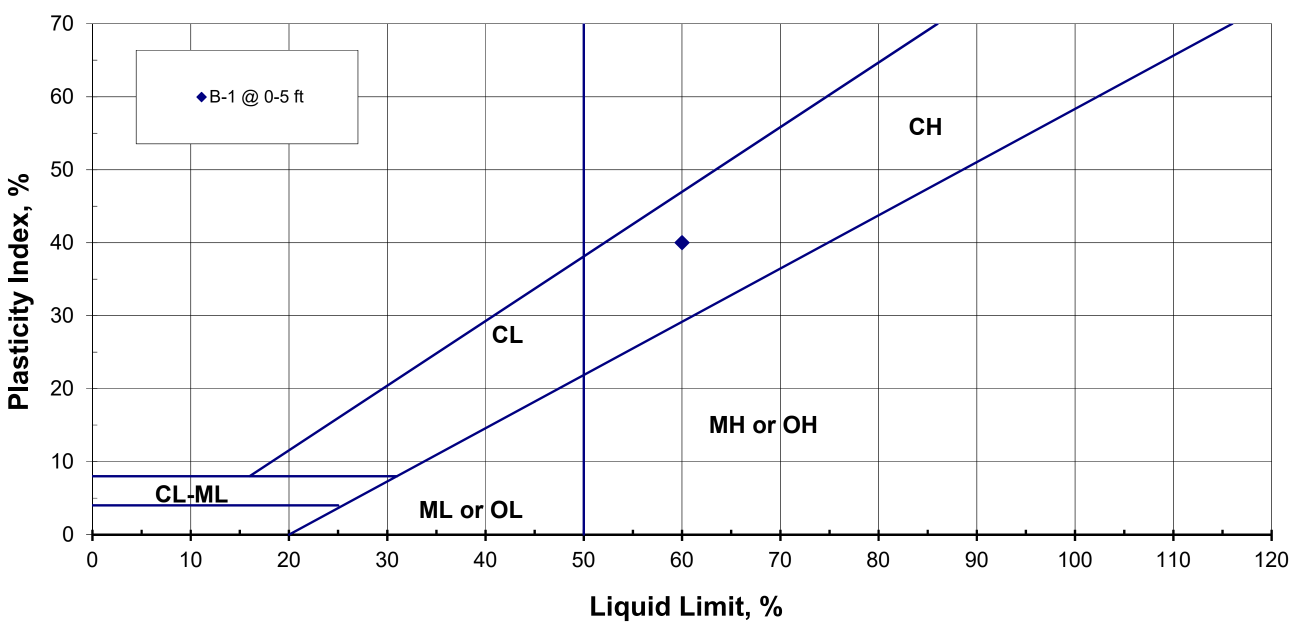
LANDMARK CONSULTANTS, INC.

CLIENT: Sanders, Inc.
PROJECT: ICPW Maintenance Building -- Brawley, CA
JOB No.: LE16213
DATE: 11/17/16

ATTERBERG LIMITS (ASTM D4318)

Sample Location	Sample Depth (ft)	Liquid Limit (LL)	Plastic Limit (PL)	Plasticity Index (PI)	USCS Classification
B-1	0-5	60	20	40	CH

PLASTICITY CHART



LANDMARK Geo-Engineers and Geologists

Project No.: LE16213

Atterberg Limits Test Results

Plate C-1

LANDMARK CONSULTANTS, INC.

CLIENT: Sanders, Inc.
PROJECT: ICPW Maintenance Building -- Brawley, CA
JOB No.: LE16213
DATE: 11/17/16

CHEMICAL ANALYSIS

		Caltrans Method
Boring:	B-1	
Sample Depth, ft:	0-5	
pH:	8.0	**643**
Electrical Conductivity (mmhos):	3.6	**424**
Resistivity (ohm-cm):	--	**643**
Chloride (Cl), ppm:	2,500	**422**
Sulfate (SO4), ppm:	5,341	**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.: LE16213

Selected Chemical Test Results

Plate C-2

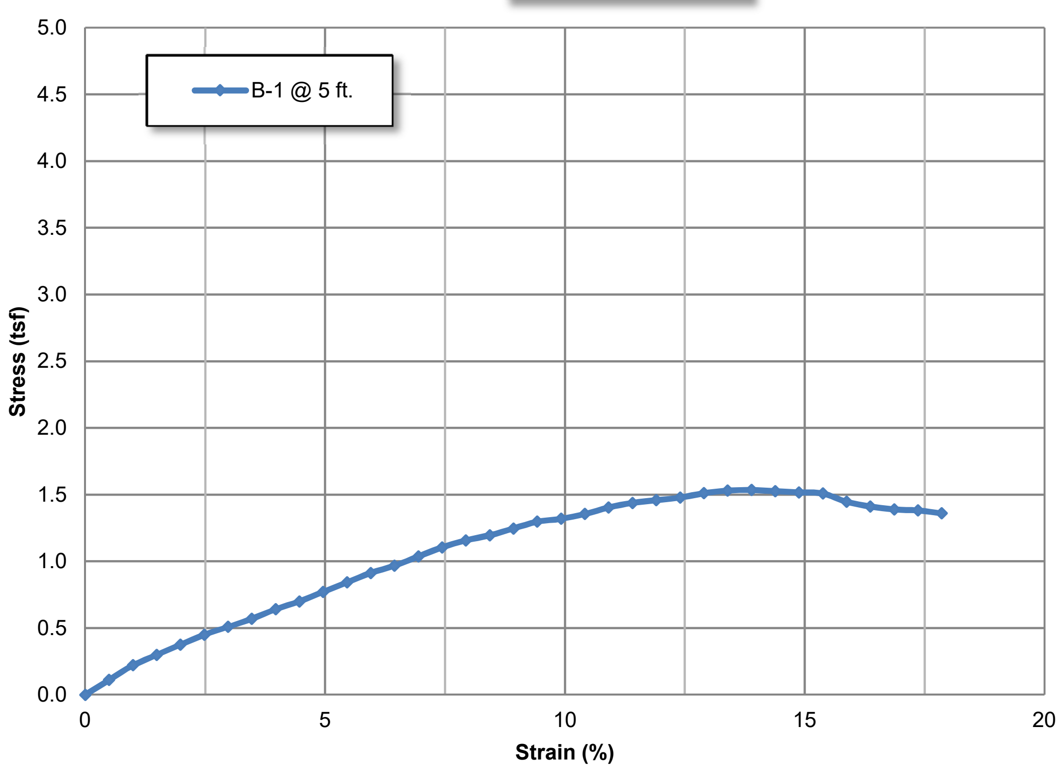
LANDMARK CONSULTANTS, INC.

CLIENT: Sanders, Inc.
PROJECT: ICPW Maintenance Building -- Brawley, CA
JOB NO: LE16213
DATE: 11/17/2016

UNCONFINED COMPRESSION TEST (ASTM D2166)

Boring No.	Sample Depth (ft)	Natural Moisture Content (%)	Unit Dry Weight (pcf)	Maximum Compressive Strength (tsf)	Cohesion (tsf)	Failure Strain (%)
B-1	5	24.8	97.8	1.53	0.77	13.9

Stress - Strain Plot



LANDMARK Geo-Engineers and Geologists

Project No.: LE16213

Unconfined Compression Test Results

Plate C-3

APPENDIX D

Liquefaction Evaluation and Settlement Calculation

Project Name: **ICPW Maintenance Building -- Brawley, CA**
Project No.: **LE16213**
Location: **B-1**

Maximum Credible Earthquake	7	
Design Ground Motion	0.53	g
Total Unit Weight,	110	pcf
Water Unit Weight,	62.4	pcf
Depth to Groundwater	20	ft
Hammer Effenciency	90	
Required Factor of Safety	1.3	

Boring Data						Sampling Corrections							Corrected	Fines	SPT Clean	Cyclical	Cyclical	Factor	Volumetric	Induced
Depth		Blow Counts		Liquefiable	Overburden	Sampler	SPT	Energy	Borehole	Rod	Liner	Overburden	SPT	Content	Sands	Resistance	Stress	of	Strain (%)	Subsidence
(ft)	(m)	SPT	Mod. Cal.	Soil (0 / 1)	Pressure	Diameter	N_m	C_E	C_B	C_R	C_L	C_N	$(N_1)_{60}$	%	$(N_1)_{60CS}$	$CRR_{M7.5}$	CSR	Safety		(inch)
6	1.83		15	0	660	0.67	10	1.50	1.0	0.75	1	1.70	19	95	28	0.345	0.340	Non-Liq.	0.00	0.00
11	3.35		27	0	1210	0.67	18	1.50	1.0	0.80	1	1.32	29	95	39		0.337	Non-Liq.	0.00	0.00
16	4.88	20		0	1760	1	20	1.50	1.0	0.85	1	1.10	28	95	39		0.333	Non-Liq.	0.00	0.00
21	6.40	13		0	2248	1	13	1.50	1.0	0.95	1	0.97	18	95	27	0.311	0.338	1.10	0.00	0.00
26	7.92	8		0	2486	1	8	1.50	1.0	0.95	1	0.92	11	95	18	0.190	0.372	0.61	0.00	0.00
31	9.45	20		0	2724	1	20	1.50	1.0	0.95	1	0.88	25	95	35		0.395	Non-Liq.	0.00	0.00
36	10.97	14		0	2962	1	14	1.50	1.0	1.00	1	0.85	18	95	26	0.305	0.407	0.90	0.00	0.00
41	12.50	12		0	3200	1	12	1.50	1.0	1.00	1	0.81	15	95	23	0.247	0.409	0.72	0.00	0.00
46	14.02	8		0	3438	1	8	1.50	1.0	1.00	1	0.78	9	95	16	0.176	0.402	0.52	0.00	0.00
51	15.54	13		1	3676	1	13	1.50	1.0	1.00	1	0.76	15	40	23	0.250	0.390	0.76	1.40	0.50
	0.00																			
	0.00																			
	0.00																			
	0.00																			

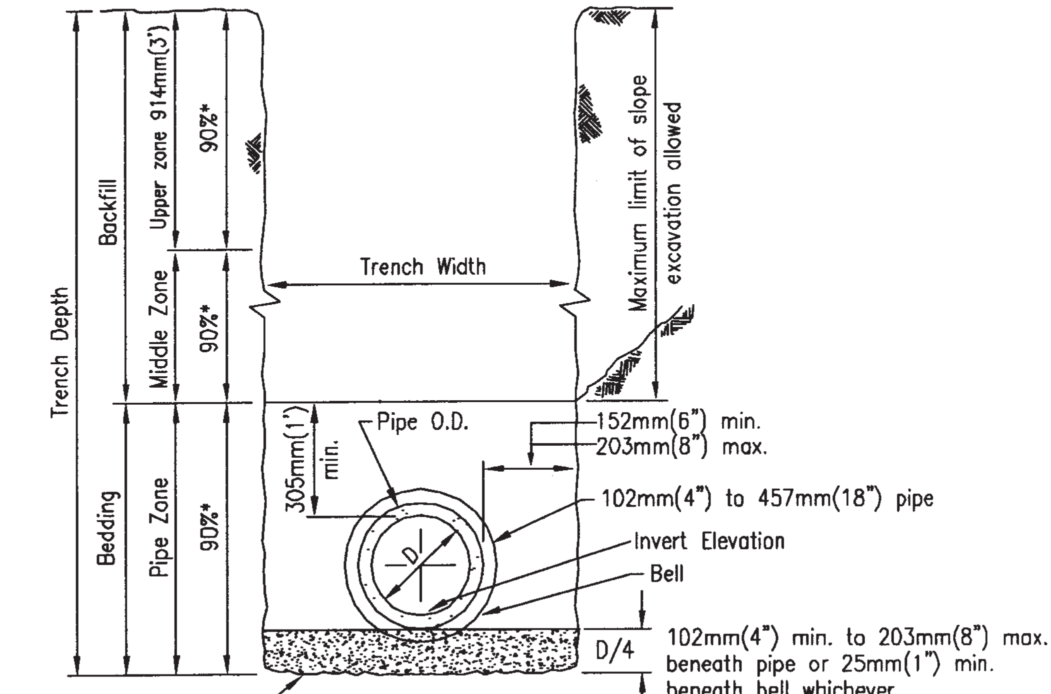
Based on *Proceeding of the NCEER Workshop on Evaluation of Liquefaction Resistance of Soils*, Technical Report NCEER-97-0022, December 31, 1997.

Total Settlement **0.50**

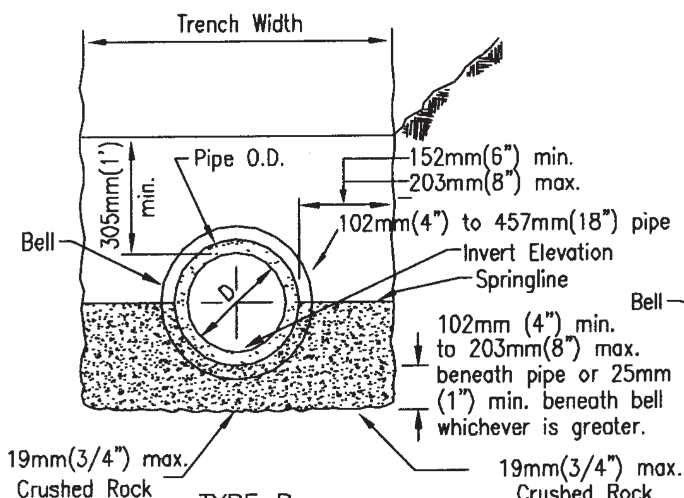
Corrections to SPT (Modified from Skempton, 1986) as listed by Robertson and Wride.

Factor	Equipment Variable	Term	Correction
Overburden Pressure		C_N	$(P_a/\sigma_{VO})^{0.5}$
			C_N<=2
Energy Ratio	Donut Hammer	C_E	0.5 to 1.0
	Safety Hammer		0.7 to 1.2
	Automatic-trip Donut type Hammer		0.8 to 1.3
Borehole Diameter	2.6 inch to 6 inch	C_B	1
	6 inch		1.05
	8 inch		1.15
Rod Length	10 feet to 13 feet	C_R	0.75
	13 feet to 19.8 ft.		0.85
	19.8 ft. to 33 ft.		0.95
	33 ft. to 98 ft.		1
	> 98 ft.		<1.0
Sampling Method	Standard Sampler	C_L	1
	Sampler without liners		1.1 to 1.3

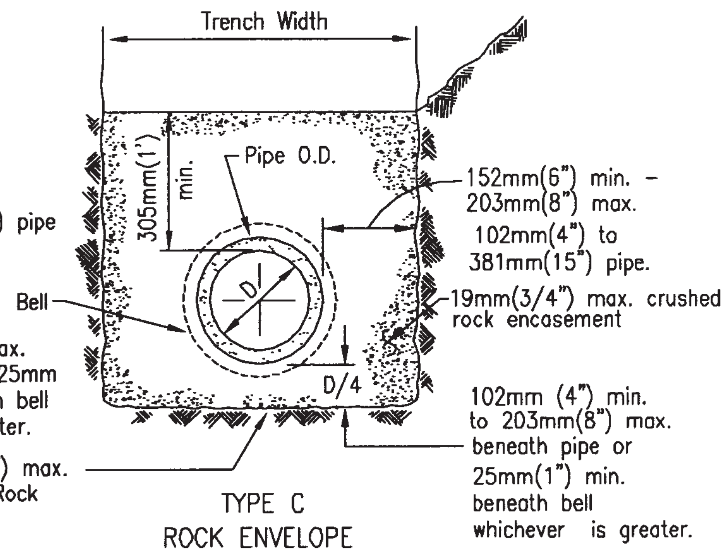
APPENDIX E



TYPE A
STANDARD INSTALLATION



TYPE B
ROCK TO SPRINGLINE



TYPE C
ROCK ENVELOPE

NOTES

1. For trenching in improved streets, see Standard Drawings G-24 or G-25 for trench resurfacing.
2. (*) indicates minimum relative compaction.
3. Minimum depth of cover from the top of pipe to finish grade for all sanitary sewer installations shall be 914mm(3') For cover less tnan 914mm(3'), see Standard Drawing S-7 for concrete encasement.
4. See Type A installation for details not shown for Types B and C.

LANDMARK Geo-Engineers and Geologists **Project No.: LE16213**	**Pipe Bedding and Trench Backfill Recommendations**	**Plate E-1**

APPENDIX F

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Addendum No. 2 – Geotechnical Report
ICPW Maintenance Building
4736 Hwy 111
Brawley, California
LCI Report No. LE16213

Dear Mr. Aguilera:

Landmark prepared a geotechnical report in 2016 for design and construction of the proposed replacement of the existing maintenance building at the Imperial County Public Works (ICPW) Department facility located at 4736 Hwy 111 in northern Brawley, California. It is our understanding that the west side of the building will be extended approximately 20 feet to the west. A small building with a basement will be demolished in order to facilitate the expansion. The basement is planned to be partially demolished. This addendum addresses the backfill of the basement with engineered fill.

The top 4 feet of the concrete basement walls may be demolished and broken into the basement area. The concrete sections should be spread on the floor of the basement to minimize cavities. The interior of the basement should be backfilled with granular soil to a level about 6 inches above any concrete sections. A pogo stick or other means of mechanical compaction should be used to densify the granular soil between sections of concrete in the basement area to at least 90% compaction. Due to the existing soils having a hydrocarbon odor, the existing soils are not to be used in backfilling the basement excavation. Pit run sand or clean silty soils should be used to backfill the basement area. The backfill should be placed in placed in maximum 8-inch lifts (loose), uniformly moisture conditioned to at least 2 percent above optimum moisture content and compacted to a minimum of 90% of the maximum density determined in accordance with ASTM D1557 methods. Compaction testing should be performed on each 1 foot compacted lift.

If you have any questions or comments regarding our findings, please call our office at (760) 370-3000.

Respectfully Submitted,
Landmark Consultants, Inc.

REGISTERED PROFESSIONAL ENGINEER
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