Geotechnical Report

Membrane Bio-Reactor Treatment Facility Imperial Wastewater Treatment Plant Upgrades

Imperial, California

Prepared for:

Albert A. Webb Associates
3788 McCray Street
Imperial, CA 92506

LANDMARK
Geo-Engineers and Geologists

Prepared by:

Landmark Consultants, Inc.
780 N. 4th Street
El Centro, CA 92243
(760) 370-3000

June 2016

LANDMARK
Geo-Engineers and Geologists

780 N. 4th Street
El Centro, CA 92243
(760) 370-3000
(760) 337-8900 fax

77-948 Wildcat Drive
Palm Desert, CA 92211
(760) 360-0665
(760) 360-0521 fax

June 23, 2016

Mr. Brian Knoll, PE
Albert A. Webb Associates
3788 McCray Street
Riverside, CA 92506

Geotechnical Report
MBR Building and Basin
Imperial WWPT
Imperial, California
LCI Report No. LE16096

Dear Mr. Knoll:

This geotechnical report is provided for design and construction of the proposed improvements to the existing Imperial Wastewater Treatment plant located at the northeast corner of 15th Street and North N Street in northeast Imperial, 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:

The findings of this study indicate that the site is, in general, predominantly underlain by stiff to very stiff clay/silty clay (CH-CL) to a depth of 31.5 feet. Interbedded silty sand/sandy silt (SM/ML) and clayey silt (ML) layers of about 3 to 4 feet were encountered at a depth of 4 to 7 and 14 to 21 feet below ground surface.

The risk of liquefaction induced settlement is low (estimated settlement of less than ¾ inches at 14 to 17 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 4.0 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.

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

Respectfully Submitted,
Landmark Consultants, Inc.

PROFESSIONAL GEOLOGIST
STEVEN K. WILLIAMS
CERTIFIED ENGINEERING GEOLOGIST
CEG 2261
STATE OF CALIFORNIA

Steven K. Williams, PG, CEG
Senior Engineering Geologist

REGISTERED PROFESSIONAL ENGINEER
JULIAN R. AVALOS
No. 73339
EXPIRES 12-31-16
CIVIL
STATE OF CALIFORNIA

Julian R. Avalos, PE
Senior Engineer

REGISTERED PROFESSIONAL ENGINEER
JEFFREY O. LYON
No. 31921
EXPIRES 12-31-16
CIVIL
STATE OF CALIFORNIA

Jeffrey O. Lyon, PE
President

Distribution:
Client (4)

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 improvements to the existing Imperial Wastewater Treatment plant located at the northeast corner of 15th Street and North N Street in northeast Imperial, California (See Vicinity Map, Plate A-1). The proposed improvements will consist of the removal of an existing oxidation/aeration basin and the construction of a Membrane Bio-Reactor Treatment facility which will consist of a 16 feet deep concrete MBR, Aeration and Anoxic basins with associated equipment building and UV Disinfection basins. A site plan for the proposed improvements was provided by the client prior to initiation of the field investigation.

Site development will include building pad preparation, basin excavations, 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 upper 30 feet of 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
- Lateral earth pressures
- 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 site for the presence of environmentally hazardous materials or conditions, groundwater mounding, or landscape suitability of the soil.

1.3 Authorization

Mr. Brian Knoll of Webb Associates provided authorization by written agreement to proceed with our work on May 24, 2016. We conducted our work according to our written proposal dated May 11, 2016.

Section 2
METHODS OF INVESTIGATION

2.1 Field Exploration

Subsurface exploration was performed on May 31, 2016 using 2R Drilling of Ontario, California to advance two (2) borings to depths of 31.5 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 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 borings were backfilled with the excavated material. The backfill was loosely placed and was not compacted to the requirements specified for engineered fill.

The subsurface logs are presented on Plates B-1 and B-2 in Appendix B. A key to the log symbols is presented on Plate B-3. 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) – used for soil classification and expansive soil design criteria
- Particle Size Analyses (ASTM D422) – used for soil classification and liquefaction evaluation
- Unit Dry Densities (ASTM D2937) and Moisture Contents (ASTM D2216) – used for insitu soil parameters
- Direct Shear (ASTM D3080) – used for soil strength determination
- 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 logs (Appendix B) and on Plates C-1 through C-4 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 Imperial Wastewater Treatment Plant facility is rectangular in plan view and is located at the northeast corner of 15th Street and North N Street in northeast Imperial, California. The existing south oxidation basin will be removed to allow construction of the proposed MBR facility.

A second oxidation basin is located adjacent to the north side of the proposed MBR facility location Plant. Headwork's, an aeration basin and influent pump stations are located adjacent to the east side of the proposed MBR facility area. The existing operation building, clarifiers, sludge pumping station building and the UV disinfection structure are located at the southeast side of the wastewater plant. Sludge drying beds are located to the north side of the wastewater plant. Existing underground power lines and raw water supply lines cross the wastewater plant in east to west and north to south directions.

Adjacent properties are flat-lying and are approximately at the same elevation with this site. The Imperial Public Works maintenance yard and a 2.0 MG above ground treated water steel storage tank lies to the south side of the site. P Street and the Date Canal are located along the east side of the project site with agricultural land beyond. The Union Pacific Railroad tracks are located along the west side of the project site, with the Imperial Irrigation District Headquarters Yard and IID substation beyond. Robertson's Ready Mix concrete plant and the Morningside Residential subdivision lies to the north side of the project site.

The project site lies at an elevation of approximately 65 feet below mean sea level (MSL) (El. 935 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. 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

Subsurface soils encountered during the field exploration conducted on May 31, 2016 consist of dominantly stiff to very stiff clay/silty clay (CH-CL) to a depth of 31.5 feet. Interbedded silty sand/sandy silt (SM/ML) and clayey silt (ML) layers of about 3 to 4 feet were encountered at a depth of 4 to 7 and 14 to 21 feet below ground surface.

The native surface clays likely exhibit moderate to high swell potential (Expansion Index, EI = 51 to 110) 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).

3.4 Groundwater

Groundwater was encountered in the borings at about 16 to18 feet during the time of exploration, but may rise with time to approximately 10 feet below ground surface at this site. Dewatering should be anticipated for wet well construction and piping installed below 10 feet in depth. 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, 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 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. 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, 2000a) indicates that the nearest mapped Earthquake Fault Zone is the Imperial fault located approximately 2.4 miles northeast 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.

CBC General Ground Motion Parameters: 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, 2014) 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, 2015) 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 0.72g 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 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.** 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 06025C1725C).
- **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 0.72g was used in the analysis with a 10 foot groundwater depth and a threshold factor of safety (FS) of 1.3. Computer printouts of the liquefaction analyses are provided in Appendix E.

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 silt layer at a depth of 14 to 17 feet below ground surface. The likely triggering mechanism for liquefaction appears to be strong groundshaking associated with the rupture of the Superstition Hills and Imperial faults. The analysis is summarized in the table below.

Table 3. Summary of Liquefaction Analysis

Boring Location	Depth To First Liquefiable Zone (ft)	Potential Induced Settlement (in)
B-1	---	Unlikely
B-2	14	0.68

Liquefaction Induced Settlements: ***Based on empirical relationships, total induced settlements are estimated to be less than ¾ 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 less than ¾ inch of liquefaction induced differential settlement at the project site.

Because of the depth of the liquefiable layer, the 14 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.

Liquefaction Induced Ground Failure: Based on research from Ishihara (1985) and Youd and Garris (1995) small ground fissure or sand boil formation is unlikely because of the thickness of the overlying unliquefiable soil. Sand boils are conical piles of sand derived from the upward flow of groundwater caused by excess porewater pressures created during strong ground shaking. Sand boils are not inherently damaging by themselves, but are an indication that liquefaction occurred at depth (Jones, 2003).

Liquefaction induced lateral spreading is not expected to occur at this site due to the planar topography. According to Youd (2005), if the liquefiable layer lies at a depth greater that about twice the height of a free face, lateral spread is not likely to develop. No slopes or free faces occur at this site except for the shallow basins, which depths are substantially above the first liquefiable layer.

Mitigation: Based on an estimate of less than ¾ inch of liquefaction induced settlement, no ground improvement or deep foundation mitigation 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.

Buildings Subgrade Preparation (Shallow Foundations): The existing surface soil within the building pad/foundation areas should be removed to 30 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 (clays) or 2 to 6% above optimum (silts), and recompacted to 85 to 90% (clays) or 87 to 92% (silts) 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. Any loose and organic material from the bottom of the existing oxidation/aeration basin shall be completed removed and not used as a backfill or engineered fill. Imported fill soil (for foundations designed for expansive soil conditions) should have a Plasticity Index less than 25 and sulfates (SO_4) less than 4,000 ppm.

If foundation designs are to be utilized which do not include provisions for expansive soil, an engineered building support pad consisting of 3.5 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.

Below Grade Structures Site Preparation: The MBR, aeration and anoxic basins are planned to be constructed at the location of the existing oxidation/aeration basin and are anticipated to be founded at approximately 16 feet below existing grade. The subsurface silts at the proposed bottom of excavation are saturated; consequently, the subgrade has a high potential for pumping under equipment loads. Therefore, the subgrade for the new MBR, aeration and anoxic basins should be overexcavated 24 inches and replaced with drainage rock (ASTM C33, Size 57 or 467). The bottom of the excavation should be covered with a geotextile filter fabric (Mirafi 180 or better) lapped at sides and ends in accordance with manufacture's installations guidelines. The 2.0 ft thick layer of drainage rock should be end dumped onto the filter fabric and spread evenly by excavators or dozers. Upon completing placement of the drainage rock a small vibratory compactor (walk-behind or equivalent) should be used to densify the crushed rock layer. Following densification of the drainage rock, a second layer of filter fabric should be placed over the drainage rock.

Excavation for the MBR facility (approximately 16 feet depth) will encounter the groundwater table (10 feet bgs). Therefore, seepage and pumping subgrade conditions should be anticipated. An adequately designed dewatering system, such as well points or sumps, will be required to control groundwater seepage and prevent running ground conditions. During construction groundwater should be maintained a minimum of 2 feet below the bottom of the excavation. The responsibility for dewatering and selection of an appropriate system for dewatering is beyond the scope of this report.

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).

<u>Moisture Control and Drainage:</u> If clay soils are used at building pads (without 3.5 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.

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

Structural concrete mat foundations are suitable to support the MBR facility building. The mats shall be founded on a layer of properly prepared and compacted soil as described in Section 4.1. The foundations may be designed using an allowable soil bearing pressure of 1,500 psf for compacted native 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).

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).

Structural mats may be designed for a modulus of subgrade reaction (Ks) of 50 pci when placed on compacted native soil or a subgrade modulus of 250 pci when placed on 24 inches of crushed rock (below grade structures) or a subgrade modulus of 300 pci when placed on 3.5 feet of granular fill (buildings). 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.

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 or crushed rock) 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 or crushed rock) 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.

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 either a uniformly thick structural mats (10 inches or greater) or should be designed in accordance with Chapter 18 of the 2013 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 3.5 feet of granular fill 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 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 3.5 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 18-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 4-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 housekeeping slabs) shall be a minimum of 4 inches thick and should be placed on a minimum of 4 inches of compacted (90%) 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. A 15-mil polypropylene vapor barrier shall be placed over native soils prior to placing sand underlayment. Area slabs with shade structures shall have an 18-inch deep perimeter footing and shall have interior grade beams at 15 feet on center. Planters that trap water between sidewalks and foundations are not allowed.

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.

4.4 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-4). The native soils were found to have severe levels of sulfate ion concentration (3,930 to 3,954 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 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 (5,080 to 9,760 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 4 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 four (4) inches around steel reinforcing or embedded components (anchor bolts, etc.) exposed to native soil or landscape water (to 18 inches above grade). If the 4-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 should not be placed under floor slabs.***

4.5 Excavations

All site excavations should conform to CalOSHA requirements for Type B soil (if site is dewatered or Type C soils for non-dewatered excavations). 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. These temporary deep excavations will require slope inclinations no steeper than 1½(H):1(V) unless trench shoring is used. If excavations are planned below groundwater (10 feet below ground surface), all excavation slopes should be excavated according to OSHA Standards for Type C soils. Dewatering of the excavation site will be required prior to start of excavation (2 ft. below bottom of excavation).

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.

All discussions in this section regarding stable excavation slopes assume minimal equipment vibration and adequate setback of excavated material and construction equipment from the top of the excavation. We recommended that the minimum setback distance be equal to the depth of excavation and at least 10 feet from the crown of the slope. If excavated materials are stockpiled adjacent to the excavation, the weight of the material should be considered as a surcharge load for slope stability.

Excavation for the MBR facility (approximately 16 feet depth) will encounter the groundwater table (10 feet bgs). Therefore, seepage and pumping subgrade conditions should be anticipated. An adequately designed dewatering system, such as well points or sumps, will be required to control groundwater seepage and prevent running ground conditions. The responsibility for dewatering and selection of an appropriate system for dewatering is beyond the scope of this report.

4.6 Lateral Earth Pressures

Earth retaining structures, such as retaining walls, should be designed to resist the soil pressure imposed by the retained soil mass. Walls with granular drained backfill may be designed for an assumed static earth pressure equivalent to that exerted by a fluid weighing 60 pcf (native) and 45 pcf (granular) for unrestrained (active) conditions (able to rotate 0.1% of wall height), and 100 (native) and 60 pcf (granular) for restrained (at-rest) conditions. These values should be verified at the actual wall locations during construction.

When applicable (unbalanced retaining wall greater than 6 feet high) seismic earth pressure on walls may be assumed to exert a uniform pressure distribution of 7.5H psf against the back of the wall. The total seismic load is assumed to act as a point load at 0.6H above the base of the wall. The term H is the height of the backfill against a retaining wall in feet. The recommended value 7.5H was derived from the following formula:

$$P_e = \tfrac{3}{8} (k_h)\gamma H^2$$

where: $k_h = 0.75a_{max}$ (a_{max} is a pseudo-static maximum of 0.20g)
$\gamma = 125$ pcf

which equates to $P_e = 7.0H^2$ (acting as a point load at 0.6H from base of wall)

A pseudo-static a_{max} is typically used in slope stability analysis.

Surcharge loads should be considered if loads are applied within a zone between the face of the wall and a plane projected behind the wall 45 degrees upward from the base of the wall. The increase in lateral earth pressure acting uniformly against the back of the wall should be taken as 50% of the surcharge load within this zone. Areas of the retaining wall subjected to traffic loads should be designed for a uniform surcharge load equivalent to two feet of native soil.

Walls should be provided with backdrains to reduce the potential for the buildup of hydrostatic pressure. The drainage system should consist of a composite HDPE drainage panel or a 2-foot wide zone of free draining crushed rock placed adjacent to the wall and extending 2/3 the height of the wall. The gravel should be completely enclosed in an approved filter fabric to separate the gravel and backfill soil. A perforated pipe should be placed perforations down at the base of the permeable material at least six inches below finished floor elevations. The pipe should be sloped to drain to an appropriate outlet that is protected against erosion. Walls should be properly waterproofed. The project geotechnical engineer should approve any alternative drain system.

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.

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 improvements to the existing Imperial Wastewater Treatment plant located at the northeast corner of 15th Street and North N Street in northeast Imperial, 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.

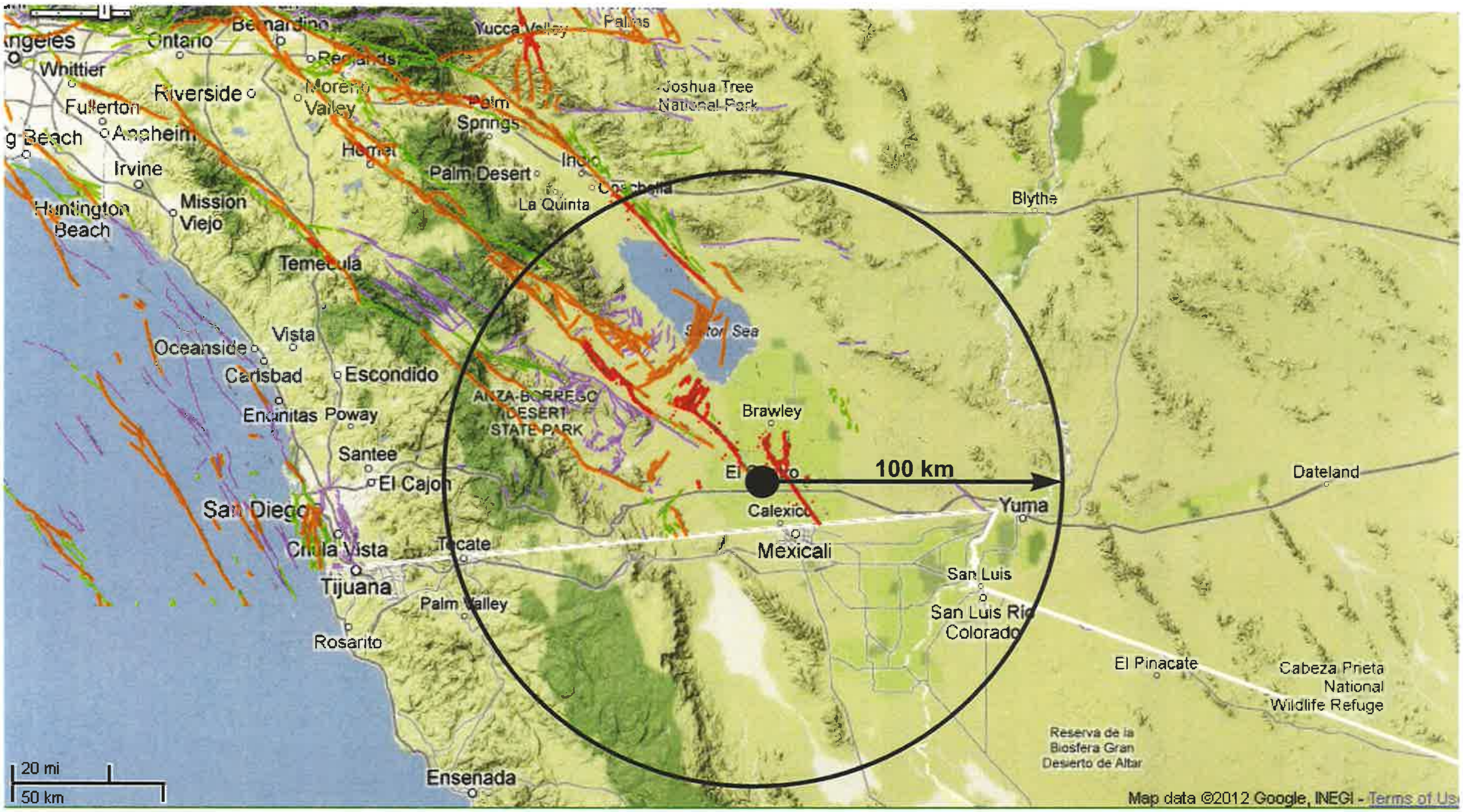
5.2 Additional Services

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.

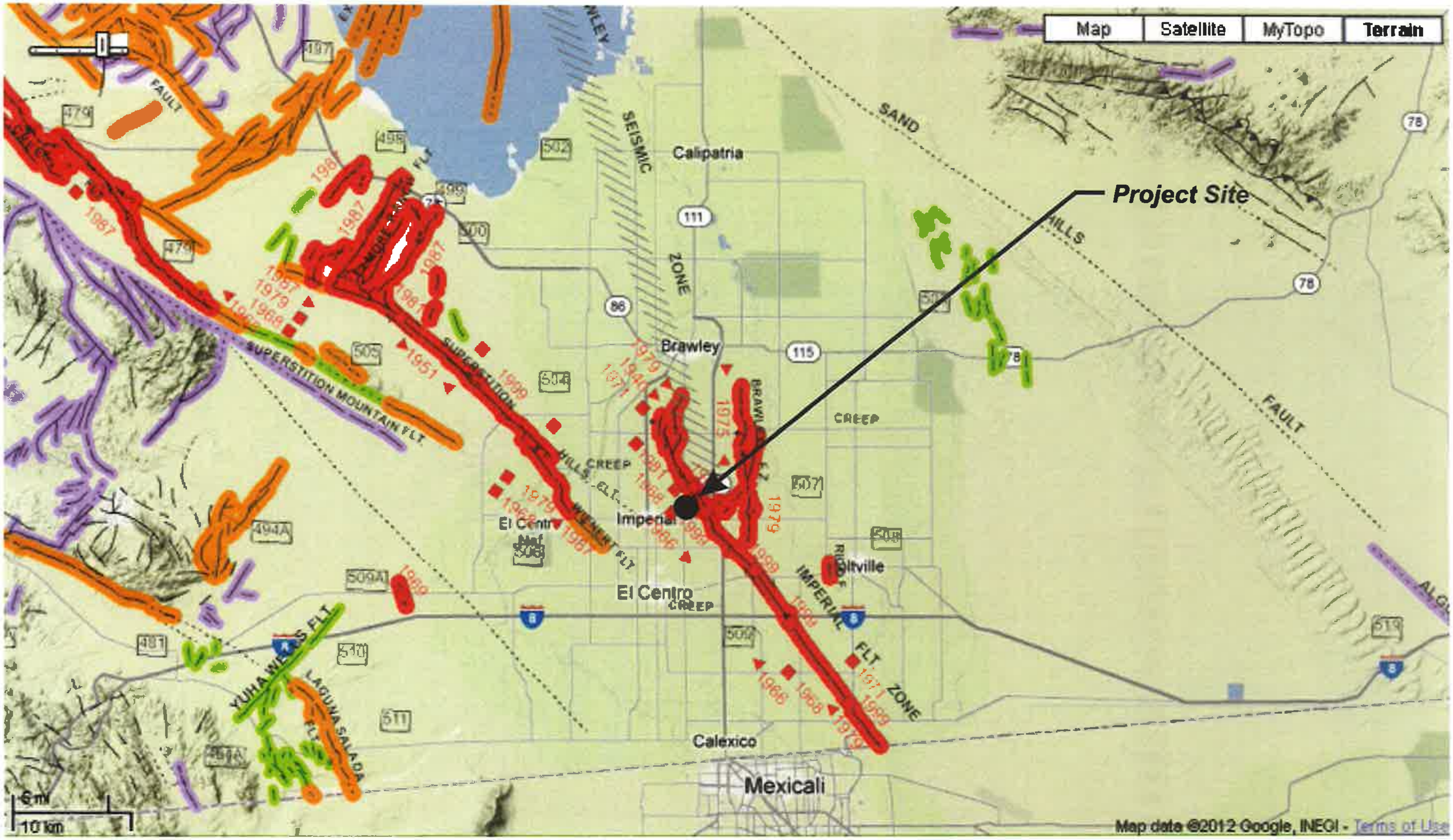


Source: California Geological Survey 2010 Fault Activity Map of California
http://www.quake.ca.gov/gmaps/FAM/faultactivitymap.html#

LANDMARK
Geo-Engineers and Geologists
Project No.: LE16096

Regional Fault Map

Figure 1



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.: LE16096

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 milllon 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.

LANDMARK
Geo-Engineers and Geologists
Project No.: LE16096

Fault Map Legend

Figure 3a

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.

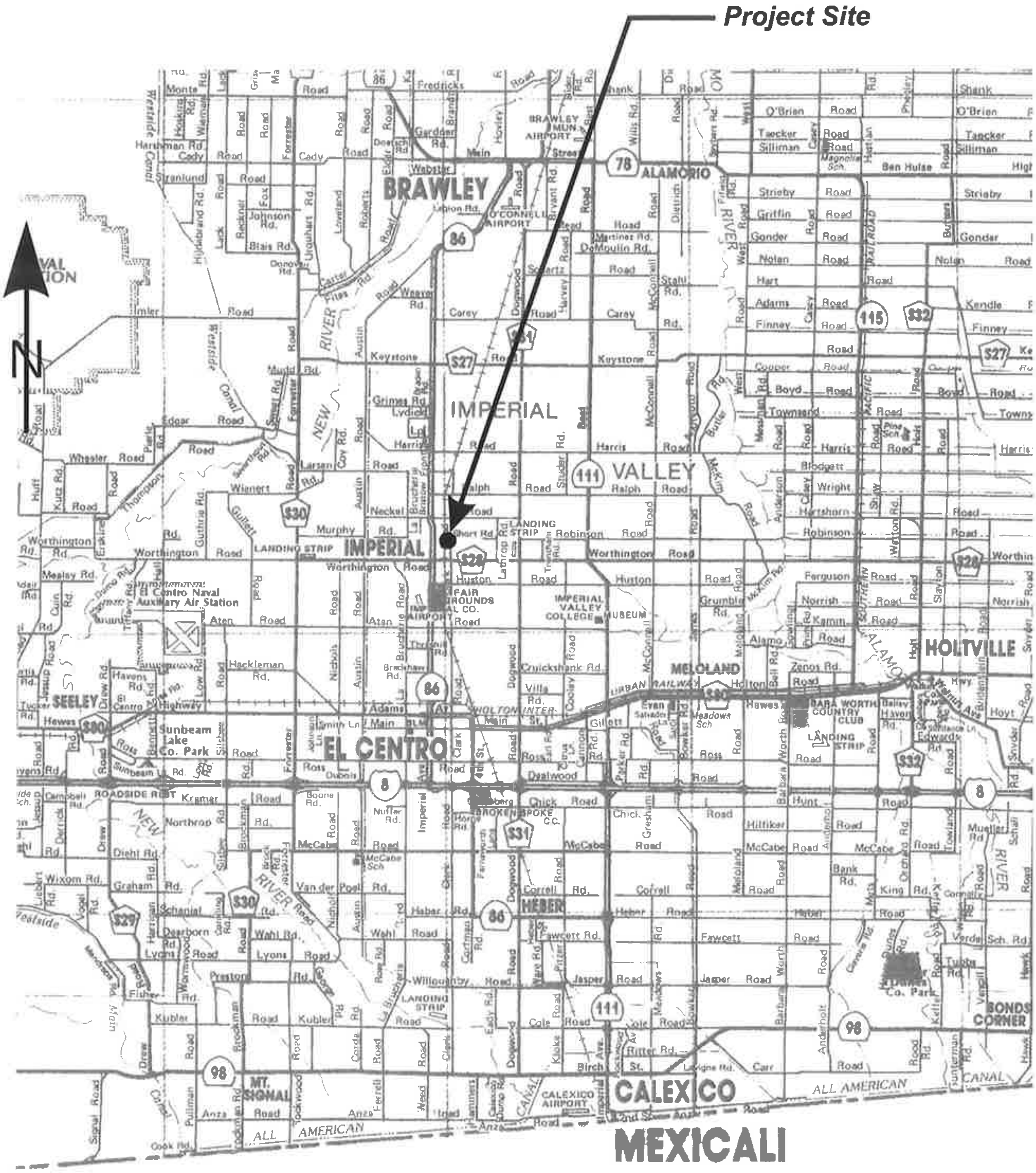
Structural discontinuity (offshore) separating differing Neogene structural domains. May indicate discontinuities between basement rocks.

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				Displacement during historic time (e.g. San Andreas fault 1906). Includes areas of known fault creep.	
			200				
Quaternary	Late Quaternary	Holocene				Displacement during Holocene time.	Fault offsets seafloor sediments or strata of Holocene age.
			11,700				
Quaternary	Late Quaternary	Pleistocene				Faults showing evidence of displacement during late Quaternary time.	Fault cuts strata of Late Pleistocene age.
			700,000				
Quaternary	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.

APPENDIX A

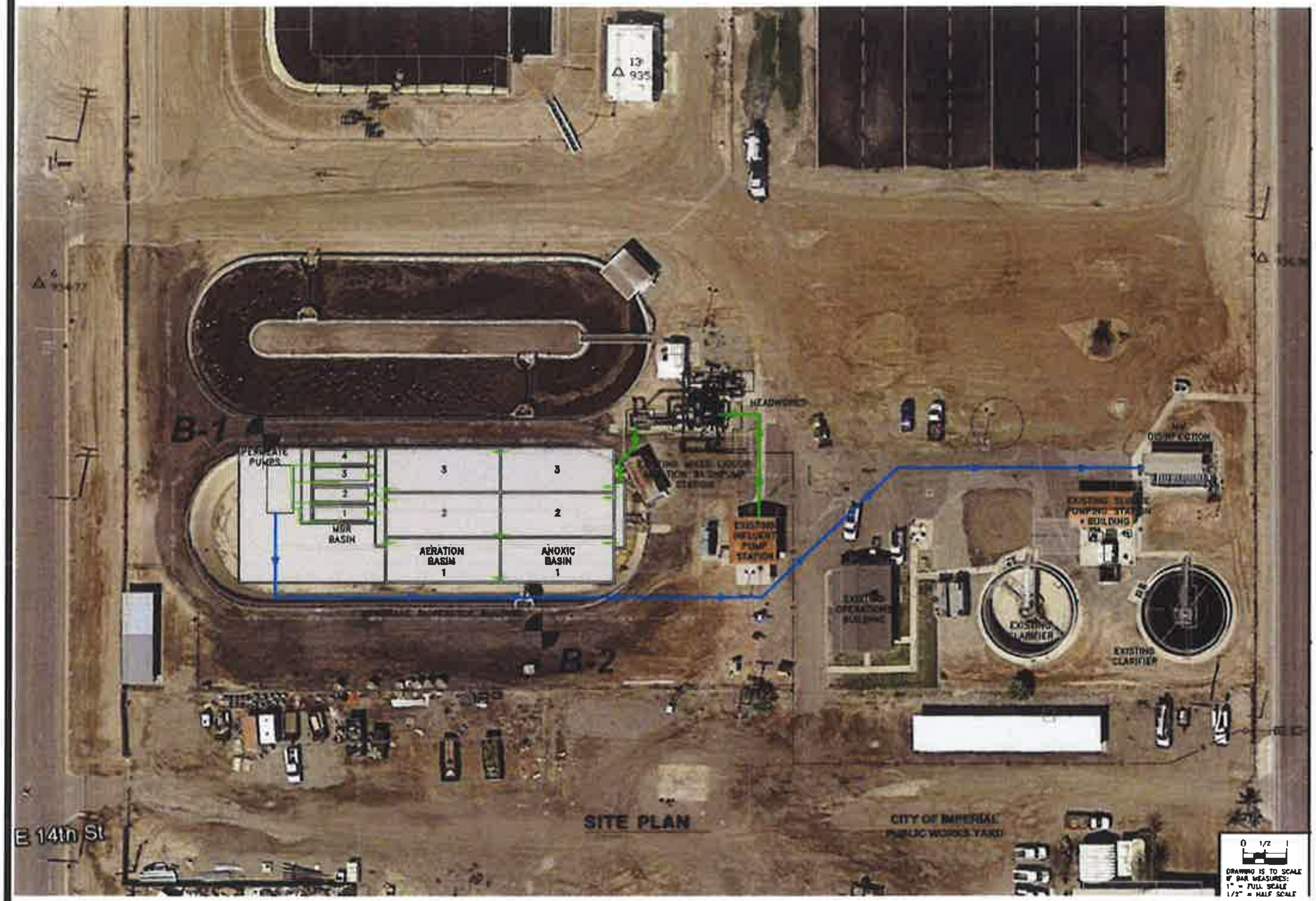


LANDMARK
Geo-Engineers and Geologists
a DBE/MBE/SBE Company

Project No.: LE16096

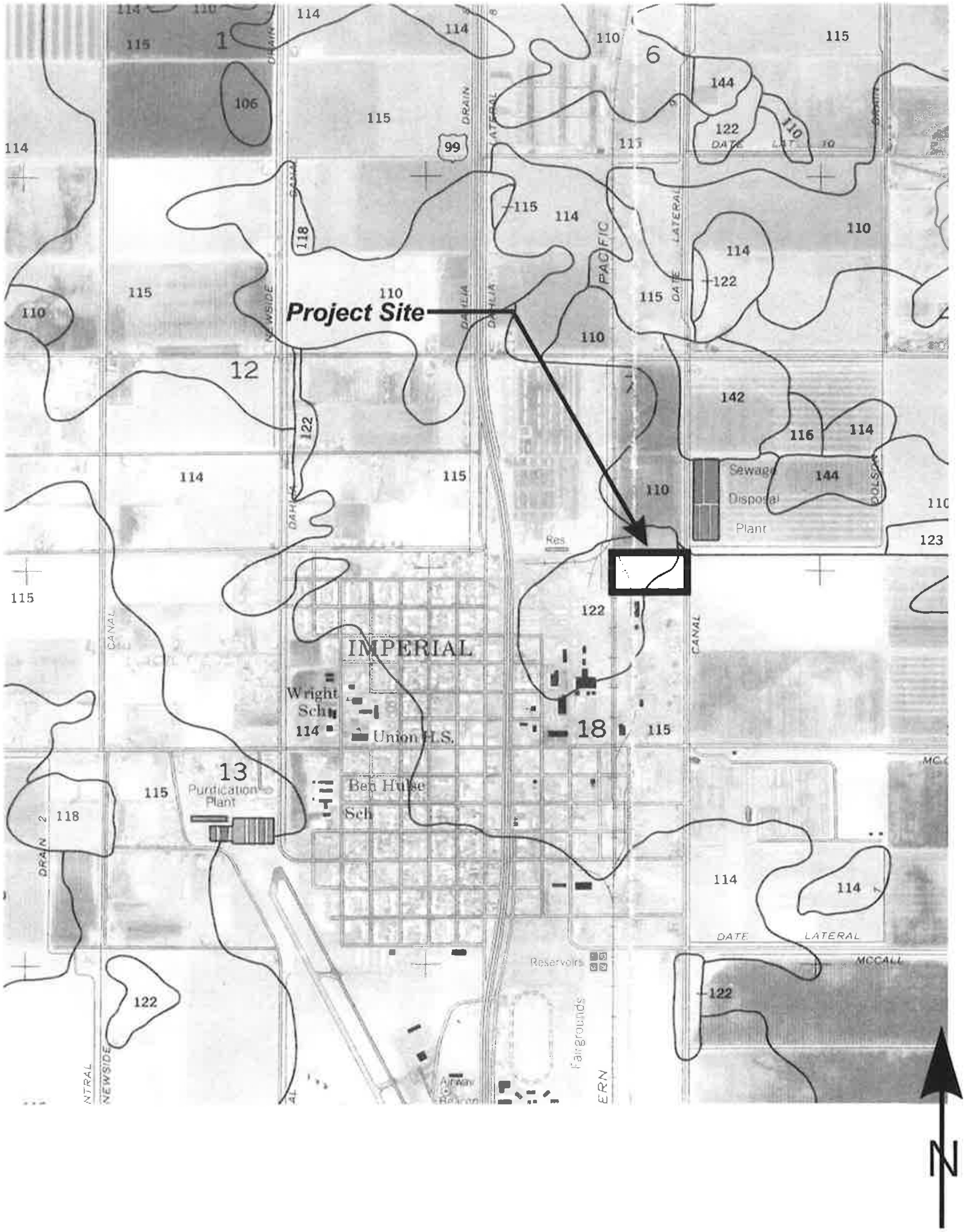
Vicinity Map

Plate A-1



N

LANDMARK Geo-Engineers and Geologists *a DBE/MBE/SBE Company* **Project No.: LE16096**	**Site and Exploration Map**	**Plate A-2**



LANDMARK
Geo-Engineers and Geologists
a DBE/MBE/SBE Company

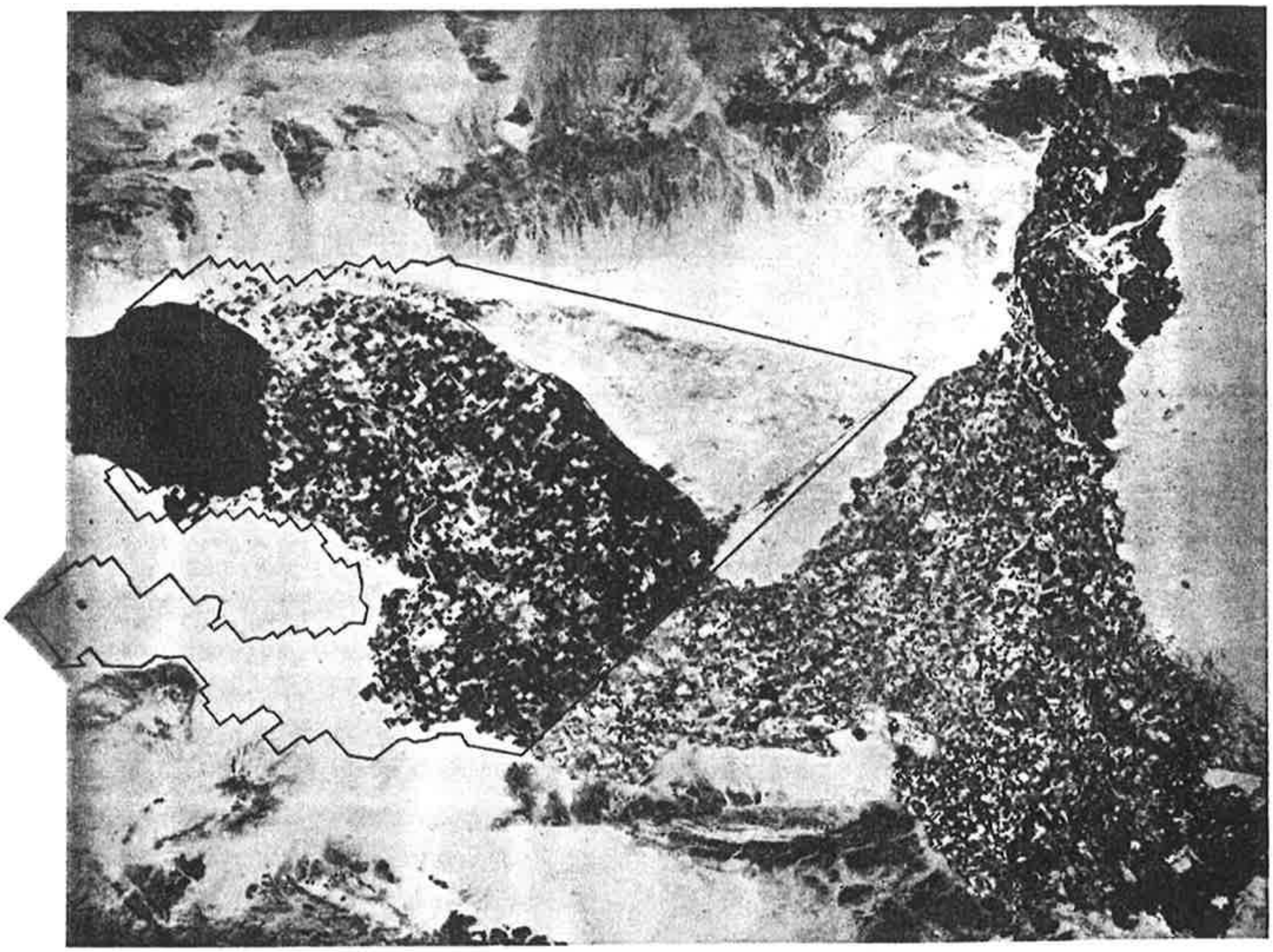
Project No.: LE16096

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		Fragments > 3 inches	Percentage passing sieve number--				Liquid limit	Plasticity index
			Unified	AASHTO		4	10	40	200		
	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		Fragments > 3 inches	Percentage passing sieve number--				Liquid limit	Plasticity index
			Unified	AASHTO		4	10	40	200		
	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

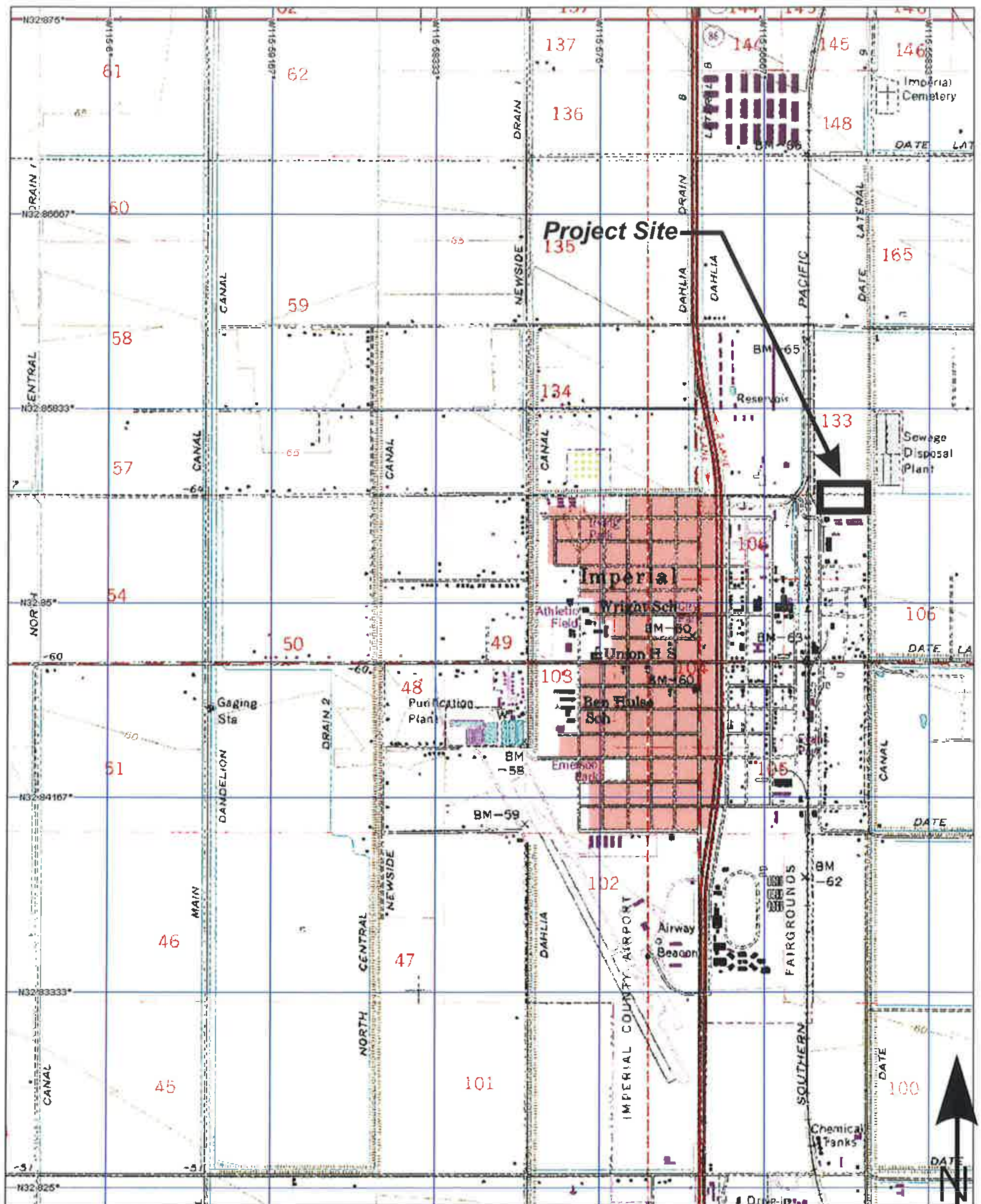
Soil name and map symbol	Depth	USDA texture	Classification		Frag- ments > 3 inches	Percentage passing sieve number--				Liquid limit	Plas- ticity index
			Unified	AASHTO		4	10	40	200		
	In				Pct					Pct	
121---------------- Meloland	0-12	Fine sand-------	SM, SP-SM	A-2, A-3	0	95-100	90-100	75-100	5-30	---	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 Unified	Classification AASHTO	Frag- ments > 3 inches	Percentage passing sieve number-- 4	10	40	200	Liquid limit	Plas- ticity index
	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.



3-D TopoQuads Copyright © 1999 DeLorme Yarmouth, ME 04096 Source Data: USGS

700 ft Scale: 1 : 24,000 Detail: 13-1 Datum: WGS84

LANDMARK
Geo-Engineers and Geologists
a DBE/MBE/SBE Company

Project No.: LE16096

Topographic Map

Plate A-4

APPENDIX B

LOG OF BORING No. B-1

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
					Recycled AC Aggregate (4-in)			
					FAT CLAY (CH): Dark brown, very moist, high plasticity.			LL=54% PI=35%
5			5	0.75	SILTY CLAY (CL): Reddish brown, very moist, medium stiff, medium plasticity.			
10			23	2.0	Very stiff	97.1	27.7	c=0.67 tsf
15			4	0.5	Soft			
20				0.5	CLAYEY SILT (ML): Brown, saturated, very soft, low plasticity.			
			23	2.0	FAT CLAY (CH): Dark brown, very moist, very stiff, high plasticity.	99.5	26.4	
25			13	3.0				
30			27	4.0	Very stiff to hard			
35					Total Depth = 31.5' Groundwater encountered at a depth of 18.0 ft. at time of drilling Backfilled with excavated soil			
40								
45								
50								
55								
60								

DATE DRILLED: 5/31/16
LOGGED BY: J. Avalos
SURFACE ELEVATION: Approximately -65'

TOTAL DEPTH: 31.5 Feet
TYPE OF BIT: Hollow Stem Auger
HAMMER WT.: 140 lbs.

DEPTH TO WATER: 18.0 ft.
DIAMETER: 8 in.
DROP: 30 in.

PROJECT No. LE16096

LANDMARK
Geo-Engineers and Geologists

PLATE B-1

LOG OF BORING No. B-2

DEPTH	SAMPLE	USCS CLASS.	BLOW COUNT	POCKET PEN. (tsf)	DESCRIPTION OF MATERIAL	DRY DENSITY (pcf)	MOISTURE CONTENT (% dry wt.)	OTHER TESTS
					Recycled AC Aggregate (3-in)			
					SILTY CLAY/CLAY (CL-CH): Brown, very moist, medium to high plasticity.			LL=49% PI=33%
5			17		SILTY SAND/SANDY SILT (SM/ML): Tannish brown, very moist, medium dense, with fine grained sand.	99.7	17.3	Passing #200 = 50.2%
10			10	0.5	SILTY CLAY (CL): Brown, very moist, medium stiff to stiff, medium plasticity.			LL=31% PI=10%
15			17	0.5	CLAYEY SILT (ML): Brown, saturated, very soft, low plasticity.	95.3	28.1	ϕ=28° c=0.11 tsf
20			14	2.0	FAT CLAY (CH): Dark brown, very moist, very stiff, high plasticity.			
25			21	4.0		93.8	29.8	c=1.29 tsf
30			20		CLAYEY SILT (ML): Brown, saturated, very stiff, low plasticity.			
					Total Depth = 31.5'			
					Groundwater encountered at a depth of 16.0 ft. at time of drilling			
35					Backfilled with excavated soil			
40								
45								
50								
55								
60								

DATE DRILLED:	5/31/16	TOTAL DEPTH:	31.5 Feet	DEPTH TO WATER:	16.0 ft.
LOGGED BY:	J. Avalos	TYPE OF BIT:	Hollow Stem Auger	DIAMETER:	8 in.
SURFACE ELEVATION:	Approximately -65'	HAMMER WT.:	140 lbs.	DROP:	30 in.

PROJECT No. LE16096

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**		**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. LE16096**	**Key to Logs**	**Plate B-3**

APPENDIX C

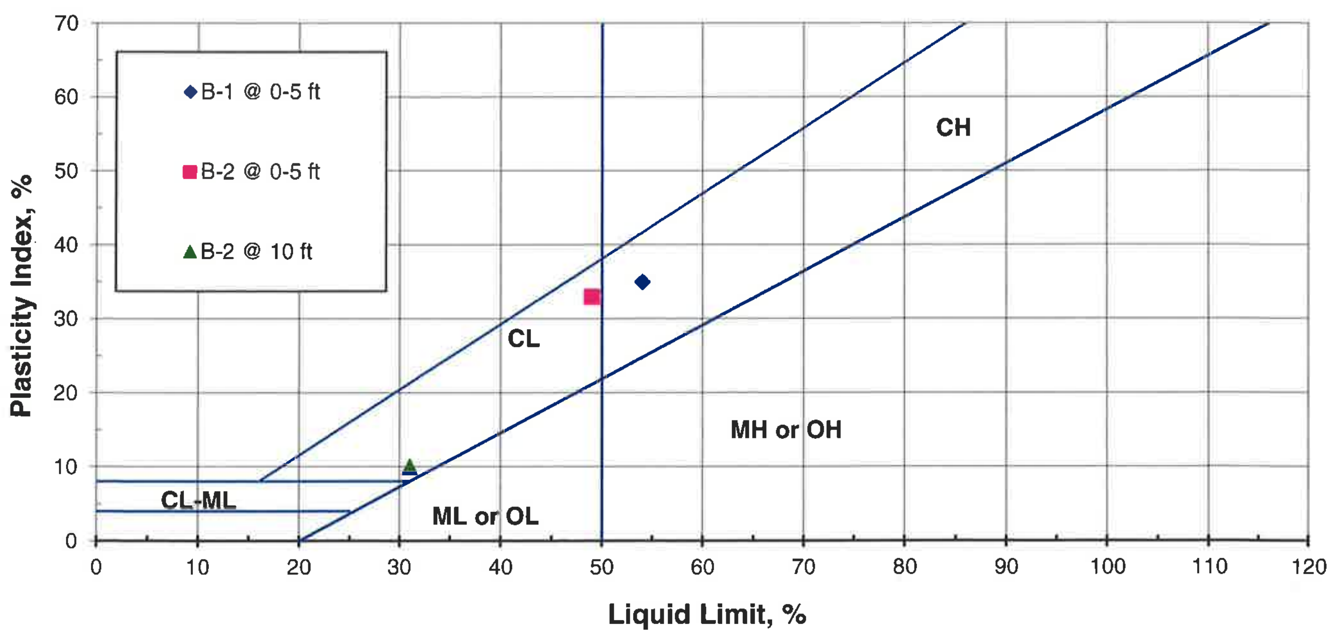
LANDMARK CONSULTANTS, INC.

CLIENT: Webb & Associates
PROJECT: MBR Building and Basin - Imperial WWTP
JOB No.: LE16096
DATE: 06/07/16

ATTERBERG LIMITS (ASTM D4318)

Sample Location	Sample Depth (ft)	Liquid Limit (LL)	Plastic Limit (PL)	Plasticity Index (PI)	USCS Classification
B-1	0-5	54	19	35	CH
B-2	0-5	49	16	33	CL
B-2	10	31	21	10	CL

PLASTICITY CHART



LANDMARK Geo-Engineers and Geologists

Project No.: LE16096

Atterberg Limits Test Results

Plate C-1

LANDMARK CONSULTANTS, INC.

CLIENT: Webb & Associates
PROJECT: MBR Building and Basin - Imperial WWTP -- Imperial, CA
JOB NO: LE16096
DATE: 6/9/2016

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	10	27.7	97.1	1.34	0.67	5.1
B-2	25	29.8	93.8	2.58	1.29	8.8

LANDMARK Geo-Engineers and Geologists

Project No.: LE16096

Unconfined Compression Test Results

Plate C-2

LANDMARK CONSULTANTS, INC.

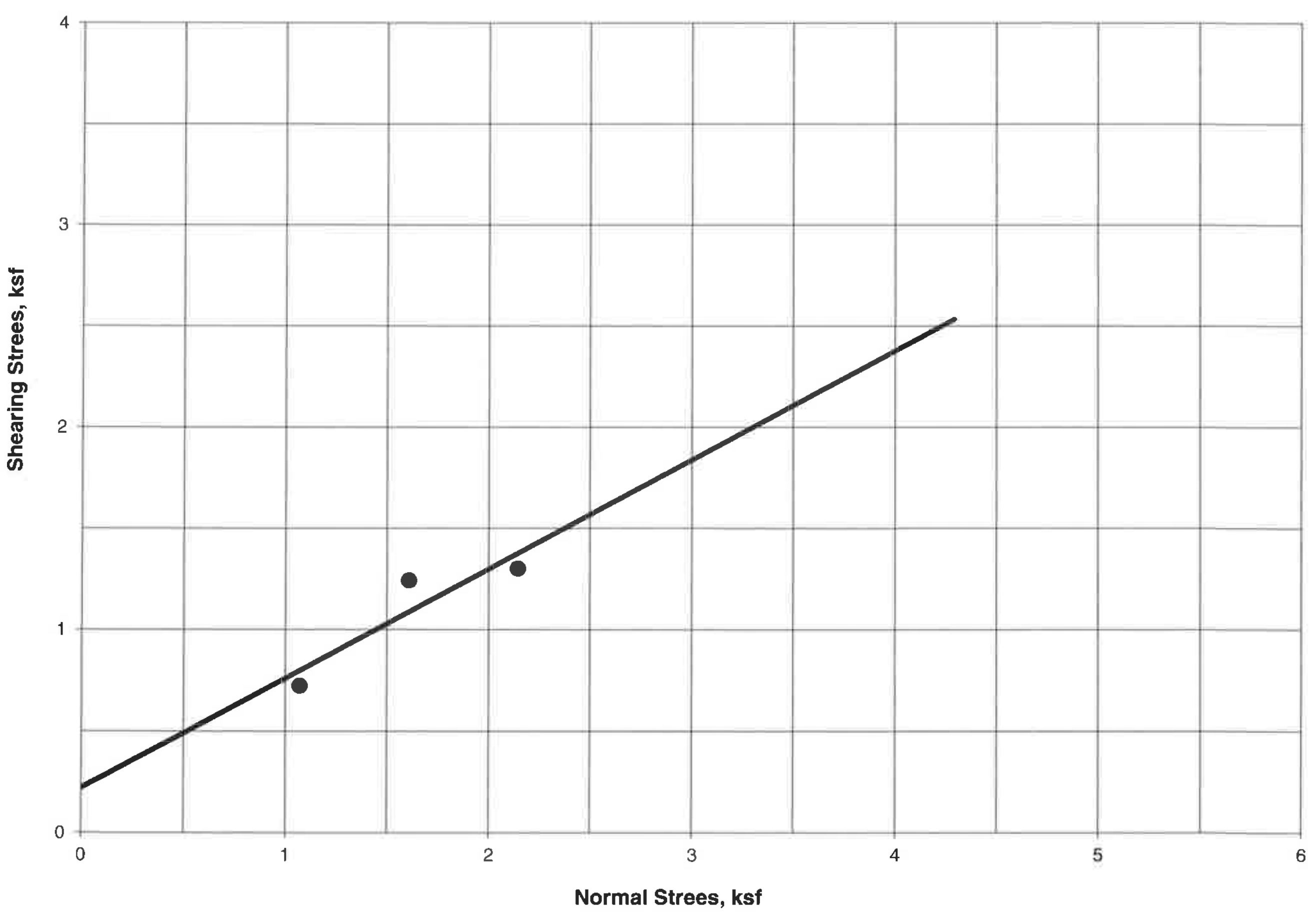
CLIENT: Webb & Associates
PROJECT: MBR Building and Basin - Imperial WWTP
PROJECT No: LE16096 **DATE:** 6/7/2016

DIRECT SHEAR TEST - INSITU (ASTM D3080)

SAMPLE LOCATION: B-2 @ 15 ft
SAMPLE DESCRIPTION: Clayey Silt (ML)

Angle of Internal Friction:	**28°**	**Initial Dry Density:**	**95.3 pcf**
Cohesion:	**0.22 ksf**	**Initial Moisture Content:**	**28.1%**

DIRECT SHEAR TEST RESULTS



LANDMARK
Geo-Engineers and Geologists
PROJECT No: LE16096

Direct Shear Test Results

Plate C-3

LANDMARK CONSULTANTS, INC.

CLIENT: Webb & Associates
PROJECT: MBR Building and Basin - Imperial WWTP
JOB No.: LE16096
DATE: 06/07/16

CHEMICAL ANALYSIS

Boring:	B-1	B-5	**Caltrans**
Sample Depth, ft:	0-5	0-5	**Method**
pH:	7.60	7.31	**643**
Electrical Conductivity (mmhos):	4.73	6.92	**424**
Resistivity (ohm-cm):	100	140	**643**
Chloride (Cl), ppm:	5,080	9,760	**422**
Sulfate (SO4), ppm:	3,930	3,954	**417**

General Guidelines for Soil Corrosivity

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

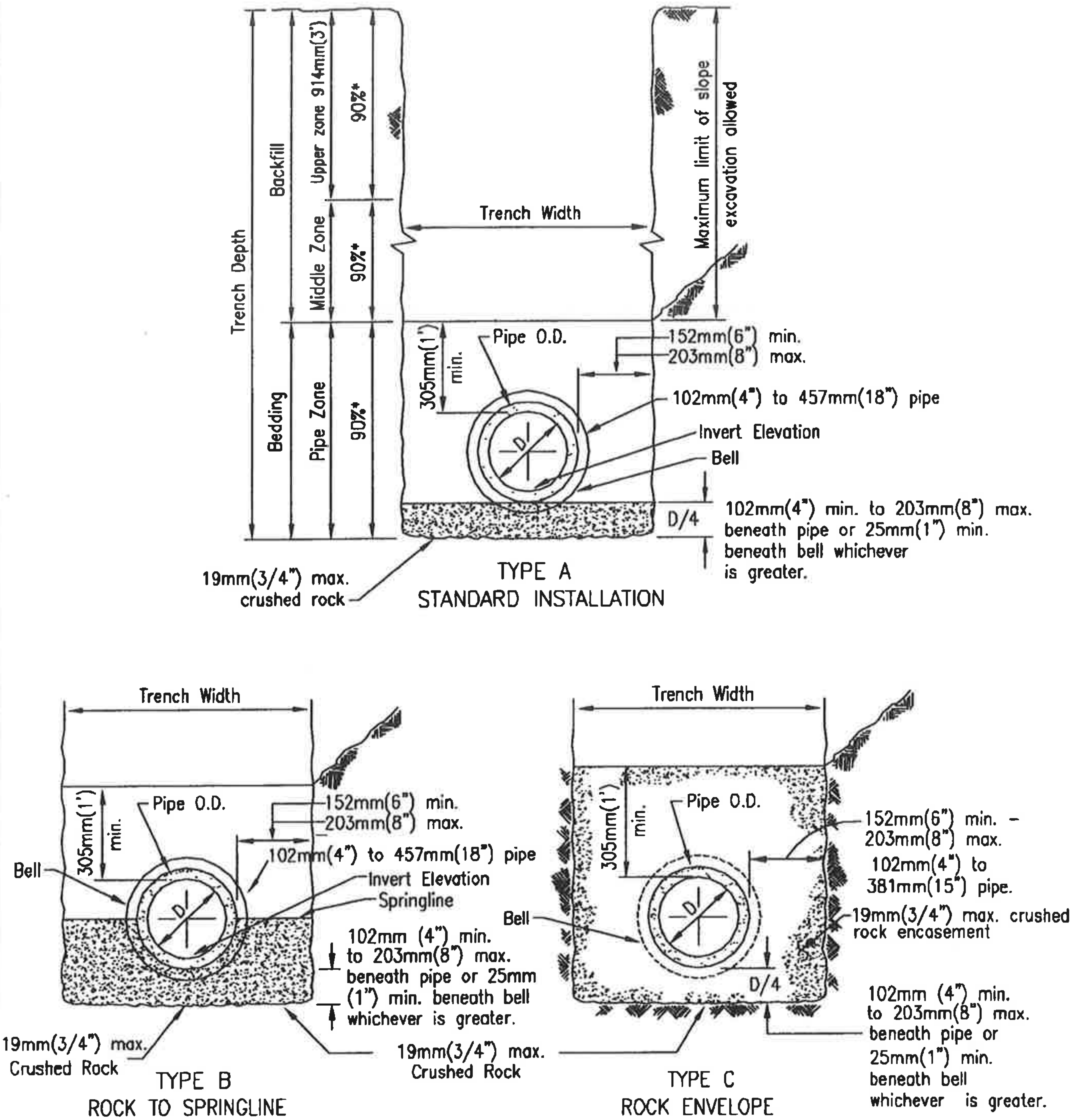
LANDMARK
Geo-Engineers and Geologists

Project No.: LE16096

Selected Chemical Test Results

Plate C-4

APPENDIX D



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.: LE16096	**Pipe Bedding and Trench Backfill Recommendations**	**Plate D-1**

APPENDIX E

Liquefaction Evaluation and Settlement Calculation

Project Name: **MBR Building and Basin - Imperial WWTP**
Project No.: **LE16096**
Location: **B-1**

Maximum Credible Earthquake	7
Design Ground Motion	0.72 g
Total Unit Weight,	115 pcf
Water Unit Weight,	62.4 pcf
Depth to Groundwater	10 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)
5	1.52	5		0	575	1	5	1.50	1.0	0.75	1	1.70	10	95	16	0.178	0.463	Non-Liq.	0.00	0.00
10	3.05		23	0	1150	0.67	15	1.50	1.0	0.80	1	1.36	25	90	35		0.458	Non-Liq.	0.00	0.00
15	4.57	4		0	1413	1	4	1.50	1.0	0.85	1	1.22	6	90	12	0.135	0.553	0.29	0.00	0.00
20	6.10		23	1	1676	0.67	15	1.50	1.0	0.95	1	1.12	25	75	35		0.615	Non-Liq.	0.00	0.00
25	7.62	13		0	1939	1	13	1.50	1.0	0.95	1	1.04	19	95	28	0.349	0.654	0.64	0.00	0.00
30	9.14		27	0	2202	0.67	18	1.50	1.0	0.95	1	0.98	25	95	35		0.675	Non-Liq.	0.00	0.00
	0.00			0	0	0.67	0	1.50	1.0	#N/A	1	#DIV/0!	#N/A	95	#N/A	#N/A	#DIV/0!	#N/A	0.00	
	0.00			0	0	0.67	0	1.50	1.0	#N/A	1	#DIV/0!	#N/A	95	#N/A	#N/A	#DIV/0!	#N/A	0.00	
	0.00			0	0	0.67	0	1.50	1.0	#N/A	1	#DIV/0!	#N/A	95	#N/A	#N/A	#DIV/0!	#N/A	0.00	
	0.00			0	0	0.67	0	1.50	1.0	#N/A	1	#DIV/0!	#N/A	95	#N/A	#N/A	#DIV/0!	#N/A	0.00	
	0.00			0	0	0.67	0	1.50	1.0	#N/A	1	#DIV/0!	#N/A	95	#N/A	#N/A	#DIV/0!	#N/A	0.00	
	0.00			0	0	0.67	0	1.50	1.0	#N/A	1	#DIV/0!	#N/A	95	#N/A	#N/A	#DIV/0!	#N/A	0.00	
	0.00			0	0	0.67	0	1.50	1.0	#N/A	1	#DIV/0!	#N/A	95	#N/A	#N/A	#DIV/0!	#N/A	0.00	
	0.00			0	0	0.67	0	1.50	1.0	#N/A	1	#DIV/0!	#N/A	95	#N/A	#N/A	#DIV/0!	#N/A	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 **0.00**

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 Safety Hammer Automatic-trip Donut type Hammer	C_E	0.5 to 1.0 0.7 to 1.2 0.8 to 1.3
Borehole Diameter	2.6 inch to 6 inch 6 inch 8 inch	C_B	1 1.05 1.15
Rod Length	10 feet to 13 feet 13 feet to 19.8 ft. 19.8 ft. to 33 ft. 33 ft. to 98 ft. > 98 ft.	C_R	0.75 0.85 0.95 1 <1.0
Sampling Method	Standard Sampler Sampler without liners	C_L	1 1.1 to 1.3

Liquefaction Evaluation and Settlement Calculation

Project Name: **MBR Building and Basin - Imperial WWTP**
Project No.: **LE16096**
Location: **B-2**

Maximum Credible Earthquake	7
Design Ground Motion	0.72 g
Total Unit Weight,	115 pcf
Water Unit Weight,	62.4 pcf
Depth to Groundwater	10 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)
5	1.52		17	1	575	0.67	11	1.50	1.0	0.75	1	1.70	22	50	31		0.463	Non-Liq.	0.00	0.00
10	3.05	10		0	1150	1	10	1.50	1.0	0.80	1	1.36	16	90	25	0.275	0.458	0.72	0.00	0.00
15	4.57		17	1	1413	0.67	11	1.50	1.0	0.85	1	1.22	18	75	26	0.306	0.553	0.66	1.14	0.68
20	6.10	14		0	1676	1	14	1.50	1.0	0.95	1	1.12	22	95	32		0.615	Non-Liq.	0.00	0.00
25	7.62		21	0	1939	0.67	14	1.50	1.0	0.95	1	1.04	21	95	30		0.654	Non-Liq.	0.00	0.00
30	9.14	20		1	2202	1	20	1.50	1.0	0.95	1	0.98	28	75	39		0.675	Non-Liq.	0.00	0.00
	0.00			0	0	0.67	0	1.50	1.0	#N/A	1	#DIV/0!	#N/A	30	#N/A	#N/A	#DIV/0!	#N/A	0.00	
	0.00			0	0	0.67	0	1.50	1.0	#N/A	1	#DIV/0!	#N/A	95	#N/A	#N/A	#DIV/0!	#N/A	0.00	
	0.00			0	0	0.67	0	1.50	1.0	#N/A	1	#DIV/0!	#N/A	30	#N/A	#N/A	#DIV/0!	#N/A	0.00	
	0.00			0	0	0.67	0	1.50	1.0	#N/A	1	#DIV/0!	#N/A	85	#N/A	#N/A	#DIV/0!	#N/A	0.00	
	0.00			0	0	0.67	0	1.50	1.0	#N/A	1	#DIV/0!	#N/A	7.8	#N/A	#N/A	#DIV/0!	#N/A	0.00	
	0.00			0	0	0.67	0	1.50	1.0	#N/A	1	#DIV/0!	#N/A	74	#N/A	#N/A	#DIV/0!	#N/A	0.00	
	0.00			0	0	0.67	0	1.50	1.0	#N/A	1	#DIV/0!	#N/A	95	#N/A	#N/A	#DIV/0!	#N/A	0.00	
	0.00			0	0	0.67	0	1.50	1.0	#N/A	1	#DIV/0!	#N/A	95	#N/A	#N/A	#DIV/0!	#N/A	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 **0.68**

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 Safety Hammer Automatic-trip Donut type Hammer	C_E	0.5 to 1.0 0.7 to 1.2 0.8 to 1.3
Borehole Diameter	2.6 inch to 6 inch 6 inch 8 inch	C_B	1 1.05 1.15
Rod Length	10 feet to 13 feet 13 feet to 19.8 ft. 19.8 ft. to 33 ft. 33 ft. to 98 ft. > 98 ft.	C_R	0.75 0.85 0.95 1 <1.0
Sampling Method	Standard Sampler Sampler without liners	C_L	1 1.1 to 1.3

APPENDIX F

REFERENCES

American Society of Civil Engineers (ASCE), 2005, Minimum Design Loads for Buildings and Other Structures: ASCE Standard 7-05.

Arango I., 1996, Magnitude Scaling Factors for Soil Liquefaction Evaluations: ASCE Geotechnical Journal, Vol. 122, No. 11.

Bennett, M. J., McLaughlin, P. V., Sarmiento, J. S., and Youd, T. L., 1984. Geotechnical Investigation of Liquefaction Sites, Imperial Valley, California. U.S. Geological Survey Open-File Report 84-252.

Bray, J. D., Sancio, R. B., Riemer, M. F. and Durgunoglu, T., (2004), Liquefaction Susceptibility of Fine-Grained Soils: Proc. 11th Inter. Conf. in Soil Dynamics and Earthquake Engineering and 3rd Inter. Conf. on Earthquake Geotechnical Engineering., Doolin, Kammerer, Nogami, Seed, and Towhata, Eds., Berkeley, CA, Jan. 7-9, V.1, pp. 655-662.

California Building Standards Commission, 2010, 2010 California Building Code. California Code of Regulations, Title 24, Part 2, Vol. 2 of 2.

California Division of Mines and Geology (CDMG), 1996, California Fault Parameters: available at http://www.consrv.ca.gov/dmg/shezp/fltindex.html.

California Division of Mines and Geology (CDMG), 1962, Geologic Map of California – San Diego-El Centro Sheet: California Division of Mines and Geology, Scale 1:250,000.

California Geological Survey (CGS), 2012, Fault Activity Map of California http://www.quake.ca.gov/gmaps/FAM/faultactivitymap.html#.

California Geological Survey (CGS), 2012, Alquist-Priolo Earthquake Fault Zone Maps. http://www.quake.ca.gov/gmaps/ap/ap_maps.htm

Cao, T., Bryant, W. A., Rowshandel, B., Branum, D., and Wills, C. J., 2003, The revised 2002 California probabilistic seismic hazards maps: California Geological Survey: http://www.conservation.ca.gov/cgs/rghm/psha.

Cetin, K. O., Seed, R. B., Der Kiureghian, A., Tokimatsu, K., Harder, L. F., Jr., Kayen, R. E., and Moss, R. E. S., 2004, Standard penetration test-based probabilistic and deterministic assessment of seismic soil liquefaction potential: ASCE JGGE, Vol., 130, No. 12, p. 1314-1340.

Dibblee, T. W., 1954, Geology of the Imperial Valley region, California, in: Jahns, R. H., ed., Geology of Southern California: California Division of Mines Bull. 170, p. 21-28.

Ellsworth, W. L., 1990, Earthquake History, 1769-1989 in: The San Andreas Fault System, California: U.S. Geological Survey Professional Paper 1515, 283 p.

Ishihara, K. (1985), Stability of natural deposits during earthquakes, Proc. 11th Int. Conf. On Soil Mech. And Found. Engrg., Vol. 1, A. A. Balkema, Rotterdam, The Netherlands, 321-376.

Jennings, C. W., 1994, Fault activity map of California and Adjacent Areas: California Division of Mines and Geology, DMG Geologic Map No. 6.

Jones, A. L., 2003, An Analytical Model and Application for Ground Surface Effects from Liquefaction, PhD. Dissertation, University of Washington, 362 p.

Jones, L. and Hauksson, E., 1994, Review of potential earthquake sources in Southern California: Applied Technology Council, Proceedings of ATC 35-1.

Morton, P. K., 1977, Geology and mineral resources of Imperial County, California: California Division of Mines and Geology, County Report No. 7, 104 p.

Mualchin, L., 1996, A Technical Report to Accompany the Caltrans California Seismic Hazard Map 1996 (Based on Maximum Credible Earthquakes): California Department of Transportation, 65 p.

Mualchin, L. and Jones, A. L., 1992, Peak acceleration from maximum credible earthquakes in California (Rock and Stiff Soil Sites): California Division of Mines and Geology, DMG Open File Report 92-01.

Naeim, F. and Anderson, J. C., 1993, Classification and evaluation of earthquake records for design: Earthquake Engineering Research Institute, NEHRP Report.

National Research Council, Committee of Earthquake Engineering, 1985, Liquefaction of Soils during Earthquakes: National Academy Press, Washington, D.C.

Post-Tensioning Institute (PTI), 2004, Design of Post-Tensioned Slabs-on-Ground. 106 p.

Post-Tensioning Institute (PTI), 2007, Standard Requirements for Design of Shallow Post-Tensioned Concrete Foundations on Expansive Soils. 16 p.

Robertson, P. K. and Wride, C. E., 1996, Cyclic Liquefaction and its Evaluation based on the SPT and CPT, Proceeding of the NCEER Workshop on Evaluation of Liquefaction Resistance of Soils, NCEER Technical Report 97-0022, p. 41-88.

Rymer, M.J., Treiman, J.A., Kendrick, K.J., Lienkaemper, J.J., Weldon, R.J., Bilham, R., Wei, M., Fielding, E.J., Hernandez, J.L., Olson, B.P.E., Irvine, P.J., Knepprath, N., Sickler, R.R., Tong, .X., and Siem, M.E., 2011, Triggered surface slips in southern California associated with the 2010 El Mayor-Cucapah, Baja California, Mexico, earthquake: U.S. Geological Survey Open-File Report 2010-1333 and California Geological Survey Special Report 221, 62 p., available at http://pubs.usgs.gov/of/ 2010/1333/.

Seed, Harry B., Idriss, I. M., and Arango I., 1983, Evaluation of liquefaction potential using field performance data: ASCE Geotechnical Journal, Vol. 109, No. 3.

Seed, Harry B., et al, 1985, Influence of SPT Procedures in Soil Liquefaction Resistance Evaluations: ASCE Geotechnical Journal, Vol. 113, No. 8.

Seed, R. B., Cetin, K. O., Moss, R. E. S., Kammerer, A. M., Wu, J., Pestana, J. M. Riemer, M. F., Sancio, R. B., Bray, J. D., Kayen, R. E., and Faris, A., 2003, Recent advances in soil liquefaction engineering: a unified and consistent framework: University of California, Earthquake Engineering Research Center Report 2003-06, 71 p.

Sharp, R. V., 1982, Tectonic setting of the Imperial Valley region: U.S. Geological Survey Professional Paper 1254, p. 5-14.

Sylvester, A. G., 1979, Earthquake damage in Imperial Valley, California May 18, 1940, as reported by T. A. Clark: Bulletin of the Seismological Society of America, v. 69, no. 2, p. 547-568.

Tokimatsu, K. and Seed H. B., 1987, Evaluation of settlements in sands due to earthquake shaking: ASCE Geotechnical Journal, v. 113, no. 8.

U.S. Geological Survey (USGS), 1982, The Imperial Valley California Earthquake of October 15, 1979: Professional Paper 1254, 451 p.

U.S. Geological Survey (USGS), 1990, The San Andreas Fault System, California, Professional Paper 1515.

U.S. Geological Survey (USGS), 1996, National Seismic Hazard Maps: available at http://gldage.cr.usgs.gov

U.S. Geological Survey (USGS), 2009, Earthquake Ground Motion Parameters, Version 5.0.9a: available at http://earthquake.usgs.gov/research/hazmaps/design/

U.S. Geological Survey (USGS), 2013, US Seismic Design Maps Web Application, available at http://geohazards.usgs.gov/designmaps/us/application.php

Wire Reinforcement Institute (WRI), 2003, Design of Slab-on-Ground Foundations, Tech Facts TF 700-R-03, 23 p.

Youd, T. L., 2005, Liquefaction-induced flow, lateral spread, and ground oscillation, GSA Abstracts with Programs, Vol. 37, No. 7, p. 252.

Youd, T. L. and Garris, C. T., 1995, Liquefaction induced ground surface disruption: ASCE Geotechnical Journal, Vol. 121, No. 11.

Youd, T. L. and Wieczorek, G. F., 1984. Liquefaction During 1981 and Previous Earthquakes Near Westmorland California. U.S. Geological Survey Open-File Report 84-680.

Youd, T. L., Hansen, C. M., and Bartlett, S. F., 1995, Revised Multilinear Regression Equations of Prediction of Lateral Spread Displacement: Journal of Geotechnical and Geoenvironmental Engineering, Vol. 128, No. 12, p. 1007-1017.

Youd, T. L. et. al., 2001, Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils: Journal of Geotechnical and Geoenvironmental Engineering, Vol. 127, No. 10, p. 817-833.

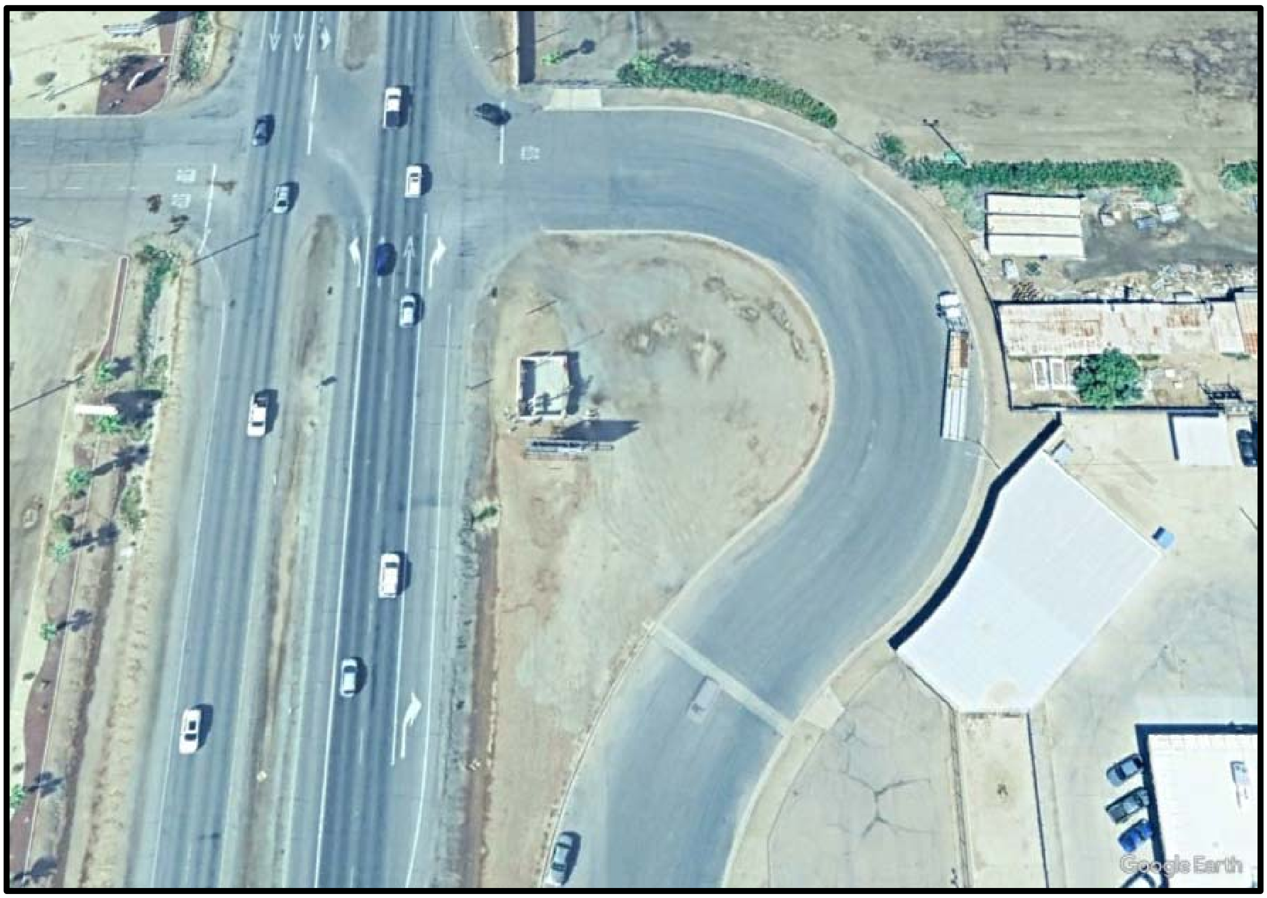
Zimmerman, R. P., 1981, Soil survey of Imperial County, California, Imperial Valley Area: U.S. Dept. of Agriculture Soil Conservation Service, 112 p.

Geotechnical Report

Imperial Sewer Lift Station
SEC Hwy 86 and Claypool Drive
Imperial, California

Prepared for:

Dynamic Consulting Engineers, Inc.
2415 Imperial Business Park Drive, Suite B
Imperial, CA 92243



LANDMARK
Geo-Engineers and Geologists

Prepared by:

Landmark Consultants, Inc.
780 N. 4th Street
El Centro, CA 92243
(760) 337-1100

March 2025

LANDMARK
Geo-Engineers and Geologists

780 N. 4th Street
El Centro, CA 92243
(760) 370-3000
(760) 337-8900 fax

77-948 Wildcat Drive
Palm Desert, CA 92211
(760) 360-0665
(760) 360-0521 fax

March 4, 2025

Mr. Carlos Beltran, PE
Dynamic Consulting Engineers, Inc.
2415 Imperial Business Park Dr., Suite B
Imperial, California

Geotechnical Report
Sewer Lift Station
SEC Hwy 86 and Claypool Drive
Imperial, California
LCI Report No. LE25036

Dear Mr. Beltran:

This geotechnical report is provided for design and construction of the proposed Imperial Sewer Lift Station located at the southeast corner of Hwy 86 and Claypool Drive in Imperial, 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:

- The findings of this study indicate that the subsurface soils consist of surficial fat clays (CH) and silty clays (CL) to a depth of about 11 feet. Medium dense to very dense silty sand (SM) and sandy silt (ML) extend from 11 to 21 feet. A very stiff silty clay (CL) layer was encountered from 21 to 28 feet. Medium dense to very dense silty sand (SM) and sandy silt (ML) soils extend from 28 to 34 feet. Interbedded layers of stiff to very stiff clayey silt/silty clay (ML/CL) and loose sandy silt (ML) were encountered from 34 to 50 feet, the maximum depth of exploration.

- The clay soils are very 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 6.5 sacks Type V cement per cubic yard).

- Liquefaction induced settlement of less than 1-inch at 12 to 32 feet below ground surface is estimated for this site. There is a very low risk of ground rupture should liquefaction occur. Most of the liquefaction occurs in the loose silt/sand layers encountered at depths between 12 to 18 feet. The wet well is designed to be founded at a depth of 20 to 25 feet. Liquefaction settlement is expected to be less than ¼-inch at the base of the lift station. The differential movement between the wet well and adjacent soil is anticipated to be about 0.75 inch. It is not believed that mitigation for potential liquefaction settlement is warranted at this site unless a structure is settlement sensitive. However, piping connections to the lift station should include provisions for differing settlement (about 0.75 inch) between the pipeline and the wet well shaft.

- The excavation for the sewer lift station and sewer main will encounter groundwater (anticipated groundwater at about 8.0 ft below ground surface). Therefore, seepage and pumping subgrade conditions should be anticipated. An adequately designed dewatering system (well points) will be required to control groundwater seepage and prevent running ground conditions. Due to an existing loose clayey/silty layer encountered between 11 to 21 feet depth, the use of a shoring system or large diameter casing should be planned.

- All reinforcing bars, anchor bolts and hold down bolts shall have a minimum concrete cover of 4.0 inches unless epoxy coated (ASTM D3963/A934).

We did not encounter soil conditions that would preclude constructing the lift station at this site 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. 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
PETER E. LABRUCHERIE
No. 3293
GEOTECHNICAL
STATE OF CALIFORNIA

PROFESSIONAL GEOLOGIST
STEVEN K. WILLIAMS
CERTIFIED ENGINEERING GEOLOGIST
CEG 2261
STATE OF CALIFORNIA

Peter E. LaBrucherie, PE, GE
Principal Geotechnical Engineer

Steven K. Williams, PG, CEG
Senior Engineering Geologist

JA

Julian R. Avalos, GE
Senior Geotechnical Engineer

REGISTERED PROFESSIONAL ENGINEER
JULIAN R. AVALOS
No. 3164
GEOTECHNICAL
STATE OF CALIFORNIA

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 Imperial Sewer Lift Station located at the southeast corner of Hwy 86 and Claypool Drive in Imperial, California (See Vicinity Map, Plate A-1). The proposed new sewer lift station will consist of approximately 12-foot diameter reinforced concrete pipe (RCP) or precast manhole, founded approximately 20 to 25 feet below existing grade elevation. The extended slab for the pump station will be supported on shallow spread or continuous footings and the wet well will be supported on a mat foundation. A site plan for the proposed development was provided by the client.

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
- Allowable soil bearing pressures and expected settlements
- Lateral earth pressures
- 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 site for the presence of environmentally hazardous materials or conditions, groundwater mounding, or landscape suitability of the soil.

1.3 Authorization

Carlos Beltran of Dynamic Consulting Engineers, Inc. provided authorization by email to proceed with our work on February 18, 2025. We conducted our work according to our written proposal dated February 18, 2025.

Section 2
METHODS OF INVESTIGATION

2.1 Field Exploration

Subsurface exploration was performed on February 25, 2025 using Kehoe Testing and Engineering, Inc. of Huntington Beach, California to advance one (1) electric cone penetrometer (CPT) sounding to an approximate depth of 50 feet below existing ground surface. The sounding was made at the location shown on the Site and Exploration Plan (Plate A-2). The approximate sounding location was established in the field and plotted on the site map by sighting to discernible site features. A shallow (3-foot deep) hand auger boring (3-inch diameter) was made adjacent to the CPT sounding in order to obtain near surface soil samples for laboratory analysis.

CPT soundings provide a continuous profile of the soil stratigraphy with readings every 2.5cm (1 inch) in depth. Direct sampling for visual and physical confirmation of soil properties has been used by our firm to establish direct correlations with CPT exploration in this geographical region.

The CPT exploration was conducted by hydraulically advancing an instrumented 15cm2 conical probe into the ground at a rate of 2cm per second using a 30-ton truck as a reaction mass. An electronic data acquisition system recorded a nearly continuous log of the resistance of the soil against the cone tip (Qc) and soil friction against the cone sleeve (Fs) as the probe was advanced. Empirical relationships (Robertson and Campanella, 1989) were then applied to the data to give a continuous profile of the soil stratigraphy. Interpretation of CPT data provides correlations for SPT blow count, phi (ϕ) angle (soil friction angle), undrained shear strength (Su) of clays and over-consolidation ratio (OCR). These correlations may then be used to evaluate vertical and lateral soil bearing capacities and consolidation characteristics of the subsurface soil.

An interpretive log of the CPT sounding is presented on Plate B-1 in Appendix B. A key to the interpretation of CPT sounding 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 soil samples obtained from hand auger borings made adjacent to the CPT location to aid in classification and evaluation of selected engineering properties of the near surface 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:

- Moisture-Density Relationship (ASTM D1557)
- Chemical Analyses (soluble sulfates & chlorides, pH, and resistivity) (Caltrans Methods)

The laboratory test results are presented 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 either extrapolated from correlations with the subsurface CPT data or from data obtained from the field and laboratory testing program.

Section 3
DISCUSSION

3.1 Site Conditions

The sewer lift station site is located at the southeast corner of Hwy 86 and Claypool Drive in Imperial, California. The existing lift station is fenced by a perimeter block wall. A wooden power pole is located adjacent to the southwest corner of the wall, with a billboard advertising lying at the south side of the lift station. The site is bounded on the west by State Hwy 86, a divided 4-lane highway and the north and east by Claypool Drive. Adjacent properties are flat-lying and are approximately at the same elevation as this site. The Imperial County Airport is located on the west side of Hwy 86 and the Imperial County Fairgrounds lies to the north side. Commercial/industrial buildings and some vacant lots are located to the south and east sides of the lift station project site.

The project site lies at an elevation of approximately 60 feet below mean sea level (MSL) (El. 940 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 lakebed 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 ºF. 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.

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, 2025) for Google Earth indicates that surficial deposits at the project site consist predominantly of silty clay loams 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 February 25, 2025 consist of surficial fat clays (CH) and silty clays (CL) to a depth of about 11 feet. Medium dense to very dense silty sand (SM) and sandy silt (ML) extend from 11 to 21 feet. A very stiff silty clay (CL) layer was encountered from 21 to 28 feet. Medium dense to very dense silty sand (SM) and sandy silt (ML) soils extend from 28 to 34 feet. Interbedded layers of stiff to very stiff clayey silt/silty clay (ML/CL) and loose sandy silt (ML) were encountered from 34 to 50 feet, the maximum depth of exploration.

The subsurface log (Plate B-1) depicts the stratigraphic relationships of the subsurface soil encountered at the point of exploration. Variations in subsurface stratigraphy may occur at the project site. 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.

3.4 Groundwater

Groundwater was not noted in the CPT sounding, but is typically encountered at approximately 8 to 10 feet below ground surface in the vicinity of the project 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.

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. We have performed a computer-aided search of known faults or seismic zones that lie within a 37-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, 2025b). 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, 2025a) indicates that the nearest zoned fault is the Superstition Hills fault located approximately 2.1 miles west 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.

2022 CBC General Ground Motion Parameters: 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 (ASCE, 2016) for structures on Site Class D 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 (CBC, 2023). **This project site has been classified as Site Class D and has a S_1 value of 0.63, which would require a site-specific ground motion hazard analysis**. However, ASCE 7-16 Section 11.4.8 Supplement 3 provides 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:

- Site Class D sites: A ground motion hazard analysis is not required where the value of the parameter S_{M1} determined by Equation 11.4-2 is increased by 50% for all applications of S_{M1} in ASCE 7-16. The resulting value of the parameter S_{D1} determined by ASCE 7-16 Equation 11.4-4 shall be used for all applications of S_{D1} in ASCE 7-16.

- Site Class E sites: A ground motion hazard analysis is not required:
  a. Where the equivalent lateral force procedure is used for design and the value of C_S is determined by ASCE 7-16 Equation 12.8-2 for all values of T, or
  b. Where (i) the value of S_{ai} is determined by ASCE 7-16 Equation 15.7-7 for all values of T_i and (ii) the value of the parameter S_{D1} is replaced with $1.5S_{D1}$ in ASCE 7-16 Equation 15.7-10 and ASCE 7-16 Equation 15.7-11.

Based on the project site being classified as Site Class D, the structural engineer should increase the parameter S_{M1} provided in Table 2 by 50% for all applications of S_{M1} in ASCE 7-16. If a site-specific ground motion hazard analysis is required for the project, our office should be consulted to perform a site-specific ground motion hazard analysis. **Design earthquake ground motion parameters are provided in Table 2.**

The 2022 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, 2025) 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 2022 CBC Section 1803.5.12.2 is estimated at 0.84g for the project site.

3.7 Seismic and Other Hazards

- **Groundshaking.** The primary seismic hazard at the project site is the potential for strong ground shaking during earthquakes along the Imperial, Brawley, and Superstition Hills faults.
- **Surface Rupture.** The California Geological Survey (2025b) 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 a currently mapped A-P Earthquake Fault Zone; therefore, surface fault rupture is considered to be low at the project site.
- **Liquefaction.** 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). The potential for liquefaction at the site is discussed in more detail in Section 3.8. Liquefaction induced lateral spreading is not expected to occur at this site due to the planar topography.

<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. Obsidian Butte and Red Hill, located at the south end of the Salton Sea approximately 25 miles north of the project site, are small remnants of volcanic domes. The domes erupted about 1,800 to 2,500 years ago (Wright et al, 2015). The subsurface brine fluids around the domes have a high heat flow and are currently being utilized to produce geothermal energy.
- **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 06025C1725C 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 and shallow groundwater, 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 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.

Methods of Analysis: The computer program CLiq (Version 3.5.3.10, Geologismiki, 2024) was utilized for liquefaction assessment at the project site. The estimated settlements have been adjusted for transition zones between layers. Computer printouts of the liquefaction analyses are provided in Appendix D.

The liquefaction potential at the project site was evaluated using the 1998 NCEER Liquefaction Workshop (NCEER, 1998 and Youd, et.al., 2001). The 1997 NCEER methods utilize 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 tip pressures $Q_{tn,cs}$. The analysis was performed using a PGA_M value of 0.98g was used in the analysis with a 10-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. The CPT tip pressures (Qc) were adjusted to an equivalent clean sand pressure ($Q_{tn,cs}$) in accordance with NCEER (1998).

The soils encountered at the points of exploration included saturated silts and silty sands that could liquefy during a Maximum Considered Earthquake. Liquefaction can occur within several isolated silt and sand layers between depths of 12 to 32 feet. 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 less than 1-inch should liquefaction occur.*** Most of the liquefaction occurs in the loose silt/sand layers encountered at depths between 12 to 18 feet. The wet well project is designed to be founded at a depth of 20 to 25 feet, liquefaction settlement is expected to be less than ¼-inch at the base of the station. The differential movement between the wet well and adjacent soil is anticipated to be about 0.75 inch.

Liquefaction Induced Ground Failure: Based on research from Ishihara (1985) and Youd and Garris (1995) small ground fissure or sand boil formation is unlikely because of the thickness of the overlying unliquefiable soil. Sand boils are conical piles of sand derived from the upward flow of groundwater caused by excess porewater pressures created during strong ground shaking. Sand boils are not inherently damaging by themselves, but are an indication that liquefaction occurred at depth (Jones, 2003). Liquefaction induced lateral spreading is not expected to occur at this site due to the planar topography. According to Youd (2005), if the liquefiable layer lies at a depth greater than about twice the height of a free face, lateral spread is not likely to develop. No slopes or free faces occur at this site.

Mitigation: It is not believed that mitigation for potential liquefaction settlement is warranted at this site unless a structure is settlement sensitive. However, piping connections to the lift station should include provisions for differing settlement (about 0.75 inch) between the pipeline and the wet well shaft. Increased slope within the last 20 feet to the wet well shaft and a series of flexible (rubber gasketed) joints near the wet well is suggested.

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.

Wet Well Backfill: Following completion of concrete placement for the wet well foundation, the remaining excavation area against the foundation may be backfilled with native soil in 0.5 foot maximum lifts and compacted to a minimum of 90% of ASTM D1557 maximum dry density at a minimum of optimum moisture. See Section 4.6 for wet well bottom preparation.

Small Equipment Pad Preparation: The exposed surface soil within the small equipment mat foundation areas such as a generator or switchboard should be removed to 12 inches below the bottom of the mat foundations to 2 feet beyond the edges of the foundation. Exposed subgrade should be scarified to a depth of 12 inches, uniformly moisture conditioned to a minimum of 2% to 6% above optimum moisture content, and recompacted to a minimum of 90% of the maximum density determined in accordance with ASTM D1557 methods.

A 12 inch layer of Caltrans Class 2 aggregate base, compacted in maximum 6 inch lifts to at least 95% of ASTM D1557 maximum density at 2% below to 4% above optimum moisture shall be placed over the compacted subgrade prior to placing mat foundations.

Following completion of concrete placement for the mat foundation, the remaining excavation area against the foundation may be backfilled with native soil in 6 inch maximum lifts and compacted to a minimum of 90% of ASTM D1557 maximum dry density at a 2% to 6% above optimum moisture.

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.

4.2 Utility Trench Backfill

Utility Trench Backfill: 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.

Backfill Materials: 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).

4.3 Foundations and Settlements

The lift station may be designed for an allowable soil bearing pressure of 2,500 pounds per square foot (psf) at the base of the station (around 20 feet depth). Footings and equipment foundations which are embedded a minimum of 18 inches into native soil or compacted backfill around the pump wet-well may be designed for an allowable bearing pressure of 1,500 psf. It is suggested that a rigid mat be used for structures placed over wet-well backfill. Horizontal sliding can be resisted with passive earth pressure equivalent to 250 pounds per cubic foot (pcf) of fluid pressure and a coefficient of friction of 0.25. Groundwater buoyant forces and lateral loads should be considered in the wet well design.

<u>Small Equipment Flat Plate Structural Mats</u>: Structural concrete mat foundations may be designed using an allowable soil bearing pressure of 2,000 psf when the foundation is supported on 12 inches of compacted Class 2 aggregate base. The allowable soil pressure may be increased by one-third for short term loads induced by winds or seismic events. The structural mat shall have a double mat of steel and a minimum thickness of 12 inches. Structural mats may be designed for a modulus of subgrade reaction (Ks) of 150 pci when placed on 12 inches of compacted Class 2 aggregate base. An allowable friction coefficient of 0.35 may also be used at the base of the mat to resist lateral sliding.

Resistance to horizontal loads will be developed by passive earth pressure on the sides of footings and frictional resistance developed along the base of footings. Passive resistance to lateral earth pressure may be calculated using an equivalent fluid pressure of 250 pcf to resist lateral loadings. An allowable friction coefficient of 0.35 may also be used at the base of the footings to resist lateral sliding.

4.4 Slabs-On-Grade

<u>Structural Concrete:</u> Structural concrete slabs are those slabs (foundations) that underlie structures or covered housekeeping slabs (shades). Concrete slabs and flatwork shall be a minimum of 5 inches thick due to expansive soil conditions. Concrete slab and flatwork 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 drying shrinkage 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).

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).

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 S2 (severe) levels of sulfate ion concentration (6,132 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, 2019) 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 – Option 1	Over 20,000	V (plus Pozzolon)	0.45	4,500
S3 – Option 2	Over 20,000	V	0.40	5,000

Note: From ACI 318-19 Table 19.3.1.1 and Table 19.3.2.1

A minimum of 6.5 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. 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 (5,200 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.

Foundation designs shall provide a minimum concrete cover of four (4) inches around steel reinforcing or embedded components (anchor bolts, etc.) exposed to native soil or landscape water (to 18 inches above grade). If the 4-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. Additionally, the concrete should be thoroughly vibrated at footings during placement to decrease the permeability of the concrete.

4.6 Excavations for Sewer Lift Station

All site excavations to 4 feet 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. 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. If excavations are planned below groundwater (about 8 feet below ground surface), all excavation slopes should be excavated according to OSHA Standards for Type C soils. All discussions in this section regarding stable excavation slopes assumes minimal equipment vibration and adequate setback of excavated material and construction equipment from the top of the excavation. We recommended that the minimum setback distance be equal to the depth of excavation and at least 10 feet from the crown of the slope. If excavated materials are stockpiled adjacent to the excavation, the weight of the material should be considered as a surcharge load for slope stability.

Due to an existing medium dense sandy/silty layer encountered between 11 to 21 feet depth, the use of a shoring system should be planned. Dewatering of the excavation site will be required prior to start of excavation. Dewatering systems should provide adequate filters so that fine silts are not pumped from depth. Pumping of the fine soils can result in area settlement.

The excavation for the sewer lift station will encounter the groundwater table. Therefore, seepage and pumping subgrade conditions should be anticipated. An adequately designed dewatering system (well points) will be required to control groundwater seepage and prevent running ground conditions. The bottom of pump station should be underlain by a minimum of 18 inches of 1.5-inch crushed rock (ASTM C33, size 467) encapsulated in a geotextile filter fabric.

The responsibility for dewatering and the selection and performance of an appropriate system is the contractor's responsibility. The contractor is cautioned to evaluate soil moisture and groundwater conditions at the time of bidding. This report should be made available to dewatering contractors for theirs initial assessment of the site conditions. However, it is the contractor's own risk to interpret the information contained in this report.

4.7 Lateral Earth Pressures

Earth retaining structures, such as retaining walls, should be designed to resist the soil pressure imposed by the retained soil mass. Walls with granular drained backfill may be designed for an assumed static earth pressure equivalent to that exerted by a fluid weighing 60 (45 sand) pcf for unrestrained (active) conditions (able to rotate 0.1% of wall height), and 100 (60 sand) pcf for restrained (at-rest) conditions. These values should be verified at the actual wall locations during construction.

When applicable (Seismic Design Category D, E or F), retaining wall structures where the backfill is greater than 6 feet high shall be designed in addition to the static loading (active or at-rest condition) with an additional seismic lateral pressure increasing linearly with depth and the resultant acting as a point load at 0.4H above the base of the wall. The term H is the height of the backfill against a retaining wall in feet. The seismic load increment, shall be determined using the following equations for different wall type and backfill conditions:

Basement (restrained) walls with level backfill: $\Delta K_{ae} = \frac{1}{2}\gamma H^2(0.68\ PGA_M/g)$

Cantilever (unrestrained) wall with level backfill: $\Delta K_{ae} = \frac{1}{2}\gamma H^2(0.42\ PGA_M/g)$

Cantilever (unrestrained) wall with sloping backfill*: $\Delta K_{ae} = \frac{1}{2}\gamma H^2(0.70\ PGA_M/g)$

*Applicable for sloping backfill that is no steeper than 2:1 (horizontal:vertical).

Where:

ΔK_{ae} = Seismic Lateral Force (plf) based on seismic pressure
γ = 125 pcf
H = Height of retained soil (ft)
g = A PGA_M value of 0.84g has been determined for the project site.

Surcharge loads should be considered if loads are applied within a zone between the face of the wall and a plane projected behind the wall 45 degrees upward from the base of the wall. The increase in lateral earth pressure acting uniformly against the back of the wall should be taken as 50% of the surcharge load within this zone. Areas of the retaining wall subjected to traffic loads should be designed for a uniform surcharge load equivalent to two feet of native soil.

Walls should be provided with backdrains to reduce the potential for the buildup of hydrostatic pressure. The drainage system should consist of a composite HDPE drainage panel, or a 2-foot-wide zone of free draining crushed rock placed adjacent to the wall and extending 2/3 the height of the wall. The gravel should be completely enclosed in an approved filter fabric to separate the gravel and backfill soil. A perforated pipe should be placed perforations down at the base of the permeable material at least six inches below finished floor elevations. The pipe should be sloped to drain to an appropriate outlet that is protected against erosion. Walls should be properly waterproofed. The project geotechnical engineer should approve any alternative drain system.

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 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.

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 Imperial Sewer Lift Station located at the southeast corner of Hwy 86 and Claypool Drive in Imperial, California. The conclusions and professional opinions of this report are invalid if:

- The sewer lift structure is 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 extent 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 the 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 in 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.'s 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 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
REFERENCES

American Concrete Institute (ACI), 2015, ACI Manual of Concrete Practice 302.1R-15.

American Concrete Institute (ACI), 2019, ACI Manual of Concrete Practice 318-19.

American Society of Civil Engineers (ASCE), 2016, Minimum Design Loads for Buildings and Other Structures: ASCE Standard 7-16.

Boulanger, R. W., and Idriss, I. M., 2014, CPT and SPT Based Liquefaction Triggering Procedures, Report UCD/CGM-14/01, Department of Civil and Environmental Engineering, University of California, Davis, CA, 138 p.

California Building Standards Commission, 2023, 2022 California Building Code. California Code of Regulations, Title 24, Part 2, Vol. 2 of 2.

Caltrans, 2020, Highway Design Manual.

Caltrans, 2020, Caltrans Geotechnical Manual.

California Geological Survey (CGS), 2008, Guidelines for Evaluating and Mitigating Seismic Hazards in California, Special Publication 117A, 98p.

California Geological Survey (CGS), 2025a, Fault Activity Map of California https://maps.conservation.ca.gov/cgs/fam/.

California Geological Survey (CGS), 2025b, Alquist-Priolo Earthquake Fault Zone Maps. http://maps.conservation.ca.gov/cgs/informationwarehouse/index.html?map=regulatorymaps

Geologismiki, 2025, CLiq Computer Program v. 3.5.3.10, www.geologismiki.gr

Federal Emergency Management Agency (FEMA), 2008, Flood Insurance Rate Map (FIRM), Imperial County, California and Incorporated Areas. Dated September 26, 2008.

Idriss, I. M. and Boulanger, R. W., 2008, Soil liquefaction during earthquakes. Monograph MNO-12. Earthquake Engineering Research Institute, Oakland, CA. 261 p.

Idriss, I. M. and Boulanger, R. W., 2010, SPT-base liquefaction triggering procedures. Report UCD/CGM-10-02. Dept. of Civil and Environmental Engineering, University of California – Davis, CA. 259 p.

Ishihara, K. (1985), Stability of natural deposits during earthquakes, Proc. 11th Int. Conf. On Soil Mech. And Found. Engrg., Vol. 1, A. A. Balkema, Rotterdam, The Netherlands, 321-376.

Jones, A. L., 2003, An Analytical Model and Application for Ground Surface Effects from Liquefaction, PhD. Dissertation, University of Washington, 362 p.

Loeltz, O. J., Irelan, B., Robison, J. H., and Olmsted, F. H., 1975, Geohydrologic Reconnaissance of the Imperial Valley, California. USGS Professional Paper 486-K.

Martin, G. R. and Lew, M., 1999, Recommended Procedures for Implementation of DMG Special Publication 117 Guidelines for Analyzing and Mitigating Liquefaction Hazards in California. 63 p.

McCrink, T. P., Pridmore, C. L., Tinsley, J. C., Sickler, R. R., Brandenberg, S. J., and Stewart, J. P., 2011, Liquefaction and Other Ground Failures in Imperial County, California, from the April 4, 2010, El Mayor—Cucapah Earthquake, CGS Special Report 220, USGS Open File Report 2011-1071, 84 p.

Morton, P. K., 1977, Geology and mineral resources of Imperial County, California: California Division of Mines and Geology, County Report No. 7, 104 p.

National Center for Earthquake Engineering Research (NCEER), 1997, Proceedings of the NCEER Workshop on Liquefaction Resistance of Soils. Salt Lake City, Utah, NCEER Technical Report NCEER-97-0022.

Norris and Webb, 1990, Geology of California, 2nd Edition, John Wiley and Sons.

Post-Tensioning Institute (PTI), 2007a, Standard Requirements for Analysis of Shallow Concrete Foundations on Expansive Soils (3rd Edition).

Post-Tensioning Institute (PTI), 2007b, Standard Requirements for Design of Shallow Post-Tensioned Concrete Foundations on Expansive Soils (2nd Edition).

Robertson, P. K. 2009. “Interpretation of cone penetration tests—A unified approach.” Can. Geotech. J. 46 (11): 1337–1355.

Robertson, P. K., 2014, Seismic liquefaction CPT-based methods: EERI 1st Workshop on Geotechnical Earthquake Engineering – Liquefaction Evaluation, Mapping, Simulation and Mitigation. UC San Diego Campus, 10/12/2014.

Robertson, P. K. and Shao, L. 2010. Estimation of Seismic Compression in Dry Soils Using the CPT, 5th International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics, San Diego, CA.

Robertson, P. K. and Wride, C. E., 1997, Cyclic Liquefaction and its Evaluation based on the SPT and CPT, Proceeding of the NCEER Workshop on Evaluation of Liquefaction Resistance of Soils, NCEER Technical Report 97-0022, p. 41-88.

Rymer, M.J., Treiman, J.A., Kendrick, K.J., Lienkaemper, J.J., Weldon, R.J., Bilham, R., Wei, M., Fielding, E.J., Hernandez, J.L., Olson, B.P.E., Irvine, P.J., Knepprath, N., Sickler, R.R., Tong, .X., and Siem, M.E., 2011, Triggered surface slips in southern California associated with the 2010 El Mayor-Cucapah, Baja California, Mexico, earthquake: U.S. Geological Survey Open-File Report 2010-1333 and California Geological Survey Special Report 221, 62 p., available at http://pubs.usgs.gov/of/2010/1333/

Structural Engineers Association of California (SEAOC), 2025, Seismic Design Maps Web Application, available at https://seismicmaps.org/

Tokimatsu, K., and Seed, H. B., 1987, "Evaluation of settlements in sands due to earthquake shaking," J. Geotechnical Eng., ASCE 113(GT8), 861–78.

UC Davis, 2025. California Soil Resource Lab SoilWeb App for Google Earth. https://casoilresource.lawr.ucdavis.edu/

USDA Natural Resources Conservation Service, 2025, Web Soil Survey Website. https://websoilsurvey.sc.egov.usda.gov/App/HomePage.htm

Wire Reinforcement Institute (WRI/CRSI), 2003, Design of Slab-on-Ground Foundations, Tech Facts TF 700-R-03, 23 p.

Wright, H. M., J. A. Vazquez, D. E. Champion, A. T. Calvert, M. T. Mangan, M. Stelten, K. M. Cooper, C. Herzig, and A. Schriener Jr.,2015, Episodic Holocene eruption of the Salton Buttes rhyolites, California, from paleomagnetic, U-Th, and Ar/Ar dating, Geochem. Geophys. Geosyst., 16, 1198–1210, doi:10.1002/2015GC005714.

Youd, T. L., 2005, Liquefaction-induced flow, lateral spread, and ground oscillation, GSA Abstracts with Programs, Vol. 37, No. 7, p. 252.

Youd, T. L. and Garris, C. T., 1995, Liquefaction induced ground surface disruption: ASCE Geotechnical Journal, Vol. 121, No. 11.

Youd, T. L., et al., 2001, “Liquefaction resistance of soils: Summary report from the 1996 NCEER and 1998 NCEER/NSF workshops on evaluation of liquefaction resistance of soils,” Journal Geotechnical and Geoenvironmental Engineering, Volume 127 No. 10 pp. 817–833.

Zhang, G., Robertson, P.K. and Brachman, R.W.I., (2002), “Estimating Liquefaction Induced Ground Settlements From CPT for Level Ground”, *Canadian Geotechnical Journal,* 39(5): 1168-1180.

Zimmerman, R. P., 1981, Soil survey of Imperial County, California, Imperial Valley Area: U.S. Dept. of Agriculture Soil Conservation Service, 112 p.

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)
Superstition Hills	2.1	3.3	6.6	23 ± 2	4 ± 2
Imperial	3.4	5.5	7	62 ± 6	20 ± 5
Brawley *	5.6	8.9			
Superstition Mountain	7.7	12.3	6.6	24 ± 2	5 ± 3
Rico *	9.9	15.9			
Northern Centinela*	12.2	19.5			
Route 247*	12.2	19.6			
Yuha*	14.2	22.7			
Shell Beds	16.1	25.8			
Yuha Well *	16.3	26.1			
Laguna Salada	19.4	31.0	7	67 ± 7	3.5 ± 1.5
Vista de Anza*	19.6	31.3			
Borrego (Mexico)*	19.8	31.7			
Painted Gorge Wash*	19.8	31.7			
Elmore Ranch	21.1	33.7	6.6	29 ± 3	1 ± 0.5
Cerro Prieto *	23.6	37.8			
Ocotillo*	23.7	37.9			
Pescadores (Mexico)*	25.4	40.6			
Cucapah (Mexico)*	26.7	42.8			
Elsinore - Coyote Mountain	27.0	43.1	6.8	39 ± 4	4 ± 2
San Jacinto - Borrego	27.5	44.0	6.6	29 ± 3	4 ± 2
Algodones *	36.5	58.4			

* Note: Faults not included in CGS database.

Table 2
2022 California Building Code (CBC) and ASCE 7-16 Seismic Parameters

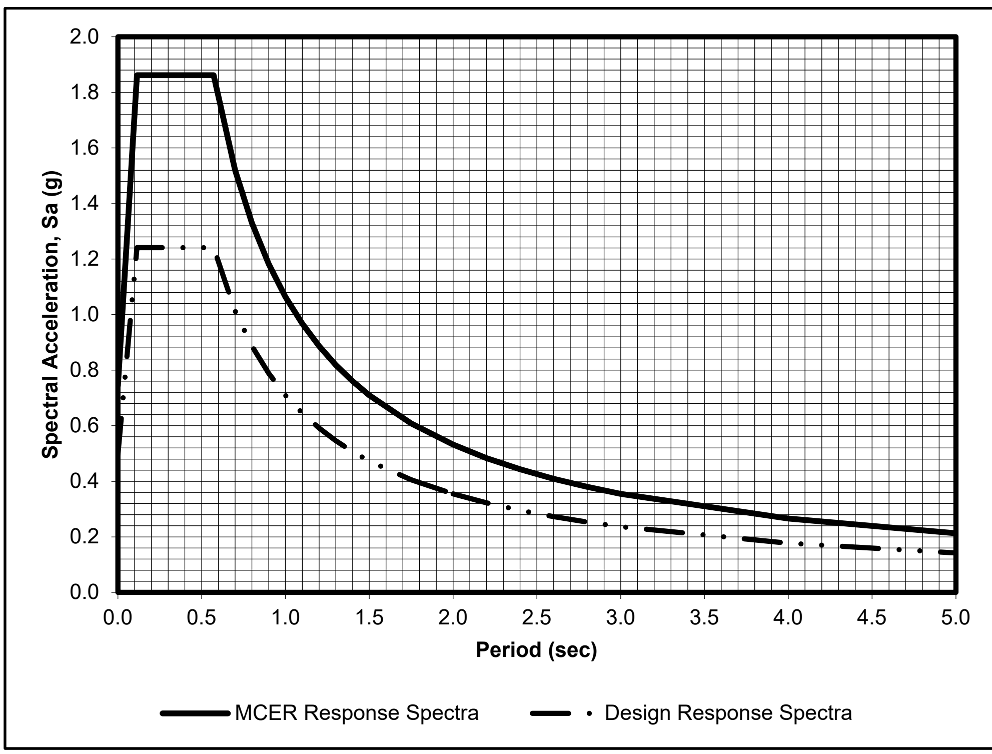
			ASCE 7-16 Reference
Soil Site Class:	**D**		Table 20.3-1
Latitude:	32.8329	N	
Longitude:	-115.5695	W	
Risk Category:	II		
Seismic Design Category:	D		

Maximum Considered Earthquake (MCE) Ground Motion

Mapped MCE_R Short Period Spectral Response	$\mathbf{S_s}$	1.862 g	ASCE Figure 22-1	
Mapped MCE_R 1 second Spectral Response	$\mathbf{S_1}$	0.626 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)	$\mathbf{S_{MS}}$	1.862 g	= Fa * S_s	ASCE Equation 11.4-1
MCE_R Spectral Response Acceleration Parameter (1.0 s)	$\mathbf{S_{M1}}$	1.064 g	= Fv * S_1	ASCE Equation 11.4-2

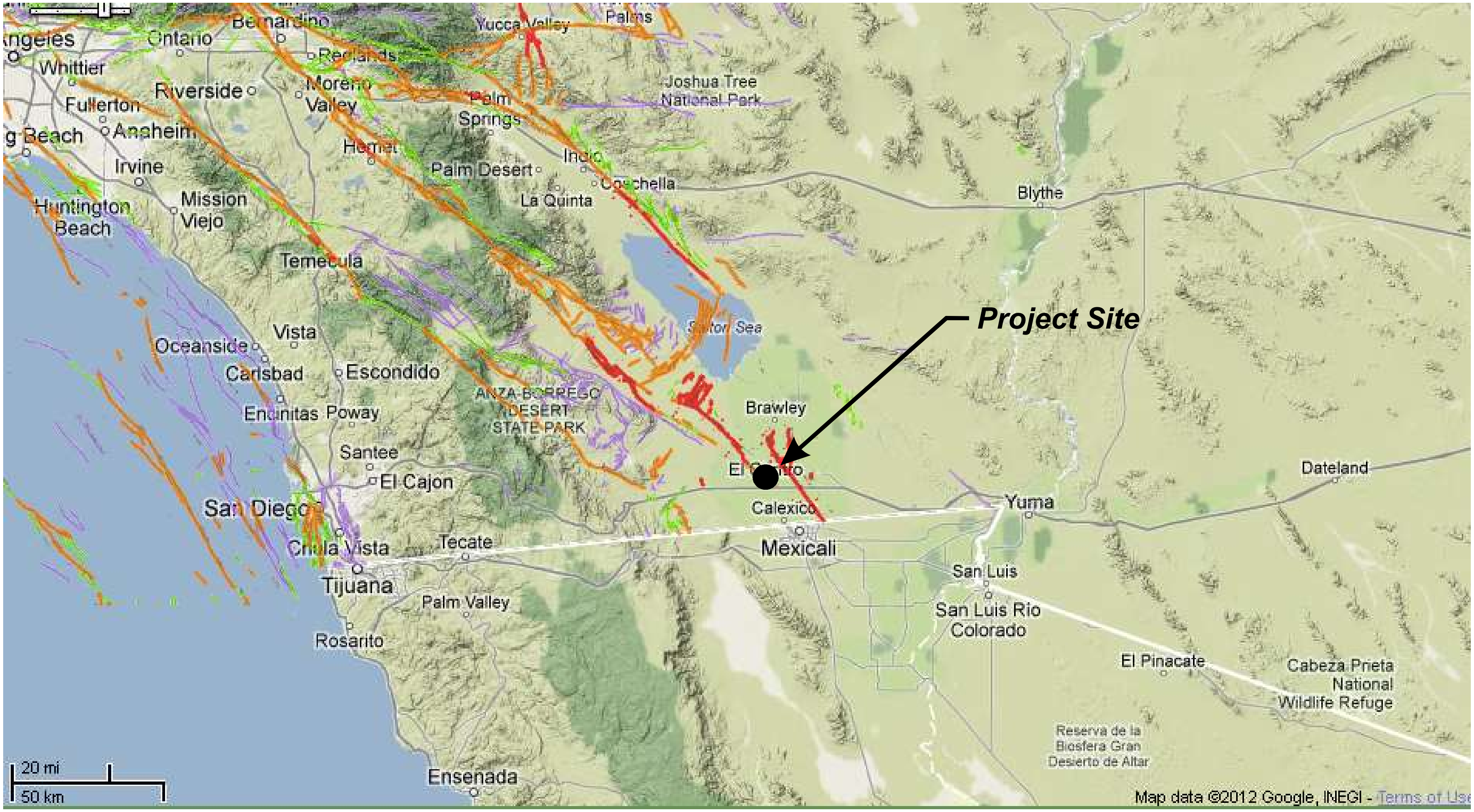
Design Earthquake Ground Motion

Design Spectral Response Acceleration Parameter (0.2 s)	$\mathbf{S_{DS}}$	1.241 g	= 2/3*S_{MS}	ASCE Equation 11.4-3
Design Spectral Response Acceleration Parameter (1.0 s)	$\mathbf{S_{D1}}$	0.709 g	= 2/3*S_{M1}	ASCE Equation 11.4-4
Risk Coefficient at Short Periods (less than 0.2 s)	$\mathbf{C_{RS}}$	0.957		ASCE Figure 22-17
Risk Coefficient at Long Periods (greater than 1.0 s)	$\mathbf{C_{R1}}$	0.927		ASCE Figure 22-18
	$\mathbf{T_L}$	8.00 sec		ASCE Figure 22-12
	$\mathbf{T_O}$	0.11 sec	=0.2*S_{D1}/S_{DS}	
	$\mathbf{T_S}$	0.57 sec	=S_{D1}/S_{DS}	
Peak Ground Acceleration	$\mathbf{PGA_M}$	0.84 g		ASCE Equation 11.8-1



Period T (sec)	Sa (g)	MCE_R Sa (g)
0.00	0.50	0.74
0.11	1.24	1.86
0.57	1.24	1.86
0.70	1.01	1.52
0.80	0.89	1.33
0.90	0.79	1.18
1.00	0.71	1.06
1.10	0.64	0.97
1.20	0.59	0.89
1.30	0.55	0.82
1.40	0.51	0.76
1.50	0.47	0.71
1.75	0.41	0.61
2.00	0.35	0.53
2.20	0.32	0.48
2.40	0.30	0.44
2.60	0.27	0.41
2.80	0.25	0.38
3.00	0.24	0.35
4.00	0.18	0.27
5.00	0.14	0.21

FIGURES



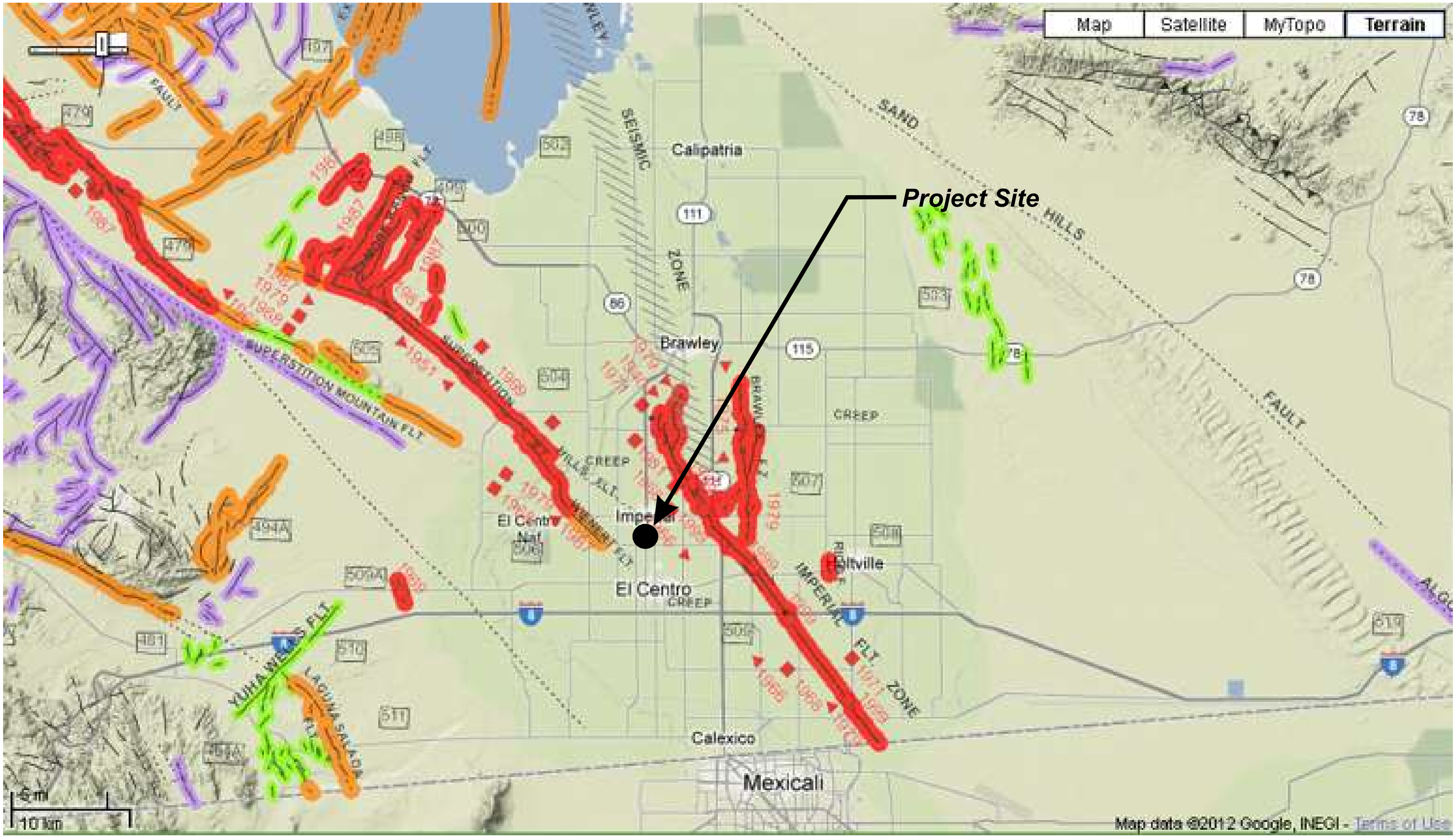
Source: California Geological Survey 2010 Fault Activity Map of California
http://www.quake.ca.gov/gmaps/FAM/faultactivitymap.html#

LANDMARK Geo-Engineers and Geologists

Project No.: LE25036

Regional Fault Map

Figure 1



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.: LE25036	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

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	0–200			Displacement during historic time (e.g. San Andreas fault 1906). Includes areas of known fault creep.	
Quaternary	Late Quaternary	Holocene	200–11,700			Displacement during Holocene time.	Fault offsets seafloor sediments or strata of Holocene age.
Quaternary	Late Quaternary	Pleistocene	11,700–700,000			Faults showing evidence of displacement during late Quaternary time.	Fault cuts strata of Late Pleistocene age.
Quaternary	Early Quaternary	Pleistocene	700,000–1,600,000*			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			1,600,000*–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.

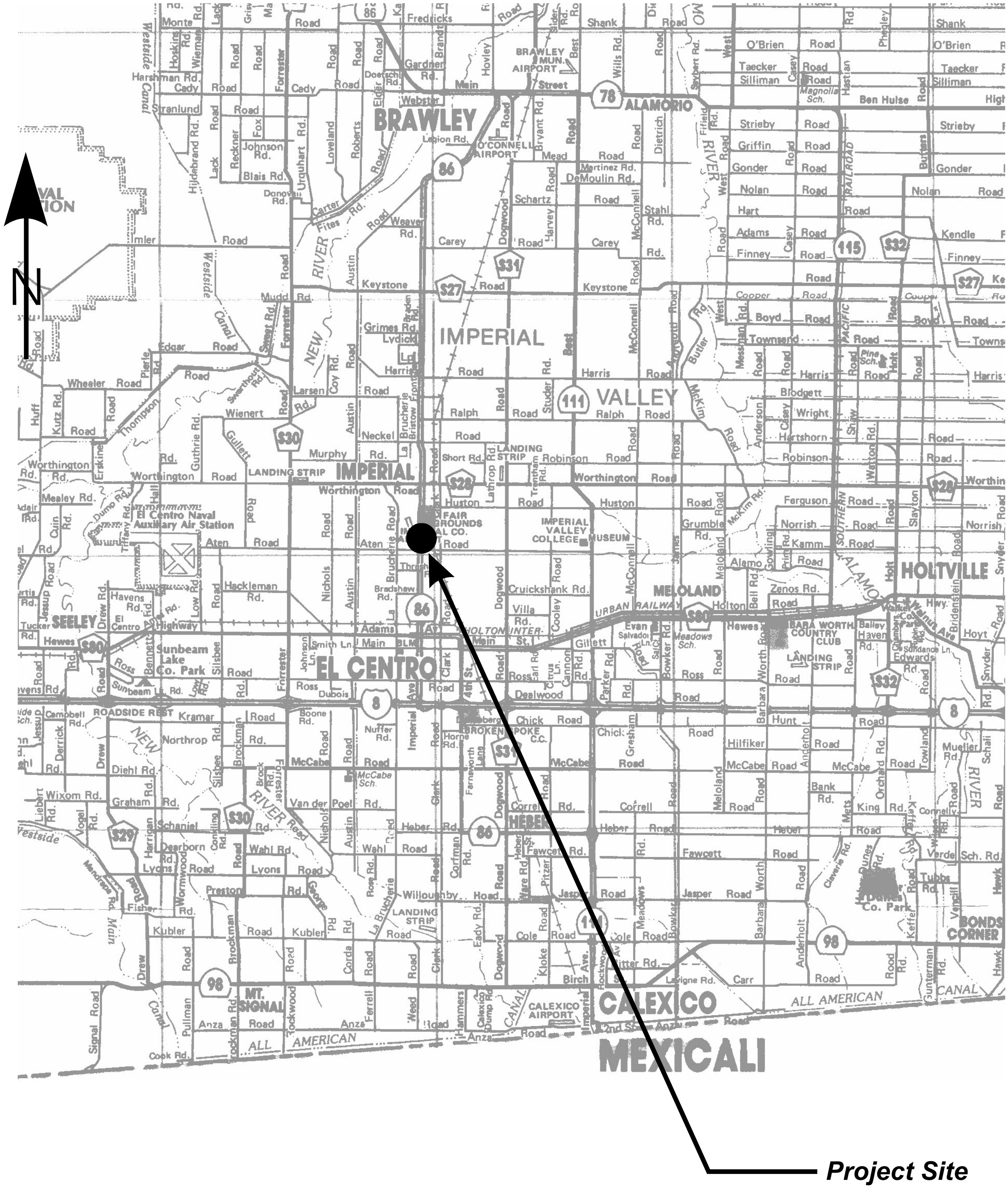
* Quaternary now recognized as extending to 2.6 Ma (Walker and Geissman, 2009). Quaternary faults in this map were established using the previous 1.6 Ma criterion.

LANDMARK
Geo-Engineers and Geologists
Project No.: LE25036

Fault Map Legend

Figure 3

APPENDIX A



LANDMARK
Geo-Engineers and Geologists

Project No.: LE25036

Vicinity Map

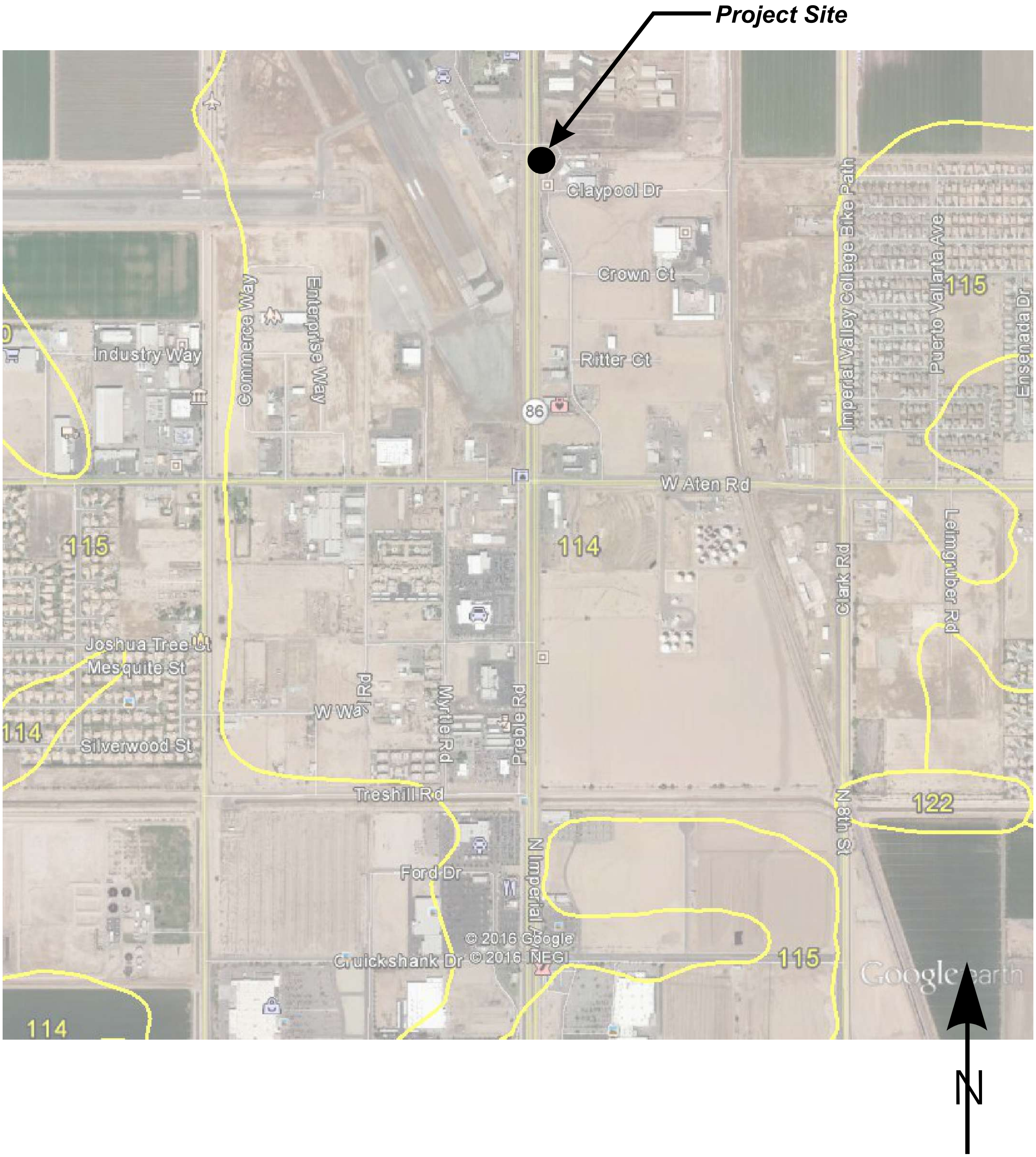
Plate A-1

LANDMARK
Geo-Engineers and Geologists

Project No.: LE25036

Site and Exploration Map

Plate A-2

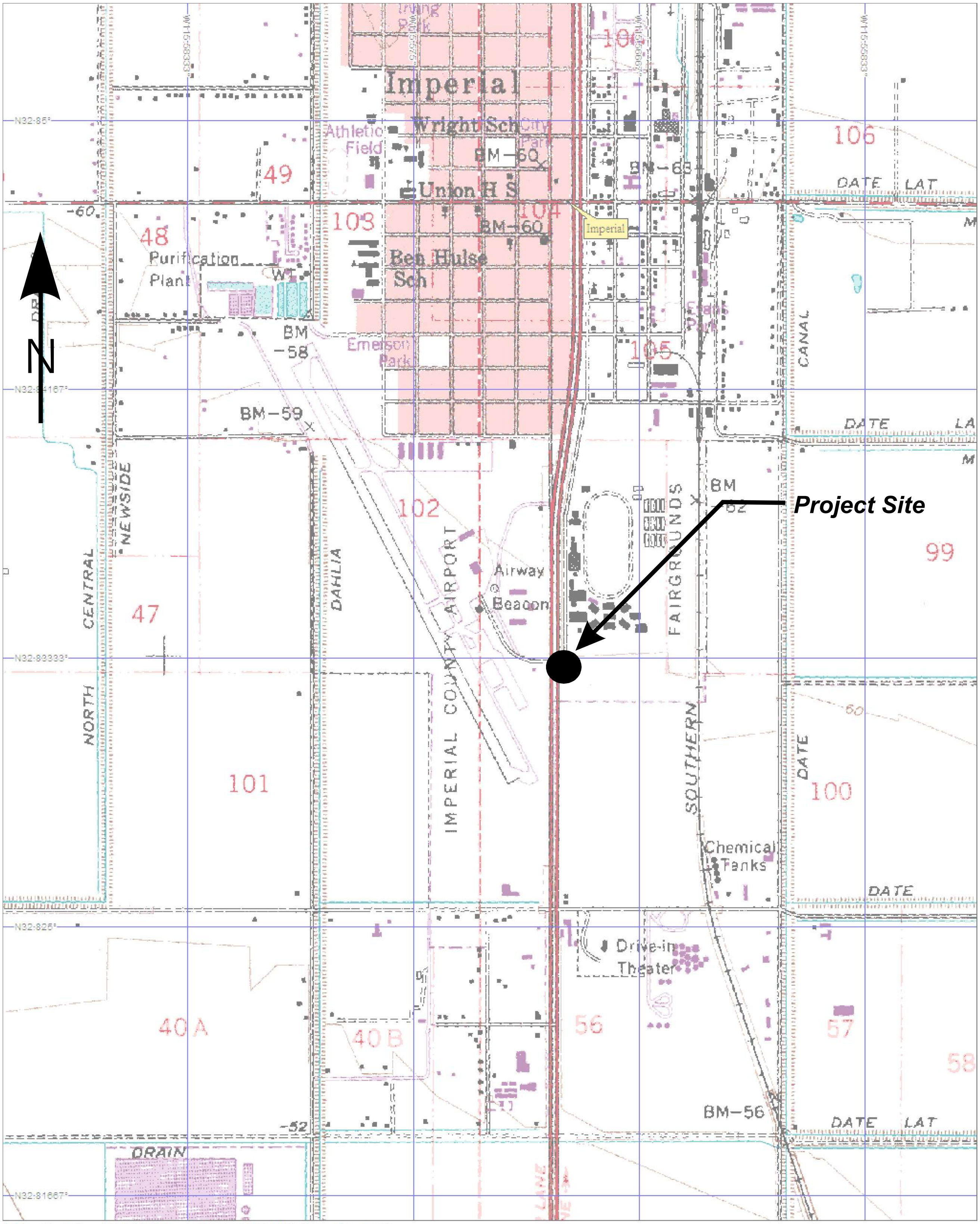


LANDMARK
Geo-Engineers and Geologists

Project No.: LE25036

Soil Survey Map

Plate
A-3



3-D TopoQuads Copyright © 1999 DeLorme Yarmouth, ME 04096 Source Data: USGS

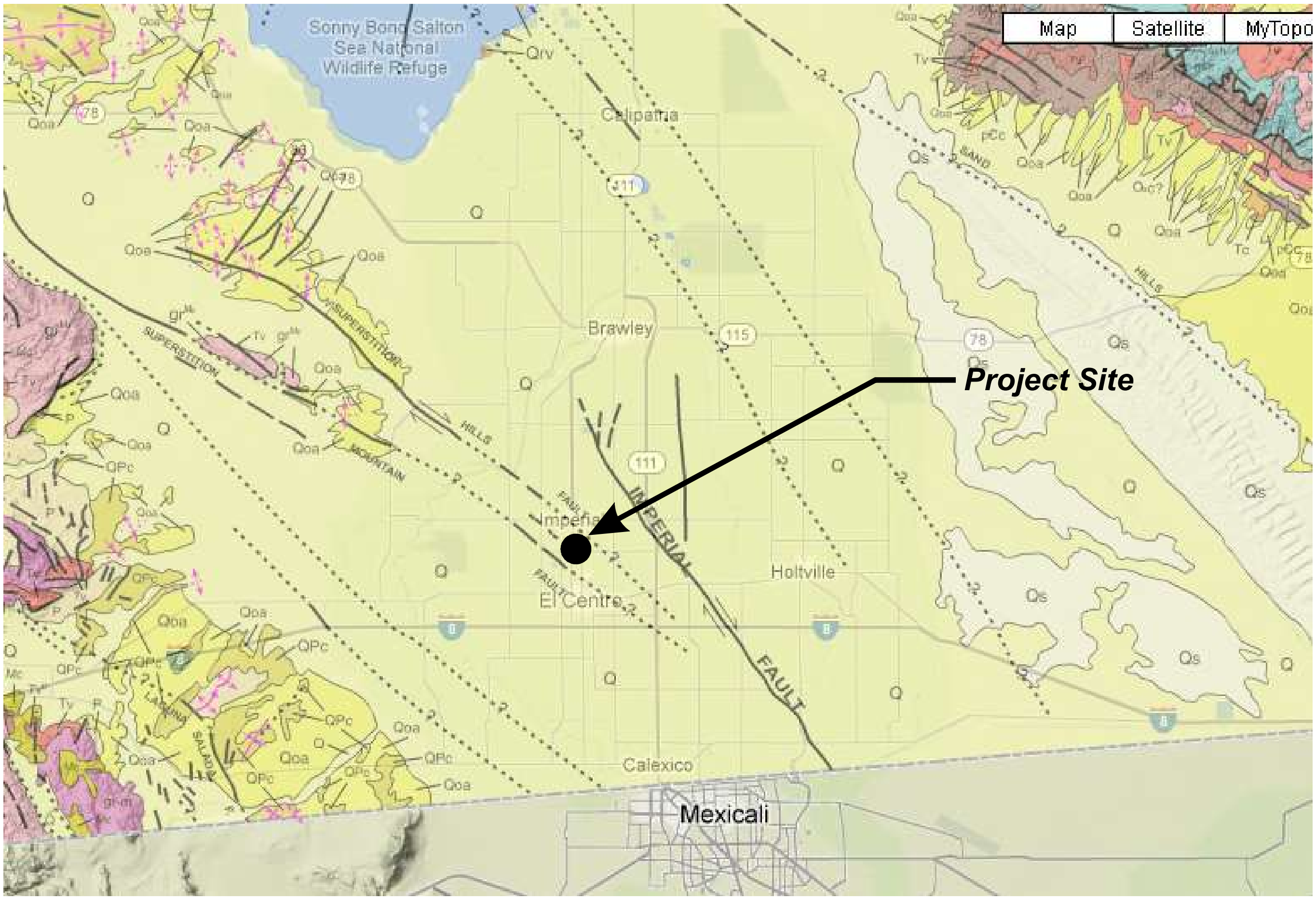
500 ft Scale: 1 : 17,600 Detail: 13-5 Datum: WGS84

LANDMARK
Geo-Engineers and Geologists

Project No.: LE25036

Topographic Map

Plate A-4



GEOLOGIC LEGEND

Quaternary Deposits: Qs, Q, Qls, Qg, Qoa, QPc

Quaternary Volcanic Rocks: Qrv, Qrv, Qv, Qv

Tertiary Sedimentary Rocks: Tc, P, M, Mc, Oc, Ocс, E, Ec, Ep

Tertiary Volcanic Rocks: Tv, Tv, Ti

Tertiary Plutonic Rocks: gr

Mesozoic Sedimentary and Metasedimentary Rocks: TK, K, Ku, Kl, KJf, KJf, KJf, J, Ŧ, sch, ls

Mesozoic Mixed Rocks: gr-m

Mesozoic Metavolcanic Rocks: Mzv, mv

Mesozoic Plutonic Rocks: gr, um, gb, gr

Paleozoic Sedimentary and Metasedimentary Rocks: Pz, Pm, C, D, SO, C

Paleozoic Mixed Rocks: m

Paleozoic Metavolcanic Rocks: Pzv

Paleozoic Plutonic Rocks: gr

Pre-Cambrian Rocks: pC, pCc, gr

SYMBOLS

Geologic boundary

Fault traces - solid where well located, dashed where approximately located or inferred, dotted where concealed, and queried where continuation or existence is uncertain. Ball and bar on downthrown side (relative or apparent). Arrows indicate direction of lateral movement (relative or apparent).

Thrust fault (barbs on upper plate).

Regional strike and dip of stratified rocks.

Regional strike and dip of stratified rocks (overturned).

Anticlinal fold.

Synclinal fold.

Monoclinal fold.

N

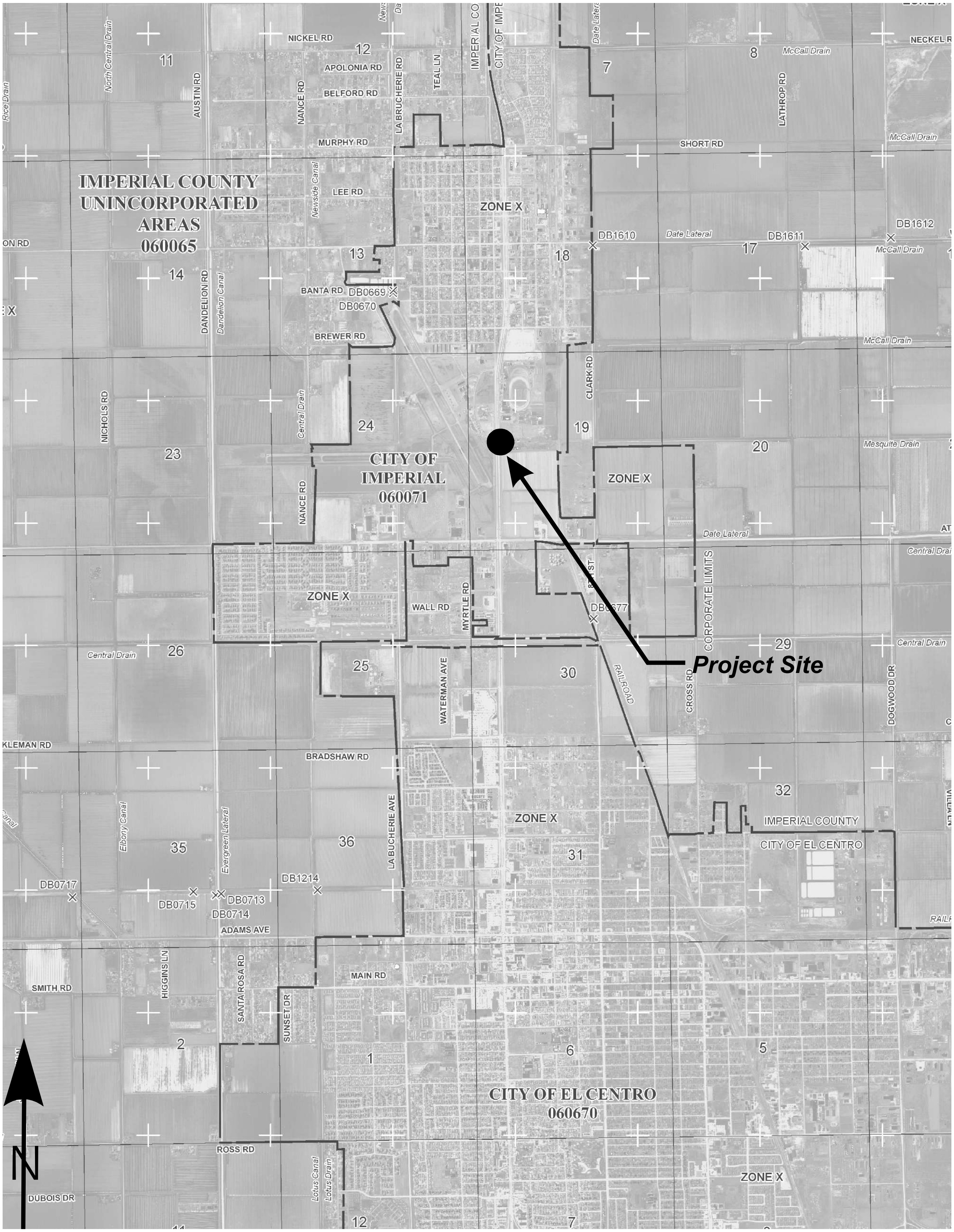
Site Location
Lat: 32.8329N Long: -115.5695W

LANDMARK
Geo-Engineers and Geologists

Project No.: LE25036

Regional Geologic Map

Plate A-5



LANDMARK
Geo-Engineers and Geologists

Project No.: LE25036

FEMA Flood Zones

Plate A-6

LEGEND

SPECIAL FLOOD HAZARD AREAS SUBJECT TO INUNDATION BY THE 1% ANNUAL CHANCE FLOOD

The 1% annual flood (100-year flood), also known as the base flood, is the flood that has a 1% chance of being equaled or exceeded in any given year. The Special Flood Hazard Area is the area subject to flooding by the 1% annual chance flood. Areas of Special Flood Hazard include Zones A, AE, AH, AO, AR, A99, V, and VE. The Base Flood Elevation is the water-surface elevation of the 1% annual chance flood.

ZONE A	No Base Flood Elevations determined.
ZONE AE	Base Flood Elevations determined.
ZONE AH	Flood depths of 1 to 3 feet (usually areas of ponding); Base Flood Elevations determined.
ZONE AO	Flood depths of 1 to 3 feet (usually sheet flow on sloping terrain); average depths determined. For areas of alluvial fan flooding, velocities also determined.
ZONE AR	Special Flood Hazard Area formerly protected from the 1% annual chance flood by a flood control system that was subsequently decertified. Zone AR indicates that the former flood control system is being restored to provide protection from the 1% annual chance or greater flood.
ZONE A99	Area to be protected from 1% annual chance flood by a Federal flood protection system under construction; no Base Flood Elevations determined.
ZONE V	Coastal flood zone with velocity hazard (wave action); no Base Flood Elevations determined.
ZONE VE	Coastal flood zone with velocity hazard (wave action); Base Flood Elevations determined.

FLOODWAY AREAS IN ZONE AE

The floodway is the channel of a stream plus any adjacent floodplain areas that must be kept free of encroachment so that the 1% annual chance flood can be carried without substantial increases in flood heights.

OTHER FLOOD AREAS

ZONE X	Areas of 0.2% annual chance flood; areas of 1% annual chance flood with average depths of less than 1 foot or with drainage areas less than 1 square mile; and areas protected by levees from 1% annual chance flood.

OTHER AREAS

ZONE X	Areas determined to be outside the 0.2% annual chance floodplain.
ZONE D	Areas in which flood hazards are undetermined, but possible.

COASTAL BARRIER RESOURCES SYSTEM (CBRS) AREAS

OTHERWISE PROTECTED AREAS (OPAs)

CBRS areas and OPAs are normally located within or adjacent to Special Flood Hazard Areas.

1% annual chance floodplain boundary

0.2% annual chance floodplain boundary

Floodway boundary

Zone D boundary

CBRS and OPA boundary

Boundary dividing Special Flood Hazard Area Zones and boundary dividing Special Flood Hazard Areas of different Base Flood Elevations, flood depths or flood velocities.

~~ 513 ~~~	Base Flood Elevation line and value; elevation in feet*
(EL 987)	Base Flood Elevation value where uniform within zone; elevation in feet*

* Referenced to the North American Vertical Datum of 1988

A ——— A	Cross section line
23 - - - - - - 23	Transect line
87°07'45", 32°22'30"	Geographic coordinates referenced to the North American Datum of 1983 (NAD 83), Western Hemisphere
$^{24}76^{000m}$N	1000-meter Universal Transverse Mercator grid values, zone 11N
600000 FT	5000-foot grid ticks: California State Plane coordinate system, zone VI (FIPSZONE 0406), Lambert Conformal Conic projection
DX5510 ×	Bench mark (see explanation in Notes to Users section of this FIRM panel)
● M1.5	River Mile

APPENDIX B

CLIENT: Dynmic Consulting Engineers
PROJECT: Imperial Sewer Lift Station - Imperial, CA

CONE PENETROMETER: Kehoe Testing & Engineering Truck Mounted Electric Cone with 30 ton reaction weight

LOCATION: See Site and Boring Location Plan

DATE: 2/25/2025

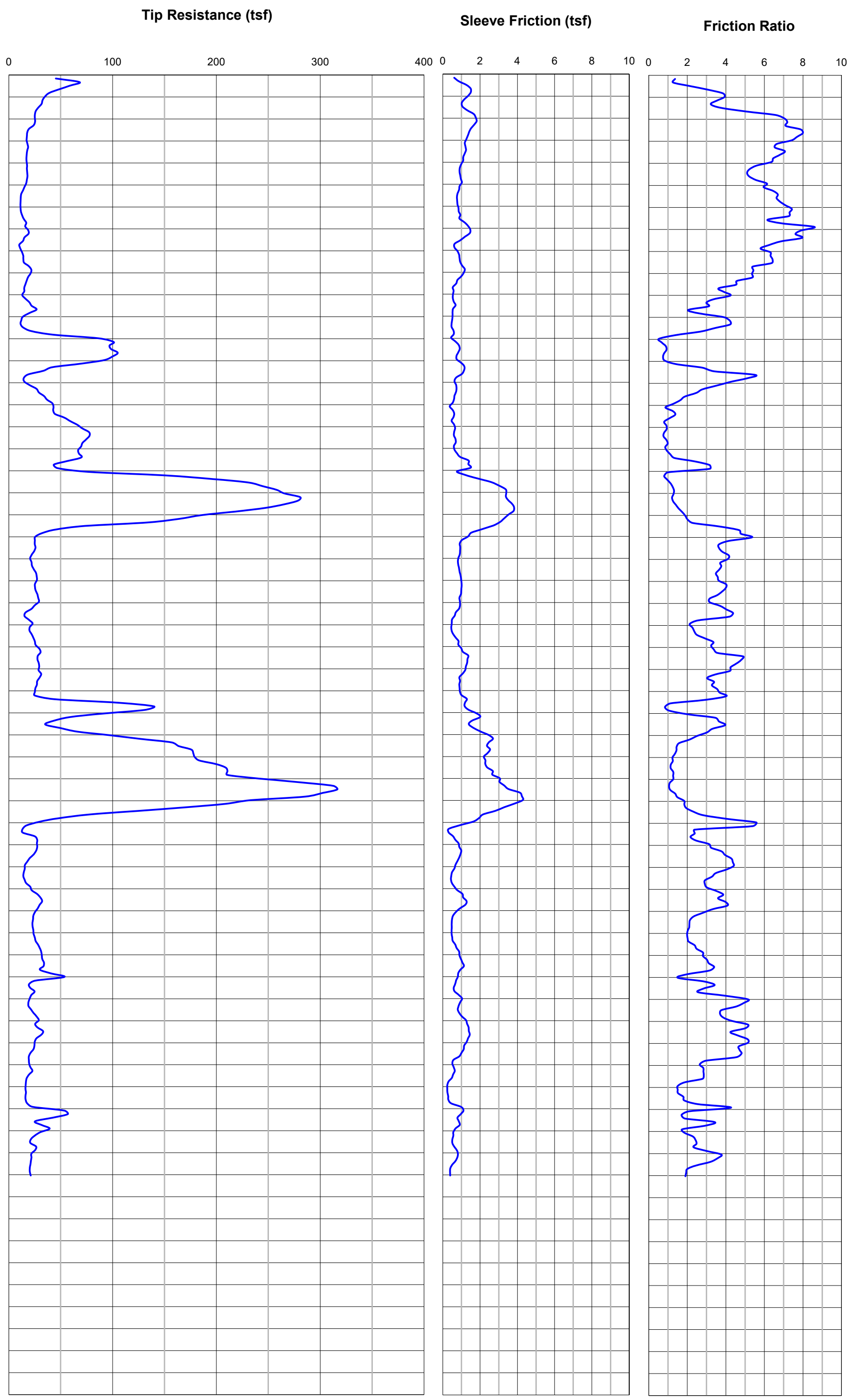
CONE SOUNDING DATA CPT-1

DEPTH

INTERPRETED SOIL PROFILE
From Robertson and Campanella (1989

GROUND ELEVATION +/-

Sandy Silt to Clayey Silt	ML	very dense
Clay	CL/CH	very stiff
Clay	" "	very stiff
Clay	" "	very stiff
Clay	" "	very stiff
Clay	" "	stiff
Clay	" "	stiff
Clay	" "	stiff
Clay	" "	stiff
Clay	" "	stiff
Clayey Silt to Silty Clay	ML/CL	very stiff
Sandy Silt to Clayey Silt	ML	medium dense
Sand to Silty Sand	SP/SM	dense
Clayey Silt to Silty Clay	ML/CL	very stiff
Sandy Silt to Clayey Silt	ML	medium dense
Silty Sand to Sandy Silt	SM/ML	medium dense
Sand to Silty Sand	SP/SM	medium dense
Silty Sand to Sandy Silt	SM/ML	medium dense
Sand	SP	very dense
Sand to Silty Sand	SP/SM	very dense
Clayey Silt to Silty Clay	ML/CL	hard
Silty Clay to Clay	CL	very stiff
Clayey Silt to Silty Clay	ML/CL	very stiff
Clayey Silt to Silty Clay	" "	very stiff
Clayey Silt to Silty Clay	" "	very stiff
Clayey Silt to Silty Clay	" "	very stiff
Silty Clay to Clay	CL	very stiff
Clayey Silt to Silty Clay	ML/CL	very stiff
Silty Sand to Sandy Silt	SM/ML	medium dense
Sandy Silt to Clayey Silt	ML	medium dense
Sand to Silty Sand	SP/SM	dense
Sand	SP	very dense
Sand	" "	very dense
Sandy Silt to Clayey Silt	ML	dense
Clayey Silt to Silty Clay	ML/CL	very stiff
Silty Clay to Clay	CL	very stiff
Clayey Silt to Silty Clay	ML/CL	stiff
Clayey Silt to Silty Clay	" "	very stiff
Sandy Silt to Clayey Silt	ML	loose
Sandy Silt to Clayey Silt	" "	loose
Sandy Silt to Clayey Silt	" "	loose
Clayey Silt to Silty Clay	ML/CL	very stiff
Silty Clay to Clay	CL	very stiff
Clay	CL/CH	very stiff
Silty Clay to Clay	CL	very stiff
Clayey Silt to Silty Clay	ML/CL	very stiff
Sandy Silt to Clayey Silt	ML	loose
Sandy Silt to Clayey Silt	" "	loose
Clayey Silt to Silty Clay	ML/CL	very stiff
Clayey Silt to Silty Clay	" "	very stiff



END OF SOUNDING AT 50 ft.

Project No.
LE25036

LANDMARK
Geo-Engineers and Geologists

PLATE
B-1

LANDMARK CONSULTANTS, INC.
CONE PENETROMETER INTERPRETATION (based on Robertson & Campanella, 1989, refer to Key to CPT logs)

Project: Imperial Sewer Lift Station - Imperial, CA **Project No:** LE25036 **Date:** 2/25/2025

CONE SOUNDING: CPT-1
Est. GWT (ft): 8
Phi Correlation: **0** 0-Schm(78),1-R&C(83),2-PHT(74)

Base Depth (m)	Base Depth (ft)	Avg Tip Qc, tsf	Avg Friction Ratio, %	Soil Classification	USCS	Density or Consistency	Est. Density (pcf)	SPT N(60)	Norm. Qc1n	Est. % Fines	Rel. Dens. Dr (%)	**Nk:** Phi (deg.)	**17** Su (tsf)	OCR
0.15	0.5	57.36	1.60	Silty Sand to Sandy Silt	SM/ML	very dense	115	13	108.4	30	118	45		
0.30	1.0	40.73	3.62	Clayey Silt to Silty Clay	ML/CL	hard	120	16		55			2.39	>10
0.45	1.5	30.88	3.49	Clayey Silt to Silty Clay	ML/CL	very stiff	120	12		60			1.81	>10
0.60	2.0	25.38	6.22	Clay	CL/CH	very stiff	125	20		85			1.49	>10
0.75	2.5	22.76	7.39	Clay	CL/CH	very stiff	125	18		95			1.33	>10
0.93	3.0	17.43	7.70	Clay	CL/CH	very stiff	125	14		100			1.02	>10
1.08	3.5	18.00	6.75	Clay	CL/CH	very stiff	125	14		100			1.05	>10
1.23	4.0	17.12	6.55	Clay	CL/CH	stiff	125	14		100			0.99	>10
1.38	4.5	17.55	5.31	Clay	CL/CH	very stiff	125	14		90			1.02	>10
1.53	5.0	17.37	5.64	Clay	CL/CH	very stiff	125	14		95			1.00	>10
1.68	5.5	13.05	6.43	Clay	CL/CH	stiff	125	10		100			0.75	>10
1.83	6.0	11.27	7.11	Clay	CL/CH	stiff	125	9		100			0.64	>10
1.98	6.5	13.06	6.91	Clay	CL/CH	stiff	125	10		100			0.75	>10
2.13	7.0	17.06	7.80	Clay	CL/CH	stiff	125	14		100			0.98	>10
2.28	7.5	15.97	7.48	Clay	CL/CH	stiff	125	13		100			0.91	>10
2.45	8.0	11.18	6.12	Clay	CL/CH	stiff	125	9		100			0.63	>10
2.60	8.5	14.15	6.37	Clay	CL/CH	stiff	125	11		100			0.80	>10
2.75	9.0	20.64	5.39	Clay	CL/CH	very stiff	125	17		90			1.18	>10
2.90	9.5	17.27	4.82	Clay	CL/CH	stiff	125	14		90			0.99	>10
3.05	10.0	14.21	3.92	Silty Clay to Clay	CL	stiff	125	8		95			0.80	>10
3.20	10.5	19.21	3.16	Clayey Silt to Silty Clay	ML/CL	very stiff	120	8		75			1.10	>10
3.35	11.0	19.97	2.87	Clayey Silt to Silty Clay	ML/CL	very stiff	120	8		75			1.14	>10
3.50	11.5	12.36	3.96	Clay	CL/CH	stiff	125	10		100			0.69	8.14
3.65	12.0	50.37	1.52	Silty Sand to Sandy Silt	SM/ML	medium dense	115	11	63.2	35	59	36		
3.80	12.5	98.94	0.82	Sand to Silty Sand	SP/SM	dense	115	18	122.8	15	79	39		
3.95	13.0	99.15	0.78	Sand to Silty Sand	SP/SM	dense	115	18	121.7	15	78	39		
4.13	13.5	48.61	2.54	Sandy Silt to Clayey Silt	ML	medium dense	115	14	59.1	50	57	36		
4.28	14.0	16.03	4.92	Clay	CL/CH	stiff	125	13		100			0.90	>10
4.43	14.5	25.27	2.91	Clayey Silt to Silty Clay	ML/CL	very stiff	120	10		70			1.45	>10
4.58	15.0	37.55	1.59	Silty Sand to Sandy Silt	SM/ML	medium dense	115	8	44.2	45	48	35		
4.73	15.5	43.53	1.16	Silty Sand to Sandy Silt	SM/ML	medium dense	115	10	50.7	40	52	35		
4.88	16.0	62.55	0.93	Sand to Silty Sand	SP/SM	medium dense	115	11	72.2	25	63	37		
5.03	16.5	76.52	0.83	Sand to Silty Sand	SP/SM	medium dense	115	14	87.5	20	69	38		
5.18	17.0	69.12	0.93	Sand to Silty Sand	SP/SM	medium dense	115	13	78.3	25	65	37		
5.33	17.5	64.76	1.63	Silty Sand to Sandy Silt	SM/ML	medium dense	115	14	72.7	35	63	37		
5.48	18.0	55.76	2.44	Sandy Silt to Clayey Silt	ML	medium dense	115	16	62.1	50	58	36		
5.65	18.5	193.15	0.96	Sand	SP	very dense	110	30	213.3	10	95	41		
5.80	19.0	256.10	1.28	Sand	SP	very dense	110	39	280.7	15	103	42		
5.95	19.5	274.79	1.29	Sand	SP	very dense	110	42	298.9	15	105	43		
6.10	20.0	217.13	1.73	Sand to Silty Sand	SP/SM	very dense	115	39	234.4	20	98	42		
6.25	20.5	124.49	2.65	Silty Sand to Sandy Silt	SM/ML	dense	115	28	133.3	35	81	39		
6.40	21.0	34.15	4.94	Clay	CL/CH	very stiff	125	27		85			1.96	>10
6.55	21.5	25.24	3.81	Clayey Silt to Silty Clay	ML/CL	very stiff	120	10		90			1.43	>10
6.70	22.0	22.27	4.02	Silty Clay to Clay	CL	very stiff	125	13		95			1.26	>10
6.85	22.5	22.73	3.71	Silty Clay to Clay	CL	very stiff	125	13		95			1.28	>10
7.00	23.0	26.70	3.57	Clayey Silt to Silty Clay	ML/CL	very stiff	120	11		85			1.52	>10
7.18	23.5	25.45	3.94	Silty Clay to Clay	CL	very stiff	125	15		90			1.44	>10
7.33	24.0	28.30	3.29	Clayey Silt to Silty Clay	ML/CL	very stiff	120	11		85			1.61	>10
7.48	24.5	20.91	4.04	Silty Clay to Clay	CL	very stiff	125	12		100			1.17	>10
7.63	25.0	19.09	2.95	Clayey Silt to Silty Clay	ML/CL	very stiff	120	8		100			1.07	>10
7.78	25.5	20.73	2.37	Clayey Silt to Silty Clay	ML/CL	very stiff	120	8		90			1.16	>10
7.93	26.0	24.85	3.17	Clayey Silt to Silty Clay	ML/CL	very stiff	120	10		90			1.40	>10
8.08	26.5	28.87	4.15	Silty Clay to Clay	CL	very stiff	125	16		95			1.64	>10
8.23	27.0	28.91	4.33	Silty Clay to Clay	CL	very stiff	125	17		95			1.64	>10
8.38	27.5	29.39	3.28	Clayey Silt to Silty Clay	ML/CL	very stiff	120	12		85			1.67	>10
8.53	28.0	25.91	3.50	Clayey Silt to Silty Clay	ML/CL	very stiff	120	10		95			1.46	>10
8.68	28.5	58.00	2.71	Sandy Silt to Clayey Silt	ML	medium dense	115	17	54.7	60	55	36		
8.85	29.0	120.21	1.30	Sand to Silty Sand	SP/SM	dense	115	22	112.7	30	76	39		
9.00	29.5	45.54	3.69	Clayey Silt to Silty Clay	ML/CL	hard	120	18		75			2.61	>10
9.15	30.0	70.82	2.96	Sandy Silt to Clayey Silt	ML	medium dense	115	20	65.6	55	60	36		
9.30	30.5	149.06	1.72	Sand to Silty Sand	SP/SM	dense	115	27	137.2	30	82	39		
9.45	31.0	177.58	1.34	Sand to Silty Sand	SP/SM	dense	115	32	162.5	25	87	40		
9.60	31.5	197.40	1.18	Sand	SP	dense	110	30	179.6	20	90	41		
9.75	32.0	220.66	1.27	Sand	SP	very dense	110	34	199.7	20	93	41		
9.90	32.5	302.55	1.09	Sand	SP	very dense	110	47	272.4	15	102	42		
10.05	33.0	273.85	1.57	Sand to Silty Sand	SP/SM	very dense	115	50	245.3	20	99	42		
10.20	33.5	166.97	2.04	Silty Sand to Sandy Silt	SM/ML	dense	115	37	148.7	30	84	40		
10.38	34.0	51.12	4.16	Clayey Silt to Silty Clay	ML/CL	hard	120	20		80			2.94	>10
10.53	34.5	14.56	3.38	Silty Clay to Clay	CL	stiff	125	8		100			0.78	4.09
10.68	35.0	26.45	2.59	Clayey Silt to Silty Clay	ML/CL	very stiff	120	11		90			1.48	>10
10.83	35.5	25.88	3.66	Clayey Silt to Silty Clay	ML/CL	very stiff	120	10		100			1.45	>10
10.98	36.0	17.88	4.35	Clay	CL/CH	stiff	125	14		100			0.98	4.18
11.13	36.5	14.68	3.43	Silty Clay to Clay	CL	stiff	125	8		100			0.79	3.83
11.28	37.0	19.82	3.15	Clayey Silt to Silty Clay	ML/CL	very stiff	120	8		100			1.09	8.56
11.43	37.5	29.94	3.81	Clayey Silt to Silty Clay	ML/CL	very stiff	120	12		100			1.68	>10
11.58	38.0	27.55	3.41	Clayey Silt to Silty Clay	ML/CL	very stiff	120	11		100			1.54	>10
11.73	38.5	23.21	2.20	Sandy Silt to Clayey Silt	ML	loose	115	7	19.5	95	24	31		

LANDMARK CONSULTANTS, INC.
CONE PENETROMETER INTERPRETATION (based on Robertson & Campanella, 1989, refer to Key to CPT logs)

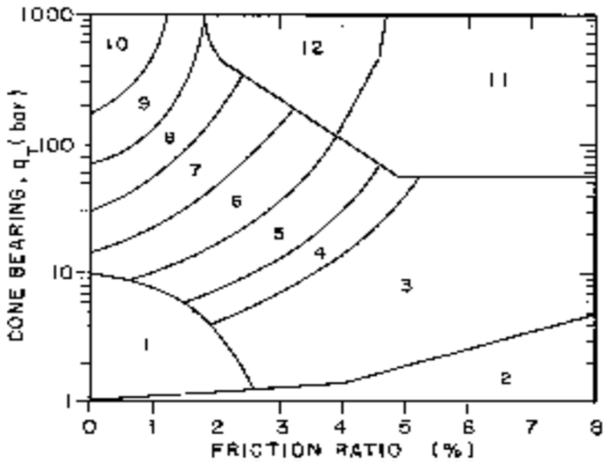
Project: Imperial Sewer Lift Station - Imperial, CA **Project No:** LE25036 **Date:** 2/25/2025

CONE SOUNDING: CPT-1
Est. GWT (ft): 8
Phi Correlation: **0** 0-Schm(78),1-R&C(83),2-PHT(74)

Base Depth (m)	Base Depth (ft)	Avg Tip Qc, tsf	Avg Friction Ratio, %	Soil Classification	USCS	Density or Consistency	Est. Density (pcf)	SPT N(60)	Norm. Qc1n	Est. % Fines	Rel. Dens. Dr (%)	**Nk:** Phi (deg.)	**17** Su (tsf)	OCR
11.88	39.0	23.46	2.04	Sandy Silt to Clayey Silt	ML	loose	115	7	19.6	95	24	31		
12.05	39.5	26.27	2.15	Sandy Silt to Clayey Silt	ML	loose	115	8	21.9	90	28	32		
12.20	40.0	30.97	2.70	Sandy Silt to Clayey Silt	ML	loose	115	9	25.7	90	32	33		
12.35	40.5	33.12	3.18	Clayey Silt to Silty Clay	ML/CL	very stiff	120	13		95			1.87	>10
12.50	41.0	41.40	2.23	Sandy Silt to Clayey Silt	ML	medium dense	115	12	34.0	75	41	34		
12.65	41.5	21.82	3.03	Clayey Silt to Silty Clay	ML/CL	very stiff	120	9		100			1.20	8.70
12.80	42.0	22.18	3.90	Silty Clay to Clay	CL	very stiff	125	13		100			1.22	6.32
12.95	42.5	19.73	4.40	Clay	CL/CH	very stiff	125	16		100			1.08	3.91
13.10	43.0	26.55	3.96	Silty Clay to Clay	CL	very stiff	125	15		100			1.48	8.27
13.25	43.5	28.67	4.80	Clay	CL/CH	very stiff	125	23		100			1.60	7.00
13.40	44.0	27.58	4.96	Clay	CL/CH	very stiff	125	22		100			1.53	6.43
13.58	44.5	23.15	4.73	Clay	CL/CH	very stiff	125	19		100			1.27	4.78
13.73	45.0	19.39	3.39	Clayey Silt to Silty Clay	ML/CL	very stiff	120	8		100			1.05	6.21
13.88	45.5	21.00	2.84	Clayey Silt to Silty Clay	ML/CL	very stiff	120	8		100			1.14	6.88
14.03	46.0	16.67	2.04	Clayey Silt to Silty Clay	ML/CL	stiff	120	7		100			0.89	4.68
14.18	46.5	16.15	1.61	Sandy Silt to Clayey Silt	ML	very loose	115	5	12.5	100	11	30		
14.33	47.0	27.50	2.68	Clayey Silt to Silty Clay	ML/CL	very stiff	120	11		100			1.52	>10
14.48	47.5	41.36	2.33	Sandy Silt to Clayey Silt	ML	loose	115	12	31.8	80	39	33		
14.63	48.0	33.70	2.17	Sandy Silt to Clayey Silt	ML	loose	115	10	25.8	85	32	33		
14.78	48.5	22.24	2.40	Clayey Silt to Silty Clay	ML/CL	very stiff	120	9		100			1.21	7.00
14.93	49.0	24.36	3.04	Clayey Silt to Silty Clay	ML/CL	very stiff	120	10		100			1.34	8.14
15.10	49.5	21.27	3.08	Clayey Silt to Silty Clay	ML/CL	very stiff	120	9		100			1.15	6.32
15.25	50.0	20.48	1.95	Sandy Silt to Clayey Silt	ML	very loose	115	6	15.4	100	17	30		

Simplified Soil Classification Chart

After Robertson & Campanella (1989)



Geotechnical Parameters from CPT Data:

Equivalent SPT N(60) blow count = Qc/(Qc/N Ratio)

N1(60) = Cn*N(60) Normalized SPT blow count

Cn = 1/(p'o)^0.5 < 1.6 max. from Liao & Whitman (1986)

p'o = effective overburden pressure (tsf) using unit densities given below and estimated groundwater table.

Dr = Relative density (%) from Jamiolkowski et. al. (1986) relationship = -98 +68*log(Qc/p'o^0.5) where Qc, p'o in tonne/sqm

Note: 1 tonne/sqm = 0.1024 tsf, 1 bar =1.0443 tsf

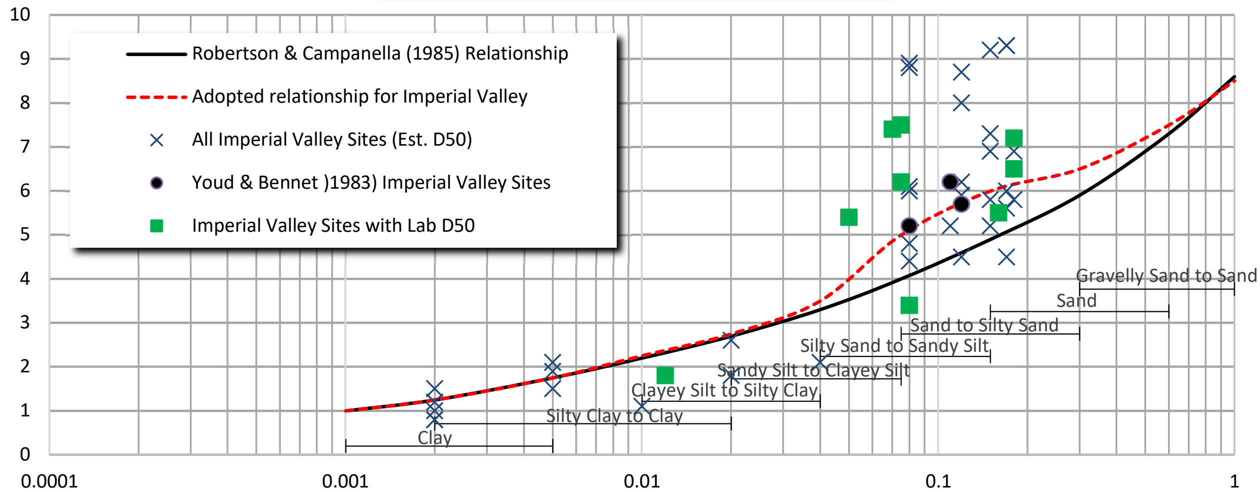
Phi = Friction Angle estimated from either:

1. Roberton & Campanella (1983) chart:
   Phi = 5.3 + 24*(log(Qc/p'o))+3(log(Qc/p'o))^2
2. Peck, Hansen & Thornburn (1974) N-Phi Correlation
3. Schmertman (1978) chart [Phi = 28+0.14*Dr for fine uniform sands]

Su = undrained shear strength (tsf) = (Qc-p'o)/Nk where Nk varies from 10 to 22, 17 for OC clays

OCR = Overconsolidation Ratio estimated from Schmertman (1978) chart using Su/p'o ratio and estimated normal consolidated Su/p'o

Variation of Qc/N Ratio with Grain Size



Note: Assumed Properties and Adopted Qc/N Ratio based on correlations from Imperial Valley, California soils

Table of Soil Types and Assumed Properties

Zone	Soil Classification	UCS	Density (pcf)	R&C Qc/N	Adopted Qc/N	Est. PI	Fines (%)	D50 (mm)
1	Sensitive fine grained	ML	120	2	2	NP-15	65-100	0.02
2	Organic Material	OL/OH	120	1	1	--	--	--
3	Clay	CL/CH	125	1	1.25	25-40+	90-100	0.002
4	Silty Clay to Clay	CL	125	1.5	2	15-40	90-100	0.01
5	Clayey Silt to Silty Clay	ML/CL	120	2	2.75	25-May	90-100	0.02
6	Sandy Silt to Clayey Silt	ML	115	2.5	3.5	NP-10	65-100	0.04
7	Silty Sand to Sandy Silt	SM/ML	115	3	5	NP	35-75	0.075
8	Sand to Silty Sand	SP/SM	115	4	6	NP	May-35	0.15
9	Sand	SP	110	5	6.5	NP	0-5	0.3
10	Gravelly Sand to Sand	SW	115	6	7.5	NP	0-5	0.6
11	Overconsolidated Soil	--	120	1	1	NP	90-100	0.01
12	Sand to Clayey Sand	SP/SC	115	2	2	NP-5	--	---

Su (tsf)	Consistency
0-0.13	very soft
0.13-.25	soft
0.25-0.5	firm
0.5-1.0	stiff
1.0-2.0	very stiff
>2.0	hard
Dr (%)	**Relative Density**
0-15	very loose
15-35	loose
35-65	medium dense
65-85	dense
>85	very dense

LANDMARK
Geo-Engineers and Geologists

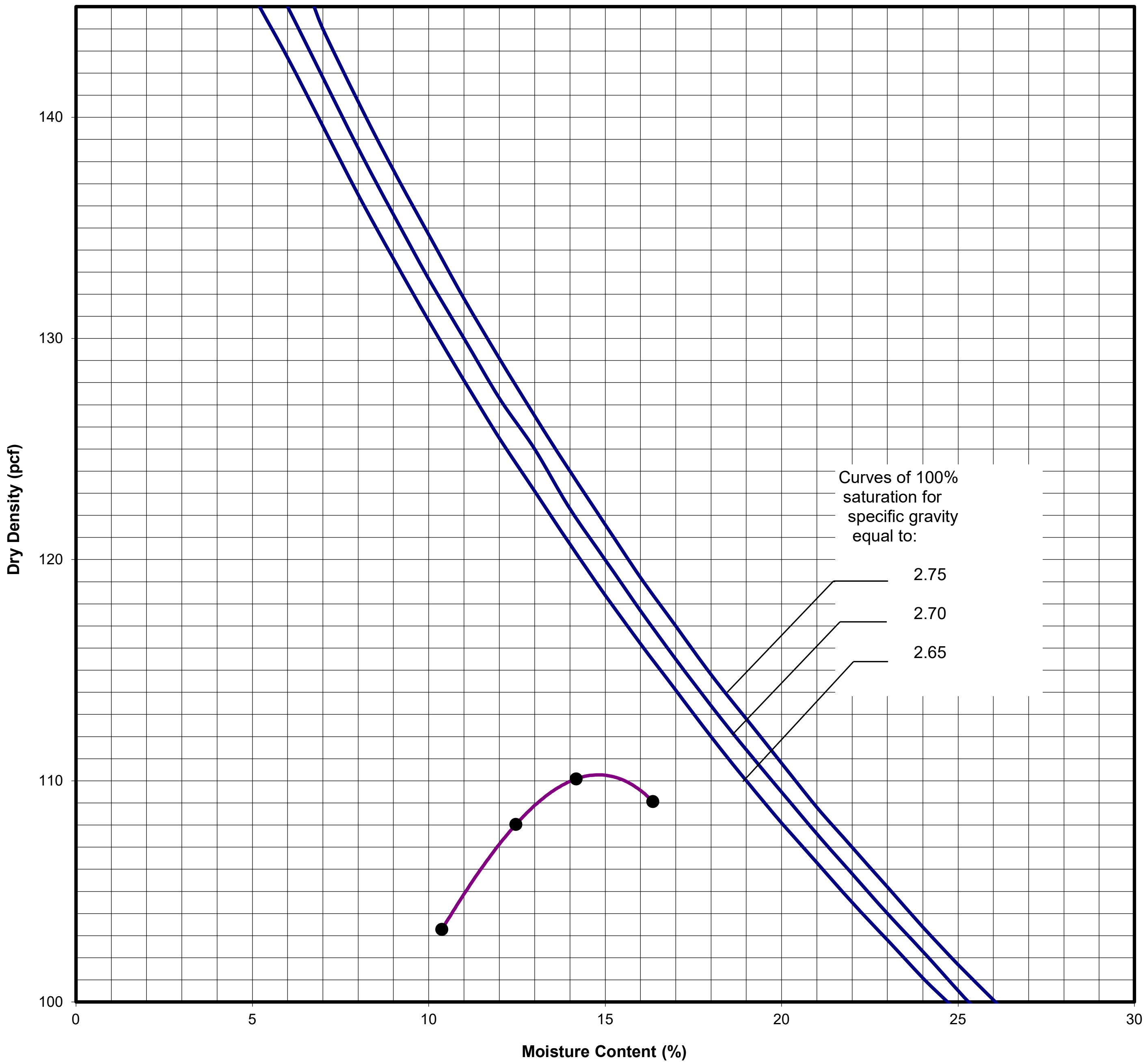
Project No: LE25036

Key to CPT Interpretation of Logs

Plate B-2

APPENDIX C

Client:	Dynamic Consulting Engineers	**Soil Description:**	Silty Clay/Clay (CL-CH)
Project:	Imperial Sewer Lift Station-Imperial,	**Sample Location:**	CPT-1 @ 0-3'
Project No.:	LE25036	**Test Method:**	D1557-A
Date:	2/27/2025	**Maximum Dry Density (pcf):**	**110.5**
Lab. No.:	EC25-62	**Optimum Moisture Content (%):**	**15.0**



LANDMARK Geo-Engineers and Geologists

Project No.: LE25036

Moisture Density Relationship

Plate C-1

LANDMARK CONSULTANTS, INC.

CLIENT: Dynamic Consulting Engineers, Inc.
PROJECT: Imperial Sewer Lift Station - SEC Hwy 86 and Claypool Drive
JOB No.: LE25036
DATE: 02/21/25

CHEMICAL ANALYSIS

		Caltrans Method
Boring:	CPT-1	
Sample Depth, ft:	0-3	
pH:	7.45	643
Electrical Conductivity (mmhos):	7.94	424
Resistivity (ohm-cm):	140	643
Chloride (Cl), ppm:	5,200	422
Sulfate (SO4), ppm:	6,132	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.: **LE25036**

Selected Chemical Test Results

Plate C-2

APPENDIX D

Landmark Consultants, Inc.
780 N. 4th Street
El Centro, CA 92243

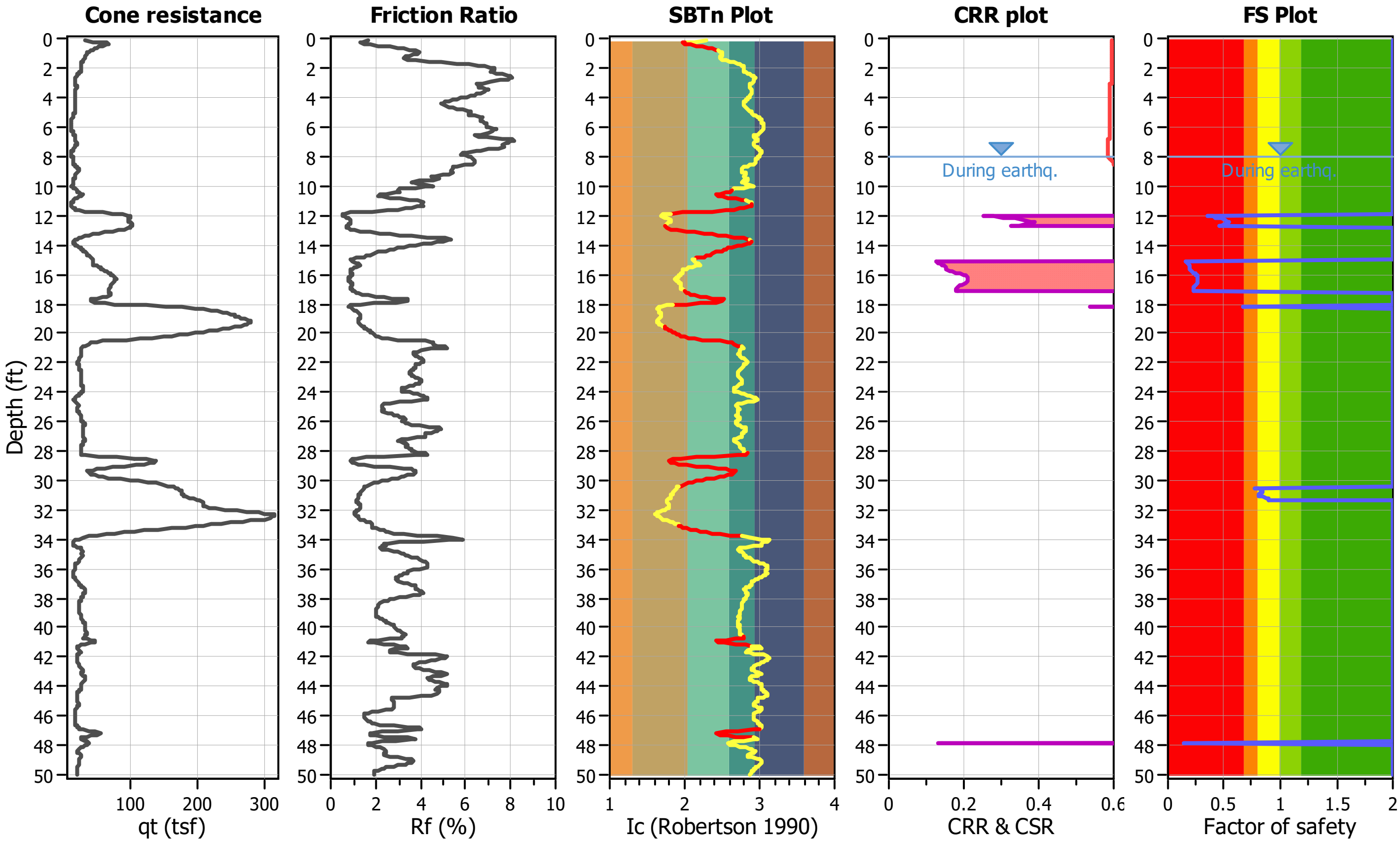
LIQUEFACTION ANALYSIS REPORT

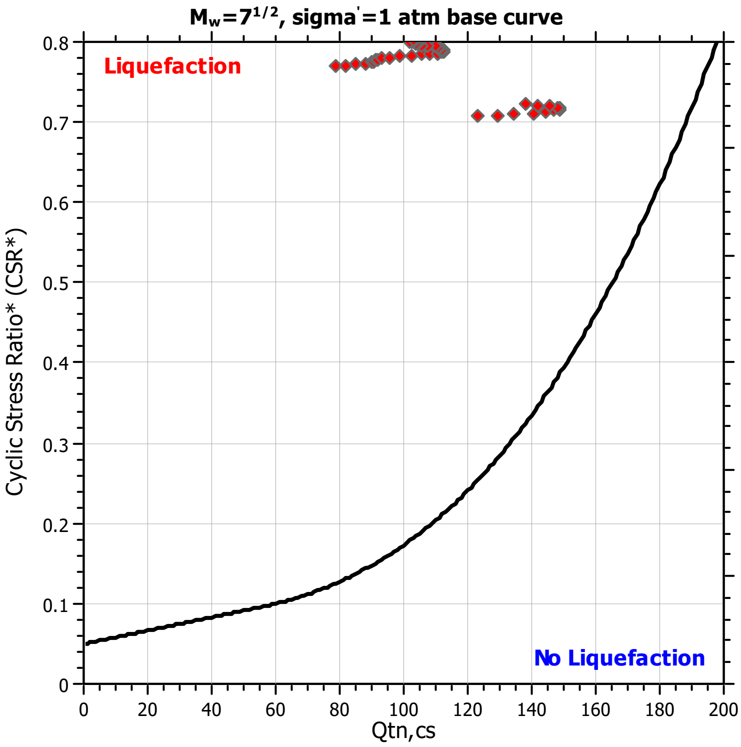
Project title : Imperial Sewer Lift Station **Location : Imperial, CA**

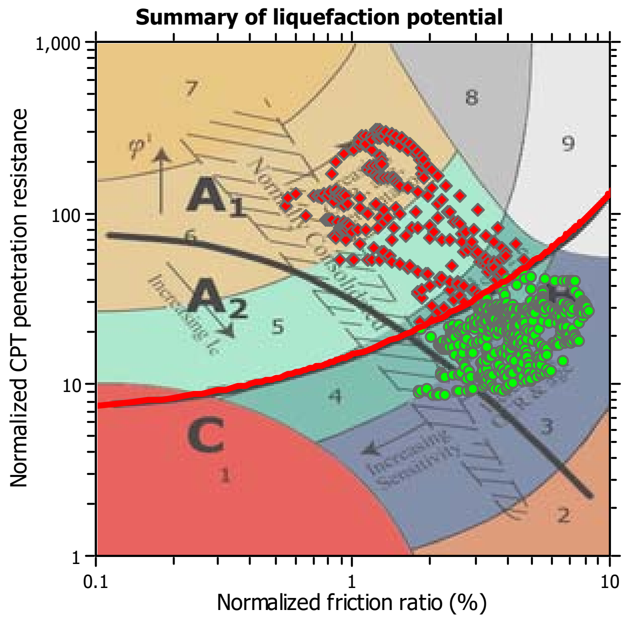
CPT file : CPT-1

Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	8.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	8.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	7.00	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.84	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

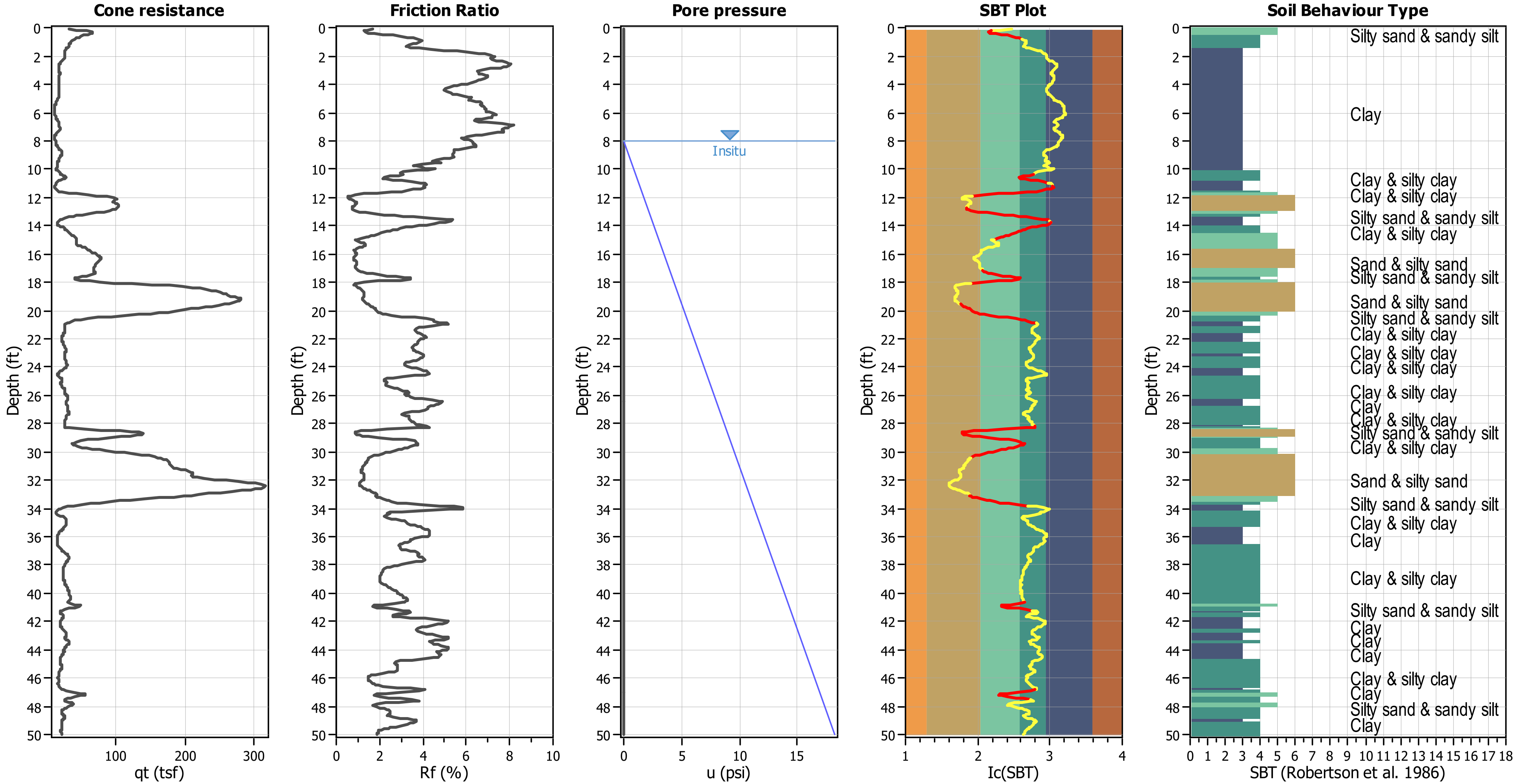






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

CPT basic interpretation plots



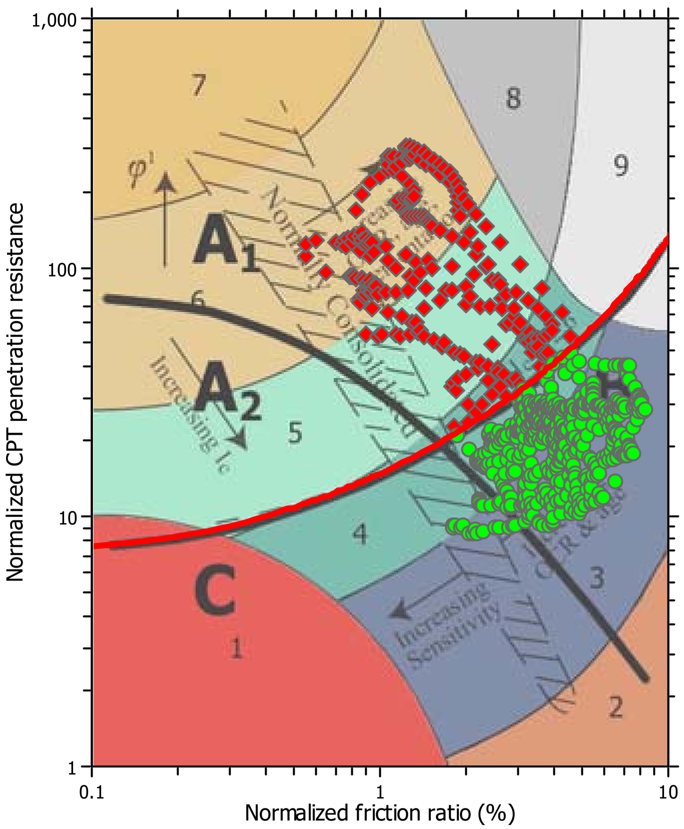
Input parameters and analysis data

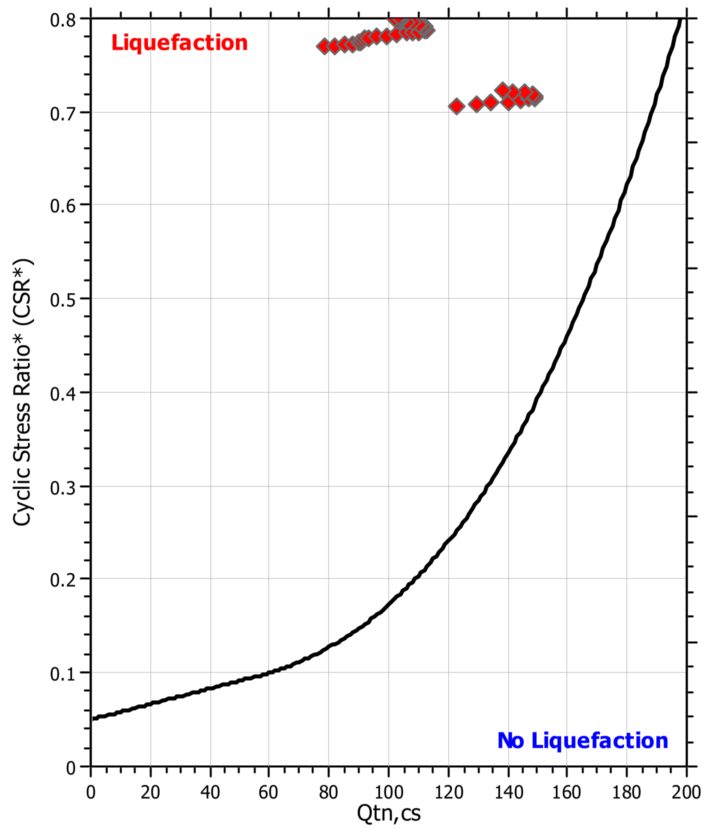
Analysis method:	NCEER (1998)	Depth to water table (erthq.):	8.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.84	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	8.00 ft	Fill height:	N/A	Limit depth:	N/A

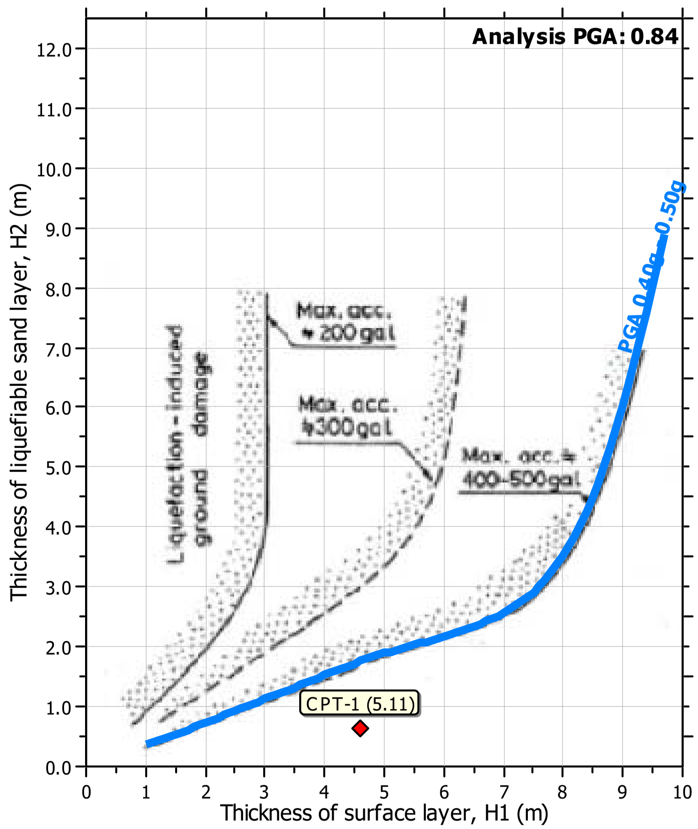
SBT legend

- 1. Sensitive fine grained
- 2. Organic material
- 3. Clay to silty clay
- 4. Clayey silt to silty
- 5. Silty sand to sandy silt
- 6. Clean sand to silty sand
- 7. Gravely sand to sand
- 8. Very stiff sand to
- 9. Very stiff fine grained

Liquefaction analysis summary plots



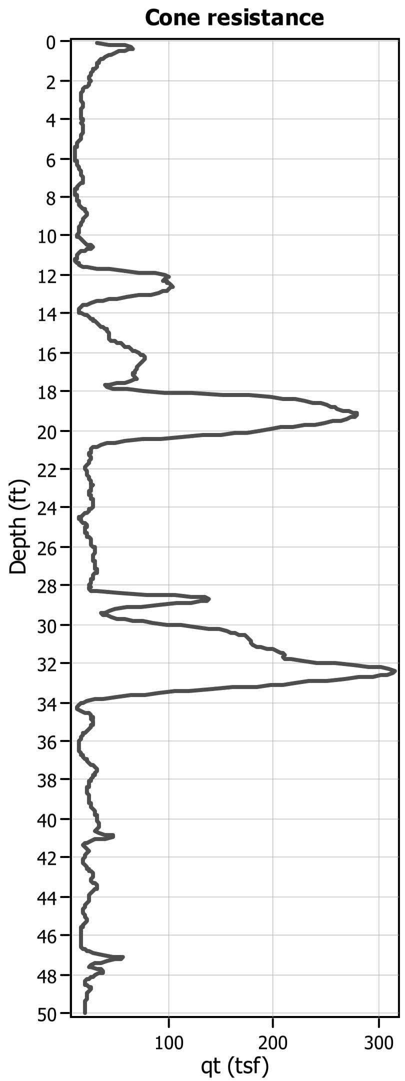


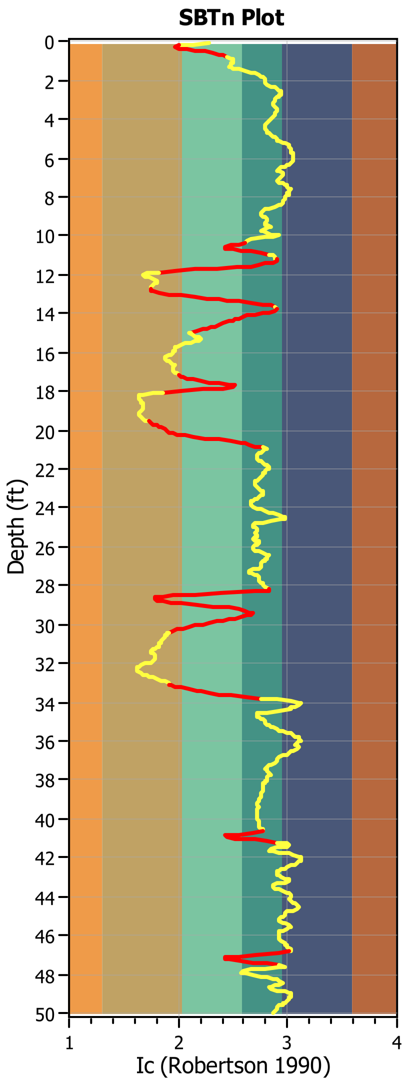


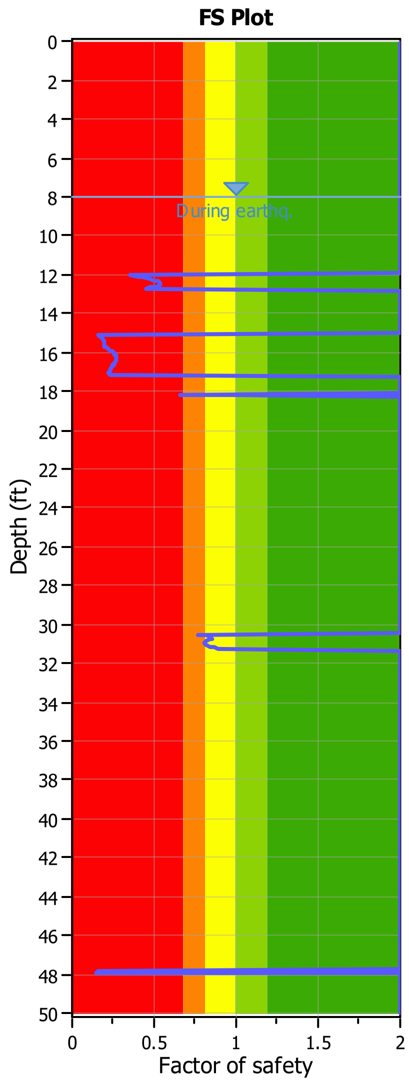
Input parameters and analysis data

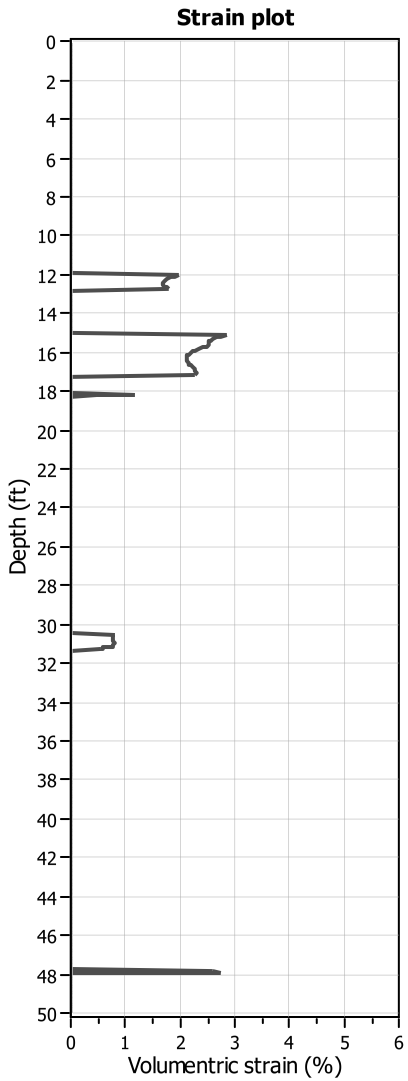
Analysis method:	NCEER (1998)	Depth to water table (erthq.):	8.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.84	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	8.00 ft	Fill height:	N/A	Limit depth:	N/A

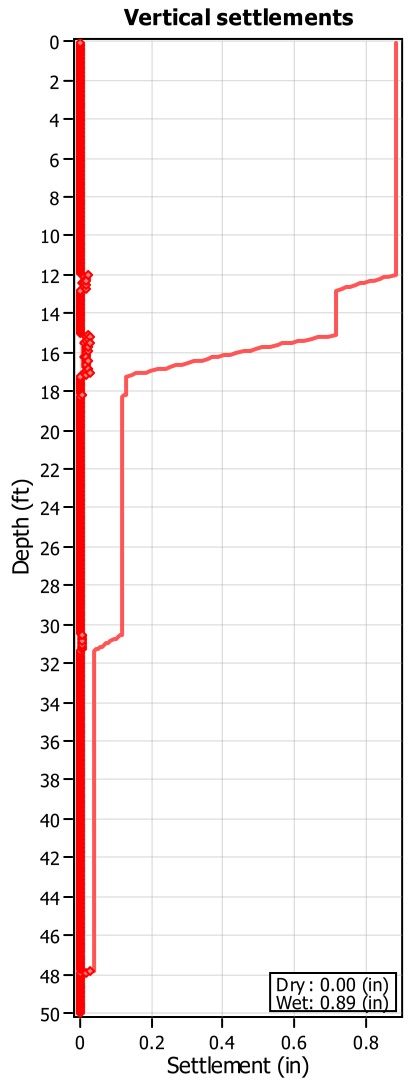
Estimation of post-earthquake settlements











Abbreviations

q_t:	Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c:	Soil Behaviour Type Index
FS:	Calculated Factor of Safety against liquefaction
Volumentric strain:	Post-liquefaction volumentric strain

:: Post-earthquake settlement due to soil liquefaction ::

Depth (ft)	$Q_{tn,cs}$	FS	e_v (%)	DF	Settlement (in)	Depth (ft)	$Q_{tn,cs}$	FS	e_v (%)	DF	Settlement (in)
8.03	163.81	2.00	0.00	1.00	0.00	8.08	168.15	2.00	0.00	1.00	0.00
8.17	171.02	2.00	0.00	1.00	0.00	8.21	173.26	2.00	0.00	1.00	0.00
8.27	174.08	2.00	0.00	1.00	0.00	8.35	174.46	2.00	0.00	1.00	0.00
8.41	175.20	2.00	0.00	1.00	0.00	8.46	176.20	2.00	0.00	1.00	0.00
8.55	176.03	2.00	0.00	1.00	0.00	8.62	175.41	2.00	0.00	1.00	0.00
8.67	176.60	2.00	0.00	1.00	0.00	8.76	179.50	2.00	0.00	1.00	0.00
8.79	182.54	2.00	0.00	1.00	0.00	8.86	182.83	2.00	0.00	1.00	0.00
8.94	181.61	2.00	0.00	1.00	0.00	9.01	179.64	2.00	0.00	1.00	0.00
9.06	176.67	2.00	0.00	1.00	0.00	9.13	173.81	2.00	0.00	1.00	0.00
9.20	168.24	2.00	0.00	1.00	0.00	9.27	161.40	2.00	0.00	1.00	0.00
9.33	152.86	2.00	0.00	1.00	0.00	9.39	148.17	2.00	0.00	1.00	0.00
9.47	148.24	2.00	0.00	1.00	0.00	9.52	151.49	2.00	0.00	1.00	0.00
9.59	144.66	2.00	0.00	1.00	0.00	9.66	133.51	2.00	0.00	1.00	0.00
9.72	125.48	2.00	0.00	1.00	0.00	9.78	127.25	2.00	0.00	1.00	0.00
9.88	131.44	2.00	0.00	1.00	0.00	9.92	131.74	2.00	0.00	1.00	0.00
9.98	135.11	2.00	0.00	1.00	0.00	10.04	132.53	2.00	0.00	1.00	0.00
10.11	127.34	2.00	0.00	1.00	0.00	10.20	118.54	2.00	0.00	1.00	0.00
10.25	117.56	2.00	0.00	1.00	0.00	10.32	119.99	2.00	0.00	1.00	0.00
10.38	123.85	2.00	0.00	1.00	0.00	10.44	121.09	2.00	0.00	1.00	0.00
10.52	112.95	2.00	0.00	1.00	0.00	10.59	104.77	2.00	0.00	1.00	0.00
10.64	101.77	2.00	0.00	1.00	0.00	10.70	104.56	2.00	0.00	1.00	0.00
10.78	110.65	2.00	0.00	1.00	0.00	10.83	117.78	2.00	0.00	1.00	0.00
10.90	123.59	2.00	0.00	1.00	0.00	10.96	128.33	2.00	0.00	1.00	0.00
11.03	129.83	2.00	0.00	1.00	0.00	11.10	128.69	2.00	0.00	1.00	0.00
11.16	125.98	2.00	0.00	1.00	0.00	11.22	125.25	2.00	0.00	1.00	0.00
11.31	125.77	2.00	0.00	1.00	0.00	11.36	124.70	2.00	0.00	1.00	0.00
11.45	122.29	2.00	0.00	1.00	0.00	11.51	120.39	2.00	0.00	1.00	0.00
11.56	117.58	2.00	0.00	1.00	0.00	11.63	109.77	2.00	0.00	1.00	0.00
11.69	97.13	2.00	0.00	1.00	0.00	11.75	91.69	2.00	0.00	1.00	0.00
11.82	100.48	2.00	0.00	1.00	0.00	11.90	111.42	2.00	0.00	1.00	0.00
11.95	113.96	2.00	0.00	1.00	0.00	12.04	123.09	0.36	1.97	1.00	0.02
12.09	129.34	0.40	1.89	1.00	0.01	12.14	134.10	0.43	1.84	1.00	0.01
12.22	140.47	0.48	1.77	1.00	0.02	12.28	144.17	0.50	1.73	1.00	0.01
12.37	147.19	0.53	1.70	1.00	0.02	12.41	148.70	0.54	1.69	1.00	0.01
12.47	149.05	0.54	1.68	1.00	0.01	12.56	148.14	0.53	1.69	1.00	0.02
12.62	145.44	0.51	1.72	1.00	0.01	12.67	141.93	0.48	1.75	1.00	0.01
12.73	138.24	0.45	1.79	1.00	0.01	12.81	135.58	2.00	0.00	1.00	0.00
12.87	135.38	2.00	0.00	1.00	0.00	12.95	135.88	2.00	0.00	1.00	0.00
13.00	132.34	2.00	0.00	1.00	0.00	13.07	125.68	2.00	0.00	1.00	0.00
13.13	121.92	2.00	0.00	1.00	0.00	13.20	124.51	2.00	0.00	1.00	0.00
13.28	131.95	2.00	0.00	1.00	0.00	13.34	139.67	2.00	0.00	1.00	0.00
13.40	149.09	2.00	0.00	1.00	0.00	13.46	160.23	2.00	0.00	1.00	0.00
13.53	170.03	2.00	0.00	1.00	0.00	13.59	167.91	2.00	0.00	1.00	0.00
13.68	159.91	2.00	0.00	1.00	0.00	13.72	150.18	2.00	0.00	1.00	0.00
13.79	144.42	2.00	0.00	1.00	0.00	13.86	139.49	2.00	0.00	1.00	0.00
13.93	135.34	2.00	0.00	1.00	0.00	13.98	132.38	2.00	0.00	1.00	0.00
14.06	129.46	2.00	0.00	1.00	0.00	14.12	126.47	2.00	0.00	1.00	0.00
14.17	122.66	2.00	0.00	1.00	0.00	14.26	119.40	2.00	0.00	1.00	0.00

:: Post-earthquake settlement due to soil liquefaction :: (continued)

Depth (ft)	$Q_{tn,cs}$	FS	e_v (%)	DF	Settlement (in)	Depth (ft)	$Q_{tn,cs}$	FS	e_v (%)	DF	Settlement (in)
14.32	115.68	2.00	0.00	1.00	0.00	14.40	112.35	2.00	0.00	1.00	0.00
14.44	108.02	2.00	0.00	1.00	0.00	14.50	102.99	2.00	0.00	1.00	0.00
14.59	98.22	2.00	0.00	1.00	0.00	14.65	94.40	2.00	0.00	1.00	0.00
14.70	91.68	2.00	0.00	1.00	0.00	14.79	89.97	2.00	0.00	1.00	0.00
14.83	87.93	2.00	0.00	1.00	0.00	14.90	82.12	2.00	0.00	1.00	0.00
14.98	77.92	2.00	0.00	1.00	0.00	15.04	75.85	2.00	0.00	1.00	0.00
15.10	78.62	0.16	2.85	1.00	0.02	15.18	81.65	0.17	2.76	1.00	0.03
15.24	85.14	0.18	2.67	1.00	0.02	15.32	87.99	0.19	2.60	1.00	0.02
15.37	90.08	0.19	2.55	1.00	0.02	15.43	90.84	0.19	2.53	1.00	0.02
15.52	91.21	0.19	2.52	1.00	0.03	15.55	91.39	0.19	2.52	1.00	0.01
15.62	91.68	0.19	2.51	1.00	0.02	15.70	93.10	0.20	2.48	1.00	0.02
15.76	95.71	0.21	2.42	1.00	0.02	15.82	99.06	0.22	2.35	1.00	0.02
15.88	102.70	0.23	2.29	1.00	0.02	15.97	105.76	0.24	2.23	1.00	0.02
16.03	108.38	0.25	2.19	1.00	0.02	16.09	110.34	0.26	2.16	1.00	0.02
16.15	111.77	0.27	2.13	1.00	0.01	16.23	112.42	0.27	2.12	1.00	0.02
16.28	112.41	0.27	2.12	1.00	0.01	16.34	112.07	0.27	2.13	1.00	0.02
16.43	111.78	0.27	2.13	1.00	0.02	16.48	111.52	0.26	2.14	1.00	0.01
16.54	110.91	0.26	2.15	1.00	0.02	16.61	109.74	0.26	2.17	1.00	0.02
16.68	107.77	0.25	2.20	1.00	0.02	16.73	105.75	0.24	2.23	1.00	0.02
16.81	104.21	0.23	2.26	1.00	0.02	16.88	103.47	0.23	2.27	1.00	0.02
16.93	102.93	0.23	2.28	1.00	0.01	17.03	102.23	0.22	2.29	1.00	0.03
17.07	101.96	0.22	2.30	1.00	0.01	17.13	103.31	0.23	2.28	1.00	0.02
17.22	105.68	2.00	0.00	1.00	0.00	17.27	109.23	2.00	0.00	1.00	0.00
17.32	114.36	2.00	0.00	1.00	0.00	17.41	119.69	2.00	0.00	1.00	0.00
17.47	124.89	2.00	0.00	1.00	0.00	17.54	129.38	2.00	0.00	1.00	0.00
17.60	135.51	2.00	0.00	1.00	0.00	17.65	142.65	2.00	0.00	1.00	0.00
17.72	148.70	2.00	0.00	1.00	0.00	17.79	149.65	2.00	0.00	1.00	0.00
17.86	139.15	2.00	0.00	1.00	0.00	17.92	119.32	2.00	0.00	1.00	0.00
18.00	117.81	2.00	0.00	1.00	0.00	18.05	133.70	2.00	0.00	1.00	0.00
18.12	152.08	2.00	0.00	1.00	0.00	18.18	170.14	0.66	1.16	1.00	0.01
18.24	197.33	0.97	0.47	1.00	0.00	18.34	219.52	2.00	0.00	1.00	0.00
18.40	233.92	2.00	0.00	1.00	0.00	18.45	246.69	2.00	0.00	1.00	0.00
18.52	256.46	2.00	0.00	1.00	0.00	18.58	265.11	2.00	0.00	1.00	0.00
18.64	272.70	2.00	0.00	1.00	0.00	18.73	278.55	2.00	0.00	1.00	0.00
18.78	283.24	2.00	0.00	1.00	0.00	18.84	286.06	2.00	0.00	1.00	0.00
18.91	289.15	2.00	0.00	1.00	0.00	18.98	293.06	2.00	0.00	1.00	0.00
19.03	298.77	2.00	0.00	1.00	0.00	19.13	303.32	2.00	0.00	1.00	0.00
19.17	306.51	2.00	0.00	1.00	0.00	19.26	305.81	2.00	0.00	1.00	0.00
19.31	304.34	2.00	0.00	1.00	0.00	19.37	300.57	2.00	0.00	1.00	0.00
19.47	295.45	2.00	0.00	1.00	0.00	19.50	289.06	2.00	0.00	1.00	0.00
19.56	291.75	2.00	0.00	1.00	0.00	19.64	293.73	2.00	0.00	1.00	0.00
19.71	292.26	2.00	0.00	1.00	0.00	19.76	288.27	2.00	0.00	1.00	0.00
19.83	281.71	2.00	0.00	1.00	0.00	19.89	273.06	2.00	0.00	1.00	0.00
19.95	262.85	2.00	0.00	1.00	0.00	20.01	251.22	2.00	0.00	1.00	0.00
20.10	239.37	2.00	0.00	1.00	0.00	20.15	226.74	2.00	0.00	1.00	0.00
20.23	213.79	2.00	0.00	1.00	0.00	20.28	198.72	2.00	0.00	1.00	0.00
20.34	185.24	2.00	0.00	1.00	0.00	20.44	178.48	2.00	0.00	1.00	0.00
20.49	182.11	2.00	0.00	1.00	0.00	20.54	187.32	2.00	0.00	1.00	0.00

:: Post-earthquake settlement due to soil liquefaction :: (continued)

Depth (ft)	$Q_{tn,cs}$	FS	e_v (%)	DF	Settlement (in)	Depth (ft)	$Q_{tn,cs}$	FS	e_v (%)	DF	Settlement (in)
20.62	184.98	2.00	0.00	1.00	0.00	20.68	177.19	2.00	0.00	1.00	0.00
20.74	171.50	2.00	0.00	1.00	0.00	20.83	172.27	2.00	0.00	1.00	0.00
20.88	172.48	2.00	0.00	1.00	0.00	20.95	170.46	2.00	0.00	1.00	0.00
21.00	162.36	2.00	0.00	1.00	0.00	21.08	153.23	2.00	0.00	1.00	0.00
21.13	143.27	2.00	0.00	1.00	0.00	21.22	137.83	2.00	0.00	1.00	0.00
21.28	134.16	2.00	0.00	1.00	0.00	21.36	133.69	2.00	0.00	1.00	0.00
21.41	134.06	2.00	0.00	1.00	0.00	21.47	134.46	2.00	0.00	1.00	0.00
21.55	135.12	2.00	0.00	1.00	0.00	21.60	136.33	2.00	0.00	1.00	0.00
21.66	137.78	2.00	0.00	1.00	0.00	21.75	138.83	2.00	0.00	1.00	0.00
21.80	138.67	2.00	0.00	1.00	0.00	21.86	137.26	2.00	0.00	1.00	0.00
21.93	135.35	2.00	0.00	1.00	0.00	22.01	133.60	2.00	0.00	1.00	0.00
22.05	131.67	2.00	0.00	1.00	0.00	22.13	130.47	2.00	0.00	1.00	0.00
22.19	130.22	2.00	0.00	1.00	0.00	22.25	130.93	2.00	0.00	1.00	0.00
22.35	131.27	2.00	0.00	1.00	0.00	22.38	131.26	2.00	0.00	1.00	0.00
22.44	131.10	2.00	0.00	1.00	0.00	22.52	130.70	2.00	0.00	1.00	0.00
22.59	130.37	2.00	0.00	1.00	0.00	22.64	130.74	2.00	0.00	1.00	0.00
22.73	131.76	2.00	0.00	1.00	0.00	22.78	133.20	2.00	0.00	1.00	0.00
22.86	133.95	2.00	0.00	1.00	0.00	22.92	134.92	2.00	0.00	1.00	0.00
22.98	136.44	2.00	0.00	1.00	0.00	23.06	138.38	2.00	0.00	1.00	0.00
23.13	140.09	2.00	0.00	1.00	0.00	23.17	140.43	2.00	0.00	1.00	0.00
23.24	140.22	2.00	0.00	1.00	0.00	23.32	139.39	2.00	0.00	1.00	0.00
23.37	138.36	2.00	0.00	1.00	0.00	23.44	136.57	2.00	0.00	1.00	0.00
23.51	134.52	2.00	0.00	1.00	0.00	23.56	131.58	2.00	0.00	1.00	0.00
23.63	127.41	2.00	0.00	1.00	0.00	23.69	124.40	2.00	0.00	1.00	0.00
23.75	123.07	2.00	0.00	1.00	0.00	23.84	123.93	2.00	0.00	1.00	0.00
23.90	124.58	2.00	0.00	1.00	0.00	23.96	126.51	2.00	0.00	1.00	0.00
24.02	129.10	2.00	0.00	1.00	0.00	24.09	131.80	2.00	0.00	1.00	0.00
24.16	133.28	2.00	0.00	1.00	0.00	24.22	133.49	2.00	0.00	1.00	0.00
24.28	132.21	2.00	0.00	1.00	0.00	24.35	130.20	2.00	0.00	1.00	0.00
24.42	128.29	2.00	0.00	1.00	0.00	24.49	125.85	2.00	0.00	1.00	0.00
24.54	121.66	2.00	0.00	1.00	0.00	24.61	114.42	2.00	0.00	1.00	0.00
24.67	106.67	2.00	0.00	1.00	0.00	24.75	100.10	2.00	0.00	1.00	0.00
24.81	95.74	2.00	0.00	1.00	0.00	24.88	94.55	2.00	0.00	1.00	0.00
24.95	94.33	2.00	0.00	1.00	0.00	25.00	94.86	2.00	0.00	1.00	0.00
25.07	94.94	2.00	0.00	1.00	0.00	25.14	94.92	2.00	0.00	1.00	0.00
25.20	93.69	2.00	0.00	1.00	0.00	25.29	93.50	2.00	0.00	1.00	0.00
25.34	95.29	2.00	0.00	1.00	0.00	25.41	100.64	2.00	0.00	1.00	0.00
25.49	106.13	2.00	0.00	1.00	0.00	25.54	110.51	2.00	0.00	1.00	0.00
25.61	114.89	2.00	0.00	1.00	0.00	25.67	118.65	2.00	0.00	1.00	0.00
25.72	122.31	2.00	0.00	1.00	0.00	25.82	123.04	2.00	0.00	1.00	0.00
25.87	121.67	2.00	0.00	1.00	0.00	25.92	121.48	2.00	0.00	1.00	0.00
26.00	123.87	2.00	0.00	1.00	0.00	26.07	127.76	2.00	0.00	1.00	0.00
26.14	131.18	2.00	0.00	1.00	0.00	26.20	136.03	2.00	0.00	1.00	0.00
26.26	143.62	2.00	0.00	1.00	0.00	26.32	152.36	2.00	0.00	1.00	0.00
26.38	158.56	2.00	0.00	1.00	0.00	26.46	159.82	2.00	0.00	1.00	0.00
26.54	158.24	2.00	0.00	1.00	0.00	26.60	156.48	2.00	0.00	1.00	0.00
26.64	155.35	2.00	0.00	1.00	0.00	26.72	153.48	2.00	0.00	1.00	0.00
26.78	150.70	2.00	0.00	1.00	0.00	26.84	148.24	2.00	0.00	1.00	0.00

:: Post-earthquake settlement due to soil liquefaction :: (continued)

Depth (ft)	$Q_{tn,cs}$	FS	e_v (%)	DF	Settlement (in)	Depth (ft)	$Q_{tn,cs}$	FS	e_v (%)	DF	Settlement (in)
26.93	147.40	2.00	0.00	1.00	0.00	26.99	146.85	2.00	0.00	1.00	0.00
27.05	144.71	2.00	0.00	1.00	0.00	27.11	139.21	2.00	0.00	1.00	0.00
27.18	131.68	2.00	0.00	1.00	0.00	27.23	124.48	2.00	0.00	1.00	0.00
27.33	120.52	2.00	0.00	1.00	0.00	27.37	119.95	2.00	0.00	1.00	0.00
27.44	122.41	2.00	0.00	1.00	0.00	27.51	124.51	2.00	0.00	1.00	0.00
27.57	125.29	2.00	0.00	1.00	0.00	27.63	124.29	2.00	0.00	1.00	0.00
27.70	123.96	2.00	0.00	1.00	0.00	27.77	124.66	2.00	0.00	1.00	0.00
27.85	126.18	2.00	0.00	1.00	0.00	27.91	126.77	2.00	0.00	1.00	0.00
27.96	127.49	2.00	0.00	1.00	0.00	28.05	129.07	2.00	0.00	1.00	0.00
28.10	132.22	2.00	0.00	1.00	0.00	28.15	138.84	2.00	0.00	1.00	0.00
28.25	143.58	2.00	0.00	1.00	0.00	28.30	140.15	2.00	0.00	1.00	0.00
28.36	115.78	2.00	0.00	1.00	0.00	28.45	109.99	2.00	0.00	1.00	0.00
28.49	125.65	2.00	0.00	1.00	0.00	28.55	137.72	2.00	0.00	1.00	0.00
28.64	143.68	2.00	0.00	1.00	0.00	28.69	146.91	2.00	0.00	1.00	0.00
28.75	148.45	2.00	0.00	1.00	0.00	28.81	147.24	2.00	0.00	1.00	0.00
28.89	141.53	2.00	0.00	1.00	0.00	28.94	133.39	2.00	0.00	1.00	0.00
29.02	129.45	2.00	0.00	1.00	0.00	29.07	136.64	2.00	0.00	1.00	0.00
29.14	146.63	2.00	0.00	1.00	0.00	29.22	151.40	2.00	0.00	1.00	0.00
29.28	147.26	2.00	0.00	1.00	0.00	29.37	143.06	2.00	0.00	1.00	0.00
29.40	143.05	2.00	0.00	1.00	0.00	29.47	144.49	2.00	0.00	1.00	0.00
29.55	144.39	2.00	0.00	1.00	0.00	29.62	142.76	2.00	0.00	1.00	0.00
29.67	141.60	2.00	0.00	1.00	0.00	29.74	142.30	2.00	0.00	1.00	0.00
29.81	143.45	2.00	0.00	1.00	0.00	29.86	145.10	2.00	0.00	1.00	0.00
29.92	146.64	2.00	0.00	1.00	0.00	29.99	149.96	2.00	0.00	1.00	0.00
30.06	155.09	2.00	0.00	1.00	0.00	30.13	161.70	2.00	0.00	1.00	0.00
30.19	168.74	2.00	0.00	1.00	0.00	30.26	174.75	2.00	0.00	1.00	0.00
30.35	179.44	2.00	0.00	1.00	0.00	30.40	182.03	2.00	0.00	1.00	0.00
30.45	184.74	2.00	0.00	1.00	0.00	30.52	187.91	0.77	0.81	1.00	0.01
30.59	191.57	0.81	0.79	1.00	0.01	30.65	193.86	0.84	0.77	1.00	0.01
30.72	194.51	0.85	0.77	1.00	0.01	30.79	193.66	0.84	0.77	1.00	0.01
30.86	192.25	0.82	0.78	1.00	0.01	30.92	191.11	0.81	0.79	1.00	0.01
30.98	190.69	0.80	0.79	1.00	0.01	31.05	191.65	0.81	0.79	1.00	0.01
31.13	193.68	0.84	0.77	1.00	0.01	31.17	196.48	0.87	0.58	1.00	0.00
31.26	198.45	0.89	0.57	1.00	0.01	31.32	199.00	0.90	0.57	1.00	0.00
31.37	200.73	2.00	0.00	1.00	0.00	31.43	204.29	2.00	0.00	1.00	0.00
31.51	209.60	2.00	0.00	1.00	0.00	31.58	212.73	2.00	0.00	1.00	0.00
31.64	213.83	2.00	0.00	1.00	0.00	31.72	213.82	2.00	0.00	1.00	0.00
31.76	215.06	2.00	0.00	1.00	0.00	31.83	220.09	2.00	0.00	1.00	0.00
31.92	226.65	2.00	0.00	1.00	0.00	31.98	231.81	2.00	0.00	1.00	0.00
32.03	231.28	2.00	0.00	1.00	0.00	32.11	246.37	2.00	0.00	1.00	0.00
32.16	260.10	2.00	0.00	1.00	0.00	32.22	270.01	2.00	0.00	1.00	0.00
32.28	277.28	2.00	0.00	1.00	0.00	32.35	281.06	2.00	0.00	1.00	0.00
32.42	281.52	2.00	0.00	1.00	0.00	32.48	279.17	2.00	0.00	1.00	0.00
32.56	274.40	2.00	0.00	1.00	0.00	32.63	269.85	2.00	0.00	1.00	0.00
32.70	275.41	2.00	0.00	1.00	0.00	32.75	276.28	2.00	0.00	1.00	0.00
32.85	272.34	2.00	0.00	1.00	0.00	32.88	262.92	2.00	0.00	1.00	0.00
32.95	251.06	2.00	0.00	1.00	0.00	33.02	237.92	2.00	0.00	1.00	0.00
33.08	226.93	2.00	0.00	1.00	0.00	33.14	215.49	2.00	0.00	1.00	0.00

:: Post-earthquake settlement due to soil liquefaction :: (continued)

Depth (ft)	$Q_{tn,cs}$	FS	e_v (%)	DF	Settlement (in)	Depth (ft)	$Q_{tn,cs}$	FS	e_v (%)	DF	Settlement (in)
33.22	203.35	2.00	0.00	1.00	0.00	33.27	185.02	2.00	0.00	1.00	0.00
33.34	170.17	2.00	0.00	1.00	0.00	33.40	156.91	2.00	0.00	1.00	0.00
33.48	147.74	2.00	0.00	1.00	0.00	33.55	141.94	2.00	0.00	1.00	0.00
33.60	141.09	2.00	0.00	1.00	0.00	33.66	146.13	2.00	0.00	1.00	0.00
33.75	155.14	2.00	0.00	1.00	0.00	33.81	163.80	2.00	0.00	1.00	0.00
33.87	162.90	2.00	0.00	1.00	0.00	33.97	155.70	2.00	0.00	1.00	0.00
34.00	144.88	2.00	0.00	1.00	0.00	34.06	130.19	2.00	0.00	1.00	0.00
34.15	111.04	2.00	0.00	1.00	0.00	34.21	92.89	2.00	0.00	1.00	0.00
34.26	84.63	2.00	0.00	1.00	0.00	34.36	82.29	2.00	0.00	1.00	0.00
34.38	83.54	2.00	0.00	1.00	0.00	34.45	85.73	2.00	0.00	1.00	0.00
34.52	88.68	2.00	0.00	1.00	0.00	34.60	91.40	2.00	0.00	1.00	0.00
34.66	95.64	2.00	0.00	1.00	0.00	34.72	101.33	2.00	0.00	1.00	0.00
34.78	106.89	2.00	0.00	1.00	0.00	34.84	111.74	2.00	0.00	1.00	0.00
34.91	114.46	2.00	0.00	1.00	0.00	34.97	116.12	2.00	0.00	1.00	0.00
35.06	118.10	2.00	0.00	1.00	0.00	35.13	120.83	2.00	0.00	1.00	0.00
35.20	123.93	2.00	0.00	1.00	0.00	35.24	126.12	2.00	0.00	1.00	0.00
35.31	126.75	2.00	0.00	1.00	0.00	35.39	127.01	2.00	0.00	1.00	0.00
35.44	127.14	2.00	0.00	1.00	0.00	35.51	125.93	2.00	0.00	1.00	0.00
35.58	124.59	2.00	0.00	1.00	0.00	35.64	122.37	2.00	0.00	1.00	0.00
35.70	119.71	2.00	0.00	1.00	0.00	35.76	117.07	2.00	0.00	1.00	0.00
35.83	115.27	2.00	0.00	1.00	0.00	35.91	114.14	2.00	0.00	1.00	0.00
35.96	112.49	2.00	0.00	1.00	0.00	36.02	108.48	2.00	0.00	1.00	0.00
36.10	103.96	2.00	0.00	1.00	0.00	36.17	100.54	2.00	0.00	1.00	0.00
36.25	99.07	2.00	0.00	1.00	0.00	36.31	97.75	2.00	0.00	1.00	0.00
36.37	95.95	2.00	0.00	1.00	0.00	36.44	94.40	2.00	0.00	1.00	0.00
36.49	93.42	2.00	0.00	1.00	0.00	36.56	93.49	2.00	0.00	1.00	0.00
36.65	94.12	2.00	0.00	1.00	0.00	36.68	95.94	2.00	0.00	1.00	0.00
36.75	97.68	2.00	0.00	1.00	0.00	36.83	100.92	2.00	0.00	1.00	0.00
36.89	103.66	2.00	0.00	1.00	0.00	36.95	108.57	2.00	0.00	1.00	0.00
37.05	113.12	2.00	0.00	1.00	0.00	37.08	118.18	2.00	0.00	1.00	0.00
37.15	122.88	2.00	0.00	1.00	0.00	37.23	126.72	2.00	0.00	1.00	0.00
37.29	128.21	2.00	0.00	1.00	0.00	37.34	130.33	2.00	0.00	1.00	0.00
37.42	132.90	2.00	0.00	1.00	0.00	37.48	136.42	2.00	0.00	1.00	0.00
37.57	137.54	2.00	0.00	1.00	0.00	37.61	137.21	2.00	0.00	1.00	0.00
37.68	135.69	2.00	0.00	1.00	0.00	37.73	131.98	2.00	0.00	1.00	0.00
37.80	126.16	2.00	0.00	1.00	0.00	37.86	118.03	2.00	0.00	1.00	0.00
37.96	110.51	2.00	0.00	1.00	0.00	38.02	104.88	2.00	0.00	1.00	0.00
38.07	101.10	2.00	0.00	1.00	0.00	38.13	97.25	2.00	0.00	1.00	0.00
38.20	93.44	2.00	0.00	1.00	0.00	38.26	90.52	2.00	0.00	1.00	0.00
38.32	88.88	2.00	0.00	1.00	0.00	38.41	88.15	2.00	0.00	1.00	0.00
38.46	87.75	2.00	0.00	1.00	0.00	38.52	87.52	2.00	0.00	1.00	0.00
38.59	87.19	2.00	0.00	1.00	0.00	38.65	86.59	2.00	0.00	1.00	0.00
38.74	85.96	2.00	0.00	1.00	0.00	38.80	85.53	2.00	0.00	1.00	0.00
38.85	85.72	2.00	0.00	1.00	0.00	38.92	85.61	2.00	0.00	1.00	0.00
38.98	85.26	2.00	0.00	1.00	0.00	39.04	85.28	2.00	0.00	1.00	0.00
39.12	85.69	2.00	0.00	1.00	0.00	39.18	86.53	2.00	0.00	1.00	0.00
39.24	87.30	2.00	0.00	1.00	0.00	39.31	89.23	2.00	0.00	1.00	0.00
39.38	91.75	2.00	0.00	1.00	0.00	39.44	94.64	2.00	0.00	1.00	0.00

:: Post-earthquake settlement due to soil liquefaction :: (continued)

Depth (ft)	$Q_{tn,cs}$	FS	e_v (%)	DF	Settlement (in)	Depth (ft)	$Q_{tn,cs}$	FS	e_v (%)	DF	Settlement (in)
39.51	96.85	2.00	0.00	1.00	0.00	39.57	98.86	2.00	0.00	1.00	0.00
39.64	100.77	2.00	0.00	1.00	0.00	39.70	103.20	2.00	0.00	1.00	0.00
39.77	106.06	2.00	0.00	1.00	0.00	39.85	108.47	2.00	0.00	1.00	0.00
39.91	109.73	2.00	0.00	1.00	0.00	39.97	110.16	2.00	0.00	1.00	0.00
40.03	110.99	2.00	0.00	1.00	0.00	40.10	112.64	2.00	0.00	1.00	0.00
40.16	114.73	2.00	0.00	1.00	0.00	40.23	116.66	2.00	0.00	1.00	0.00
40.30	118.17	2.00	0.00	1.00	0.00	40.35	120.32	2.00	0.00	1.00	0.00
40.44	122.23	2.00	0.00	1.00	0.00	40.50	120.78	2.00	0.00	1.00	0.00
40.58	118.17	2.00	0.00	1.00	0.00	40.63	115.80	2.00	0.00	1.00	0.00
40.69	114.72	2.00	0.00	1.00	0.00	40.77	107.55	2.00	0.00	1.00	0.00
40.84	96.66	2.00	0.00	1.00	0.00	40.89	87.92	2.00	0.00	1.00	0.00
40.95	86.93	2.00	0.00	1.00	0.00	41.03	90.28	2.00	0.00	1.00	0.00
41.08	98.20	2.00	0.00	1.00	0.00	41.14	102.89	2.00	0.00	1.00	0.00
41.23	105.30	2.00	0.00	1.00	0.00	41.29	105.17	2.00	0.00	1.00	0.00
41.35	104.90	2.00	0.00	1.00	0.00	41.41	102.38	2.00	0.00	1.00	0.00
41.47	99.27	2.00	0.00	1.00	0.00	41.56	96.73	2.00	0.00	1.00	0.00
41.62	98.26	2.00	0.00	1.00	0.00	41.68	104.96	2.00	0.00	1.00	0.00
41.77	111.96	2.00	0.00	1.00	0.00	41.80	119.00	2.00	0.00	1.00	0.00
41.87	125.07	2.00	0.00	1.00	0.00	41.94	129.71	2.00	0.00	1.00	0.00
42.00	130.27	2.00	0.00	1.00	0.00	42.07	128.28	2.00	0.00	1.00	0.00
42.13	124.72	2.00	0.00	1.00	0.00	42.21	121.69	2.00	0.00	1.00	0.00
42.26	118.74	2.00	0.00	1.00	0.00	42.34	116.35	2.00	0.00	1.00	0.00
42.40	114.26	2.00	0.00	1.00	0.00	42.45	113.70	2.00	0.00	1.00	0.00
42.55	114.44	2.00	0.00	1.00	0.00	42.60	117.41	2.00	0.00	1.00	0.00
42.69	119.44	2.00	0.00	1.00	0.00	42.74	121.82	2.00	0.00	1.00	0.00
42.79	126.55	2.00	0.00	1.00	0.00	42.89	131.08	2.00	0.00	1.00	0.00
42.92	136.24	2.00	0.00	1.00	0.00	42.99	139.03	2.00	0.00	1.00	0.00
43.06	139.71	2.00	0.00	1.00	0.00	43.13	141.27	2.00	0.00	1.00	0.00
43.18	142.28	2.00	0.00	1.00	0.00	43.27	141.96	2.00	0.00	1.00	0.00
43.32	142.20	2.00	0.00	1.00	0.00	43.38	140.40	2.00	0.00	1.00	0.00
43.45	140.28	2.00	0.00	1.00	0.00	43.51	141.61	2.00	0.00	1.00	0.00
43.57	141.75	2.00	0.00	1.00	0.00	43.64	142.64	2.00	0.00	1.00	0.00
43.72	140.80	2.00	0.00	1.00	0.00	43.78	142.02	2.00	0.00	1.00	0.00
43.84	141.00	2.00	0.00	1.00	0.00	43.91	139.37	2.00	0.00	1.00	0.00
43.99	137.26	2.00	0.00	1.00	0.00	44.03	134.70	2.00	0.00	1.00	0.00
44.12	132.15	2.00	0.00	1.00	0.00	44.18	130.53	2.00	0.00	1.00	0.00
44.23	130.54	2.00	0.00	1.00	0.00	44.30	130.33	2.00	0.00	1.00	0.00
44.36	128.93	2.00	0.00	1.00	0.00	44.43	126.00	2.00	0.00	1.00	0.00
44.51	122.65	2.00	0.00	1.00	0.00	44.57	118.37	2.00	0.00	1.00	0.00
44.62	111.42	2.00	0.00	1.00	0.00	44.70	103.70	2.00	0.00	1.00	0.00
44.75	96.03	2.00	0.00	1.00	0.00	44.82	92.51	2.00	0.00	1.00	0.00
44.91	91.42	2.00	0.00	1.00	0.00	44.97	92.58	2.00	0.00	1.00	0.00
45.02	94.08	2.00	0.00	1.00	0.00	45.10	96.47	2.00	0.00	1.00	0.00
45.16	97.86	2.00	0.00	1.00	0.00	45.21	98.35	2.00	0.00	1.00	0.00
45.30	97.81	2.00	0.00	1.00	0.00	45.35	95.31	2.00	0.00	1.00	0.00
45.42	93.91	2.00	0.00	1.00	0.00	45.49	92.27	2.00	0.00	1.00	0.00
45.54	88.94	2.00	0.00	1.00	0.00	45.61	83.83	2.00	0.00	1.00	0.00
45.69	78.31	2.00	0.00	1.00	0.00	45.74	72.62	2.00	0.00	1.00	0.00

:: Post-earthquake settlement due to soil liquefaction :: (continued)

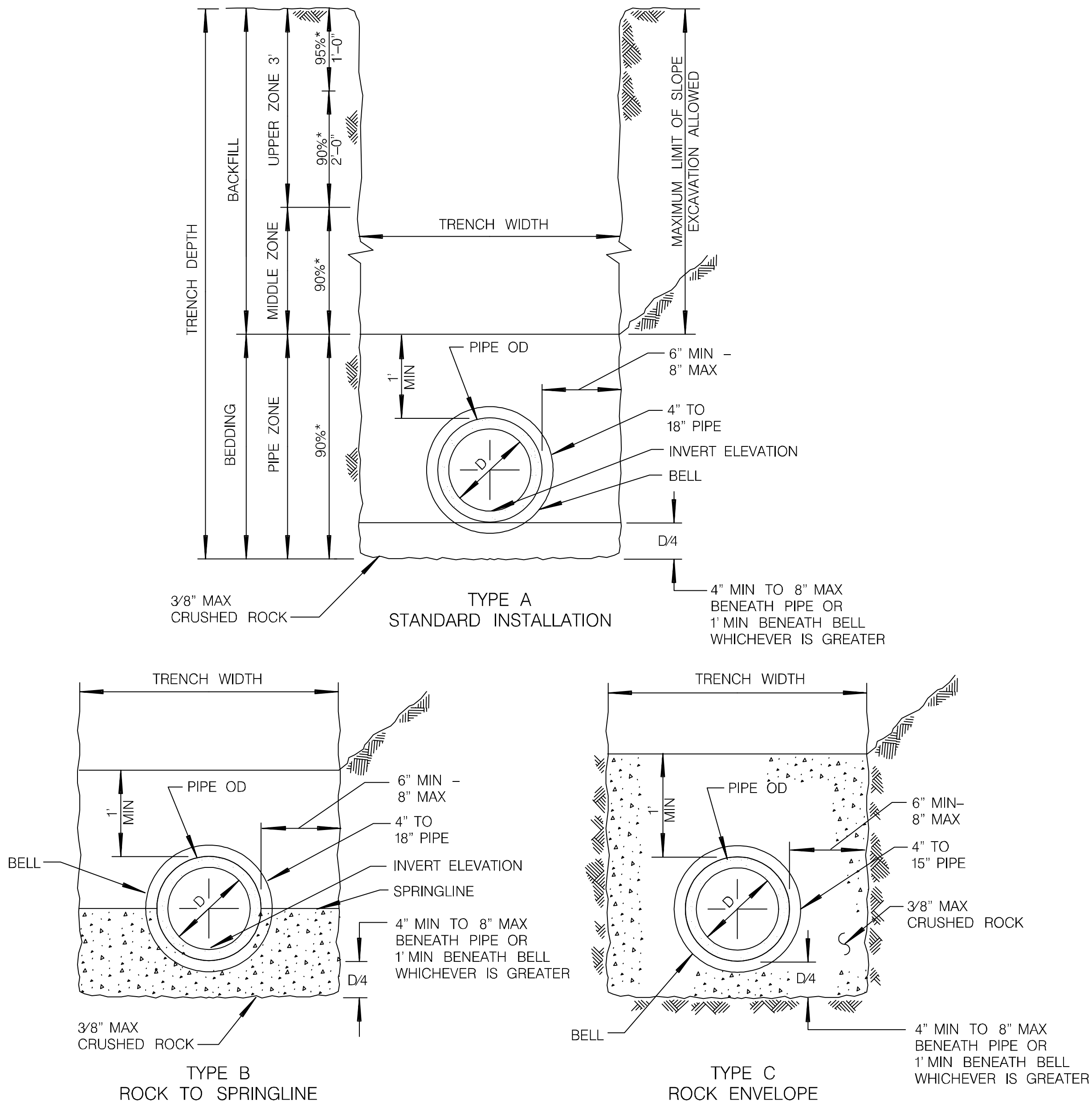
Depth (ft)	$Q_{tn,cs}$	FS	e_v (%)	DF	Settlement (in)	Depth (ft)	$Q_{tn,cs}$	FS	e_v (%)	DF	Settlement (in)
45.83	69.83	2.00	0.00	1.00	0.00	45.88	67.56	2.00	0.00	1.00	0.00
45.94	67.35	2.00	0.00	1.00	0.00	46.01	67.27	2.00	0.00	1.00	0.00
46.07	67.19	2.00	0.00	1.00	0.00	46.16	67.43	2.00	0.00	1.00	0.00
46.21	68.04	2.00	0.00	1.00	0.00	46.27	69.29	2.00	0.00	1.00	0.00
46.36	70.09	2.00	0.00	1.00	0.00	46.41	71.23	2.00	0.00	1.00	0.00
46.47	72.02	2.00	0.00	1.00	0.00	46.53	74.02	2.00	0.00	1.00	0.00
46.61	78.91	2.00	0.00	1.00	0.00	46.66	91.30	2.00	0.00	1.00	0.00
46.76	102.81	2.00	0.00	1.00	0.00	46.80	113.78	2.00	0.00	1.00	0.00
46.86	119.27	2.00	0.00	1.00	0.00	46.93	123.15	2.00	0.00	1.00	0.00
46.99	114.85	2.00	0.00	1.00	0.00	47.08	104.10	2.00	0.00	1.00	0.00
47.13	94.73	2.00	0.00	1.00	0.00	47.18	90.21	2.00	0.00	1.00	0.00
47.26	87.96	2.00	0.00	1.00	0.00	47.32	92.23	2.00	0.00	1.00	0.00
47.42	99.65	2.00	0.00	1.00	0.00	47.47	107.47	2.00	0.00	1.00	0.00
47.52	113.34	2.00	0.00	1.00	0.00	47.59	115.53	2.00	0.00	1.00	0.00
47.65	116.14	2.00	0.00	1.00	0.00	47.71	108.48	2.00	0.00	1.00	0.00
47.78	96.72	2.00	0.00	1.00	0.00	47.86	87.15	0.16	2.62	1.00	0.03
47.91	81.69	0.15	2.76	1.00	0.02	47.99	81.52	2.00	0.00	1.00	0.00
48.05	83.92	2.00	0.00	1.00	0.00	48.10	86.86	2.00	0.00	1.00	0.00
48.17	88.85	2.00	0.00	1.00	0.00	48.24	89.92	2.00	0.00	1.00	0.00
48.30	89.57	2.00	0.00	1.00	0.00	48.37	89.64	2.00	0.00	1.00	0.00
48.43	88.33	2.00	0.00	1.00	0.00	48.51	89.48	2.00	0.00	1.00	0.00
48.57	89.71	2.00	0.00	1.00	0.00	48.64	90.55	2.00	0.00	1.00	0.00
48.69	93.03	2.00	0.00	1.00	0.00	48.76	97.46	2.00	0.00	1.00	0.00
48.83	102.61	2.00	0.00	1.00	0.00	48.91	105.47	2.00	0.00	1.00	0.00
48.97	107.91	2.00	0.00	1.00	0.00	49.02	108.83	2.00	0.00	1.00	0.00
49.11	108.40	2.00	0.00	1.00	0.00	49.16	106.94	2.00	0.00	1.00	0.00
49.22	104.65	2.00	0.00	1.00	0.00	49.29	102.53	2.00	0.00	1.00	0.00
49.34	97.59	2.00	0.00	1.00	0.00	49.45	92.85	2.00	0.00	1.00	0.00
49.48	87.84	2.00	0.00	1.00	0.00	49.56	86.26	2.00	0.00	1.00	0.00
49.61	84.55	2.00	0.00	1.00	0.00	49.68	81.81	2.00	0.00	1.00	0.00
49.74	79.11	2.00	0.00	1.00	0.00	49.81	78.55	2.00	0.00	1.00	0.00
49.87	78.25	2.00	0.00	1.00	0.00	49.94	78.23	2.00	0.00	1.00	0.00
50.00	78.42	2.00	0.00	1.00	0.00						

Total estimated settlement: 0.89

Abbreviations

$Q_{tn,cs}$: Equivalent clean sand normalized cone resistance
FS: Factor of safety against liquefaction
e_v (%): Post-liquefaction volumentric strain
DF: e_v depth weighting factor
Settlement: Calculated settlement

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.: LE25036**	**Pipe Bedding and Trench Backfill Recommendations**	**Plate E-1**

SOUTHLAND GEOTECHNICAL

FOUNDATION ENGINEERS AND MATERIALS LABS

GEOTECHNICAL INVESTIGATION
WATER & WASTEWATER TREATMENT PLANT EXPANSION
IMPERIAL, CALIFORNIA

242 NORTH 8TH STREET • EL CENTRO, CALIFORNIA 92243 • (619) 352-1242
79-607 COUNTRY CLUB DRIVE, SUITE 5 • BERMUDA DUNES, CALIFORNIA 92201 • (619) 360-0665
2211 EAST PALO VERDE STREET • YUMA, ARIZONA 85365 • (602) 344-8844

GEOTECHNICAL INVESTIGATION
WATER & WASTEWATER TREATMENT PLANT EXPANSION
IMPERIAL, CALIFORNIA

*   *   *

Prepared for:

Black & Veatch
1400 South Potomac Street, Suite 200
Aurora, Colorado 80012

*   *   *

Prepared by:

Southland Geotechnical
242 North 8th Street
El Centro, California, 92243

Report No. S93211
November 1993

OUTHLAND
EOTECHNICAL

FOUNDATION ENGINEERS AND MATERIALS LABS

November 12, 1993

Black & Veatch
1400 South Potomac Street, Suite 200
Aurora, Colorado 80012
Attn: Michael Johnson, PE

Geotechnical Investigation
Water & Wastewater Treatment Plant Expansion
Imperial, California
Report No. S93211

Dear Mr. Johnson:

We are pleased to present this geotechnical report for the proposed expansion to the water and wastewater treatment plants for the City of Imperial, California. Our geotechnical investigation was conducted according to your request for our services. The enclosed report provides a description of the investigation performed and our recommendations for geotechnical design and construction of the project.

The subsurface soils below these sites were found to be prone to liquefaction during earthquakes. Proposed process units at the wastewater treatment plants may be supported on conventional concrete base slabs acting as partially to fully compensating mat foundations for structures with small settlements. Reccomendations for foundation designs, earthwork and pavement reconstruction are contained within the text of this report.

We appreciate the opportunity to provide our professional services and would encourage any questions or comments regarding our findings.

Respectfully Submitted,
SOUTHLAND GEOTECHNICAL

Shelton L. Stringer

Shelton L. Stringer, P.E.
Senior Engineer

Jeffrey O. Lyon

Jeffrey O. Lyon, P.E.
Principal Engineer

SLS/kmyw

REGISTERED PROFESSIONAL ENGINEER
Shelton L. Stringer
No. 48696
EXP. 6/30/96
CIVIL
STATE OF CALIFORNIA

REGISTERED PROFESSIONAL ENGINEER
JEFFREY O. LYON
No. 31921
Exp. 12-31-96
CIVIL
STATE OF CALIFORNIA

242 NORTH 8TH STREET • EL CENTRO, CALIFORNIA 92243 • (619) 352-1242
79-607 COUNTRY CLUB DRIVE, SUITE 5 • BERMUDA DUNES, CALIFORNIA 92201 • (619) 360-0665
2211 EAST PALO VERDE STREET • YUMA, ARIZONA 85365 • (602) 344-8844

TABLE OF CONTENTS

EXHIBITS

Plate

APPENDICES

Appendix

I. INTRODUCTION

A. Project Description

This report presents the findings of our geotechnical investigation for the proposed improvements to the water and wastewater treatment plants for the City of Imperial, California. The water treatment plant is located at the northwest corner of 4th and "B" Streets and the wastewater treatment plant is located at the northwest corner of 14th Street and Clark Road ("P" Street) (See Vicinity Map, Plate 1).

Improvements to the Water Treatment Plant Expansion includes construction of a new 7.0-MGD WTP. The new facility will include two rapid mix basins, two flocculation basins, two settling basins, and four filters, Additional improvements will include a new raw water pump station and renovation of the existing operations building A raw water pipeline will be placed in Banta Road for approximately 2500 linear feet, paralleling the existing raw water supply from the Newside Canal to the southside of the WTP.

Improvements to the Wastewater Treatment Plant Expansion includes modification of the existing treatment plant to increase treatment capacity to 1.8 MGD. New facilities include a raw sewage pumping station, new aeration basin, new secondary clarifier, new UV disinfection facility, and new sludge drying beds. The project also includes approximately 12,600 linear feet of finished water line and sewer line beginning at 14th and P Streets, thence south in P Street and Clark Road to Aten Road, thence west in Aten Road across the railroad tracks.

B. Purpose and Scope of Work

The purpose of this geotechnical study was to investigate the upper 50 to 60 ft of subsurface soils in order to provide professional opinions regarding geotechnical constraints at this site for proposed construction. The scope of our services included field investigation and in-situ testing at selected locations of the site soils; laboratory testing for physical characteristics and strength parameters; review of seismicity in the project vicinity; analysis of all data collected; and the presentation of this report with comments, opinions and recommendations regarding:

- Site geology and seismicity
- Generalized subsurface soil and groundwater conditions encountered
- An evaluation of the soil liquefaction potential, possible effects on facilities and proposed mitigation methods
- Groundwater elevations for design of uplift resistance on buried structures
- Site preparation guidelines for fill quality, along with fill placement and compaction procedures for structures and embankments.
- Assessment of groundwater conditions and the necessity for dewatering during construction.
- Recommendations for geotechnical engineering criteria for design of new structures, including:
  - Passive soil pressures
  - Friction values between foundations and the underlying materials.
  - Allowable bearing pressures under foundations as well as estimated total and differential settlement.
- Corrosion potential of on-site soils
- Flexible pavement section thickness

Evaluation of the site for presence of potential environmental hazards was not included in the scope of our work.

C. Authorization

Authorization to proceed with our work was provided by the written Agreement for Geotechnical Services with Black & Veatch on October 13, 1993. The work was performed according to this agreement.

II. METHODS OF INVESTIGATION

A. Field Exploration

The subsurface exploration was conducted on August 18 thru 21, 1993 by drilling 20 borings to approximate depths of 13.5 to 61.5 ft with an 8-inch diameter hollow stem, continuous flight auger. The borings were made at the locations shown on the Site and Exploration Plans, Plates 2 and 3. Boring locations were initially established by paced or taped measurements and later surveyed by Tesco Engineering of El Centro for coordinates and ground elevations. The borings were backfilled with auger cuttings afterwards and roadway surfaces patched with cold-mix asphaltic concrete. Temporary piezometers were set in three boreholes to obtain stabilized groundwater levels. The 2-inch PVC piezometers consist of 10 ft of 0.010 in. slotted sections encapsulated with a filter sock and gravel-packed with No. 16 silica sand.

The augers were advanced with a truck mounted, CME 55 drill rig equipped with a CME automatic hammer for performing Standard Penetration Tests (ASTM D1586). Either a 2-inch O.D. diameter, split spoon ; a 3-inch O.D. diameter, Shelby tube; or a 3-inch O.D. diameter, California Split Barrel (ring) sampler was used to obtain relatively undisturbed, soil samples ahead of the auger tip at selected intervals. Blow counts were recorded (without correction for overburden pressure) to advance the sampler from 6 to 18 inches into undisturbed soil. Blow counts recorded for the automatic hammer operating at about 90% efficiency, were corrected to the 60% energy level normally achieved by rope and cathead systems. Blow counts for the ring sampler were further adjusted by a factor of 0.67 to account for the larger sampler diameter as compared to the standard 2-inch O.D. SPT sampler.

Logs of the borings were prepared during exploration by a staff geologist and edited after examination of retrieved samples in the laboratory. The subsurface logs were completed in final form after analysis of all data and are presented on Plates 4 through 23 in the Exhibits section of the report. Soils encountered have been classified in accordance with the *Unified Soils Classification System*. A Key to the Logs is presented on Plate 24. 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 since boundaries are not always distinct in nature.

B. Laboratory Testing

Laboratory tests were performed on selected soil samples to assist in classification and the establishing engineering properties. The tests were performed in general accordance with 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:

- Moisture Contents (ASTM D2216)
- Unit Dry Densities
- Atterberg Limits (ASTM D4318)
- Grain Size Analyses (ASTM D422)
- Expansion Index (UBC 29-2 and ASTM 4829)
- Unconfined Compression shear tests (ASTM D2166)
- Direct Shear (ASTM D3080)
- One-dimensional Consolidation (ASTM D2435)
- R Value (ASTM D2844)
- Chemical Analyses (Soluble Sulfates & Chlorides, pH, and Conductivity)

The laboratory test results are presented on the subsurface logs and on Plates 25 through 38 in the Exhibits section of the report.

III. DISCUSSION

A. Site Conditions

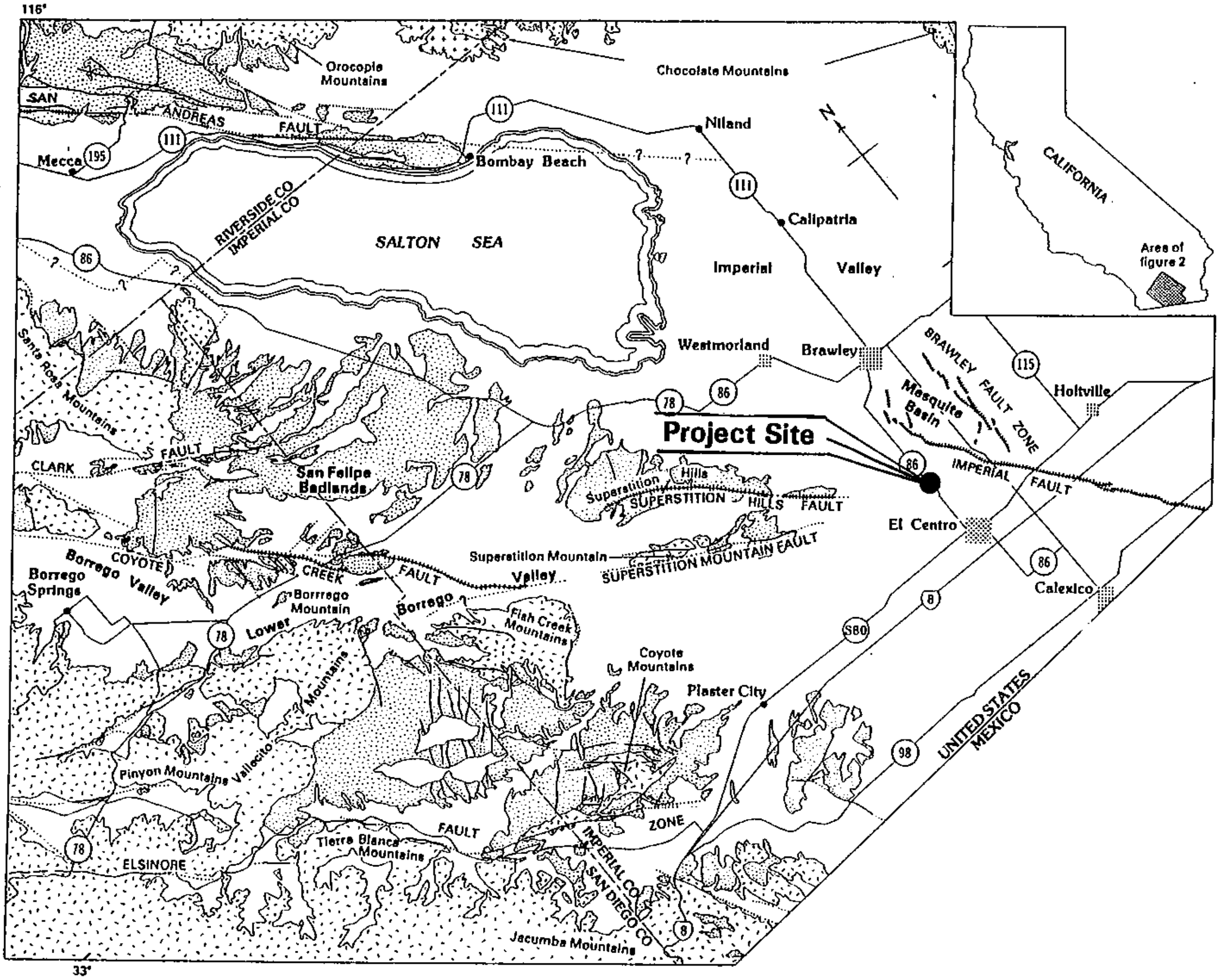
The project sites are planar and have very little, if any, vegetation covering each site. Adjacent existing facilities at the water treatment plant include an operations building, ground level steel tank storage reservoir, an elevated tank, four water storage and settling ponds, and raw water pump station and pipelines. Above ground fuel tanks and pump island with earthen containment areas exists near the proposed flocculation basins and should be relocated with this project. Adjacent existing facilities at the wastewater treatment plant includes an aeration basin, mixed liquor pumping station, control building, clarifier, sludge drying beds, and wastewater storage ponds.

The project sites are within a wide planar valley (Imperial Valley) and lies approximately 57 to 65 feet below mean sea level (943 to 935 ft site datum) in the arid southeastern region of the California low desert. Annual rainfall in this region is less than 3 inches per year with 4 months of average summertime temperature above 100 degrees F. Winter temperatures are mild, seldom reaching freezing.

B. Geologic Setting

The project sites are located in the Salton Trough physiographic province. The Salton Trough is a geologic structural depression resulting from large scale regional faulting. The trough is bounded on the northeast by the San Andreas Fault and the southwest by faults of the San Jacinto Fault Zone. The Salton Trough represents the northward extension of the Gulf of California, which has experienced continual in-filling with both marine and non-marine sediments since the Miocene Era (30 million years before present).

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.



Reference: U.S.G.S. Professional Paper 1254

EXPLANATION

Quaternary (Holocene) alluvium, dune sand, and lake deposits

Cenozoic intrusive rocks or their volcanic equivalents

Cenezoic stratified rocks and interbedded volcanic rocks

Pre-Cenozoic crystalline rocks

Contact

Faults active in Cenozoic time—Solid where exposed; dashed or queried where inferred; dotted where concealed. Hatching where historical movement has occurred

SOUTHLAND GEOTECHNICAL

FOUNDATION ENGINEERS AND MATERIALS LABS

Project No: S93211

Regional Generalized Geologic Map

Figure 1

The sites are directly underlain by Holocene (0 - 11,000 year B.P.) Cahuilla Lake beds, which consist of interbedded lenticular and tabular silt, sand, and clay. The Holocene lake deposits are considered to be less than 100 feet thick. The Pleistocene Brawley Formation underlies the Cahuilla Lake beds. The Brawley Formation consists of at least 2,000 feet of gray clay, sand, and pebbles, which in turn overlie about 6,000 feet of the late Pliocene Borrego Formation. The Borrego Formation consists of lacustrine gray clay and sand. The Borrego Formation overlies an indeterminate thickness of the Pliocene marine Imperial Formation, Alveson Andesite, and Miocene continental sediments of the Split Mountain Formation.

Base rock consisting of Mesozoic granite and probably Paleozoic metamorphic rocks are considered to be at a depth between 15,000 - 20,000 feet. Thicknesses of the various geologic formations are necessarily approximate due to the lack of site specific published data.

C. Seismicity

The Imperial Valley is located in an area of active earthquake activity. Strong earthquakes on May 19, 1940 and October 15, 1979 along the Imperial Fault, measuring 7.1 and 6.6M, respectively, triggered widespread liquefaction, as evidenced by sand volcanos, and horizontal offsets from 2 to 10 feet throughout the valley. A 5.8 magnitude event occurred along the Brawley Fault on the evening of October 15, 1979 as an aftershock to the 6.6 event on the Imperial Fault. On November 24, 1987, a 6.6 magnitude event along the Superstition Hills Fault caused over 15 miles of right lateral offset (26 in. maximum) and triggered liquefaction in areas from the Salton Sea to Seeley.

Several times a year, Imperial County will witness minor tremors, experience a moderate quake every 5 years, a damaging quake every twelve years, and be subjected to a major quake of magnitude 7.0 at least once during an average time span of 53 years.

Although earthquake predictions of time, place, and magnitudes have not been scientifically developed, significant geologic information and statistical analysis have been compiled, analyzed, and published intensely by various agencies over the past 20 years.

Our research of active or potential faulting in the areas in close proximity to the site (less than 50 miles) indicates that three active faults cross within six miles of the project site as shown on Table 1.) The Maximum Credible Event (MCE) listed was determined from published geologic information available for each fault. The MCE corresponds to an estimated probability of occurrence of 10% in 50 years (equivalent to an average return period of about 475 years). This is the probability of occurrence used by the *Structural Engineers's Association of California* (SEAOC)(Ref 3) to establish the *Uniform Building Code* seismic (Z) factor (Ref 4) that corresponds numerically to the effective peak acceleration (EPA).

The *Working Group of California Earthquake Probabilities* published a 1988 USGS report (Ref 1) that assigned a 50% conditional probability of occurrence in the 30-year period of 1988 to 2018 of a magnitude 6.5 event or greater along the Imperial Fault.

The project sites do not lie within a State of California, *Alquist-Priolo* Special Study Zone. Fault rupture is not anticipated to occur at the project sites because of the well-delineated fault lines through this region.

The inferred submerged portion of the Superstition Mountain and Hill Faults are shown to traverse within 0.4 mi and 1.0 mi from the WTP and WWTP, respectively, by the 1966 mapping by Morton (Ref7). However, discussion with Robert Sharp (USGS Menlo Park, CA) in July 1989 indicates that the early mapping of inferred/submerged faults by Morton had little basis and are unsubstantiated. Mapping by Sharp after the Superstition Hills earthquake of 1987 shows rupture along the fault line as far south as Worthington Road (Bulletin of Seismological Society of America-April 1989). The rupture is approximately 2.5 miles west of Imperial, indicating that the Superstition Hills Fault does not project through the city.

To evaluate the potential for liquefaction and assess the intensity of ground motion, a estimation of the horizontal peak ground acceleration (PGA) has been made. Ground motions are dependent primarily on the magnitude and distance to the seismogenic (rupture) zone. Accelerations are also dependent upon the attenuation of rock and soil deposits, direction of the rupture, type of fault, and other factors. For these reasons, ground motions may vary considerably in the same general area. Strong motion data sets tend to scatter considerably from an average relationship. The scatter can be statistically quantified by a standard deviation about the mean. The mean plus one standard deviation (mean+1σ) acceleration corresponds to an 84% confidence level of not being exceeded. The standard of practice is to use either mean or mean+1σ site attenuation relationships when estimating ground acceleration.

Deterministic estimates of site acceleration are presented on Table 1. Our estimates were derived using fault magnitude-distance relationships developed by Joyner and Boore (1982, 1988) from compilation of accelerograph measurements taken in the Imperial Valley during the October 1979 event (6.6 M) (Ref 2). The deterministic estimates may be used to compare the relative seismic hazard each fault presents to the sites and is considered conservative because the fault is assumed to rupture at the closest distance to the sites.

The PGA is considered to be an inconsistent scaling factor to compare to the UBC Z factor (EPA) (Ref 4) and is generally a poor indicator of structural damage during an earthquake. This is because the duration and frequency of strong ground motion in addition to local subsurface conditions and structural details are all important factors influencing structural performance. Repeatable high ground acceleration (RHGA), however, is sometimes used as an approximation of EPA. The RHGA is generally is taken to be 65% of the PGA for earthquake events within 20 miles of the site and 100% of the PGA for events greater than 20 miles in distance (Ref: Ploessel and Slossen, 1974).

Table 1

FAULT DISTANCE / MAGNITUDE & ESTIMATES OF PEAK GROUND ACCELERATION (PGA)

Fault Name	Activity	Distance & Direction from Site*	Maximum Credible Event*	Maximum Historical Event	Deterministic Site PGA (g)**
Imperial	A	3.6 (2.4) mi	7.2	7.1 (1940)	0.55 (0.62)
Superstition Hills	A	2.0 (3.2) mi	7.0	6.6 (1987)	0.58 (0.52)
Brawley	A	6.1 (5.0) mi E	7.0	5.8 (1979)	0.38 (0.43)
Borrego Mtn	A	26.6 (27.2) mi NW	7.0	6.5 (1942)	0.09
Coyote Creek	A	45.3 (45.8) mi NW	7.5	6.5 (1968)	0.06
San Andreas (Southern)	A	36.9 (36.6) mi N	8.0	6.5 (1948)	0.10
Superstition Mountains	PA	6.8 (7.6) mi	7.0		0.36 (0.33)
Sand Hills	PA	24.6 (23.5) mi	7.5		0.14
Laguna Salada	PA	19.5 (20.7) mi SW	7.5		0.17
Elsinore	PA	22.4 (23.7) mi SW	7.5	6.0 (1910)	0.15

A - Active PA - Potentially Active

* The first number is for the water treatment plant and the second number in parenthesis is for the wastewater treatment plant.

** The estimates of the PGA are based on site attenuation relationships of Joyner and Boore (1982, 1988) - mean value larger of the two components. (Ref 2).

The primary seismic hazards to the project sites are the Imperial, Brawley, and Superstition Hills Faults. The corresponding site PGA and RHGA for the Maximum Credible Event along these faults are given in the table below.

Fault	Closest Distance (miles)	MCE	PGA mean (g)	PGA Mean+ 1σ (g)	RHGA Mean (g)
Imperial	2.4	7.2	0.62	1.19	0.41
Superstition Hills	2.0	7.0	0.58	1.10	0.37
Brawley	5.0	7.0	0.43	0.82	0.28

A network of accelerographs stations are monitored by the California *Office of Strong Motion Studies* and the USGS. Selected stations in proximity to the sites are given in the following table to compare historical ground motions to estimated ground motions anticipated for the MCE given above.

Accelerograph Station	Distance from Site (miles)	6.6 M Imperial Earthquake of 10/19/79 PGA (g)	6.6 M Superstition Hills Earthquake of 11/24/87 PGA (g)
El Centro Array			
#8	3.5	0.64	0.35
#9	4.1	0.40	0.30
Reference:		USGS Prof. Paper 1254	USGS OFR 88-672

D. Subsurface Soils

The subsurface soils consist generally of lacustrine (lake bed) deposits of silty clays, clayey silts, silts, sandy silts, and silty sand that vary from soft to very stiff or loose to dense in consistency or compactness. The reader is referred to the individual boring logs for a detailed description of the subsurface soil condition at each process unit or pipeline alignment.

The surface silty clays exhibited moderate to high swell potential (9 to 11 percent) when tested in accordance with Uniform Building Code Standard 29-2 methods. The silty clay is expansive when wetted. Development of concrete slab-on-grade and pavements should include provisions for mitigating the swelling forces as well as the strength reductions caused by soil saturation or capillary rise in moisture upon sealing the ground surface to evaporation.

Stabilized groundwater levels were encountered in the borings at about 7 to 11.4 ft during the time of exploration. Groundwater levels may fluctuate with precipitation, drainage, and site grading. The groundwater levels detected should not be interpreted to represent an accurate or permanent condition.

E. Liquefaction

Liquefaction is a phenomenon, which occurs when granular soils below the water table are subjected to vibratory motions, such as produced by earthquakes. When this occurs, an increase in pore water pressure develops as the soil tends to reduce in volume. If this increase in pore water pressure is sufficient to significantly reduce the vertical effective stress, the soil strength decreases and the soil behaves as a liquid (similar to quicksand). Liquefaction can produce excessive settlement or failure of shallow bearing foundations.

Four conditions are generally required before liquefaction can occur: (1) the soils must be saturated; (2) the soils must be loosely packed; (3) the soils must be relatively cohesionless; and (4) groundshaking of sufficient intensity must occur to function as a trigger mechanism.

The soils encountered at the points of exploration included saturated silts and sandy silts. The potential for liquefaction at the project sites is high. Liquefaction analyses of the subsurface soils at the project sites were performed using the Seed, et. al. 1985 method (See Ref. No.9 and

Appendix A). The results of the analysis is that the silt strata is likely to liquefy during a strong earthquake. The safety factors against liquefaction computed are estimated to be range from about 0.14 to 0.40 for a horizontal acceleration of 0.60g estimated to result from the maximum credible earthquake. (A safety factor less than 1.0 indicates a liquefiable condition). We have estimated immediate settlements upon liquefaction may be approximately 2 1/2 to 6 inches using the 1987 Tokimatsu and Seed method (Ref. No. 10). The following table shows the liquefiable safety factors and estimated seismic induced settlements upon liquefaction.

Cost effective means of mitigate liquefaction damage for the intended structures include rigid mat foundations using flexible piping connections. Other mitigative measures do not appear to be warranted because the seismic induced settlement is expected to result in small differential settlement across the below grade structures other than across the filter complex. Differential settlements upon liquefaction are expected to be small because the net structure loads are minimal compared to the removed overburden soil weight and the structures are to be encapsulated in a non-liquefiable clay liner above the liquefiable soil zone.

Process Area	Depth of Liquefiable Zone (ft)	Safety Factor against Liquefaction	Est. Seismic induced Settlement
WTP-Flocc. Basins & Filter Complex	23 to 33 (west) to none (east)	0.40 to > 1	0 to 2 1/2
WTP Raw Water Pump Station	17 to 22 & 38-48	0.14	6
WWTP	13 to 18	0.2	4

In addition to the liquefiable zone identified above, the soft clayey silt strat while considered too clayey to liquefy (Ref 11) may experience significant loss of shear strength from cyclic ground motion.

III. RECOMMENDATIONS

A. Site Preparation and Earthwork

Any debris or vegetation such as grass ,trees, or weeds that may exist on the site at the time of construction should be removed from the construction area. Existing pipelines should be re-routed around proposed structures with abandoned sections removed and backfilled under controlled conditions. Any root balls should be completely excavated. Organic strippings should be hauled from the site and not be incorporated into any engineered fills. In areas to receive pavements or concrete slabs, the ground surface should be scarified to 12 inches, moisture conditioned, and recompacted to the criteria for native soils.

The on-site native silty clays and clayey silts may be used as engineered fill when moisture conditioned to 5 to 10 % above optimum moisture content, placed in maximum 8 inch lifts (loose), and compacted to 85-90 % of ASTM D1557 maximum density. The surface 2 ft of native soils at slabs-on-grade foundations shall removed and replaced as specified above or replaced with granular imported fill soils. The fine grained soils are highly susceptible to *pumping*; therefore construction equipment should be selected to avoid creating an unstable subgrade condition. Discing and drying of the subgrade soils may be required. Any pumping soils should be aerated or mixed with lime to provide a stable subgrade.

All imported fill soils (if required) should be non-expansive, granular soils meeting the USCS classifications of SM, SP, or SW with a maximum rock size of 3 inches and 5 to 20% passing the No. 200 sieve. Imported fill soils should be approved by the soils engineer prior to hauling to the site. The imported fill should be placed in lifts no greater than 8 inches in loose thickness and compacted to a minimum of 95% of ASTM D1557 maximum dry density at optimum moisture plus or minus 2 percent.

Positive drainage should be maintained away from all structures (5 ft minimum) to prevent ponding and subsequent saturation of the native clays soils. Drainage should be maintained for paved areas and water should not be permitted to pond on or near paved areas.

All site preparation and fill placement should be observed and tested by a representative of our firm. This is emphasized during the excavation and scarification process in order to detect any undesirable materials or conditions such as soft areas that may be encountered in the construction area.

B. Excavation Conditions

Temporary excavations above groundwater level left unshored during the construction period may be relatively stable at slope ratios of 1.5(H):1(V). Direct rainfall and drying of slopes may cause erosion or sloughing. However, limited working area at the wastewater treatment plant may require a sheet-pile, earth retention system. Sheet-pile, earth retention loads may be approximated as braced excavation loads. The apparent strut loads for braced excavations, may be based on the earth pressure diagram on Plate 39.

The specifications should clearly state that all excavations be constructed in accordance with the Federal and State OSHA requirements. The contractor has sole responsibility for the safety of his personnel. This fact should be clearly stated in the project documents. Based on present safety regulations of the OSHA, shoring and/or bracing of excavations will be required where personnel are working within excavations deeper than 5 ft. Allowances are made for sloped excavation walls in lieu of shoring or bracing. The slope ratio presented in the preceding paragraph is not necessarily the same as those required for personnel safety.

All discussions in this section regarding stable excavation slopes assumes minimal equipment vibration and adequate setback of excavated material and construction equipment from the top of the excavation. We recommend that the minimum setback distance be equal to the depth of excavation and at least 5 feet from the crown of the slope. If excavated materials are stockpiled adjacent to the excavation, the weight of this material should be considered as a surcharge load for slope stability requiring further analysis by our firm prior to project inception.

Based upon the groundwater measurements in the borings and proposed base elevations of the structures, the excavations for several of the process units will be founded below the groundwater table. These excavations below the ground water level will require dewatering. Such excavations areas are expected to be relatively small in plan dimensions. Pumping from perimeter trench drains and sumps may be an appropriate method of dewatering for most of the process units. However, the influent pumping station at the wastewater treatment plant will require other means to control groundwater as described below. The sump may need to be protected with filters to reduce the potential for internal erosion and piping in the soils adjacent to the sump. A 18 to 24-inch thick layer of 1-inch crushed rock (or 12 inches of rock with grid reinforcement such as Tensar SS-1 (BX 1100) or equivalent should be considered for stabilizing the excavation bottoms at these structures. To control groundwater and loss of ground for excavations at the wastewater treatment plant, the sheet piling should extend to at least 20 ft depth to cutoff the pervious silt strata at 13 to 18 ft depth.

The wet well for the influent pumping station that extends below the pumping station facility can be installed using a drill pier rig and a large diameter auger and CMP casing driven in conjunction with the excavation. The CMP casing will cutoff seepage and prevent loss of ground. The annulus between the CMP casing and concrete man-hole can be grouted after the manhole installation.

C. Foundation Conditions for Process Units

The process units are to be partially embedded in soils so that the base foundation slab may act as a fully compensating raft. Settlements of the structures are negliable because the net bearing pressure is decreased to zero from the removal of soil overburden. The proposed process units approximate dimensions, preliminary subgrade elevations and anticipated groundwater and subgrade conditions are given below.

Table 2A

Water Treatment Plant Improvements

Process Unit	Approx. Plan Dimesion s (ft)	Structure Height (ft)	Approx Depth below grade (ft)	Hydraulic Load (ft)	Approx GWT depth (ft)	Anticipated Subgrade Condition	Ref. Boring
Rapid Mix & Flocculation Basins	30x50	16	8	12	7	Stiff, Silty Clay	B-1 to B-3
Sedimentation Basins	20x40	16	8	12	7	Stiff, Silty Clay	B-1 to B-3
Filter Complex	55x60	20	12	16	7	Soft Clayey Silt	B-4 to B-6
Chemical Storage Feed Slab	23x46	-	0	-	7	Stiff Silty Clay	B-1 to B-6
Blower Slab	n/a	-	0	-	7	Stiff, Silty Clay	-
Raw Water Pump Station	10x10	n/a	10	8	7.5	Firm to stiff Clayey Silt/Silty Clay	B-7 & B-8

Table 2B
Wastewater Treatment Plant Improvements

Process Unit	Approx. Plan Dimesions (ft)	Structure Height (ft)	Approx Depth below grade (ft)	Hydraulic Load (ft)	Approx GWT depth (ft)	Anticipated Subgrade Condition	Ref. Boring
Influent Pumping Station	27x39	40	30	12	11 1/2	very stiff Silty Clay	B-11
Grit Chambers	7.5 dia.	12	6	9	11 1/2	loose Silt	B-11
Aeration Basin	95x270	-	10	9	9	medium dense Silt	B-12
Secondary Clarifier	60 dia	19	15	15	11	loose Sandy Silt	B-13
Sludge Pumping Station	30x40	30	15	10	11 1/2	medium dense Sandy Silt	B-14
Ultraviolet Disinfection Chamber	7x65	n/a	12	6	10 1/2	very stiff Silty Clay	B-15

Information presented above was obtained from Black & Veatch (design engineers) and should be considered preliminary.

Bottom heave of the excavations may occur as the excavations are left exposed for a extended period of time. Some of the heave occurs as "elastic" rebound as the soil is removed. This portion is removed as the soil is excavated. Longer term rebound may occur as the structure is left empty for extended periods of time. However, the amount of this rebound is estimated to be less than 1 inch and is not considered detrimental to the structure.

D. Mat Foundations

The process units are planned to be designed with a thick concrete base that can serve as a mat foundation for the structure. Mat rigidity can be estimated by using a modulus of subgrade reaction of 100 pci. The allowable net soil pressure induced by uniform mat loading and maximum toe pressure due to overturning moments, induced by wind or seismic events, should not exceed 1000 and 1500 psf respectively.

The subgrade soils at several of the process units are soft or loose and below the groundwater level. Consequently, the subgrade has a high potential for pumping with repetitive construction traffic. Therefore, the subgrade for these process units identified above should be overexcavated 18 to 24 inches and replaced with 1-inch crushed rock that is wheel rolled to a dense state. A 4-inch thick, concrete mud slab may be placed over the crushed rock to provide a working platform. The thickness of the crushed rock may be reduced to 12-inches by the use of a geogrid reinforcement such as Tensar SS-1 (BX1100) or equivalent subgrade reinforcing fabric. Elsewhere, the subgrade for the process units should be compacted for the upper 12 inches to the criteia given in the earthwork section.

Resistance to horizontal loadings will be developed by passive earth pressure on the side 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 footings of 300 pcf to resist lateral loadings. The top of one foot embedment should not be considered in computing passive resistance unless the adjacent area is confined by a slab or pavement. A friction coefficient of 0.35 may also be used beneath footings with granular subbase. An adhesion of 250 psf may be used beneath foundations founded on a cohesive subgrade to resist lateral loadings to resist lateral loadings. The larger of the calculated values should be reduced by 50 percent if both equivalent fluid pressure and friction are combined for use in computing resistance to lateral loads.

E. Lateral Earth Pressures

Lateral earth pressures for use in retaining wall design acting as equivalent fluid pressures without surcharge loads or hydrostatic pressure may be assumed to be:

Lateral Pressures and Sliding Resistance	Sand Backfill	Native Silt/Clay
Passive Pressure	300 pcf	250 pcf
Active Pressure (cantilever walls) able to rotate 0.1% of structure height	33 pcf	50 pcf
At-Rest Pressure (braced walls)	55 pcf	70 pcf
Dynamic Lateral Earth Pressure acting at midheight of structure (Mononobe-Okabe Method)	50 pcf	50 pcf
Allowable Base Lateral Sliding Resistance Dead load X Coefficient of Friction: Base Area X Adhesion:	 0.35	 0.25(silt) 250 psf(clay)

Note: The equivalent fluid pressures given for sand backfill assumes that the sand has been placed on a slope no steeper than 1(V) on 1(H) that intersects the base of the excavation.

Upward sloping backfill or surcharge loads from nearby footings can create larger lateral pressures. Should any walls be considered for retaining sloped backfill or placed adjacent to foundations, our office should be contacted for recommended design parameters. Surcharge loads should be taken into consideration if loads are applied within a zone from the face of the wall and a plane projected 45 degrees upward from the base of the wall. The increase in lateral earth pressure should be taken as 50% of the surcharge load within this zone. Retaining walls subjected to traffic loads should include a uniform surcharge load equivalent to 2 ft of native soil. The native sands at this site are naturally free draining which should allow relief of hydrostatic pressure under normal conditions. Although the dynamic lateral earth pressure is assumed to have a triangular pressure distribution, the resultant force should be taken to act through the mid-height of the retaining structure. When dynamic lateral earth pressures are taken into account, a reduced factor of safety may be used for design.

F. Slabs-On-Grade

Concrete slabs/flatwork should be a minimum of 5 inches thick due to expansive soil conditions. The concrete slabs should be underlain by a minimum of 4 inches of concrete sand or crushed aggregate base compacted to 95% of ASTM D1557 maximum density and moistened to approximately optimum moisture just prior to the concrete pour. The underlying subgrade should be moisture conditioned and compacted for 24 inches to the criteria given in the earthwork section.

Concrete slab/flatwork reinforcement should consist of a minimum of No. 3's @ 18 in. O.C. bothways placed at slab mid-height to resist swell forces and crack separation. Steel and slab recommendations are minimums only and should be verified with UBC Standard 29-4 design method for expansive soils or other accepted methods by the structural engineer/architect knowing the actual project loadings. All reinforcing steel in slabs should be continually inspected by the project architect or soils engineer during the concrete pour to insure proper location within the slab.

Control joints should be provided in all concrete slabs-on-grade at a maximum spacing of 32 times slab thickness as recommended by *American Concrete Institute* (ACI) guidelines with all joints forming 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 (1/4 of slab depth) within 8 hrs of concrete placement. Construction (cold) joints should either be thickened butt-joint with 1/2 inch dowels at 24-inches on center or a thickened keyed-joint to resist vertical deflection at the joint. All cold/construction joints in exterior flatwork should be sealed to prevent moisture or foreign material intrusion. Precautions should be taken to prevent curling of slabs in this arid desert region.

G. Concrete Mix

The native soils are known to have moderate to severe sulfate ion concentration (approximately 0.20%). Sulfate ions can attack the cementitious material in concrete, causing weakening of the cement matrix and eventual deterioration by ravelling. The *Uniform Building Code* recommends that increased quantities of Type II Portland Cement be used at a low water/cement ratio when concrete is subjected to moderate sulfate concentrations and that Type V Portland Cement or Type II Cement with 15-20% flyash replacement be used when the concrete is subjected to severe sulfate concentration.

For these reasons, we recommend that a minimum of 6 sacks per cubic yard of concrete of Type V Portland Cement with a maximum water/cement ratio of 0.45 (by weight) be used for concrete placed in contact with native soils on this project. Additionally, water-tightness of structures can be improved by plasticizing admixtures, thorough vibration of the fresh concrete, and introduction of fibrilated polypropylene fibers. All of the methods will reduce rebar corrosion and add longevity to the structures.

H. Seismic Design

This site is subject to strong ground shaking due to frequent 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. The minimum seismic design factors should comply with the latest edition of the *Uniform Building Code* for Seismic Zone 4 using a Site Coefficient of 1.5 for Soil Type S_3. The 1991 UBC assumes that the *average* area within Seismic Zone 4 has a 10% probability of experiencing an effective ground acceleration (EPA) of 0.4g or greater in 50 years.

The intent of the UBC lateral force requirements is to provide a structural design that will resist collapse from a major earthquake but may undergo some structural and non-structural damage. SEAOC urges special care be exercised so that all components of the design are all fully met and that adequate quality assurance and control be exercised during project construction for sites lying within 5 miles of the Imperial Fault. If further information on seismic design is desired, a site-specific probabilistic seismic analysis should be performed.

I. Pavements

Pipeline excavations in Banta Road, P Street, Clark Road and Aten Road will require removal and replacement of pavements. Pavements should be designed in accordance with *CALTRANS* or other acceptable methods. Since no traffic loadings were provided by the project engineer or owner, we have assumed traffic loadings for comparative evaluation. The owner or design engineer should determine the appropriate traffic conditions for the pavements. Maintenance of proper drainage is necessary to prolong the service life of the pavements.

Based on the current State of California *CALTRANS* methods and assumed traffic loads, we are providing the following asphaltic concrete pavement sections:

Table 4

PAVEMENTS

R-Value Subgrade Soils - 10 Design Method - *CALTRANS 1988*

Traffic Index	Reconstruction Pavement	Flexible Pavements	
		Asphaltic Concrete Thickness (in.)	Aggregate Base Thickness (in.)
6.0	Clark & Aten Rd.	4.0	12.0
5.0	Banta Rd.	3.0	9.0

Notes:

1) Asphaltic concrete should be Caltrans, Type B, 3/4 or 1/2 in. maximum-medium grading, compacted to a minimum of 95% of the 75-blow Marshall density (ASTM D1559).
2) Aggregate base should be Caltrans Class 2 (3/4 in. maximum), compacted to a minimum of 95% of ASTM D1557 maximum dry density.
3) All pavements should be placed on 8 inches of moisture conditioned, (2% above optimum) native soils compacted to a minimum of 90% of ASTM D1557 maximum dry density.

IV. LIMITATIONS AND ADDITIONAL SERVICES

A. Limitations

The recommendations and conclusions within this report are based on current information regarding the proposed improvements to the Water and Wastewater Treatment Plants in Imperial, California. The conclusions and recommendations are invalid if structural loads change; the additional services section is not followed; the report is used for adjacent or other property; changes of grade and/or changes in groundwater occur between the issuance of this report and construction; or if any other change is implemented, which materially alters the project from that proposed at the time this report was prepared.

The conclusions and recommendations in this report are based on selected points of field exploration, laboratory testing, and our understanding of the proposed project. Our analysis of data and the recommendations presented herein are based on the assumption that soil conditions do not vary significantly from those found at specific boring locations. However, it is possible that variations in soil conditions could exist between and beyond the exploration points or that groundwater elevations may change. These conditions may require additional studies, consultation, and possible design revisions.

This report was prepared in accordance with the generally accepted, geotechnical engineering standards of practice that existed in Imperial County at the time the report was prepared. No other warranty, expressed or implied is made. Due to potential changes in the *Geotechnical Engineering Standards of Practice*, this report should be considered invalid for periods in excess of 2 years from the report date.

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

B. Additional Services

The recommendations made in this report are based on the assumption that an adequate program of tests and inspections will be performed during construction to verify the field applicability of subsurface conditions and compliance of the recommendations that are the basis of this report. Because of our experience and familiarity with the project, Southland Geotechnical should be retained as the geotechnical consultant to provide the tests and inspections.

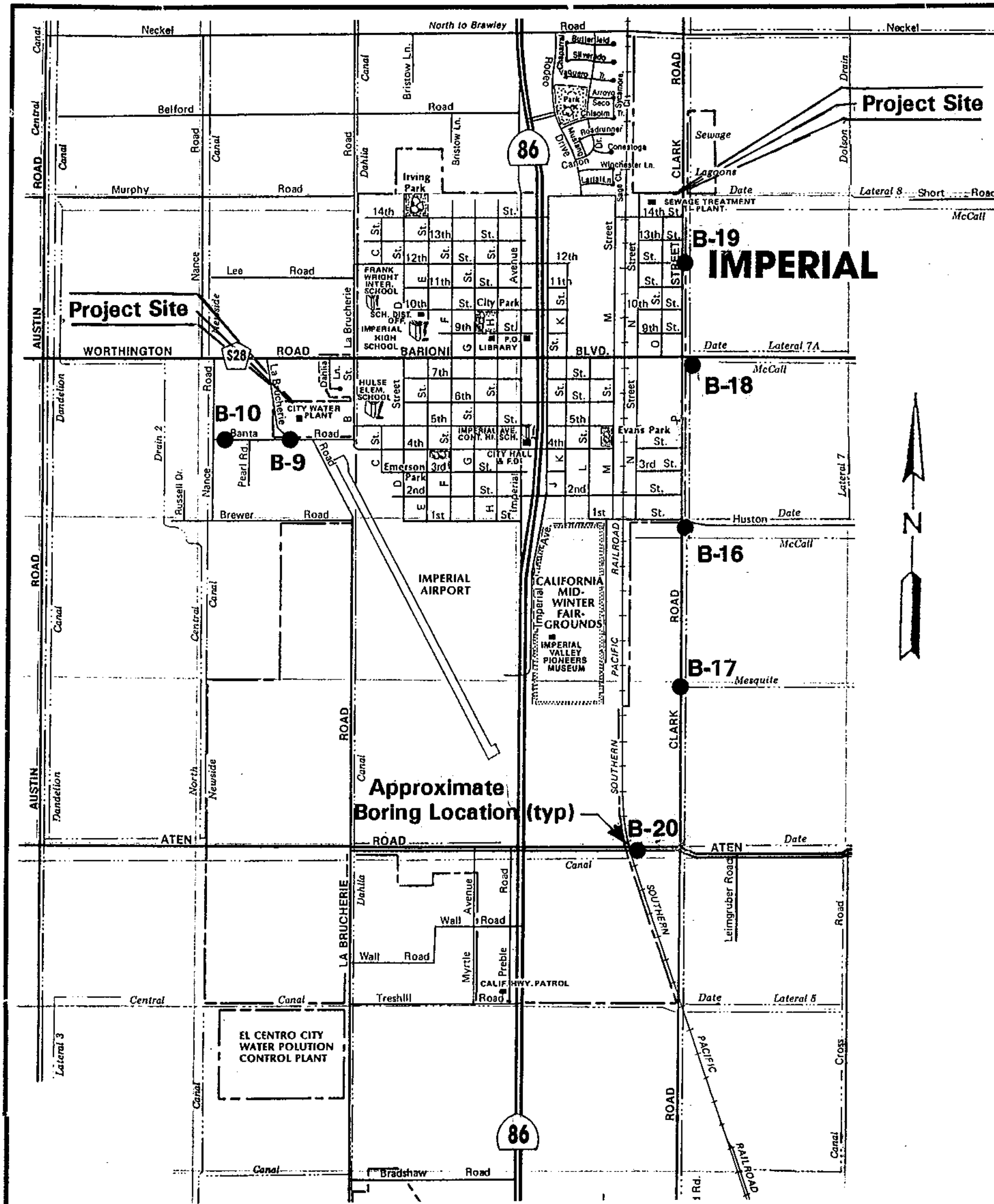
These tests and inspections should include, but not necessarily be limited to the following:

- Full-time observation and testing by the geotechnical consultant of record during site clearing, grading, excavation, placement of fills, and subgrade preparation, backfilling of utility trenches;
- Inspection of foundation excavations and reinforcing steel prior to concrete placement;
- Consultation as may be required during construction.

In addition, the project plans and specifications should be reviewed by us to verify compatibility with our recommendations and conclusions. Additional information concerning the scope and cost of these services can be obtained from our office.

-o0o-

EXHIBITS



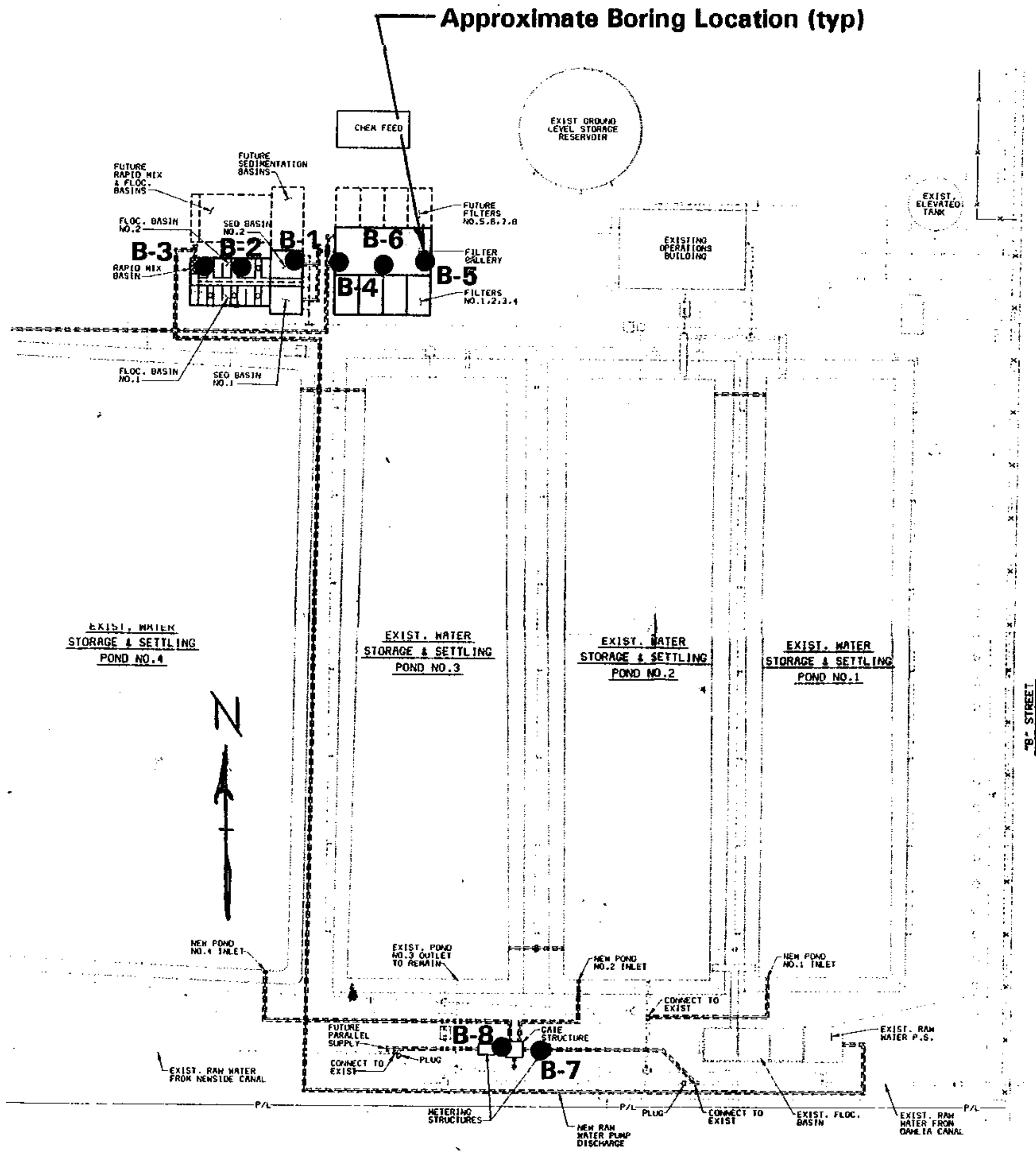
SOUTHLAND GEOTECHNICAL

FOUNDATION ENGINEERS AND MATERIALS LABS

Project No: S93211

Vicinity Map

Plate 1

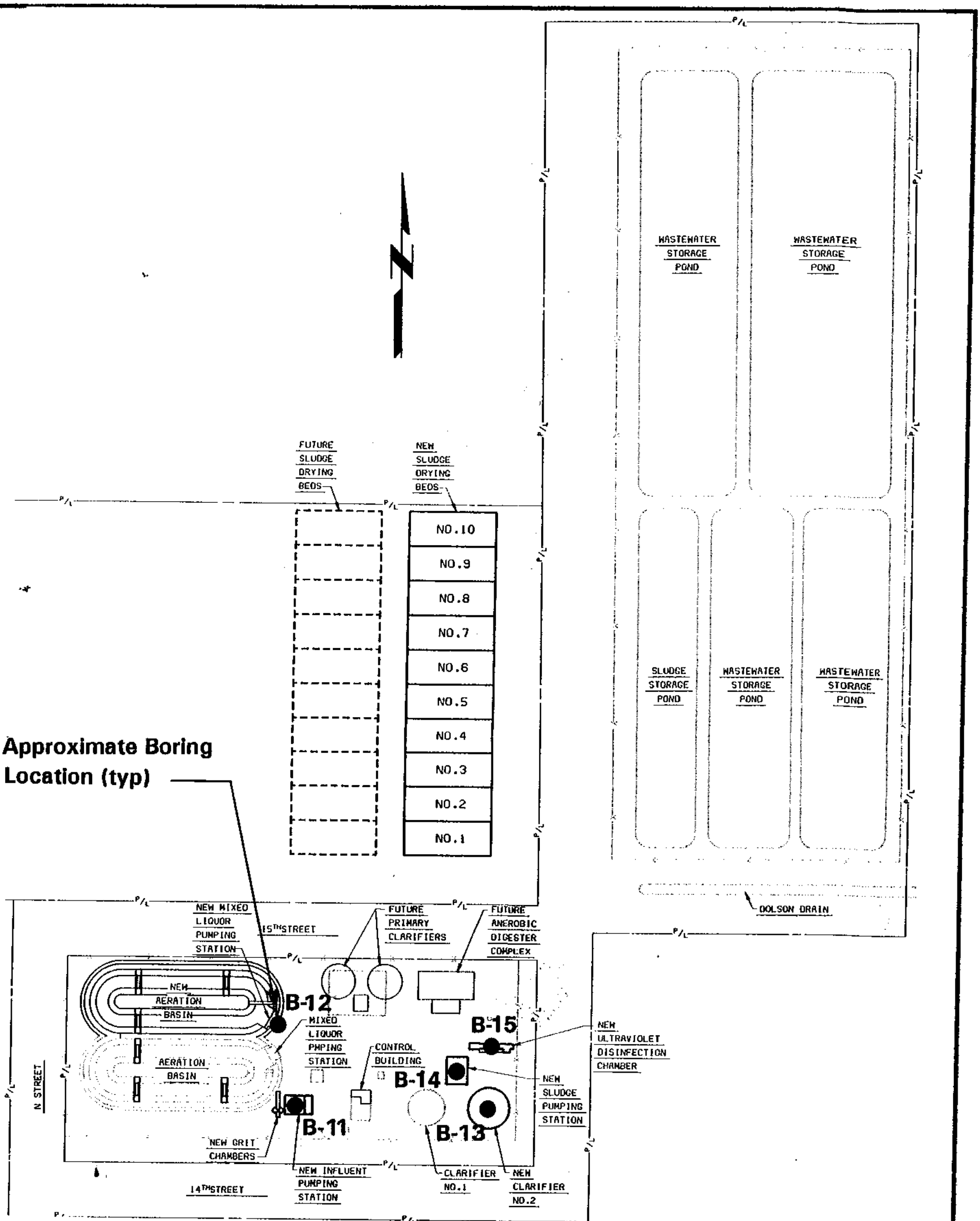


SOUTHLAND GEOTECHNICAL

Project No: S93211

Site And Exploration Plan
Water Treatment Plant Expansion

Plate 2



SOUTHLAND GEOTECHNICAL

Project No: S93211

Site And Exploration Plan
Wastewater Treatment Plant Expansion

Plate 3

CLIENT: Black & Veatch
METHOD OF DRILLING: CME 55 with autohammer
PROJECT: Imperial Water and Wastewater Plant Expansion
DATE OBSERVED:10/18/93
LOCATION: N 1,887,948 E 6,767,372 (Sedimentation Basins)
LOGGED BY: K. Harmon

LOG OF BORING B-1

SHEET 1 OF 1

DESCRIPTION OF MATERIAL

SURFACE ELEV. +/- 942.1

DEPTH (FEET)	CLASSIFICATION	SAMPLE TYPE	BLOWS/ FOOT	POCKET PEN. (TSF)	DESCRIPTION OF MATERIAL	MOISTURE CONTENT (%)	DRY UNIT WT. (PCF)	UNCONFINED COMPRESSION (TSF)	LIQUID LIMIT	PLASTICITY INDEX	PASSING # 200
					SILTY CLAY (CL): Brown, stiff to very stiff, wet						
5			9	1.5							
10			17	3.5		25.9	97.1				
					GWT @ 14 ft during drilling						
15			11	2.75	- with layers of clayey silt below 10 ft	28.1	95.7	0.70			
20			8								
					End of Boring @ 21.5 ft						
25											
30											
35											
40											

Project No:
S93211

SOUTHLAND GEOTECHNICAL

Plate
4

CLIENT: Black & Veatch
PROJECT: Imperial Water and Wastewater Plant Expansion
LOCATION: N 1,887,938 E 6,767,336 (Flocculation Basins)

METHOD OF DRILLING: CME 55 with autohammer
DATE OBSERVED: 10/18/93
LOGGED BY: K. Harmon

LOG OF BORING B-2

SHEET 1 OF 2

DESCRIPTION OF MATERIAL

SURFACE ELEV. +/- 942.2

DEPTH (FEET)	CLASSIFICATION	SAMPLE TYPE	BLOWS/ FOOT	POCKET PEN. (TSF)	DESCRIPTION OF MATERIAL	MOISTURE CONTENT (%)	DRY UNIT WT. (PCF)	UNCONFINED COMPRESSION (TSF)	LIQUID LIMIT	PLASTICITY INDEX	PASSING # 200
					SILTY CLAY/CLAYEY SILT (CL/ML): Brown, firm, wet						
5			8	0.75		15.5	116.1				
10			12	1	SILTY CLAY (CL): Brown, stiff, wet, - with lenses of silty sand						
15			8	1	- with lenses of sandy silt						
20			14	1.5		25.6	97.8	1.15			
25			12		SANDY SILT (ML): Brown, medium dense, saturated						85
30			14								
35			17	2.5	SILTY CLAY (CL): Brown, very stiff, wet, with lenses of silt						
40			9		CLAYEY SILT (ML): Brown, stiff, wet, with some sandy silt						

Project No: S93211

SOUTHLAND GEOTECHNICAL

Plate 5a

CLIENT: Black & Veatch | METHOD OF DRILLING: CME 55 with autohammer
PROJECT: Imperial Water and Wastewater Plant Expansion | DATE OBSERVED: 10/18/93
LOCATION: N 1,887,938 E 6,767,336 | LOGGED BY: K. Harmon

LOG OF BORING B-2

DESCRIPTION OF MATERIAL

SURFACE ELEV. +/- 942.2

DEPTH (FEET)	CLASSIFICATION	SAMPLE TYPE	BLOWS/ FOOT	POCKET PEN. (TSF)	DESCRIPTION OF MATERIAL	MOISTURE CONTENT (%)	DRY UNIT WT. (PCF)	UNCONFINED COMPRESSION (TSF)	LIQUID LIMIT	PLASTICITY INDEX	PASSING # 200
			12								
50			15	3	CLAY (CH): Brown, stiff to very stiff, wet with lenses of clayey silt						
					End of Boring @ 51.5 ft						
55											
60											
65											
70											
75											
80											
85											

Project No: S93211

SOUTHLAND GEOTECHNICAL

CLIENT: Black & Veatch
METHOD OF DRILLING: CME 55 with autohammer
PROJECT: Imperial Water and Wastewater Plant Expansion
DATE OBSERVED:10/19/93
LOCATION: N 1,887,936 E 6,767,303 (Rapid Mix Basin)
LOGGED BY: K. Harmon

LOG OF BORING B-3

DESCRIPTION OF MATERIAL

SURFACE ELEV. +/- 942.2

DEPTH (FEET)	CLASSIFICATION	SAMPLE TYPE	BLOWS/ FOOT	POCKET PEN. (TSF)	DESCRIPTION OF MATERIAL	MOISTURE CONTENT (%)	DRY UNIT WT. (PCF)	UNCONFINED COMPRESSION (TSF)	LIQUID LIMIT	PLASTICITY INDEX	PASSING # 200
			9	2.5	SILTY CLAY (CL): Brown, stiff, damp to 2 ft then moist to wet						
5				0.5	- firm	28.5	96.2				
			10	3	- stiff to very stiff below 7 ft	31.3	91.0				
10											
			20	2		26.7	96.2				
15											
					GWT @ 17 ft - 0.25 hr after drilling						
			6		SILT (ML): Brown, loose, saturated, with some clay						
20											
			14	2	SILTY CLAY (CL): Brown, stiff, wet						
25					End of Boring @ 23.5 ft						
30											
35											
40											

Project No:
S93211

SOUTHLAND GEOTECHNICAL

Plate
6

CLIENT: Black & Veatch
PROJECT: Imperial Water and Wastewater Plant Expansion
LOCATION: N 1,887,951 E 6,767,428 (Filter Complex)

METHOD OF DRILLING: CME 55 with autohammer
DATE OBSERVED: 10/19/93
LOGGED BY: K. Harmon

LOG OF BORING B-4

SHEET 1 OF 1

DESCRIPTION OF MATERIAL

SURFACE ELEV. +/- 942.1

DEPTH (FEET)	CLASSIFICATION	SAMPLE TYPE	BLOWS/ FOOT	POCKET PEN. (TSF)	DESCRIPTION OF MATERIAL	MOISTURE CONTENT (%)	DRY UNIT WT. (PCF)	UNCONFINED COMPRESSION (TSF)	LIQUID LIMIT	PLASTICITY INDEX	PASSING # 200
					SILTY CLAY (CL): Brown, stiff to very stiff, damp grading to wet, with lenses of clayey silt				39	24	
5			9	1							
10			15	2.5		25.5	100.1				
15			5	0.5	CLAYEY SILT (ML): Brown, firm, wet to saturated, with lenses of silty clay	31.8	91.1				
20			32	3.5	SILTY CLAY (CL): Brown, very stiff, wet with lenses of clayey silt	22.4	103.2				
25			11		SANDY SILT (ML): Brown, medium dense, saturated						
					End of Boring @ 26.5 ft						
30											
35											
40											

CLIENT: Black & Veatch
PROJECT: Imperial Water and Wastewater Plant Expansion
LOCATION: N 1,887,955 E 6,767,482 (Filter Complex)

METHOD OF DRILLING: CME 55 with autohammer
DATE OBSERVED: 10/19/93
LOGGED BY: K. Harmon

LOG OF BORING B-5

SHEET 1 OF 1

DESCRIPTION OF MATERIAL

SURFACE ELEV. +/- 941.9

DEPTH (FEET)	BLOWS/ FOOT	POCKET PEN. (TSF)	DESCRIPTION OF MATERIAL	MOISTURE CONTENT (%)	DRY UNIT WT. (PCF)	UNCONFINED COMPRESSION (TSF)	LIQUID LIMIT	PLASTICITY INDEX	PASSING # 200
			SILTY CLAY (CL): Brown, very stiff to stiff, moist to wet						
	18	2.75							
5									
			GWT @ 7.1 ft - 10/28/93						
	11	2.25							
10									
	3		- soft	31.0	92.5		30	16	
15									
	3		CLAYEY SILT (ML): Brown, soft, saturated, sandy						68
20									
	5			30.7	91.1	0.22			
25	6	0.5	SILTY CLAY/CLAYEY SILT (CL/ML): Brown, firm, wet						
			End of Boring @ 26.5 ft						
			A 25 ft temporary piezometer was set in the borehole.						
30									
35									
40									

Project No:
S93211

SOUTHLAND GEOTECHNICAL

Plate
8

CLIENT: Black & Veatch | METHOD OF DRILLING: CME 55 with autohammer
PROJECT: Imperial Water and Wastewater Plant Expansion | DATE OBSERVED: 10/19/93
LOCATION: N 1,887,955 E 6,767,460 (Filter Complex) | LOGGED BY: K. Harmon

LOG OF BORING B-6

DESCRIPTION OF MATERIAL

SURFACE ELEV. +/- 941.9

DEPTH (FEET)	CLASSIFICATION	SAMPLE TYPE	BLOWS/ FOOT	POCKET PEN. (TSF)	DESCRIPTION OF MATERIAL	MOISTURE CONTENT (%)	DRY UNIT WT. (PCF)	UNCONFINED COMPRESSION (TSF)	LIQUID LIMIT	PLASTICITY INDEX	PASSING # 200
					SILTY CLAY (CL): Brown, stiff, damp to wet						
5			9	2							
10			6		CLAYEY SILT (ML): Brown, firm, wet to saturated, with some fine sand	28.7	96.5				
15			6								
20			9		- firm to stiff	30.7	98.2				
25			2		- very soft, with layer of sandy silt and some silty clay						
30			8	1	SILTY CLAY (CL): Brown, firm to stiff, wet - with layer of clayey silt						
35			11	2							
40			9	1.5	SILTY CLAY/CLAYEY SILT (CL/ML): Brown, stiff, wet, interbedded						

Project No: S93211

SOUTHLAND GEOTECHNICAL

Plate 9a

CLIENT: Black & Veatch
PROJECT: Imperial Water and Wastewater Plant Expansion
LOCATION: N 1,887,955 E 6,767,460 (Filter Complex)

METHOD OF DRILLING: CME 55 with autohammer
DATE OBSERVED: 10/19/93
LOGGED BY: K. Harmon

LOG OF BORING B-6

SHEET 2 OF 2

DEPTH (FEET)	CLASSIFICATION	SAMPLE TYPE	BLOWS/ FOOT	POCKET PEN. (TSF)	DESCRIPTION OF MATERIAL	MOISTURE CONTENT (%)	DRY UNIT WT. (PCF)	UNCONFINED COMPRESSION (TSF)	LIQUID LIMIT	PLASTICITY INDEX	PASSING # 200
			12	1.5	SILTY CLAY/CLAYEY SILT (CL/ML): Brown, stiff, wet, interbedded						
50			15	2.75	SILTY CLAY (CL): Brown, stiff to very stiff, wet,						
55			14		CLAYEY SILT (ML): Brown, stiff, wet,						
60			33		SILT (ML): Brown, medium dense, saturated, with some fine sand and clay	25.9	98.9				
					End of Boring @ 61.5 ft						
65											
70											
75											
80											
85											

Project No: S93211

SOUTHLAND GEOTECHNICAL

Plate 9b

CLIENT: Black & Veatch | METHOD OF DRILLING: CME 55 with autohammer
PROJECT: Imperial Water and Wastewater Plant Expansion | DATE OBSERVED:10/19/93
LOCATION: N 1,887,457 E 6,767,565 (Raw Water Pump Station) | LOGGED BY: K. Harmon

LOG OF BORING B-7

DESCRIPTION OF MATERIAL

SURFACE ELEV. +/- 943.1

DEPTH (FEET)	CLASSIFICATION	SAMPLE TYPE	BLOWS/ FOOT	POCKET PEN. (TSF)	DESCRIPTION OF MATERIAL	MOISTURE CONTENT (%)	DRY UNIT WT. (PCF)	UNCONFINED COMPRESSION (TSF)	LIQUID LIMIT	PLASTICITY INDEX	PASSING # 200
					SILTY CLAY (CL): Brown, stiff, damp to wet				44	26	
5			8		CLAYEY SILT (ML): Brown, firm, wet						
10			5	0.2	- with some fine sand below 10 ft	30.8	93.8				
15			3		- soft, saturated						
20			2		SANDY SILT (ML): Brown, very loose, saturated						76
25			15	2	SILTY CLAY (CL): Brown, stiff, wet	26.3	98.9				
30			27		SANDY SILT (ML): Brown, medium dense, saturated	24.5	99.6				
35			11		SILTY CLAY/CLAYEY SILT (CL/ML): Brown, stiff, wet, interbedded						
40			9		SILT (ML): Brown, loose, saturated with some clay and fine sand						

Project No: S93211

SOUTHLAND GEOTECHNICAL

Plate 10a

CLIENT: Black & Veatch
PROJECT: Imperial Water and Wastewater Plant Expansion
LOCATION: N 1,887,457 E 6,767,565 (Raw Water Pump Station)

METHOD OF DRILLING: CME 55 with autohammer
DATE OBSERVED: 10/19/93
LOGGED BY: K. Harmon

LOG OF BORING B-7

DESCRIPTION OF MATERIAL

SURFACE ELEV. +/- 943.1

DEPTH (FEET)	CLASSIFICATION	SAMPLE TYPE	BLOWS/ FOOT	POCKET PEN. (TSF)	DESCRIPTION OF MATERIAL	MOISTURE CONTENT (%)	DRY UNIT WT. (PCF)	UNCONFINED COMPRESSION (TSF)	LIQUID LIMIT	PLASTICITY INDEX	PASSING # 200
			18		SILT (ML): Brown, medium dense, saturated with some clay and fine sand	27.4	98.2				
50			20	3	SILTY CLAY (CL): Brown, very stiff, wet,						
					End of Boring @ 51.5 ft						
55											
60											
65											
70											
75											
80											
85											

Project No: S93211

SOUTHLAND GEOTECHNICAL

Plate 10b

CLIENT: Black & Veatch
METHOD OF DRILLING: CME 55 with autohammer
PROJECT: Imperial Water and Wastewater Plant Expansion
DATE OBSERVED:10/19/93
LOCATION: N 1,887,457 E 6,767,520 (Raw Water Pump Station)
LOGGED BY: K. Harmon

LOG OF BORING B-8

DESCRIPTION OF MATERIAL

SURFACE ELEV. +/- 942.7

DEPTH (FEET)	CLASSIFICATION	SAMPLE TYPE	BLOWS/ FOOT	POCKET PEN. (TSF)	DESCRIPTION OF MATERIAL	MOISTURE CONTENT (%)	DRY UNIT WT. (PCF)	UNCONFINED COMPRESSION (TSF)	LIQUID LIMIT	PLASTICITY INDEX	PASSING # 200
			8	1	SILTY CLAY (CL): Brown, stiff, moist						
5			9		CLAYEY SILT (ML): Brown, firm to stiff, wet GWT @ 7.5 ft- 10/28/93	31.3	91.9				
10			9	3	SILTY CLAY/CLAYEY SILT (CL/ML): Brown, stiff, wet, interbedded						
15			11		SILT (ML): Brown, medium dense, saturated with clay layer						
20			27	4	SILTY CLAY (CL): Brown, very stiff, wet	20.7	106.2				
25					End of Boring @ 23.5 ft A Temporary piezometer was set in an adjacent borehole.						
30											
35											
40											

CLIENT: Black & Veatch METHOD OF DRILLING: CME 55 with autohammer
PROJECT: Imperial Water and Wastewater Plant Expansion DATE OBSERVED:10/20/93
LOCATION: N 1,887,433 E 6,766,703 (Banta Road @ La Brucherie) LOGGED BY: K. Harmon

LOG OF BORING B-9

SHEET 1 OF 1

SURFACE ELEV. +/- 940.9

DEPTH (FEET)	CLASSIFICATION	SAMPLE TYPE	BLOWS/ FOOT	POCKET PEN. (TSF)	DESCRIPTION OF MATERIAL	MOISTURE CONTENT (%)	DRY UNIT WT. (PCF)	UNCONFINED COMPRESSION (TSF)	LIQUID LIMIT	PLASTICITY INDEX	PASSING # 200
					PAVEMENT: 2 in. A.C. over 5 in. Aggregate Base						
5			11	2.5	CLAYEY SILT (ML): Brown, stiff, wet	27.0	95.6				
10			15	2.25	SILTY CLAY (CL): Brown, stiff, wet,	27.3	95.3				
15			17	2	- with lenses of clayey silt						
					End of Boring @ 16.5 ft						
20					No Groundwater encountered						
25											
30											
35											
40											

Project No:
S93211

SOUTHLAND GEOTECHNICAL

Plate
12

CLIENT: Black & Veatch
PROJECT: Imperial Water and Wastewater Plant Expansion
LOCATION: N 1,887,415 E 6,765,766 (Banta Road, E of Nance)

METHOD OF DRILLING: CME 55 with autohammer
DATE OBSERVED: 10/20/93
LOGGED BY: K. Harmon

LOG OF BORING B-10

DESCRIPTION OF MATERIAL

SURFACE ELEV. +/- 940.0

DEPTH (FEET)	CLASSIFICATION	SAMPLE TYPE	BLOWS/ FOOT	POCKET PEN. (TSF)	DESCRIPTION OF MATERIAL	MOISTURE CONTENT (%)	DRY UNIT WT. (PCF)	UNCONFINED COMPRESSION (TSF)	LIQUID LIMIT	PLASTICITY INDEX	PASSING # 200
					PAVEMENT: 2 in. A.C. over 5 in. Aggregate Base						
			10		CLAYEY SILT (ML): Brown, stiff, wet	27.9	94.5				
5											
			20	4	SILTY CLAY (CL): Brown, very stiff to stiff, wet, - with thin silty sand lenses	24.0	102.9				
10											
			15	1.5	- with lenses of silt						
15					End of Boring @ 14 ft						
					No Groundwater encountered						
20											
25											
30											
35											
40											

Project No: S93211

SOUTHLAND GEOTECHNICAL

Plate 13

CLIENT: Black & Veatch
PROJECT: Imperial Water and Wastewater Plant Expansion
LOCATION: N 1,891,208 E 6,772,756 (Influent Pumping Station)

METHOD OF DRILLING: CME 55 with autohammer
DATE OBSERVED:10/20/93
LOGGED BY: K. Harmon

LOG OF BORING B-11

DESCRIPTION OF MATERIAL

SURFACE ELEV. +/- 936.8

DEPTH (FEET)	CLASSIFICATION	SAMPLE TYPE	BLOWS/ FOOT	POCKET PEN. (TSF)	DESCRIPTION OF MATERIAL	MOISTURE CONTENT (%)	DRY UNIT WT. (PCF)	UNCONFINED COMPRESSION (TSF)	LIQUID LIMIT	PLASTICITY INDEX	PASSING # 200
					SILTY CLAY (CL): Brown, stiff, damp to wet				44	28	
5			3		SILT (ML): Brown, loose, wet, with some fine sand and clay						
10			15	3.5	SILTY CLAY (CL): Brown, stiff to very stiff, wet GWT @ 11.4 ft - 10/28/93	27.2	96.6				
15			8		SANDY SILT (ML): Brown, loose, saturated						
20			14	3.25	SILTY CLAY (CL): Brown, stiff to very stiff, wet	23.6	100.0	3.21			
25			12	2							
30			36	4	- hard, with sandy silt	25.9	97.2				
35			7		CLAYEY SILT (ML/CL): Brown, firm, wet to saturated, with silty clay	29.5	93.4		35	16	100
40			17	1.25	SILTY CLAY (CL): Brown, stiff, wet						

Project No: S93211

SOUTHLAND GEOTECHNICAL

Plate 14a

CLIENT: Black & Veatch
PROJECT: Imperial Water and Wastewater Plant Expansion
LOCATION: N 1,891,208 E 6,772,756 (Influent Pumping Station)

METHOD OF DRILLING: CME 55 with autohammer
DATE OBSERVED: 10/20/93
LOGGED BY: K. Harmon

LOG OF BORING B-11

SHEET 2 OF 2

DESCRIPTION OF MATERIAL

SURFACE ELEV. +/- 936.8

DEPTH (FEET)	CLASSIFICATION	SAMPLE TYPE	BLOWS/ FOOT	POCKET PEN. (TSF)	DESCRIPTION OF MATERIAL	MOISTURE CONTENT (%)	DRY UNIT WT. (PCF)	UNCONFINED COMPRESSION (TSF)	LIQUID LIMIT	PLASTICITY INDEX	PASSING # 200
			15	4	SILTY CLAY (CL): Brown, very stiff, wet						
50			23	3.5		26.8	94.9				
					End of Boring @ 51.5 ft						
55					A temporary piezometer was set in the borehole.						
60											
65											
70											
75											
80											
85											

CLIENT: Black & Veatch
METHOD OF DRILLING: CME 55 with autohammer
PROJECT: Imperial Water and Wastewater Plant Expansion
DATE OBSERVED: 10/20/93
LOCATION: N 1,891,331 E 6,772,756 (Mixed Liquor Pumping Station)
LOGGED BY: K. Harmon

LOG OF BORING B-12

SHEET 1 OF 1

DEPTH (FEET)	CLASSIFICATION	SAMPLE TYPE	BLOWS/ FOOT	POCKET PEN. (TSF)	DESCRIPTION OF MATERIAL SURFACE ELEV. +/- 934.6	MOISTURE CONTENT (%)	DRY UNIT WT. (PCF)	UNCONFINED COMPRESSION (TSF)	LIQUID LIMIT	PLASTICITY INDEX	PASSING # 200
5			9		SANDY SILT (ML): Brown, loose, wet	27.7	95.3				
10			14		SILT/CLAYEY SILT (ML): Brown, medium dense/firm, wet, with some fine sand and clay	28.5	93.2	0.80			
15			2		-very loose						86
					End of Boring @ 16.5 ft						
20											
25											
30											
35											
40											

Project No: S93211

SOUTHLAND GEOTECHNICAL

Plate 15

CLIENT: Black & Veatch
PROJECT: Imperial Water and Wastewater Plant Expansion
LOCATION: N 1,891,199 E 6,773,058 (Secondary Clarifer)

METHOD OF DRILLING: CME 55 with autohammer
DATE OBSERVED:10/20/93
LOGGED BY: K. Harmon

LOG OF BORING B-13

SURFACE ELEV. +/- 936.6

DEPTH (FEET)	CLASSIFICATION	SAMPLE TYPE	BLOWS/ FOOT	POCKET PEN. (TSF)	DESCRIPTION OF MATERIAL	MOISTURE CONTENT (%)	DRY UNIT WT. (PCF)	UNCONFINED COMPRESSION (TSF)	LIQUID LIMIT	PLASTICITY INDEX	PASSING # 200
					SILTY CLAY (CL): Brown, firm, damp to wet						
5			8		SILT (ML): Brown, loose, wet						
10			18		SILTY CLAY (CL): Brown, very stiff, wet, with silt lenses	25.2	95.4				
15			5		SANDY SILT (ML): Brown, loose, saturated, with some clay						
20			10	1	SILTY CLAY (CL): Brown, stiff, wet	31.0	91.3				
25			11	2							
30			18	2.5	- with pockets of silty sand	27.8	94.0				
35			12	2	SILTY CLAY/CLAYEY SILT (CL/ML): Brown, stiff,wet interbedded						
40			18	4	SILTY CLAY (CL): Brown, very stiff, wet						

Project No: S93211

SOUTHLAND GEOTECHNICAL

Plate 16a

CLIENT: Black & Veatch
PROJECT: Imperial Water and Wastewater Plant Expansion
LOCATION: N 1,891,199 E 6,773,058 (Secondary Clarifer)

METHOD OF DRILLING: CME 55 with autohammer
DATE OBSERVED: 10/20/93
LOGGED BY: K. Harmon

LOG OF BORING B-13

SHEET 2 OF 2

DEPTH (FEET)	CLASSIFICATION	SAMPLE TYPE	BLOWS/ FOOT	POCKET PEN. (TSF)	DESCRIPTION OF MATERIAL SURFACE ELEV. +/- 936.6	MOISTURE CONTENT (%)	DRY UNIT WT. (PCF)	UNCONFINED COMPRESSION (TSF)	LIQUID LIMIT	PLASTICITY INDEX	PASSING # 200
			17	1.5	SILTY CLAY (CL): Brown, very stiff, wet	33.7	84.8				
50			12	2							
55			13		SILT (ML): Grayish brown, medium dense, saturated, with some very fine sand and clay	26.3	98				
60			30		-no recovery						
			55		SILTY SAND (SM): Grayish brown, very dense, saturated						
65					End of Boring @ 63.5 ft						
70											
75											
80											
85											

Project No: S93211

SOUTHLAND GEOTECHNICAL

Plate 16b

CLIENT: Black & Veatch
PROJECT: Imperial Water and Wastewater Plant Expansion
LOCATION: N 1,891,247 E 6,773,029 (Sludge Pumping Station)

METHOD OF DRILLING: CME 55 with autohammer
DATE OBSERVED: 10/20/93
LOGGED BY: K. Harmon

LOG OF BORING B-14

SHEET 1 OF 1

DESCRIPTION OF MATERIAL

SURFACE ELEV. +/- 937.2

DEPTH (FEET)	CLASSIFICATION	SAMPLE TYPE	BLOWS/ FOOT	POCKET PEN. (TSF)	DESCRIPTION OF MATERIAL	MOISTURE CONTENT (%)	DRY UNIT WT. (PCF)	UNCONFINED COMPRESSION (TSF)	LIQUID LIMIT	PLASTICITY INDEX	PASSING # 200
0–5			13		SANDY CLAY (CL): Brown, stiff, wet, with green mottling	21.7	98.7				
5–10			6		CLAYEY SILT (ML): Brown, firm, wet						
10–15			12	3.5	SILTY CLAY (CL): Brown, stiff to very stiff, wet, with clayey silt	27.0	93.6	0.93			
15–18.5			12		SANDY SILT (ML): Brown, medium dense, saturated, with						
18.5–23.5			16	1.5	SILTY CLAY (CL): Brown, stiff, wet	25.8	97.2				
25					End of Boring @ 23.5 ft						

Depth scale: 5, 10, 15, 20, 25, 30, 35, 40

Project No: S93211

SOUTHLAND GEOTECHNICAL

Plate 17

CLIENT: Black & Veatch
PROJECT: Imperial Water and Wastewater Plant Expansion
LOCATION: N 1,891,305 E 6,773,060 (UV Disinfection Chamber)

METHOD OF DRILLING: CME 55 with autohammer
DATE OBSERVED: 10/20/93
LOGGED BY: K. Harmon

LOG OF BORING B-15

SHEET 1 OF 1

DEPTH (FEET)	CLASSIFICATION	SAMPLE TYPE	BLOWS/ FOOT	POCKET PEN. (TSF)	DESCRIPTION OF MATERIAL SURFACE ELEV. +/- 936.0	MOISTURE CONTENT (%)	DRY UNIT WT. (PCF)	UNCONFINED COMPRESSION (TSF)	LIQUID LIMIT	PLASTICITY INDEX	PASSING # 200
5			6		CLAYEY SILT (ML): Brown, firm, wet						
10			16	3	SILTY CLAY (CL): Brown, very stiff, wet, GWT @ 13 ft while drilling	29.4	92.0				
15			18		SILTY SAND (SM): Brown, medium dense, saturated	22.5	99.5				
					End of Boring @ 16.5 ft						
20											
25											
30											
35											
40											

Project No:
S93211

SOUTHLAND GEOTECHNICAL

Plate
18

CLIENT: Black & Veatch
METHOD OF DRILLING: CME 55 with autohammer
PROJECT: Imperial Water and Wastewater Plant Expansion
DATE OBSERVED:10/21/93
LOCATION: N 1,886,230 E 6,773,159 (Clark Road @ Huston)
LOGGED BY: K. Harmon

LOG OF BORING B-16

SHEET 1 OF 1

DESCRIPTION OF MATERIAL

SURFACE ELEV. +/- 938.8

DEPTH (FEET)	CLASSIFICATION	SAMPLE TYPE	BLOWS/ FOOT	POCKET PEN. (TSF)	DESCRIPTION OF MATERIAL	MOISTURE CONTENT (%)	DRY UNIT WT. (PCF)	UNCONFINED COMPRESSION (TSF)	LIQUID LIMIT	PLASTICITY INDEX	PASSING # 200
					PAVEMENT: 3 in. A.C. over 7 in. Aggregate Base						
5			15		CLAYEY SILT (ML): Brown, stiff, wet	27.7	94.5				
10			9		GWT @ 10 ft while drilling SILT (ML): Brown, loose, saturated, with very fine sand and clay	26.8	95.0				
15			23		SANDY SILT (ML); Brown, medium dense, saturated						
					End of Boring @ 16.5 ft						
20											
25											
30											
35											
40											

Project No:
S93211

SOUTHLAND GEOTECHNICAL

CLIENT: Black & Veatch
METHOD OF DRILLING: CME 55 with autohammer
PROJECT: Imperial Water and Wastewater Plant Expansion
DATE OBSERVED: 10/21/93
LOCATION: N 1,883,074 E 6,773,175 (Clark Road @ Mesquite Drain)
LOGGED BY: K. Harmon

LOG OF BORING B-17

SHEET 1 OF 1

DESCRIPTION OF MATERIAL

SURFACE ELEV. +/- 941.6

DEPTH (FEET)	CLASSIFICATION	SAMPLE TYPE	BLOWS/ FOOT	POCKET PEN. (TSF)	DESCRIPTION OF MATERIAL	MOISTURE CONTENT (%)	DRY UNIT WT. (PCF)	UNCONFINED COMPRESSION (TSF)	LIQUID LIMIT	PLASTICITY INDEX	PASSING # 200
					PAVEMENT: 3-1/4 in. A.C. over 5 in. Aggregate Base						
			19	2.75	SILTY CLAY (CL): Brown, very stiff to stiff, moist to wet	25.9	93.9				
5											
			13	3		27.5	96.4				
10											
			8		SANDY SILT (ML): Brown, loose, wet to saturated						
15					End of Boring @ 13.5 ft						
					No Groundwater encountered at time of drilling.						
20											
25											
30											
35											
40											

Project No:
S93211

SOUTHLAND GEOTECHNICAL

Plate
20

CLIENT: Black & Veatch
PROJECT: Imperial Water and Wastewater Plant Expansion
LOCATION: N 1,888,751 E 6,773,152 (Clark Road @ Barioni Bvd)

METHOD OF DRILLING: CME 55 with autohammer
DATE OBSERVED: 10/21/93
LOGGED BY: K. Harmon

LOG OF BORING B-18

SHEET 1 OF 1

SURFACE ELEV. +/- 935.3

DEPTH (FEET)	CLASSIFICATION	SAMPLE TYPE	BLOWS/ FOOT	POCKET PEN. (TSF)	DESCRIPTION OF MATERIAL	MOISTURE CONTENT (%)	DRY UNIT WT. (PCF)	UNCONFINED COMPRESSION (TSF)	LIQUID LIMIT	PLASTICITY INDEX	PASSING # 200
					PAVEMENT: 3 in. A.C. over 5 in. Aggregate Base						
5			5	1	SILTY CLAY (CL): Brown, stiff, wet, - with 0.5 ft layer of gray clayey silt with ash						
					GWT @ 8 ft while drilling						
10			8		SILTY CLAY/SANDY SILT (ML): Brown, firm/loose, wet to saturated, interbedded	25.6	99.8				
15			7		CLAYEY SILT (ML); Brown, firm, wet to saturated, with some fine sand	32.7	90.3				
					End of Boring @ 16.5 ft						
20											
25											
30											
35											
40											

CLIENT: Black & Veatch | METHOD OF DRILLING: CME 55 with autohammer

PROJECT: Imperial Water and Wastewater Plant Expansion | DATE OBSERVED: 10/21/93

LOCATION: N 1,890,368 E 6,773,145 (Clark Road @ 12th Street) | LOGGED BY: K. Harmon

LOG OF BORING B-19

SHEET 1 OF 1

DESCRIPTION OF MATERIAL

SURFACE ELEV. +/- 935.0

DEPTH (FEET)	CLASSIFICATION	SAMPLE TYPE	BLOWS/ FOOT	POCKET PEN. (TSF)	DESCRIPTION OF MATERIAL	MOISTURE CONTENT (%)	DRY UNIT WT. (PCF)	UNCONFINED COMPRESSION (TSF)	LIQUID LIMIT	PLASTICITY INDEX	PASSING # 200
					PAVEMENT: A.C. over Aggregate Base						
			3		CLAYEY SILT (ML): Brown, soft, wet to saturated - with some fine sand, roots, and wood chips						
5											
			3		- with some fine sand	29.9	93				
10											
			0		VITIFIED CLAY PIPE: fuel oil odor noticeable						
15					Auger Refusal @ 14 ft on possible concrete No Groundwater encountered at time of drilling.						
20											
25											
30											
35											
40											

Project No: S93211

SOUTHLAND GEOTECHNICAL

Plate 22

CLIENT: Black & Veatch
PROJECT: Imperial Water and Wastewater Plant Expansion
LOCATION: N 1,880,826 E 6,772,497 (Aten Road, E of SPRR)

METHOD OF DRILLING: CME 55 with autohammer
DATE OBSERVED: 10/21/93
LOGGED BY: K. Harmon

LOG OF BORING B-20

SHEET 1 OF 1

DESCRIPTION OF MATERIAL

SURFACE ELEV. +/- 945.7

DEPTH (FEET)	CLASSIFICATION	SAMPLE TYPE	BLOWS/ FOOT	POCKET PEN. (TSF)	DESCRIPTION OF MATERIAL	MOISTURE CONTENT (%)	DRY UNIT WT. (PCF)	UNCONFINED COMPRESSION (TSF)	LIQUID LIMIT	PLASTICITY INDEX	PASSING # 200
					PAVEMENT: 3 in. A.C. over 10 in. Aggregate Base						
					SAND (SW): Grayish brown, dense, moist (FILL)						
5			15	2.5	SILTY CLAY (CL): Brown, stiff to very stiff, moist to wet						
10			17	3.5							
15			7		CLAYEY SILT (ML); Brown, firm, wet to saturated, GWT @ 15 ft while drilling	30.3	86.5				
					End of Boring @ 16.5 ft						
20											
25											
30											
35											
40											

DEFINITION OF TERMS

PRIMARY DIVISIONS			SYMBOLS	SECONDARY DIVISIONS
COARSE GRAINED SOILS MORE THAN HALF OF MATERIAL IS LARGER THAN No. 200 SIEVE SIZE	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 SIZE	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, gravelly clays, sandy clays lean clays.
			OL	Organic silts and organic silty clays of low plasticity.
	SILTS AND CLAYS LIQUID LIMIT IS GREATER 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"

U.S. STANDARD SERIES SIEVE — CLEAR SQUARE SIEVE OPENNINGS

RELATIVE DENSITY

SANDS, GRAVELS AND NON-PLASTIC SILTS	BLOWS/FOOT*
VERY LOOSE	0 - 4
LOOSE	4 - 10
MEDIUM DENSE	10 - 30
DENSE	30 - 50
VERY DENSE	OVER 50

CONSISTENCY

CLAYS AND PLASTIC SILTS	STRENGTH**	BLOWS/FOOT*
VERY SOFT	0 - 1/4	0 - 2
SOFT	1/4 - 1/2	2 - 4
FIRM	1/2 - 1	4 - 8
STIFF	1 - 2	8 - 16
VERY STIFF	2 - 4	16 - 32
HARD	OVER 4	OVER 32

* NUMBER OF BLOWS OF 140 POUND HAMMER FALLING 30-INCHES TO DRIVE A 2-INCH O.D. (1-3/8-INCH I.D.) SPLIT SPOON (ASTM D-1586).

** UNCONFINED COMPRESSIVE STRENGTH IN TONS/SQ.FT. AS DETERMINED BY LABORATORY TESTING OR APPROXIMATED BY THE STANDARD PENETRATION TEST (ASTM D-1586), 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 POUND HAMMER FALLING 30 INCHES. (CORRECTED FOR SAMPLER DIAMETER)
   STANDARD PENETRATION TEST - NUMBER OF BLOWS PER FOOT
   SHELBY TUBE - 3 INCH NOMINAL DIAMETER TUBE HYDRAULICALLY PUSHED.
2. P.P.= POCKET PENETROMETER (TONS/SQ.FOOT)
3. NR = NO RECOVERY
4. GWT - GROUND WATER TABLE OBSERVED @ SPECIFIED TIME

KEY TO LOGS

SOUTHLAND GEOTECHNICAL

CLIENT: Black & Veatch
PROJECT: Imperial WTP & WWTP Improvements
JOB NO: S93211
DATE: 11/10/93

EXPANSION INDEX TEST (UBC 29-2 & ASTM 4829)

Sample Location & Depth (ft)	Initial Moisture (%)	Compacted Dry Density (pcf)	Final Moisture (%)	Volumetric Swell (%)	Expansion Index (EI)	Expansive Potential
B-4 @ 1-4 ft	12.6	106.2	26.3	8.2	**89**	Medium
B-7 @ 0-5 ft	13.6	104.0	27.7	8.8	**97**	High
B-11 @ 1-5 ft	11.6	105.8	28.3	10.7	**110**	High

UBC CLASSFICATION	
0-20	Very Low
20-50	Low
50-90	Medium
90-130	High
130+	Very High

Note: The measured EI have been adjusted to the estimated EI at 50% saturation in accordance with Section 10.1.2 of ASTM D4829.

SOUTHLAND GEOTECHNICAL

Project No: S93211

Expansion Index Test Results

Plate 25

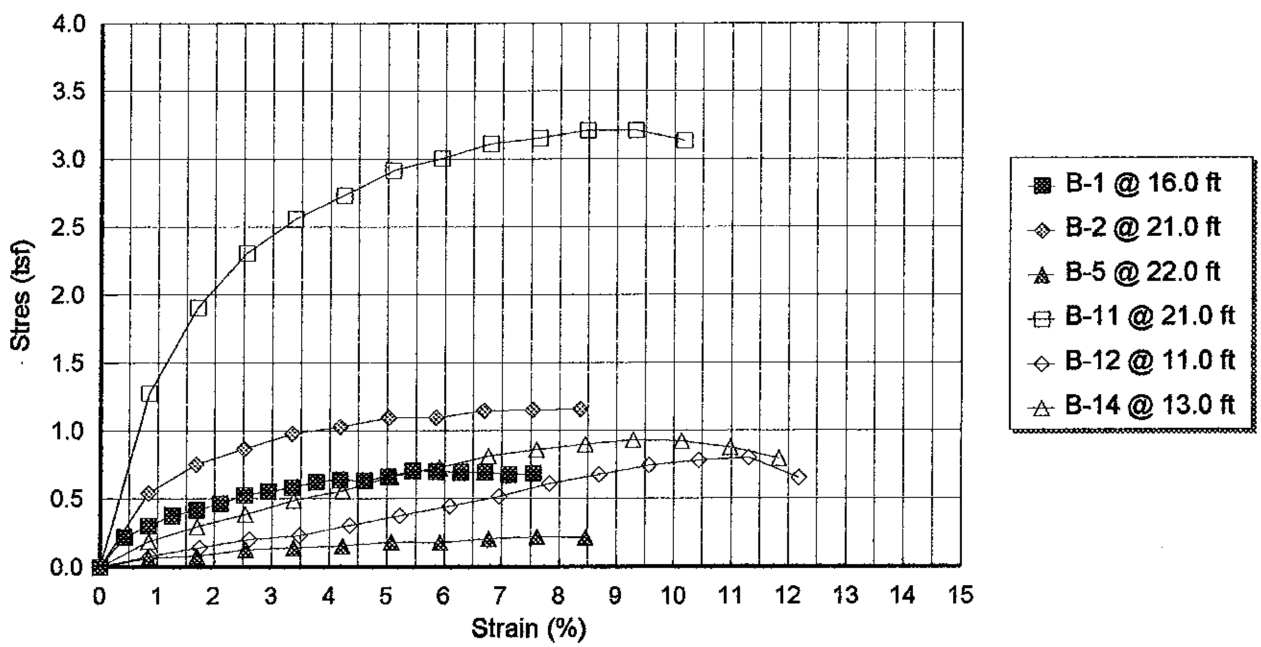
SOUTHLAND GEOTECHNICAL

CLIENT: Black & Veatch
PROJECT: Imperial WTP & WWTP Improvements
JOB NO: S93211
DATE: 11/11/93

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	16.0	28.1	95.7	0.70	0.35	5.4
B-2	21.0	25.6	97.8	1.15	0.58	8.4
B-5	22.0	30.7	91.1	0.22	0.11	7.6
B-11	21.0	23.6	100.0	3.21	1.60	8.5
B-12	11.0	28.5	93.2	0.80	0.40	11.3
B-14	13.0	27.9	93.6	0.93	0.47	9.3

STRESS-STRAIN PLOT



SOUTHLAND GEOTECHNICAL
FOUNDATION ENGINEERS AND MATERIAL LABS
Project No: S93211

Unconfined Compression Test Results

Plate 26

SOUTHLAND GEOTECHNICAL

CLIENT: Black & Veatch
PROJECT: Imperial WTP & WWTP Improvements
JOB NO: S93211
DATE: 11/09/93

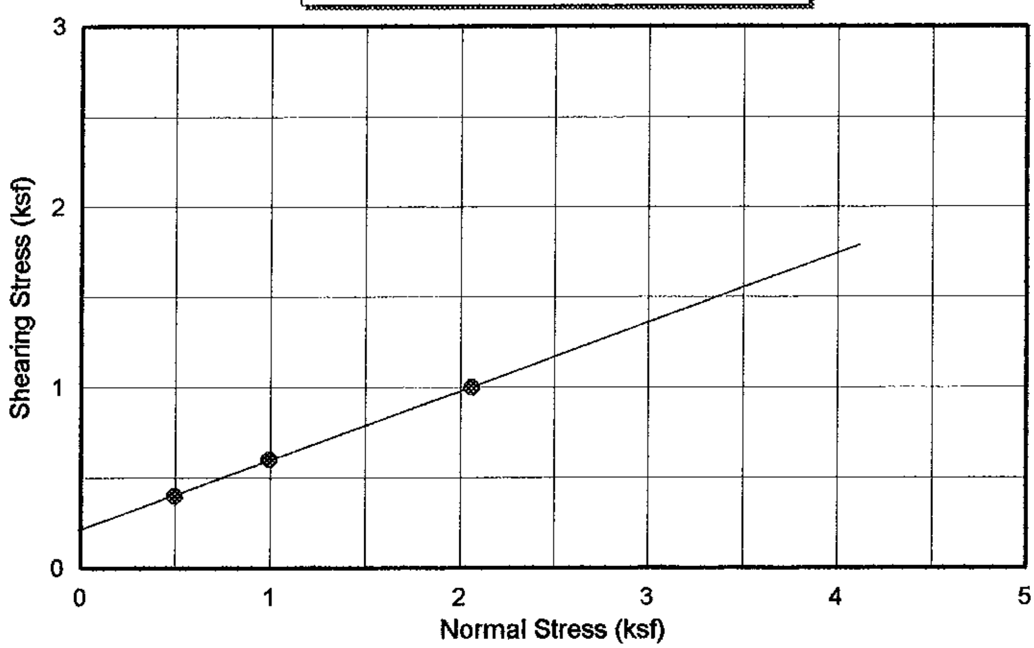
DIRECT SHEAR TEST - INSITU & SATURATED (ASTM D3080)

BORING NO: B-4
DEPTH, ft: 15-16.5
SAMPLE DESCRIPTION: Clayey Silt (ML)

Specimen:	1	2	3	Avg.
Moisture Content, %:	31.3	31.4	32.6	31.8
Dry Density, pcf:	90.8	92.3	90.2	91.1
Normal Stress, ksf:	0.50	1.00	2.06	
Shearing Stress, ksf:	0.40	0.60	1.00	

Angle of Internal Friction: **21** degrees
Cohesion: **0.21** ksf

DIRECT SHEAR TEST RESULTS



FOUNDATION ENGINEERS AND MATERIAL LABS
Project No: **S93211**

Direct Shear Test Results

Plate 27

SOUTHLAND GEOTECHNICAL

CLIENT: Black & Veatch
PROJECT: Imperial WTP & WWTP Improvements
JOB NO: S93211
DATE: 11/09/93

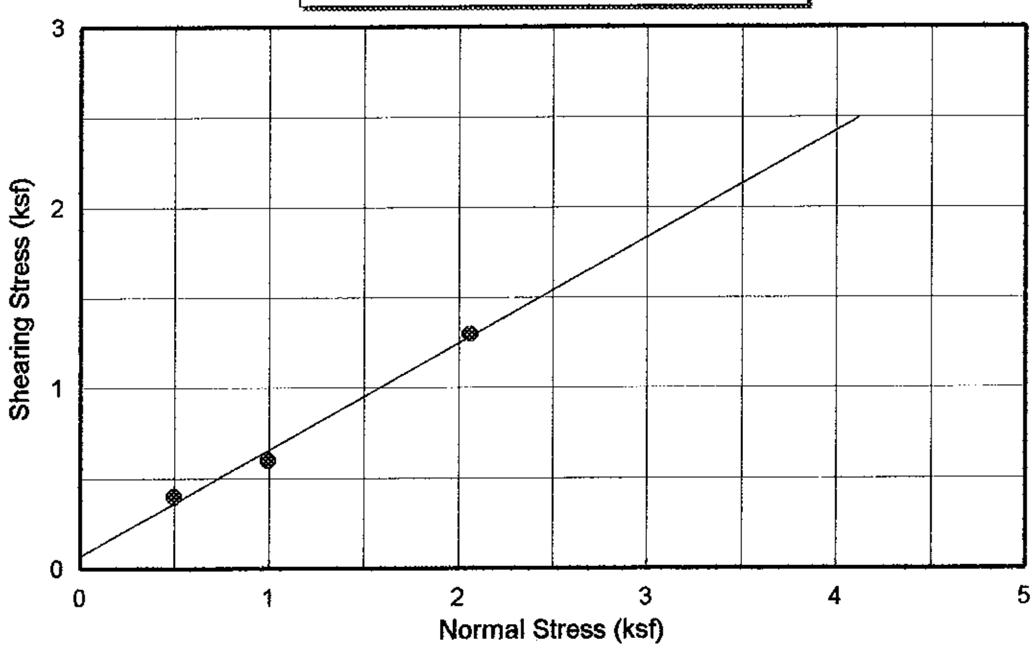
DIRECT SHEAR TEST - INSITU & SATURATED (ASTM D3080)

BORING NO: B-12
DEPTH, ft: 5-6.5
SAMPLE DESCRIPTION: Sandy Silt (ML)

Specimen:	1	2	3	Avg.
Moisture Content, %:	27.5	27.7	27.8	27.7
Dry Density, pcf:	97.3	95.2	93.3	95.3
Normal Stress, ksf:	0.50	1.00	2.06	
Shearing Stress, ksf:	0.40	0.60	1.30	

Angle of Internal Friction: **30** degrees
Cohesion: **0.07** ksf

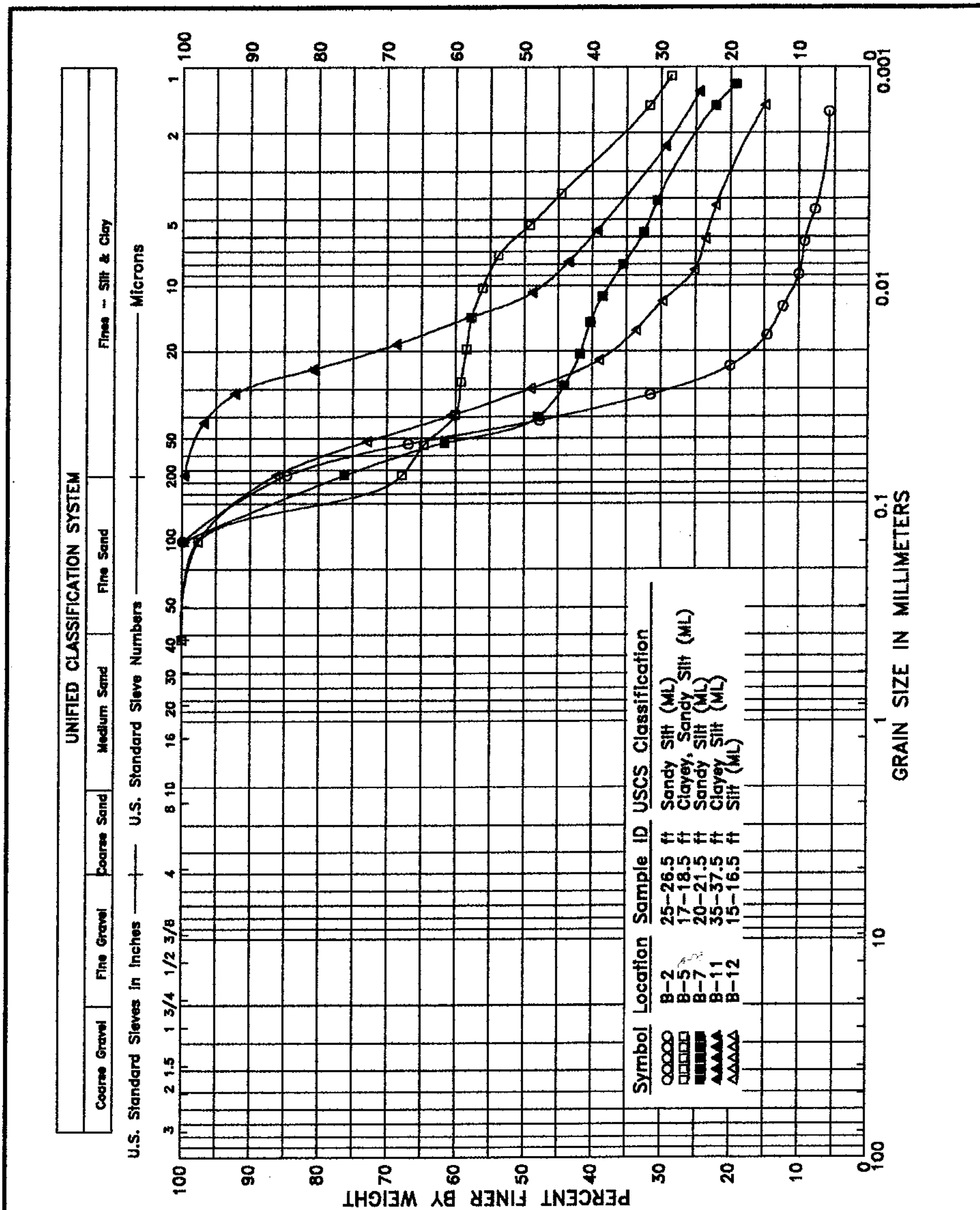
DIRECT SHEAR TEST RESULTS



SOUTHLAND GEOTECHNICAL
FOUNDATION ENGINEERS AND MATERIAL LABS
Project No: S93211

Direct Shear Test Results

Plate 28



SOUTHLAND GEOTECHNICAL

Project No: S93211

Grain Size Distribution

Plate 29

SOUTHLAND GEOTECHNICAL

CLIENT: Black & Veatch
PROJECT: Imperial WTP & WWTP Improvements
JOB NO: S93211
DATE: 11/10/93

CHEMICAL ANALYSES

Boring:	B-3	B-5	B-8	B-12	B-13
Sample Depth, ft:	2	7	2	0-5	0-5
pH:	8.9	7.6	7.5	7.4	7.4
Electrical Conductivity: millimhos/cm (Saturated soil extract)	59.0	2.5	2.8	42.5	3.0
Chloride (Cl), ppm:	1,900	2,100	1,100	2,900	1,400
Sulfate (SO4), ppm:	1,200	400	450	1,875	975

Note: Tests were performed by Agricultural Technical Service, Inc. of Brawley, California under subcontract to us.

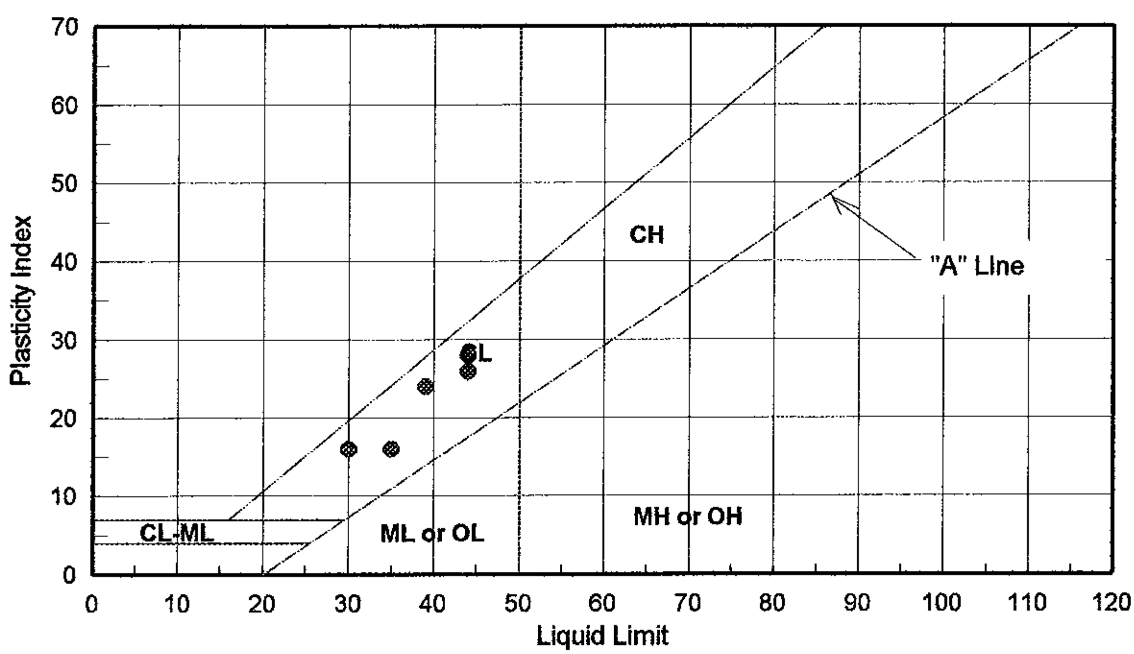
SOUTHLAND GEOTECHNICAL

CLIENT: Black & Veatch
PROJECT: Imperial WTP & WWTP Improvements
JOB NO: S93211
DATE: 11/11/93

ATTERBERG LIMITS (ASTM D4318)

Sample Location	Sample Depth (ft)	Liquid Limit (LL)	Plastic Limit (PL)	Plasticity Index (PI)	USCS Classif-ation
B-4	1-4	39	15	24	CL
B-5	12-13.5	30	14	16	CL
B-7	0-5	44	18	26	CL
B-11	1-5	44	16	28	CL
B-11	35-36.5	35	19	16	CL

PLASTICITY CHART

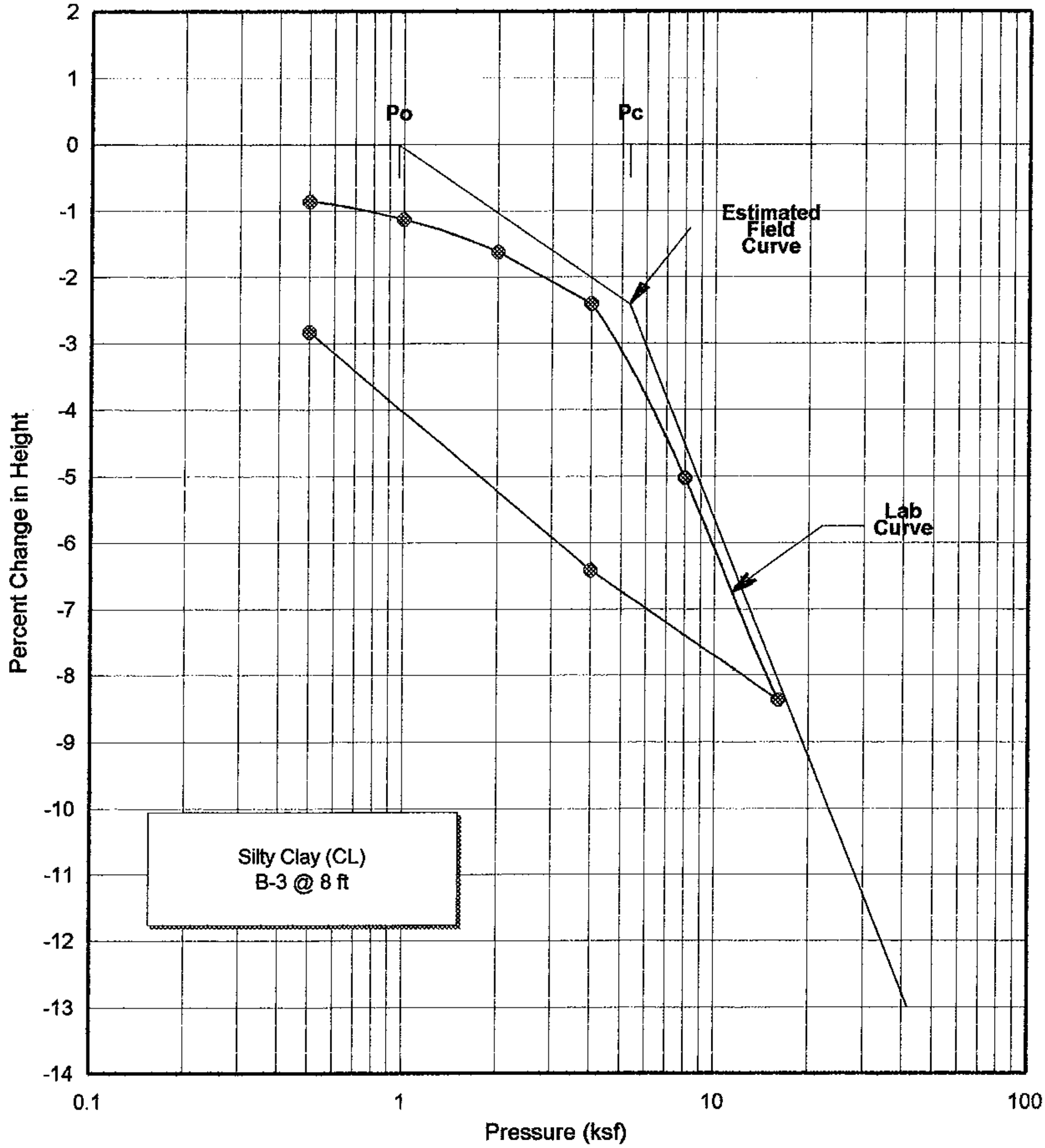


Project No: S93211

Atterberg Limits Test Results

Plate 31

ONE DIMENSIONAL CONSOLIDATION TEST (ASTM D2435)



Results of Test:			Initial	Final
Overburden Pressure, Po:	1.0 ksf	Dry Density, pcf:	91.0	90.5
Preconsol. Pressure, Pc:	5.3 ksf	Water Content, %:	31.3	32.9
Compression Index, Cc:	0.223	Void Ratio, e:	0.884	0.895
Recompression. Index, Cr:	0.061	Saturation, %:	96.2	99.9

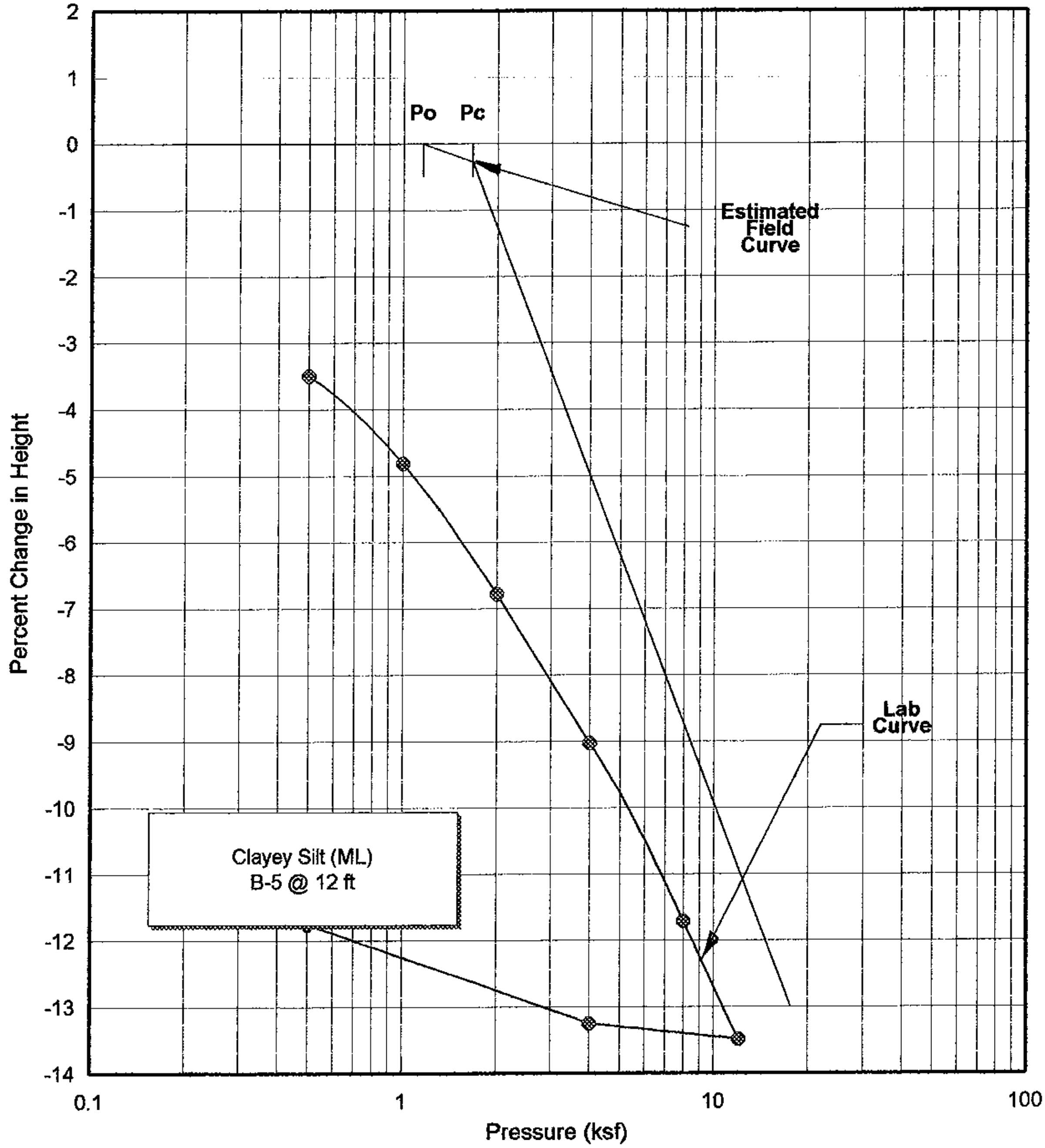
SOUTHLAND GEOTECHNICAL

Project No: S93211

Consolidation Test Results

Plate 32

ONE DIMENSIONAL CONSOLIDATION TEST (ASTM D2435)



Results of Test:				Initial	Final
Overburden Pressure, Po:	1.2 ksf		Dry Density, pcf:	92.5	103.1
Preconsol. Pressure, Pc:	1.7 ksf		Water Content, %:	31.0	23.8
Compression Index, Cc:	0.229		Void Ratio, e:	0.840	0.651
Recompression. Index, Cr:	0.030		Saturation, %:	99.6	98.9

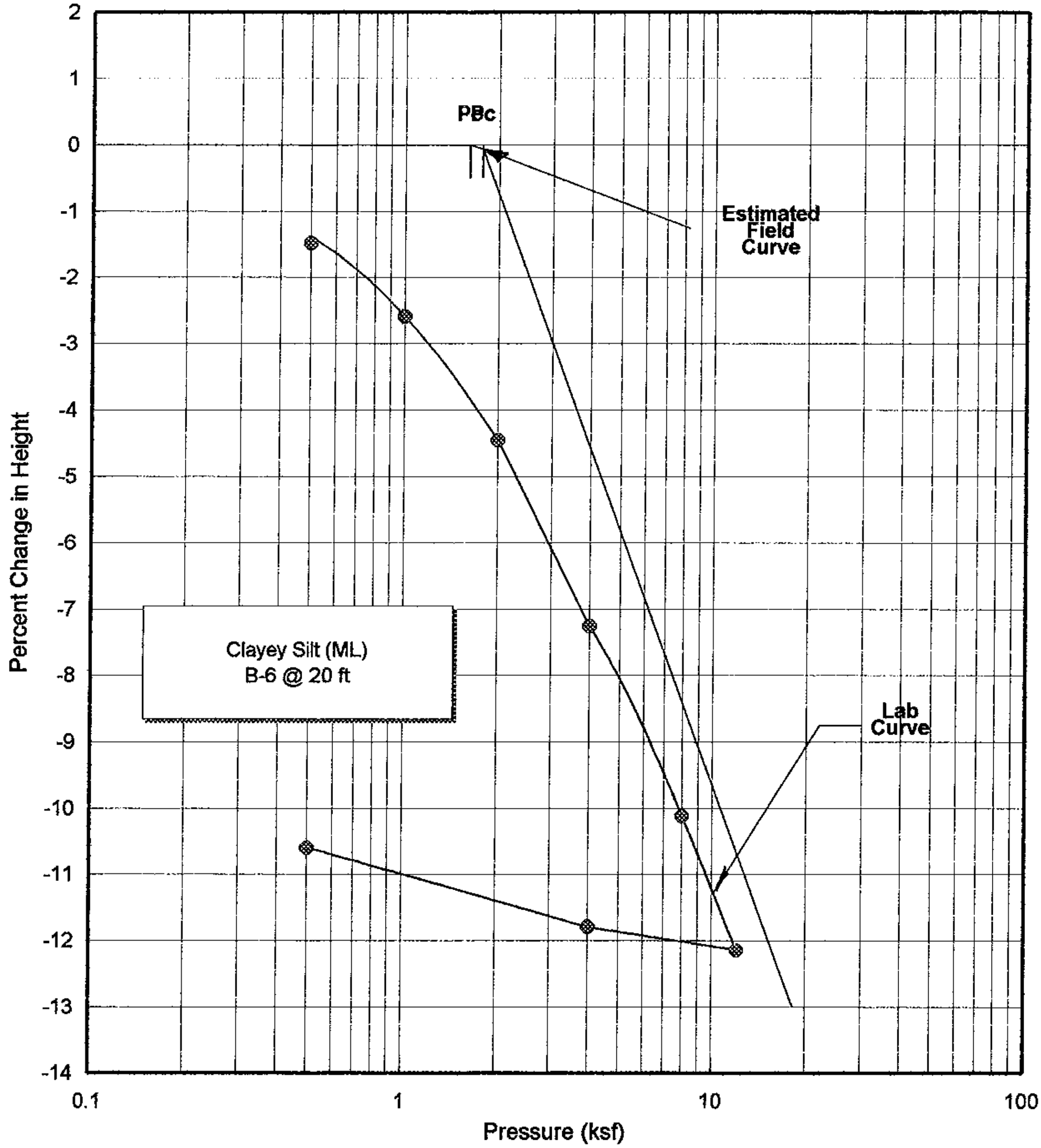
SOUTHLAND GEOTECHNICAL

Project No: S93211

Consolidation Test Results

Plate 33

ONE DIMENSIONAL CONSOLIDATION TEST (ASTM D2435)



Results of Test:				Initial	Final
Overburden Pressure, Po:	1.6 ksf		Dry Density, pcf:	91.9	100.8
Preconsol. Pressure, Pc:	1.8 ksf		Water Content, %:	30.7	25.2
Compression Index, Cc:	0.235		Void Ratio, e:	0.843	0.680
Recompression. Index, Cr:	0.024		Saturation, %:	98.2	100.0

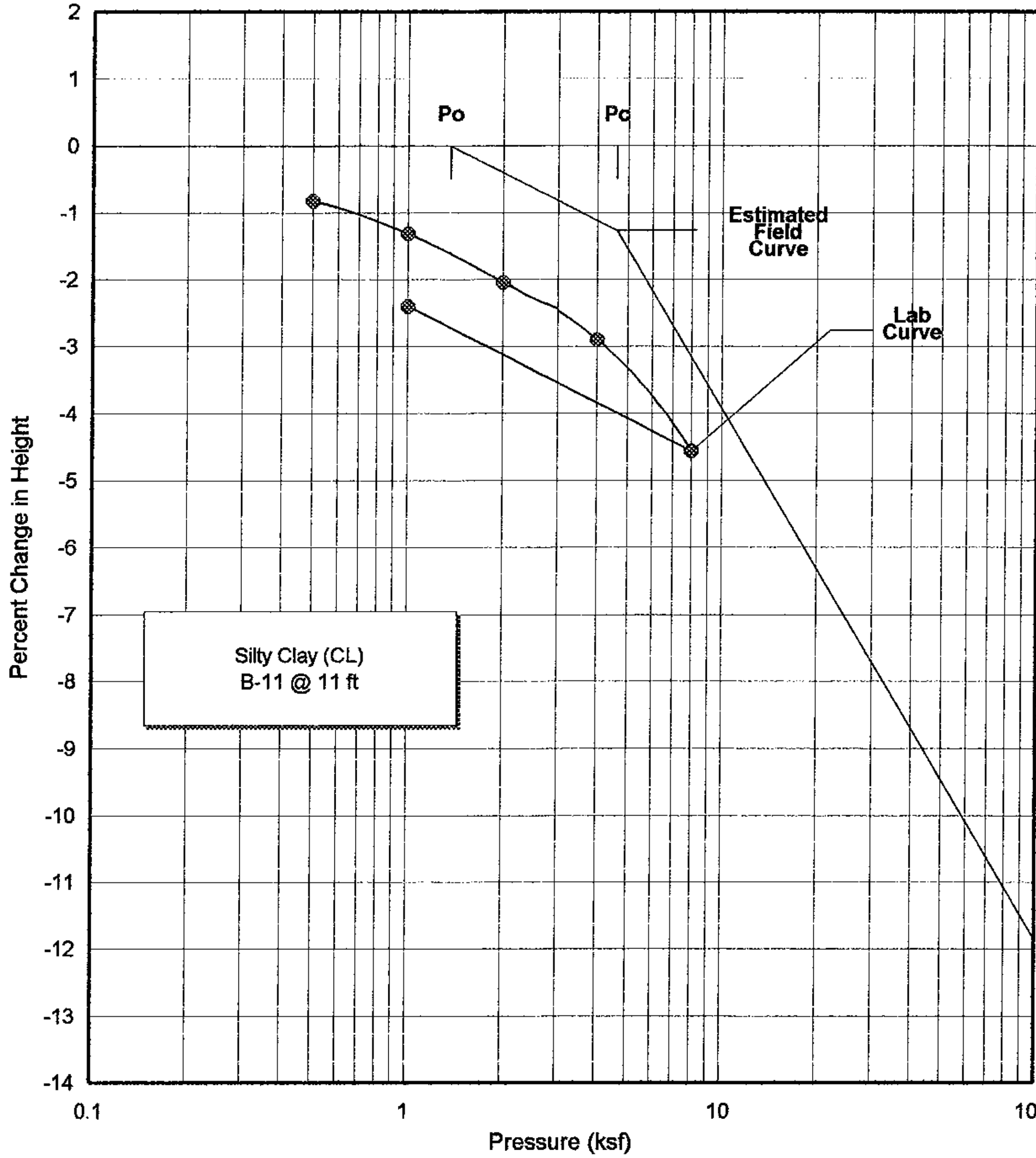
SOUTHLAND GEOTECHNICAL

Project No: S93211

Consolidation Test Results

Plate 34

ONE DIMENSIONAL CONSOLIDATION TEST (ASTM D2435)



Results of Test:				Initial	Final
Overburden Pressure, Po:	1.4 ksf		Dry Density, pcf:	99.8	98.4
Preconsol. Pressure, Pc:	4.6 ksf		Water Content, %:	23.7	26.7
Compression Index, Cc:	0.134		Void Ratio, e:	0.697	0.721
Recompression. Index, Cr:	0.041		Saturation, %:	91.8	99.9

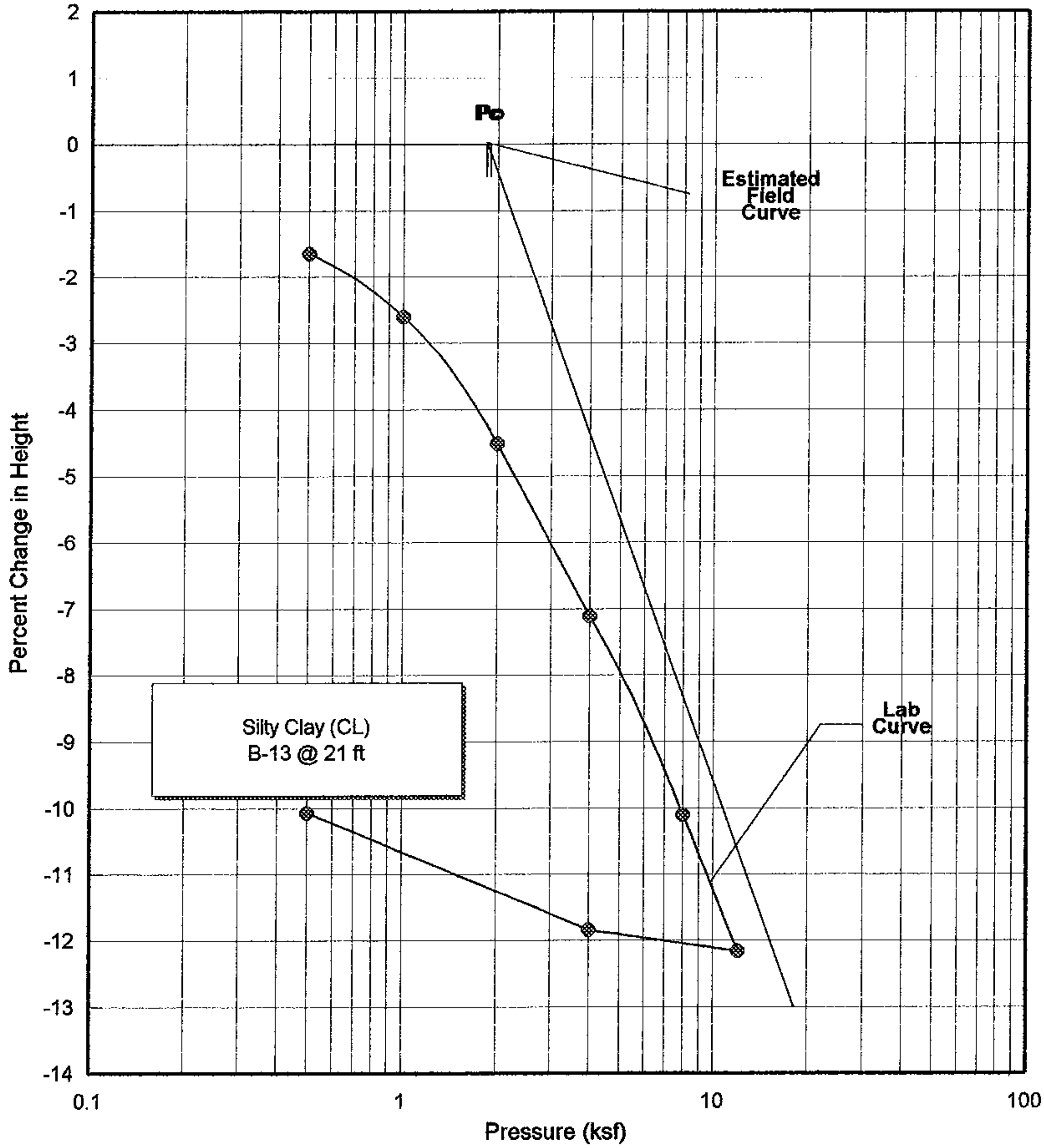
SOUTHLAND GEOTECHNICAL

Project No: S93211

Consolidation Test Results

Plate 35

ONE DIMENSIONAL CONSOLIDATION TEST (ASTM D2435)



Results of Test:

				Initial	Final
Overburden Pressure, Po:	1.9 ksf		Dry Density, pcf:	91.3	99.1
Preconsol. Pressure, Pc:	1.8 ksf		Water Content, %:	31.0	26.4
Compression Index, Cc:	0.243		Void Ratio, e:	0.855	0.709
Recompression. Index, Cr:	0.036		Saturation, %:	98.0	100.0

SOUTHLAND GEOTECHNICAL

Project No: S93211

Consolidation Test Results

Plate 36

SOUTHLAND GEOTECHNICAL

CLIENT: Black & Veatch
PROJECT: Imperial WTP & WWTP Improvements
JOB NO: S93211
DATE: 11/10/93

==

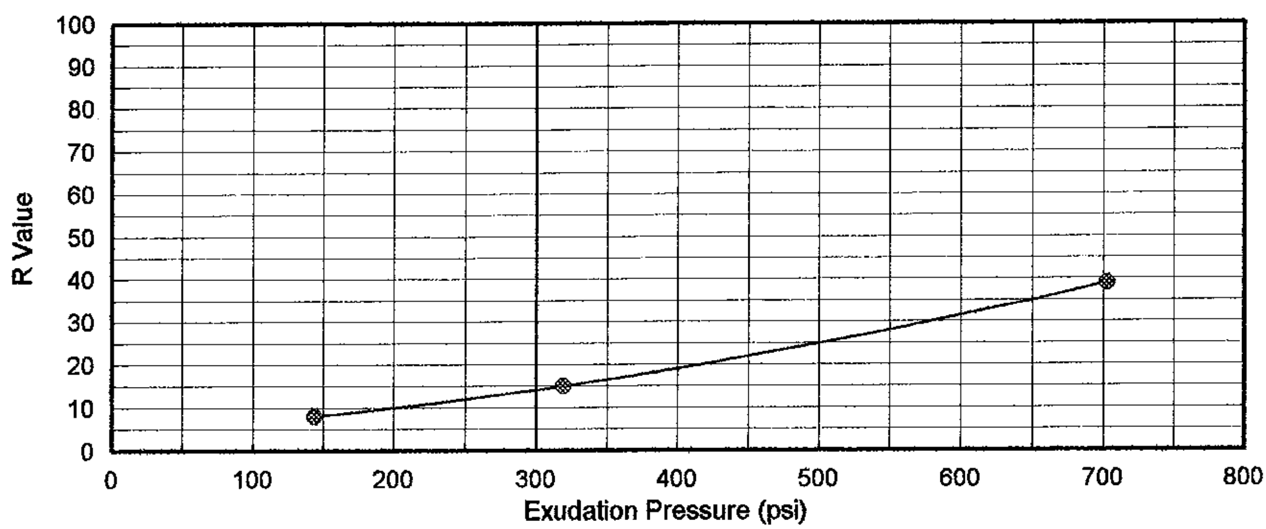
R VALUE TEST (ASTM D2844)

==

SAMPLE DESCRIPTION: Clayey Silt (ML)
SAMPLE LOCATION: B-10 @ 1-2 ft

Specimen ID:	A	B	C
Moisture Content, %:	21.6%	19.9%	18.7%
Dry Density, pcf:	102.7	110.0	112.9
Compaction foot pressure, psi:	160	300	350
Specimen Height, in.:	2.70	2.55	2.47
Stabilometer, Ph @ 1000 lb:	67	53	35
Stabilometer, Ph @ 2000 lb:	141	118	75
Displacement:	5.26	4.88	4.44
Expansion pressure, psf:	31	113	188
Exudation pressure, psi:	144	319	702
Equilibrum R Value:	8	15	39
R Value at 300 psi:		**14**	

EXUDATION PRESSURE CHART



FOUNDATION ENGINEERS AND MATERIAL LABS
Project No: S93211

R Value Test Results

Plate 37

SOUTHLAND GEOTECHNICAL

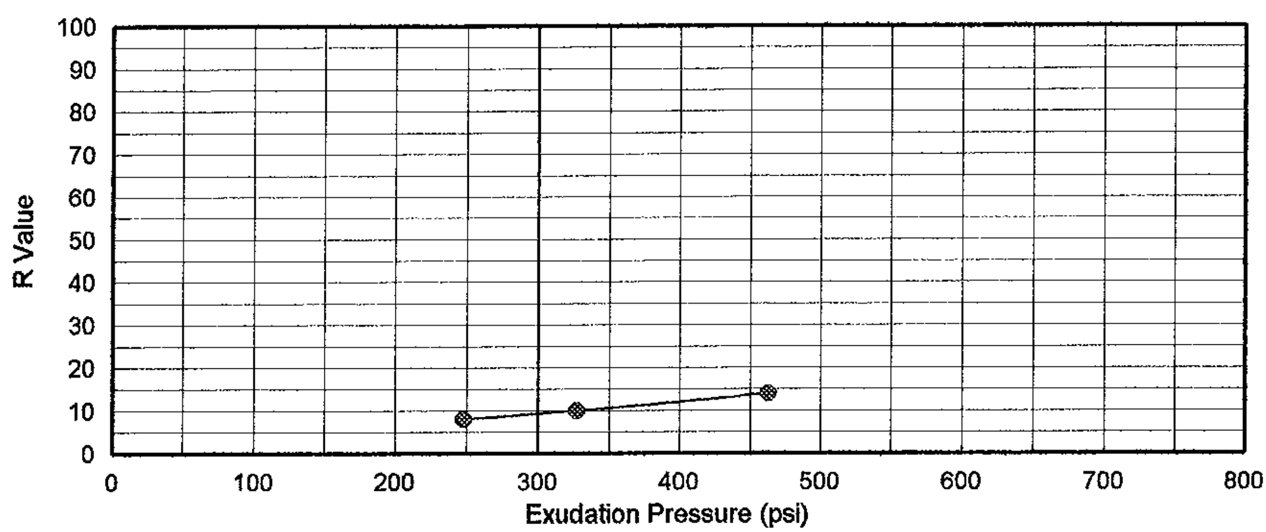
CLIENT: Black & Veatch
PROJECT: Imperial WTP & WWTP Improvements
JOB NO: S93211
DATE: 11/10/93

R VALUE TEST (ASTM D2844)

SAMPLE DESCRIPTION: Clayey Silt (ML)
SAMPLE LOCATION: B-19 @ 0-5 ft

Specimen ID:	A	B	C
Moisture Content, %:	16.4%	18.1%	19.4%
Dry Density, pcf:	112.6	110.1	106.0
Compaction foot pressure, psi:	150	110	800
Specimen Height, in.:	2.58	2.51	2.47
Stabilometer, Ph @ 1000 lb:	56	59	63
Stabilometer, Ph @ 2000 lb:	132	137	141
Displacement:	3.48	3.72	3.97
Expansion pressure, psf:	74	17	26
Exudation pressure, psi:	463	327	247
Equilibrum R Value:	14	10	8
R Value at 300 psi:		**9**	

EXUDATION PRESSURE CHART

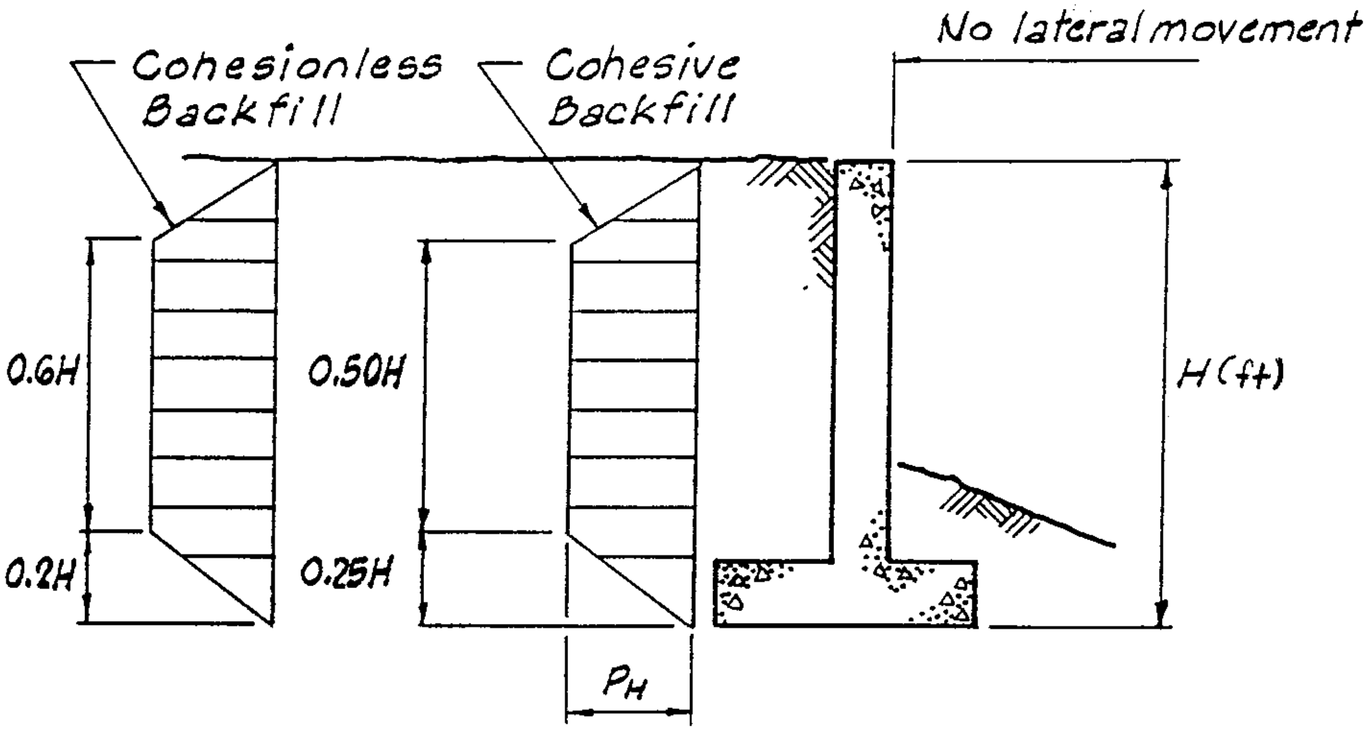


FOUNDATION ENGINEERS AND MATERIAL LABS

Project No: S93211

R Value Test Results

Plate 38



P_H = 30 H in PSF - Cohesionless Backfill
= 24 H in PSF - Cohesive Backfill

Assumed:
γ_e = 115 PCF - Cohesionless Backfill
γ_e = 120 PCF - Cohesive Backfill

Note:
These earth pressure diagrams are for temporary conditions and are applicable for computation of strut loads for a braced excavation.

SOUTHLAND GEOTECHNICAL
FOUNDATION ENGINEERS AND MATERIALS LABS
Project No: S93,211

Earth Pressure Diagram for Temporary Conditions

Plate 39

APPENDIX A

LIQUEFACTION ANALYSIS DOCUMENTATION

PROCEDURE

(1) FOR SOILS AT DEPTHS SHALLOWER THAN 10 ft, MULTIPLY MEASURED N-VALUES (N_m) BY 0.75 TO ALLOW FOR ENERGY LOSS IN THE DRIVE RODS.

(2) CONVERT N_m-VALUES TO $(N_1)_{60}$-VALUES USING EQUATION (a). C_N CORRECTION CURVES ARE SHOWN ON FIGURE 1 AND C_E CORRECTION FACTORS ARE SHOWN IN TABLE 1.

(3) FOR SANDS, SILTY SANDS, AND SILTS USE MAGNITUDE CORRELATION CURVES SHOWN ON FIGURE 2 TO DETERMINE THE STRESS RATIO NEEDED FOR LIQUEFACTION. FOR EARTHQUAKE MAGNITUDES OTHER THAN 7½, USE CORRECTION FACTORS IN TABLE 2 TO SCALE THE ABCISSA OF FIGURE 2.

(4) REDUCE THE STRESS RATIO NEEDED FOR LIQUEFACTION USING EQUATION (b) TO ALLOW FOR THE REDUCTION DUE TO INCREASED CONFINING PRESSURE. USE CURVES SHOWN ON FIGURE 3.

(5) DETERMINE THE EARTHQUAKE-INDUCED STRESS RATIO FROM FIGURE 4 AND EQUATION (c).

(6) EVALUATE THE SAFETY FACTOR AGAINST LIQUEFACTION FROM EQUATION (d).

NOTES: Figures and tables referenced are shown on Plates 42 and 43.

See Seed and others (1983), Seed (1983), Seed and others (1984), Tokimatsu and Seed (1984), and Seed and others (1985) for additional information.

EQUATIONS

$$(N_1)_{60} = C_N \cdot C_E \cdot N_m \quad \text{(a)}$$

$$\tau_\ell/\sigma_0' = \tau/\sigma_v' \cdot K_\sigma \quad \text{(b)}$$

$$\tau_{av}/\sigma_0' = 0.65 \cdot a_{max} \cdot \sigma_0/\sigma_0' \cdot r_d \quad \text{(c)}$$

$$F.S. = \tau_\ell/\sigma_0' \div \tau_{av}/\sigma_0' \quad \text{(d)}$$

$(N_1)_{60}$ = Normalized penetration resistance corrected to a delivered drill-rod energy of 60% of the theoretical free-fall energy (blows/ft)

τ_ℓ/σ_0' = Cyclic stress ratio needed to cause liquefaction

τ_{av}/σ_0' = Average earthquake-induced cyclic stress ratio

F.S. = Factor of safety against liquefaction

C_N = Depth-normalization factor (figure 1)

C_E = Correction factor to convert measured N-values to a delivered drill-rod energy of 60% of the theoretical free-fall energy (table 1)

N_m = Field SPT penetration resistance (corrected if <10 ft deep)(blows/ft)

τ/σ_v = Cyclic stress ratio needed for liquefaction (figure 2)

K_σ = Cyclic stress ratio overburden-correction-factor (figure 3)

a_{max} = Peak horizontal ground acceleration at site (g)

σ_0 = Total overburden pressure (tsf)

σ_0' = Effective overburden pressure (tsf)

r_d = Stress reduction factor (figure 4)

EMPIRICAL PREDICTION OF LIQUEFACTION POTENTIAL FOR SANDS SEED AND OTHERS (1985) METHOD

Table 1. Summary of Energy Ratios for SPT Procedures (from Tokimatsu and Seed, 1984).

	Country	Hammer Type	Hammer Release	Estimated Rod Energy (%)	Correction Factor for 60% Rod Energy
	I. JAPAN**				
	A.	Donut	Free-Fall	78	78/60 = 1.3
*	B.	Donut	Rope & Pulley with special throw release	67	67/60 = 1.12
	II. USA				
*	A.	Safety	Rope & Pulley	60	60/60 = 1.0
	B.	Donut	Rope & Pulley	60/1.33 = 45	45/60 = 0.75
	III. ARGENTINA				
	A.	Donut	Rope & Pulley	45	45/60 = 0.75
	IV. EUROPE				
*	A.	Donut	Free-Fall	60	60/60 = 1.0
	V. CHINA				
*	A.	Donut	Free-Fall	60	60/60 = 1.0
	B.	Donut	Rope & Pulley	60X 0.825 = 50	50/60 = 0.83

*Prevalent method in this country today.

**Japanese SPT results have additional corrections for borehole diameter and frequency effects.

Table 2. Scaling Factors for Effect of Earthquake Magnitude on Effective Cyclic Stress Ratio (from Tokimatsu and Seed, 1984).

Earthquake Magnitude, M (1)	Number of Representative Cycles at 0.65 τ_{max} (2)	Scaling Factor for Stress Ratio, r_m (3)
8-1/2	26	0.89
7-1/2	15	1.0
6-3/4	10	1.13
6	5	1.32
5-1/4	2-3	1.5

TABLES OF CORRECTION FACTORS FOR SEED AND OTHERS (1985) METHOD

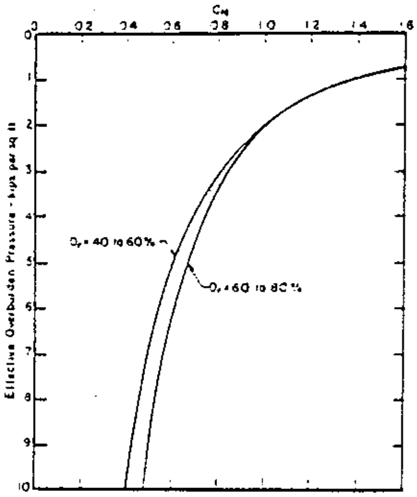


Figure 1. Recommended curves for determination of C_N (from Seed, 1983).

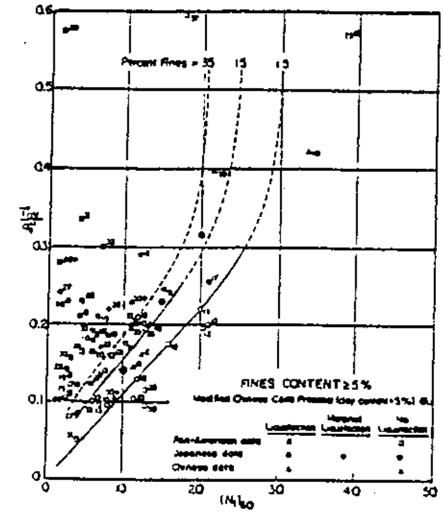


Figure 2. Relationships between stress ratio causing liquefaction and $(N_1)_{60}$ values for silty sands for magnitude 7.5 earthquakes (from Seed and others, 1985).

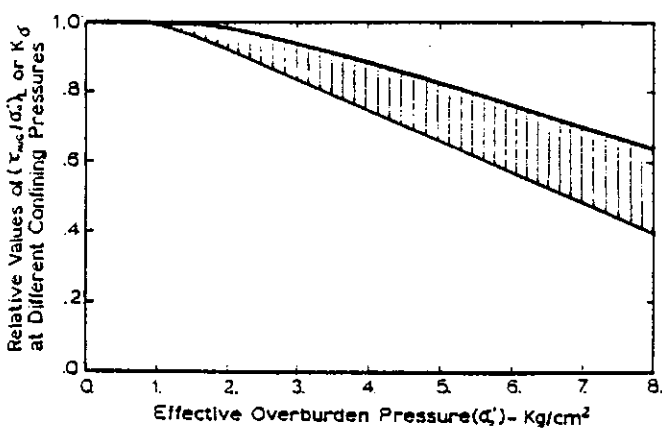


Figure 3. Typical reduction in cyclic stress ratio causing liquefaction with increase in initial confining pressure (from Seed, 1983).

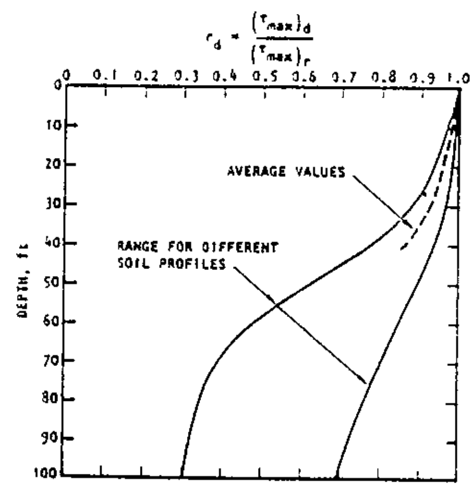
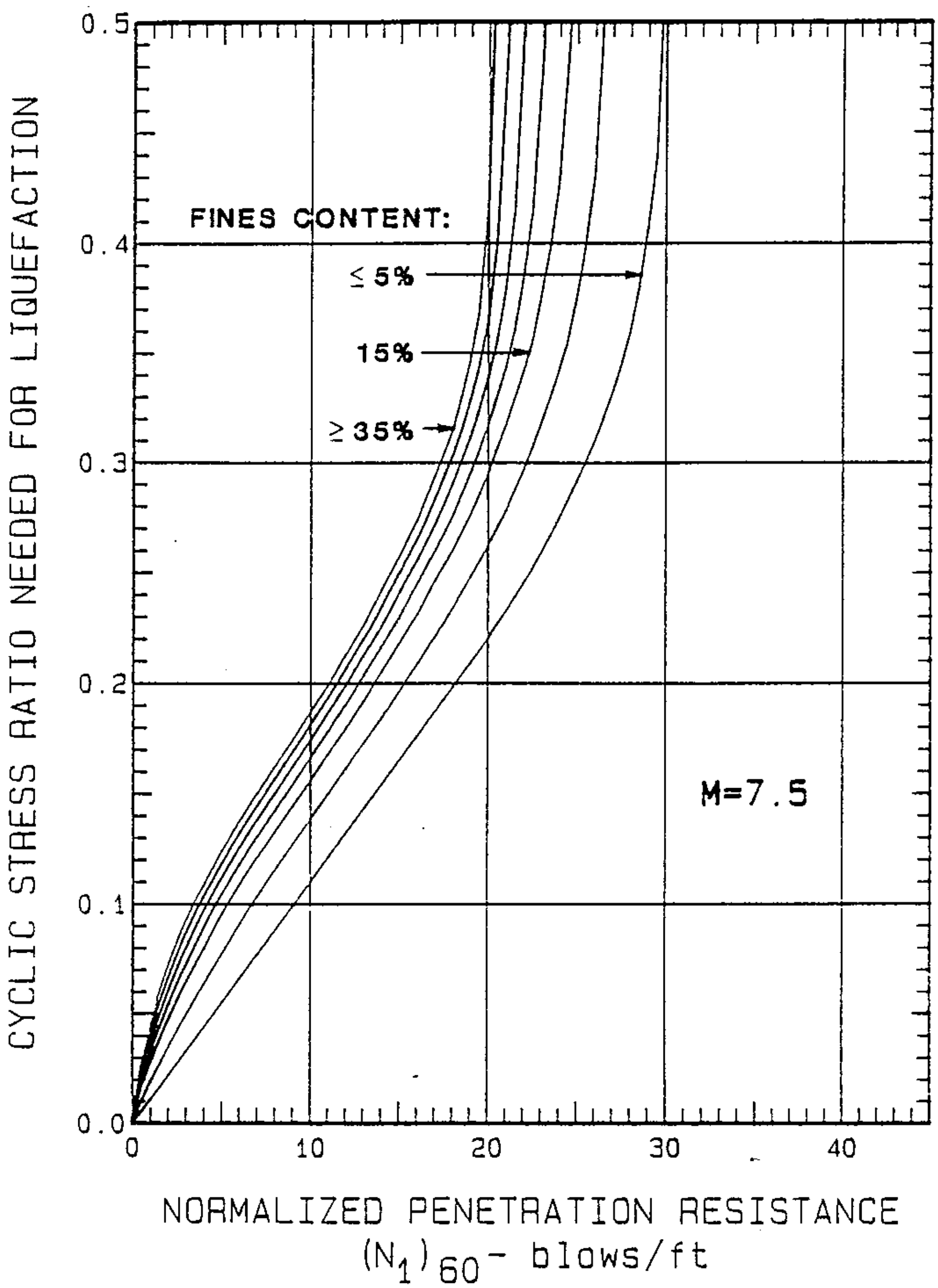


Figure 4. Range of values of r_d for different soil profiles (from Seed and Idriss, 1982).

EMPIRICAL CURVES FOR SEED AND OTHERS (1985) METHOD



DESIGN CURVES FOR EVALUATING FIELD LIQUEFACTION RESISTANCE OF SANDS UNDER LEVEL GROUND

Seed and Others (1985) Method

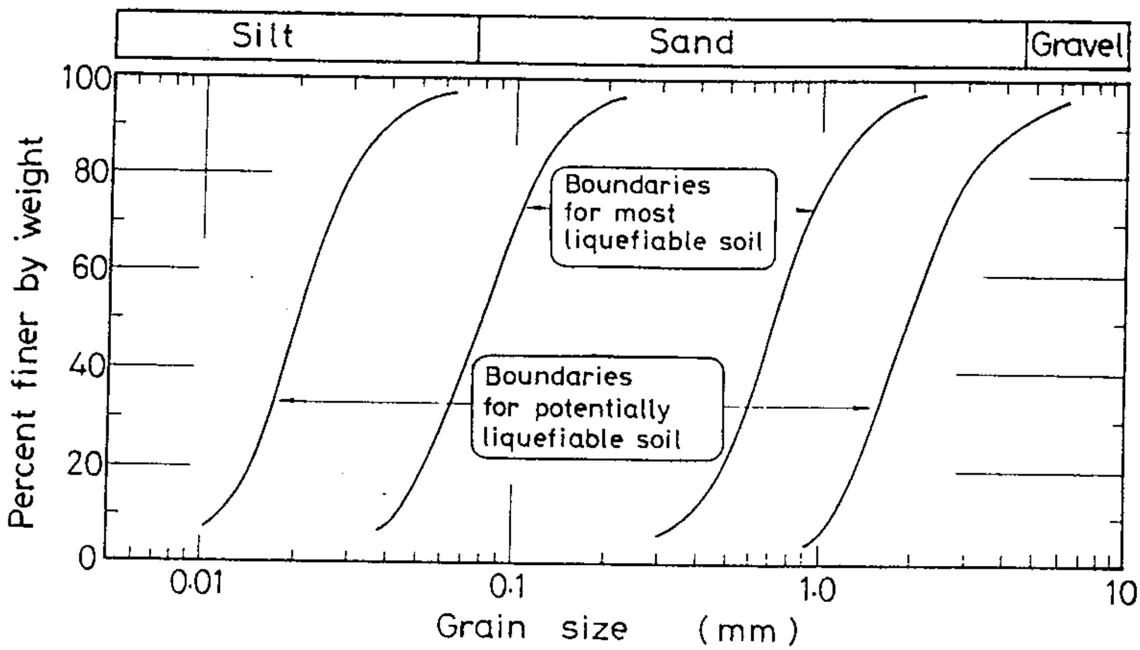


FIGURE 2-19 Limits in the gradation curves separating liquefiable and unliquefiable soils. Source: Tsuchida (1970).

```
******************
*                *
* L I Q U E F Y 2 *
*                *
******************
```

EMPIRICAL PREDICTION OF
EARTHQUAKE-INDUCED LIQUEFACTION POTENTIAL

JOB NUMBER: S93211 DATE: Friday, November 12, 1993

JOB NAME: IMPERIAL WTP & WWTP IMPROVEMENTS

LIQUEFACTION CALCULATION NAME: BORING B-2, MCE

SOIL-PROFILE NAME: S211-2

GROUND WATER DEPTH: 7.5 ft

DESIGN EARTHQUAKE MAGNITUDE: 7.20

SITE PEAK GROUND ACCELERATION: 0.600 g

K sigma BOUND: M

rd BOUND: M

N60 CORRECTION: 1.00

FIELD SPT N-VALUES < 10 FT DEEP ARE CORRECTED FOR SHORT LENGTH OF DRIVE RODS

NOTE: Relative density values listed below are estimated using equations of Giuliani and Nicoll (1982).

```
**********************
*                    *
*  SOIL PROFILE LOG  *
*                    *
**********************
```

SOIL PROFILE NAME: S211-2

LAYER #	BASE DEPTH (ft)	SPT FIELD-N (blows/ft)	LIQUEFACTION SUSCEPTIBILITY	WET UNIT WT. (pcf)	FINES %<#200	D (mm) 50	DEPTH OF SPT (ft)
1	8.0	8.0	UNSUSCEPTIBLE (0)	125.0	100.0	0.005	6.25
2	13.0	12.0	UNSUSCEPTIBLE (0)	125.0	100.0	0.005	11.25
3	18.0	8.0	UNSUSCEPTIBLE (0)	125.0	100.0	0.005	16.25
4	23.0	14.0	UNSUSCEPTIBLE (0)	125.0	100.0	0.005	21.25
5	28.0	12.0	SUSCEPTIBLE (1)	115.0	85.0	0.045	26.25
6	33.0	14.0	SUSCEPTIBLE (1)	115.0	85.0	0.045	31.25
7	38.0	17.0	UNSUSCEPTIBLE (0)	125.0	100.0	0.005	36.25
8	43.0	9.0	UNSUSCEPTIBLE (0)	125.0	100.0	0.005	41.25

LIQUEFACTION ANALYSIS SUMMARY

Seed and Others [1985] Method

SOIL NO.	CALC. DEPTH (ft)	TOTAL STRESS (tsf)	EFF. STRESS (tsf)	FIELD N (B/ft)	Est.D r (%)	C N	CORR. (N1)60 (B/ft)	LIQUE. STRESS RATIO	r d	INDUC. STRESS RATIO	LIQUE. SAFETY FACTOR
1	0.25	0.016	0.016	8	~	@	@	@	@	@	@ @
1	0.75	0.047	0.047	8	~	@	@	@	@	@	@ @
1	1.25	0.078	0.078	8	~	@	@	@	@	@	@ @
1	1.75	0.109	0.109	8	~	@	@	@	@	@	@ @
1	2.25	0.141	0.141	8	~	@	@	@	@	@	@ @
1	2.75	0.172	0.172	8	~	@	@	@	@	@	@ @
1	3.25	0.203	0.203	8	~	@	@	@	@	@	@ @
1	3.75	0.234	0.234	8	~	@	@	@	@	@	@ @
1	4.25	0.266	0.266	8	~	@	@	@	@	@	@ @
1	4.75	0.297	0.297	8	~	@	@	@	@	@	@ @
1	5.25	0.328	0.328	8	~	@	@	@	@	@	@ @
1	5.75	0.359	0.359	8	~	@	@	@	@	@	@ @
1	6.25	0.391	0.391	8	~	@	@	@	@	@	@ @
1	6.75	0.422	0.422	8	~	@	@	@	@	@	@ @
1	7.25	0.453	0.453	8	~	@	@	@	@	@	@ @
1	7.75	0.484	0.477	8	~	~	~	~	~	~	~~
2	8.25	0.516	0.492	12	~	~	~	~	~	~	~~
2	8.75	0.547	0.508	12	~	~	~	~	~	~	~~
2	9.25	0.578	0.524	12	~	~	~	~	~	~	~~
2	9.75	0.609	0.539	12	~	~	~	~	~	~	~~
2	10.25	0.641	0.555	12	~	~	~	~	~	~	~~
2	10.75	0.672	0.570	12	~	~	~	~	~	~	~~
2	11.25	0.703	0.586	12	~	~	~	~	~	~	~~
2	11.75	0.734	0.602	12	~	~	~	~	~	~	~~
2	12.25	0.766	0.617	12	~	~	~	~	~	~	~~
2	12.75	0.797	0.633	12	~	~	~	~	~	~	~~
3	13.25	0.828	0.649	8	~	~	~	~	~	~	~~
3	13.75	0.859	0.664	8	~	~	~	~	~	~	~~
3	14.25	0.891	0.680	8	~	~	~	~	~	~	~~
3	14.75	0.922	0.696	8	~	~	~	~	~	~	~~
3	15.25	0.953	0.711	8	~	~	~	~	~	~	~~
3	15.75	0.984	0.727	8	~	~	~	~	~	~	~~
3	16.25	1.016	0.743	8	~	~	~	~	~	~	~~
3	16.75	1.047	0.758	8	~	~	~	~	~	~	~~
3	17.25	1.078	0.774	8	~	~	~	~	~	~	~~
3	17.75	1.109	0.790	8	~	~	~	~	~	~	~~
4	18.25	1.141	0.805	14	~	~	~	~	~	~	~~
4	18.75	1.172	0.821	14	~	~	~	~	~	~	~~
4	19.25	1.203	0.837	14	~	~	~	~	~	~	~~
4	19.75	1.234	0.852	14	~	~	~	~	~	~	~~
4	20.25	1.266	0.868	14	~	~	~	~	~	~	~~
4	20.75	1.297	0.883	14	~	~	~	~	~	~	~~
4	21.25	1.328	0.899	14	~	~	~	~	~	~	~~
4	21.75	1.359	0.915	14	~	~	~	~	~	~	~~
4	22.25	1.391	0.930	14	~	~	~	~	~	~	~~
4	22.75	1.422	0.946	14	~	~	~	~	~	~	~~
5	23.25	1.452	0.960	12	51	0.983	11.8	0.218	0.947	0.559	0.39
5	23.75	1.481	0.974	12	51	0.983	11.8	0.218	0.946	0.561	0.39
5	24.25	1.509	0.987	12	51	0.983	11.8	0.218	0.944	0.563	0.39
5	24.75	1.538	1.000	12	51	0.983	11.8	0.218	0.943	0.566	0.38
5	25.25	1.567	1.013	12	51	0.983	11.8	0.217	0.941	0.568	0.38
5	25.75	1.596	1.026	12	51	0.983	11.8	0.217	0.939	0.569	0.38
5	26.25	1.624	1.039	12	51	0.983	11.8	0.217	0.937	0.571	0.38
5	26.75	1.653	1.053	12	51	0.983	11.8	0.217	0.934	0.572	0.38
5	27.25	1.682	1.066	12	51	0.983	11.8	0.217	0.932	0.574	0.38
5	27.75	1.711	1.079	12	51	0.983	11.8	0.217	0.930	0.575	0.38
6	28.25	1.739	1.092	14	54	0.928	13.0	0.235	0.928	0.576	0.41
6	28.75	1.768	1.105	14	54	0.928	13.0	0.234	0.926	0.577	0.41
6	29.25	1.797	1.118	14	54	0.928	13.0	0.234	0.923	0.579	0.41
6	29.75	1.826	1.131	14	54	0.928	13.0	0.234	0.921	0.580	0.40
6	30.25	1.854	1.145	14	54	0.928	13.0	0.234	0.919	0.580	0.40
6	30.75	1.883	1.158	14	54	0.928	13.0	0.234	0.916	0.581	0.40
6	31.25	1.912	1.171	14	54	0.928	13.0	0.234	0.913	0.581	0.40
6	31.75	1.941	1.184	14	54	0.928	13.0	0.234	0.910	0.582	0.40
6	32.25	1.969	1.197	14	54	0.928	13.0	0.234	0.907	0.582	0.40
6	32.75	1.998	1.210	14	54	0.928	13.0	0.234	0.904	0.582	0.40
7	33.25	2.028	1.225	17	~	~	~	~	~	~	~~
7	33.75	2.059	1.240	17	~	~	~	~	~	~	~~
7	34.25	2.091	1.256	17	~	~	~	~	~	~	~~
7	34.75	2.122	1.272	17	~	~	~	~	~	~	~~
7	35.25	2.153	1.287	17	~	~	~	~	~	~	~~
7	35.75	2.184	1.303	17	~	~	~	~	~	~	~~
7	36.25	2.216	1.319	17	~	~	~	~	~	~	~~
7	36.75	2.247	1.334	17	~	~	~	~	~	~	~~
7	37.25	2.278	1.350	17	~	~	~	~	~	~	~~
7	37.75	2.309	1.366	17	~	~	~	~	~	~	~~
8	38.25	2.341	1.381	9	~	~	~	~	~	~	~~
8	38.75	2.372	1.397	9	~	~	~	~	~	~	~~
8	39.25	2.403	1.413	9	~	~	~	~	~	~	~~
8	39.75	2.434	1.428	9	~	~	~	~	~	~	~~
8	40.25	2.466	1.444	9	~	~	~	~	~	~	~~
8	40.75	2.497	1.459	9	~	~	~	~	~	~	~~
8	41.25	2.528	1.475	9	~	~	~	~	~	~	~~
8	41.75	2.559	1.491	9	~	~	~	~	~	~	~~
8	42.25	2.591	1.506	9	~	~	~	~	~	~	~~
8	42.75	2.622	1.522	9	~	~	~	~	~	~	~~

```
******************
*                *
* L I Q U E F Y 2 *
*                *
******************
```

EMPIRICAL PREDICTION OF
EARTHQUAKE-INDUCED LIQUEFACTION POTENTIAL

JOB NUMBER: S93211 DATE: Friday, November 12, 1993

JOB NAME: IMPERIAL WTP & WWTP IMPROVEMENTS

LIQUEFACTION CALCULATION NAME: BORING B-7, MCE

SOIL-PROFILE NAME: S211-7

GROUND WATER DEPTH: 8.0 ft

DESIGN EARTHQUAKE MAGNITUDE: 7.20

SITE PEAK GROUND ACCELERATION: 0.600 g

K sigma BOUND: M

rd BOUND: M

N60 CORRECTION: 1.00

FIELD SPT N-VALUES < 10 FT DEEP ARE CORRECTED FOR SHORT LENGTH OF DRIVE RODS

NOTE: Relative density values listed below are estimated using equations of Giuliani and Nicoll (1982).

```
***********************
*                     *
*   SOIL PROFILE LOG  *
*                     *
***********************
```

SOIL PROFILE NAME: S211-7

LAYER #	BASE DEPTH (ft)	SPT FIELD-N (blows/ft)	LIQUEFACTION SUSCEPTIBILITY	WET UNIT WT. (pcf)	FINES %<#200	D (mm) 50	DEPTH OF SPT (ft)
1	8.0	8.0	UNSUSCEPTIBLE (0)	125.0	100.0	0.005	6.25
2	13.0	5.0	UNSUSCEPTIBLE (0)	125.0	100.0	0.005	11.25
3	18.0	3.0	UNSUSCEPTIBLE (0)	125.0	100.0	0.005	16.25
4	23.0	2.0	SUSCEPTIBLE (1)	120.0	76.0	0.045	21.25
5	28.0	15.0	UNSUSCEPTIBLE (0)	125.0	100.0	0.005	26.25
6	33.0	27.0	SUSCEPTIBLE (1)	120.0	76.0	0.045	31.25
7	38.0	11.0	UNSUSCEPTIBLE (0)	125.0	100.0	0.005	36.25
8	43.0	9.0	SUSCEPTIBLE (1)	120.0	85.0	0.003	41.25
9	48.0	18.0	SUSCEPTIBLE (1)	120.0	85.0	0.003	46.25
10	51.5	3.0	UNSUSCEPTIBLE (0)	125.0	100.0	0.005	51.25

LIQUEFACTION ANALYSIS SUMMARY

Seed and Others [1985] Method

SOIL NO.	CALC. DEPTH (ft)	TOTAL STRESS (tsf)	EFF. STRESS (tsf)	FIELD N (B/ft)	Est.D r (%)	C N	CORR. (N1)60 (B/ft)	LIQUE. STRESS RATIO	r d	INDUC. STRESS RATIO	LIQUE. SAFETY FACTOR
1	0.25	0.016	0.016	8	~	@	@	@	@	@	@@
1	0.75	0.047	0.047	8	~	@	@	@	@	@	@@
1	1.25	0.078	0.078	8	~	@	@	@	@	@	@@
1	1.75	0.109	0.109	8	~	@	@	@	@	@	@@
1	2.25	0.141	0.141	8	~	@	@	@	@	@	@@
1	2.75	0.172	0.172	8	~	@	@	@	@	@	@@
1	3.25	0.203	0.203	8	~	@	@	@	@	@	@@
1	3.75	0.234	0.234	8	~	@	@	@	@	@	@@
1	4.25	0.266	0.266	8	~	@	@	@	@	@	@@
1	4.75	0.297	0.297	8	~	@	@	@	@	@	@@
1	5.25	0.328	0.328	8	~	@	@	@	@	@	@@
1	5.75	0.359	0.359	8	~	@	@	@	@	@	@@
1	6.25	0.391	0.391	8	~	@	@	@	@	@	@@
1	6.75	0.422	0.422	8	~	@	@	@	@	@	@@
1	7.25	0.453	0.453	8	~	@	@	@	@	@	@@
1	7.75	0.484	0.484	8	~	@	@	@	@	@	@@
2	8.25	0.516	0.508	5	~	~	~	~	~	~	~~
2	8.75	0.547	0.523	5	~	~	~	~	~	~	~~
2	9.25	0.578	0.539	5	~	~	~	~	~	~	~~
2	9.75	0.609	0.555	5	~	~	~	~	~	~	~~
2	10.25	0.641	0.570	5	~	~	~	~	~	~	~~
2	10.75	0.672	0.586	5	~	~	~	~	~	~	~~
2	11.25	0.703	0.602	5	~	~	~	~	~	~	~~
2	11.75	0.734	0.617	5	~	~	~	~	~	~	~~
2	12.25	0.766	0.633	5	~	~	~	~	~	~	~~
2	12.75	0.797	0.649	5	~	~	~	~	~	~	~~
3	13.25	0.828	0.664	3	~	~	~	~	~	~	~~
3	13.75	0.859	0.680	3	~	~	~	~	~	~	~~
3	14.25	0.891	0.696	3	~	~	~	~	~	~	~~
3	14.75	0.922	0.711	3	~	~	~	~	~	~	~~
3	15.25	0.953	0.727	3	~	~	~	~	~	~	~~
3	15.75	0.984	0.743	3	~	~	~	~	~	~	~~
3	16.25	1.016	0.758	3	~	~	~	~	~	~	~~
3	16.75	1.047	0.774	3	~	~	~	~	~	~	~~
3	17.25	1.078	0.790	3	~	~	~	~	~	~	~~
3	17.75	1.109	0.805	3	~	~	~	~	~	~	~~
4	18.25	1.140	0.820	2	21	1.053	2.1	0.077	0.961	0.521	0.15
4	18.75	1.170	0.835	2	21	1.053	2.1	0.077	0.960	0.525	0.15
4	19.25	1.200	0.849	2	21	1.053	2.1	0.077	0.959	0.529	0.15
4	19.75	1.230	0.863	2	21	1.053	2.1	0.077	0.958	0.532	0.14
4	20.25	1.260	0.878	2	21	1.053	2.1	0.077	0.957	0.536	0.14
4	20.75	1.290	0.892	2	21	1.053	2.1	0.077	0.955	0.539	0.14
4	21.25	1.320	0.907	2	21	1.053	2.1	0.077	0.954	0.542	0.14
4	21.75	1.350	0.921	2	21	1.053	2.1	0.077	0.952	0.544	0.14
4	22.25	1.380	0.935	2	21	1.053	2.1	0.077	0.951	0.547	0.14
4	22.75	1.410	0.950	2	21	1.053	2.1	0.077	0.949	0.549	0.14
5	23.25	1.441	0.965	15	~	~	~	~	~	~	~~
5	23.75	1.472	0.980	15	~	~	~	~	~	~	~~
5	24.25	1.503	0.996	15	~	~	~	~	~	~	~~
5	24.75	1.534	1.012	15	~	~	~	~	~	~	~~
5	25.25	1.566	1.027	15	~	~	~	~	~	~	~~
5	25.75	1.597	1.043	15	~	~	~	~	~	~	~~
5	26.25	1.628	1.059	15	~	~	~	~	~	~	~~
5	26.75	1.659	1.074	15	~	~	~	~	~	~	~~
5	27.25	1.691	1.090	15	~	~	~	~	~	~	~~
5	27.75	1.722	1.106	15	~	~	~	~	~	~	~~
6	28.25	1.753	1.121	27	74	0.925	25.0	Infin	0.928	0.566	Infin
6	28.75	1.783	1.135	27	74	0.925	25.0	Infin	0.926	0.567	Infin
6	29.25	1.813	1.149	27	74	0.925	25.0	Infin	0.923	0.568	Infin
6	29.75	1.843	1.164	27	74	0.925	25.0	Infin	0.921	0.569	Infin
6	30.25	1.873	1.178	27	74	0.925	25.0	Infin	0.919	0.569	Infin
6	30.75	1.903	1.193	27	74	0.925	25.0	Infin	0.916	0.570	Infin
6	31.25	1.933	1.207	27	74	0.925	25.0	Infin	0.913	0.570	Infin
6	31.75	1.963	1.221	27	74	0.925	25.0	Infin	0.910	0.570	Infin
6	32.25	1.993	1.236	27	74	0.925	25.0	Infin	0.907	0.570	Infin
6	32.75	2.023	1.250	27	74	0.925	25.0	Infin	0.904	0.571	Infin
7	33.25	2.053	1.265	11	~	~	~	~	~	~	~~
7	33.75	2.084	1.281	11	~	~	~	~	~	~	~~
7	34.25	2.116	1.297	11	~	~	~	~	~	~	~~
7	34.75	2.147	1.312	11	~	~	~	~	~	~	~~
7	35.25	2.178	1.328	11	~	~	~	~	~	~	~~
7	35.75	2.209	1.344	11	~	~	~	~	~	~	~~
7	36.25	2.241	1.359	11	~	~	~	~	~	~	~~
7	36.75	2.272	1.375	11	~	~	~	~	~	~	~~
7	37.25	2.303	1.391	11	~	~	~	~	~	~	~~
7	37.75	2.334	1.406	11	~	~	~	~	~	~	~~
8	38.25	2.365	1.421	9	40	0.812	7.3	0.155	0.866	0.562	0.28
8	38.75	2.395	1.436	9	40	0.812	7.3	0.155	0.862	0.561	0.28
8	39.25	2.425	1.450	9	40	0.812	7.3	0.155	0.858	0.560	0.28
8	39.75	2.455	1.464	9	40	0.812	7.3	0.155	0.854	0.559	0.28
8	40.25	2.485	1.479	9	40	0.812	7.3	0.155	0.850	0.557	0.28
8	40.75	2.515	1.493	9	40	0.812	7.3	0.155	0.845	0.555	0.28
8	41.25	2.545	1.508	9	40	0.812	7.3	0.155	0.840	0.553	0.28
8	41.75	2.575	1.522	9	40	0.812	7.3	0.155	0.836	0.551	0.28
8	42.25	2.605	1.536	9	40	0.812	7.3	0.154	0.831	0.549	0.28
8	42.75	2.635	1.551	9	40	0.812	7.3	0.154	0.826	0.547	0.28
9	43.25	2.665	1.565	18	56	0.770	13.9	0.244	0.821	0.545	0.45
9	43.75	2.695	1.580	18	56	0.770	13.9	0.243	0.816	0.543	0.45
9	44.25	2.725	1.594	18	56	0.770	13.9	0.243	0.811	0.541	0.45
9	44.75	2.755	1.608	18	56	0.770	13.9	0.243	0.806	0.539	0.45
9	45.25	2.785	1.623	18	56	0.770	13.9	0.243	0.801	0.536	0.45
9	45.75	2.815	1.637	18	56	0.770	13.9	0.243	0.796	0.534	0.45
9	46.25	2.845	1.652	18	56	0.770	13.9	0.242	0.791	0.531	0.46
9	46.75	2.875	1.666	18	56	0.770	13.9	0.242	0.786	0.529	0.46
9	47.25	2.905	1.680	18	56	0.770	13.9	0.242	0.781	0.526	0.46
9	47.75	2.935	1.695	18	56	0.770	13.9	0.242	0.776	0.524	0.46

```
******************
*                *
* L I Q U E F Y 2 *
*                *
******************
```

EMPIRICAL PREDICTION OF
EARTHQUAKE-INDUCED LIQUEFACTION POTENTIAL

JOB NUMBER: S93211 DATE: Friday, November 12, 1993

JOB NAME: IMPERIAL WTP & WWTP IMPROVEMENTS

LIQUEFACTION CALCULATION NAME: BORING B-12, MCE

SOIL-PROFILE NAME: S211-12

GROUND WATER DEPTH: 9.0 ft

DESIGN EARTHQUAKE MAGNITUDE: 7.20

SITE PEAK GROUND ACCELERATION: 0.600 g

K sigma BOUND: M

rd BOUND: M

N60 CORRECTION: 1.00

FIELD SPT N-VALUES < 10 FT DEEP ARE CORRECTED FOR SHORT LENGTH OF DRIVE RODS

NOTE: Relative density values listed below are estimated using equations of Giuliani and Nicoll (1982).

```
************************
*                      *
*   SOIL PROFILE LOG   *
*                      *
************************
```

SOIL PROFILE NAME: S211-12

LAYER #	BASE DEPTH (ft)	SPT FIELD-N (blows/ft)	LIQUEFACTION SUSCEPTIBILITY	WET UNIT WT. (pcf)	FINES %<#200	D (mm) 50	DEPTH OF SPT (ft)
1	8.0	9.0	UNSUSCEPTIBLE (0)	125.0	100.0	0.005	6.25
2	13.0	14.0	SUSCEPTIBLE (1)	120.0	86.0	0.030	11.25
3	18.0	2.0	SUSCEPTIBLE (1)	120.0	86.0	0.030	16.25

LIQUEFACTION ANALYSIS SUMMARY

Seed and Others [1985] Method

SOIL NO.	CALC. DEPTH (ft)	TOTAL STRESS (tsf)	EFF. STRESS (tsf)	FIELD N (B/ft)	Est.D_r (%)	C_N	CORR. (N1)60 (B/ft)	LIQUE. STRESS RATIO	r_d	INDUC. STRESS RATIO	LIQUE. SAFETY FACTOR
1	0.25	0.016	0.016	9	~	@	@	@	@	@	@ @
1	0.75	0.047	0.047	9	~	@	@	@	@	@	@ @
1	1.25	0.078	0.078	9	~	@	@	@	@	@	@ @
1	1.75	0.109	0.109	9	~	@	@	@	@	@	@ @
1	2.25	0.141	0.141	9	~	@	@	@	@	@	@ @
1	2.75	0.172	0.172	9	~	@	@	@	@	@	@ @
1	3.25	0.203	0.203	9	~	@	@	@	@	@	@ @
1	3.75	0.234	0.234	9	~	@	@	@	@	@	@ @
1	4.25	0.266	0.266	9	~	@	@	@	@	@	@ @
1	4.75	0.297	0.297	9	~	@	@	@	@	@	@ @
1	5.25	0.328	0.328	9	~	@	@	@	@	@	@ @
1	5.75	0.359	0.359	9	~	@	@	@	@	@	@ @
1	6.25	0.391	0.391	9	~	@	@	@	@	@	@ @
1	6.75	0.422	0.422	9	~	@	@	@	@	@	@ @
1	7.25	0.453	0.453	9	~	@	@	@	@	@	@ @
1	7.75	0.484	0.484	9	~	@	@	@	@	@	@ @
2	8.25	0.515	0.515	14	61	@	@	@	@	@	@ @
2	8.75	0.545	0.545	14	61	@	@	@	@	@	@ @
2	9.25	0.575	0.567	14	61	1.264	17.7	0.331	0.981	0.388	0.85
2	9.75	0.605	0.582	14	61	1.264	17.7	0.331	0.980	0.398	0.83
2	10.25	0.635	0.596	14	61	1.264	17.7	0.331	0.979	0.407	0.81
2	10.75	0.665	0.610	14	61	1.264	17.7	0.331	0.978	0.415	0.80
2	11.25	0.695	0.625	14	61	1.264	17.7	0.331	0.977	0.424	0.78
2	11.75	0.725	0.639	14	61	1.264	17.7	0.331	0.976	0.432	0.77
2	12.25	0.755	0.654	14	61	1.264	17.7	0.331	0.975	0.439	0.75
2	12.75	0.785	0.668	14	61	1.264	17.7	0.331	0.973	0.446	0.74
3	13.25	0.815	0.682	2	22	1.145	2.3	0.081	0.972	0.453	0.18
3	13.75	0.845	0.697	2	22	1.145	2.3	0.081	0.971	0.459	0.18
3	14.25	0.875	0.711	2	22	1.145	2.3	0.081	0.970	0.466	0.17
3	14.75	0.905	0.726	2	22	1.145	2.3	0.081	0.969	0.471	0.17
3	15.25	0.935	0.740	2	22	1.145	2.3	0.081	0.968	0.477	0.17
3	15.75	0.965	0.754	2	22	1.145	2.3	0.081	0.967	0.482	0.17
3	16.25	0.995	0.769	2	22	1.145	2.3	0.081	0.966	0.487	0.17
3	16.75	1.025	0.783	2	22	1.145	2.3	0.081	0.965	0.492	0.16
3	17.25	1.055	0.798	2	22	1.145	2.3	0.081	0.964	0.497	0.16
3	17.75	1.085	0.812	2	22	1.145	2.3	0.081	0.962	0.502	0.16

SOUTHLAND GEOTECHNICAL

ESTIMATION OF LIQUEFACTION AND INDUCED SETTLEMENT
FROM MAX. CREDIBLE EARTHQUAKE

Project:IMPERIAL WTP EXPANSION
Job No:S93211
Date:11/12/93

M: 7.2 | Ce: 1.00 (Conversion to N60) | 1 ft
PGA: 0.60 g | GWT: 7.5 ft | 0.0 (in radii or half width)
Boring:B-2 | rn: 1.05 | 0 psf
1

Methods:Liquefaction Analysis from Seed and Others (1985) and LIQUEFY2
Settlements from Tokimatsu and Seed (1987), ASCE GT Vol 113, No.8

Layer	Base Depth (ft)	Field N (bpf)	Suscept	Total Unit Wt (pcf)	% Fines	D50	SPT Depth (ft)	Effect. P'o (tsf)	Layer Thick. (ft)	rd	Induced CSR	Corr N'60	Est Dr	Level Threshold CSR	Corr CSR	SF	Volumetric Strain (%)	Liquifeable Layer,ft	Settlement (in.)	Note
1	8.0	8	0	125	100	0.005	6.25	0.391	8.0	0.982	0.383	8.1	N/A	N/A	N/A	N/A	0.0	N/A	0.00	
2	13.0	12	0	125	100	0.005	11.3	0.586	5.0	0.968	0.453	14.8	N/A	N/A	N/A	N/A	0.0	N/A	0.00	
3	18.0	8	0	125	100	0.005	16.3	0.743	5.0	0.954	0.509	9.0	N/A	N/A	N/A	N/A	0.0	N/A	0.00	
4	23.0	14	0	125	100	0.005	21.3	0.899	5.0	0.939	0.541	15.1	N/A	N/A	N/A	N/A	0.0	N/A	0.00	
5	28.0	12	1	115	85	0.045	26.3	1.039	5.0	0.925	0.564	12.0	51%	0.217	0.217	0.38	2.3	5.0	1.38	
6	33.0	14	1	115	85	0.045	31.3	1.171	5.0	0.911	0.580	13.6	54%	0.234	0.234	0.40	2.1	5.0	1.26	
7	38.0	17	0	125	100	0.005	36.3	1.319	5.0	0.885	0.580	15.5	N/A	N/A	N/A	N/A	0.0	N/A	0.00	
8	43.0	9	0	125	100	0.005	41.3	1.475	5.0	0.825	0.551	8.0	N/A	N/A	N/A	N/A	0.0	N/A	0.00	
9								N/A	N/A	1.000	N/A	N/A	N/A	N/A	N/A	N/A	0.0	N/A	0.00	
10								N/A	N/A	1.000	N/A	N/A	N/A	N/A	N/A	N/A	0.0	N/A	0.00	

Total: 2.64 in.

M: 7.2 | Ce: 1.00 (Conversion to N60) | 1 ft
PGA: 0.60 g | GWT: 8.0 ft | 0.0 (in radii or half width)
Boring:B-7 | rn: 1.05 | 0 psf
1

Methods:Liquefaction Analysis from Seed and Others (1985) and LIQUEFY2
Settlements from Tokimatsu and Seed (1987), ASCE GT Vol 113, No.8

Layer	Base Depth (ft)	Field N (bpf)	Suscept	Total Unit Wt (pcf)	% Fines	D50	SPT Depth (ft)	Effect. P'o (tsf)	Layer Thick. (ft)	rd	Induced CSR	Corr N'60	Est Dr	Level Threshold CSR	Corr CSR	SF	Volumetric Strain (%)	Liquifeable Layer,ft	Settlement (in.)	Note
1	8.0	8	0	125	100	0.005	6.25	0.391	8.0	0.982	0.383	8.1	N/A	N/A	N/A	N/A	0.0	N/A	0.00	
2	13.0	5	0	125	100	0.005	11.3	0.602	5.0	0.968	0.441	5.9	N/A	N/A	N/A	N/A	0.0	N/A	0.00	
3	18.0	3	0	125	100	0.005	16.3	0.758	5.0	0.954	0.498	3.4	N/A	N/A	N/A	N/A	0.0	N/A	0.00	
4	23.0	2	1	120	76	0.045	21.3	0.907	5.0	0.939	0.533	2.1	21%	0.077	0.077	0.14	6.0	5.0	3.60	
5	28.0	15	0	125	100	0.005	26.3	1.059	5.0	0.925	0.555	15.0	N/A	N/A	N/A	N/A	0.0	N/A	0.00	
6	33.0	27	1	120	76	0.045	31.3	1.192	5.0	0.911	0.576	25.4	74%	#################			0.0	0.0	0.00	
7	38.0	11	0	125	100	0.005	36.3	1.344	5.0	0.885	0.576	10.1	N/A	N/A	N/A	N/A	0.0	N/A	0.00	
8	43.0	9	1	120	85	0.003	41.3	1.492	5.0	0.825	0.549	7.8	40%	0.155	0.158	0.29	2.9	5.0	1.74	
9	48.0	18	1	120	85	0.003	46.3	1.636	5.0	0.765	0.519	15.2	56%	0.242	0.242	0.47	2.0	5.0	1.20	
10	51.5	3	0	125	100	0.005	51.3	1.788	3.5	0.705	0.485	2.4	N/A	N/A	N/A	N/A	0.0	N/A	0.00	

Total: 6.54 in.

M: 7.2 | Ce: 1.00 (Conversion to N60) | 1 ft
PGA: 0.60 g | GWT: 9.0 ft | 0.0 (in radii or half width)
Boring:B-12 | rn: 1.05 | 1 psf
1

Methods:Liquefaction Analysis from Seed and Others (1985) and LIQUEFY2
Settlements from Tokimatsu and Seed (1987), ASCE GT Vol 113, No.8

Layer	Base Depth (ft)	Field N (bpf)	Suscept	Total Unit Wt (pcf)	% Fines	D50	SPT Depth (ft)	Effect. P'o (tsf)	Layer Thick. (ft)	rd	Induced CSR	Corr N'60	Est Dr	Level Threshold CSR	Corr CSR	SF	Volumetric Strain (%)	Liquifeable Layer,ft	Settlement (in.)	Note
1	8.0	9	0	125	100	0.005	6.25	0.391	8.0	0.982	0.383	9.1	N/A	N/A	N/A	N/A	0.0	0.0	0.00	
2	13.0	14	1	120	86	0.030	11.3	0.625	5.0	0.968	0.420	16.4	61%	0.331	0.331	0.79	1.7	0.0	1.02	
3	18.0	2	1	120	86	0.030	16.3	0.769	5.0	0.954	0.481	2.2	22%	0.081	0.081	0.17	6.0	0.0	3.60	
4								N/A	N/A	1.000	N/A	N/A	N/A	N/A	N/A	N/A	0.0	N/A	0.00	
5								N/A	N/A	1.000	N/A	N/A	N/A	N/A	N/A	N/A	0.0	N/A	0.00	
6								N/A	N/A	1.000	N/A	N/A	N/A	N/A	N/A	N/A	0.0	N/A	0.00	
7								N/A	N/A	1.000	N/A	N/A	N/A	N/A	N/A	N/A	0.0	0.0	0.00	
8								N/A	N/A	1.000	N/A	N/A	N/A	N/A	N/A	N/A	0.0	0.0	0.00	

Total: 4.62 in.

EVALUATION OF SETTLEMENTS IN SANDS DUE TO EARTHQUAKE SHAKING

By Kohji Tokimatsu, A. M. ASCE, and
H. Bolton Seed, Hon. M. ASCE

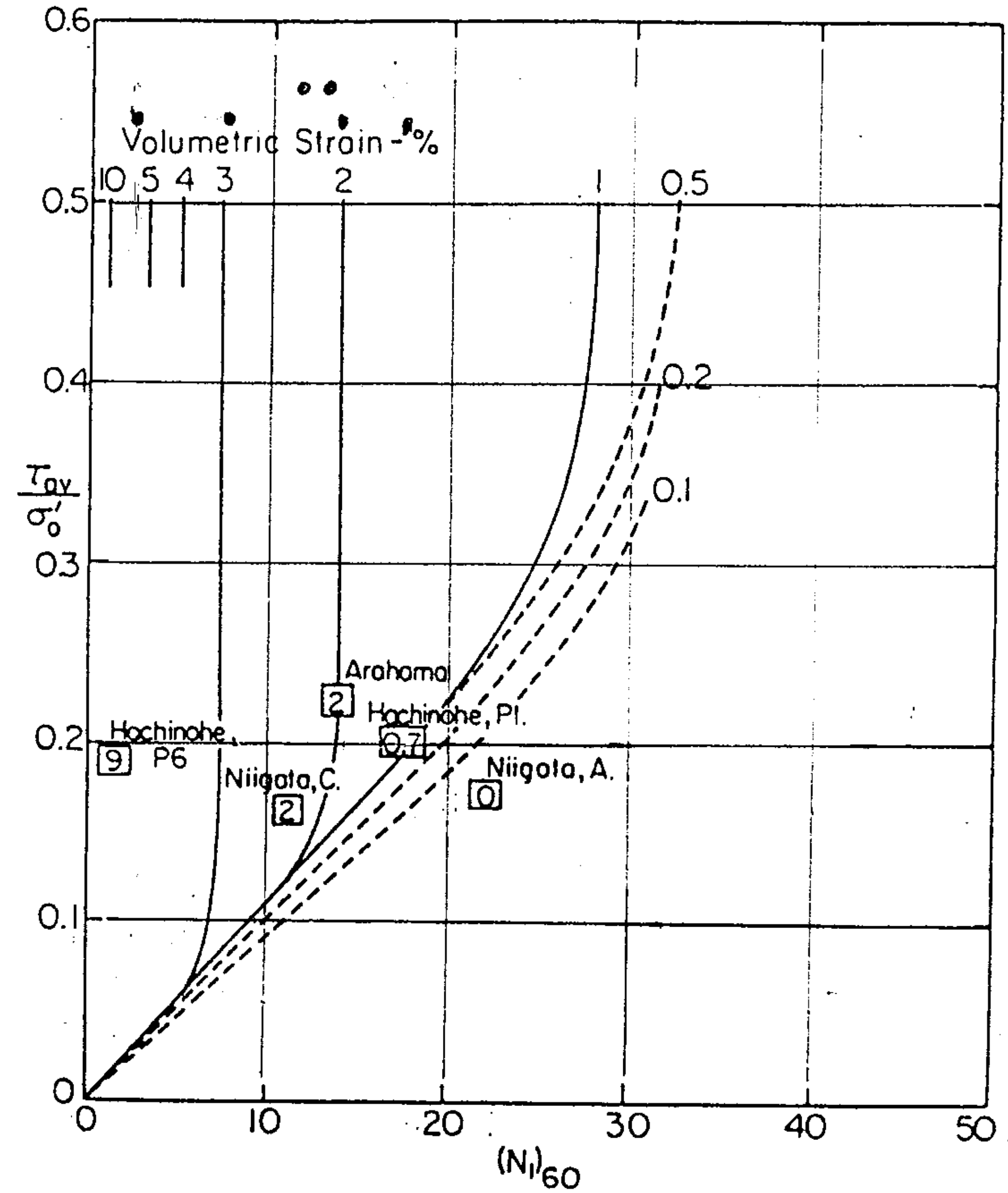


FIG. 9.—Comparison of Proposed Chart for Determination of Volumetric Strain with Field Performance of Saturated Sands

APPENDIX B

REFERENCES

1. Working Group on California Earthquake Probabilities, (1988), *Probabilities of Large Earthquakes Occurring in California on the San Andreas Fault,* U.S. Geological Survey Open-File Report 88-398.

2. Joyner, William B. and Boore, David M., 1988, *Measurements, Characterization, and Prediction of Strong Ground Motion.*

3. SEAOC (Structural Engineers Association of California), 1990, *Recommended Lateral Force Requirements and Commentary.*

4. ICBO (International Conference of Building Officials) (1988), *Uniform Building Code, 1988 Edition.*

5. U.S. Geological Survey, 1982, Professional Paper 1254, *The Imperial Valley California Earthquake of October 15, 1979.*

6. U.S. Geological Survey, 1987, *Open File Report 87-672, Strong Motion Data from the Superstition Hills Earthquake of November 24, 1987.*

7. California Division of Mines and Geology, *1977 County Report No. 7, Geology and Mineral Resources of Imperial County, California.*

8. Personal Communication - Robert Sharp, USGS, Menlo Park, CA in July 1989.

9. Blake, Thomas, F., 1986 *LIQUEFY 2, A computer program for the Empirical Prediction of Earthquake-Induced Liquefaction Potential.*

10. Tokimatsu and Seed, 1987, ASCE Geotechnical Journal, Vol. 113, No. 8.

11. Committee of Earthquake Engineering, National Research Council, 1985, *Liquefaction of Soils During Earthquakes.*