GEOTECHNICAL INVESTIGATION
DOGWOOD ROAD BRIDGE REPLACEMENT PROJECT
IMPERIAL COUNTY, CALIFORNIA

ENVIRONMENTAL
GEOTECHNICAL / CONSTRUCTION MATERIALS TESTING
INFRASTRUCTURE ENGINEERING
MUNICIPAL SERVICES
ASSET MANAGEMENT / CERTIFICATION

September 30, 2016
Revised July 25, 2017

NV5 West, Inc.
15092 Avenue of Science, Suite 200
San Diego, California 92128
(858) 715-5800
www.NV5.com

A NV5 Company – Offices Nationwide

Geotechnical Investigation

Dogwood Road Bridge Replacement Project
Imperial County, California

Prepared for:

County of Imperial
Department of Public Works
155 S. 11th Street
El Centro, California 92243

Attention: Mr. William Brunet, Director of Public Works

Project No.: 226816-00103.02

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Attention: **Mr. William Brunet, Director of Public Works**

Subject: Geotechnical Investigation

Project: Dogwood Road Bridge Replacement Project
Imperial County, California

Dear Mr. Brunet:

As requested, NV5 West, Inc. (NV5) is pleased to submit the results of our geotechnical investigation for the proposed Dogwood Road bridge replacement project in Imperial County, California. The purpose of this investigation was to evaluate the subsurface conditions at the bridge and approach areas and to provide geotechnical recommendations pertaining to the design and construction of the replacement bridge. The accompanying report includes a discussion of the subsurface soil conditions observed during our study, a review of available relevant geotechnical documents and geotechnical engineering analyses. Based on the results of the subsurface exploration, subsequent testing of the retrieved soil samples, and engineering analyses, it was concluded that the construction of the proposed project is geotechnically feasible provided the recommendations contained herein are appropriately incorporated into the design and implemented during construction.

It is recommended that the forthcoming project specifications, be reviewed by NV5 for consistency with the report prior to the bid process in order to avoid possible conflicts, misinterpretations, and inadvertent omissions, etc. It should also be noted that the applicability and final evaluation of recommendations presented herein are contingent upon construction phase field monitoring by NV5 in light of the widely acknowledged importance of geotechnical consultant continuity through the various design, planning and construction stages of a project.

NV5 appreciates the opportunity to provide this geotechnical engineering service for this project and looks forward to continuing our role as your geotechnical engineering consultant.

Respectfully submitted,

NV5 West, Inc.

Gene Custenborder

Gene Custenborder, CEG 1319
Senior Engineering Geologist

GENE CUSTENBORDER
No 1319
CERTIFIED ENGINEERING GEOLOGIST
Exp 11·30·16

Sam Koohi, PhD, PE 85010
Engineering Manager

YASSER KOOHI
No. C 85010
Exp. 03/31/18
CIVIL

Reviewed by,

Guillaume Gau, GE 2986
Senior Vice President

GUILLAUME GAU
No. GE 2986
Exp. 9/30/16
GEOTECHNICAL

GC/SK/GG:ma

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1.0 INTRODUCTION

This report presents the results of our geotechnical investigation conducted for the Dogwood Road Bridge replacement project in Imperial County, California. Included in this report is a summary of the data collected along with our findings, conclusions and geotechnical design recommendations for the project. This report has been prepared for the exclusive use of the client and their consultants in the design of the proposed project. In particular, it should be noted that this report should be considered by prospective construction bidders only as a source of general information, subject to interpretation and refinement by their own expertise and experience; particularly with regard to construction feasibility. Contract requirements set forth by the project plans and specifications will supersede any general observations and specific recommendations presented in this report.

1.1 Project Description

The Dogwood Road Bridge replacement project will replace the existing Dogwood Road Bridge across the Central Main Canal in Imperial County, California. We understand that the existing bridge, (BR. No. 58C-0042), was built in 1967. It is a three-span reinforced concrete slab with a total length of 82 feet. The superstructure consists of a concrete slab with integral caps at the bents. The substructure consists of diaphragm type abutments supported on driven piles. The bents also consist of driven pile extensions. The existing bridge has been affected by soil subsidence which has resulted in the loss of freeboard and the accumulation of debris at the upstream end of the bridge, causing a waterway restriction and the potential for future immersion.

Based on preliminary design information provided by NV5 Infrastructure, it is understood that the proposed new bridge will be a two-span structure approximately 91.5 feet in length, which will span the waterway with a mid-span pier within the canal. The bridge will be built from precast-prestressed voided slab girders with a reinforced cast-in-place composite concrete deck and a pier and abutments supported on cast-in-steel-shell (CISS) piles. It is anticipated that the new roadway profile will be raised 4 feet 2 inches over the current roadway. The locations of the project site in relation to surrounding streets and landmarks is presented on *Figure 1, Site Location Map*.

1.2 Purpose and Scope of Investigation

The purpose of the geotechnical investigation was to evaluate the subsurface conditions at the proposed bridge site and to provide geotechnical recommendations and parameters for the design and construction of the proposed replacement bridge. The scope of services for this project included the following tasks:

- Review of readily available background data, including in-house geotechnical data, in-house geotechnical reports, published geologic maps, topographic maps, seismic hazard maps and literature relevant to the subject site.

- Performing a site reconnaissance to observe the general surficial site conditions, check for accessibility, and select the exploratory boring locations.

- Coordinating with entities having an interest in the field exploration activities including the design team, the exploration subcontractor, Underground Service Alert and agencies associated with one-call notification.

- Conducting a subsurface investigation which included drilling four borings to a maximum depth of approximately 71.5 feet below the existing grade (see *Appendix A*).

- Performing laboratory testing on selected representative bulk and relatively undisturbed soil samples obtained during the field exploration program to evaluate the geotechnical engineering properties of these materials (see *Appendix B*).

- Performing an assessment of general seismic conditions and geotechnical hazards affecting the area and potential impacts on the subject project.

- Engineering evaluation of the data collected to develop geotechnical design parameters and recommendations for the proposed construction.

- Preparation of this report including reference maps and graphics, presenting our findings, conclusions and geotechnical design recommendations specifically addressing the following items:

    - Evaluation of general subsurface conditions and description of types, distribution, and engineering characteristics of subsurface materials.

    - Evaluation of project feasibility including excavatability, trench stability, and suitability of on-site soils for backfill.

    - Recommendations including site earthwork, foundation design and other geotechnical parameters to be used for the design of the project.

    - General construction considerations for the project.

1.3 Site Description

The project site is located north of the intersection of South Dogwood Road and Willoughby Road over the Central Main Canal in Imperial County, California. Dogwood Road is a heavily traveled north/south roadway that parallels highway 111 to the east. The bridge crosses the Central Main Canal perpendicularly. The earthen channel is unlined with side slopes inclined at approximately 2 to 1 (horizontal to vertical). The terrain at and around the bridge site is relatively flat except for the embankments on the various irrigation canals in the area. The existing bridge deck is currently at an elevation of approximately 4 feet below mean sea level.

The area around the site is generally agricultural. A solar generating plant (Ormat Nevada, Inc.), and a construction and aggregate production yard (Pyramid Construction and Aggregates, Inc.) are located on the west side of Dogwood Road just north of the bridge site. Existing improvements in the vicinity of the project include several unlined canals and drainages, various underground pipelines and overhead utilities.

2.0 FIELD EXPLORATION PROGRAM

Before starting our field exploration program, a field reconnaissance was conducted to observe site conditions and mark the location of our planned explorations. In accordance with state law, Underground Service Alert was notified of our operations for underground utility marking at the locations of exploration prior to excavation.

On March 24rd and 25th, 2016, four exploratory borings were drilled at the project site. The exploratory borings were drilled with a hollow stem auger drill rig to a maximum depth of approximately 71.5 feet below the existing ground surface. The borings were logged by an NV5 engineer and representative soil samples encountered were obtained for visual soils classification and laboratory testing. The soil conditions encountered in the borings were visually examined, classified, and logged in general accordance with the Unified Soil Classification System. The logs of the exploratory test borings are presented in *Appendix A, Exploratory Boring Logs*. The approximate locations of the borings are shown on *Figure 2, Geotechnical Map. The Log of Test Boring Sheets* are included as *Appendix B*.

Bulk and relatively undisturbed drive samples of the soils encountered in the borings were obtained in the field during our subsurface evaluation. The samples were tagged in the field and transported to our laboratory for observation and testing. The drive samples were obtained using the California Modified Split Spoon and Standard Penetration Test (SPT) samplers, as described below.

California Modified Split Spoon Sampler

The split barrel drive sampler was driven with a 140-pound hammer allowed to drop freely 30 inches in general accordance with ASTM D1587. The number of blows for the last two of three 6-inch intervals were recorded during sampling and are presented in the logs of borings. The sampler has external and internal diameters of approximately 3.0 and 2.4 inches, respectively, and the inside of the sampler is lined with 1-inch-long brass rings. The relatively undisturbed soil samples within the rings were removed, sealed, and transported to the laboratory for observation and testing.

Standard Penetration Test (SPT) Sampler

A split barrel sampler was driven with a 140-pound hammer allowed to drop freely 30 inches in general accordance with ASTM D1586. The numbers of blows for the last two of three 6-inch intervals were recorded during sampling and are presented in the logs of borings (i.e., N-value). The sampler has external and internal diameters of 2.0 and 1.5 inches, respectively. The soil samples obtained in the interior of the barrel were measured, removed, sealed and transported to the laboratory for observation and testing.

3.0 LABORATORY SOIL TESTING

Laboratory testing was performed on selected representative bulk and relatively undisturbed soil samples obtained from the exploratory borings to aid in the material classifications and to evaluate engineering properties of the materials encountered. The following tests were performed:

- In-situ density and moisture content (ASTM D2937 and ASTM D2216).
- Particle size analyses and No. 200-wash (ASTM D422 and ASTM D1140).
- Hydrometer Analysis (ASTM D4221)
- Expansion Index (ASTM D4829).
- R-Value (ASTM D2844)
- Atterberg Limits (ASTM D4318).
- Direct Shear (ASTM D3080).
- Maximum Density-Optimum Moisture Content (ASTM D1557).
- Corrosivity series including sulfate content, chloride content, pH-value, and resistivity (CTM 417, 422, and 532/643).

Testing was performed in general accordance with applicable ASTM standards and California Test Methods. A summary of the laboratory testing program and the laboratory test results are presented in *Appendix C, Laboratory Test Results*.

4.0 GEOLOGY

4.1 Geologic Setting

The project site is located in Imperial County within the Colorado Desert geomorphic province. This province is a low-lying barren desert basin (in part about 245 feet below sea level) dominated by the Salton Sea. The province is a depressed block between active branches of the San Andreas fault system. The fault branches are buried by recent alluvial deposits. The dominant structural features related to the San Andreas fault system consist of northwest-trending faults and fault zones. The major northwest-trending fault zones include the Imperial fault, the Superstition Hills fault, the Elsinore fault and the San Andreas fault. The province is characterized by the ancient beach lines and silt deposits of extinct Lake Cahuilla.

4.2 Geologic Materials

Geologic units encountered in the subsurface exploration consisted natural deposits mapped as Quaternary Lake Deposits (Ql) on published geologic maps. *Figure 3, Regional Geologic Map,* shows the geology of the site area based on published geologic mapping. The natural deposits generally consisted of tan to red-brown to dark brown, moist to wet, medium stiff to stiff, silty to sandy clay sand with moderate to high plasticity, and brown, wet to saturated, loose to medium dense sand to clayey sand. Minor, locally derived fill soils, associated with construction of the roadways and canals are also present. Descriptions of the materials encountered are also presented on the *Logs of Exploratory Borings in Appendix A*, and the *Log of Test boring Sheets in Appendix B*.

4.3 Groundwater

Groundwater was encountered in borings B-2 at a depth of approximately five feet below the existing ground surface and in B-3 at a depth of approximately ten feet below the existing ground surface (approximate elevation nine and 14 feet below mean sea level, respectively). Groundwater levels are also indicated on the *Log of Test Boring Sheets in Appendix B*. Groundwater was not encountered in borings B-1 located northerly of the bridge site and B-4 located southerly of the site. Groundwater conditions can be expected to vary due to seasonal precipitation, irrigation, and other factors.

5.0 SEISMIC AND GEOTECHNICAL HAZARDS

The findings of our seismic and geotechnical hazards evaluation for the proposed project are summarized in the sections below.

5.1 Faults

The numerous faults in southern California include active, potentially active, and inactive faults. As used in this report, the definitions of fault terms are based on those developed for the Alquist-Priolo Special Studies Zones Act of 1972 and published by the California Division of Mines and Geology (Hart and Bryant, 1997).

Active faults are defined as those that have experienced surface displacement within Holocene time (approximately the last 11,000 years) and/or have been included within any of the state-designated Earthquake Fault Zones (previously known as Alquist-Priolo Special Studies Zones). Faults are considered potentially active if they exhibit evidence of surface displacement since the beginning of Quaternary time (approximately two million years ago) but not since the beginning of Holocene time. Inactive faults are those that have not had surface movement since the beginning of Quaternary time.

Review of geologic maps and literature pertaining to the general site area indicates that the site is not located within an Earthquake Fault Zone delineated by the State of California for the hazard of fault surface rupture. In addition, there are no known major or active faults mapped or known to cross or trend toward the project site. Evidence of active faulting was not encountered during our field investigation or in our research of available published geologic maps. Therefore, the potential for damage due to surface rupture of faults at the project site is considered low during the design life of the proposed project.

5.1.1 Active Faults

The closest known active faults to the site are the Cerro Prieto fault and the Imperial fault, located approximately 6.1 miles southwest and 6.9 miles northeast of the project site, respectively. Other important active faults that could affect the project area and their distance to the site are included in included in the following *Table 1*. *Figure 4, Regional Fault Map*, depicts the site in relation to known active faults in the region.

Table 1
Distance From the site to Known Active Faults

Fault	Approximate Distance From Site
Cerro Prieto	6.1 miles
Imperial fault	6.9 miles
Brawley fault	8.9 miles
Superstition Hills fault	9.4 miles
San Jacinto fault	15.4 miles
Laguna Salada fault	15.7 miles
Elsinore fault	28.0 miles
San Andreas fault	45.4 miles

5.2 Ground Shaking

Although the site could be subjected to strong ground shaking in the event of an earthquake, this hazard is common in California and the effects of ground shaking can be attenuated if the improvements are designed and constructed in conformance with current building codes and engineering practices (see *Section 6.5, Seismic Design*).

5.3 Liquefaction and Dynamic Settlement

Liquefaction and dynamic settlement of soils can be caused by ground shaking during earthquakes. Research and historical data indicate that loose, relatively clean granular soils are susceptible to liquefaction and dynamic settlement, whereas the stability of the majority of clayey silts, silty clays and clays is not adversely affected by ground shaking. Liquefaction is generally known to occur in saturated loose cohesionless soils at depths shallower than approximately 50 feet. The potential for liquefaction under the same conditions of ground shaking intensity and duration will decrease for sands that are more well graded, more irregular and gritty, coarser and denser. Also, a pronounced decrease in liquefaction potential will occur with the increase in fine-grained (i.e., silt and clay) content. Idriss and Boulanger (2008, 2014) suggested that if the plasticity index of the soil is greater than 7, the soil can be considered non-liquefiable. Dynamic settlement due to earthquake shaking can occur in both dry and saturated sands. The potential consequences of liquefaction to engineered structures include loss of bearing capacity, buoyancy forces on underground structures (including pipelines), increased lateral earth pressures on retaining walls, and lateral spreading.

The proposed bridge will be located within the poorly to moderately consolidated alluvial materials. The subsurface exploration program encountered loose to medium dense alluvial sand with varying contents of clay and silt, along with a shallow ground water table. The alluvial sands were encountered between approximately 35 feet to 50 feet below existing grade.

Liquefaction analyses were performed using the program SPTLIQ (InfraGEO Software, 2017). The proposed procedure by Boulanger and Idriss (2014) was used, which consists of comparing a Cyclic Stress Ratio (CSR, earthquake “load”) to the Cyclic Resistance Ratio (CRR, soil “strength”) of the soil. The CRR calculations were based upon input data obtained from the test borings. All of the potential liquefaction induced settlements were performed using the method proposed by Idriss and Boulanger (2008) method.

Liquefaction analyses were performed utilizing the field and laboratory test data. A design peak ground acceleration (PGA) value of 0.65g and a modal earthquake moment magnitude of Mw=6.7 was used based on the results of seismic deaggregation of the site, The groundwater level (GWL) utilized in the analyses was 5 feet below existing grade. Appendix E, Liquefaction Analyses Results, contains the input data file and a graphical output identifying the potentially liquefiable zones.

Based on our analysis, it is estimated that up to 1.6 inches of total seismic settlement could occur within the bridge footprint for the design-event earthquake. In addition, differential settlements up to 0.8 inch could be expected. In summary, the analyses indicate that there is a potential for liquefaction, seismically-induced settlement and associated ground damage for the design-event earthquake. Foundation recommendations to address the liquefaction potential are included in Section 6.2.

5.4 Slope Stability and Landslides

There are no high or steep slopes on or in close proximity to the project site. Based on the investigation, there appears to be no indications of landslides or deep-seated instability at the site.

5.5 Tsunami and Seiches

Although the site and surrounding areas are at an elevation approximately at or several feet below mean sea level, the site is approximately 80 miles from the Sea of Cortez, the closest sea to the site. Due to the distance, tsunamis (seismic sea waves) are not considered a hazard at the site.

The site is not located near to or downslope of, any large body of water that could affect the site in the event of an earthquake-induced failure or seiche (oscillation in a body of water due to earthquake shaking). The Salton Sea is located approximately 28.6 miles north of the site and is therefore not considered a hazard to the site in terms of a seismically induced seiche.

5.6 Subsidence

The site is located in an area with documented ground subsidence. According to an evaluation report for the Dogwood Road Bridge (NV5, 2015), the existing bridge structure had over 24 inches of freeboard when it was first built in 1967. According to survey data provided by the County of Imperial and the Imperial Irrigation District (IID), the structure appears to have settled about 23 inches since 1981. The subsidence data was collected through surveys from 1981 to 2012. The report concluded that the subsidence is due to withdrawal of groundwater. Typically soil subsidence occurs when groundwater (near the surface or in a deep aquifer) is lowered past its historical level. This occurrence results in an increase of effective stress within a soil layer which typically translates into additional soil consolidation. The report also concluded that the subsidence is likely to continue at an average rate of 0.72 inches per year. The recommended proposed foundation system cannot be designed to mitigate the on-going soil subsidence.

6.0 DESIGN RECOMMENDATIONS

6.1 General

Based on the results of the field exploration and engineering analyses, it is NV5’s opinion that the proposed Dogwood Road Bridge Replacement is feasible from a geotechnical standpoint, provided the recommendations in this report are incorporated into the design plans and implemented during construction. The following sections present detailed recommendations and parameters pertaining to the geotechnical engineering design for this project.

6.2 Foundation Recommendations

Based on the results of our investigation, Cast-In-Steel-Shell (CISS) piles are recommended for foundation support of the proposed Dogwood Road Bridge replacement. CISS piles are recommended to reduce the potential for construction difficulties due to caving of the loose sandy layers and to withstand the large lateral loads due to potential extreme events. Detailed recommendations including vertical pile tip elevations, lateral capacity, estimated settlement, pile design and construction considerations, and embankment recommendations are provided in the following sections of the report.

6.2.1 Vertical Design and Pile Tip Elevations

The anticipated pile diameters and structural loads of the CISS piles at the abutments and piers were provided by the structural engineer and are summarized in Appendix D in Table D-1.

The ultimate axial capacity for 30-inch diameter CISS piles in both compression and tension for the provided load cases were computed using the static method of analysis using the computer program AXIALCAP (InfraGEO Software, 2017). The axial capacity, in both compression and tension, for 30-inch diameter CISS piles were calculated using the method proposed by NAVFAC 7.02 manual (1986).

The results of liquefaction analysis are presented in Appendix E. Based on the analysis, a medium dense sandy layer from 35 to 40 feet below existing ground was identified as liquefiable during a design seismic event with a total seismic settlement potential of 1.6 inches. Therefore, we have taken into account in our analysis the potential for occurrence of downdrag forces on the piles due to liquefaction.

The specified tip elevations for abutment and pier piles in both static and extreme events for compression, uplift and lateral loads are presented in Appendix D in Table D-1. We use a factor of safety of 2.0 for skin friction for static (service loading) case and 1.0 for seismic case including the effects of liquefaction. We neglected end bearing in our analysis because the movement associated with mobilizing the end bearing is typically beyond tolerable structural limits. The axial pile capacity curves versus embedment are presented in Appendix D.

6.2.2 Lateral Capacity

It is our understanding that the structural engineer will use the computer program LPILE to perform pushover analyses and determine the depth of fixity of the proposed CISS piles. We have estimated the relevant geotechnical input parameters for LPILE based on the current investigation. The recommended geotechnical input parameters for LPILE are presented in Table 2.

Table 2
Recommended Soil Parameters for LPILE

Elevation (feet)	Soil Type	P-Y Model	Unit Weight(pcf)	ϕ (deg)	C (psf)	K (pci)	ε_{50} (in/in)
El. 996.85- El. 995.00(above water table)	Clay	Soft Clay	120	0	200	-	0.02
El. 995.00- El. 991.85 (below water table)	Clay	Soft Clay	57.6	0	200	-	0.02
El. 991.85- El. 966.85	Clay	Soft Clay	57.6	0	500	-	0.02
El. 966.85- El. 946.85 (use p-multiplier=0.1 for extreme event)	Sand	API Sand	57.6	32	0	40	-
El. 946.85- El. 926.85	Sand	API Sand	57.6	34	0	60	-

6.2.3 Estimated Settlement

We estimate the settlement of the proposed bridge supported on CISS piles in the manner recommended to be less than 0.5 inch. A detailed settlement analysis for deep foundations was beyond the scope of this study. We would be pleased to perform a detailed settlement analysis on a case-by-case basis, if requested.

6.2.4 CISS Pile Design and Construction Considerations

Caltrans standard specifications and special provisions for “Cast-in-Steel-Shell (CISS) Piling” should apply for construction of the CISS piles. Groundwater is expected during pile construction. The design groundwater level is at 5 feet below the existing surface. However, at the time of construction, the groundwater elevation may be different due to seasonal fluctuations or other conditions.

If the composite section action is used in the analysis, the design engineer must assure that a reliable bond exists between the casing and concrete. Welded studs or shear rings may be required, especially for large diameter piles.

Open ended or closed ended pipes can be used as casing for CISS piles. The close ended piles are more difficult to drive in very dense granular soils, very hard cohesive soils or soft rock. Generally, pipe piles up to 400 mm in diameter will be plugged during driving while diameters 600 mm and greater will not be plugged. After the pipe is plugged, an open-ended pipe behaves like a displacement pile and driving becomes more difficult. The Geotechnical Engineer may recommend center relief drilling against excessive blow counts or high driving stresses to reach the planned tip elevation. A drivability analyses can be performed to calculate the pile driving stresses and check the required thickness of the pipe.

For open ended piles a plug two diameters in length can usually seal the bottom of the pile, but a seal course may be required for some combinations of high water pressure and sandy soils.

6.3 Approach Fill Earthwork

The preliminary project plans indicate up to approximately 6 feet of fill is planned at the approach to the two abutments. We do not anticipate fill settlement issue at the abutments provided that the embankments are constructed and compacted according to Caltrans specifications. The portion of abutment inside the channel is proposed at a gradient of 2 to 1 (horizontal to vertical) and should be constructed in accordance with Caltrans specifications.

The entire footprint of areas to receive new fill shall be cleared and grubbed of any vegetation. Any loose soils or undocumented fill should be removed and stockpiled for re-use as compacted fill. Excavations in the onsite soils can be accomplished by conventional heavy-duty excavating equipment in good operating condition. After clearing/grubbing and removal of any loose soils or undocumented fill, the subgrade shall be proof-rolled with loaded heavy equipment under the observation of competent geotechnical personnel. If loose, soft, or pumping areas are observed additional excavation shall be performed as recommended by the geotechnical professional.

If proof rolling is successful, the subgrade shall be scarified and recompacted. Abutment wall backfill shall conform to Caltrans Standard Specifications for structural fill. All earthwork and grading should conform to Sections 16 through 22 of the current edition of Caltrans Standard Specifications. All new fill placed within 150 feet of the abutments should have a relative compaction of 95 percent (based on ASTM D1557) and an expansion index (EI) of less than 50. Positive surface drainage away from foundation areas should be maintained.

6.4 Temporary Excavations

Temporary, shallow excavations with vertical side slopes less than 4 feet high will generally be stable, although there is a potential for localized sloughing; in these soil types vertical excavations greater than 4 feet high should not be attempted without proper shoring to prevent local instabilities. Shoring may be accomplished with hydraulic shores and trench plates, and/or trench boxes, soldier piles and lagging. The actual method of a shoring system should be provided and by a contractor experienced in installing temporary shoring under similar soil conditions and designed by an experienced licensed professional. If soldier piles and lagging are to be used, we should be contacted for additional recommendations.

All trench excavations and access pits should be shored in accordance with CalOSHA regulations. For your planning purposes, the native soil materials may be considered a Type C, as defined the current CalOSHA soil classification.

The excavation support system should be designed to resist lateral earth pressures of the soil and hydrostatic pressures. It is common practice for an experienced contractor to design and install shoring structure. The preliminary shoring design parameters are provided as follows for reference. The final design of the temporary shoring should be reviewed by the project geotechnical engineer.

For the design of a cantilever soldier piles and lagging shoring system the structure should be designed to resist the lateral earth, water, and surcharge loadings. For the subsurface conditions at this site, the unfactored earth pressure distribution (p in psf) can be calculated as follows:

$$P = K.\gamma.H + Surcharge\ 1$$

Where

H= height of the excavation

γ = soil unit weight, where for above water ground is 120 pcf, and for below water level is γ'=58 pcf

K_0=0.5 At rest earth pressures should be assumed for the geotechnical design, where the wall support does not allow lateral displacement

K_a=0.3 active earth pressure should be assumed for the geotechnical design, where the wall support allow for lateral yielding

Surcharge 1: The surcharge for typical construction activities, a minimum of 2 feet equivalent soil surcharge is recommended

Hydrostatic pressures acting below the groundwater table should be considered in shoring designs.

Stockpiled (excavated) materials should be placed no closer to the edge of a trench excavation than a distance defined by a line drawn upward from the bottom of the trench at an inclination of 1(H): 1(V), but no closer than 4 feet. All trench excavations should be made in accordance with CalOSHA requirements.

6.5 Seismic Design

Seismic design parameters were developed as per the guidelines outlined in the 2012 IBC (2013 CBC) and ASCE 7-10 (W/March 2013 errata) Standard. NV5 should be contacted to provide revisions to these parameters if other codes are specified. The seismic design parameters for Site Class “D” and Risk Category IV (essential facilities) were developed using a JAVA ™ application, Java Ground Motion Parameter Calculator–Version 5.0.9 available on the USGS website (http://earthquake.usgs.gov) and Caltrans ARS Online (v2.3.07). The preliminary seismic design parameters for the project site are presented in the following Table 3.

Table 3
2012 IBC (2013 CBC) Seismic Design Parameters And ASCE 7-10 Standard and Caltrans ARS Online (v2.3.07)

Parameter	Value
Site Class; (Table 1613.3.2)	D
Maximum Magnitude (MMax)	6.7
Shear Wave Velocity, V_{s30} (m/s)	270
Peak Ground Acceleration (PGA)	0.65g

The values for the envelope of the spectral response based on ARS Online Version 2.3.07 (March 2016) are presented in the following Table 4. The spectral acceleration curve for the envelope is presented in Figure 5.

Table 4
Spectral Envelope Values Based on Caltrans ARS Online (v2.3.07)

Period (sec)	S_A
0.01	0.652
0.05	0.978
0.1	1.164
0.15	1.334
0.2	1.470
0.25	1.465
0.3	1.462
0.4	1.365
0.5	1.294
0.6	1.225
0.7	1.175
0.85	1.090
1	1.018
1.2	0.863
1.5	0.705
2	0.543
3	0.333
4	0.235
5	0.188

6.6 Retaining Wall Design

Conventional cantilever retaining walls backfilled with compacted non-expansive granular soil may be designed for active pressures of 36 pcf of equivalent fluid weight for well-drained, level backfill. For at rest (restrained condition), an equivalent fluid pressure of 55 pcf may be used in design. For seismic design consideration, an equivalent fluid pressure of 12 pcf with an inverted triangular distribution may be used. This value is in addition to the static pressure. A passive pressure of 400 psf per foot of depth is recommended.

Retaining wall backfill should be free draining, non-expansive material compacted at near optimum moisture conditions to at least 90% of maximum dry density as determined by ASTM D 1557.

Lateral earth pressure to be resisted by the retaining walls or similar structures should be increased to allow for surcharge loads. The surcharge considered should include the loads from any structures traffic or other temporary loads that would influence wall design.

A backdrain or an equivalent system of backfill drainage should be incorporated into retaining wall design. Backfill immediately behind retaining structures should be a free-draining granular material a minimum of 1 foot thick and extending to within 12 inches of the top of the backfill. The granular backfill should be capped with about 12 inches of on-site soils. Efflorescence on the face of walls can be mitigated by water proofing which, if done, should be according to the Architect or Civil Engineer's recommendations. Surface runoff should be directed away from the retaining wall and backfill and towards a suitable drainage disposal system.

Compaction on the uphill side of walls within a horizontal distance equal to one wall height should be performed by hand-operated or other lightweight compaction equipment. This is intended to reduce potential "locked-in" lateral pressures caused by compaction with heavy grading equipment.

6.7 Pavement Recommendations

Preliminary pavement sections were developed for the project based on laboratory R-value tests performed on near surface soil samples. Laboratory testing indicated R-values of 5 and 8 for the near-surface soils (Appendix C). Various pavement sections were calculated using an R-value of 5 and assumed traffic indices ranging from 6.0 to 12.0. The project Civil Engineer should select the appropriate pavement section based on the anticipated traffic loads. NV5 can provide alternate sections based on other traffic loadings, if requested. Based on these design parameters, analysis in general accordance with the current Cal-Trans Highway Design Manual, and assuming compliance with site preparation recommendations, NV5 recommends the pavement structural sections in the following Table 5:

Table 5

Flexible Asphalt Pavement Sections

Traffic Index (TI)	*Pavement Section*	
	AC[(1)] *(inches)*	*AB*[(2)] *(inches)*
6.0	4.0	13.0
8.0	5.0	18.0
10.0	6.0	24.0
12.0	7.5	30.0

(1) Asphalt Concrete;

(2) Crushed Aggregate Base (CAB), Green Book section 200-2.2, compacted to at least 95% relative compaction (ASTM D-1557);

Note: The upper 12-inches of subgrade soils should be compacted to at least 95% relative compaction (ASTM D-1557).

It is recommended that R-value testing be performed on representative soil samples after rough grading operations on the upper 2 feet to confirm applicability of the above pavement sections.

The aggregate base should conform to the Crushed Aggregate Base per Greenbook requirements, Section 200-2.2. The base course should be compacted to a minimum dry density of 95% of the materials maximum density as determined by the ASTM D1557 test procedure. Field testing should be used to verify compaction, aggregate gradation, and compacted thickness.

The asphalt concrete pavement should be compacted to 95% of the unit weight as tested in accordance with the Hveem procedure. The maximum lift thickness should be two inches. The asphalt concrete material shall conform to Type III, Class C2 or C3, 2009 edition of the Greenbook Standard Specifications for Public Works Construction. An approved mix design should be submitted 30 days prior to placement. The mix design should include proportions of materials, maximum density and required lay-down temperature range. Field testing should be used to verify oil content, aggregate gradation, compaction, compacted thickness, and lay-down temperature.

If the paved areas are to be used during construction, or if the type and frequency of traffic is greater than assumed in the design, the pavement section should be re-evaluated for the anticipated traffic.

The aggregate base should be compacted to a minimum dry density of 95% of the materials maximum density as determined by the ASTM D1557 test procedure. Field testing should be used to verify compaction, aggregate gradation, and compacted thickness.

6.7 Soil Corrosion

Caltrans Corrosion Guidelines (version 2.0, November 2012) define a corrosive area as an area where the soil contains more than 500 ppm of chlorides, more than 2,000 ppm of sulfates or has a pH of less than 5.5. Laboratory testing was performed on a representative sample of the on-site soils to evaluate chloride and sulfate content as well as pH and minimum resistivity. Table 6 presents the results of the corrosivity testing.

Table 6
Corrosion Test Data

Boring No.	Depth (feet)	pH	Resistivity (ohm-cm)	Chloride Content (ppm)	Sulfate Content (ppm)
B-3	10-11.5	8.7	900	32	170

The onsite soils do not classify as corrosive in accordance with Caltrans criterial. It is our recommendation that a corrosion specialist be consulted to evaluate the need for corrosion protection mitigation.

6.8 Backfill

The on-site silt and clay soil may not be suitable for backfill of trenches or buried structures. The on-site sandy soils may be used for backfill provided they are free of any contaminated soil, debris, organic matter, or other deleterious materials. Any rock or other soil fragments greater than 3 inches in size should not be used in backfill. All imported backfill, if any, should consist of granular, non-expansive soil with an Expansion Index of 20 or less. Import material should be evaluated by our firm prior to transport to the site and not contain any contaminated soil, expansive soil, debris, organic matter, or other deleterious materials.

The moisture content of the backfill should be maintained within 2% of optimum moisture content during compaction, and backfill should be placed in loose horizontal lifts not more than 8 inches in loose thickness and compacted to at least 90 percent of the maximum dry density as evaluated by the latest version of ASTM D1557. Backfill should be mechanically compacted. Flooding or jetting is not recommended.

7.0 DESIGN REVIEW AND CONSTRUCTION MONITORING

Geotechnical review of plans and specifications is of paramount importance in engineering practice. The poor performance of many structures has been attributed to inadequate geotechnical review of construction documents. Additionally, observation and testing of the subgrade will be important to the performance of the proposed improvements. The following sections present our recommendations relative to the review of construction documents and the monitoring of construction activities.

7.1 Plans and Specifications

The design plans and specifications should be reviewed and approved by NV5 prior to bidding and construction, as the geotechnical recommendations may need to be reevaluated in the light of the actual design configuration. This review is necessary to evaluate whether the recommendations contained in this report and future reports have been properly incorporated into the project plans and specifications.

7.2 Construction Monitoring

Site preparation, removal of unsuitable soils, assessment of imported fill materials, fill placement, and other earthwork operations should be observed and tested. The substrata exposed during the construction may differ from that encountered in the test borings. Continuous observation by a representative of NV5 during construction allows for evaluation of the soil/rock conditions as they are encountered, and allows the opportunity to recommend appropriate revisions where necessary.

8.0 LIMITATIONS

The recommendations and opinions expressed in this report are based on NV5's review of background documents and on information developed during this study. It should be noted that this study did not evaluate the possible presence of hazardous materials on any portion of the site. More detailed limitations of the supplemental geotechnical study are presented in the ASFE's information bulletin in *Appendix F*.

Due to the limited nature of our field explorations, conditions not observed and described in this report may be present on the site. Uncertainties relative to subsurface conditions can be reduced through additional subsurface exploration. Additional subsurface evaluation and laboratory testing can be performed upon request. It should be understood that conditions different from those anticipated in this report may be encountered during the proposed structure construction operations.

Site conditions, including ground-water level, can change with time as a result of natural processes or the activities of man at the subject site or at nearby sites. Changes to the applicable laws, regulations, codes, and standards of practice may occur as a result of government action or the broadening of knowledge. The findings of this report may, therefore, be invalidated over time, in part or in whole, by changes over which NV5 has no control.

NV5's recommendations for this site are, to a high degree, dependent upon appropriate quality control of subgrade preparation, fill/backfill placement, and foundation construction. Accordingly, the recommendations are made contingent upon the opportunity for NV5 to observe grading operations and foundation excavations for the proposed construction. If parties other than NV5 are engaged to provide such services, 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 recommendations in this report and/or by providing alternative recommendations.

This document is intended to be used only in its entirety. No portion of the document, by itself, is designed to completely represent any aspect of the project described herein. NV5 should be contacted if the reader requires additional information or has questions regarding the content, interpretations presented, or completeness of this document.

NV5 has endeavored to perform this study using the degree of care and skill ordinarily exercised under similar circumstances by reputable geotechnical professionals with experience in this area in similar soil/rock conditions. No other warranty, either expressed or implied, is made as to the conclusions and recommendations contained in this study.

9.0 SELECTED REFERENCES

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Figures

Reference: Google Maps 2016

NV5
An NV5 West, Inc. Company – Offices Nationwide
10592 Avenue of Science, Suite 200
San Diego, CA
Tel: (858) 715-5800, Fax: (858) 715-5810

Project No: **22816-00103**
Drawn: **SK**
Date: **July 2016**

Site Location Map
Dogwood Rd Bridge Replacement
Imperial County, California

Figure No. 1

0 200 400 600 800 1000

Approximate scale in feet

MAP SYMBOLS

◐ Approximate location of exploratory boring

B-4

GEOLOGIC UNITS

Ql Quaternary lake deposits

→ N →

Base map adapted from Google Maps 2016

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10592 Avenue of Science, Suite 200
San Diego, CA
Tel: (858) 715-5800, Fax: (858) 715-5810

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Drawn: **GC**

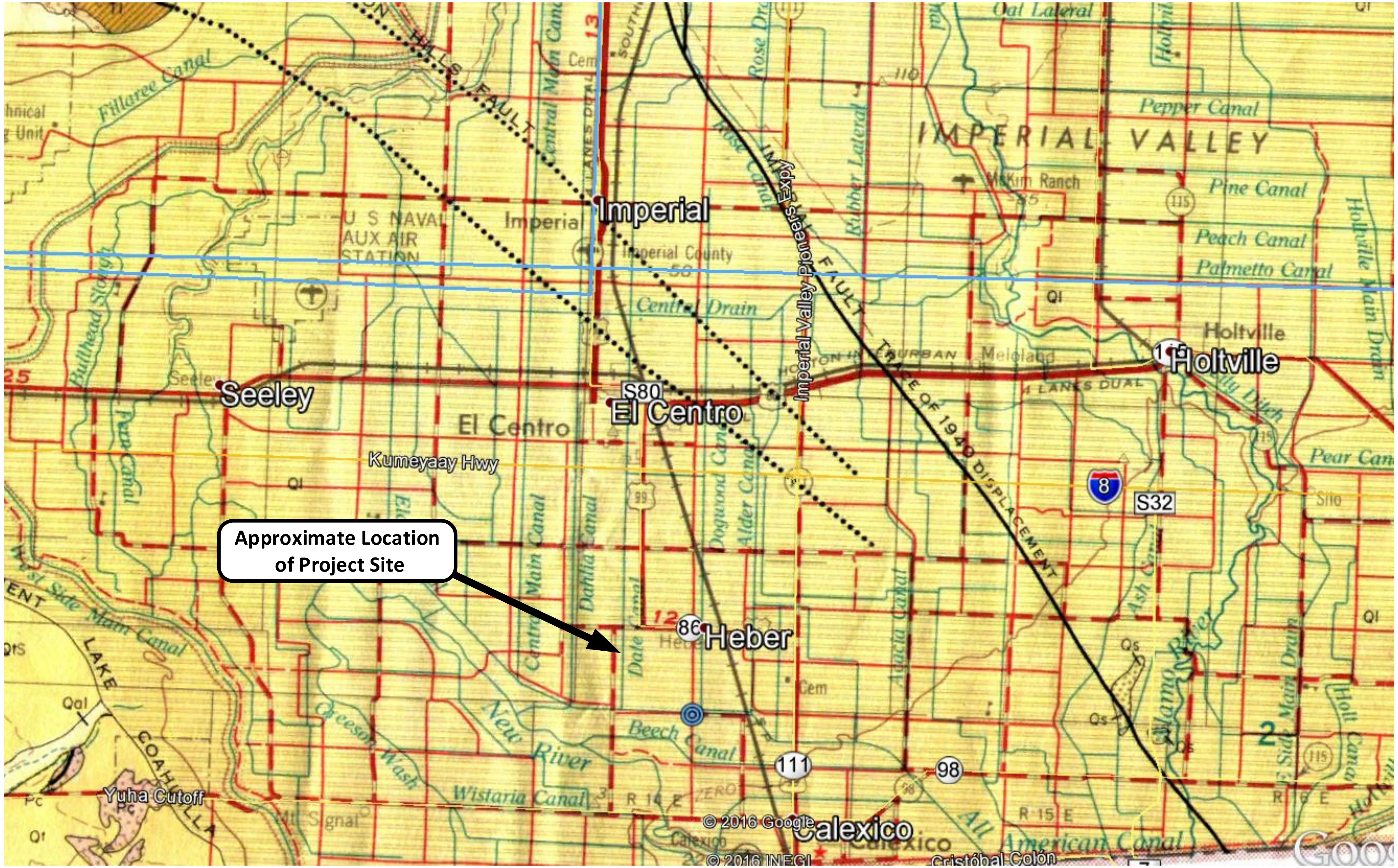
Date: **July 2016**

Geotechnical Map

Dogwood Road Bridge Replacement

Imperial County, California

Figure No. 2



DESCRIPTION OF MAP UNITS

Qs	Dune sand
Qal	Alluvium
Ql	Lake deposits

N

0 1 2 3 4 5 miles

Scale Approximate

Reference: Jenkins, Olaf P, compiled by Strand, R.G., 1962, Geologic Map of California, San Diego-El Centro Sheet: California Department of Conservation, Division of Mines and Geology, map scale 1:250,000.

NV5
An NV5 West, Inc. Company – Offices Nationwide
10592 Avenue of Science, Suite 200
San Diego, CA
Tel: (858) 715-5800, Fax: (858) 715-5810

Project No: **226816-00103.02**
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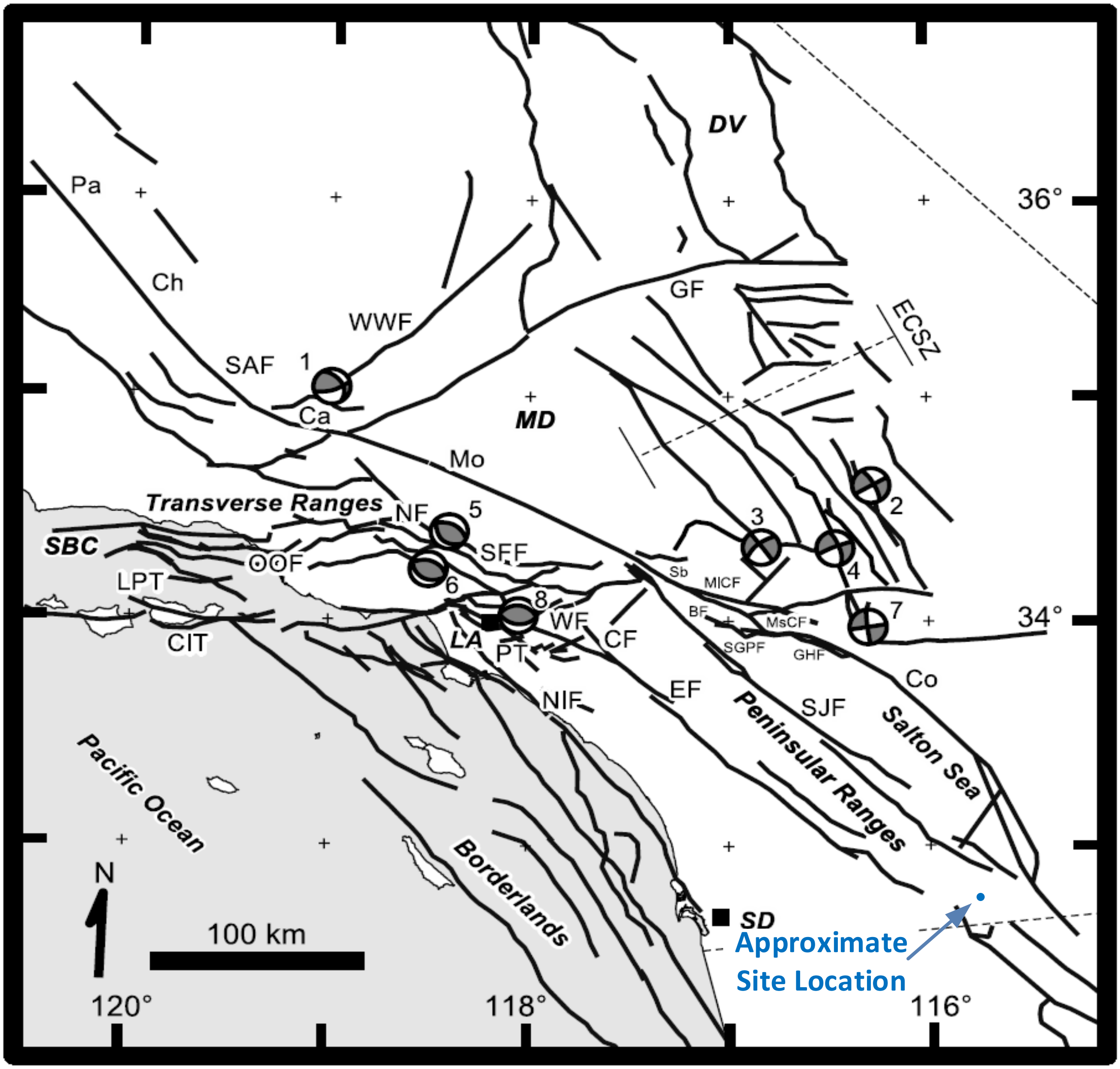
Regional Geologic Map
Dogwood Road Bridge Replacement
Imperial County, California

Figure No. 3



Map of southern California showing the geographic regions, faults and focal mechanisms of the more significant earthquakes. **Regions:** Death Valley, DV; Mojave Desert MD; Los Angeles, LA; Santa Barbara Channel, SBC; and San Diego, SD. **Indicated Faults:** Banning fault, BF; Channel Island thrust, CIT; Chino fault, CF; Eastern California Shear Zone, ECSZ; Elsinore fault, EF; Garlock fault, GF; Garnet Hill fault, GHF; Lower Pitas Point thrust, LPT; Mill Creek fault, MICF; Mission Creek fault, MsCF; Northridge fault, NF; Newport Inglewood fault, NIF; offshore Oak Ridge fault, OOF; Puente Hills thrust, PT; San Andreas fault (sections: Parkfield, Pa; Cholame, Ch; Carrizo; Ca; Mojave, Mo; San Bernardino, Sb; and Coachella, Co); San Fernando fault, SFF; San Gorgonio Pass fault, SGPF; San Jacinto fault, SJF; Whittier fault, WF; and White Wolf fault, WWF. **Earthquake Focal Mechanisms:** 1952 Kern County, 1; 1999 Hector Mine, 2; 1992 Big Bear, 3; 1992 Landers, 4; 1971 San Fernando, 5; 1994 Northridge, 6; 1992 Joshua Tree, 7; and 1987 Whittier Narrows, 8.

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For Schematic Use Only-Not a Construction Drawing

NV5
An NV5 West, Inc. Company – Offices Nationwide
10592 Avenue of Science, Suite 200
San Dlego, CA
Tel: (858) 715-5800, Fax: (858) 715-5810

Project No: **226816-00103.02**
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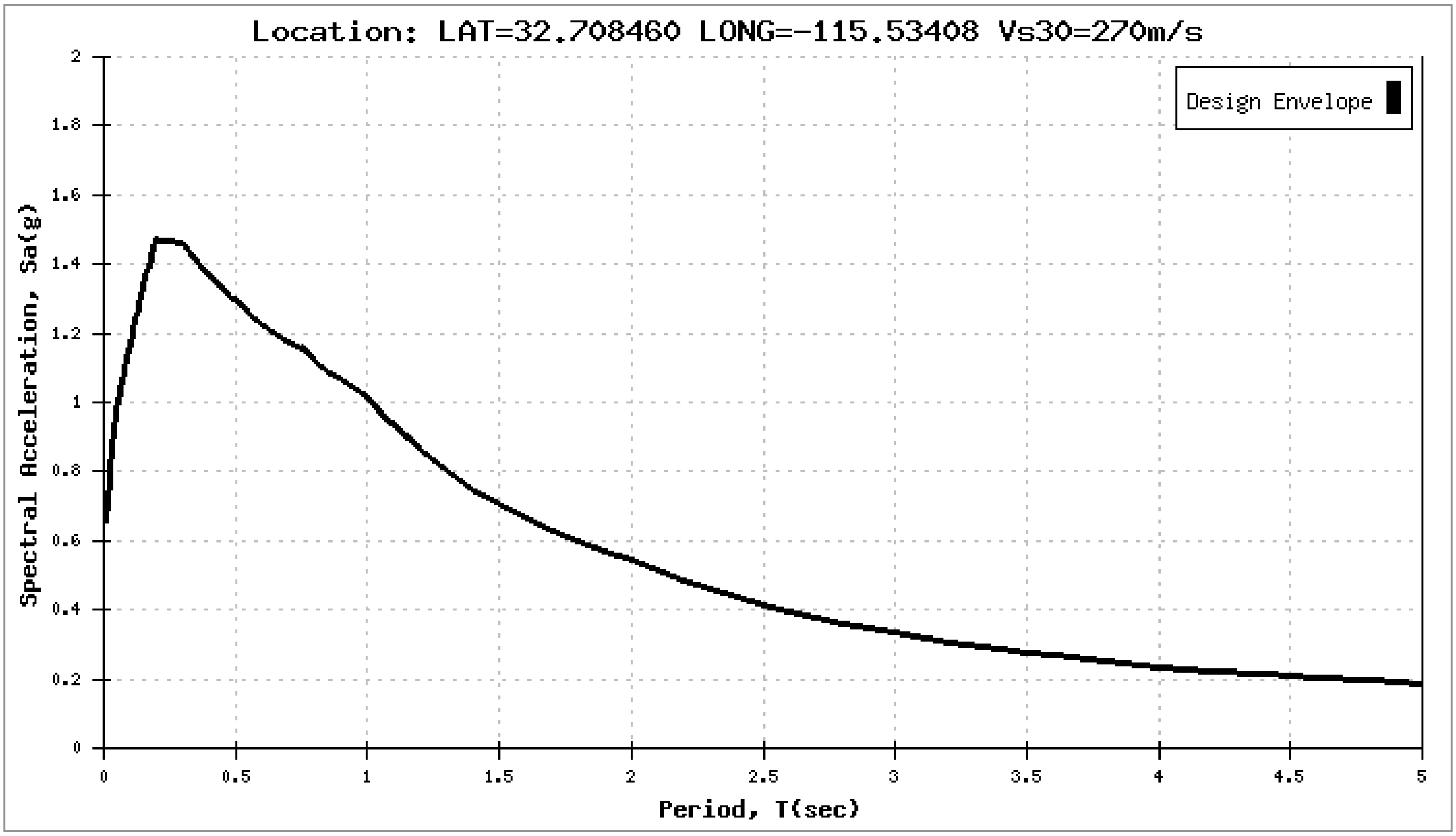
Date: **July 2016**

Regional Fault Map
Dogwood Road Bridge Replacement
San Diego County, California

Figure No. 4



Reference: Caltrans ARS Online (v2.3.07)

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An NV5 West, Inc. Company – Offices Nationwide
10592 Avenue of Science, Suite 200
San DIego, CA
Tel: (858) 715-5800, Fax: (858) 715-5810

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Seismic Design Envelope
Dogwood Rd Bridge Replacement
Imperial County, California

Figure No. 5

Appendix A

Exploratory Boring Logs

Logs of Exploratory Borings

Bulk and relatively undisturbed drive samples were obtained in the field during our subsurface evaluation. The samples were tagged in the field and transported to our laboratory for observation and testing. The drive samples were obtained using the Standard Penetration Test (SPT) samplers as described below.

California Modified Split Spoon Sampler

The split barrel drive sampler is driven with a 140-pound hammer allowed to drop freely 30 inches in general accordance with ASTM D1587. The number of blows per foot recorded during sampling is presented in the logs of exploratory borings. The sampler has external and internal diameters of approximately 3.0 and 2.4 inches, respectively, and the inside of the sampler is lined with 1-inch-long brass rings. The relatively undisturbed soil sample within the rings is removed, sealed, and transported to the laboratory for observation and testing.

Standard Penetration Test (SPT) Sampler

The split barrel sampler is driven with a 140-pound hammer allowed to drop freely 30 inches in general accordance with ASTM D1586. The number of blows per foot recorded during sampling is presented in the logs of exploratory borings. The sampler has external and internal diameters of 2.0 and 1.5 inches, respectively. The soil sample obtained in the interior of the barrel is measured, removed, sealed and transported to the laboratory for observation and testing.

LOG SYMBOLS:

■ Bulk/Bag sample

▮ California sampler
(2-1/2 inch outside diameter)

⧗ Modified California sampler
(3 inch outside diameter)

☒ Standard penetration
Split spoon sampler
(2 inch outside diameter)

‖ NX size core barrel
(2-5/8 inch outside diameter)

⦀ Shelby tube

▼ Water level
(level after completion)

▽ Water level
(level where first encountered)

Abbreviations:

SA - Sieve Analysis
P200 - Percent passing #200 sieve
AL - Atterberg Limits
LL - Liquid limit
DS - Direct shear test
'R' - R-value test
CS - Corrosivity test
EI - UBC expansion index
MD - Laboratory compaction test
CN - Consolidation test

General Notes:

1. Lines separating strata on the logs represent approximate boundaries only. Actual transitions may be gradual.
2. No warranty is provided as to the continuity of soil conditions between individual sample locations.
3. Logs represent general soil conditions observed at the point of exploration on the date indicated.
4. In general, unified soil classification designations presented on the logs were evaluated by visual methods only. Therefore, actual designations (based on laboratory tests) may vary.

Consistency criteria based on field tests

Relative density	SPT* (# blows/ft)	Relative density (%)
Very Loose	<4	0 - 15
Loose	4 - 10	15 - 35
Medium Dense	10 - 30	35 - 65
Dense	30 - 50	65 - 85
Very dense	>50	85 - 100

		Torvane	Pocket** penetrometer
Consistency	SPT (# blows/ft)	Undrained shear strength (tsf)	Unconfined compressive strength
Very soft	<2	<0.13	<0.25
Soft	2 - 4	0.13 - 0.25	0.25 - 0.5
Medium stiff	4 - 8	0.25 - 0.5	0.5 - 1.0
Stiff	8 - 15	0.5 - 1.0	1.0 - 2.0
Very stiff	15 - 30	1.0 - 2.0	2.0 - 4.0
Hard	>30	>2.0	>4.0

* Number of blows of 140 pounds hammer falling 30 inches to drive a 2 inch C.D. (1 3/8" I.D.) split barrel samler (ASTM - 1386 standard penetration test)

** Unconfined compressive strength in Tons/ft^2. Read from pocket penetrometer

Moisture content

Description	Field test
Dry	Absence of moisture, dusty, dry to the touch
Moist	Damp but no visible water
Wet	Visible free water, usually soil is below water table

Cementation

Description	Field test
Weakly	Crumbles or breaks with handling or slight finger pressure
Moderately	Crumbles or breaks with considerable finger pressure
Strongly	Will not crumble or break with finger pressure

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NV5 West, Inc.
15092 Avenue of Science, Suire 200,
San Diego, CA 92128
Tel: (858) 715-5800, Fax: (858) 715-5810

Project No: **226816-00103.02**
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Date: **July 2016**

Title: **Log Legend**
Project: **Dogwood Road Bridge Replacement**
Imperial County, California

Chart 1

Soil Classification Chart

Major Divisions			Symbols		Typical Descriptions
			Graph	Letter	
Coarse Grained Soils More than 50& of material is larger than No. 200 sieve size	Gravel and Gravelly soils more than 50% of coarse fraction retained on No. 4 sieve	Clean Gravels (Little or no fines)	●	GW	Well-Graded Gravel, Gravel SAND mixtures, little of no fines
			○	GP	Poorly-Graded Gravels, Gravel - SAND mixtures, little or no fines
		Gravels with fines (Appreciable amount of fines)	○	GM	Silty Gravels, Gravel- SAND- Silt mixture
			▨	GC	Clayey Gravels, Gravel - SAND - Clay mixtures
	Sand and Sandy Soils More than 50% of coarse fraction passing on No.4 sieve	Clean SANDS (Little or no fines)	▒	SW	Well-Graded SANDS, Gravelly, SANDS, little or no fines
			░	SP	Poorly - Graded SANDS, Gravelly SAND, little or no fines
		Sands with Fines (Appreciable amount of fines)	░	SM	Silty SANDS, SAND-Silt mixtures
			▨	SC	Clayey SANDS, SAND - Clay mixtures
Fine grained soils More than 50% of material is smaller than No. 200 sieve size	Silts and Clays	Liquid Limit less than 50	▥	ML	Inorganic Silts and 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, Lean Clays
			☰	OL	Organic Silts and organic Silty Clays of low Plasticity
	Silts and Clays	Liquid Limit Greater than 50	▥	MH	Inorganic Silts, micaceous or diatomaceous fine SAND or Silty Soils
			▨	CH	Inorganic Clays of high Plasticity
			≋	OH	Organic Clays of medium to High Plasticity, organic Silts
Highly organic soils			⁂	PT	Peat, Humus, swamp soils with High organic contents

NOTE: Dual symbols are used to indicate borderline soil classifications.

NV5 West, Inc.
15092 Avenue of Science, Suire 200,
San Diego, CA 92128
Tel: (858) 715-5800, Fax: (858) 715-5810

Project No: **226816-00103.02**
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Date: **July 2016**

Title: **Log Legend**
Project: **Dogwood Road Bridge Replacement**
Imperial County, California

Chart No. 2

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Project: **Dogwood Rd Bridge Replacement**
Project Location: Imperial County, California
Project Number: 226816-00103.02

Boring B-1
Sheet 1 of 1

Date(s) Drilled	March 24, 2016	Logged By	SR	Checked By	SK
Drilling Method	Hollow Stem Auger	Boring Diameter	6-inch	Approximate Surface Elevation	996 feet
Drilling Contractor	Pacific Drilling	Sampling Method	California Split Spoon and Standard Penetration Test	Hammer Data	140 pound, auto chain, 30" drop
Drill Rig Type:	Hollow Stem Auger	Location:	See Geotechnical Map, Figure 2	Lat. - Long.:	32.7130 - -115.5343

Depth (ft)	Sample Type	Blows / 6 in. (N)	Sample ID	USCS Class.	**MATERIAL DESCRIPTION** This log is an integral part of the accompanying report and must be used together with the report for relevant interpretation. The descriptions contained hereon apply only at this boring location and at the time of excavation. Subsurface data are a simplified summary of actual conditions encountered and may vary at other locations and with the passage of time.	Moisture Content %	Dry Weight (pcf)	Other Tests and Remarks
0				CL	silty CLAY-firm to stiff-reddish brown-moist			
5	▨	5 7 6	Cal 1	CL	silty CLAY-stiff-brown-moist-with stain	26.1	95.7	
10	☒	2 3 4	SPT 1	CH	silty CLAY-firm-brown-moist-high plasticity	24.1		
15	▨	5 8 12	Cal 2	CL	sandy CLAY-stiff-brown-moist-medium to high plasticity	25.8	99.8	
					Total depth : 16.5 Ground water not encountered No caving Boring backfilled on 3/24/2016			
20								
25								
30								

Sample Type

▨ Cal. Mod. ☒ SPT ▩ Bulk ◣ Other ● No Recovery

NV5

Project: **Dogwood Rd Bridge Replacement**
Project Location: Imperial County, California
Project Number: 226816-00103.02

Boring B-2

Sheet 1 of 3

Date(s) Drilled	March 25, 2016	Logged By	SR	Checked By	SK
Drilling Method	Hollow Stem Auger	Boring Diameter	6-inch	Approximate Surface Elevation	996 feet
Drilling Contractor	SG Drilling	Sampling Method	California Split Spoon and Standard Penetration Test	Hammer Data	140 pound, auto chain, 30" drop
Drill Rig Type:	Hollow Stem Auger	Location: See Geotechnical Map, Figure 2		Lat. - Long.: 32.7089 - -115.5342	

MATERIAL DESCRIPTION

This log is an integral part of the accompanying report and must be used together with the report for relevant interpretation. The descriptions contained hereon apply only at this boring location and at the time of excavation. Subsurface data are a simplified summary of actual conditions encountered and may vary at other locations and with the passage of time.

Depth (ft)	Sample Type	Blows / 6 in. (N)	Sample ID	USCS Class.	MATERIAL DESCRIPTION	Moisture Content %	Dry Weight (pcf)	Other Tests and Remarks
0				CL-ML	0-2':SILT-CLAY-firm-moist-medium plasticity			
5	☒	2 2 4	SPT 1	CH	CLAY-soft-light brown-wet	29.7		▽ groundwater @ 5'
10	●	2 3 3						No recovery
15	☒	4 5 7	SPT 2	CH	CLAY-soft-dark brown-wet-high plasticity-with reddish orange mottling	28.9	99.3	
20	▨	4 5 6	Cal 2	SC CL	clayey SAND- loose to medium dense-brown-wet sandy CLAY-soft-dark brown-wet-with fine silty sand	25.5	100.1	
25	☒	2 2 3	SPT 3			29.3		
30					decreasing sand with depth			

Sample Type

▨ Cal. Mod. ☒ SPT ▩ Bulk ◪ Other ● No Recovery

Project:	**Dogwood Rd Bridge Replacement**
Project Location:	Imperial County, California
Project Number:	226816-00103.02

Boring B-2

Sheet 2 of 3

Date(s) Drilled	March 25, 2016	Logged By	SR	Checked By	SK
Drilling Method	Hollow Stem Auger	Boring Diameter	6-inch	Approximate Surface Elevation	996 feet
Drilling Contractor	Pacific Drilling	Sampling Method	California Split Spoon and Standard Penetration Test	Hammer Data	140 pound, auto chain, 30" drop
Drill Rig Type:	Hollow Stem Auger	Location: See Geotechnical Map, Figure 2		Lat. - Long.: 32.7089 - -115.5342	

MATERIAL DESCRIPTION

This log is an integral part of the accompanying report and must be used together with the report for relevant interpretation. The descriptions contained hereon apply only at this boring location and at the time of excavation. Subsurface data are a simplified summary of actual conditions encountered and may vary at other locations and with the passage of time.

Depth (ft)	Sample Type	Blows / 6 in. (N)	Sample ID	USCS Class.	MATERIAL DESCRIPTION	Moisture Content %	Dry Weight (pcf)	Other Tests and Remarks
30	▨	15 17 21	Cal 2	CH	Decreased sand @30' CLAY-stiff-brown-wet-high plasticity-orange mottling	27.9	97.0	
35	☒	6 7 7	SPT 4	SC	clayey SAND-loose to medium dense-brown-wet	23.4		
40	▨	9 11 20	Cal 4	SC SC	Increase in clay content from 40'-41' clayey SAND-dense-brown-wet-fine with black laminations	24.2	97.8	
45	☒	20 25 17		SC	clayey SAND-dense- brown-fine-with black laminations	23.8		
50	▨	8 5 10		SC	clayey SAND-medium dense- brown-fine-with black laminations	24.1	99.5	
55	☒	7 9 17		SC		23.7		
60					Increasing clay content, grading to clayey SAND (SC)			

Sample Type

▨ Cal. Mod. ☒ SPT ▩ Bulk ◣ Other ● No Recovery

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Project:	**Dogwood Rd Bridge Replacement**	**Boring**	**B-2**
Project Location:	Imperial County, California		
Project Number:	226816-00103.02	Sheet 3 of 3	

Date(s) Drilled	March 25, 2016	Logged By	SR	Checked By	SK
Drilling Method	Hollow Stem Auger	Boring Diameter	6-inch	Approximate Surface Elevation	996 feet
Drilling Contractor	Pacific Drilling	Sampling Method	California Split Spoon and Standard Penetration Test	Hammer Data	140 pound, auto chain, 30" drop
Drill Rig Type:	Hollow Stem Auger	Location: See Geotechnical Map, Figure 2		Lat. - Long.:	32.7089 - -115.5342

MATERIAL DESCRIPTION

This log is an integral part of the accompanying report and must be used together with the report for relevant interpretation. The descriptions contained hereon apply only at this boring location and at the time of excavation. Subsurface data are a simplified summary of actual conditions encountered and may vary at other locations and with the passage of time.

Depth (ft)	Sample Type	Blows / 6 in. (N)	Sample ID	USCS Class.	MATERIAL DESCRIPTION	Moisture Content %	Dry Weight (pcf)	Other Tests and Remarks
60	▨	6 11 17	Cal 6	SC	clayey SAND-medium dense-brown,wet-with black lamination- decreasing clay grades to sand	22.0	103.3	
65	☒	6 15 15		SC	clayey SAND-dense-brown-wet-limited recovery			
70	●	3 2 5			Increasing clay with depth	22.8		No recovery
					Total depth : 71.5' Ground water encountered at 5' No caving Boring backfilled with cuttings and bentonite chips on 3/25/2016			
75								
80								
85								
90								

Sample Type

▨ Cal. Mod. ☒ SPT ▩ Bulk ◩ Other ● No Recovery

NV5

Project: **Dogwood Rd Bridge Replacement**
Project Location: Imperial County, California
Project Number: 226816-00103.02

Boring B-3
Sheet 1 of 3

Date(s) Drilled	March 24, 2016	Logged By	SR	Checked By	SK
Drilling Method	Hollow Stem Auger	Boring Diameter	6-inch	Approximate Surface Elevation	996 feet
Drilling Contractor	Pacific Drilling	Sampling Method	California Split Spoon and Standard Penetration Test	Hammer Data	140 pound, auto chain, 30" drop
Drill Rig Type:	Hollow Stem Auger	Location:	See Geotechnical Map, Figure 2	Lat. - Long.:	32.7086 - -115.5342

MATERIAL DESCRIPTION

This log is an integral part of the accompanying report and must be used together with the report for relevant interpretation. The descriptions contained hereon apply only at this boring location and at the time of excavation. Subsurface data are a simplified summary of actual conditions encountered and may vary at other locations and with the passage of time.

Depth (ft)	Sample Type	Blows / 6 in. (N)	Sample ID	USCS Class.	MATERIAL DESCRIPTION	Moisture Content %	Dry Weight (pcf)	Other Tests and Remarks
0				CL	silty CLAY-firm-tan-reddish brown-moist-medium plasticity			
5	▨	4 5 9	Cal 1	CH	CLAY-soft to firm-dark brown-moist-medium to high plasticity	26.4	94.5	
10	☒	2 2 3	SPT 1	SC	clayey SAND-loose-brown-wet-fine	25.9		▼ groundwater @ 10'
15	▨	4 5 8	Cal 2	CH	CLAY-stiff-dark brown-wet-high plasticity-with reddish orange molting	30.7	92.8	
20	☒	2 2 3	SPT 2	SC CH	clayey SAND to sandy CLAY- loose-brown-wet-micaceous CLAY-soft-dark brown-wet-reddish brown molting	24.3		
25	▨	4 4 4	Cal 3	SC CH SC	25'-26':increasing sand increasing clay below 26' clayey SAND-loose-brown-wet-fine	25	97.6	
30								

Sample Type

▨ Cal. Mod. ☒ SPT ▩ Bulk ◣ Other ● No Recovery

NV5

Project:	**Dogwood Rd Bridge Replacement**	**Boring**	**B-3**
Project Location:	Imperial County, California		
Project Number:	226816-00103.02	Sheet	2 of 3

Date(s) Drilled	March 24, 2016	Logged By	SR	Checked By	SK
Drilling Method	Hollow Stem Auger	Boring Diameter	6-inch	Approximate Surface Elevation	996 feet
Drilling Contractor	Pacific Drilling	Sampling Method	California Split Spoon and Standard Penetration Test	Hammer Data	140 pound, auto chain, 30" drop
Drill Rig Type:	Hollow Stem Auger	Location:	See Geotechnical Map, Figure 2	Lat. - Long.:	32.7086 - -115.5342

MATERIAL DESCRIPTION

This log is an integral part of the accompanying report and must be used together with the report for relevant interpretation. The descriptions contained hereon apply only at this boring location and at the time of excavation. Subsurface data are a simplified summary of actual conditions encountered and may vary at other locations and with the passage of time.

Depth (ft)	Sample Type	Blows / 6 in. (N)	Sample ID	USCS Class.	MATERIAL DESCRIPTION	Moisture Content %	Dry Weight (pcf)	Other Tests and Remarks
30	⊠	7 8 4	SPT 3	SC	clayey SAND-loose to medium dense-brown-wet	25.8		
								flowing sand-adding water
35	⊠	5 9 10	SPT 4	SC	clayey SAND medium dense-brown-wet-fine	24.3		
40	▨	5 7 12	Cal 4	SC		25.5	97.1	
					black laminated sand			
45	⊠	8 10 8	SPT 5	SC	increasing clay at 46' clayey SAND-medium dense- brown-fine-with black laminations	23.7		
50	●	1 3 4		SC				No recovery
55	⊠	2 3 5	SPT 6	SC	clayey SAND-loose- brown-fine-with black laminations	20.4		
60								

Sample Type

▨ Cal. Mod. ⊠ SPT ▩ Bulk ◣ Other ● No Recovery

N|V|5	Project:	**Dogwood Rd Bridge Replacement**	**Boring B-3**
	Project Location:	Imperial County, California	
	Project Number:	226816-00103.02	Sheet 3 of 3

Date(s) Drilled	March 24, 2016	Logged By	SR	Checked By	SK
Drilling Method	Hollow Stem Auger	Boring Diameter	6-inch	Approximate Surface Elevation	996 feet
Drilling Contractor	Pacific Drilling	Sampling Method	California Split Spoon and Standard Penetration Test	Hammer Data	140 pound, auto chain, 30" drop
Drill Rig Type:	Hollow Stem Auger	Location: See Geotechnical Map, Figure 2		Lat. - Long.: 32.7086 - -115.5342	

MATERIAL DESCRIPTION

This log is an integral part of the accompanying report and must be used together with the report for relevant interpretation. The descriptions contained hereon apply only at this boring location and at the time of excavation. Subsurface data are a simplified summary of actual conditions encountered and may vary at other locations and with the passage of time.

Depth (ft)	Sample Type	Blows / 6 in. (N)	Sample ID	USCS Class.	MATERIAL DESCRIPTION	Moisture Content %	Dry Weight (pcf)	Other Tests and Remarks
60	☒	3 3 5	SPT 7	SC	clayey SAND-loose-brown-wet-fine-black laminations	23.3		
65	▨	13 23 50/5"	Cal 5	SC	clayey SAND-loose-brown-wet-fine-black laminations	24.2	101.5	
70	☒	6 8 8	SPT 8	SC	Becomes clayey SAND (SC) to sandy CLAY (CL)	28		
					Total depth : 71.5' Ground water encountered at 10' No caving Boring backfilled with cuttings and bentonite chips on 3/24/2016			
75								
80								
85								
90								

Sample Type

▨ Cal. Mod. ☒ SPT ▩ Bulk ◩ Other ● No Recovery

Project: **Dogwood Rd Bridge Replacement**
Project Location: Imperial County, California
Project Number: 226816-00103.02

Boring B-4
Sheet 1 of 1

Date(s) Drilled	March 24, 2016	Logged By	SR	Checked By	SK
Drilling Method	Hollow Stem Auger	Boring Diameter	6-inch	Approximate Surface Elevation	994 feet
Drilling Contractor	Pacific Drilling	Sampling Method	California Split Spoon and Standard Penetration Test	Hammer Data	140 pound, auto chain, 30" drop
Drill Rig Type:	Hollow Stem Auger	Location: See Geotechnical Map, Figure 2		Lat. - Long.: 32.7130 - -115.5343	

MATERIAL DESCRIPTION

This log is an integral part of the accompanying report and must be used together with the report for relevant interpretation. The descriptions contained hereon apply only at this boring location and at the time of excavation. Subsurface data are a simplified summary of actual conditions encountered and may vary at other locations and with the passage of time.

Depth (ft)	Sample Type	Blows / 6 in. (N)	Sample ID	USCS Class.	MATERIAL DESCRIPTION	Moisture Content %	Dry Weight (pcf)	Other Tests and Remarks
0				CL	silty CLAY-firm-tan-reddish brown-moist-medium plasticity			
5	☒	2 3 5	SPT 1	CH	CLAY-soft-dark brown-moist-medium to high plasticity	23.9		
10	▨	5 9 10	Cal 1	CH	CLAY-stiff-dark brown-moist-high plasticity	27.4	94.6	
15	☒	3 4 4	SPT 2	SC	clayey SAND-loose-brown-wet-medium to high plasticity	20		
					Total depth : 16.5' Ground water not encountered No caving Boring backfilled with cuttings on 3/24/2016			
20								
25								
30								

Sample Type

▨ Cal. Mod. ☒ SPT ▩ Bulk ◣ Other ● No Recovery

Appendix B

Log of Test Boring Sheets

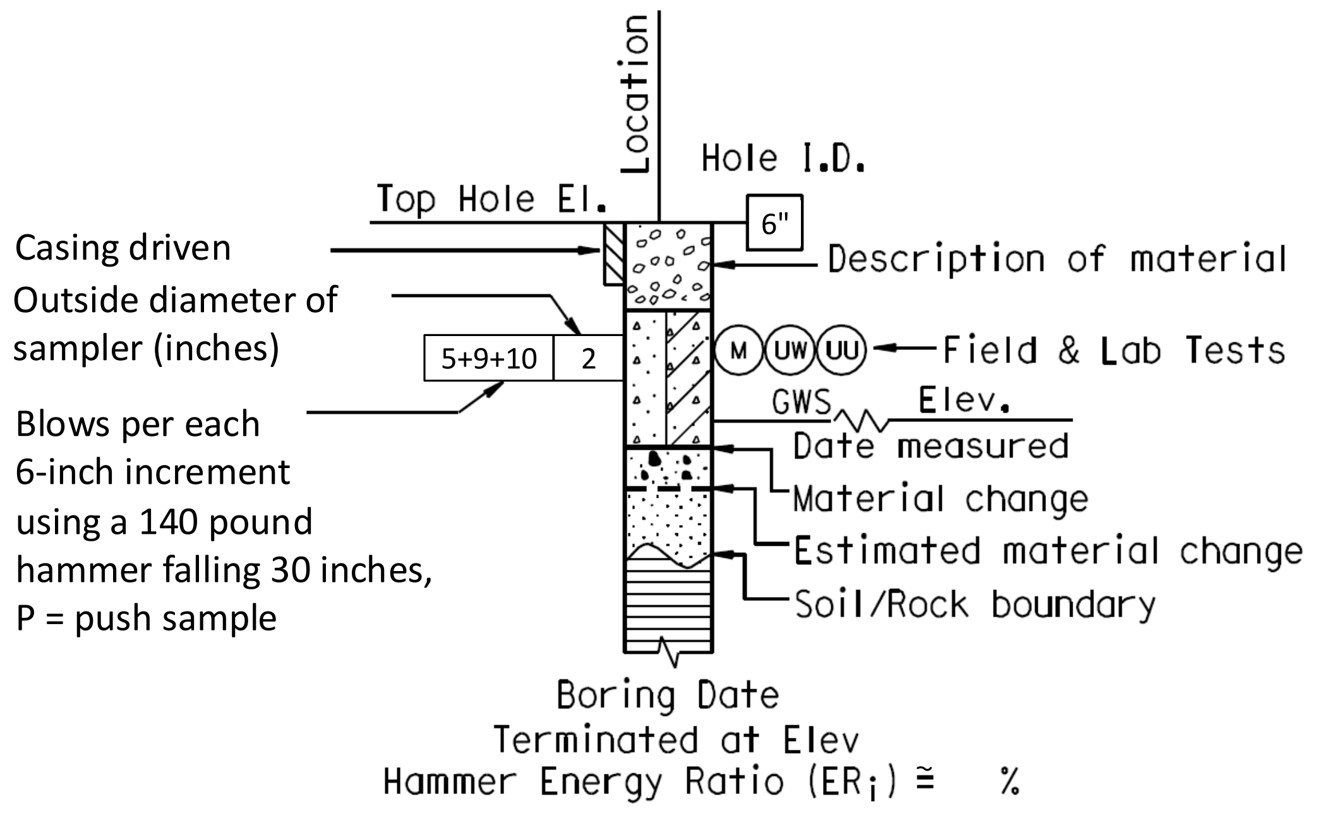
REFERENCE: CALTRANS SOIL & ROCK LOGGING, CLASSIFICATION, AND PRESENTATION MANUAL (2010)

CEMENTATION	
Description	Criteria
Weak	Crumbles or breaks with handling or little finger pressure.
Moderate	Crumbles or breaks with considerable finger pressure.
Strong	Will not crumble or break with finger pressure.

CONSISTENCY OF COHESIVE SOILS				
Description	Unconfined Compressive Strength (tsf)	Pocket Penetrometer Measurement (tsf)	Torvane Measurement (tsf)	Field Approximation
Very Soft	< 0.25	< 0.25	< 0.12	Easily penetrated several inches by fist
Soft	0.25 to 0.50	0.25 to 0.50	0.12 to 0.25	Easily penetrated several inches by thumb
Medium Stiff	0.50 to 1.0	0.50 to 1.0	0.25 to 0.50	Penetrated several inches by thumb with moderate effort
Stiff	1 to 2	1 to 2	0.50 to 1.0	Readily indented by thumb but penetrated only with great effort
Very Stiff	2 to 4	2 to 4	1.0 to 2.0	Readily indented by thumbnail
Hard	> 4.0	> 4.0	> 2.0	Indented by thumbnail with difficulty

BOREHOLE IDENTIFICATION		
Symbol	Hole Type	Description
Size	A	Auger Boring
Size	R P	Rotary drilled boring Rotary percussion boring (air)
Size	R	Rotary drilled diamond core
Size	HD HA	Hand driven (1-inch soil tube) Hand Auger
●	D	Dynamic Cone Penetration Boring
▲	CPT	Cone Penetration Test (ASTM D 5778-95)
⬚	O	Other
Note: Size in inches.		

PLASTICITY OF FINE-GRAINED SOILS	
Description	Criteria
Nonplastic	A 1/8-inch thread cannot be rolled at any water content.
Low	The thread can barely be rolled and the lump cannot be formed when drier than the plastic limit.
Medium	The thread is easy to roll and not much time is required to reach the plastic limit. The thread cannot be rerolled after reaching the plastic limit. The lump crumbles when drier than the plastic limit.
High	It takes considerable time rolling and kneading to reach the plastic limit. The thread can be rerolled several times after reaching the plastic limit. The lump can be formed without crumbling when drier than the plastic limit.



HOLLOW STEM AUGER

REVISION	DATE	COMMENTS

No. GE 2986
Exp. 09/30/16

PREPARED UNDER THE DIRECT SUPERVISION OF:
2986
G.E. No.
9/27/16
DATE
6/30/17
REG. EXP.

No. 61599
Exp. 6-30-17

COUNTY OF IMPERIAL PUBLIC WORKS DEPARTMENT
APPROVED FOR CONSTRUCTION BY:
WILLIAM S. BRUNET, P.E.
ROAD COMMISSIONER
61599
R.C.E. No.
DATE
6/30/17
REG. EXP.

PUBLIC WORKS DEPARTMENT
COUNTY OF IMPERIAL
EL CENTRO, CALIFORNIA

DATE 5/6/2016
DRAWN GC
SCALE AS SHOWN
CHECKED GG

LOG OF TEST BORINGS
DOGWOOD ROAD BRIDGE REPLACEMENT
OVER CENTRAL MAIN CANAL
BRIDGE NO. 58C-0042

SOIL LEGEND
S-11
SHEET 19 OF 21

REFERENCE: CALTRANS SOIL & ROCK LOGGING, CLASSIFICATION, AND PRESENTATION MANUAL (2010)

GROUP SYMBOLS AND NAMES

Graphic/Symbol	Group Names
GW	Well-graded GRAVEL Well-graded GRAVEL with SAND
GP	Poorly graded GRAVEL Poorly graded GRAVEL with SAND
GW-GM	Well-graded GRAVEL with SILT Well-graded GRAVEL with SILT and SAND
GW-GC	Well-graded GRAVEL with CLAY (or SILTY CLAY) Well-graded GRAVEL with CLAY and SAND (or SILTY CLAY and SAND)
GP-GM	Poorly graded GRAVEL with SILT Poorly graded GRAVEL with SILT and SAND
GP-GC	Poorly graded GRAVEL with CLAY (or SILTY CLAY) Poorly graded GRAVEL with CLAY and SAND (or SILTY CLAY and SAND)
GM	SILTY GRAVEL SILTY GRAVEL with SAND
GC	CLAYEY GRAVEL CLAYEY GRAVEL with SAND
GC-GM	SILTY, CLAYEY GRAVEL SILTY, CLAYEY GRAVEL with SAND
SW	Well-graded SAND Well-graded SAND with GRAVEL
SP	Poorly graded SAND Poorly graded SAND with GRAVEL
SW-SM	Well-graded SAND with SILT Well-graded SAND with SILT and GRAVEL
SW-SC	Well-graded SAND with CLAY (or SILTY CLAY) Well-graded SAND with CLAY and GRAVEL (or SILTY CLAY and GRAVEL)
SP-SM	Poorly graded SAND with SILT Poorly graded SAND with SILT and GRAVEL
SP-SC	Poorly graded SAND with CLAY (or SILTY CLAY) Poorly graded SAND with CLAY and GRAVEL (or SILTY CLAY and GRAVEL)
SM	SILTY SAND SILTY SAND with GRAVEL
SC	CLAYEY SAND CLAYEY SAND with GRAVEL
SC-SM	SILTY, CLAYEY SAND SILTY, CLAYEY SAND with GRAVEL
PT	PEAT
	COBBLES COBBLES and BOULDERS BOULDERS

Graphic/Symbol	Group Names
CL	Lean CLAY Lean CLAY with SAND Lean CLAY with GRAVEL SANDY lean CLAY SANDY lean CLAY with GRAVEL GRAVELLY lean CLAY GRAVELLY lean CLAY with SAND
CL-ML	SILTY CLAY SILTY CLAY with SAND SILTY CLAY with GRAVEL SANDY SILTY CLAY SANDY SILTY CLAY with GRAVEL GRAVELLY SILTY CLAY GRAVELLY SILTY CLAY with SAND
ML	SILT SILT with SAND SILT with GRAVEL SANDY SILT SANDY SILT with GRAVEL GRAVELLY SILT GRAVELLY SILT with SAND
OL	ORGANIC lean CLAY ORGANIC lean CLAY with SAND ORGANIC lean CLAY with GRAVEL SANDY ORGANIC lean CLAY SANDY ORGANIC lean CLAY with GRAVEL GRAVELLY ORGANIC lean CLAY GRAVELLY ORGANIC lean CLAY with SAND
OL	ORGANIC SILT ORGANIC SILT with SAND ORGANIC SILT with GRAVEL SANDY ORGANIC SILT SANDY ORGANIC SILT with GRAVEL GRAVELLY ORGANIC SILT GRAVELLY ORGANIC SILT with SAND
CH	Fat CLAY Fat CLAY with SAND Fat CLAY with GRAVEL SANDY fat CLAY SANDY fat CLAY with GRAVEL GRAVELLY fat CLAY GRAVELLY fat CLAY with SAND
MH	Elastic SILT Elastic SILT with SAND Elastic SILT with GRAVEL SANDY elastic SILT SANDY elastic SILT with GRAVEL GRAVELLY elastic SILT GRAVELLY elastic SILT with SAND
OH	ORGANIC fat CLAY ORGANIC fat CLAY with SAND ORGANIC fat CLAY with GRAVEL SANDY ORGANIC fat CLAY SANDY ORGANIC fat CLAY with GRAVEL GRAVELLY ORGANIC fat CLAY GRAVELLY ORGANIC fat CLAY with SAND
OH	ORGANIC elastic SILT ORGANIC elastic SILT with SAND ORGANIC elastic SILT with GRAVEL SANDY ORGANIC elastic SILT SANDY ORGANIC elastic SILT with GRAVEL GRAVELLY ORGANIC elastic SILT GRAVELLY ORGANIC elastic SILT with SAND
OL/OH	ORGANIC SOIL ORGANIC SOIL with SAND ORGANIC SOIL with GRAVEL SANDY ORGANIC SOIL SANDY ORGANIC SOIL with GRAVEL GRAVELLY ORGANIC SOIL GRAVELLY ORGANIC SOIL with SAND

FIELD AND LABORATORY TESTING

(C)	Consolidation (ASTM D 2435)
(CL)	Collapse Potential (ASTM D 5333)
(CP)	Compaction Curve (CTM 216)
(CR)	Corrosivity Testing (CTM 643, CTM 422, CTM 417)
(CU)	Consolidated Undrained Triaxial (ASTM D 4767)
(DS)	Direct Shear (ASTM D 3080)
(EI)	Expansion Index (ASTM D 4829)
(M)	Moisture Content (ASTM D 2216)
(OC)	Organic Content-% (ASTM D 2974)
(P)	Permeability (CTM 220)
(PA)	Particle Size Analysis (ASTM D 422)
(PI)	Plasticity Index (AASHTO T 90) Liquid Limit (AASHTO T 89)
(PL)	Point Load Index (ASTM D 5731)
(PM)	Pressure Meter
(PP)	Pocket Penetrometer
(R)	R-Value (CTM 301)
(SE)	Sand Equivalent (CTM 217)
(SG)	Specific Gravity (AASHTO T 100)
(SL)	Shrinkage Limit (ASTM D 427)
(SW)	Swell Potential (ASTM D 4546)
(TV)	Pocket Torvane
(UC)	Unconfined Compression-Soil (ASTM D 2166) Unconfined Compression-Rock (ASTM D 2938)
(UU)	Unconsolidated Undrained Triaxial (ASTM D 2850)
(UW)	Unit Weight (ASTM D 4767)
(VS)	Vane Shear (AASHTO T 223)

APPARENT DENSITY OF COHESIONLESS SOILS

Description	SPT N_{60} (Blows / 12 inches)
Very loose	0 - 4
Loose	5 - 10
Medium Dense	11 - 30
Dense	31 - 50
Very Dense	> 50

MOISTURE

Description	Criteria
Dry	Absence of moisture, dusty, dry to the touch
Moist	Damp but no visible water
Wet	Visible free water, usually soil is below water table

PERCENT OR PROPORTION OF SOILS

Description	Criteria
Trace	Particles are present but estimated to be less than 5%
Few	5 to 10%
Little	15 to 25%
Some	30 to 45%
Mostly	50 to 100%

PARTICLE SIZE

Description		Size
Boulder		> 12"
Cobble		3" to 12"
Gravel	Coarse	3/4" to 3"
	Fine	No. 4 to 3/4"
Sand	Coarse	No. 10 to No. 4
	Medium	No. 40 to No. 10
	Fine	No. 200 to No. 40

REVISION	DATE	COMMENTS

No. GE 2986
Exp. 09/30/16

PREPARED UNDER THE DIRECT SUPERVISION OF:

2986	G.E. No.
9/27/16	DATE
6/30/17	REG. EXP.

WILLIAM S. BRUNET
No. 61599
Exp. 6-30-17

COUNTY OF IMPERIAL PUBLIC WORKS DEPARTMENT
APPROVED FOR CONSTRUCTION BY:

WILLIAM S. BRUNET, P.E.
ROAD COMMISSIONER

61599	R.C.E. No.
DATE	
6/30/17	REG. EXP.

PUBLIC WORKS DEPARTMENT
COUNTY OF IMPERIAL
EL CENTRO, CALIFORNIA

DATE 5/6/2016
DRAWN GC
SCALE AS SHOWN
CHECKED GG

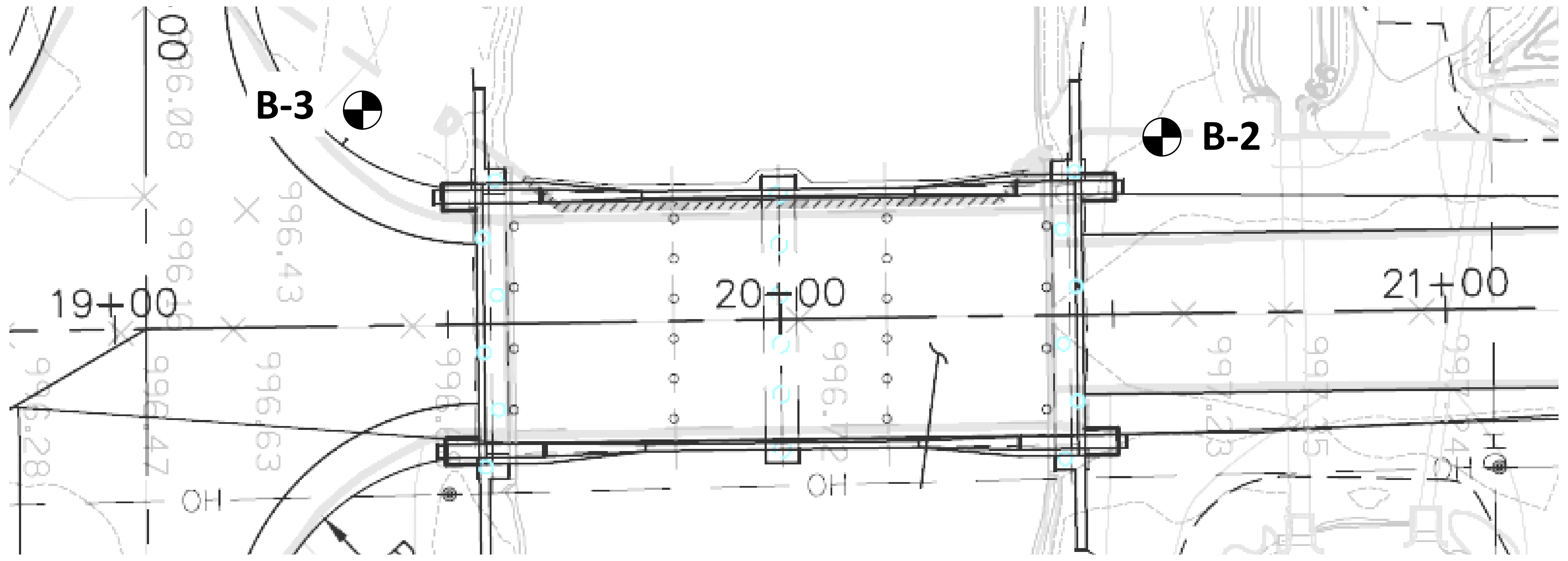
LOG OF TEST BORINGS

DOGWOOD ROAD BRIDGE REPLACEMENT
OVER CENTRAL MAIN CANAL
BRIDGE NO. 58C-0042

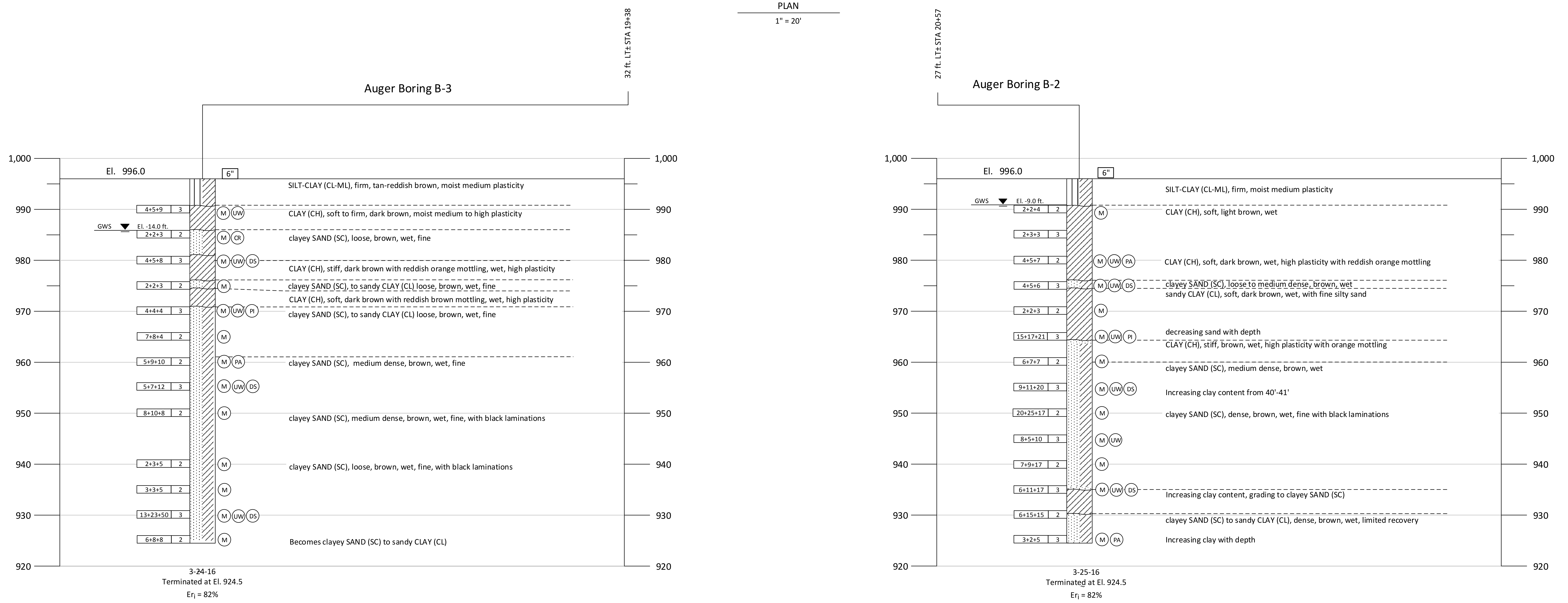
SOIL LEGEND

S-12

REFERENCE: **CALTRANS SOIL & ROCK LOGGING, CLASSIFICATION, AND PRESENTATION MANUAL (2010)**



PLAN
1" = 20'



PROFILE
Horizontal 1" = 1'
Vertical 1" = 10'

REVISION	DATE	COMMENTS

GUILLAUME GAU
No. GE 2986
Exp. 9/30/18

PREPARED UNDER THE DIRECT SUPERVISION OF:
2986
G.E. No.
9/27/16
DATE
9/30/18
REG. EXP.

JOHN A. GAY
No. 62028
Exp. 9-30-17

COUNTY OF IMPERIAL PUBLIC WORKS DEPARTMENT
APPROVED FOR CONSTRUCTION BY:
JOHN A. GAY, P.E.
ROAD COMMISSIONER
62028
R.C.E. No.
DATE
9/30/17
REG. EXP.

PUBLIC WORKS DEPARTMENT
COUNTY OF IMPERIAL
EL CENTRO, CALIFORNIA

DATE 5/6/2016
DRAWN GC
SCALE AS SHOWN
CHECKED GG

LOG OF TEST BORINGS-3

DOGWOOD ROAD BRIDGE REPLACEMENT
OVER CENTRAL MAIN CANAL
BRIDGE NO. 58C-0042

LOG OF TEST BORINGS

S-13

SHEET 21 OF 21

Appendix C

Laboratory Test Results

SUMMARY OF LABORATORY TEST RESULTS

In-situ Moisture and Density Tests

The in-situ moisture contents and dry densities of selected samples obtained from the test borings were evaluated in general accordance with the latest version of D-2216 and D2937 laboratory test methods. The method involves obtaining the moist weight of the sample and then drying the sample to obtain is dry weight. The moisture content is calculated by taking the difference between the wet and dry weights, dividing it by the dry weight of the sample and expressing the result as a percentage. The results of the in-situ moisture content and density tests are presented in the following table and on the logs of exploratory borings in Appendix A.

RESULTS OF MOISTURE CONTENT AND DENSITY TESTS
(ASTM D2216)

Sample Location	Moisture Content (percent)	Dry Density (pounds per cubic foot)
Boring B-1 @ 6-6.5 feet	26.1	95.7
Boring B-1 @ 10-11.5 feet	24.1	-
Boring B-1 @ 15-16.5 feet	25.8	99.8
Boring B-2 @ 5-6.5 feet	29.7	-
Boring B-2 @ 15-16.5 feet	28.9	99.3
Boring B-2 @ 25-26.5 feet	29.3	-
Boring B-2 @ 31-31.5 feet	27.9	97.0
Boring B-2 @ 35-36.5 feet	23.4	-
Boring B-2 @ 45-46.5 feet	23.8	-
Boring B-2 @ 51-51.5 feet	24.1	99.5
Boring B-2 @ 55-56.5 feet	23.7	-
Boring B-2 @ 65-66.5 feet	22.0	-
Boring B-2 @ 70-71.5 feet	22.8	-
Boring B-3 @ 6-6.5 feet	26.4	94.5
Boring B-3 @ 10-11.5 feet	25.9	-
Boring B-3 @ 20-21.5 feet	24.3	-
Boring B-3 @ 26-26.5 feet	25.0	97.6
Boring B-3 @ 30-31.5 feet	25.8	-
Boring B-3 @ 35-36.5 feet	24.3	-
Boring B-3 @ 45-46.5 feet	23.7	-
Boring B-3 @ 55-56.5 feet	20.4	-

Sample Location	Moisture Content (percent)	Dry Density (pounds per cubic foot)
Boring B-3 @ 60-61.5 feet	23.3	-
Boring B-3 @ 70-71.5 feet	28.0	-
Boring B-4 @ 5-6.5 feet	23.9	-
Boring B-4 @ 11-11.5 feet	27.4	94.6
Boring B-4 @ 15-16.5 feet	20.0	-

Classification

Soils were visually and texturally classified in accordance with the Unified Soil Classification System. Soil classifications are indicated on the logs of the exploratory borings in Appendix A.

Particle-size Distribution Tests

An evaluation of the grain-size distribution of selected soil samples was performed in general accordance with the latest version of ASTM D-422 (including –200 wash). These test results were utilized in evaluating the soil classifications in accordance with the Unified Soil Classification System. Particle size distribution test results are presented on the laboratory test sheets attached in this appendix.

Atterberg Limits

Atterberg limits test was performed in accordance with ASTM D4318. This test was useful in classification of the soil. Test results are attached in this appendix.

Expansion Index

Expansion index test was performed in accordance with ASTM D4829. This test was useful in evaluating the potential expansion of the soil. Test results are attached in this appendix.

Maximum Density

Maximum density test was performed in accordance with ASTM D1557. This test was useful in evaluating the compaction of the soil in the field. Test results are attached in this appendix.

R Value

R Value test was performed in accordance with ASTM D2844. This test was useful in evaluating the response of the compacted soil. Test results are attached in this appendix.

Soil Corrosivity Tests

Soluble sulfate, chloride, resistively and pH tests were performed in accordance with California Test Methods 643, 417 and 422 to assess the degree of corrosivity of the subgrade soils with regard to concrete and normal grade steel. The results of the test are presented in the following table and attached in this appendix.

RESULTS OF CORROSIVITY TESTS
(CTM 417, CTM 422)

Location	pH	Resistivity (ohm-cm)	Sulfate (ppm)	Chloride (ppm)
B3, 10-11.5	8.7	900	170	32

Direct shear

A direct shear test was performed on a representative undisturbed sample in accordance with ASTM D3080 to evaluate the shear strength characteristics of the on-site materials. The test method consists of placing the soil sample in the direct shear device, applying a series of normal stresses, and then shearing the sample at the constant rate of shearing deformation. The shearing force and horizontal displacements are measured and recorded as the soil specimen is sheared. The shearing is continued well beyond the point of maximum stress until the stress reaches a constant or residual value. The results of the tests are presented in the following table and attached in this appendix.

RESULTS OF DIRECT SHEAR TEST
(ASTM D3080)

Location	Peak Angle of Internal Friction (degrees)	Peak Cohesion Intercept (psf)	Notes
B2, 21-21.5	24	650	-
B2, 41-41.5	33	121	-
B2, 61-61.5	33	347	-
B3, 16-16.5	22	385	-

Location	Peak Angle of Internal Friction (degrees)	Peak Cohesion Intercept (psf)	Notes
B3,41-41.5	33	120	-
B3,66-66.5	35	61	-

NV5

Moisture and Density Test Report

(ASTM 2216,2937)

Date: May 05, 2016
Client: County of Imperial
Address: 1002 State Street
El Centro, CA 92243
Project: Dogwood Road Bridge Replacement
Project Add: El Centro, CA

Job Number: 22816-00103 PH2 T2.2
Report Number: 4264
Lab Number: 112657-112675

Sampled By: Sean Roy
Date Rcvd: 3/28/16

Lab Number	112657	112658	112659	112677	112678
Exploration No.	B1-01	B1-SPT1	B1-02	B2-SPT1	B2-SPT2
Depth, feet	6-6.5	10-11.5	15-16.5	5-6.5	15-16.5
Moisture Content, %	26.1	24.1	25.8	29.7	28.9
Dry Density, pcf.	95.7	- -	99.8	- -	99.3

Lab Number	112680	112681	112682	112684	112685
Exploration No.	B2-SPT3	B2-D2	B2-SPT4	B2-SPT5	B2-D4
Depth, feet	25-26.5	31-31.5	35-36.5	45-46.5	51-51.5
Moisture Content, %	29.3	27.9	23.4	23.8	24.1
Dry Density, pcf.	- -	97.0	- -	- -	99.5

Lab Number	112686	112688	112689	112660	112661
Exploration No.	B2-SPT6	B2-SPT7	B2-SPT8	B3-01	B3-STP1
Depth, feet	55-56.5	65-66.5	70-71.5	6-6.5	10-11.5
Moisture Content, %	23.7	22.0	22.8	26.4	25.9
Dry Density, pcf.	- -	- -	- -	94.5	- -

Lab Number	112663	112664	112665	11266	112668
Exploration No.	B3-SPT2	B3-D3	B3-SPT3	B3-SPT4	B3-SPT5
Depth, feet	20-21.5	26-26.5	30-31.5	35-36.5	45-46.5
Moisture Content, %	24.3	25.0	25.8	24.3	23.7
Dry Density, pcf.	- -	97.6	- -	- -	- -

Moisture and Density Test Report

(ASTM 2216,2937)

Job No: 22816-00103 PH2 T2.2
Job Name: Dogwood Road Bridge Replacement
Client: County of Imperial
Report No: 4264

Lab Number	112669	112670	112672	112673	112674
Exploration No.	B3-SPT6	B3-SPT7	B3-SPT8	B4-SPT1	B4-01
Depth, feet	55-56.5	60-61.5	70-71.5	5-6.5	11-11.5
Moisture Content, %	20.4	23.3	28.0	23.9	27.4
Dry Density, pcf.	- -	- -	- -	- -	94.6

Lab Number	112675
Exploration No.	B4-SPT2
Depth, feet	15-16.5
Moisture Content, %	20.0
Dry Density, pcf.	- -

Reviewed by: ______________________
Sam Koohi, PE
Engineering Manager

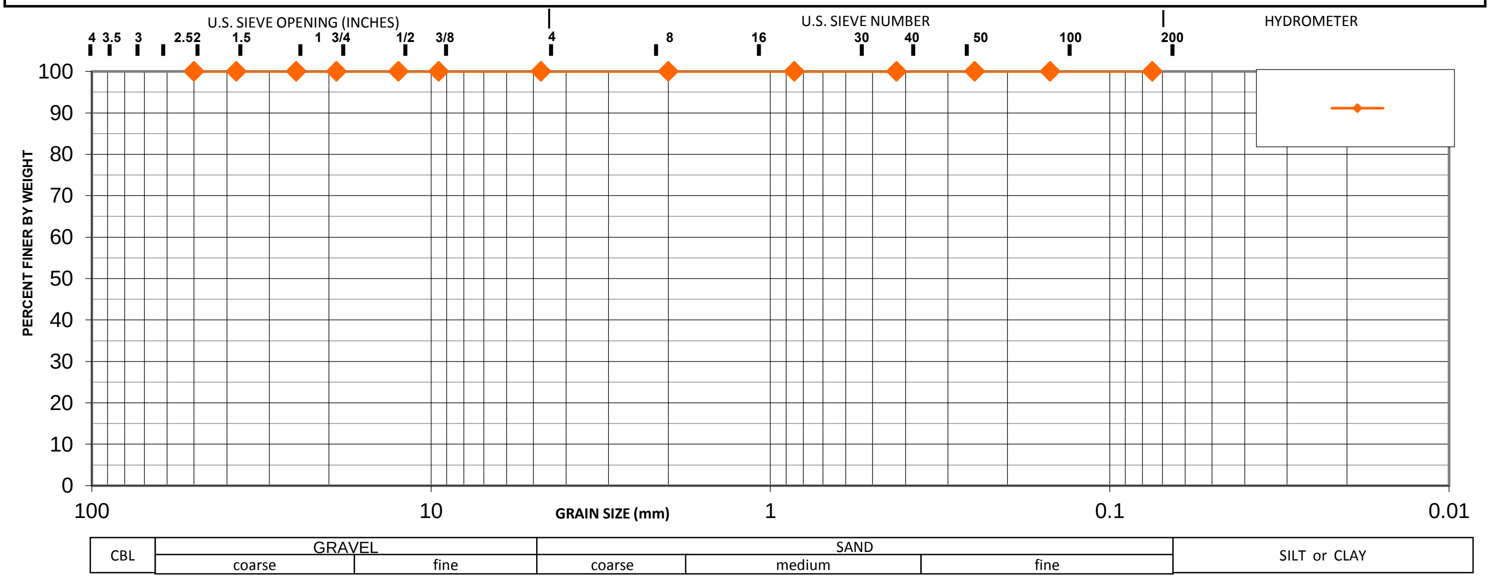
REPORT OF SIEVE ANALYSIS TEST

ASTM D422 - Soil

Date: May 5, 2016
Client: County of Imperial
Address: 1002 State Street
El Centro, CA 92243
Project : Dogwood Road Bridge Replacement
Project Address: El Centro, CA

Job Number: 226816-00103 PH2
Report Number: 4264
Lab Number: 112678

Material	CLAY (CH)				
Color	Brown				
Sample Location	B2-SPT2 @ 15-16.5 ft.				
Date Sampled	3/24/2016				
Sampled By	Sean Roy				
Date Tested	4/11/2016				
Tested By	Darrel Delgado				



Sample ID:	112678					
Sieve Size	% Passing					
63mm (2 1/2")						
50mm (2")						
37.5mm (1 1/2")	100					
25mm (1")	100					
19mm (3/4")	100					
12.5mm (1/2")	100					
9.5mm (3/8")	100					
4.75mm (#4)	100					
2mm (#10)	100					
850µm (#20)	100					
425µm (#40)	100					
250µm (#60)	100					
150 µm (#100)	100					
75 um (#200) washµ	100					
Fineness Modulus	0 0					
Shape (sand & gravel)	N.R.					
Hardness (sand & gravel)	N.R.					
Specific Gravity	2.65					
Coef. of Curvature (C_C)	0 5					
Coef. of Uniformity (C_U)	27.4					
% Gravel	0					
% Sand	0					
% Fines	100.0					
USCS Class:	**CH**					

Notes: *Hardness: H&D = Hard & Durable; W&F = Weathered & Friable*
N.R.: Not Recorded; N/A: Not Available.

Respectfully Submitted,
NV5 West, Inc.

Sam Koohi, PE
Engineering Manager

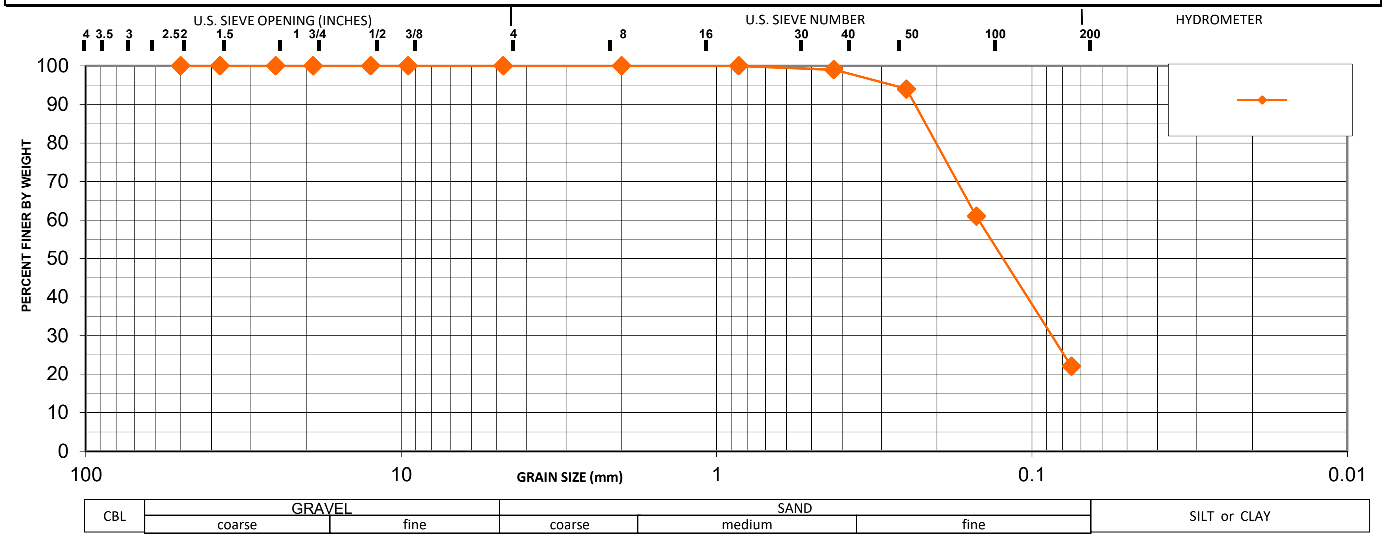
REPORT OF SIEVE ANALYSIS TEST

ASTM D422 - Soil

Date: May 5, 2016
Client: County of Imperial
Address: 1002 State Street
El Centro, CA 92243
Project : Dogwood Road Bridge Replacement
Project Address: El Centro, CA

Job Number: 226816-00103 PH2
Report Number: 4264
Lab Number: 112689

Material	clayey SAND (SC)					
Color	Brown					
Sample Location	B2-SPT8 @ 70-71.5 ft.					
Date Sampled	3/24/2016					
Sampled By	Sean Roy					
Date Tested	4/11/2016					
Tested By	Darrel Delgado					



Sample ID:	**112689**					
Sieve Size	% Passing					
63mm (2 1/2")						
50mm (2")						
37.5mm (1 1/2")	**100**					
25mm (1")	**100**					
19mm (3/4")	**100**					
12.5mm (1/2")	**100**					
9.5mm (3/8")	**100**					
4.75mm (#4)	**100**					
2mm (#10)	**100**					
850µm (#20)	**100**					
425µm (#40)	**99**					
250µm (#60)	**94**					
150 µm (#100)	**61**					
75 um (#200) washµ	**22**					
Fineness Modulus	1.2					
Shape (sand & gravel)	N.R.					
Hardness (sand & gravel)	N.R.					
Specific Gravity	2.65					
Coef. of Curvature (C_C)	25.0					
Coef. of Uniformity (C_U)	72.7					
% Gravel	0					
% Sand	78					
% Fines	22.0					
USCS Class:	**SC**					

Notes: *Hardness: H&D = Hard & Durable; W&F = Weathered & Friable*
N.R.: Not Recorded; N/A: Not Available.

Respectfully Submitted,
NV5 West, Inc.

Sam Koohi, PE
Engineering Manager

NV5

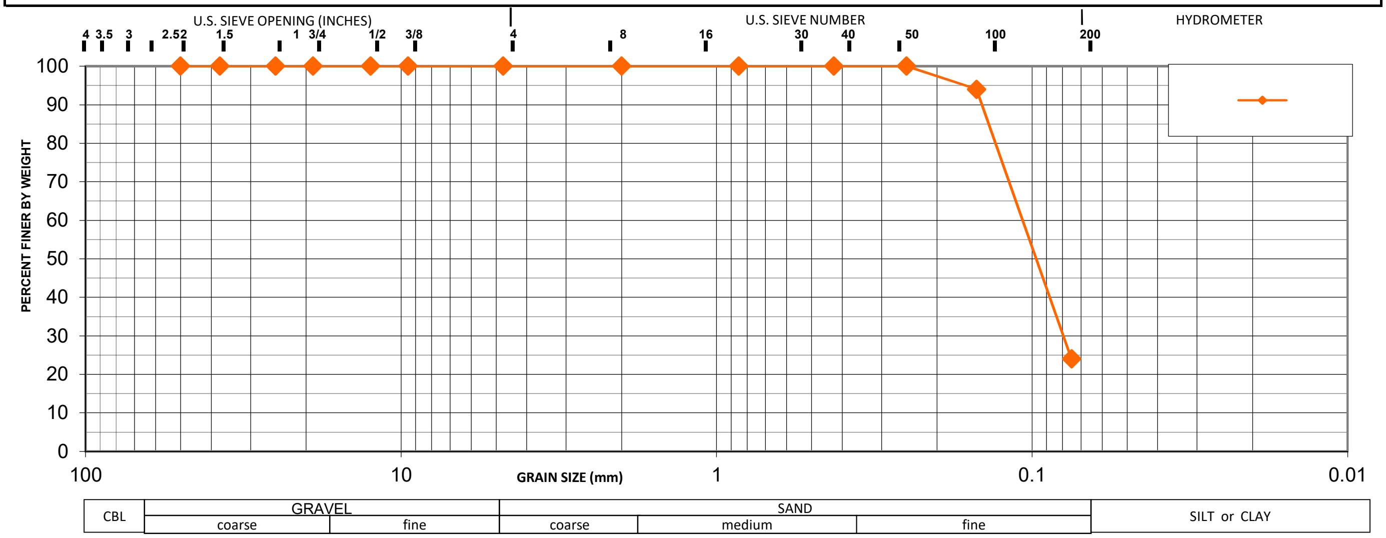
REPORT OF SIEVE ANALYSIS TEST

ASTM D422 - Soil

Date: May 5, 2016
Client: County of Imperial
Address: 1002 State Street
El Centro, CA 92243
Project : Dogwood Road Bridge Replacement
Project Address: El Centro, CA

Job Number: 226816-00103 PH2
Report Number: 4264
Lab Number: 112666

Material	clayey SAND (SC)					
Color	Brown					
Sample Location	B3-SPT4 @ 35-36.5 ft.					
Date Sampled	3/24/2016					
Sampled By	Sean Roy					
Date Tested	4/11/2016					
Tested By	Darrel Delgado					



Sample ID:	112666					
Sieve Size	% Passing					
63mm (2 1/2")						
50mm (2")						
37.5mm (1 1/2")	**100**					
25mm (1")	**100**					
19mm (3/4")	**100**					
12.5mm (1/2")	**100**					
9.5mm (3/8")	**100**					
4.75mm (#4)	**100**					
2mm (#10)	**100**					
850µm (#20)	**100**					
425µm (#40)	**100**					
250µm (#60)	**100**					
150 µm (#100)	**94**					
75 um (#200) washµ	**24**					
Fineness Modulus	0.9					
Shape (sand & gravel)	N.R.					
Hardness (sand & gravel)	N.R.					
Specific Gravity	2.65					
Coef. of Curvature (C_C)	37.5					
Coef. of Uniformity (C_U)	67.9					
% Gravel	0					
% Sand	76					
% Fines	24.0					
USCS Class:	**SC**					

Notes: *Hardness: H&D = Hard & Durable; W&F = Weathered & Friable*
N.R.: Not Recorded; N/A: Not Available.

Respectfully Submitted,
NV5 West, Inc.

Sam Koohi, PE
Engineering Manager

NV5

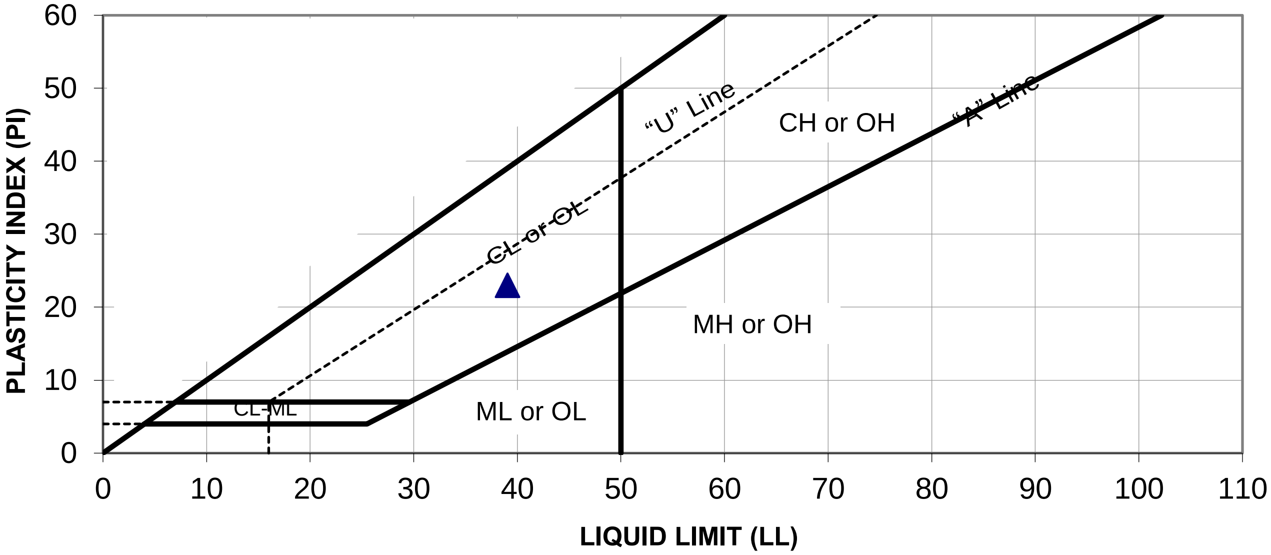
REPORT OF LIQUID LIMIT, PLASTIC LIMIT & PLASTICITY INDEX TESTS

(ASTM 4318)

Date:	5/12/2016	Job Number:	226816-00103 PH2 T2.2
Client:	County of Imperial	Report Number:	4264
Address:	1002 State Street El Centro, CA 92243	Lab Number:	112664
Project :	Dogwood Road Bridge Replacement		
Project Address :	El Centro, CA		

Sample Location: B3-D3
Sampled By: Sean Roy
Sample Identification: 112664
Depth: 26-26.5

Date Sampled: 3/24/2016 Date Received: 4/11/2016 Date Tested: 4/11/2006



SUMMARY OF TEST RESULTS

SAMPLE ID	SOURCE / LOCATION	DEPTH/ ELEV.	%>#40	TEST RESULT			USCS	
				LL	PL	PI	Class	Group Name
112664	B3-D3	26-26.5		39	16	23	CL	lean CLAY

Reviewed By: ____________________

Sam Koohi, PE
Engineering Manager

NV5

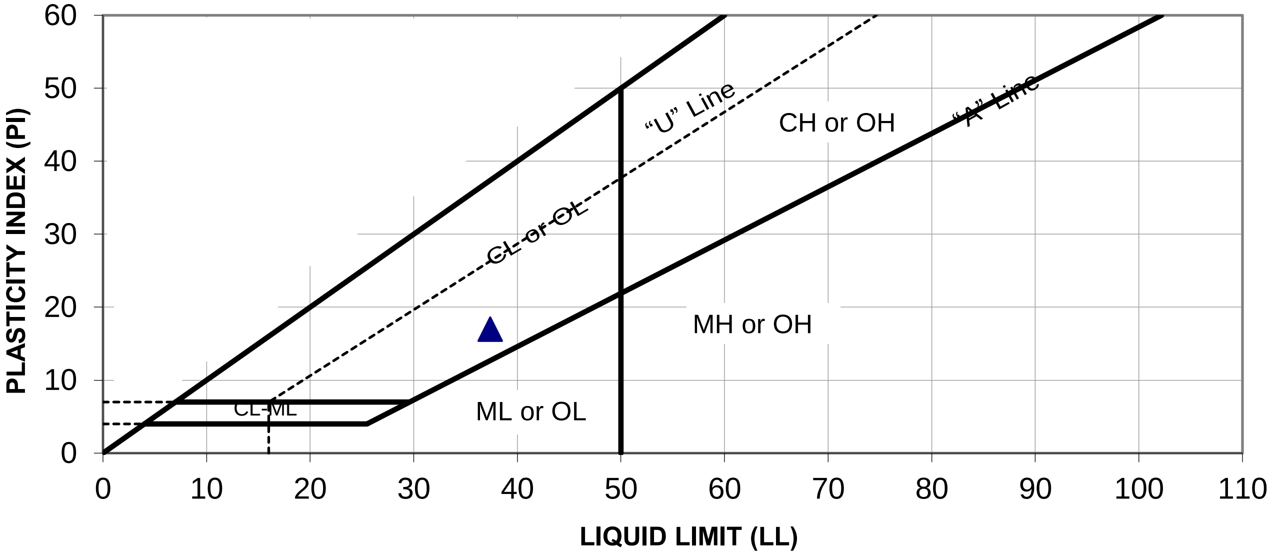
REPORT OF LIQUID LIMIT, PLASTIC LIMIT & PLASTICITY INDEX TESTS

(ASTM 4318)

Date: 5/12/2016
Client: County of Imperial
Address: 1002 State Street
El Centro, CA 92243
Project : Dogwood Road Bridge Replacement
Project Address : El Centro, CA

Job Number: 226816-00103 PH2 T2.2
Report Number: 4264
Lab Number: 112681

Sample Location: B2-D2
Sampled By: Sean Roy
Sample Identification: 112681
Depth: 31-31.5
Date Sampled: 3/24/2016 Date Received: 4/11/2016 Date Tested: 4/11/2006



SUMMARY OF TEST RESULTS

SAMPLE ID	SOURCE / LOCATION	DEPTH/ ELEV.	%>#40	TEST RESULT			USCS	
				LL	PL	PI	Class	Group Name
112681	B2-D2	31-31.5		37	20	17	CL	lean CLAY

Reviewed By: ______________________
Sam Koohi, PE
Engineering Manager

EXPANSION INDEX TEST

(ASTM D4829)

Date: May 5, 2016
Client: County of Imperial
Address: 1002 State Street
El Centro, CA 92243
Project: Dogwood Road Bridge Replacement
Project Add: El Centro, CA

Job Number: 226816-00103 PH2 T2.2
Report Number: 4264

Sampled By: Sean Roy
Date Received: 3/28/16

Lab Number	112676
Location	0-5 ft. Depth
Sample No.	B1
Initial Moisture Content, %	11.6
Final Moisture Content, %	23.8
Dry Density, pcf	105.1
Saturation, %	51.8
Expansion Index	**68**
Potential Expansion	**Medium**

Respectfully submitted,
NV5 West, Inc.

Sam Koohi, PE
Engineering Manager

REPORT OF MOISTURE/DENSITY RELATIONSHIP TEST

(ASTM D1557/D698)

Date: May 5, 2016
Client: County of Imperial
Address: 1002 State Street
El Centro, CA 92243

Job Number: 226816-00103
Report Number: 4264
Lab Number: 112690

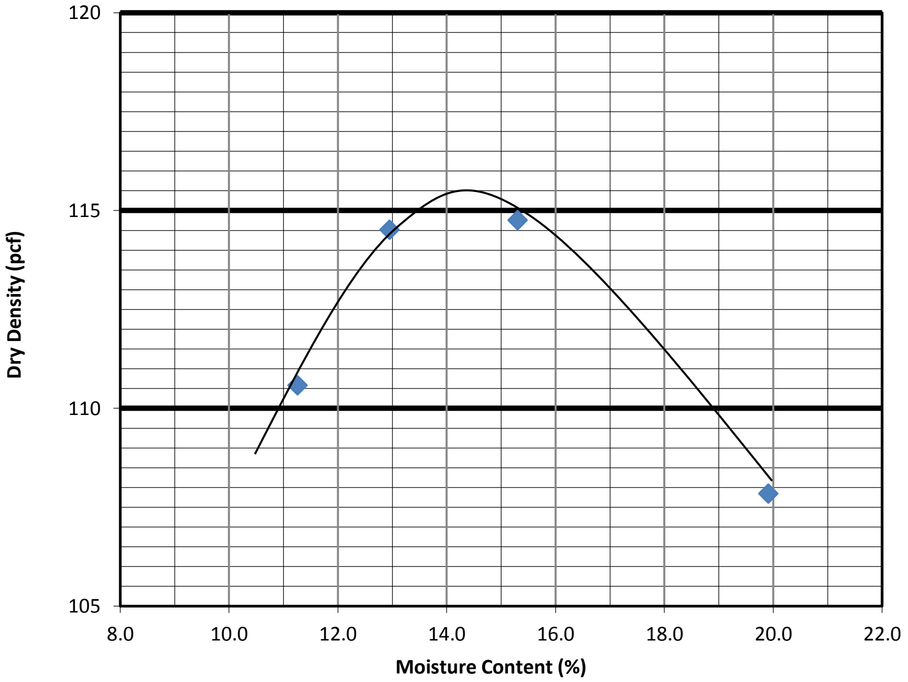
Project: Dogwood Road Bridge Replacement
Project Address: El Centro, CA
Material: Brown CLAY (CH)
Location: B4 at 0-5ft
Date Sampled: 3/24/16
Sampled By: Sean Roy

Mold Size: 6 inch
ASTM D1557 C

Maximum Dry Density = 115.5 pcf

Optimum Moisture = 14.5%

Dry Density vs Moisture Relationship



Distribution
Client
File

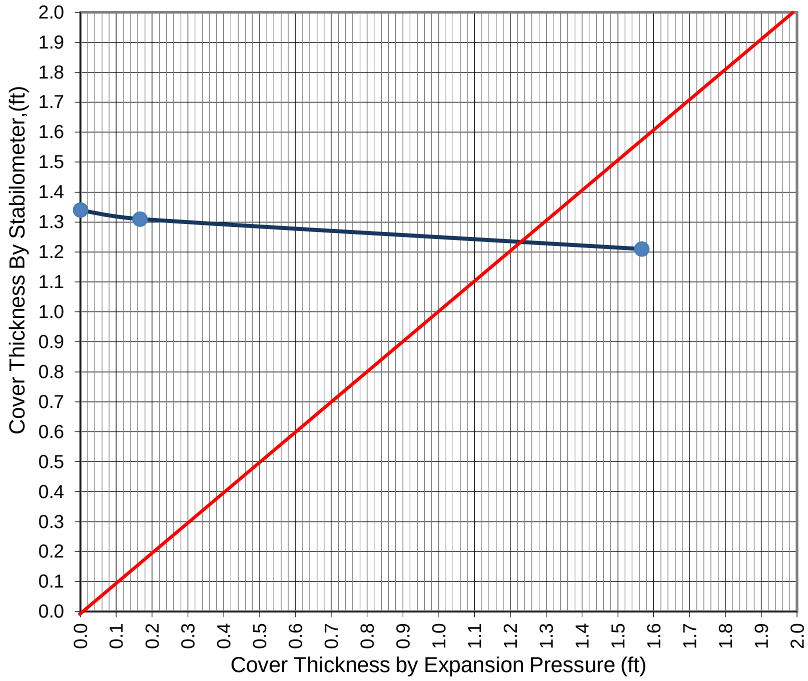
Reviewed By: ____________________
Sam Koohi, PE
Engineering Manager

RESISTANCE "R" VALUE TEST

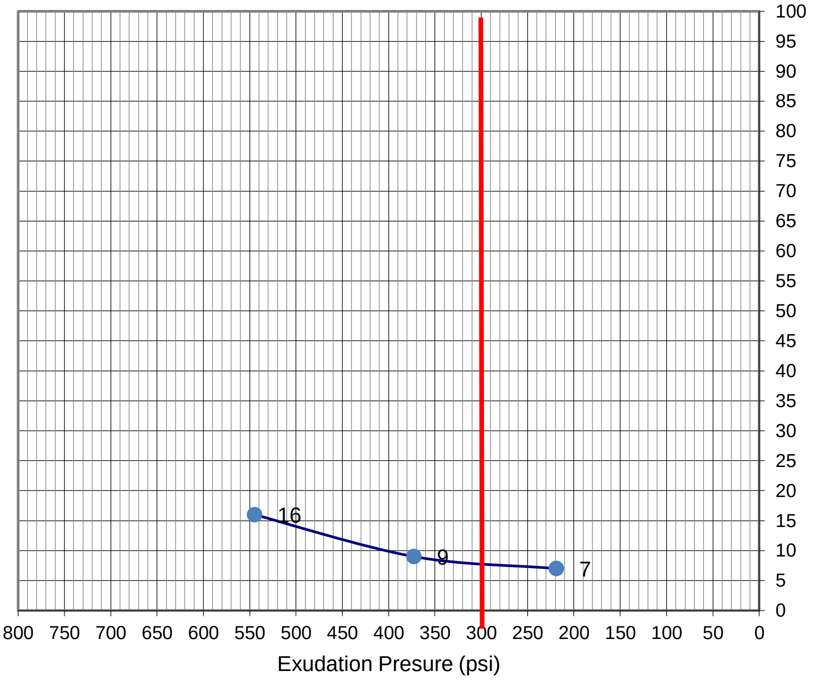
(CTM301 Caltrans / ASTM D2844)

Date:	5/5/2016	Job Number:	22816-00103 PH2
Client:	County of Imperial	Report Number:	4264
Address:	P.O. Box 129007	Lab Number:	112676
	El Centro, CA 92243	Boring No./Depth:	B1, 0-5'
Project :	Dogwood Road Bridge Replacement		
Project Address :	El Centro, CA		

EXPANSION PRESSURE CHART



EXUDATION PRESSURE CHART



TEST SPECIMEN	A	B	C	D
COMP. FOOT PRESSURE, psi	70	100	165	
INITIAL MOISTURE %	2.8	2.8	2.8	
MOISTURE @ COMPACTION %	14.8	13.1	11.4	
DRY DENSITY, pcf	112.3	115.6	120.4	
EXUDATION PRESSURE, psi	219	373	545	
STABILOMETER VALUE 'R'	7	9	16	

R-VALUE BY EXUDATION	8
R-VALUE BY EXPANSION	15
R-VALUE AT EQUILIBRIUM	**8**

Respectfully Submitted,

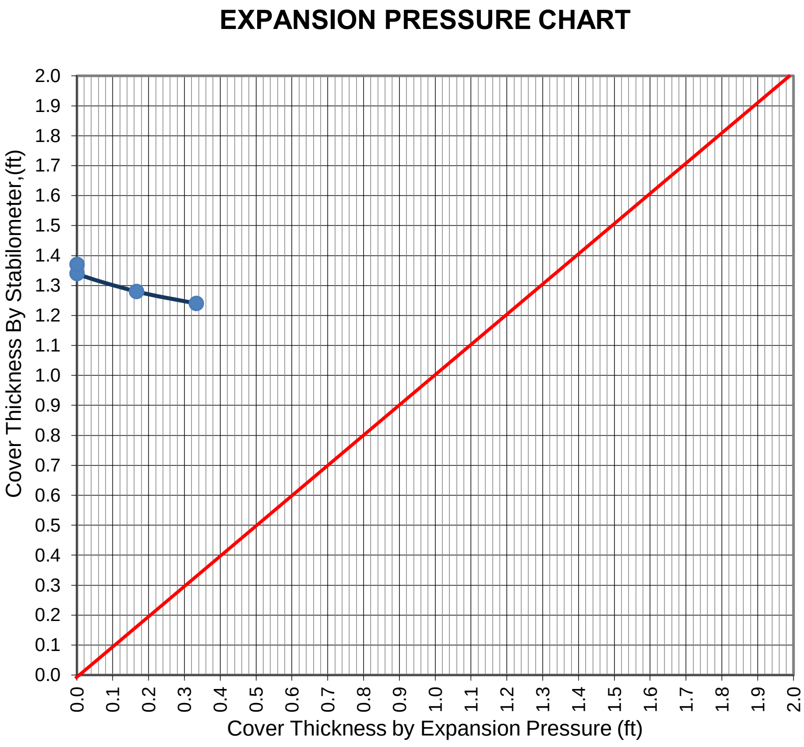
NV5 West, Inc.

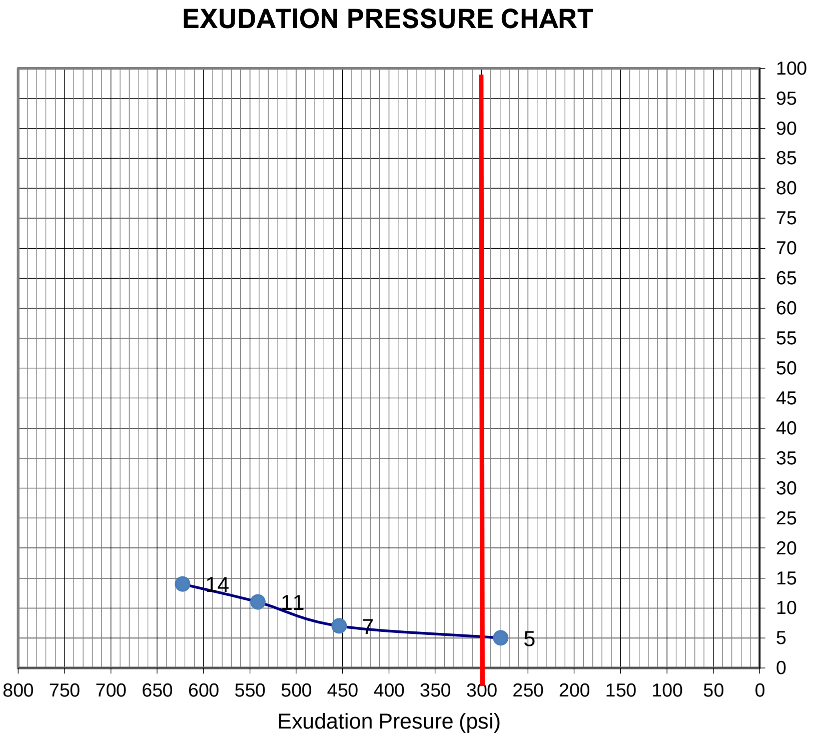
Sam Koohi, PE
Engineering Manager

RESISTANCE "R" VALUE TEST

(CTM301 Caltrans / ASTM D2844)

Date:	5/5/2016	Job Number:	22816-00103 PH2
Client:	County of Imperial	Report Number:	4264
Address:	P.O. Box 129007 El Centro, CA 92243	Lab Number:	112690
		Boring No./Depth:	B4, 0-5'
Project :	Dogwood Road Bridge Replacement		
Project Address :	El Centro, CA		


EXPANSION PRESSURE CHART


EXUDATION PRESSURE CHART

TEST SPECIMEN	A	B	C	D
COMP. FOOT PRESSURE, psi	160	105	85	60
INITIAL MOISTURE %	2.8	2.8	2.8	2.8
MOISTURE @ COMPACTION %	14.0	15.7	17.4	20.0
DRY DENSITY, pcf	114.0	110.5	107.8	102.1
EXUDATION PRESSURE, psi	623	541	454	279
STABILOMETER VALUE 'R'	14	11	7	5

R-VALUE BY EXUDATION	5
R-VALUE BY EXPANSION	0
R-VALUE AT EQUILIBRIUM	**5**

Respectfully Submitted,

NV5 West, Inc.

Sam Koohi, PE
Engineering Manager

LABORATORY REPORT

Telephone (619) 425-1993 Fax 425-7917 Established 1928

CLARKSON LABORATORY AND SUPPLY INC.
350 Trousdale Dr. Chula Vista, Ca. 91910 www.clarksonlab.com
ANALYTICAL AND CONSULTING CHEMISTS

Date: April 14, 2016
Purchase Order Number: 16-0361
Sales Order Number: 30906
Account Number: NV5.SD

To:
--
NV5 West Inc
15092 Avenue of Science #200
San Diego, CA 92128
Attention: Michelle Albrecht

Laboratory Number: SO5967 Customers Phone: 858-715-5800
Fax: 858-715-5810
Sample Designation:
--
One soil sample received on 04/07/16 at 3:00pm, taken on 04/07/16 from Dogwood Road Bridge Job# 226816-00103 marked as B3-SPT 1 @ 10-11.5' Lab# 112661.

Analysis By California Test 643, 1999, Department of Transportation Division of Construction, Method for Estimating the Service Life of Steel Culverts.

pH 8.7

Water Added (ml)	Resistivity (ohm-cm)
10	7200
5	4000
5	2900
5	1600
5	940
5	900
5	940
5	980

29 years to perforation for a 16 gauge metal culvert.
38 years to perforation for a 14 gauge metal culvert.
53 years to perforation for a 12 gauge metal culvert.
67 years to perforation for a 10 gauge metal culvert.
82 years to perforation for a 8 gauge metal culvert.

Water Soluble Sulfate	Calif. Test 417	0.017% (170ppm)
Water Soluble Chloride	Calif. Test 422	0.003% (32ppm)

Laura Torres

Laura Torres
LT/ilv

DIRECT SHEAR TEST (ASTM D3080)

Project No. **226816-00103**
Client: **County of Imperial**
Proj. Name: **Dogwood Road Bridge Replacement**
Location: **El Centro, CA**
Sample date: **3/24/2016** Sample Location: **16'-16.5'** Boring No: **B3-D2**

Report No.: **4264**
Lab No.: **112662**
Test Date: **4/18/2016**

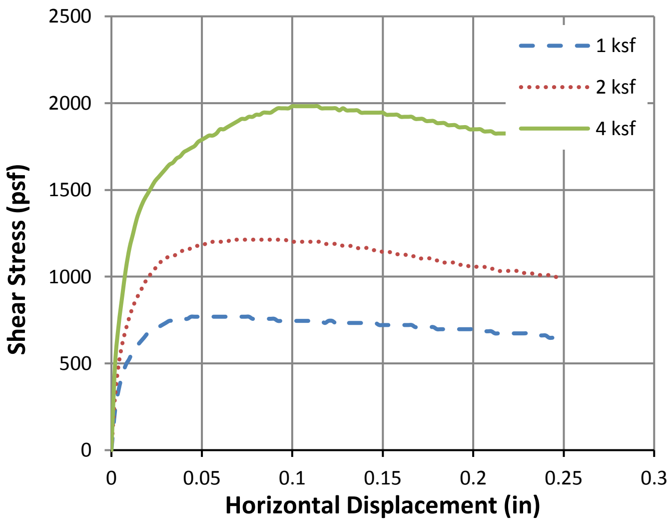
TEST DATA:

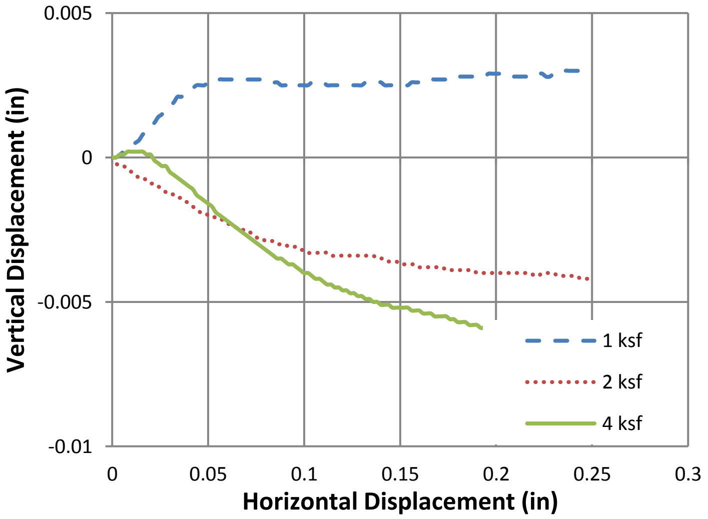
	Sample ID:	1 ksf	2 ksf	4 ksf
Initial	Water Content (%)	30.6	30.0	31.4
	Dry Density	93.1	93.5	91.8
	Saturation (%)	107.0	106.0	106.5
Final	Water Content (%)	33.0	31.8	30.8
	Dry Density	93.1	93.5	91.8
	Saturation (%)	115.4	112.3	104.4
Normal Stress (psf)		1000	2000	4000
Ultimate Shear Stress (psf)		648	997	1777
Peak Shear Stress (psf)		769	1213	1982

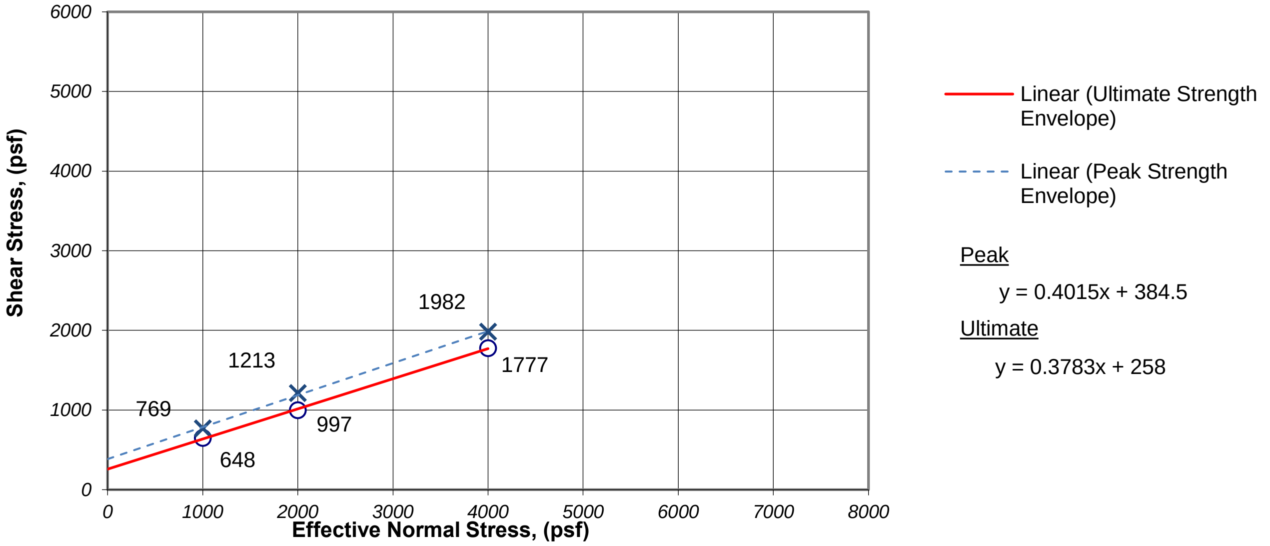
Sample Type: Undistrubed

Description: SILT (ML)

Color: Tan / Light Brown







Peak Cohesion, C'(psf): **385**
Peak Friction,Φ' (deg): **21.9**

Ultimate Cohesion, C'(psf): **258**
Ultimate Friction,Φ' (deg): **20.7**

Respectfully Submitted,
NV5 West, Inc.

Sam Koohi, PE
Engineering Manager

NV5
15092 Avenue of Science, Ste 200
San Diego CA 92128
p. 858 385 0500 f. 858 715 5810

DIRECT SHEAR TEST (ASTM D3080)

Project No.: **226816-00103**
Client: **County of Imperial**
Proj. Name: **Dogwood Road Bridge Replacement**
Location: **El Centro, CA**
Sample date: **3/24/2016**
Sample Location: **41'-41.5'**
Boring No: **B3-D4**

Report No.: **4264**
Lab No.: **112667**
Test Date: **4/15/2016**

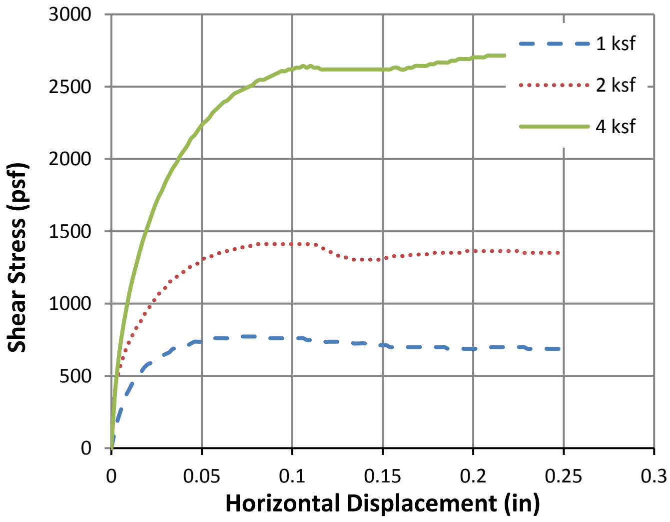
TEST DATA:

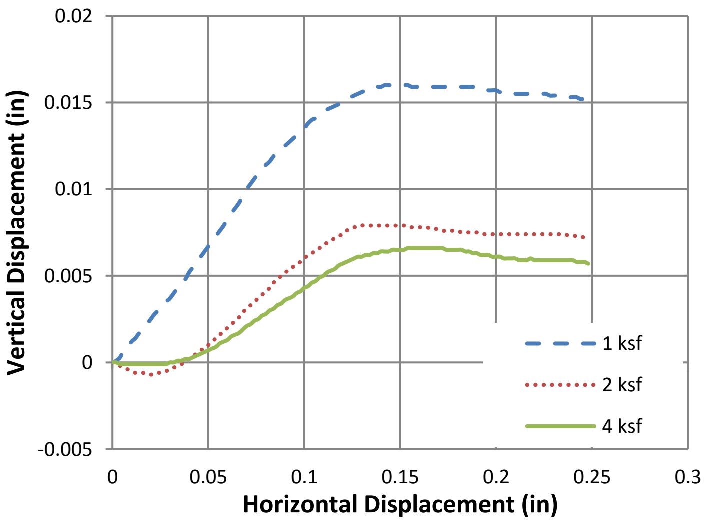
	Sample ID:	1 ksf	2 ksf	4 ksf
Initial	Water Content (%)	25.2	26.0	25.4
	Dry Density	98.9	96.5	95.8
	Saturation (%)	102.4	99.2	95.3
Final	Water Content (%)	28.7	29.0	29.8
	Dry Density	98.9	96.5	95.8
	Saturation (%)	116.3	110.6	111.7
Normal Stress (psf)		1000	2000	4000
Ultimate Shear Stress (psf)		687	1351	2595
Peak Shear Stress (psf)		772	1411	2715

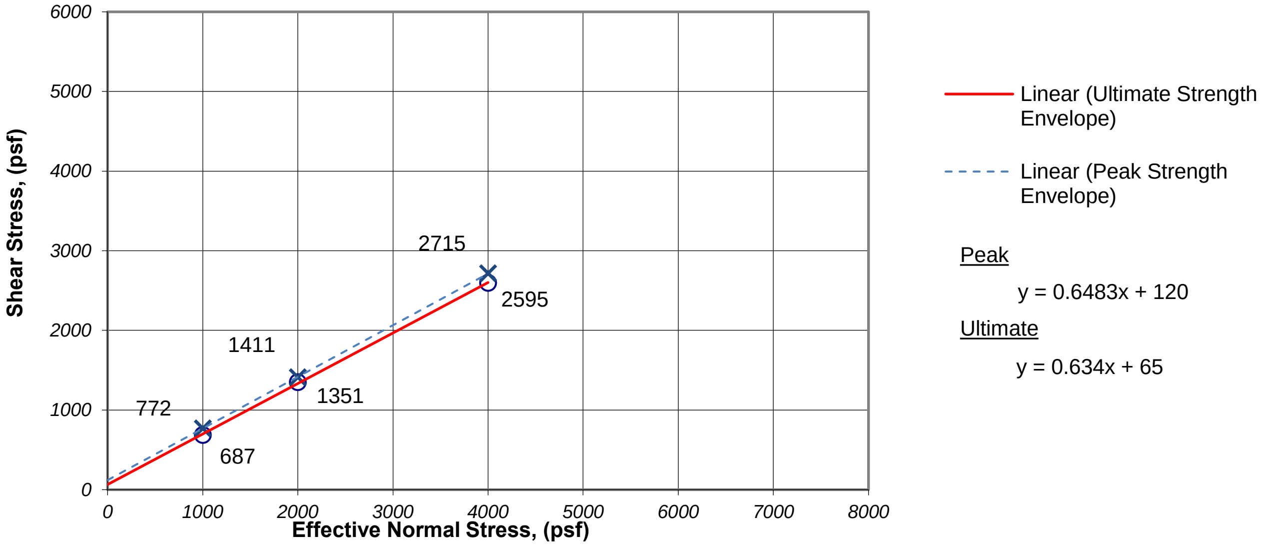
Sample Type: Undistrubed

Description: SAND (SP)

Color: Tan







Peak Cohesion, C'(psf): **120**
Peak Friction,Φ' (deg): **33.0**

Ultimate Cohesion, C'(psf): **65**
Ultimate Friction,Φ' (deg): **32.4**

Respectfully Submitted,
NV5 West, Inc.

Sam Koohi, PE
Engineering Manager

NV5
15092 Avenue of Science, Ste 200
San Diego CA 92128
p. 858 385 0500 f. 858 715 5810

DIRECT SHEAR TEST (ASTM D3080)

Project No. **226816-00103**
Client: **County of Imperial**
Proj. Name: **Dogwood Road Bridge**
Location: **El Centro, CA**
Sample date: **3/24/2016** Sample Location: **66-66.5'** Boring No: **B3-D5**

Report No.: **4264**
Lab No.: **112671**
Test Date: **4/15/2016**

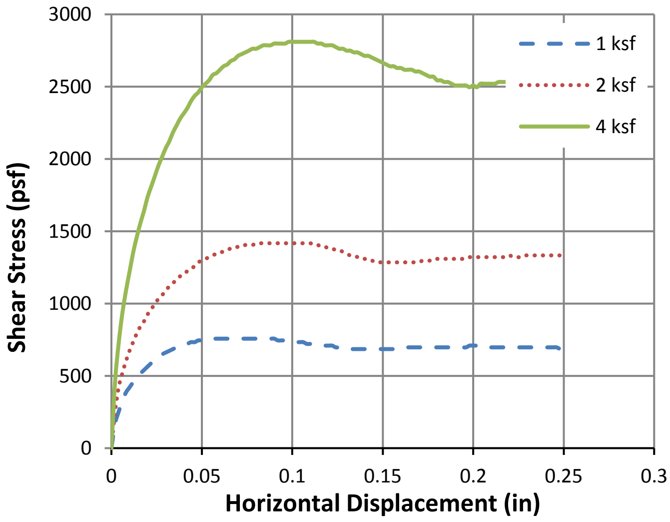
TEST DATA:

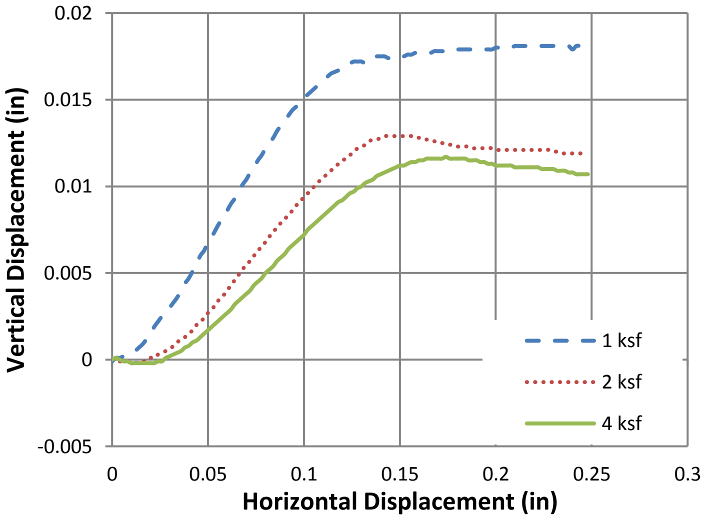
	Sample ID:	1 ksf	2 ksf	4 ksf
Initial	Water Content (%)	23.3	25.3	24.1
	Dry Density	102.5	100.5	101.6
	Saturation (%)	103.9	107.0	105.2
Final	Water Content (%)	25.5	26.1	27.3
	Dry Density	102.5	100.5	101.6
	Saturation (%)	113.9	110.4	118.9
Normal Stress (psf)		1000	2000	4000
Ultimate Shear Stress (psf)		697	1333	2544
Peak Shear Stress (psf)		757	1417	2810

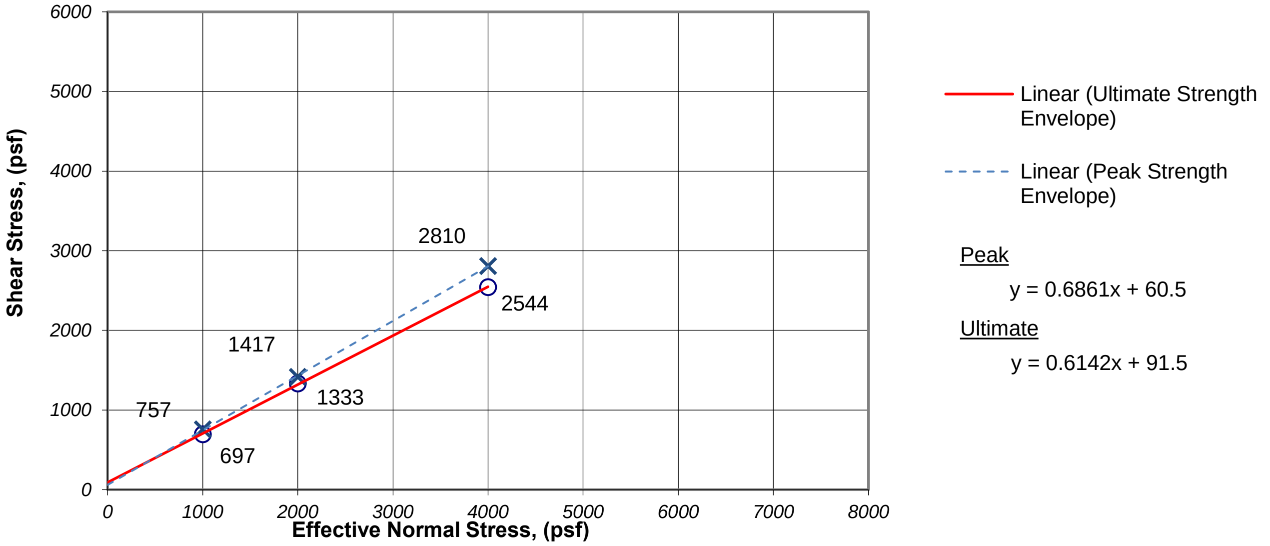
Sample Type: Undistrubed

Description: SAND (SP)

Color: Tan







Peak Cohesion, C'(psf): **61**
Peak Friction,Φ' (deg): **34.5**

Ultimate Cohesion, C'(psf): **92**
Ultimate Friction,Φ' (deg): **31.6**

Respectfully Submitted,
NV5 West, Inc.

Sam Koohi, PE
Engineering Manager

NV5
15092 Avenue of Science, Ste 200
San Diego CA 92128
p. 858 385 0500 f. 858 715 5810

DIRECT SHEAR TEST (ASTM D3080)

Project No. **226816-00103**
Client: **County of Imperial**
Proj. Name: **Dogwood Road Bridge Replacement**
Location: **El Centro, CA**
Sample date: **3/24/2016** Sample Location: **21'-21.5'** Boring No: **B2-D1**

Report No.: **4264**
Lab No.: **112679**
Test Date: **4/25/2016**

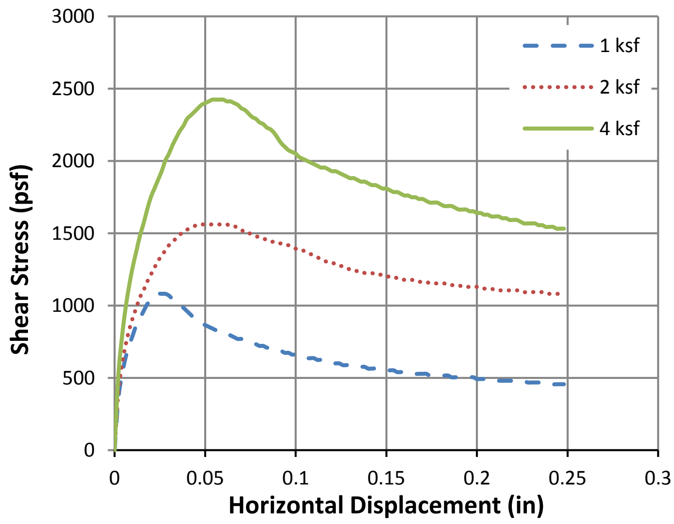
TEST DATA:

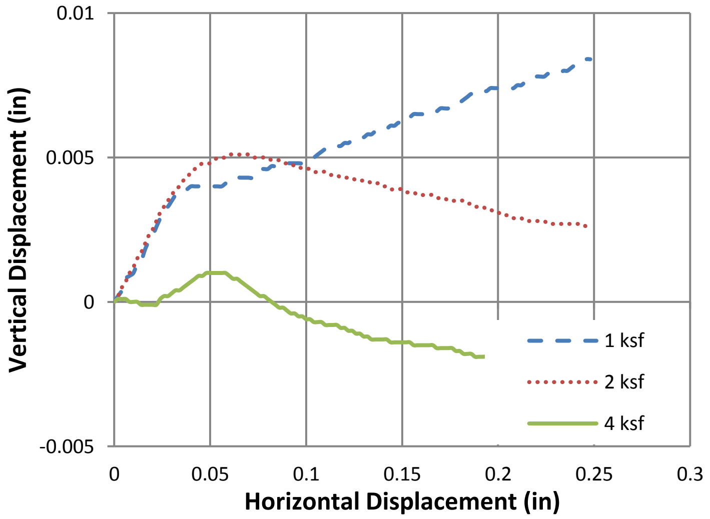
	Sample ID:	1 ksf	2 ksf	4 ksf
Initial	Water Content (%)	24.6	25.7	26.1
	Dry Density	99.1	99.9	101.4
	Saturation (%)	100.6	107.3	113.2
Final	Water Content (%)	28.6	29.8	28.6
	Dry Density	99.1	99.9	101.4
	Saturation (%)	116.7	124.3	123.9
Normal Stress (psf)		1000	2000	4000
Ultimate Shear Stress (psf)		456	1081	1519
Peak Shear Stress (psf)		1081	1561	2424

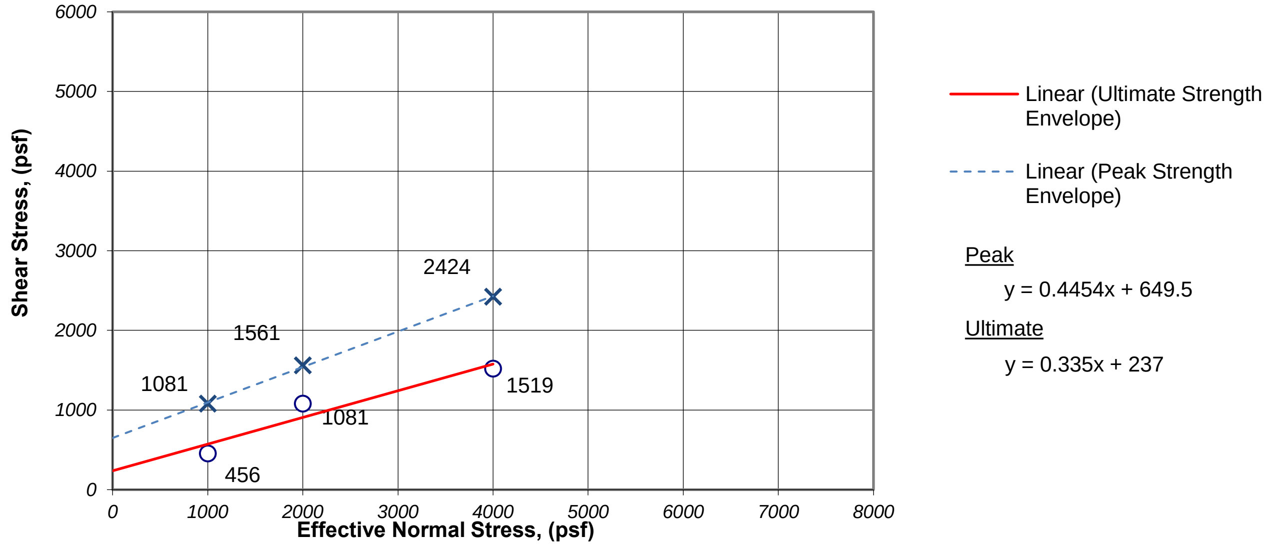
Sample Type: Undistrubed

Description: CLAY (CL)

Color: Light Brown







Peak Cohesion, C'(psf): **650**
Peak Friction,Φ' (deg): **24.0**

Ultimate Cohesion, C'(psf): **237**
Ultimate Friction,Φ' (deg): **18.5**

Respectfully Submitted,
NV5 West, Inc.

Sam Koohi, PE
Engineering Manager

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DIRECT SHEAR TEST (ASTM D3080)

Project No.: **226816-00103**
Client: **County of Imperial**
Proj. Name: **Dogwood Road Bridge Replacement**
Location: **El Centro, CA**
Sample date: **3/24/2016** Sample Location: **41'-41.5'** Boring No: **B2-D3**

Report No.: **4264**
Lab No.: **112683**
Test Date: **4/27/2016**

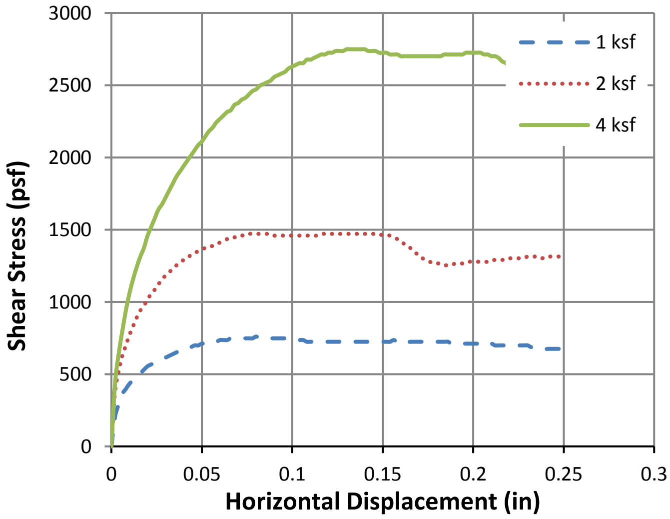
TEST DATA:

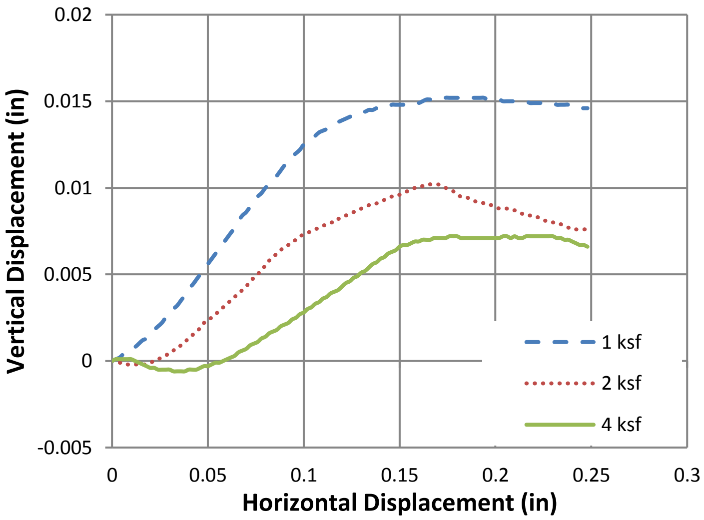
	Sample ID:	1 ksf	2 ksf	4 ksf
Initial	Water Content (%)	25.6	25.9	21.1
	Dry Density	94.4	97.9	101.1
	Saturation (%)	92.6	102.6	90.5
Final	Water Content (%)	27.5	26.5	24.3
	Dry Density	94.4	97.9	101.1
	Saturation (%)	99.7	105.0	104.5
Normal Stress (psf)		1000	2000	4000
Ultimate Shear Stress (psf)		675	1314	2460
Peak Shear Stress (psf)		760	1471	2749

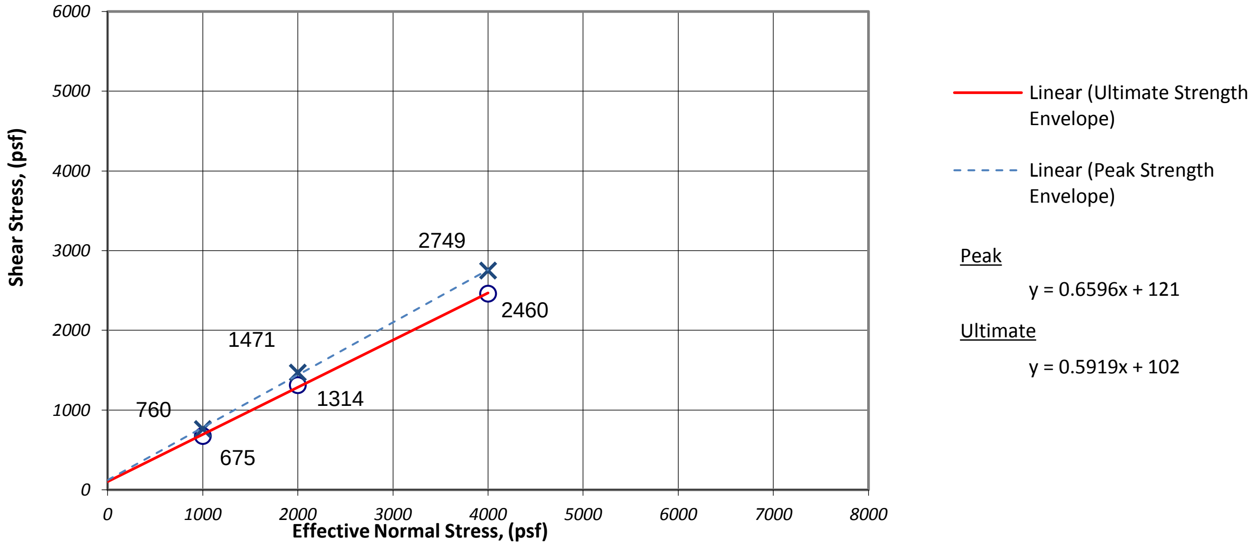
Sample Type: Undistrubed

Description: Sandy SILT (SM)

Color: Tan / Light Brown







Peak Cohesion, C'(psf): **121**
Peak Friction,Φ' (deg): **33.4**

Ultimate Cohesion, C'(psf): **102**
Ultimate Friction,Φ' (deg): **30.6**

Respectfully Submitted,
NV5 West, Inc.

Sam Koohi, PE
Engineering Manager

NV5
15092 Avenue of Science, Ste 200
San Diego CA 92128
p. 858 385 0500 f. 858 715 5810

DIRECT SHEAR TEST (ASTM D3080)

Project No. **226816-00103**
Client: **County of Imperial**
Proj. Name: **Dogwood Road Bridge Replacement**
Location: **El Centro, CA**
Sample date: **3/24/2016** Sample Location: **61'-61.5'** Boring No: **B2-D5**

Report No.: **4264**
Lab No.: **112687**
Test Date: **4/19/2016**

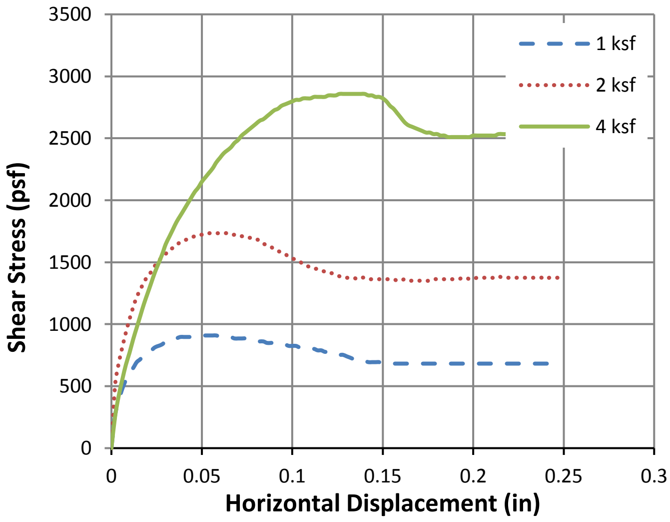
TEST DATA:

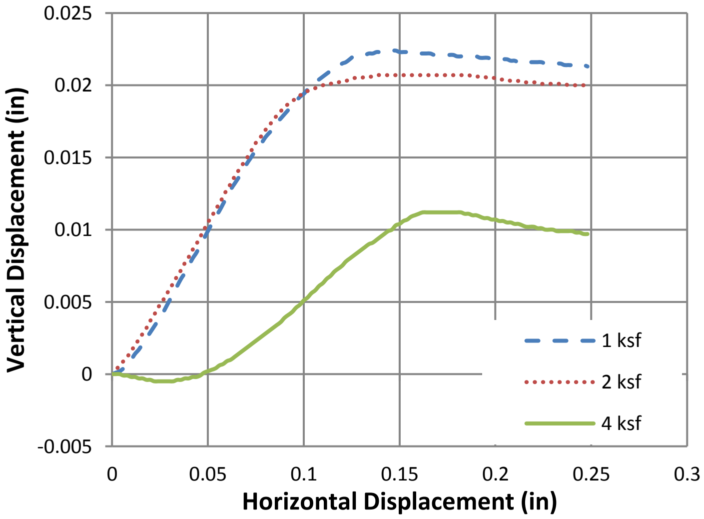
	Sample ID:	1 ksf	2 ksf	4 ksf
Initial	Water Content (%)	21.7	22.8	21.4
	Dry Density	104.6	103.3	102.1
	Saturation (%)	102.5	103.8	94.2
Final	Water Content (%)	23.8	24.6	23.7
	Dry Density	104.6	103.3	102.1
	Saturation (%)	112.3	112.2	104.6
Normal Stress (psf)		1000	2000	4000
Ultimate Shear Stress (psf)		670	1375	2558
Peak Shear Stress (psf)		909	1734	2858

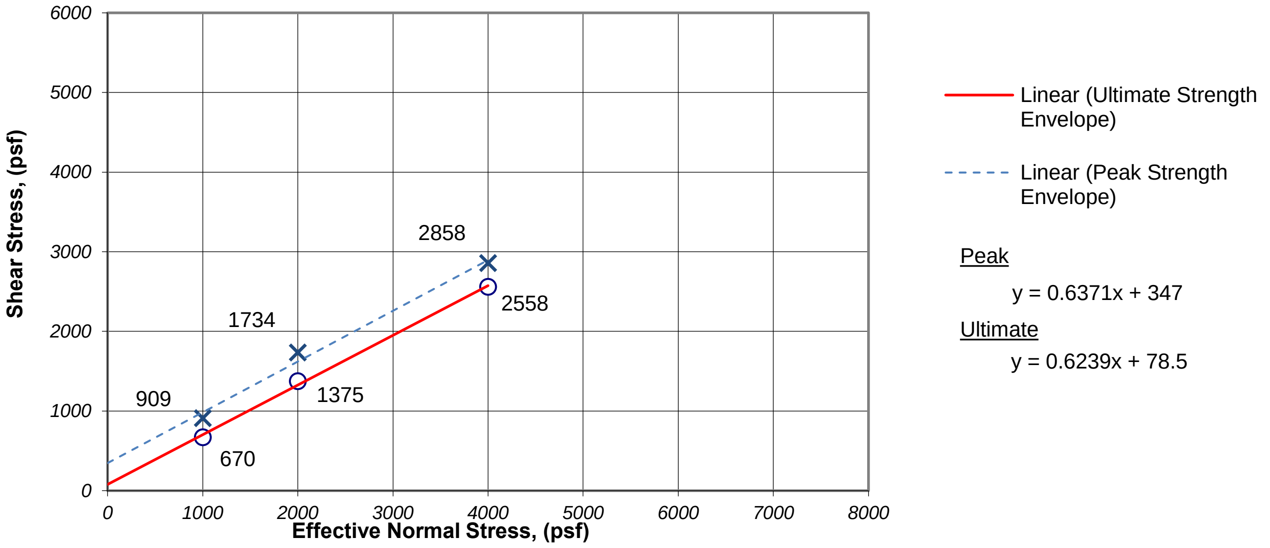
Sample Type: Undistrubed

Description: Silty Clayey SAND (SM - SC)

Color: Tan / Light Brown







Peak Cohesion, C'(psf): **347**
Peak Friction,Φ' (deg): **32.5**

Ultimate Cohesion, C'(psf): **79.5**
Ultimate Friction,Φ' (deg): **30.0**

Respectfully Submitted,
NV5 West, Inc.

Sam Koohi, PE
Engineering Manager

NV5
15092 Avenue of Science, Ste 200
San Diego CA 92128
p. 858 385 0500 f. 858 715 5810

Appendix D

Pile Design

PILE DATA TABLE

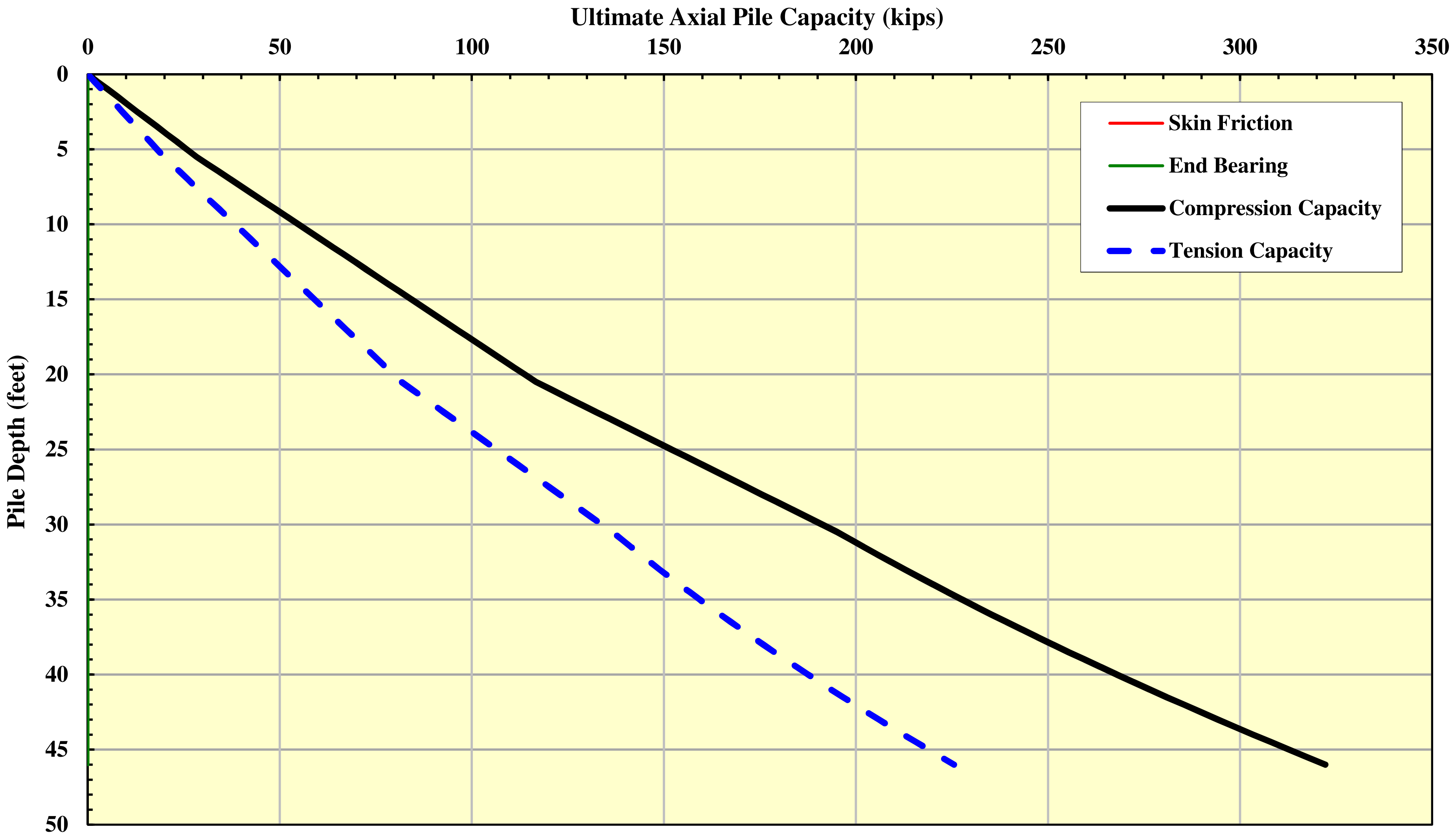
LOCATION	PILE TYPE	DESIGN LOADING (SERVICELOAD,KIPS)	NOMINAL RESISTANCE (KIPS)		CUT-OFF ELEVATIONS (FT.)	DESIGN TIP ELEVATIONS (FT.)	SPECIFIED TIP ELEVATIONS (FT.)
			COMPRESSION	TENSION			
ABUTMENT	CISS 30X0.5	160	320	0	994.42	948(a) 984(b) 933(c) 927(d)	927
PIER	CISS 30X0.5	220	440	0	994.42	937(a) 979(b) 928(c) 925(d)	925

NOTES:

1. DESIGN TIP ELEVATIONS ARE CONTROLLED BY: (a) COMPRESSION , (b) TENSION , (c) LATERAL, (d) LIQUEFACTION
2. SERVICE LOADS AND DESIGN TIP ELEVATION FOR LATERAL LOAD ARE TYPICALLY PROVIDED BY SD.
3. ESTIMATED DOWNDRAG FORCE PER PILE DUE TO LIQUEFACTION IS 185 KIPS.

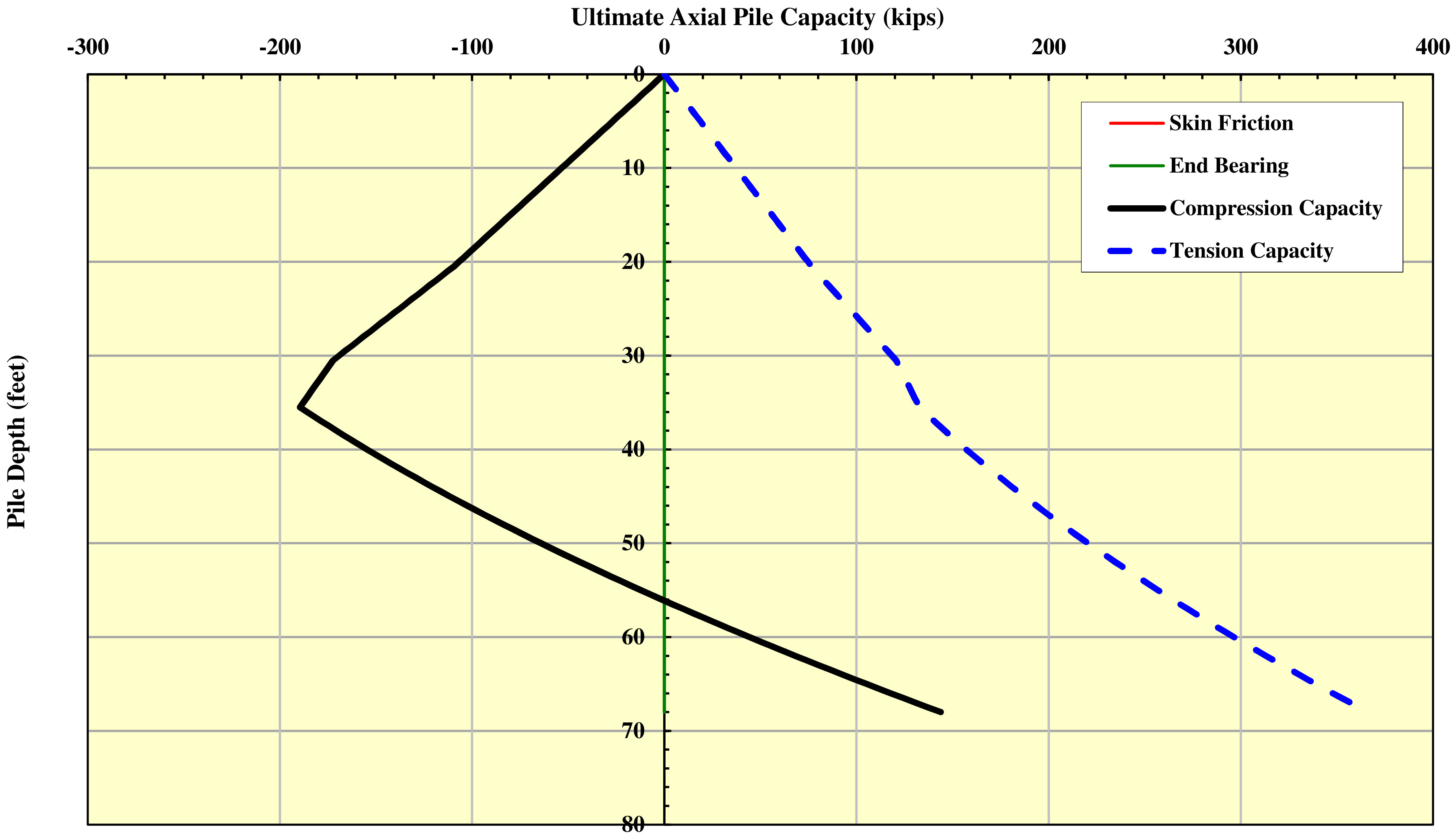
ANALYSIS OF AXIAL CAPACITY AND SETTLEMENT OF A SINGLE PILE

ULTIMATE AXIAL PILE CAPACITY VERSUS PILE DEPTH



ANALYSIS OF AXIAL CAPACITY AND SETTLEMENT OF A SINGLE PILE

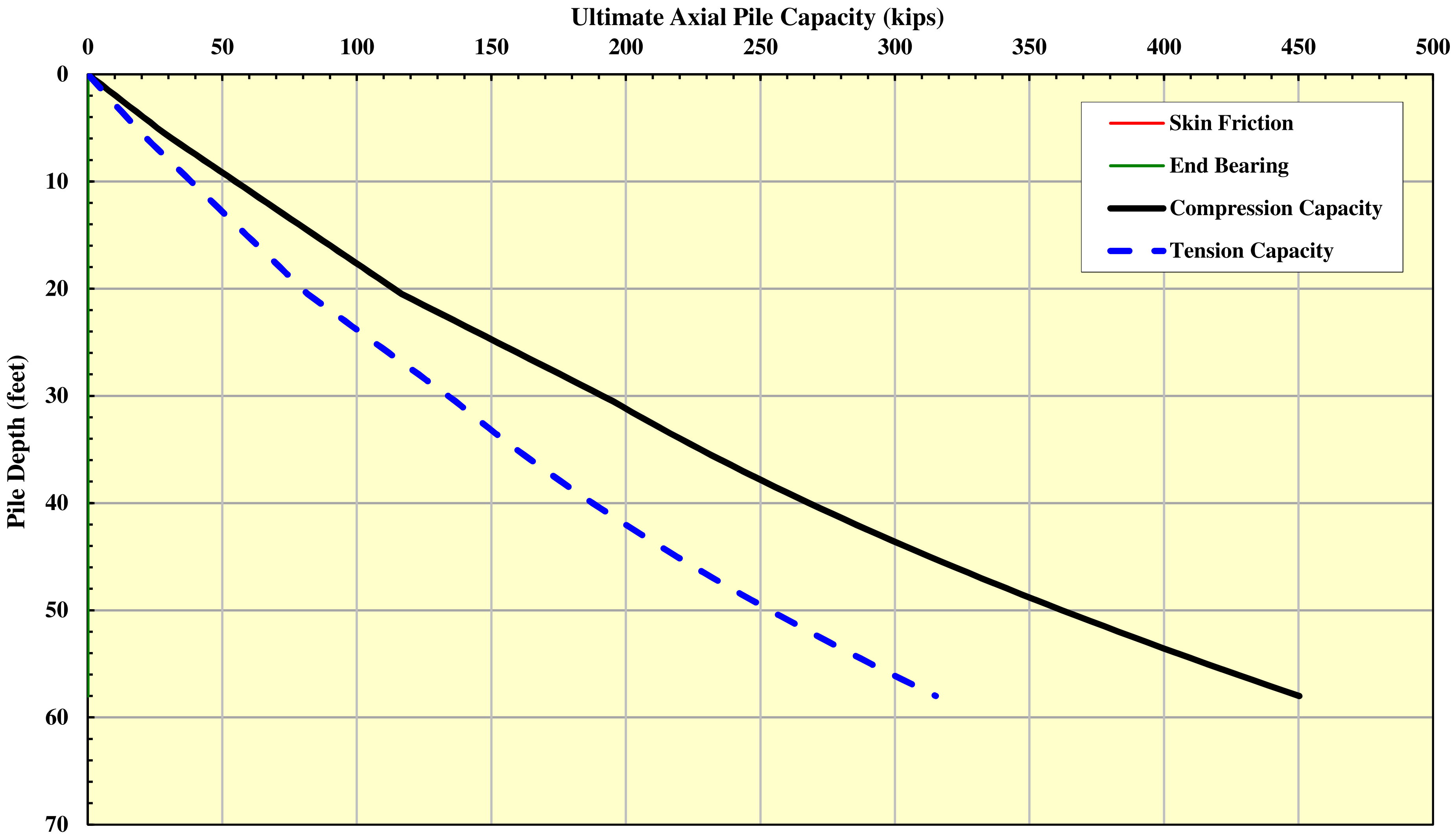
ULTIMATE AXIAL PILE CAPACITY VERSUS PILE DEPTH



ANALYSIS OF AXIAL CAPACITY AND SETTLEMENT OF A SINGLE PILE

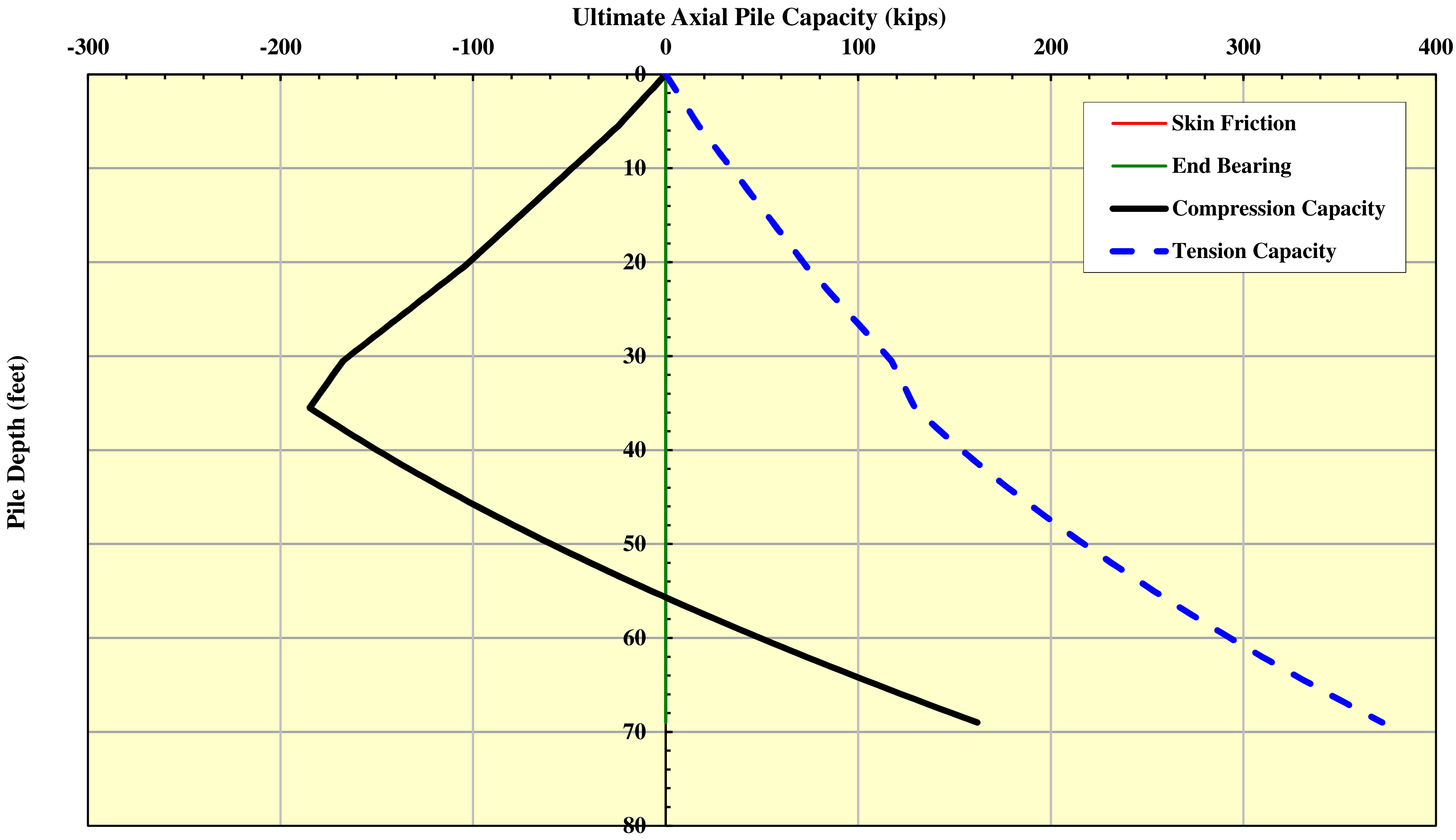
(Licensed to: NV5 West, Inc. - Irvine, California)

ULTIMATE AXIAL PILE CAPACITY VERSUS PILE DEPTH



ANALYSIS OF AXIAL CAPACITY AND SETTLEMENT OF A SINGLE PILE

ULTIMATE AXIAL PILE CAPACITY VERSUS PILE DEPTH



Appendix E

Liquefaction Analysis

SIMPLIFIED LIQUEFACTION HAZARDS ASSESSMENT USING STANDARD PENETRATION TEST (SPT) DATA

PROJECT INFORMATION

Project Name	Dogwood Road Bridge Replacement over Central Main Canal
Project No.	226816-00103.02
Project Location	Imperial County, California
Analyzed By	Carlos Amante
Reviewed By	Sam Koohi

SITE CONDITIONS

Boring No.	B-2
Ground Surface Elevation	996.00 feet
Proposed Grade Elevation	996.00 feet
Topographic Site Condition:	TSC1 (Level Ground with No Nearby Free Face)
- Ground Slope, S	0.00 %
- Free Face (L/H) Ratio	N/A
Ave. Total Unit Weight of New Fill	125.00 pcf
GWL Depth Measured During Test	5.00 feet
GWL Depth Used in Design	5.00 feet
Borehole Diameter	6.00 inches
Hammer Weight	140.00 pounds
Hammer Drop	30.00 inches
Hammer Energy Efficiency Ratio, ER	82.00 %
Hammer Distance to Ground Surface	5.00 feet

SEISMIC DESIGN PARAMETERS

Earthquake Moment Magnitude, M_w	6.70
Peak Ground Acceleration, A_{max}	0.65 g
Site to Seismic Source Distance, R	6.10 miles

SUMMARY OF RESULTS

Severity of Liquefaction:

Cumulative Thickness of Liq. Soils, H_{liq}: 5.00 feet (cumulative total thickness in the upper 65 feet)
Liquefaction Potential Index (LPI): 3.81 *** (Low risk, with minor liquefaction effects)

Seismic Settlements:	**Analysis Method**	**Upper 30 feet**	**Upper 50 feet**	**Upper 65 feet**
Seismic Compression Settlement:	Pradel (1998)	0.00 inches	0.00 inches	0.00 inches
Liquefaction-Induced Settlement:	Idriss and Boulanger (2008)	0.00 inches	1.59 inches	1.59 inches
Total Seismic Settlement:		0.00 inches	1.59 inches	1.59 inches

Seismic Lateral Displacements:	**Analysis Method**	**Upper 30 feet**	**Upper 50 feet**	**Upper 65 feet**
Cyclic Lateral Displacement:	Tokimatsu and Asaka (1998)	0.00 inches	1.33 inches	1.33 inches
Lateral Spreading Displacement:	Zhang et al. (2004)	0.00 inches	0.00 inches	0.00 inches

NOTES AND REFERENCES

\+ This method of analysis is based on observed seismic performance of level ground sites using correlation with normalized and fines-corrected SPT blow count, $(N_1)_{60cs} = f\{(N_1)_{60}, FC\}$ where $(N_1)_{60} = N_{field}\, C_N\, C_E\, C_B\, C_R\, C_S$

++ Based on criteria for liquefaction susceptibility screening selected by the user.

* FS_{liq} = Factor of Safety against liquefaction = (CRR/CSR), where CRR = $CRR_{7.5}$ MSF K_σ K_α , MSF = Magnitude Scaling Factor, $K_\sigma = f[(N_1)_{60}, \sigma'_{vo}]$, K_α =1.0, (level ground),
CSR = Cyclic Stress Ratio = 0.65 A_{max} $(\sigma_{vo}/\sigma'_{vo})$ r_d , and $CRR_{7.5}$ = Cyclic Resistance Ratio is a function of $(N_1)_{60cs}$ and corrected for an earthquake magnitude M_w of 7.5.

** Residual strength values of liquefied soils are based on correlation with post-earthquake, normalized and fines-corrected SPT blow count derived by Idriss and Boulanger (2008).

*** Based on Iwasaki et al. (1978) and Toprak and Holzer (2003)

\+ Reference: Boulanger, R.W. and Idriss, I.M. (2014), "CPT and SPT Based Liquefaction Triggering Procedures," University of California Davis, Center for Geotechnical Modeling Report No. UCD/CGM-14/01, 1-134.

SOIL PROFILE DATA

Bottom of Soil Layer Elevation (feet)	Soil Depth During Test (feet)	Material Type USCS Group Symbol (ASTM D2487)	Liquefaction Susceptibility Screening ++ Susceptible Soil? (Y/N)	Total Soil Unit Weight γ_t (pcf)	Field SPT Blow Count N_{field} (blows/ft)	Type of Soil Sampler	Fines Content FC (%)
991.00	2.50	CL-ML	N	125.00		SPT1	
988.50	6.25	CH	N	125.00		SPT1	
983.50	10.00	CH	N	125.00		SPT1	
978.50	15.00	CH	N	125.00		SPT1	
973.50	20.00	CL	N	125.00		SPT1	
968.50	25.00	CL	N	125.00		SPT1	
963.50	30.00	CH	N	125.00		SPT1	
958.50	35.00	SC	Y	125.00	14.00	SPT1	22.00
953.50	40.00	SC	Y	125.00	22.00	SPT1	22.00
948.50	45.00	SC	Y	125.00	42.00	SPT1	22.00
946.00	48.75	CL	N	125.00		SPT1	

LIQUEFACTION TRIGGERING ANALYSIS BASED ON R.W. BOULANGER AND I.M. IDRISS (2014) METHOD +

Total Vert. Stress (Design) σ_{vo} (psf)	Effective Vert. Stress (Design) σ'_{vo} (psf)	SPT Corr. For Vert. Stress C_N	SPT Corr. For Hammer Energy C_E	SPT Corr. For Borehole Size C_B	SPT Corr. For Rod Length C_R	SPT Corr. For Sampling Method C_S	Corrected SPT Blow Count N_{60}	Normalized SPT Blow Count $(N_1)_{60}$	Fines Corrected SPT Blow Count $(N_1)_{60cs}$	Shear Stress Reduction Coefficient r_d	Correction for High Overburden Stress K_σ	Cyclic Stress Ratio CSR	Cyclic Resistance Ratio CRR	Factor of Safety * FS_{liq}	Liquefaction Analysis Results	Residual Shear Strength ** S_r (psf)	Seismic Porewater Pressure Ratio r_u (%)	Cumulative Seismic Settlement (inches)	Cumulative Cyclic Lateral Displacement (inches)	Cumulative Lateral Spreading Displacement (inches)
312.50	312.50									1.00		0.42			(NL: Dry Soil)			1.59	1.33	0.00
781.25	703.25									0.99		0.46			(NL: Clay-rich Soil)			1.59	1.33	0.00
1,250.00	938.00									0.97		0.55			(NL: Clay-rich Soil)			1.59	1.33	0.00
1,875.00	1,251.00									0.95		0.60			(NL: Clay-rich Soil)			1.59	1.33	0.00
2,500.00	1,564.00									0.92		0.62			(NL: Clay-rich Soil)			1.59	1.33	0.00
3,125.00	1,877.00									0.89		0.63			(NL: Clay-rich Soil)			1.59	1.33	0.00
3,750.00	2,190.00									0.86		0.62			(NL: Clay-rich Soil)			1.59	1.33	0.00
4,375.00	2,503.00	0.90	1.37	1.05	1.00	1.00	20.1	18.1	22.9	0.83	0.97	0.61	0.28	0.46	LIQUEFY	437.96	100.00	1.59	1.33	0.00
5,000.00	2,816.00	0.88	1.37	1.05	1.00	1.00	31.6	27.7	32.5	0.80	0.94	0.60	0.87	1.46	(NL: Dense Soil)		28.35	0.36	0.38	0.00
5,625.00	3,129.00	0.91	1.37	1.05	1.00	1.00	60.3	54.7	59.5	0.77	0.87	0.58			(NL: Dense Soil)			0.00	0.05	0.00
6,093.75	3,363.75									0.75		0.57			(NL: Clay-rich Soil)			0.00	0.00	0.00

SIMPLIFIED LIQUEFACTION HAZARDS ASSESSMENT USING STANDARD PENETRATION TEST (SPT) DATA

PROJECT INFORMATION

Project Name	Dogwood Road Bridge Replacement over Central Main Canal
Project No.	226816-00103.02
Project Location	Imperial County, California
Analyzed By	Carlos Amante
Reviewed By	Sam Koohi

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Boring No.	B-2
Ground Surface Elevation	996.00 feet
Proposed Grade Elevation	996.00 feet
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- Ground Slope, S	0.00 %
- Free Face (L/H) Ratio	N/A
Ave. Total Unit Weight of New Fill	125.00 pcf
GWL Depth Measured During Test	5.00 feet
GWL Depth Used in Design	5.00 feet
Borehole Diameter	6.00 inches
Hammer Weight	140.00 pounds
Hammer Drop	30.00 inches
Hammer Energy Efficiency Ratio, ER	82.00 %
Hammer Distance to Ground Surface	5.00 feet

SEISMIC DESIGN PARAMETERS

Earthquake Moment Magnitude, M_w	6.70
Peak Ground Acceleration, A_{max}	0.65 g
Site to Seismic Source Distance, R	6.10 miles

SUMMARY OF RESULTS

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Liquefaction-Induced Settlement:	Idriss and Boulanger (2008)	0.00 inches	1.59 inches	1.59 inches
Total Seismic Settlement:		0.00 inches	1.59 inches	1.59 inches

Seismic Lateral Displacements:	**Analysis Method**	**Upper 30 feet**	**Upper 50 feet**	**Upper 65 feet**
Cyclic Lateral Displacement:	Tokimatsu and Asaka (1998)	0.00 inches	1.33 inches	1.33 inches
Lateral Spreading Displacement:	Zhang et al. (2004)	0.00 inches	0.00 inches	0.00 inches

NOTES AND REFERENCES

\+ This method of analysis is based on observed seismic performance of level ground sites using correlation with normalized and fines-corrected SPT blow count, $(N_1)_{60cs} = f\{(N_1)_{60}, FC\}$ where $(N_1)_{60} = N_{field}\, C_N\, C_E\, C_B\, C_R\, C_S$

++ Based on criteria for liquefaction susceptibility screening selected by the user.

* FS_{liq} = Factor of Safety against liquefaction = (CRR/CSR), where CRR = $CRR_{7.5}$ MSF K_σ K_α , MSF = Magnitude Scaling Factor, $K_\sigma = f[(N_1)_{60}, \sigma'_{vo}]$, K_α =1.0, (level ground),
CSR = Cyclic Stress Ratio = 0.65 A_{max} $(\sigma_{vo}/\sigma'_{vo})$ r_d , and $CRR_{7.5}$ = Cyclic Resistance Ratio is a function of $(N_1)_{60cs}$ and corrected for an earthquake magnitude M_w of 7.5.

** Residual strength values of liquefied soils are based on correlation with post-earthquake, normalized and fines-corrected SPT blow count derived by Idriss and Boulanger (2008).

*** Based on Iwasaki et al. (1978) and Toprak and Holzer (2003)

\+ Reference: Boulanger, R.W. and Idriss, I.M. (2014), "CPT and SPT Based Liquefaction Triggering Procedures," University of California Davis, Center for Geotechnical Modeling Report No. UCD/CGM-14/01, 1-134.

SOIL PROFILE DATA

Bottom of Soil Layer Elevation	Soil Depth During Test	Material Type USCS Group Symbol (ASTM D2487)	Liquefaction Susceptibility Screening ++ Susceptible Soil? (Y/N)	Total Soil Unit Weight γ_t	Field SPT Blow Count N_{field}	Type of Soil Sampler	Fines Content FC

LIQUEFACTION TRIGGERING ANALYSIS BASED ON R.W. BOULANGER AND I.M. IDRISS (2014) METHOD +

Total Vert. Stress (Design) σ_{vo}	Effective Vert. Stress (Design) σ'_{vo}	SPT Corr. For Vert. Stress C_N	SPT Corr. For Hammer Energy C_E	SPT Corr. For Borehole Size C_B	SPT Corr. For Rod Length C_R	SPT Corr. For Sampling Method C_S	Corrected SPT Blow Count N_{60}	Normalized SPT Blow Count $(N_1)_{60}$	Fines Corrected SPT Blow Count $(N_1)_{60cs}$	Shear Stress Reduction Coefficient r_d	Correction for High Overburden Stress K_σ	Cyclic Stress Ratio CSR	Cyclic Resistance Ratio CRR	Factor of Safety * FS_{liq}	Liquefaction Analysis Results	Residual Shear Strength ** S_r	Seismic Porewater Pressure Ratio r_u	Cumulative Seismic Settlement	Cumulative Cyclic Lateral Displacement	Cumulative Lateral Spreading Displacement

REFERENCES:

1. Boulanger, R.W. and Idriss, I.M. (2014), "CPT and SPT Based Liquefaction Triggering Procedures," University of California Davis, Center for Geotechnical Modeling Report No. UCD/CGM-14/01, 1-134.
2. Bray, J.D., and Sancio, R.B. (2006). "Assessment of the liquefaction susceptibility of fine-grained soils," Journal of Geotechnical and Geoenvironmental Engineering, ASCE 132 (9), 1165-1177.
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7. Kramer, S. and Wang, C.H. (2015), "Empirical Model for Estimation of the Residual Strength of Liquefied Soil," Journal of Geotechnical and Geoenvironmental Engineering, ASCE 141 (9).
8. Olson, S.M. and Johnson, C.I. (2008), "Analyzing Liquefaction-Induced Lateral Spreads Using Strength Ratios," Journal of Geotechnical and Geoenvironmental Engineering, ASCE 134 (8), 1035-1049.
9. Pradel, D. (1998), "Procedure to Evaluate Earthquake-Induced Settlements in Dry Sandy Soils," Journal of Geotechnical Engineering, ASCE 124 (4), pp. 364-368.
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SIMPLIFIED LIQUEFACTION HAZARDS ASSESSMENT USING STANDARD PENETRATION TEST (SPT) DATA

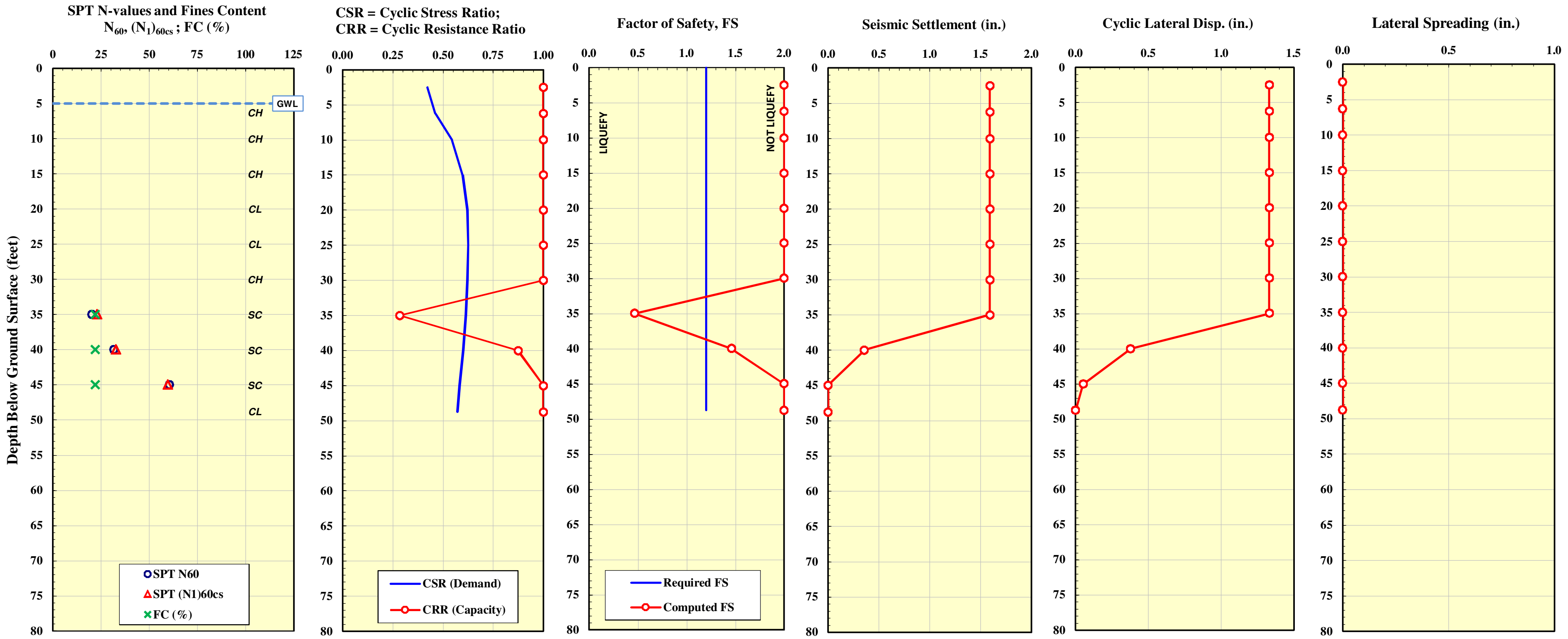
PROJECT INFORMATION	
Project Name	Dogwood Road Bridge Replacement over Central Main Canal
Project No.	226816-00103.02
Project Location	Imperial County, California
Analyzed By	Carlos Amante
Reviewed By	Sam Koohi

TOPOGRAPHIC SITE CONDITION	
Ground Slope, S	0.00 %
Free Face (L/H) Ratio	N/A

GROUNDWATER LEVEL DATA	
GWL Depth Measured During Test	5.00 feet
GWL Depth Used in Design	5.00 feet

BORING DATA	
Boring No.	B-2
Ground Surface Elevation	996.00 feet
Proposed Grade Elevation	996.00 feet
Borehole Diameter	6.00 inches
Hammer Weight	140.00 pounds
Hammer Drop	30.00 inches
Hammer Energy Efficiency Ratio, ER	82.00 %
Hammer Distance to Ground Surface	5.00 feet

SEISMIC DESIGN PARAMETERS	
Earthquake Moment Magnitude, M_w	6.70
Peak Ground Acceleration, A_{max}	0.65 g
Site to Seismic Source Distance, R	6.10 miles



Analysis Methods Used ==>>

Liquefaction Triggering:
Boulanger-Idriss (2014)

Seismic Settlements:
Above GWL: Pradel (1998)
Below GWL: Idriss and Boulanger (2008)

Cyclic Lateral Displacements:
Above GWL: Pradel (1998)
Below GWL: Tokimatsu and Asaka (1998)

Lateral Spreading:
Zhang et al. (2004)

Appendix F

ASFE Important Information about Your Geotechnical Engineering Report

Important Information about This

Geotechnical-Engineering Report

Subsurface problems are a principal cause of construction delays, cost overruns, claims, and disputes.

While you cannot eliminate all such risks, you can manage them. The following information is provided to help.

Geotechnical Services Are Performed for Specific Purposes, Persons, and Projects

Geotechnical engineers structure their services to meet the specific needs of their clients. A geotechnical-engineering study conducted for a civil engineer may not fulfill the needs of a constructor — a construction contractor — or even another civil engineer. Because each geotechnical- engineering study is unique, each geotechnical-engineering report is unique, prepared *solely* for the client. No one except you should rely on this geotechnical-engineering report without first conferring with the geotechnical engineer who prepared it. *And no one — not even you* — should apply this report for any purpose or project except the one originally contemplated.

Read the Full Report

Serious problems have occurred because those relying on a geotechnical-engineering report did not read it all. Do not rely on an executive summary. Do not read selected elements only.

Geotechnical Engineers Base Each Report on a Unique Set of Project-Specific Factors

Geotechnical engineers consider many unique, project-specific factors when establishing the scope of a study. Typical factors include: the client's goals, objectives, and risk-management preferences; the general nature of the structure involved, its size, and configuration; the location of the structure on the site; and other planned or existing site improvements, such as access roads, parking lots, and underground utilities. Unless the geotechnical engineer who conducted the study specifically indicates otherwise, do not rely on a geotechnical-engineering report that was:

- not prepared for you;
- not prepared for your project;
- not prepared for the specific site explored; or
- completed before important project changes were made.

Typical changes that can erode the reliability of an existing geotechnical-engineering report include those that affect:

- the function of the proposed structure, as when it's changed from a parking garage to an office building, or from a light-industrial plant to a refrigerated warehouse;
- the elevation, configuration, location, orientation, or weight of the proposed structure;
- the composition of the design team; or
- project ownership.

As a general rule, *always* inform your geotechnical engineer of project changes—even minor ones—and request an assessment of their impact. *Geotechnical engineers cannot accept responsibility or liability for problems that occur because their reports do not consider developments of which they were not informed.*

Subsurface Conditions Can Change

A geotechnical-engineering report is based on conditions that existed at the time the geotechnical engineer performed the study. *Do not rely on a geotechnical-engineering report whose adequacy may have been affected by*: the passage of time; man-made events, such as construction on or adjacent to the site; or natural events, such as floods, droughts, earthquakes, or groundwater fluctuations. *Contact the geotechnical engineer before applying this report to determine if it is still reliable.* A minor amount of additional testing or analysis could prevent major problems.

Most Geotechnical Findings Are Professional Opinions

Site exploration identifies subsurface conditions only at those points where subsurface tests are conducted or samples are taken. Geotechnical engineers review field and laboratory data and then apply their professional judgment to render an opinion about subsurface conditions throughout the site. Actual subsurface conditions may differ — sometimes significantly — from those indicated in your report. Retaining the geotechnical engineer who developed your report to provide geotechnical-construction observation is the most effective method of managing the risks associated with unanticipated conditions.

A Report's Recommendations Are Not Final

Do not overrely on the confirmation-dependent recommendations included in your report. *Confirmation-dependent recommendations are not final*, because geotechnical engineers develop them principally from judgment and opinion. Geotechnical engineers can finalize their recommendations *only* by observing actual subsurface conditions revealed during construction. *The geotechnical engineer who developed your report cannot assume responsibility or liability for the report's confirmation-dependent recommendations if that engineer does not perform the geotechnical-construction observation required to confirm the recommendations' applicability.*

A Geotechnical-Engineering Report Is Subject to Misinterpretation

Other design-team members' misinterpretation of geotechnical-engineering reports has resulted in costly problems. Confront that risk by having your geotechnical engineer confer with appropriate members of the design team after submitting the report. Also retain your geotechnical engineer to review pertinent elements of the design team's plans and specifications. Constructors can also misinterpret a geotechnical-engineering report. Confront that risk by having your geotechnical engineer participate in prebid and preconstruction conferences, and by providing geotechnical construction observation.

Do Not Redraw the Engineer's Logs

Geotechnical engineers prepare final boring and testing logs based upon their interpretation of field logs and laboratory data. To prevent errors or omissions, the logs included in a geotechnical-engineering report should *never* be redrawn for inclusion in architectural or other design drawings. Only photographic or electronic reproduction is acceptable, *but recognize that separating logs from the report can elevate risk.*

Give Constructors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can make constructors liable for unanticipated subsurface conditions by limiting what they provide for bid preparation. To help prevent costly problems, give constructors the complete geotechnical-engineering report, *but* preface it with a clearly written letter of transmittal. In that letter, advise constructors that the report was not prepared for purposes of bid development and that the report's accuracy is limited; encourage them to confer with the geotechnical engineer who prepared the report (a modest fee may be required) and/or to conduct additional study to obtain the specific types of information they need or prefer. A prebid conference can also be valuable. *Be sure constructors have sufficient time* to perform additional study. Only then might you be in a position to give constructors the best information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions.

Read Responsibility Provisions Closely

Some clients, design professionals, and constructors fail to recognize that geotechnical engineering is far less exact than other engineering disciplines. This lack of understanding has created unrealistic expectations that have led to disappointments, claims, and disputes. To help reduce the risk of such outcomes, geotechnical engineers commonly include a variety of explanatory provisions in their reports. Sometimes labeled "limitations," many of these provisions indicate where geotechnical engineers' responsibilities begin and end, to help others recognize their own responsibilities and risks. *Read these provisions closely.* Ask questions. Your geotechnical engineer should respond fully and frankly.

Environmental Concerns Are Not Covered

The equipment, techniques, and personnel used to perform an *environmental* study differ significantly from those used to perform a *geotechnical* study. For that reason, a geotechnical-engineering report does not usually relate any environmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated environmental problems have led to numerous project failures.* If you have not yet obtained your own environmental information, ask your geotechnical consultant for risk-management guidance. *Do not rely on an environmental report prepared for someone else.*

Obtain Professional Assistance To Deal with Mold

Diverse strategies can be applied during building design, construction, operation, and maintenance to prevent significant amounts of mold from growing on indoor surfaces. To be effective, all such strategies should be devised for the *express purpose* of mold prevention, integrated into a comprehensive plan, and executed with diligent oversight by a professional mold-prevention consultant. Because just a small amount of water or moisture can lead to the development of severe mold infestations, many mold- prevention strategies focus on keeping building surfaces dry. While groundwater, water infiltration, and similar issues may have been addressed as part of the geotechnical- engineering study whose findings are conveyed in this report, the geotechnical engineer in charge of this project is not a mold prevention consultant; *none of the services performed in connection with the geotechnical engineer's study were designed or conducted for the purpose of mold prevention. Proper implementation of the recommendations conveyed in this report will not of itself be sufficient to prevent mold from growing in or on the structure involved.*

Rely, on Your GBC-Member Geotechnical Engineer for Additional Assistance

Membership in the Geotechnical Business Council of the Geoprofessional Business Association exposes geotechnical engineers to a wide array of risk-confrontation techniques that can be of genuine benefit for everyone involved with a construction project. Confer with you GBC-Member geotechnical engineer for more information.

GEOTECHNICAL
BUSINESS COUNCIL
of the Geoprofessional Business Association

8811 Colesville Road/Suite G106, Silver Spring, MD 20910
Telephone: 301/565-2733 Facsimile: 301/589-2017
e-mail: info@geoprofessional.org www.geoprofessional.org