

Appendix B: Geology and Geotechnical Investigation

DICA Partners Water Conveyance Project

GEOTECHNICAL INVESTIGATION
PROPOSED WASTEWAY NO. 1 CROSSING
& CANAL OUTLET
SOUTH OF COACHELLA CANAL
NORTHSORE AREA
RIVERSIDE COUNTY, CALIFORNIA

-Prepared By-

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Subject: Geotechnical Investigation

Project: Proposed Wasteway No. 1 Crossing
& Canal Outlet
South of Coachella Canal
Northshore Area
Riverside County, California

Sladden Engineering is pleased to present the results of the geotechnical investigation performed for the Wasteway No. 1 crossing and canal outlet proposed for the future pipeline alignment to be located on the southside of the Coachella Canal between Arthur Street to approximately ½ mile east of Cleveland Street in the Northshore area of Riverside County, California. Our services were completed in accordance with our proposal for geotechnical engineering services dated February 6, 2018 and your authorization to proceed with the work. The purpose of our investigation was to explore the subsurface conditions at the site in order to provide recommendations for the construction of the Wasteway No. 1 bridge crossing and canal outlet. Evaluation of environmental issues and hazardous wastes was not included within the scope of services provided.

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

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

Respectfully submitted,
SLADDEN ENGINEERING

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INTRODUCTION

This report presents the results of the geotechnical investigation performed by Sladden Engineering (Sladden) for the Wasteway No. 1 crossing and canal outlet proposed in conjunction with the future irrigation pipeline alignment located on the southside of the Coachella Canal between Arthur Street to approximately ½ mile east of Cleveland Street in the Northshore area of Riverside County, California. Generally, the proposed pipeline will follow the existing alignment of Riddle Way and Coachella Canal Road. The proposed bridge and pipeline crossings will span Wasteway No. 1 located immediately east of Cleveland Street. The proposed canal outlet will be constructed approximately 2,500 feet east of Cleveland Street and just south of the Coachella Canal. The approximate location of the project alignment is indicated on the Site Location Map (Figure 1).

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

PROJECT DESCRIPTION

Based on the provided project plans (Olson, 2017a & 2017b), it is our understanding that the proposed project will consist of constructing a new bridge that will span Wasteway No 1 immediately adjacent to Cleveland Street. Based on Coachella Valley Water District's (CVWD, 2018) design guidelines, it is our understanding that the proposed bridge structure will consist of trapezoidal type reinforced concrete construction that will span approximately 43 feet. The new bridge will consist of a high strength (4,000 psi), 22-inch thick, reinforced concrete structure in accordance with CVWD design guidelines. The new bridge will be supported by a trapezoidal reinforced concrete insert.

A new irrigation line will be constructed to the immediate north of bridge structure. The irrigation line will consist of an 18-inch steel pipe that will span Wasteway No. 1 for approximately 47 feet. The new irrigation line will be supported within the existing canal.

The project will also include a canal outlet structure to be located just south of the Coachella Canal. The outlet structure will be used to supply water for a future irrigation reservoir.

Sladden anticipates that grading will consist of generally minor cuts and fills to attain the design grades. Upon completion of the project plans, Sladden should be retained in order to ensure that the recommendations presented within in this report are incorporated into the design of the proposed project.

SCOPE OF SERVICES

The purpose of our investigation was to determine specific engineering characteristics of the surface and near surface soil in order to develop foundation design criteria and recommendations for site preparation. Exploration of the site was achieved by drilling two (2) exploratory boreholes to depths of approximately 26 feet below the existing ground surface (bgs). Specifically, our site characterization consisted of the following tasks:

- Site reconnaissance to assess the existing surface conditions on and adjacent to the site.
- Advancing two (2) exploratory boreholes to depths of approximately 26 feet bgs in order to characterize the subsurface soil conditions. Representative samples of the soil were classified in the field and retained for laboratory testing and engineering analyses.
- Performing laboratory testing on selected samples to evaluate their engineering characteristics.
- Reviewing geologic literature and discussing geologic hazards.
- Performing engineering analyses to develop recommendations for foundation design and site preparation.
- The preparation of this report summarizing our work at the site.

SITE CONDITIONS

The subject alignment is located within the northern portion of Section 18 and Section 17, Township 7 South, Range 10 East, San Bernardino Base Meridian. The proposed pipeline alignment is situated just south of the Coachella Canal and generally follows two dirt roads (Riddle Road and Coachella Canal Road) between Arthur Street to approximately ½ mile east of Cleveland Street. The alignment is undeveloped and bounded by agricultural properties to the west, undeveloped land to the south and east, the Coachella Canal to the north. Wasteway No 1 and a USBR right-of-way are located near the eastern portion of Section 18.

Wasteway No. 1 is approximately 39 feet in total width and consists of 1.5 horizontal to 1 vertical (1.5H: 1V) concrete lined side slopes with a 12 foot wide concrete lined bottom. The wasteway is approximately 9 feet in depth and is located east and parallel to the Cleveland Street alignment. USBR easements extend approximately 80 feet east and 90 feet west of the wasteway (measured from centerline).

The proposed canal outlet structure will be located approximately 2,500 feet east of Cleveland Street and just south of the Coachella Canal. The Coachella Canal at the proposed location consists of concrete lined earthen embankments constructed at approximately 1.5H: 1V and less. Minor erosion in the form of shallowing incised rills are present along the embankment face.

Based on our review of the USGS (2015), the elevations range from approximately from 20 feet to 30 feet above mean sea level (MSL).

No natural ponding of water or surface seeps were observed at or near the site during our investigation conducted on February 9, 2018. Site drainage appears to be controlled by sheet flow, surface infiltration and through south trending ephemeral water courses.

GEOLOGIC SETTING

The project site is located within the Colorado Desert Physiographic Province (also referred to as the Salton Trough) that is characterized as a northwest-southeast trending structural depression extending from the Gulf of California to the Banning Pass. The Salton Trough is dominated by several northwest trending faults, most notably the San Andreas Fault system. The Salton Trough is bounded by the Santa Rosa – San Jacinto Mountains on the southwest, the San Bernardino Mountains on the north, the Little San Bernardino - Chocolate – Orocochia Mountains on the east, and extends through the Imperial Valley into the Gulf of California on the south.

A relatively thick sequence (20,000 feet) of sediment has been deposited in the Coachella Valley portion of the Salton Trough from Miocene to present times. These sediments are predominately terrestrial in nature with some lacustrine (lake) and minor marine deposits. The major contributor of these sediments has been the Colorado River. The mountains surrounding the Coachella Valley are composed primarily of Precambrian metamorphic and Mesozoic "granitic" rock.

The Salton Trough is an internally draining area with no readily available outlet to Gulf of California and with portions well below sea level (-253' msl). The region is intermittently blocked from the Gulf of California by the damming effects of the Colorado River delta (current elevation +30' msl). Between about 300AD and 1600 AD (to 1700) the Salton Trough has been inundated by the River's water, forming ancient Lake Cahuilla (max. elevation +58' msl). Since that time the floor of the Trough has been repeatedly flooded with other "fresh" water lakes (1849, 1861, and 1891), the most recent and historically long lived being the current Salton Sea (1905). The sole outlet for these waters is evaporation, leaving behind vast amounts of terrestrial sediment materials and evaporite minerals.

The proposed alignment has been mapped by Jennings (1967) to be immediately underlain by undifferentiated Quaternary-age alluvial and lake deposits (Q1 - Qal). The regional geologic setting for the site vicinity is presented on the Regional Geologic Map (Figure 2).

SUBSURFACE CONDITIONS

The subsurface conditions at the site were investigated by drilling two (2) exploratory boreholes to depths of approximately 26 feet bgs. The approximate locations of the boreholes are illustrated on the Borehole Location Plan (Figure 3). The boreholes were advanced using a track-mounted CME-75 limited access drill-rig equipped with 8-inch outside diameter hollow stem augers. A representative of Sladden was on-site to log the materials encountered.

During our field investigation a thin mantle of fill soil was encountered to a maximum depth of approximately three (3) feet bgs in the vicinity of BH-1 (proposed canal outlet). Underlying the fill soil and extending to the maximum depths explored, native alluvium was encountered. Native alluvial materials were encountered throughout the depths explored in BH-2. The native alluvial soil in both bores consists primarily of fine-grained silty sand (SM) and sand (SP) and minor proportions of sandy silt (ML). The granular native soil was found to be dry, medium dense, fine-grained and grayish brown to

yellowish brown in in-situ color. Cohesive soil was found to be very stiff and exhibited characteristics indicative of low plasticity soil.

The final logs represent our interpretation of the contents of the field logs and the results of the laboratory observations and tests of the field samples. The final logs are included in Appendix A of this report. The stratification lines represent the approximate boundaries between soil types although the transitions may be gradual and/or variable across the site.

Groundwater was not encountered to a maximum explored depth of 26 feet bgs during our field investigation. Based on our experience in the site vicinity and our review of groundwater elevations (CDWR, 2018), it is our opinion that groundwater should not be a factor during construction of the proposed project. The following table provides groundwater depths reported in the site vicinity.

TABLE 1
GROUNDWATER DEPTHS

Groundwater Level Station	Year	*Distance (Km) and Direction	Depth to Water Surface (Ft)
335703N1159888W001	2017	0.60 NE	241.9
335403N1160260W001	2017	4.4 SW	4.2
336037N1160485W001	2015	6.8 NW	164.7
336167N1161008W001	2017	11.6 NW	23.3

*Distance measured from intersection of Riddle Way and Arthur Street

SEISMICITY AND FAULTING

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

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

As previously stated, the site has been subjected to strong seismic shaking related to active faults that traverse through the region. Some of the more significant seismic events near the subject site within recent times include: M6.0 North Palm Springs (1986), M6.1 Joshua Tree (1992), M7.3 Landers (1992), M6.2 Big Bear (1992) and M7.1 Hector Mine (1999). The proposed Wasteway No. 1 crossing is located within the San Andreas Fault Zone (Figure 4).

Table 2 lists the closest known potentially active faults that was generated in part using the EQFAULT computer program (Blake, 2000), as modified using the fault parameters from The Revised 2002 California Probabilistic Seismic Hazard Maps (Cao et al, 2003). This table does not identify the probability of reactivation or the on-site effects from earthquakes occurring on any of the other faults in the region.

TABLE 2
CLOSEST KNOWN ACTIVE FAULTS

Fault Name	Distance (Km)	Maximum Event
San Andreas - Coachella	*0.0	**7.2
San Andreas - Southern	*0.0	**7.2
San Jacinto - Anza	35.8	7.2
San Jacinto - Coyote Creek	43.3	6.8
San Jacinto - Borrego	45.4	6.6

* Wasteway No. 1 crossing is located within a State of California designated fault zone (Figure 4).

** 8.1 for multiple segment rupture

2016 CBC SEISMIC DESIGN PARAMETERS

Sladden has reviewed the 2016 California Building Code (CBC) and summarized the current seismic design parameters for the proposed structures. The seismic design category for a structure may be determined in accordance with Section 1613 of the 2016 CBC or ASCE7. According to the 2016 CBC, Site Class D may be used to estimate design seismic loading for the proposed structure. The 2016 CBC Seismic Design Parameters are summarized below (USGS, 2018a). The project Design Map Reports are included within Appendix C.

Risk Category (Table 1.5-1): I/II/III

Site Class (Table 1613.3.2): D

S_s (Figure 1613.3.1): 2.809g

S₁ (Figure 1613.3.1): 1.322g

F_a (Table 1613.3.3(1)): 1.0

F_v (Table 1613.5.3(2)): 1.5

S_{ms} (Equation 16-37 {F_a X S_s}): 2.809g

S_{m1} (Equation 16-38 {F_v X S₁}): 1.983g

S_{DS} (Equation 16-39 {2/3 X S_{ms}}): 1.873g

S_{D1} (Equation 16-40 {2/3 X S_{m1}}): 1.322g

Seismic Design Category: D

GEOLOGIC HAZARDS

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

- I. Surface Rupture. Surface rupture is expected to occur along preexisting, known active fault traces. However, surface rupture could potentially splay or step from known active faults or rupture along unidentified traces. Based on our review of CGS (2012), the proposed Wasteway No. 1 crossing is located within a State of California designated fault zone (Figure 4). The bridge crossing is not considered a "structure for human occupancy" (California Code of Regulations, Title 1, Division 2, Section 3601 (e)) and is not regulated by the Alquist – Priolo Earthquake Fault Zoning Act (CGS, 2018). However, the potential impacts associated with primary surface ground rupture should be considered in the design of the irrigation pipeline, bridge crossing and associated project improvements.
- II. Ground Shaking. The site has been subjected to past ground shaking by faults that traverse through the region. Strong seismic shaking from nearby active faults is expected to produce strong seismic shaking during the design life of the proposed project. A probabilistic approach was employed to estimate the peak ground acceleration (a_{max}) that could be experienced at the site. Based on the USGS Unified Hazard Tool (USGS, 2018b) shear wave velocity (V_{s30}) of 259 m/s, the proposed bridge crossing could be subjected to ground motions on the order of 0.69. The peak ground acceleration at the site is judged to have a 475 year return period and a 10 percent chance of exceedence in 50 years.
- III. Liquefaction/Dry Sand Settlement. Liquefaction is the process in which loose, saturated granular soil loses strength as a result of cyclic loading. The strength loss is a result of a decrease in granular sand volume and a positive increase in pore pressures. Generally, liquefaction can occur if all of the following conditions apply: liquefaction-susceptible soil, groundwater within a depth of 50 feet or less, and strong seismic shaking. Based on our review of groundwater data in the site vicinity (CDWR, 2018), and our experience in the project vicinity, risks associated with liquefaction and liquefaction related hazards should be considered negligible.
- IV. Tsunamis / Seiches / Inundation. Because the site is situated at an inland location, risks associated with tsunamis are considered "negligible".

The proposed alignment is located to the immediate south of the Coachella Canal. The canal is considered an impounded body of water that may be susceptible to seiches during strong ground shaking. The canal is also transected by the San Andreas fault. In the event of surface rupture along the fault, ruptured embankments could lead to canal water inundating the site. Accordingly, risks associated with water inundation should be considered a potential hazard to the project.

- V. Slope Failure, Landsliding, Rock Falls. No signs of slope instability in the form of landslides, rock falls, earthflows or slumps were observed at or near the subject site. As such, risks associated with slope instability should be considered negligible.
- VI. Expansive Soil. Generally, the site soil consists of silty sand (SM), sand (SP) and sandy silt (ML). Based on the results of our laboratory testing (EI=1), the materials underlying the site are considered to have a "very low" expansion potential. However, the expansion potential of the surface soil should be reevaluated after remedial grading.
- VII. Settlement. Settlement resulting from the anticipated foundation loads should be minimal provided that the recommendations included in this report are considered in foundation design and construction. The estimated ultimate settlement is expected to be less than one-inch when using the recommended design criteria. As a practical matter, differential settlement between footings can be assumed as one-half of the total settlement.
- VIII. Subsidence. Land subsidence can occur in valleys where aquifer systems have been subjected to extensive groundwater pumping, such that groundwater pumping exceeds groundwater recharge. Generally, pore water reduction can result in a rearrangement of skeletal grains and could result in elastic (recoverable) or inelastic (unrecoverable) deformation of an aquifer system.
- The Coachella Valley Water District has publically acknowledged regional subsidence throughout the southern portion of the Coachella Valley and has indicated a commitment to groundwater replenishment programs including the South Valley Recharge project that are intended to limit future subsidence.
- IX. Debris Flows. Debris flows are viscous flows consisting of poorly sorted mixtures of sediment and water and are generally initiated on slopes steeper than approximately six horizontal to one vertical (6H:1V). Based on the flat nature of the site and the composition of the surface soil, we judge that risks associated with debris flows should be considered remote.
- X. Flooding and Erosion. No signs of flooding or erosion were observed during our field investigation. However, flooding and erosion should be evaluated and mitigated by the project design Civil Engineer.

CONCLUSIONS

Based on the results of our investigation, it is our professional opinion that the project should be feasible from a geotechnical perspective provided that the recommendations provided in this report are incorporated into design and carried out through construction. The main geotechnical concerns are the presence of loose native surface soil and the potential impacts associated with primary surface ground rupture along the San Andreas fault.

We recommend that remedial grading within the proposed crossing and foundation areas include over-excavation and re-compaction of all loose native soil encountered during grading. Specific recommendations for site preparation are presented in the Earthwork and Grading section of this report.

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

The following recommendations present more detailed design criteria that have been developed on the basis of our field and laboratory investigation.

EARTHWORK AND GRADING

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

- a. Stripping. Areas to be graded should be cleared of any vegetation, associated root systems, and debris. All areas scheduled to receive fill should be cleared of any undocumented fill and unsuitable materials. The strippings should be removed off site, or stockpiled for later use in landscape areas. Voids left by obstructions should be properly backfilled in accordance with the compaction recommendations of this report.
- b. Preparation Foundation Areas: In order to provide for firm and uniform foundation bearing conditions, the primary bearing soil should be overexcavated and recompacted. Overexcavation should extend to a minimum depth of 2 feet below the bottom of the trapezoidal inserts or footings. Once adequate removals have been verified, the exposed native soil should be scarified to a depth of approximately 12 inches, moisture conditioned to within two percent of optimum moisture content and compacted to at least 90 percent relative compaction. The previously removed material may then be placed as compacted engineered fill as outlined below. Removals should extend at least 3 feet laterally beyond the footing limits.

- c. Fill Placement and Compaction: Soil to be used as engineered fill should be free of organic material, debris, and other deleterious substances, and should not contain irreducible matter greater than three inches in maximum dimension. All fill materials should be placed in thin lifts, not exceeding six inches in a loose condition. If import fill is required, the material should be of a low to non-expansive nature and should meet the following criteria:

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

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

Table 3
SUMMARY OF RECOMMENDATIONS

*Remedial Grading	Excavation and recompaction within the footing areas and extending laterally for 3 feet beyond the footing limits and to a minimum of 2 feet below the bottom of the box culverts or footings.
Native / Import Engineered Fill	Place in thin lifts not exceeding 6 inches in a loose condition and compact to a minimum of 90 percent relative compaction at least 2 percent over optimum moisture content.

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

- d. Shrinkage and Subsidence. Volumetric shrinkage of the material that is excavated and replaced as controlled compacted fill should be anticipated. We estimate that this shrinkage should be between 15 and 20 percent. Subsidence of the surfaces that are scarified and compacted should be less than 1 tenth of a foot. This will vary depending upon the type of equipment used, the moisture content of the soil at the time of grading and the actual degree of compaction attained.

CONVENTIONAL SHALLOW SPREAD FOOTINGS

Conventional spread footings are expected to provide adequate support for retaining walls, wing walls or other auxiliary structures proposed for the outlet structure. All footings should be founded upon properly compacted engineered fill and should have a minimum embedment depth of 12 inches measured from the lowest adjacent finished grade. Continuous and isolated footings should have minimum widths of 12 inches and 24 inches, respectively. Continuous and isolated footings supported upon properly compacted soil may be designed using allowable (net) bearing pressures of 1800 and 2000 pounds per square foot (psf), respectively. Allowable increases of 250 psf for each additional 1 foot in width and 250 psf for each additional 6 inches in depth may be utilized, if desired. The maximum allowable bearing pressure should be 3000 psf. The allowable bearing pressure applies to combined dead and sustained live loads. The maximum foundation bearing pressure of 3000 psf should be conservatively appropriate for use in box culvert design.

The allowable bearing pressures may be increased by one-third when considering transient live loads, including seismic and wind forces. All footings should be reinforced in accordance with the project structural engineer's recommendations.

Based on the recommended allowable bearing pressures, the total static settlement of the shallow footings is anticipated to be less than one-inch, provided foundation preparations conform to the recommendations described in this report. Static differential settlement is anticipated to be approximately one-half of the total settlement for similarly loaded footings spaced up to approximately 50 feet apart.

Lateral load resistance for the spread footings will be developed by passive pressure against the sides of the footings or box culverts below grade and by friction acting at the base of the footings or box culverts. An allowable passive pressure of 250 psf per foot of depth may be used for design purposes. An allowable coefficient of friction 0.40 may be used for dead and sustained live loads to compute the frictional resistance of the footings placed directly on compact engineered fill soil. Under seismic and wind loading conditions, the passive pressure and frictional resistance may be increased by one-third.

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

RETAINING WALLS

Retaining walls/wing walls may be required to accomplish the proposed construction. Cantilever retaining walls may be designed using "active" pressures. Active pressures may be estimated using an equivalent fluid weight of 35 pcf for native backfill soil with level free-draining backfill conditions. "At Rest" pressures should be utilized for restrained walls. At rest pressures may be estimated using an equivalent fluid weight of 55 pcf for native backfill soil with level drained backfill conditions.

CORROSION SERIES

The soluble sulfate concentrations of the surface soil were determined to be 120 parts per million (ppm). The soil is considered to have a “negligible” corrosion potential with respect to concrete. The use of Type V cement and special sulfate resistant concrete mixes should not be necessary. However, based upon CVWD design criteria (CVWD, 2018), structural concrete should develop a minimum compressive strength of 4,000 psi (28 days).

The pH level of the surface soil was 9.0. Based on soluble chloride concentration testing (80 ppm) the soil is considered to have “low” corrosion with respect to normal grade steel. The minimum resistivity of the surface soil was found to be 2500 ohm-cm that suggests the site soil is considered to have a “moderate” corrosion potential with respect to ferrous metal installations. A corrosion expert should be consulted regarding appropriate corrosion protection measures.

CVWD PIPELINE DESIGN CRITERIA

The near surface soil throughout the project alignment consists of primarily of fine-grained silty sand (SM) sand (SP) and sandy silt (ML). The site soil most closely corresponds with Class III soil (SM – Silty Sand) and Class IV soil (ML – Sandy Silt). The maximum density of the surface soil was determined to be approximately 131.5 pcf using the Modified Proctor (ASTM Test Method D-1557). The maximum density associated with a Standard Proctor test method is estimated to be approximately 115 pcf. As previously indicated, no groundwater was encountered within our bores that extended to a maximum depth of approximately 26 feet. Groundwater should not be a factor in pipeline design.

It should be noted that the allowable bearing pressures previously provided within this report are intended for use in the design of conventional shallow spread footing foundations. The allowable bearing pressures for use in pipeline design will generally increase with depth. In our opinion, an allowable bearing capacity of 3000 psf should be appropriate for use in the design of pipelines varying from 5 to 10 feet in depth. The allowable bearing capacity may be increased to 4000 psf for pipelines in excess of 10 feet in depth.

UTILITY TRENCH BACKFILL

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

EXTERIOR CONCRETE FLATWORK

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

DRAINAGE

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

LIMITATIONS

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

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

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

ADDITIONAL SERVICES

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

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

REFERENCES

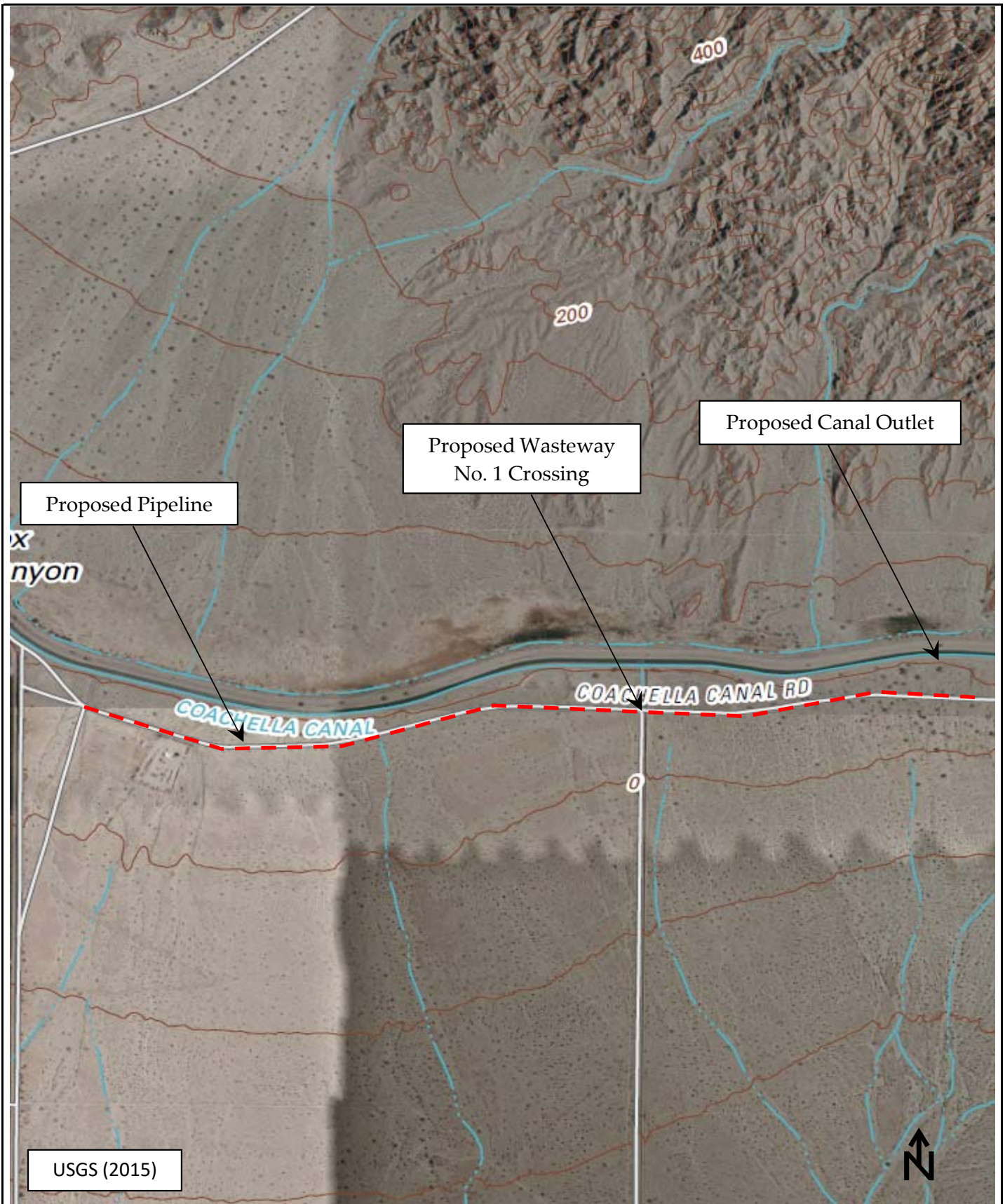
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FIGURES

SITE LOCATION MAP
REGIONAL GEOLOGIC MAP
BOREHOLE LOCATION PLAN
FAULT ZONE MAP



SITE LOCATION MAP

FIGURE

1



Sladden Engineering

Project Number:

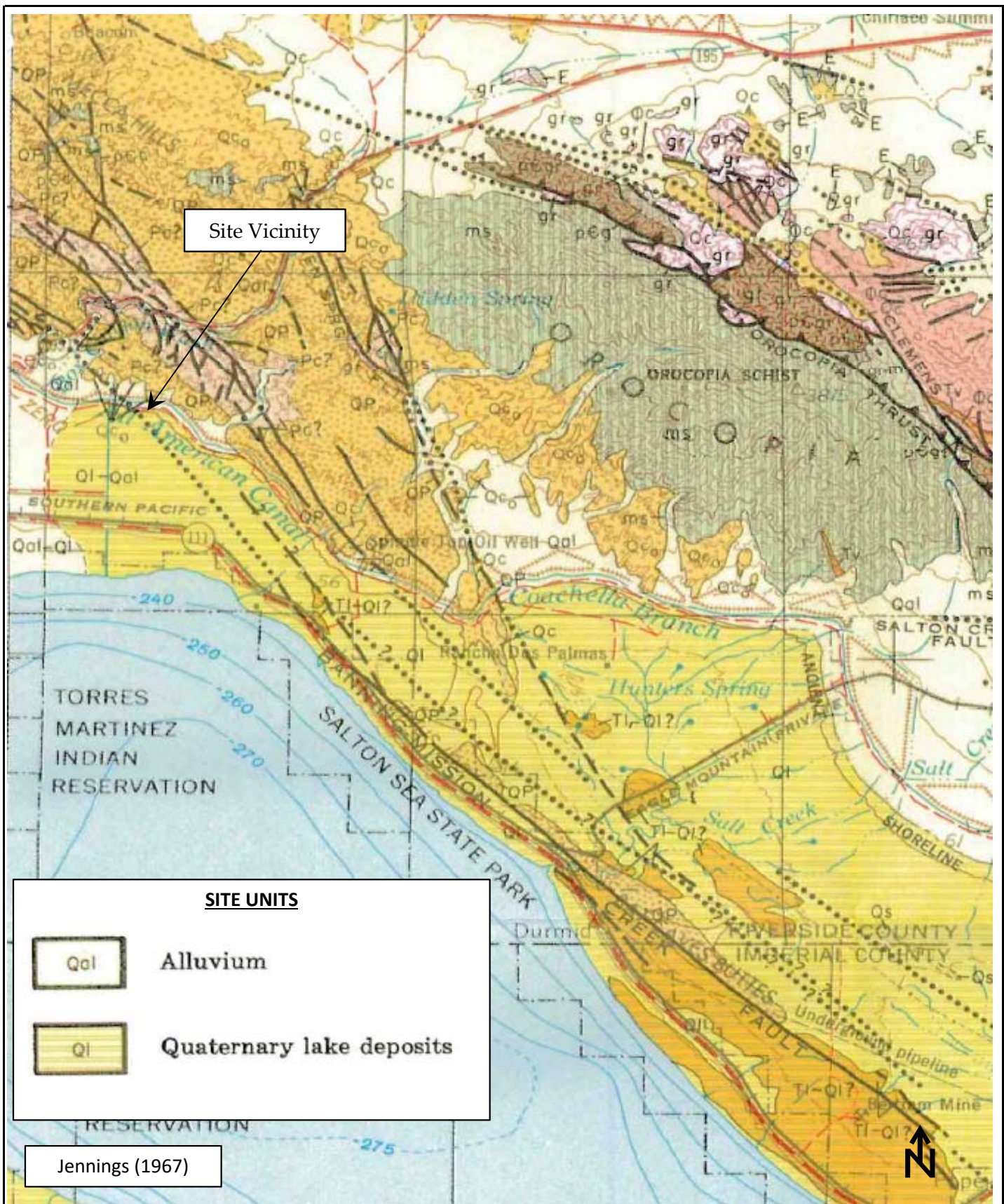
544-17109

Report Number:

18-03-136

Date:

March 21, 2018



Sladden Engineering

REGIONAL GEOLOGIC MAP

Project Number:	544-17109
Report Number:	18-03-136
Date:	March 21, 2018

FIGURE

2

MAP EXPLANATION

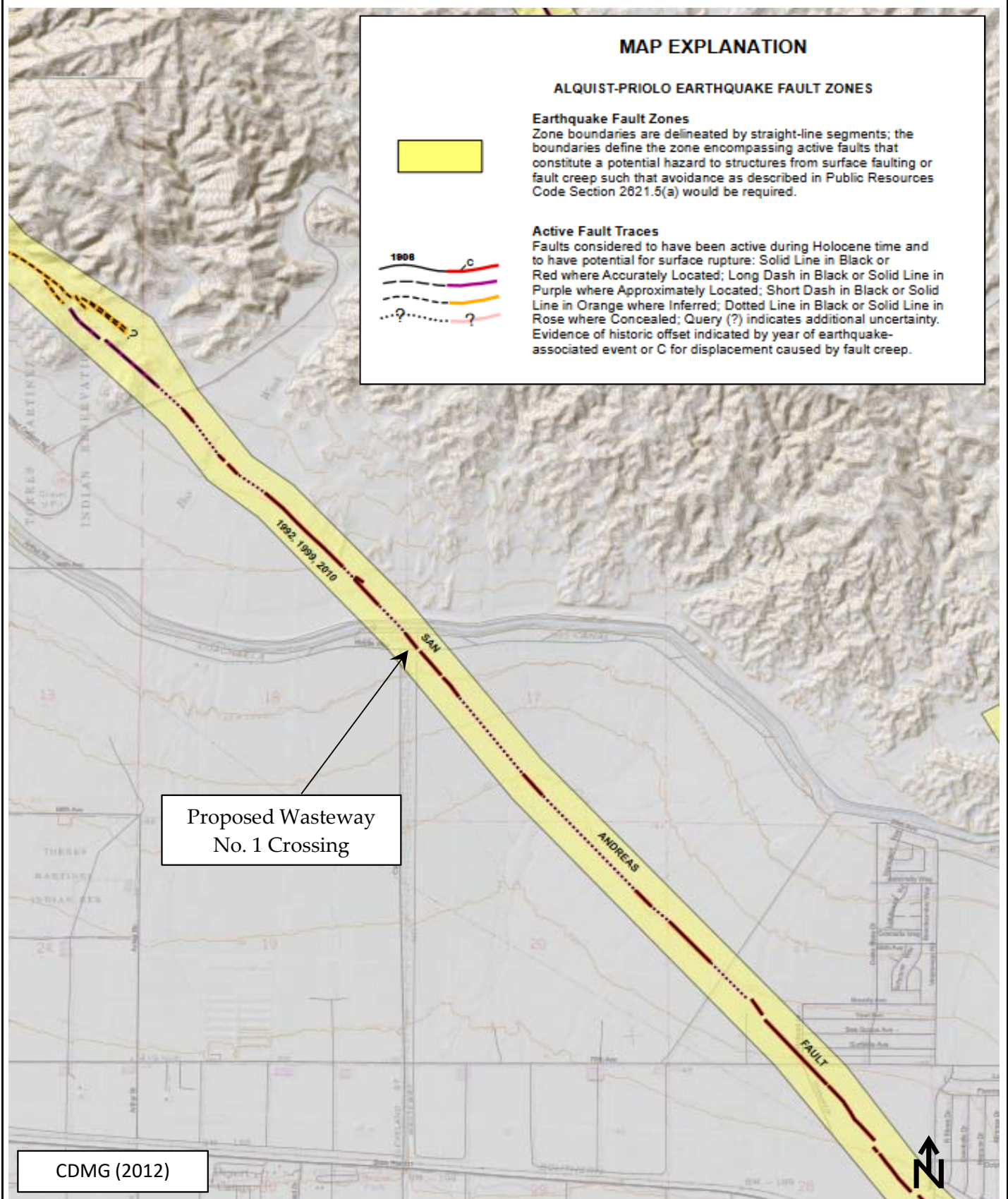
ALQUIST-PRIOLO EARTHQUAKE FAULT ZONES

Earthquake Fault Zones

Zone boundaries are delineated by straight-line segments; the boundaries define the zone encompassing active faults that constitute a potential hazard to structures from surface faulting or fault creep such that avoidance as described in Public Resources Code Section 2621.5(a) would be required.

Active Fault Traces

Faults considered to have been active during Holocene time and to have potential for surface rupture: Solid Line in Black or Red where Accurately Located; Long Dash in Black or Solid Line in Purple where Approximately Located; Short Dash in Black or Solid Line in Orange where Inferred; Dotted Line in Black or Solid Line in Rose where Concealed; Query (?) indicates additional uncertainty. Evidence of historic offset indicated by year of earthquake-associated event or C for displacement caused by fault creep.



FAULT ZONE MAP

FIGURE

4



Sladden Engineering

Project Number:

544-17109

Report Number:

18-03-136

Date:

March 21, 2018

APPENDIX A
FIELD EXPLORATION

APPENDIX A

FIELD EXPLORATION

For our field investigation two (2) exploratory bores were excavated utilizing a track-mounted limited access drill rig equipped with 8 inch (O.D.) hollow-stem augers (CME-75). Continuous logs of the materials encountered were made by a representative of Sladden Engineering. Materials encountered in the boreholes were classified in accordance with the Unified Soil Classification System which is presented in this appendix.

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

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

UNIFIED SOIL CLASSIFICATION SYSTEM

MAJOR DIVISIONS			TYPICAL NAMES		
COARSE GRAINED SOILS MORE THAN HALF IS LARGER THAN No.200 SIEVE	GRAVELS	CLEAN GRAVELS WITH LITTLE OR NO FINES	GW	WELL GRADED GRAVEL-SAND MIXTURES	
			GP	POORLY GRADED GRAVELS, GRAVEL-SAND MIXTURES	
	MORE THAN HALF COARSE FRACTION IS LARGER THAN No.4 SIEVE SIZE	GRAVELS WITH OVER 12% FINES	GM	SILTY GRAVELS, POORLY-GRADED GRAVEL-SAND-SILT MIXTURES	
			GC	CLAYEY GRAVELS, POORLY GRADED GRAVEL-SAND-CLAY MIXTURES	
	SANDS	CLEAN SANDS WITH LITTLE OR NO FINES	SW	WELL GRADED SANDS, GRAVELLY SANDS	
			SP	POORLY GRADED SANDS, GRAVELLY SANDS	
		MORE THAN HALF COARSE FRACTION IS SMALLER THAN No.4 SIEVE SIZE	SANDS WITH OVER 12% FINES	SM	SILTY SANDS, POORLY GRADED SAND-SILT MIXTURES
				SC	CLAYEY SANDS, POORLY GRADED SAND-CLAY MIXTURES
FINE GRAINED SOILS MORE THAN HALF IS SMALLER THAN No.200 SIEVE	SILTS AND CLAYS LIQUID LIMIT LESS THAN 50		ML	INORGANIC SILTS & VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS, OR CLAYEY SILTS WITH SLIGHT PLASTICITY	
			CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, CLEAN-CLAYS	
			OL	ORGANIC CLAYS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY	
	SILTS AND CLAYS: LIQUID LIMIT GREATER THAN 50		MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SANDY OR 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	

EXPLANATION OF BORE LOG SYMBOLS



California Split-spoon Sample



Unrecovered Sample



Standard Penetration Test Sample



Groundwater depth

Note: The stratification lines on the borelogs represent the approximate boundaries between the soil types; the transitions may be gradual.

SLADDEN ENGINEERING								BORE LOG			
								Drill Rig:	CME 75	Date Drilled:	2/9/2018
								Elevation:	35 Ft (MSL)	Boring No:	BH-1
Sample	Blow Counts	Bulk Sample	Expansion Index	% Minus #200	% Moisture	Dry Density	Depth (Feet)	Graphic Lithology	Description		
		1	0				2		Silty Sand (SM) with gravel; grayish brown, dry, fine- to coarse-grained (Fill).		
	15/12/12						4		No Recovery.		
							6				
							8				
	6/8/8			5.7	1.1		10				
							12		Sand (SP); grayish brown, dry, medium dense, fine- to coarse-grained (Qal).		
							14		At 13 Feet light olive gray		
	14/20/25			7.0	0.7	119.4	16		Sand (SP/SW); light grayish brown, dry, medium dense, fine- to coarse-grained (Qal).		
							18				
							20		Silty Sand (SM); light grayish brown, dry, medium dense, fine-grained (Qal).		
	6/11/17			17.9	1.5		22				
							24				
	27/50-5"						26		No Recovery.		
							28		Terminated at ~26.5 Feet bgs. No Bedrock Encountered. No Groundwater or Seepage Encountered.		
							30				
							32				
							34				
							36				
							38				
							40				
							42				
							44				
							46				
							48				
							50				

Completion Notes:

PROPOSED WASTEWAY NO 1. CROSSING & CANAL OUTLET SOUTH OF THE COACHELLA CANAL, NORTH SHORE AREA

Project No: 544-17109
Page
1

Report No: 18-03-136

**SLADDEN ENGINEERING****BORE LOG**

Drill Rig: CME 75

Date Drilled: 2/9/2018

Elevation: 35 Ft (MSL)

Boring No: BH-2

Sample	Blow Counts	Bulk Sample	Expansion Index	% Minus #200	% Moisture	Dry Density	Depth (Feet)	Graphic Lithology	Description
		2					2		Silty Sand (SM) with gravel and cobbles; grayish brown, dry, fine- to coarse-grained (Qal).
	6/9/9			12.4	0.6		4		
							6		Silty Sand (SM); light grayish brown, dry, medium dense, fine-grained (Qal).
	4/5/7			8.3	0.8		8		
							10		Sand (SP); light grayish brown, dry, medium dense, fine-grained (Qal).
							12		
	8/17/20			80.3	3.0	98.2	14		
							16		Sandy Silt (ML); pale gray and yellowish brown, dry, very stiff, low plasticity with shell fragments (Qal).
							18		
	8/8/8			27.5	1.5		20		Silty Sand (SM); yellowish brown, dry, medium dense, fine-grained (Qal).
							22		
							24		
	12/15/20						26		No Recovery.
							28		
							30		Terminated at -26.5 Feet bgs.
							32		No Bedrock Encountered.
							34		No Groundwater or Seepage Encountered.
							36		
							38		
							40		
							42		
							44		
							46		
							48		
							50		

Completion Notes:

PROPOSED WASTEWAY NO 1. CROSSING & CANAL OUTLET
SOUTH OF THE COACHELLA CANAL, NORTH SHORE AREA

Project No: 544-17109

Report No: 18-03-136

Page

2

APPENDIX B

LABORATORY TESTING

APPENDIX B

LABORATORY TESTING

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

CLASSIFICATION AND COMPACTION TESTING

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

Maximum Density-Optimum Moisture Determinations: Representative soil types were selected for maximum density determinations. This testing was performed in accordance with the ASTM Standard D1557-91, Test Method A. Graphic representations of the results of this testing are presented in this appendix. The maximum densities are compared to the field densities of the soil in order to determine the existing relative compaction to the soil.

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

SOIL MECHANIC'S TESTING

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

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

Consolidation/Hydro-Collapse Testing: One (1) relatively undisturbed samples were selected for consolidation testing. For this test, a one-inch thick test specimen was subjected to vertical loads varying from 575 psf to 11520 psf applied progressively. The consolidation at each load increment was recorded prior to placement of each subsequent load.

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



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Maximum Density/Optimum Moisture

ASTM D698/D1557

Project Number: 544-17109
Project Name: Pipeline Easement
Lab ID Number: LN6-18067
Sample Location: BH-1 Bulk 1 @ 0-5'
Description: Gray Brown Sand w/Silt (SW-SM)

March 2, 2018

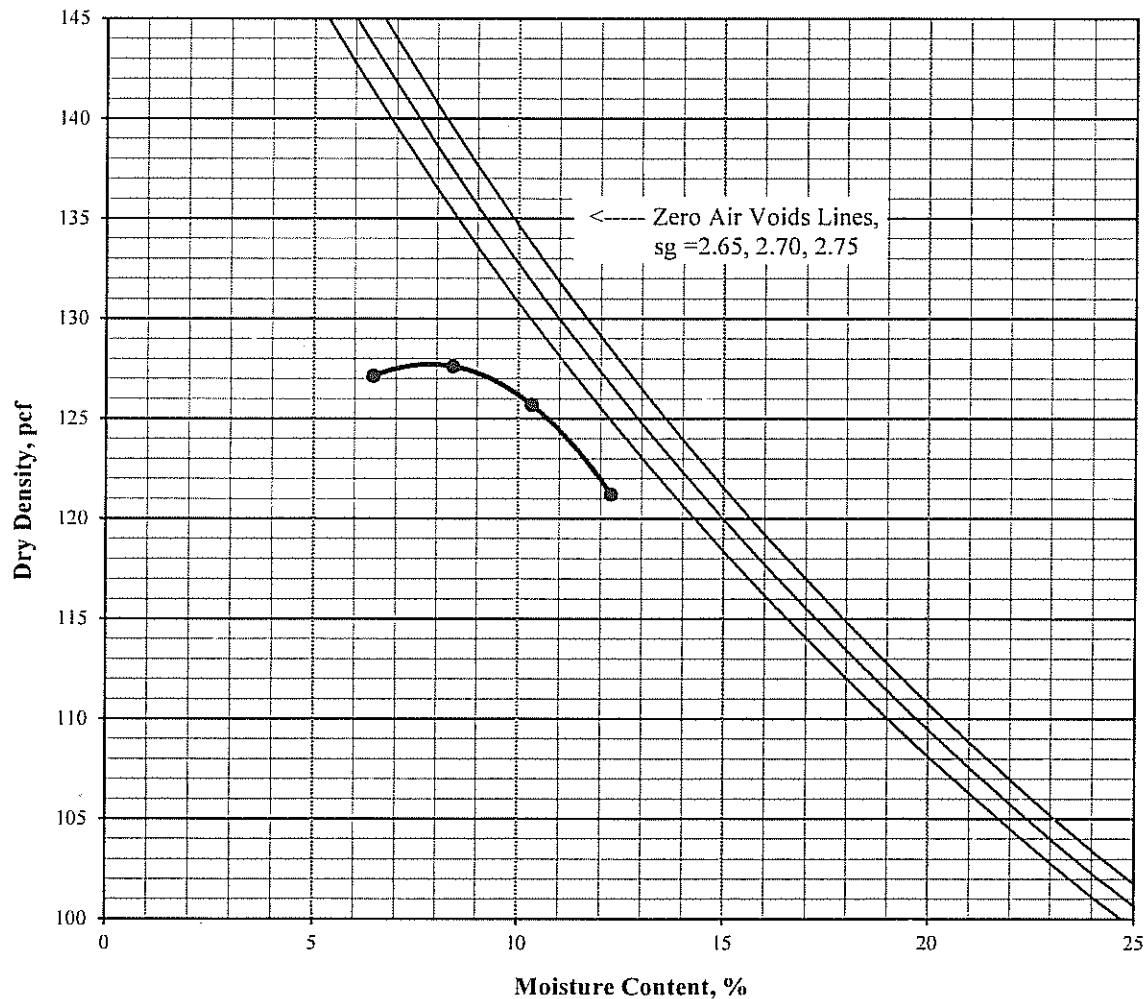
ASTM D-1557 A
Rammer Type: Machine

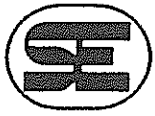
Maximum Density: 131.5 pcf

Optimum Moisture: 8%

Corrected for Oversize (ASTM D4718)

Sieve Size	% Retained
3/4"	
3/8"	
#4	12.9





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Expansion Index

ASTM D 4829

Job Number: 544-17109
Job Name: Pipeline Easement
Lab ID Number: LN6-18067
Sample ID: BH-1 Bulk 1 @ 0-5'
Soil Description: Gray Brown Sand w/Silt (SW-SM)

March 2, 2018

Wt of Soil + Ring:	582.6
Weight of Ring:	190.5
Wt of Wet Soil:	392.1
Percent Moisture:	8.2%
Sample Height, in	0.95
Wet Density, pcf:	125.1
Dry Denstiy, pcf:	115.6

% Saturation:	48.4
---------------	------

Expansion

Rack # 1

Date/Time	2/27/2018	3:17 PM
Initial Reading	0.0000	
Final Reading	0.0000	

Expansion Index

0

(Final - Initial) x 1000



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Direct Shear ASTM D 3080-04 (modified for unconsolidated condition)

Job Number: 544-17109

March 2, 2018

Job Name Pipeline Easement

Initial Dry Density: 115.8 pcf

Lab ID No. LN6-18067

Initial Moisture Content: 8.8 %

Sample ID BH-1 Bulk 1 @ 0-5'

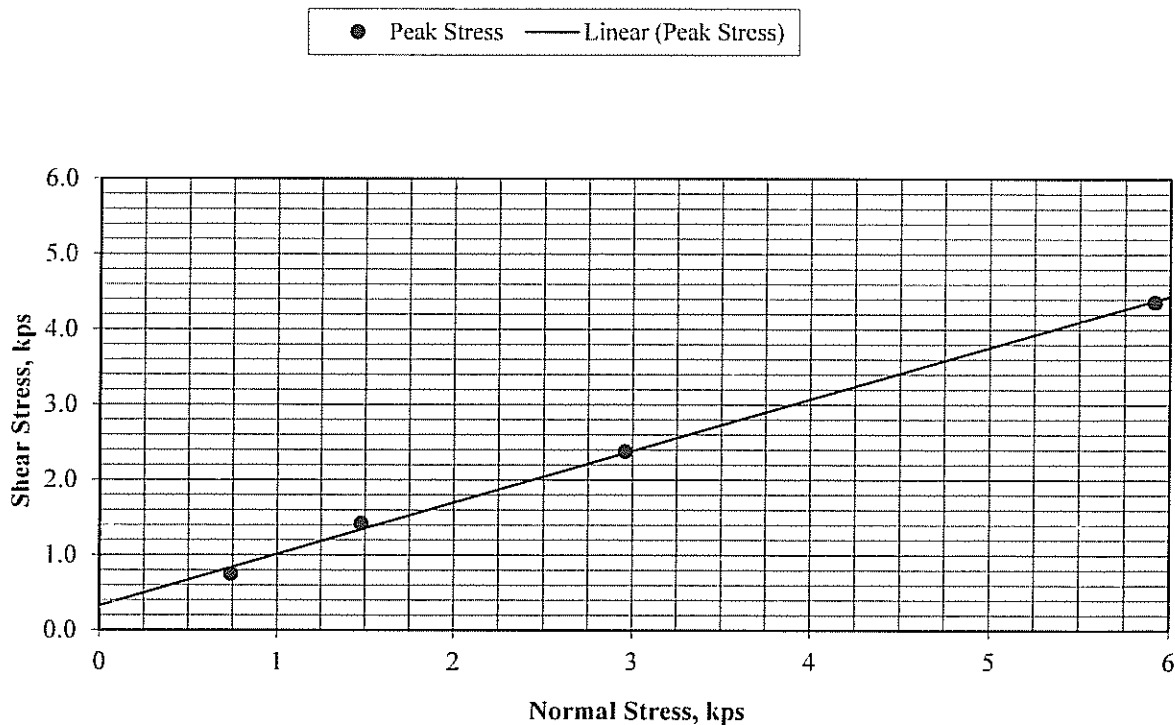
Peak Friction Angle (ϕ): 34°

Classification Gray Brown Sand w/Silt (SW-SM)

Cohesion (c): 320 psf

Sample Type Remolded @ 90% of Maximum Density

Test Results	1	2	3	4	Average
Moisture Content, %	11.9	11.9	11.9	11.9	11.9
Saturation, %	70.6	70.6	70.6	70.6	70.6
Normal Stress, kps	0.739	1.479	2.958	5.916	
Peak Stress, kps	0.748	1.422	2.379	4.365	





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Gradation

ASTM C117 & C136

Project Number: 544-17109

March 2, 2018

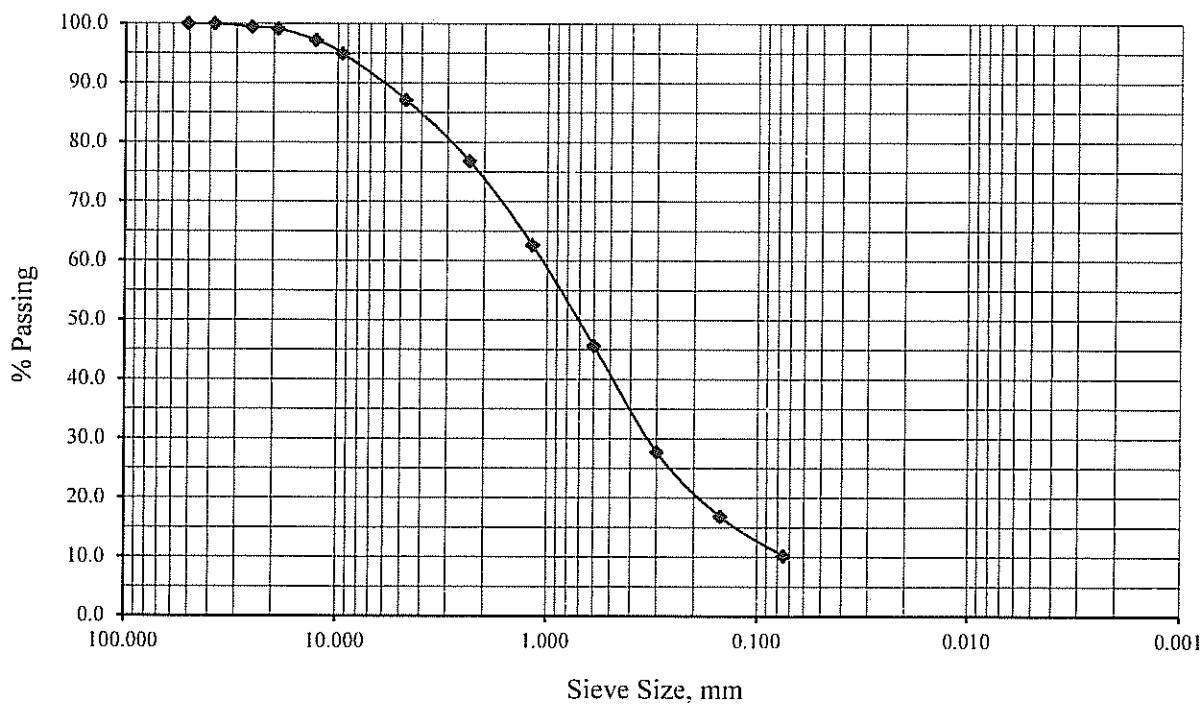
Project Name: Pipeline Easement

Lab ID Number: LN6-18067

Sample ID: BH-1 Bulk 1 @ 0-5'

Soil Classification: SW-SM

Sieve Size, in	Sieve Size, mm	Percent Passing
2"	50.8	100.0
1 1/2"	38.1	100.0
1"	25.4	99.4
3/4"	19.1	99.1
1/2"	12.7	97.2
3/8"	9.53	94.9
#4	4.75	87.1
#8	2.36	76.9
#16	1.18	62.7
#30	0.60	45.7
#50	0.30	27.7
#100	0.15	16.9
#200	0.075	10.2





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Gradation

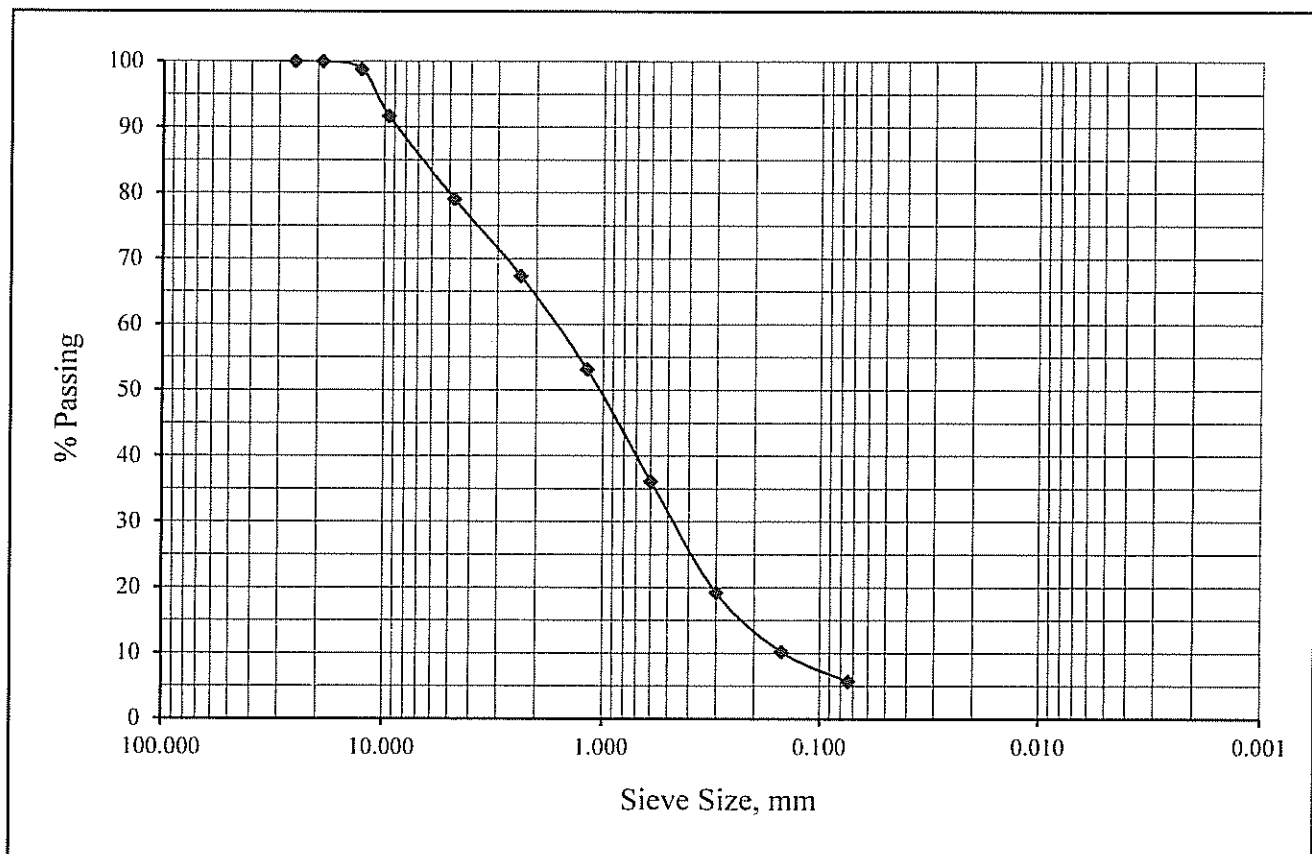
ASTM C117 & C136

Project Number: 544-17109
Project Name: Pipeline Easement
Lab ID Number: LN6-18067
Sample ID: BH-1 S-2 @ 10'

March 2, 2018

Soil Classification: SP-SM

Sieve Size, in	Sieve Size, mm	Percent Passing
1"	25.4	100.0
3/4"	19.1	100.0
1/2"	12.7	98.8
3/8"	9.53	91.7
#4	4.75	79.1
#8	2.36	67.4
#16	1.18	53.2
#30	0.60	36.2
#50	0.30	19.3
#100	0.15	10.3
#200	0.074	5.7





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Gradation

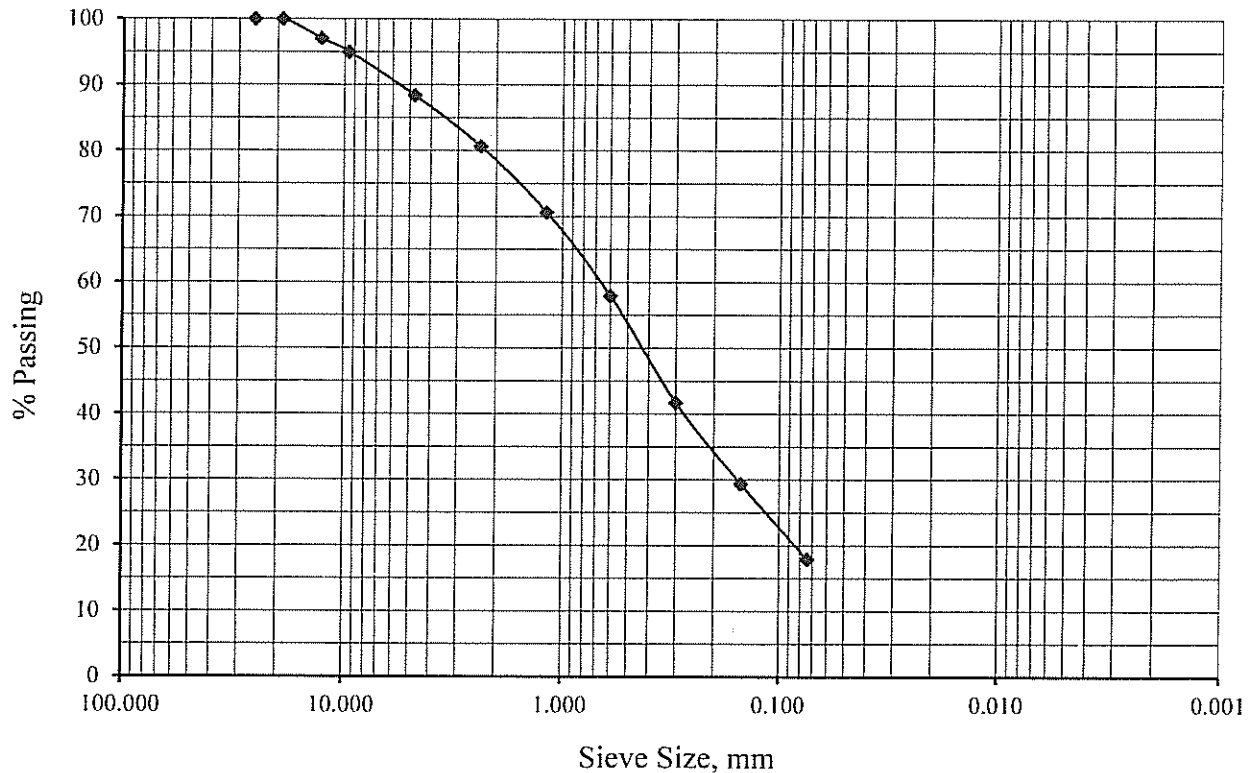
ASTM C117 & C136

Project Number: 544-17109
Project Name: Pipeline Easement
Lab ID Number: LN6-18067
Sample ID: BH-1 S-4 @ 20'

March 2, 2018

Soil Classification: SM

Sieve Size, in	Sieve Size, mm	Percent Passing
1"	25.4	100.0
3/4"	19.1	100.0
1/2"	12.7	97.0
3/8"	9.53	95.0
#4	4.75	88.4
#8	2.36	80.7
#16	1.18	70.6
#30	0.60	57.9
#50	0.30	41.7
#100	0.15	29.4
#200	0.074	17.9





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Gradation

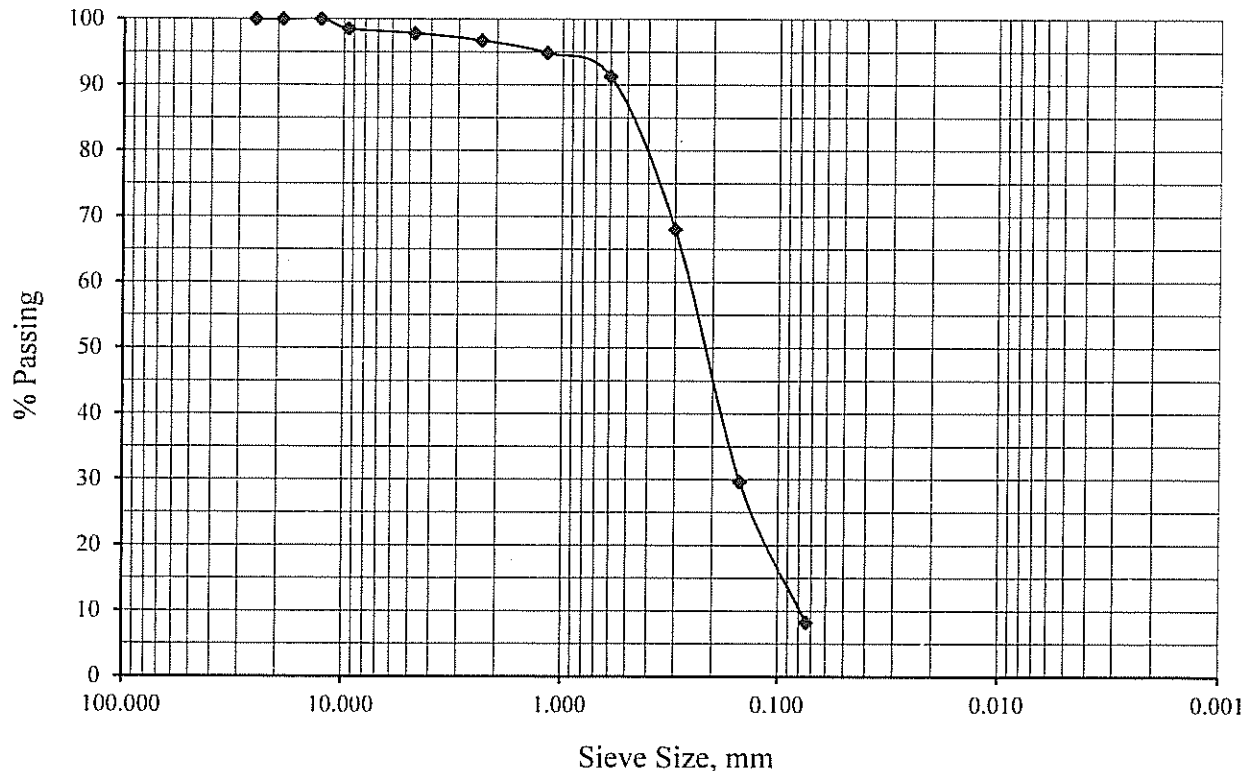
ASTM C117 & C136

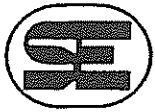
Project Number: 544-17109
Project Name: Pipeline Easement
Lab ID Number: LN6-18067
Sample ID: BH-2 S-2 @ 10'

March 2, 2018

Soil Classification: SP-SM

Sieve Size, in	Sieve Size, mm	Percent Passing
1"	25.4	100.0
3/4"	19.1	100.0
1/2"	12.7	100.0
3/8"	9.53	98.5
#4	4.75	97.8
#8	2.36	96.7
#16	1.18	94.9
#30	0.60	91.3
#50	0.30	68.0
#100	0.15	29.7
#200	0.074	8.3





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One Dimensional Consolidation

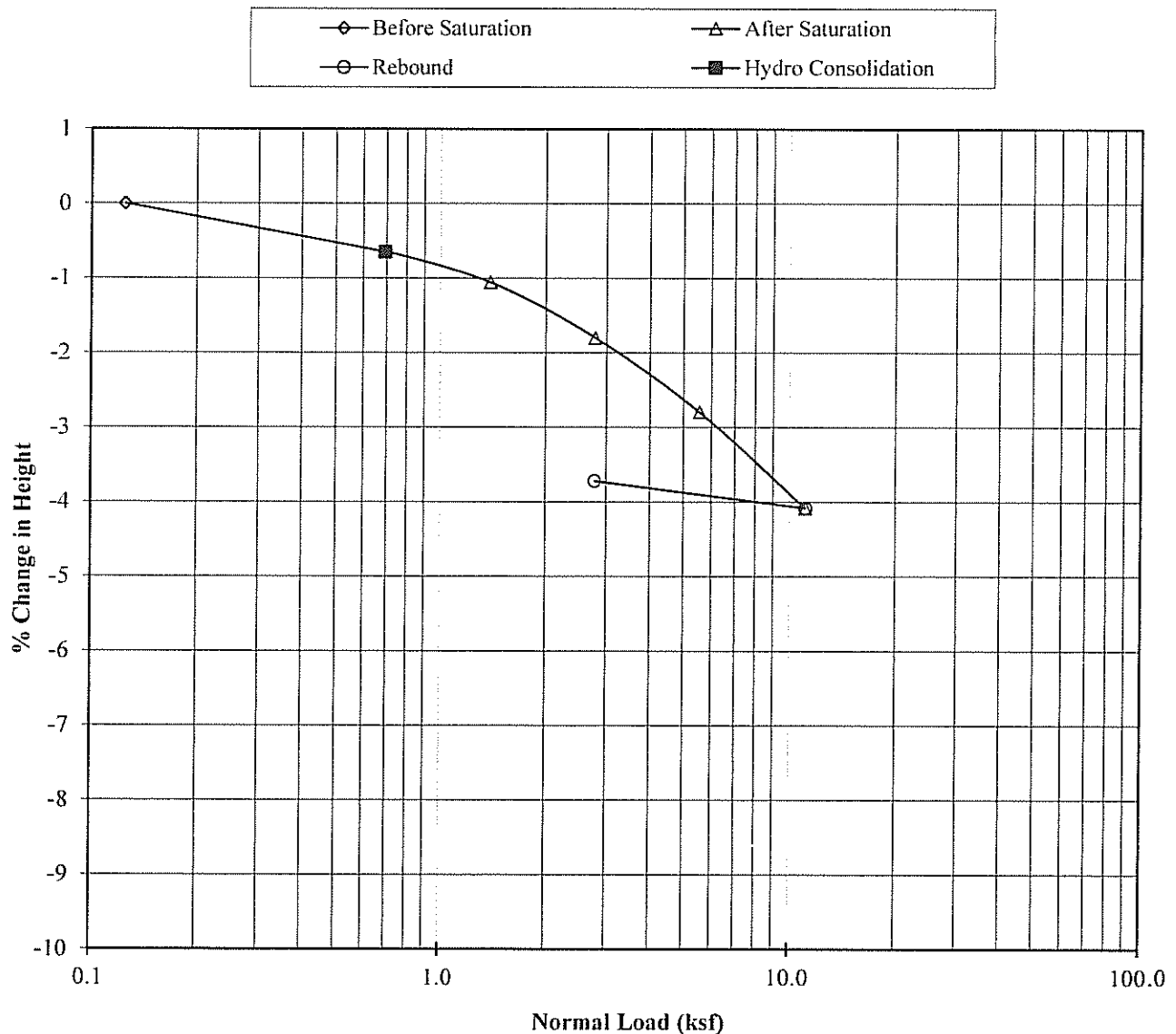
ASTM D2435 & D5333

Job Number: 544-17109
Job Name: Pipeline Easement
Lab ID Number: LN6-18067
Sample ID: BH-2 R-3 @ 15'
Soil Description: Dark Brown Silt w/Sand (ML)

March 2, 2018

Initial Dry Density, pcf: 92.1
Initial Moisture, %: 3.0
Initial Void Ratio: 0.809
Specific Gravity: 2.67

% Change in Height vs Normal Pressure Diagram





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6782 Stanton Ave., Suite C, Buena Park, CA 90621 (714) 523-0952 Fax (714) 523-1369
45090 Golf Center Pkwy, Suite F, Indio, CA 92201 (760) 863-0713 Fax (760) 863-0847
450 Egan Avenue, Beaumont, CA 92223 (951) 845-7743 Fax (951) 845-8863

Date: March 2, 2018

Account No.: 544-17109

Customer: Olson Engineering Systems, Inc.

Location: Wastewater No.1 Crossing and Canal Outlet, North Shore Area

Analytical Report

Corrosion Series

	pH per CA 643	Soluble Sulfates per CA 417 ppm	Soluble Chloride per CA 422 ppm	Min. Resistivity per CA 643 ohm-cm
BH-1 @ 0-5'	9.0	120	80	2500

APPENDIX C

SEISMIC DESIGN MAP AND REPORT DEAGGREGATION OUPUT

USGS Design Maps Summary Report

User-Specified Input

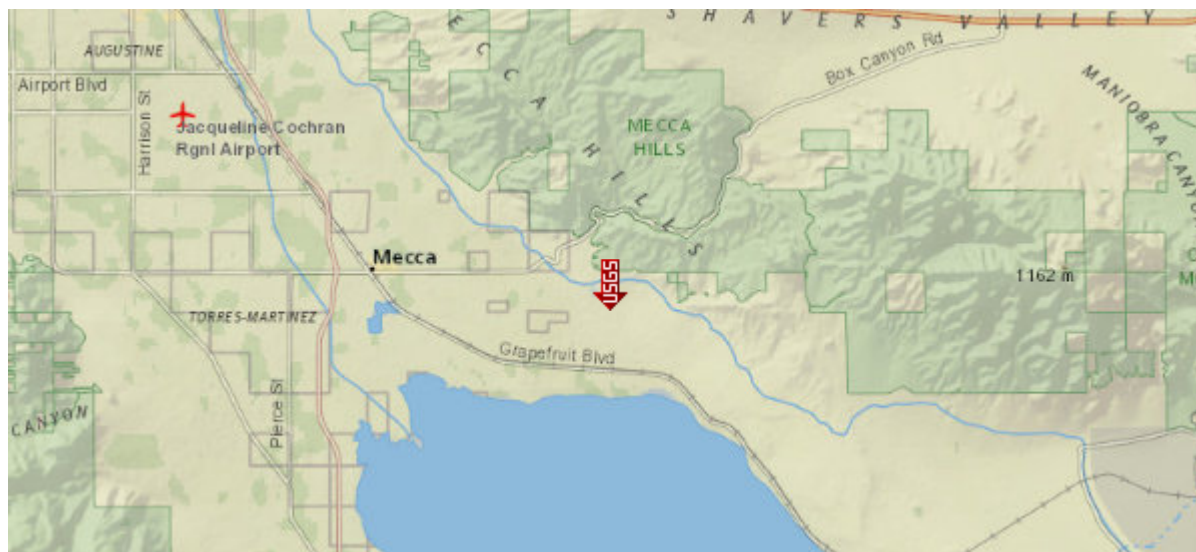
Report Title Wasteway No. 1
Wed March 21, 2018 21:11:31 UTC

Building Code Reference Document ASCE 7-10 Standard
(which utilizes USGS hazard data available in 2008)

Site Coordinates 33.56496°N, 115.97384°W

Site Soil Classification Site Class D – “Stiff Soil”

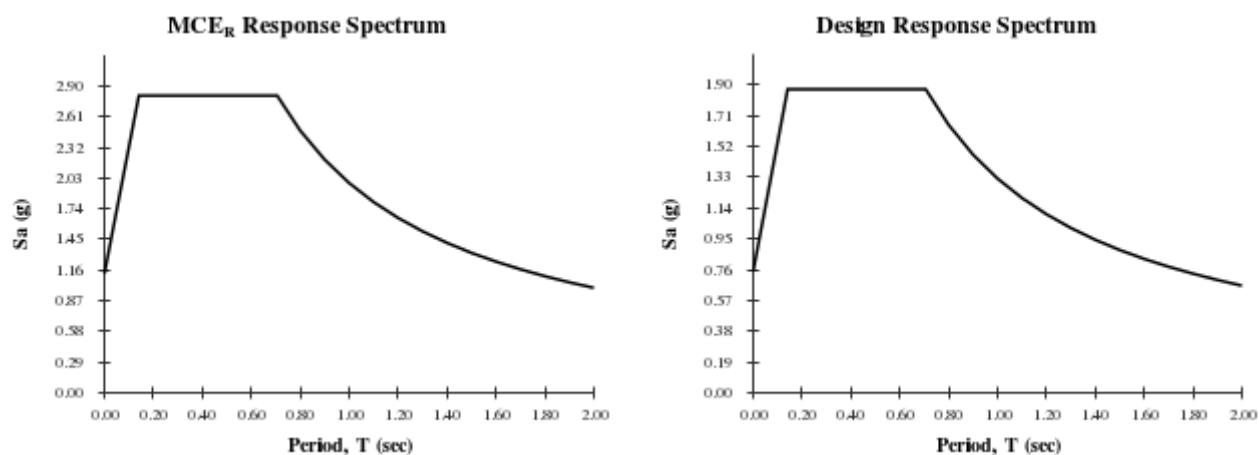
Risk Category I/II/III



USGS-Provided Output

$S_s = 2.809 \text{ g}$	$S_{MS} = 2.809 \text{ g}$	$S_{DS} = 1.873 \text{ g}$
$S_1 = 1.322 \text{ g}$	$S_{M1} = 1.983 \text{ g}$	$S_{D1} = 1.322 \text{ g}$

For information on how the S_s and S_1 values above have been calculated from probabilistic (risk-targeted) and deterministic ground motions in the direction of maximum horizontal response, please return to the application and select the “2009 NEHRP” building code reference document.



For PGA_M , T_L , C_{RS} , and C_{R1} values, please [view the detailed report](#).

Although this information is a product of the U.S. Geological Survey, we provide no warranty, expressed or implied, as to the accuracy of the data contained therein. This tool is not a substitute for technical subject-matter knowledge.



Design Maps Detailed Report

ASCE 7-10 Standard (33.56496°N, 115.97384°W)

Site Class D – “Stiff Soil”, Risk Category I/II/III

Section 11.4.1 — Mapped Acceleration Parameters

Note: Ground motion values provided below are for the direction of maximum horizontal spectral response acceleration. They have been converted from corresponding geometric mean ground motions computed by the USGS by applying factors of 1.1 (to obtain S_s) and 1.3 (to obtain S_1). Maps in the 2010 ASCE-7 Standard are provided for Site Class B. Adjustments for other Site Classes are made, as needed, in Section 11.4.3.

From [Figure 22-1](#) ^[1]

$S_s = 2.809 \text{ g}$

From [Figure 22-2](#) ^[2]

$S_1 = 1.322 \text{ g}$

Section 11.4.2 — Site Class

The authority having jurisdiction (not the USGS), site-specific geotechnical data, and/or the default has classified the site as Site Class D, based on the site soil properties in accordance with Chapter 20.

Table 20.3-1 Site Classification

Site Class	$\bar{v}_s$	$\bar{N}$ or $\bar{N}_{ch}$	$\bar{s}_u$
A. Hard Rock	>5,000 ft/s	N/A	N/A
B. Rock	2,500 to 5,000 ft/s	N/A	N/A
C. Very dense soil and soft rock	1,200 to 2,500 ft/s	>50	>2,000 psf
D. Stiff Soil	600 to 1,200 ft/s	15 to 50	1,000 to 2,000 psf
E. Soft clay soil	<600 ft/s	<15	<1,000 psf
Any profile with more than 10 ft of soil having the characteristics:			
• Plasticity index $PI > 20$, • Moisture content $w \geq 40\%$, and • Undrained shear strength $\bar{s}_u < 500 \text{ psf}$			
F. Soils requiring site response analysis in accordance with Section 21.1	See Section 20.3.1		

For SI: 1ft/s = 0.3048 m/s 1lb/ft² = 0.0479 kN/m²

Section 11.4.3 — Site Coefficients and Risk-Targeted Maximum Considered Earthquake (MCE_R) Spectral Response Acceleration Parameters

Table 11.4-1: Site Coefficient F_a

Site Class	Mapped MCE_R Spectral Response Acceleration Parameter at Short Period				
	$S_s \leq 0.25$	$S_s = 0.50$	$S_s = 0.75$	$S_s = 1.00$	$S_s \geq 1.25$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	See Section 11.4.7 of ASCE 7				

Note: Use straight-line interpolation for intermediate values of S_s

For Site Class = D and $S_s = 2.809$ g, $F_a = 1.000$

Table 11.4-2: Site Coefficient F_v

Site Class	Mapped MCE_R Spectral Response Acceleration Parameter at 1-s Period				
	$S_1 \leq 0.10$	$S_1 = 0.20$	$S_1 = 0.30$	$S_1 = 0.40$	$S_1 \geq 0.50$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	2.4
F	See Section 11.4.7 of ASCE 7				

Note: Use straight-line interpolation for intermediate values of S_1

For Site Class = D and $S_1 = 1.322$ g, $F_v = 1.500$

Equation (11.4-1):

$$S_{MS} = F_a S_S = 1.000 \times 2.809 = 2.809 \text{ g}$$

Equation (11.4-2):

$$S_{M1} = F_v S_1 = 1.500 \times 1.322 = 1.983 \text{ g}$$

Section 11.4.4 — Design Spectral Acceleration Parameters

Equation (11.4-3):

$$S_{DS} = \frac{2}{3} S_{MS} = \frac{2}{3} \times 2.809 = 1.873 \text{ g}$$

Equation (11.4-4):

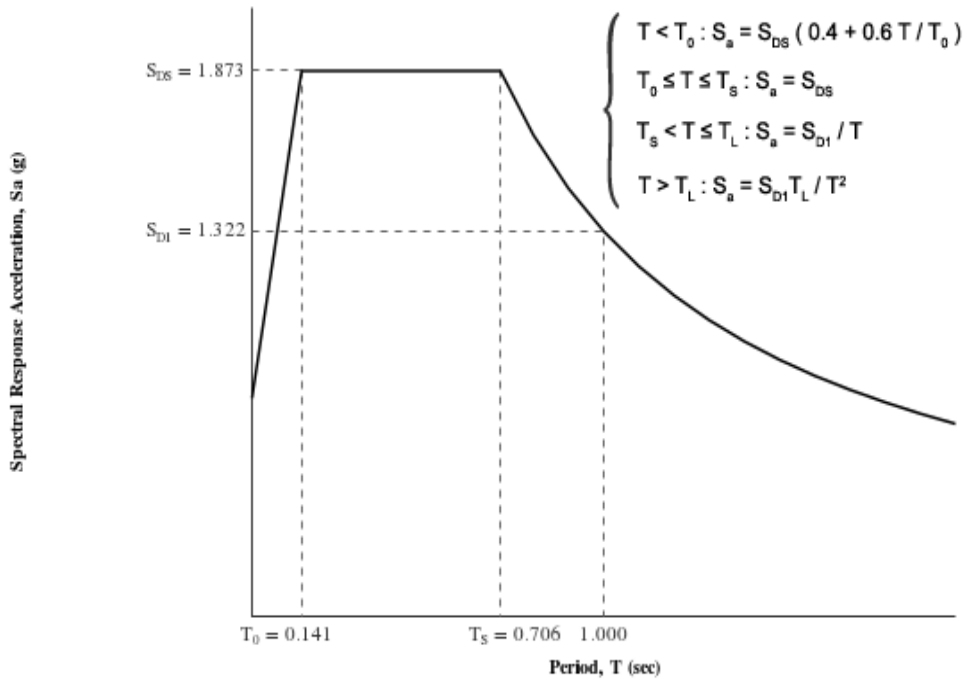
$$S_{D1} = \frac{2}{3} S_{M1} = \frac{2}{3} \times 1.983 = 1.322 \text{ g}$$

Section 11.4.5 — Design Response Spectrum

From [Figure 22-12](#) ^[3]

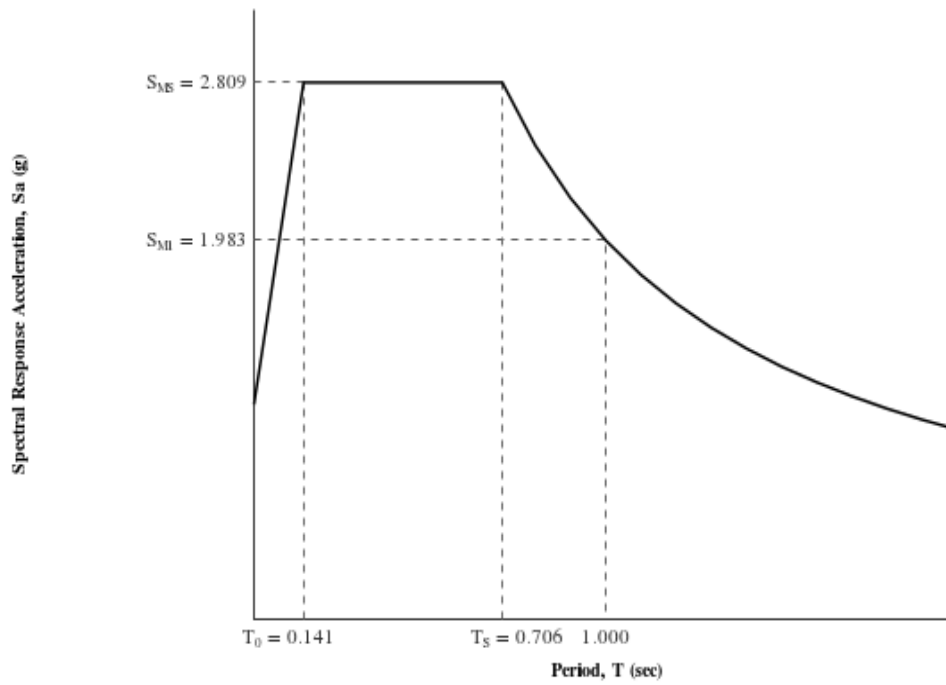
$$T_L = 8 \text{ seconds}$$

Figure 11.4-1: Design Response Spectrum



Section 11.4.6 — Risk-Targeted Maximum Considered Earthquake (MCE_R) Response Spectrum

The MCE_R Response Spectrum is determined by multiplying the design response spectrum above by 1.5.



Section 11.8.3 — Additional Geotechnical Investigation Report Requirements for Seismic Design Categories D through F

From [Figure 22-7](#) ^[4]

$$PGA = 1.088$$

Equation (11.8-1):

$$PGA_M = F_{PGA} PGA = 1.000 \times 1.088 = 1.088 \text{ g}$$

Table 11.8-1: Site Coefficient F_{PGA}

Site Class	Mapped MCE Geometric Mean Peak Ground Acceleration, PGA				
	PGA ≤ 0.10	PGA = 0.20	PGA = 0.30	PGA = 0.40	PGA ≥ 0.50
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	See Section 11.4.7 of ASCE 7				

Note: Use straight-line interpolation for intermediate values of PGA

For Site Class = D and PGA = 1.088 g, $F_{PGA} = 1.000$

Section 21.2.1.1 — Method 1 (from Chapter 21 – Site-Specific Ground Motion Procedures for Seismic Design)

From [Figure 22-17](#) ^[5]

$$C_{RS} = 0.950$$

From [Figure 22-18](#) ^[6]

$$C_{R1} = 0.922$$

Section 11.6 — Seismic Design Category

Table 11.6-1 Seismic Design Category Based on Short Period Response Acceleration Parameter

VALUE OF S_{DS}	RISK CATEGORY		
	I or II	III	IV
$S_{DS} < 0.167g$	A	A	A
$0.167g \leq S_{DS} < 0.33g$	B	B	C
$0.33g \leq S_{DS} < 0.50g$	C	C	D
$0.50g \leq S_{DS}$	D	D	D

For Risk Category = I and $S_{DS} = 1.873 g$, Seismic Design Category = D

Table 11.6-2 Seismic Design Category Based on 1-S Period Response Acceleration Parameter

VALUE OF S_{D1}	RISK CATEGORY		
	I or II	III	IV
$S_{D1} < 0.067g$	A	A	A
$0.067g \leq S_{D1} < 0.133g$	B	B	C
$0.133g \leq S_{D1} < 0.20g$	C	C	D
$0.20g \leq S_{D1}$	D	D	D

For Risk Category = I and $S_{D1} = 1.322 g$, Seismic Design Category = D

Note: When S_1 is greater than or equal to $0.75g$, the Seismic Design Category is **E** for buildings in Risk Categories I, II, and III, and **F** for those in Risk Category IV, irrespective of the above.

Seismic Design Category $\equiv$ "the more severe design category in accordance with Table 11.6-1 or 11.6-2" = E

Note: See Section 11.6 for alternative approaches to calculating Seismic Design Category.

References

1. Figure 22-1: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-1.pdf
2. Figure 22-2: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-2.pdf
3. Figure 22-12: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-12.pdf
4. Figure 22-7: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-7.pdf
5. Figure 22-17: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-17.pdf
6. Figure 22-18: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-18.pdf

U.S. Geological Survey - Earthquake Hazards Program

Unified Hazard Tool



Please do not use this tool to obtain ground motion parameter values for the design code reference documents covered by the [U.S. Seismic Design Maps web tools](#) (e.g., the International Building Code and the ASCE 7 or 41 Standard). The values returned by the two applications are not identical.

^ Input

Edition

Dynamic: Conterminous U.S. 2008 (...)

Spectral Period

Peak ground acceleration

Latitude

Decimal degrees

33.564958

Time Horizon

Return period in years

475

Longitude

Decimal degrees, negative values for western longitudes

-115.973836

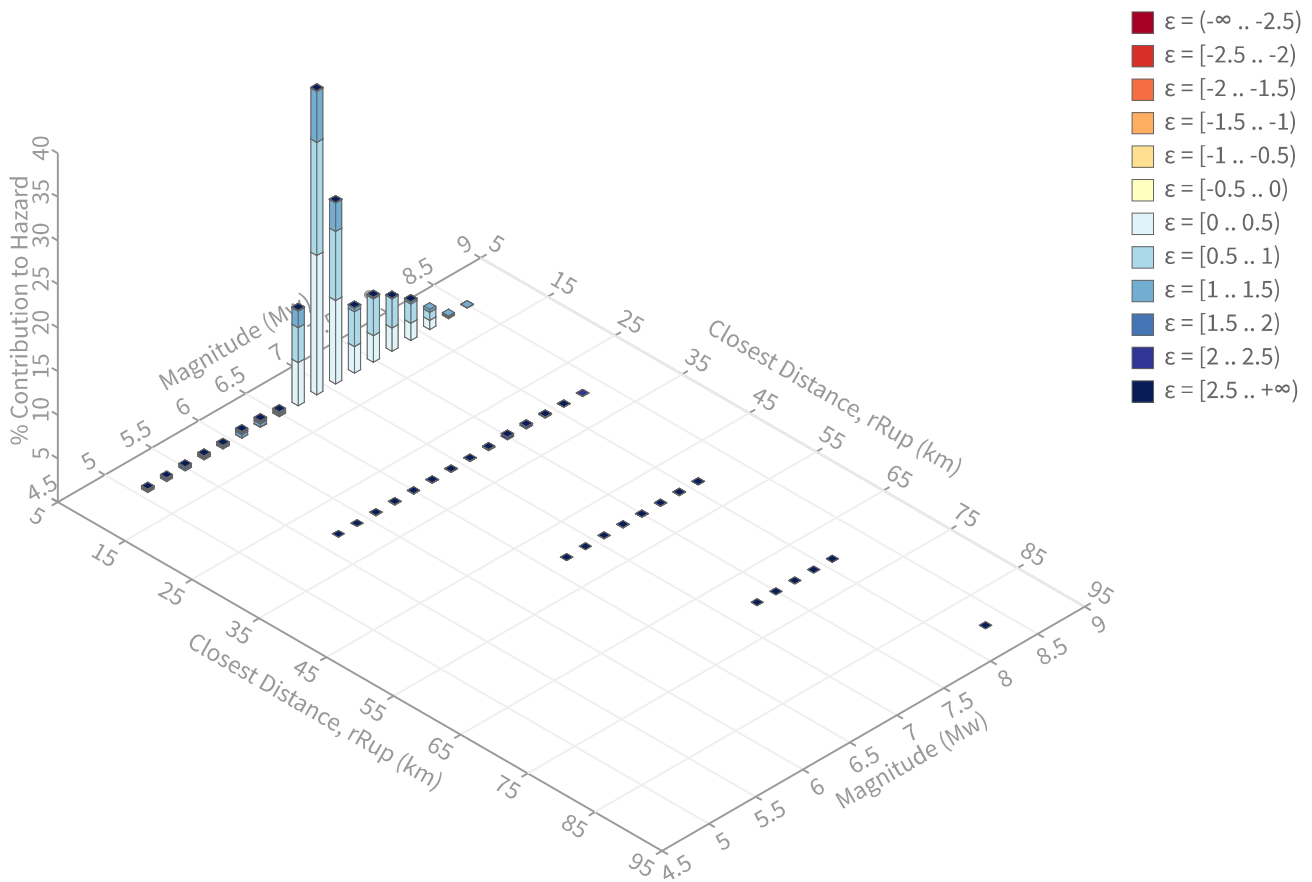
Site Class

259 m/s (Site class D)

^ Deaggregation

Component

Total



Summary statistics for, Deaggregation: Total

Deaggregation targets

Return period: 475 yrs

Exceedance rate: 0.0021052632 yr⁻¹

PGA ground motion: 0.69208721 g

Recovered targets

Return period: 498.87442 yrs

Exceedance rate: 0.0020045125 yr⁻¹

Totals

Binned: 100 %

Residual: 0 %

Trace: 0.13 %

Mean (for all sources)

r: 1.37 km

m: 7.08

ε₀: 0.67 σ

Mode (largest r-m bin)

r: 0.73 km

m: 6.9

ε₀: 0.63 σ

Contribution: 34.86 %

Mode (largest ε₀ bin)

r: 0.69 km

m: 6.9

ε₀: 0.44 σ

Contribution: 15.97 %

Discretization

r: min = 0.0, max = 1000.0, Δ = 20.0 km

m: min = 4.4, max = 9.4, Δ = 0.2

ε: min = -3.0, max = 3.0, Δ = 0.5 σ

Epsilon keys

ε0: [-∞ .. -2.5)

ε1: [-2.5 .. -2.0)

ε2: [-2.0 .. -1.5)

ε3: [-1.5 .. -1.0)

ε4: [-1.0 .. -0.5)

ε5: [-0.5 .. 0.0)

ε6: [0.0 .. 0.5)

ε7: [0.5 .. 1.0)

ε8: [1.0 .. 1.5)

ε9: [1.5 .. 2.0)

ε10: [2.0 .. 2.5)

ε11: [2.5 .. +∞]

Deaggregation Contributors

Source Set ↗ Source	Type	r	m	ϵ_0	lon	lat	az	%
aFault_MoBal	Fault							61.03
S. San Andreas : CO		0.69	6.93	0.63	115.712°W	33.350°N	134.45	51.91
S. San Andreas : BG+CO		0.40	7.35	0.50	115.712°W	33.350°N	134.45	3.25
S. San Andreas : SM+NSB+SSB+BG+CO		0.16	7.82	0.52	115.712°W	33.350°N	134.45	2.19
S. San Andreas : NSB+SSB+BG+CO		0.23	7.61	0.52	115.712°W	33.350°N	134.45	1.23
S. San Andreas : SSB+BG+CO		0.26	7.51	0.51	115.712°W	33.350°N	134.45	1.03
aFault_aPriori_D2.1	Fault							30.40
S. San Andreas : CO		0.69	6.95	0.63	115.712°W	33.350°N	134.45	14.49
S. San Andreas : BG+CO		0.40	7.35	0.49	115.712°W	33.350°N	134.45	5.20
S. San Andreas : SSB+BG+CO		0.26	7.51	0.51	115.712°W	33.350°N	134.45	2.85
S. San Andreas : NSB+SSB+BG+CO		0.23	7.61	0.52	115.712°W	33.350°N	134.45	2.80
S. San Andreas : SM+NSB+SSB+BG+CO		0.16	7.82	0.52	115.712°W	33.350°N	134.45	2.74
aFault_unseg	Fault							3.69
S. San Andreas		1.06	7.65	0.61	115.712°W	33.350°N	134.45	3.63
CAmap.21.ch.in (opt)	Grid							1.43
CAmap.24.ch.in (opt)	Grid							1.43