Pole Line Road/Olive Drive Connection
Davis, California
CAInc File No. 18-438.1

Prepared by:

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February 14, 2020

Prepared for:

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February 14, 2020  
CAInc File No. 18-438.1

Mr. Dennis Pecchia  
Wood Rodgers, Inc.  
3301 C Street, Bldg. 100-B  
Sacramento, CA 95816

Subject:  DRAFT FOUNDATION REPORT  
Pole Line Road/Olive Drive Connection  
Davis, California

Dear Mr. Pecchia,

Crawford & Associates, Inc. (CAInc) prepared this DRAFT Foundation Report for the Pole Line Road/Olive Drive Bike and Pedestrian Connection Project located in Davis, California. CAInc prepared this report in accordance with our August 7, 2017 scope of services and agreement. We will issue a Final Foundation Report once we review the Caltrans and your comments on this draft report.

Thank you for the opportunity to be part of your design team. Please call if you have questions or require additional information.

Sincerely,

Crawford & Associates, Inc.,

Hailey Wagenman  
Project Engineer

Benjamin D. Crawford, PE, GE  
Principal Geotechnical Engineer
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INTRODUCTION

1.1 PURPOSE

Crawford & Associates, Inc. (CAInc) prepared this DRAFT Foundation Report for the Pole Line Road/Olive Drive Bike and Pedestrian Connection Project in Davis, California. This report presents the results of our subsurface exploration and testing, engineering analysis, conclusions and recommendations for use in design for new bridge foundations and associated grading. CAInc will prepare a Final Foundation Report after receiving comments on this draft report.

1.2 SCOPE OF SERVICES

To prepare this report, CAInc:

- Discussed the project with Dennis Pecchia and Gerard Murdock from Wood Rogers, Inc. (WR),
- Reviewed the “General Plan No. 1” and “Column & Footing Details” As-Built plan sheets for the Pole Line Road Overcrossing project prepared by Greiner, Inc., dated January 26, 1995,
- Reviewed “Log of Test Borings No. 1, No. 2 and No. 3” As-Built plan sheets for the Pole Line Road Overcrossing project prepared by Taber Consultants, dated January 26, 1995 and,
- Reviewed the Olive Drive Bike Path/ Poleline Rd OC Ramp Connection General Plan prepared by WR, dated January 2020,
- Reviewed available published geologic and seismic mapping of the site,
- Performed subsurface exploration between May 3-4, 2018,
- Performed laboratory testing on soil samples obtained during our subsurface exploration, and
- Perfomed geotechnical engineering evaluation and analysis.

Limitations of this study are discussed in the final section of this report.

1.3 PROJECT DATUM

All elevations referenced within this report are based on the topographic survey completed by Radman Aerial Surveys, dated June 2017.

1.4 SITE DESCRIPTION

The project is located along the Olive Drive Trail located at the east end of Olive Drive in Davis, California. The proposed structure will follow the same general alignment as Olive Drive Trail and will connect to the Pole Line Road Overcrossing. Olive Drive Trail is an existing paved bike path that crosses under the Pole Line Road Overcrossing. Westbound I-80 is located directly south of the project site, and Union Pacific Railroad train tracks and 2nd street are located directly north of the project site. Site coordinates are approximately latitude 38.54593°N and longitude 121.72686°W. The site location is shown on Figure 1.

The site is within gently-rolling topography at about elevation 40 feet. Currently nearby land use is primarily commercial and residential. The bike path is primarily used for pedestrians and bikes.
1.5 PROJECT DESCRIPTION

Based on the most recent General Plan provided by WR, dated January 2020, the project will consist of a new ramp bridge structure that is about 542 ft long and 14 ft wide. The new ramp structure will require an approach embankment about 1133 ft in long near the connection point to the Olive Drive Trail. The bridge and embankment will slope down from the Pole Line Overcrossing towards Olive Drive Trail at a slope of 8%. Abutment 1 is currently planned to be supported by ten 24-inch CIDH piles, and Bents 2 through 5 are currently planned to be supported by a single, 72-inch diameter CIDH pile at each bent location.

2 AS-BUILT FOUNDATION DATA FOR POLE LINE ROAD OVERCROSSING

The Pole Line Road Overcrossing (Bridge No. 22-0193) was originally constructed in 1996 and is a seven-span-bridge that is 810±ft long. The bridge is 87±ft wide for spans 1 and 2, and 78±ft for spans 3 through 7. The existing bridge deck is depicted at elev. 53.34±ft at Begin Bridge (“BB”, Station line “A” 15+33.93) and at elev. 66.30±ft at the End Bridge (“EB”, Station line “A” 21+44.00).

The substructure of the bridge includes abutments with wingwalls, and six reinforced concrete column-bents. All supports are skewed 12±degrees from the normal line to the centerline of the bridge. Bent 5 support is the closest existing bridge support to our project area. The “General Plan No. 1” and “Column & Footing Details” As-Built plan sheets show the below information in Table 1 for Bent 5 support.

<table>
<thead>
<tr>
<th>Table 1: As-Built Elevation Data</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bridge Supports</td>
</tr>
<tr>
<td>Bent 5</td>
</tr>
</tbody>
</table>

The As-Built plan sheets do not specify whether the pile types are drilled (i.e. CIDH) or driven (i.e. precast, prestressed concrete). However, the “Column & Footing Details” As-Built plan sheet shows a pile was broken during driving on Bent 3, therefore the piles were most likely precast, prestressed concrete piles.

We show select As-Built plan sheets in Appendix A.

3 GEOLOGIC SETTING

The project is located within the Great Valley geomorphic province of California, at the very southwestern edge of the Sacramento Valley. Published geologic mapping1 indicates the immediate project vicinity is underlain by Holocene age fan levee deposits (described as natural levees deposited as long, low ridges oriented down-fan, containing coarser material than the adjoining interlevee areas). The fan levee deposits are surrounded by Holocene age alluvial fan deposits that are described as alluvial fan sediment deposited by streams emanating from mountains a debris flows, hyper-concentrated mudflows, or braided steam flows. Sediments for levee deposits and alluvial fan deposits

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1 Gutierrez, C.I., 2011, Preliminary Geologic Map of the Sacramento 30’ x 60’ Quadrangle, California: California Geological Survey, 1:100,000 scale.
include sand, gravel, silt and clay that are moderately to poorly sorted, and moderately to poorly bedded. We show a Geologic Map as Figure 3.

### 4 SUBSURFACE EXPLORATION

#### 4.1 FIELD EXPLORATION

CAInc retained Geo-Ex Subsurface Exploration (Geo-Ex) to drill and sample a total of two test borings on May 3-4, 2018. Boring depths ranged from 81.5 to 85.5 feet below ground surface (bgs).

Geo-Ex used a CME 55 truck-mounted drill rig to complete the borings with 4-inch solid-stem auger drilling equipment and 4-inch rotary wash techniques. Soil samples were recovered by means of a 2.0-inch O.D. “Standard Penetration” (SPT) split-spoon sampler with liners and a 3.0-inch O.D. “Standard California” split-spoon sampler with liners. Both samplers were advanced with standard 350 ft-lb striking force using an auto-hammer. An energy hammer analysis was not performed specific to this project/site, however Geo-Ex reports an efficiency of 77.3%. The field recorded (uncorrected) blow counts are shown on the “Log of Test Borings” (LOTB), provided in Appendix A.

CAInc’s Project Engineers, Hailey Wagenman and Gabriela Lopez, logged the test borings consistent with the Unified Soil Classification System (USCS) and the Caltrans 2010 Logging Manual. Consistency of cohesive soils were obtained in the field by means of pocket penetrometer. Selected portions of recovered soil drive samples were retained in sealed containers for laboratory testing and reference. Bulk soil samples were retained in sealed bags for laboratory testing and reference. Groundwater observations were recorded during drilling operations when encountered. At completion, test borings were backfilled with lean cement grout per the county boring permit requirements.

Taber Consultants (acquired by CAInc in 2016) completed borings B-1, B-2, B-4, B-5 and B-7 in 1990, and B-1(93) and B-2(93) in 1993 for the Pole Line Road Overcrossing Bridge Project. They also completed CPT locations B-3, B-6 and B-8 in 1990. Taber’s boring B-5 was located near proposed Bent 5 support location.

The boring locations were measured in the field with respect to existing site features and then referenced to project stationing. The boring elevations are referenced to project datum provided by Wood Rodgers, Inc. The details and locations of test borings are shown on the “Log of Test Borings” (LOTB) drawing, provided in Appendix A.

#### 4.2 SUBSURFACE CONDITIONS

The soils encountered CAInc R-18-001 and R-18-002 are generally soft to hard lean clay to sandy lean clay with localized layers of medium dense clayey sand. The soils encountered in Taber B-5 are generally soft to stiff lean clay, silt, clayey silt to silty clay with interbedded layers of dense silty-sand and sandy silts. We interpret the encountered materials as consistent with Holocene age fan levee deposits identified in the geologic mapping.

Taber’s LOTBs are attached as Appendix A.
4.3 GROUNDWATER

During the Taber field exploration, Taber Borings B-1(1990), B-2(1990), B-4(1990), B-1(1993) and B-2 (1993) encountered groundwater at elevations varying from 1 to 10 ft. We encountered groundwater during our fieldwork at elevations ranging from 16-20 ft. We present the encountered groundwater depths in in Table 2 below.

Table 2: Groundwater Data

<table>
<thead>
<tr>
<th>Well Location/ Boring ID</th>
<th>Date</th>
<th>Groundwater Surface in Feet (Elevation)</th>
</tr>
</thead>
<tbody>
<tr>
<td>CAInc R-18-001</td>
<td>5/3/18</td>
<td>16.9</td>
</tr>
<tr>
<td>CAInc R-18-002</td>
<td>5/4/18</td>
<td>20.9</td>
</tr>
<tr>
<td>Taber B1</td>
<td>1/4/90</td>
<td>9.7</td>
</tr>
<tr>
<td>Taber B2</td>
<td>1/5/90</td>
<td>6.8</td>
</tr>
<tr>
<td>Taber B4</td>
<td>1/19/90</td>
<td>5.3</td>
</tr>
<tr>
<td>Taber B-1 (93)</td>
<td>9/21/93</td>
<td>1.6</td>
</tr>
<tr>
<td>Taber B-2 (93)</td>
<td>9/22/93</td>
<td>1.8</td>
</tr>
</tbody>
</table>

Groundwater depth was not shown for Taber Boring B-5 on the Taber LOTBs.

5 LABORATORY TESTING

CAInc completed the following laboratory tests on representative soil samples from the borings:

- Moisture Content/Unit Weight (ASTM D2216/2937)
- No. 200 Sieve (ASTM D1140)
- Atterberg Limits (ASTM D4318)
- Unconfined Compression (ASTM D2166)
- Consolidation (ASTM D2435)
- Sulfate/Chloride Content (CTM 417/422)
- pH/Minimum Resistivity (CTM 643)

CAInc laboratory test results are provided in Appendix B. Laboratory testing performed by Taber Consultants is shown on the “Log of Test Borings No. 1, No.2 and No.3” As-Built plan sheets, provided in Appendix A.

6 SCOUR CONSIDERATIONS

There are no scour considerations for the Pole Line Road/Olive Drive Bike and Pedestrian Connection Bridge due to the lack of surface water.
7 CORROSION EVALUATION

Based on the test results summarized below in Table 3 and current Caltrans guidelines, the site is considered non-corrosive to structural concrete/steel foundation elements. The test results are only an indicator of soil corrosivity. Section 12 of Caltrans’ March 2018 Corrosion Guidelines (Version 3.0) provides information regarding corrosion mitigation measures for structural elements and lists additional Caltrans guideline documents regarding corrosion mitigation.

Table 3: Soil Corrosivity Test Results

<table>
<thead>
<tr>
<th>Boring / Sample No.</th>
<th>Depth (ft)</th>
<th>Elevation (ft)</th>
<th>pH</th>
<th>Minimum Resistivity (ohm-cm)</th>
<th>Chloride (ppm)</th>
<th>Sulfate (ppm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>R-18-001-9A</td>
<td>41.5</td>
<td>-1.5</td>
<td>7.62</td>
<td>1,142</td>
<td>1.7</td>
<td>32.9</td>
</tr>
<tr>
<td>R-18-002-5A</td>
<td>21.5</td>
<td>18.8</td>
<td>7.69</td>
<td>1,100</td>
<td>0.4</td>
<td>7.9</td>
</tr>
</tbody>
</table>

Caltrans currently defines a corrosive environment as an area where the soil has either a chloride concentration of 500 ppm or greater, a sulfate concentration of 1500 ppm or greater, or has a pH of 5.5 or less.

8 SEISMIC DATA AND EVALUATION

8.1 SURFACE FAULT RUPTURE

The site does not lie within an Alquist–Priolo Earthquake Fault Zone and no known active faults are mapped within or through the project area. The California Geologic Survey\(^2\) (CGS) considers a fault to be active if it has shown evidence of surface displacement within Holocene time (about the last 11,000 years). According to the United States Geologic Survey (USGS)\(^3\), the closest active fault is the Dunnigan Hills Fault at about 16.4± miles to the northwest of the site. Based on this mapping, we consider the potential for fault rupture at the site to be low.

We show nearby faults in Figure 2.

8.2 SEISMIC GROUND MOTIONS

CAInc used the Caltrans ARS Online (web-based) tool (V2.3.09)\(^4\) to calculate deterministic and probabilistic acceleration response spectra for the site based on criteria provided in Appendix B of the April 2013 Caltrans’ Seismic Design Criteria (SDC) Version 1.7.

The deterministic spectrum is defined as the average of median response spectra calculated using ground motion prediction equations developed under the “Next Generation Attenuation” (NGA) project.


\(^3\) [https://earthquake.usgs.gov/hazards/qfaults/](https://earthquake.usgs.gov/hazards/qfaults/)

These equations are applied to all faults considered active in the last 750,000 years (late-Quaternary age) that are capable of producing a moment magnitude earthquake of 6.0 or greater.

Based on Caltrans ARS Online (v2.3.09) and the 2012 Caltrans Fault Database v2b, the nearest deterministic seismic sources are the Great Valley 03a Dunnigan Hills, Great Valley 03 Mysterious Ridge, and the Great Valley 04b Gordon Valley, assigned the parameters shown in Table 4.

Table 4: Fault Data

<table>
<thead>
<tr>
<th>Fault Parameters</th>
<th>Great Valley 03a Dunnigan Hills</th>
<th>Great Valley 03 Mysterious Ridge</th>
<th>Great Valley 04b Gordon Valley</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fault Identification Number (FID)</td>
<td>95</td>
<td>82</td>
<td>104</td>
</tr>
<tr>
<td>Maximum Moment Magnitude (M_max)</td>
<td>6.4</td>
<td>7.0</td>
<td>6.7</td>
</tr>
<tr>
<td>Site-to-Fault (Rrup) Distance (km/mi)</td>
<td>13.3/8.3</td>
<td>33.4/20.8</td>
<td>29.3/18.2</td>
</tr>
<tr>
<td>Style of Faulting</td>
<td>Reverse</td>
<td>Reverse</td>
<td>Reverse</td>
</tr>
<tr>
<td>Fault Dip (degrees)</td>
<td>20</td>
<td>20</td>
<td>20</td>
</tr>
<tr>
<td>Dip Direction</td>
<td>East</td>
<td>West</td>
<td>West</td>
</tr>
</tbody>
</table>

The deterministic response spectra for the controlling seismic sources identified above was compared to the Caltrans minimum deterministic response spectrum that assumes a maximum moment magnitude 6.5, reverse slip event occurring at a distance of 8.3 miles. We also compared the deterministic results with the Caltrans probabilistic response spectrum based on data from the 2008 United States Geological Survey (USGS) National Seismic Hazard Map for a 5% in 50 year probability of exceedance (975 year return period).

Caltrans structure design practice also requires an increase to spectra due to fault proximity (near-fault factor) and when the site is located over a deep sedimentary basin (basin factor). The near-fault adjustment factor is applied for locations with a site-to-rupture plane distance (Rrup) of 25 km (15.5 miles) or less to the causative fault. The near-fault factor increase applies to this site. The basin factor increase does not apply to this site.

Based on our boring data and the above information, we recommend that structure design be based on the following Caltrans SDC parameters:
- Shear Wave Velocity, Vs30: 220 meters per second (879 ft per second);
- Soil Profile Type D;
- Maximum Moment Magnitude (M_max): 6.4;
- Peak Ground Acceleration (PGA): 0.36 g; and

We include the “ARS Curve” as Figure 4.
8.3 LIQUEFACTION EVALUATION

Liquefaction is a secondary effect associated with seismic loading. It can occur when very loose to medium dense, granular, saturated soils (generally within 50 feet of the surface), or specifically defined cohesive soils, are subjected to ground shaking. Based on the soil encountered in the field, the potential for liquefaction at this site is considered low.

8.4 SEISMICALLY INDUCED SETTLEMENT

During a seismic event, ground shaking can cause densification of granular soil above the water table that can result in settlement of the ground surface. Based on the soils encountered in CAInc borings R-18-001 and R-18-002, and Taber B-5, the potential for seismically induced ground settlement at this site is considered low.

9 CONCLUSIONS

The site is considered adequately stable with support available for proposed CIDH bridge foundations within the Holocene alluvial fan sediment deposits. Driven piles are not considered suitable due to the close proximity to existing Pole Line Road Bridge and rail road structures. Spread footings are not considered suitable due to the lack of space for excavation. No other over-riding geologic hazards (e.g. faulting, volcanoes, settlement, landslides, etc.) were identified in either published geologic mapping or field exploration/reconnaissance performed for this study. We provide the results of our analysis below.

9.1 SOIL PARAMETERS

Soil profiles were developed for this project based on the subsurface data provided in CAInc borings R-18-001 and R-18-002, and Taber boring B-5. The generalized engineering parameters for this project are based on the following:

- Average unit weights based on laboratory test results.
- Average cohesion values based on unconfined compressive strength.
- Friction angles based on published blow count correlations.
- Engineering experience and judgment.
- A groundwater elevation of 16 ft obtained from CAInc boring R-18-001.

The generalized soil parameters used in our analysis are shown in Tables 5, 6 and 7.

<table>
<thead>
<tr>
<th>Elevation (feet)</th>
<th>Soil Description</th>
<th>Total Unit Weight (lb/ft³)</th>
<th>Friction Angle, Phi (degrees)</th>
<th>Cohesion, c (psf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>36 to 19</td>
<td>Lean Clay</td>
<td>118</td>
<td>0</td>
<td>750</td>
</tr>
<tr>
<td>19 to -30</td>
<td>Lean Clay</td>
<td>130</td>
<td>0</td>
<td>2500</td>
</tr>
<tr>
<td>-30 to -45</td>
<td>Lean Clay</td>
<td>124</td>
<td>0</td>
<td>1000</td>
</tr>
</tbody>
</table>
### Table 6: Generalized Soil Parameters based on CAInc R-18-002

<table>
<thead>
<tr>
<th>Elevation (feet)</th>
<th>Soil Description</th>
<th>Total Unit Weight (lb/ft$^3$)</th>
<th>Friction Angle, Phi (degrees)</th>
<th>Cohesion, c (psf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>40 to 25</td>
<td>Lean Clay</td>
<td>118</td>
<td>0</td>
<td>1250</td>
</tr>
<tr>
<td>25 to 10</td>
<td>Lean Clay</td>
<td>134</td>
<td>0</td>
<td>3250</td>
</tr>
<tr>
<td>10 to -15</td>
<td>Lean Clay</td>
<td>124</td>
<td>0</td>
<td>1500</td>
</tr>
<tr>
<td>-15 to -35</td>
<td>Lean Clay</td>
<td>122</td>
<td>0</td>
<td>2000</td>
</tr>
<tr>
<td>-35 to -45</td>
<td>Lean Clay</td>
<td>122</td>
<td>0</td>
<td>750</td>
</tr>
</tbody>
</table>

### Table 7: Generalized Soil Parameters based on Taber B5

<table>
<thead>
<tr>
<th>Elevation (feet)</th>
<th>Soil Description</th>
<th>Total Unit Weight (lb/ft$^3$)</th>
<th>Friction Angle, Phi (degrees)</th>
<th>Cohesion, c (psf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>40.5 to 39</td>
<td>Poorly Graded Sand</td>
<td>123</td>
<td>35</td>
<td>0</td>
</tr>
<tr>
<td>39 to 34</td>
<td>Lean Clay</td>
<td>124</td>
<td>0</td>
<td>400</td>
</tr>
<tr>
<td>34 to 11.5</td>
<td>Lean Clay</td>
<td>133</td>
<td>0</td>
<td>3,600</td>
</tr>
<tr>
<td>11.5 to 0</td>
<td>Silt</td>
<td>129</td>
<td>0</td>
<td>1,700</td>
</tr>
<tr>
<td>0 to -6</td>
<td>Poorly Graded Sand with Silt</td>
<td>134</td>
<td>35</td>
<td>0</td>
</tr>
<tr>
<td>-6 to -36</td>
<td>Lean Clay</td>
<td>130</td>
<td>0</td>
<td>2,900</td>
</tr>
<tr>
<td>-36 to -40</td>
<td>Silt</td>
<td>129</td>
<td>31</td>
<td>0</td>
</tr>
</tbody>
</table>

### 9.2 FOUNDATION DATA AND LOADING

Foundation information and Pile Foundation Design Loads provided by WR are shown in Table 8 and Table 9 below.

Abutment 1, Bent 2, Bent 3, Bent 4 and Bent 5 are all being modeled with 17 feet of permanent casing. The permanent casing was modeled with grout backfill in the annular spacing between the casing and soil; therefore, the pile was modeled with 50% capacity in this section.
Table 8: Foundation Design Data Sheet

<table>
<thead>
<tr>
<th>Support No.</th>
<th>Design Method</th>
<th>Pile Type</th>
<th>Finished Grade Elevation(ft)</th>
<th>Cut-off Elevation (ft)</th>
<th>Pile Cap Size (ft)</th>
<th>Permissible Settlement – Service Load (in)¹</th>
<th>Number of Piles per Support</th>
</tr>
</thead>
<tbody>
<tr>
<td>Abut 1</td>
<td>LRFD</td>
<td>24” CIDH</td>
<td>…²</td>
<td>36.11</td>
<td>B 9 L 24</td>
<td>1</td>
<td>10</td>
</tr>
<tr>
<td>Bent 2</td>
<td>LRFD</td>
<td>72” CIDH</td>
<td>…²</td>
<td>17.97</td>
<td>N/A N/A</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>Bent 3</td>
<td>LRFD</td>
<td>72” CIDH</td>
<td>…²</td>
<td>25.40</td>
<td>N/A N/A</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>Bent 4</td>
<td>LRFD</td>
<td>72” CIDH</td>
<td>…²</td>
<td>32.55</td>
<td>N/A N/A</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>Bent 5</td>
<td>LRFD</td>
<td>72” CIDH</td>
<td>…²</td>
<td>39.69</td>
<td>N/A N/A</td>
<td>1</td>
<td>1</td>
</tr>
</tbody>
</table>

¹ Based on CALTRANS’ current practice, the total permissible settlement is one inch for multi-span structures with continuous spans or multi-column bents, one inch for single span structures with diaphragm abutments, and two inches for single span structures with seat type abutments. Different permissible settlement under service loads may be allowed if a structural analysis verifies that required level of serviceability is met.

² Finished Grade Elevations were not available at the time of this report.

Table 9: Foundation Design Loads

<table>
<thead>
<tr>
<th>Support No.</th>
<th>Service-L Limit State (kips)</th>
<th>Strength/Construction Limit State (Controlling Group, kips) (Does not include φ)</th>
<th>Extreme Limit State (Controlling Group, kips) (Does not include φ)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Total Load Per Support</td>
<td>Permanent Loads Compression Per Support Max. Per Pile</td>
<td>Tension Per Support Max. Per Pile</td>
</tr>
<tr>
<td></td>
<td>Max. Per Pile</td>
<td>Per Support</td>
<td>Compression Per Support Max. Per Pile</td>
</tr>
<tr>
<td>Abut 1</td>
<td>189</td>
<td>65</td>
<td>132</td>
</tr>
<tr>
<td>Bent 2</td>
<td>553</td>
<td>553</td>
<td>437</td>
</tr>
<tr>
<td>Bent 3</td>
<td>579</td>
<td>579</td>
<td>465</td>
</tr>
<tr>
<td>Bent 4</td>
<td>633</td>
<td>633</td>
<td>504</td>
</tr>
<tr>
<td>Bent 5</td>
<td>388</td>
<td>388</td>
<td>311</td>
</tr>
</tbody>
</table>
10 FOUNDATION RECOMMENDATIONS

10.1 ABUTMENT AND BENT FOUNDATION DESIGN RECOMMENDATIONS

The foundation design recommendations for 24-inch-diameter CIDH piles at the abutment and 72-inch diameter CIDH piles at the bents are summarized in Table 10 and Table 11 below.

Table 10: Abutment and Bents Foundation Design Recommendations

<table>
<thead>
<tr>
<th>Support Location</th>
<th>Pile Type</th>
<th>Cut-off Elev. (ft)</th>
<th>Service-I Limit State Load Per Support (kips)</th>
<th>Nominal Resistance$^1$ (kips)</th>
<th>Design Tip Elev. $^3$ (ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Total Permissible Support Settlement (inches)</td>
<td>Strength/Const.</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Comp. $\varphi_{qs} = 0.7$</td>
<td>Tens. $\varphi_{qs} = 0.7$</td>
</tr>
<tr>
<td>Abut 1</td>
<td>24” CIDH</td>
<td>36.11</td>
<td>189</td>
<td>132</td>
<td>1</td>
</tr>
<tr>
<td>Bent 2</td>
<td>72” CIDH</td>
<td>17.97</td>
<td>553</td>
<td>437</td>
<td>1</td>
</tr>
<tr>
<td>Bent 3</td>
<td>72” CIDH</td>
<td>25.40</td>
<td>579</td>
<td>465</td>
<td>1</td>
</tr>
<tr>
<td>Bent 4</td>
<td>72” CIDH</td>
<td>32.55</td>
<td>633</td>
<td>504</td>
<td>1</td>
</tr>
<tr>
<td>Bent 5</td>
<td>72” CIDH</td>
<td>39.69</td>
<td>388</td>
<td>311</td>
<td>1</td>
</tr>
</tbody>
</table>

Notes:
1) Design tip elevations are controlled by: (a-I) Compression (Strength Limit), (b-I) Tension (Strength Limit), (a-II) Compression (Extreme Event), (b-II) Tension (Extreme Event), (c) Settlement, (d) Lateral Load.
2) The Specified Tip Elevation shall not be raised above the design tip elevation.
3) The Lateral Load will be determined by WR.
## 10.2 PILE DATA TABLE

### Table 11: Pile Data Table

<table>
<thead>
<tr>
<th>Location</th>
<th>Pile Type</th>
<th>Nominal Resistance (kips)</th>
<th>Design Tip Elevations (^3)</th>
<th>Specified Tip Elevations (^4)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Compression</td>
<td>Tension</td>
<td>(ft)</td>
</tr>
<tr>
<td>Abut 1</td>
<td>24” CIDH</td>
<td>100</td>
<td>0</td>
<td>10 (a)</td>
</tr>
<tr>
<td>Bent 2</td>
<td>72” CIDH</td>
<td>1070</td>
<td>0</td>
<td>-30 (a)</td>
</tr>
<tr>
<td>Bent 3</td>
<td>72” CIDH</td>
<td>1120</td>
<td>0</td>
<td>-25 (a)</td>
</tr>
<tr>
<td>Bent 4</td>
<td>72” CIDH</td>
<td>1230</td>
<td>0</td>
<td>-37 (a)</td>
</tr>
<tr>
<td>Bent 5</td>
<td>72” CIDH</td>
<td>750</td>
<td>0</td>
<td>-12 (a)</td>
</tr>
</tbody>
</table>

Notes:
1) Design tip elevations are controlled by: (a) Compression and (d) Lateral Load, respectively.
2) The Specified Tip Elevation shall not be raised above the design tip elevation.
3) The Lateral Load will be determined by WR.
4) Specified Tip Elevations will be determined by WR after completing Lateral Load analysis.

## 10.3 CAST-IN-DRILLED- HOLE PILES

### 10.3.1 COMPRESSIVE/TENSION RESISTANCE AND SETTLEMENT

The side (compressive) resistance for the CIDH pile foundations was evaluated using Load and Resistance Factor Design (LRFD) methods and factors from AASHTO LRFD Bridge Design Specifications (6th Edition) with Caltrans amendments. End bearing contributions to axial resistance were neglected, consistent with current Caltrans guidelines for CIDH pile design. Skin friction contributions are only considered in our compressive resistance analysis. Where permanent casing is shown 50% of the capacity was utilized in determining the skin friction of the pile. Actual contributions to skin friction vary depending on load transfer along the pile shaft.

A geotechnical resistance factor (\(\phi\)) of 0.7 was used to evaluate compressive resistance at the Strength Limit State consistent with Load and Resistance Factor Design (LRFD) method for the abutment piles. The required factored nominal resistance was determined by comparing the highest Factored Strength Limit Load (\(\phi = 0.7\)) with the highest Extreme Event Load (\(\phi = 1.0\)). The higher value was then used as the required factored nominal resistance. The Controlling Strength Limit State for the 24-inch CIDH piles for Abutment 1 is 100 kips. The Controlling Strength Limit State for the 72-inch CIDH piles (1070 kips, 1120 kips, 1230 kips and 750 kips) control at Bent 2, Bent 3, Bent 4 and Bent 5, respectively.

Consistent with AASHTO LRFD BDS 10.8.3.5.1b and C10.8.3.4.1b (Side Resistance), the top 5 ft of the pile and the bottom length of pile equivalent to the shaft diameter are excluded from contributing to geotechnical capacity.
For individual pile analysis, an immediate settlement (including axial pile compression) of less than 1-inch was calculated. No significant long-term pile settlement is anticipated at this site.

We present the CIDH Pile Nominal Resistance in Appendix C.

10.3.2 LATERAL RESISTANCE

The LPILE parameters for use in lateral pile capacity analysis are provided in Appendix D. WR will be performing their own lateral pile design for this project.

10.3.3 NEGATIVE SKIN FRICTION

We do not anticipate negative skin friction at the abutment or bents.

10.3.4 PILE GROUP REDUCTION

The foundation soils predominately consist of stiff cohesive and medium dense and dense/very dense granular layers. The pile center-to-center spacing is greater than 2.5 times the pile diameter and there are no scour considerations for the structure.

According to the AASHTO LRFD BDS 6th Edition (Section10.7.3.9 – Resistance of Pile Groups in Compression) with California amendments:

- For Cohesive Soil - If the cap is in firm contact with the ground, no reduction in efficiency shall be required. If the cap is not in firm contact with the ground and if the soil is stiff, no reduction in efficiency shall be required.

- For Granular Soil - The efficiency factor, $\eta$, shall be 1.0 where the pile cap is or is not in contact with the ground for a center to center pile spacing of 2.5 diameters or greater.

Therefore, a group efficiency factor ($\eta$) of 1.0 was used in our compressive resistance pile analysis.

10.3.5 CONSTRUCTION CONSIDERATIONS - CIDH

Construct CIDH piles in conformance with Section 49-3 of the 2015 Caltrans Standard Specifications, Revised Standard Specifications, and Standard Special Provisions. If groundwater is anticipated within foundation depths, we recommend that the CIDH piles be installed by the “wet” method, including slurry drilling and concrete placed under slurry using tremie pipe to avoid construction delays should groundwater be present during construction. The slurry construction method (“wet” method) requires placement of inspection tubes to permit Gamma-Gamma Logging (GGL) and Cross-hole Sonic Logging (CSL) of the CIDH pile.

The contractor is responsible for the design and installation of temporary casing, including actual length(s) and diameter(s), to install CIDH piles according to the above specifications without defects or damage to existing utilities/ facilities. Temporary casing (if used), must be noncorrugated steel with smooth surfaces and the casing diameter should be at least 8-inches greater than the CIDH pile to help prevent binding of the drilling tool. Installing temporary casing below the specified pile tip elevation is not permissible.
The project specifications should explicitly exclude vibration and impact installation methods if noise or vibrations are of concern or otherwise not allowed due to environmental constraints, proximity of nearby residences or to protect the existing railroad tracks and other facilities (e.g., underground utilities potentially susceptible to vibration damage).

For a Type II Shaft, the bottom of the permanent casing must extend to 5 ft below the construction joint. The permanent steel casing must be placed in drilled hole, and annular space backfilled with grout (SS 49-3.02B(5), 49-3.02C(6). It is also permissible to drill/oscillate/rotate the permanent casing into place. Installation by driving or vibration is not permissible.

If an oscillator or rotator is used to construct the CIDH piles, the following is required:

- The contractor should be prepared for subsurface soil conditions that include layers of soft to hard cohesive soils and medium dense sandy layers.
- The contractor must maintain a positive fluid head within the drill road at all times. The fluid must be mineral or polymer slurry; water is not permitted.
- The contractor is to maintain a minimum 10 foot soil plug within the drill rod. The 10 foot plug is to be maintained until the drill rod reaches the specified tip elevation. At no time is the contractor to have less than the minimum 10 foot soil plug until the specified tip elevation has been reached.
- The contractor must provide access to the top of the oscillator/rotator drill rod, as requested by the Engineer, to verify the positive head and minimum soil plug are being maintained.

Prior to mobilization to the site, the foundation contractor should prepare and submit a detailed work plan for the engineer’s review and approval. The work plan should state explicitly all assumptions the contractor has made regarding earth materials and foundation construction conditions. The work plan should include details of proposed equipment, personnel, materials, methods and order or work.

The plans show pea gravel and filter fabric being utilized around the column between the finished grade and top of the footing with a 2-foot minimum. Pea gravel has the potential to settle which may cause future maintenance considerations. Where spacing is planned to be greater than 5 feet isolator casing should be utilized.

11 EARTHWORK

Site grading and earthwork should be performed in accordance with Section 17 and Section 19 of Caltrans 2015 Standard Specifications, respectively.

11.1 FILL MATERIAL

Construct embankment and place new fill in accordance with Caltrans “Standard Specifications”, including at least 95% relative compaction (CTM 216) on all fill within 150 ft of bridge abutments. Any imported fill should be approved by the soils engineer, should have 100% passing 3-inch sieve and have low expansion potential [Expansion Index (EI) < 50 and Sand Equivalent (SE) > 20]. In general, all fill material should be free of debris and organic material.
11.2 SLOPE GEOMETRY AND STABILITY

The near-surface soils are capable of providing adequate support for proposed fill heights. We expect that new embankment constructed as above, and with exterior slopes at 2:1 (horizontal:vertical), or flatter, will be stable.

Based on our experience in Davis additional embankment fills can cause long term consolidation settlement on the order of 2 to 4 inches. Settlement monitoring during and after construction is recommended through established monuments. A minimum “waiting period” of 90-days between placing embankment fill and driving piling and/or placing wall backfill is recommended. The design team should consider placing approach slabs to minimize the eventual uneven surface that will develop between the abutment and the bridge.

12 CONSTRUCTION CONSIDERATIONS

This section is provided to help identify relevant Standard Specifications and subsurface conditions that may be encountered in the field during construction. For the project described herein, we recommend that the foundation report, LOTB, and any subsequent addenda be included with project documents during the bidding process for reference purposes.

12.1 EXISTING UTILITIES

We understand that an overhead utility line exists north of the existing bike path and runs from east to west. These utilities will most likely not adversely affect pile installation. Locations of any other utilities, if present, are unknown.

12.2 EXISTING FOUNDATIONS

The proposed foundation elements for the bridge are not expected to encounter existing foundations, however the existing Pole Line Overcrossing structure foundation should be protected during construction.

12.3 EXISTING CONCRETE SECTION

The existing bike path is underlain with a concrete section approximately 3 inches thick. Contractor should expect difficult drilling and/or excavation within the concrete section. The 3-inch concrete section must be removed 5 ft laterally around the proposed Abutment 1 pile cap and embankment area.

The existing concrete section may cause adverse draining conditions at the project site. The design team should consider if it is necessary to remove or punch holes through the concrete section to assist with drainage.

See Figure 1 for approximate location of the underlying concrete.

12.4 EXCAVATION AND SHORING

Based on the subsurface conditions at this site, we expect that excavation to the indicated pile cut-off depths can be achieved using typical heavy-duty construction equipment. Soils encountered in the
boring completed at this site are considered consistent with Cal/OSHA Type B soil classification. The CIDH pile excavations are expected to encounter groundwater within the depth of pile excavation. The contractor is responsible for design and construction of excavation sloping and shoring in accordance with Cal/OSHA requirements, including verifying soil type in open excavations, and to protect existing structures, utilities and other facilities during construction.

12.5 VIBRATION

Train tracks are directly north, the Poleline Road Overpass is directly east and Highway 80 is directly south of the proposed Olive Drive bike path. Vibration is expected to occur during construction. If impact or vibratory hammers are utilized for temporary casing the vibration can be calculated using the following. Vibration criteria is discussed in Chapter 6 of the “Transportation and Construction Vibration Guidance Manual,” dated September 2013, published by Caltrans Department of Transportation. According to AASHTO the maximum vibration level for preventing damage to engineered structures is between 1.0-1.5 (in/sec). Vibration amplitudes produced by vibratory pile drivers can be calculated using equation 10 in Section 7.1.2 of the manual.

\[ PPV_{\text{Vibratory Pile Driver}} = PPV_{\text{Ref}} \left( \frac{25}{D} \right)^n \text{ (in/sec)} \]

Where:

- \( PPV_{\text{Ref}} = 0.65 \) in/sec for a reference pile driver at 25 ft
- \( D \) = distance from pile driver to the receiver in ft
- \( n = 1.1 \) (the value related to the attenuation rate through ground)

If necessary vibration monitoring should be the responsibility of the contractor.

13 RISK MANAGEMENT

Our experience, and that of our profession, clearly indicates the risks of costly design, construction, and maintenance problems can be significantly lowered by retaining the Geotechnical Engineer of Record to provide additional services during design and construction. For this project, CAInc should be retained as the Geotechnical Engineer of Record to:

- Review and provide comments on the final plans and specifications, insofar as they rely upon this report, prior to construction bidding to verify consistency with the recommendations contained herein;
- Review pile installation during construction in order to confirm anticipated bearing materials and provide additional or alternate recommendations if necessary; and,
- Update this report if design changes occur, 2 years or more lapse between this report and construction, and/or site conditions have changed.

Should there be significant change in the project or should soil conditions different from those described in this report be encountered during construction, this office should be contacted/ notified for evaluation and supplemental recommendations as necessary or appropriate.

CAInc cannot be responsible for any other parties’ interpretation of our report and recommendations contained herein, as well as subsequent addendums, letters, and discussions. If others perform the construction observation, they should review this report and either accept the conclusions and recommendations herein as their own or provide alternative recommendations.
14 LIMITATIONS

The conclusions and recommendations of this study are professional opinion based upon the indicated project criteria and the limited data described herein. It is recognized there is potential for variation in subsurface conditions and modification of conclusions and recommendations might emerge from further, more detailed study. This report is intended only for the purpose, site location, and project description indicated and construction in accordance with Caltrans practice.

As changes in appropriate standards, site conditions and technical knowledge cannot be adequately predicted; review of recommendations by this office for use after a period of two years is a condition of this report.

A review by this office of any foundation and/or grading plans and specifications or other work product insofar as they rely upon or implement the content of this report, together with the opportunity to make supplemental recommendations as indicated therefrom is considered an integral part of this study and a condition of recommendations.

Subsequently defined construction observation procedures and/or agencies are an element of work, which may affect supplementary recommendations.

Opinions and recommendations apply to current site conditions and those reasonably foreseeable for the described development—which includes appropriate operation and maintenance thereof. They cannot apply to site changes occurring, made, or induced, of which this office is not aware and has not had opportunity to evaluate.

The scope of this study specifically excluded sampling and/or testing for, or evaluation of the occurrence and distribution of, hazardous substances. No opinion is intended regarding the presence or distribution of any hazardous substances at this or nearby site.
FIGURES

Figure 1: Vicinity and Exploration Location Map
Figure 2: Fault Activity Map
Figure 3: Geologic Map
Figure 4: Seismic Design Data
Pole Line/Olive Drive Connection
Davis, California

Reference:

LEGEND:
- Approximate end of concrete
- Concrete continues past this point

Taber-B5
approximate boring location
CAInc-A-18-001
approximate boring location

BEGIN APPROACH STRUCTURE 10+11.58
ELEV 40.49
BC 10+15.69

END APPROACH STRUCTURE = BB 11+45.50
ELEV 46.04

EXIST FENCE
EXIST DRAINAGE DITCH
EXIST BIKE TRIAL
EXIST OLIVE DRIVE OFF-RAMP

EXIST UP RR TRACKS
EXIST SEWER LINE

LEGEND:
Taber-B5
CAInc-R-18-001
CAInc-A-18-002

Davis, California

approximate boring location
approximate boring location

Figure 1
Pole Line/Olive Drive Connection
Davis, California

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Reference:
Pole Line/Olive Drive Connection
Davis, California

Figure 2
Fault Activity Map
Project No. 18-438.1
Scale 1"=35,000'
Date 03/16/18

Source: USGS Google Earth Fault Overlay Map

LEGEND

Quaternary Fault (Age)
- <150 years
- <15,000 years
- <130,000 years

Quaternary Fault (Age)
- <750,000 years
- <1.6 million years

Location
- Well Constrained
- Moderately Constrained
- Inferred

Project Mgr. JJW 03/16/18
Project Eng. HFW 03/16/18
Designer
Checked By JJW 04/03/18
Drawn By HFW 03/16/18

Project Location

Hunting Creek fault (Hunting Creek-Berryessa fault zone, Hunting Creek section)
Dunnigan Hills fault
West Napa fault (West Napa fault zone, Browns Valley Section)
Vaca fault
Soda Creek fault
Green Valley fault
Cordelia fault

Sacramento

North
**Figure 3**

Geologic Map

**Project Location**


- **Qa**: Quaternary Levee & Channel deposits
- **Qb**: Quaternary Basin Deposits (Alluvium)
- **Qmr**: Quaternary Modesto-Riverbank Formations (Arokic alluvium)
- **Qr**: Quaternary Riverbank Formations (Alluvium)
- **Qm**: Quaternary Modesto Formation (Alluvium)
SEISMIC DESIGN DATA
Pole Line/Olive Drive Connection
Davis, California

### Spectral Period (s) vs. Acceleration, $Sa \ (g)$

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<tr>
<th>Period (s)</th>
<th>Spectral Acceleration, $Sa \ (g)$</th>
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<td>0.100</td>
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<td>0.200</td>
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<td>0.250</td>
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<td>0.300</td>
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**Design Response Spectrum**

The Design Response Spectrum is the upper envelope of the deterministic and probabilistic response spectrum, but not less than the Minimum Deterministic Spectrum for California. The deterministic spectrum is obtained by using the average of the 2008 Campbell-Bozorgnia and the 2008 Chiou-Youngs ground motion prediction equations. Probabilistic response spectrum is

**SEISMIC LOADING DATA**

- Soil Profile ($V_{S30}$): 722 feet/second
- Magnitude: $M = 6.4$
- Peak Ground Acceleration (PGA): 0.36g

---

Note: Seismic Loading Data provided consistent with Attachment 1 of Caltrans Memo to Designers 1-47.
APPENDIX A

Log of Test Borings
Pole Line Road Overcrossing 1995 As-Built Plans
**CITY OF DAVIS**

**PUBLIC WORKS DEPARTMENT**

**DEVELOPING INNOVATIVE DESIGN SOLUTIONS**

**3301 C St, Bldg. 100-B**

**Tel 916.341.7760**

**Fax 916.341.7767**

**Sacramento, CA 95816**

---

**OLEIVE DR BIKE PATH / POLE LINE RD OC RAMP CONNECTION**

**LOG OF TEST BORINGS NO. 2**

**C.I.P. No. 8313**

---

### SCHEDULE IDENTIFICATION

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### CONSISTENCY OF COHESIVE SOILS

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**The State of California or its officers or agents shall not be responsible for the accuracy or completeness of electronic copies of this plan sheet.**

---

**Notes:**

- **Date:** 2/11/2020
- **Project Number:** 18-438.1
- **Crawford & Associates, Inc.**
  
  1100 Corporate Way, Suite 230
  
  Sacramento, CA 95831
  
  (916) 455-4225
### Field and Laboratory Testing

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<td>(ASTM D 2216)</td>
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<td>Plasticity Index</td>
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<td>PV</td>
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<td>CBR</td>
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<td>Specific Gravity</td>
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<td>CPT</td>
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### Legend - Soil

#### Apparent Density of Cohesionless Soils

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<tr>
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<td>Medium Dense</td>
<td>10 - 15</td>
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<tr>
<td>Dense</td>
<td>15 - 20</td>
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<td>Very Dense</td>
<td>Greater than 20</td>
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#### Moisture

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<tr>
<td>Moisture Present, but no Free Water</td>
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<td>Free Water</td>
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#### Percent or Proportion of Soils

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<td>Field</td>
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<td>Mixture</td>
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#### Particle Size

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<td>Unconfined Compressibility (IL</td>
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<td>Unconfined Uniaxial Compressibility (IL</td>
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### Log of Test Boring No. 3

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<tr>
<td>C.I.P. No. 8313</td>
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</tr>
<tr>
<td>Crawford &amp; Associates, Inc.</td>
<td>1100 Corporate Way, Suite 230, Sacramento, CA 95831 (916) 455-4225</td>
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### City of Davis

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<td>Public Works Department</td>
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### Project Route County Dist

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### Notes

- The State of California or its officers or agents shall not be responsible for the accuracy or completeness of electronic copies of this plan sheet.
- Crawford & Associates, Inc. 1100 Corporate Way, Suite 230, Sacramento, CA 95831 (916) 455-4225
- Project Number 18-438.1
- 2/11/2020
APPENDIX B

Laboratory Test Results
### MOISTURE-DENSITY TESTS - D2216

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<td>CL</td>
<td>CL</td>
<td>CL</td>
<td>CL</td>
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<tr>
<td>Depth (ft.)</td>
<td>3.5</td>
<td>6</td>
<td>11</td>
<td>16</td>
<td>21</td>
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<tr>
<td>Sample Length (in.)</td>
<td>5.454</td>
<td>5.081</td>
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<td>6.029</td>
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<td>961.7</td>
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<td>Mass of Tube (g)</td>
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<td>278.4</td>
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<td>Tare No.</td>
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<td>H4</td>
<td>G9</td>
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<td>Tare (g)</td>
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<tr>
<td>Wet Soil + Tare (g)</td>
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<td>63.6</td>
<td>66.3</td>
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<td>Dry Soil (g)</td>
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<td>Water (g)</td>
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<tr>
<td>Moisture (%)</td>
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<td>31.3</td>
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<td>Dry Density (pcf)</td>
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<td>87.7</td>
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Notes:
## MOISTURE-DENSITY TESTS - D2216

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<td>USCS Symbol</td>
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<td>Depth (ft.)</td>
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<td>Sample Length (in.)</td>
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<td>Wet Soil + Tare (g)</td>
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Notes:
### MOISTURE-DENSITY TESTS - D2216

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Notes:
## MOISTURE-DENSITY TESTS - D2216

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<td>Total Mass Soil+Tube (g)</td>
<td>1201.5</td>
<td>1216.3</td>
<td>1139.0</td>
<td>881.6</td>
<td>1114.0</td>
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<tr>
<td>Mass of Tube (g)</td>
<td>272.9</td>
<td>279.7</td>
<td>276.2</td>
<td>0.0</td>
<td>231.6</td>
</tr>
<tr>
<td>Tare No.</td>
<td>G24</td>
<td>R5</td>
<td>G22</td>
<td>H20</td>
<td>R17</td>
</tr>
<tr>
<td>Tare (g)</td>
<td>13.7</td>
<td>126.6</td>
<td>13.6</td>
<td>13.4</td>
<td>130.3</td>
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<tr>
<td>Wet Soil + Tare (g)</td>
<td>83.3</td>
<td>378.2</td>
<td>81.1</td>
<td>53.4</td>
<td>496.7</td>
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<tr>
<td>Dry Soil + Tare (g)</td>
<td>71.5</td>
<td>342.8</td>
<td>68.2</td>
<td>45.3</td>
<td>417.4</td>
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<tr>
<td>Dry Soil (g)</td>
<td>57.9</td>
<td>216.2</td>
<td>54.7</td>
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<td>287.1</td>
</tr>
<tr>
<td>Water (g)</td>
<td>11.8</td>
<td>35.4</td>
<td>12.9</td>
<td>8.1</td>
<td>79.3</td>
</tr>
<tr>
<td>Moisture (%)</td>
<td>20.4</td>
<td>16.4</td>
<td>23.6</td>
<td>25.3</td>
<td>27.6</td>
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<tr>
<td>Dry Density (pcf)</td>
<td>110.9</td>
<td>115.0</td>
<td>105.8</td>
<td>94.1</td>
<td>96.8</td>
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</table>

Notes:
**Project Name:** Olive Drive Bike and Ped Connection Davis  
**CAInc File No:** 18-438.1  
**Date:** 5/29/18  
**Technician:** GL

## MOISTURE-DENSITY TESTS - D2216

<table>
<thead>
<tr>
<th>Sample No.</th>
<th>USCS Symbol</th>
<th>Depth (ft.)</th>
<th>Sample Length (in.)</th>
<th>Diameter (in.)</th>
<th>Sample Volume (ft³)</th>
<th>Total Mass Soil+Tube (g)</th>
<th>Mass of Tube (g)</th>
<th>Tare No.</th>
<th>Tare (g)</th>
<th>Wet Soil + Tare (g)</th>
<th>Dry Soil + Tare (g)</th>
<th>Dry Soil (g)</th>
<th>Water (g)</th>
<th>Moisture (%)</th>
<th>Dry Density (pcf)</th>
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</thead>
<tbody>
<tr>
<td>R-18-002-10A</td>
<td>CL</td>
<td>46</td>
<td>5.849</td>
<td>2.442</td>
<td>0.01585</td>
<td>881.2</td>
<td>0.0</td>
<td>C19</td>
<td>13.9</td>
<td>45.6</td>
<td>40.1</td>
<td>26.2</td>
<td>5.5</td>
<td>21.0</td>
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<tr>
<td>R-18-002-11A</td>
<td>CL</td>
<td>51</td>
<td>6.003</td>
<td>2.396</td>
<td>0.01566</td>
<td>872.3</td>
<td>0.0</td>
<td>B5</td>
<td>13.7</td>
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<td>62.4</td>
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<td>22.0</td>
<td>104.5</td>
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<td>R-18-002-13A</td>
<td>CL</td>
<td>61</td>
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<td>887.8</td>
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<td>36.6</td>
<td>8.9</td>
<td>24.4</td>
<td>99.1</td>
</tr>
</tbody>
</table>

**Notes:**
Project Name: Olive Drive Bike and Ped Connection Davis
CAInc File No: 18-438.1
Date: 5/25/18
Technician: CAP
Sample ID: R-18-001-4A Depth (ft): 16.0
USCS Classification: CL

**UNCONFINED COMPRESSION TEST - D2166**

- **Dry Density (pcf)**: 97.7
- **Water Content (%)**: 27.4
- **Unconfined Compressive Strength (psi)**: 14.9
- **Unconfined Compressive Strength (psf)**: 2146
- **Shear Strength (psf)**: 1072.8
- **Average Height (in)**: 6.025
- **Average Diameter (in)**: 2.415
- **Rate of strain (%)**: 1.0
- **Strain at Failure (%)**: 8.0

Notes:
Project Name: Olive Drive Bike and Ped Connection Davis
CAinc File No: 18-438.1
Date: 5/25/18
Technician: CAP
Sample ID: R-18-001-5A  Depth (ft): 21.0
USCS Classification: CL

UNCONFINED COMPRESSION TEST - D2166

| Dry Density (pcf) | 109.0 |
| Water Content (%) | 19.8  |
| Unconfined Compressive Strength (psi) | 36.5  |
| Unconfined Compressive Strength (psf) | 5256  |
| Shear Strength (psf) | 2628  |
| Average Height (in) | 6.029 |
| Average Diameter (in) | 2.393 |
| Rate of strain (%) | 1.0   |

Notes:

Olive Drive Bike and Ped Connection Davis
UNCONFINED COMPRESSION TEST - D2166
Strain at 15%
Project Name: Olive Drive Bike and Ped Connection Davis
CAInc File No: 18-438.1
Date: 5/29/18
Technician: GL
Sample ID: R-18-001-8A  Depth (ft): 36.0
USCS Classification: CL

UNCONFINED COMPRESSION TEST - D2166

Dry Density (pcf)  102.9
Water Content (%)  24.8
Unconfined Compressive Strength (psi)  34.7
Unconfined Compressive Strength (psf)  4997
Shear Strength (psf)  2498.4
Average Height (in)  5.653
Average Diameter (in)  2.412
Rate of strain (%)  1.0
Strain at Failure (%)  11.4

Notes:
Project Name: Olive Drive Bike and Ped Connection Davis  
CAInc File No: 18-438.1  
Date: 5/30/18  
Technician: HFW  
Sample ID: R-18-001-10A Depth (ft): 46.0  
USCS Classification: CL

**UNCONFINED COMPRESSION TEST - D2166**

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
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</thead>
<tbody>
<tr>
<td>Dry Density (pcf)</td>
<td>102.6</td>
</tr>
<tr>
<td>Water Content (%)</td>
<td>23.8</td>
</tr>
<tr>
<td>Unconfined Compressive Strength (psi)</td>
<td>39.6</td>
</tr>
<tr>
<td>Unconfined Compressive Strength (psf)</td>
<td>5702</td>
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<tr>
<td>Shear Strength (psf)</td>
<td>2851.2</td>
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<tr>
<td>Average Height (in)</td>
<td>5.647</td>
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<tr>
<td>Average Diameter (in)</td>
<td>2.390</td>
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<tr>
<td>Rate of strain (%)</td>
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Notes:

Strain at 15%
Project Name: Olive Drive Bike and Ped Connection Davis
CAInc File No: 18-438.1
Date: 5/30/18
Technician: HFW
Sample ID: R-18-001-12A  Depth (ft): 56.0
USCS Classification: CL

UNCONFINED COMPRESSION TEST - D2166

Dry Density (pcf) 108.9
Water Content (%) 20.5

Unconfined Compressive Strength (psi) 38.0
Unconfined Compressive Strength (psf) 5472
Shear Strength (psf) 2736
Average Height (in) 5.290
Average Diameter (in) 2.406
Rate of strain (%) 2.0
Strain at Failure (%) 9.9

Notes:
Project Name: Olive Drive Bike and Ped Connection Davis
CAInc File No: 18-438.1
Date: 5/30/18
Technician: HFW
Sample ID: R-18-001-16A Depth (ft): 75.5
USCS Classification: CL

UNCONFINED COMPRESSION TEST - D2166

Dry Density (pcf) 100.5
Water Content (%) 26.4

Unconfined Compressive Strength (psi) 6.8
Unconfined Compressive Strength (psf) 979
Shear Strength (psf) 489.6
Average Height (in) 3.330
Average Diameter (in) 1.395
Rate of strain (%) 1.0
Strain at Failure (%) 8.2

Notes:
Project Name: Olive Drive Bike and Ped Connection Davis
CAInc File No: 18-438.1
Date: 5/30/18
Technician: HFW
Sample ID: R-18-002-2A   Depth (ft): 6.0
USCS Classification: CL

UNCONFINED COMPRESSION TEST - D2166

Dry Density (pcf) 93.0
Water Content (%) 27.7

Unconfined Compressive Strength (psi) 18.4
Unconfined Compressive Strength (psf) 2650
Shear Strength (psf) 1324.8
Average Height (in) 5.744
Average Diameter (in) 2.381
Rate of strain (%) 1.0
Strain at Failure (%) 6.0

Notes:
Project Name: Olive Drive Bike and Ped Connection Davis
CAinc File No: 18-438.1
Date: 5/30/18
Technician: HFW
Sample ID: R-18-002-3A  Depth (ft): 11.0
USCS Classification: CL

UNCONFINED COMPRESSION TEST - D2166

<table>
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<tr>
<th>Parameter</th>
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<tr>
<td>Dry Density (pcf)</td>
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<tr>
<td>Water Content (%)</td>
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<tr>
<td>Unconfined Compressive Strength (psi)</td>
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<tr>
<td>Unconfined Compressive Strength (psf)</td>
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<tr>
<td>Shear Strength (psf)</td>
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<td>Average Height (in)</td>
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<td>Average Diameter (in)</td>
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<td>Rate of strain (%)</td>
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<tr>
<td>Strain at Failure (%)</td>
<td>8.9</td>
</tr>
</tbody>
</table>

Notes:
Project Name: Olive Drive Bike and Ped Connection Davis
CAInc File No: 18-438.1
Date: 5/30/18
Technician: HFW
Sample ID: R-18-002-4A  Depth (ft): 16.0
USCS Classification: CL

UNCONFINED COMPRESSION TEST - D2166

Dry Density (pcf)  108.2
Water Content (%)  20.3
Unconfined Compressive Strength (psi)  45.4
Unconfined Compressive Strength (psf)  6538
Shear Strength (psf)  3268.8
Average Height (in)  6.001
Average Diameter (in)  2.403
Rate of strain (%)  2.0
Strain at Failure (%)  12.1
Notes:
Project Name: Olive Drive Bike and Ped Connection Davis
CAInc File No: 18-438.1
Date: 5/31/18
Technician: CAP
Sample ID: R-18-002-8A  Depth (ft): 36.0
USCS Classification: CL

UNCONFINED COMPRESSION TEST - D2166

Dry Density (pcf) 94.1
Water Content (%) 25.3
Unconfined Compressive Strength (psi) 10.7
Unconfined Compressive Strength (psf) 1541
Shear Strength (psf) 770.4
Average Height (in) 5.948
Average Diameter (in) 2.469
Rate of strain (%) 2.0
Strain at Failure (%) 11.8

Notes:
Dry Density (pcf): 101.3
Water Content (%): 21.0
Unconfined Compressive Strength (psi): 23.1
Unconfined Compressive Strength (psf): 3326
Shear Strength (psf): 1663.2
Average Height (in): 5.849
Average Diameter (in): 2.442
Rate of strain (%): 2.0
Strain at Failure (%): 10.0

Notes:

Olive Drive Bike and Ped Connection Davis
UNCONFINED COMPRESSION TEST - D2166
Project Name: Olive Drive Bike and Ped Connection Davis
CAInc File No: 18-438.1
Date: 5/31/18
Technician: CAP
Sample ID: R-18-002-11A   Depth (ft): 51.0
USCS Classification: CL

UNCONFINED COMPRESSION TEST - D2166

<table>
<thead>
<tr>
<th>Property</th>
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</thead>
<tbody>
<tr>
<td>Dry Density (pcf)</td>
<td>96.2</td>
</tr>
<tr>
<td>Water Content (%)</td>
<td>27.7</td>
</tr>
<tr>
<td>Unconfined Compressive Strength (psi)</td>
<td>12.5</td>
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<tr>
<td>Unconfined Compressive Strength (psf)</td>
<td>1800</td>
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<tr>
<td>Shear Strength (psf)</td>
<td>900</td>
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<tr>
<td>Average Height (in)</td>
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</tr>
<tr>
<td>Average Diameter (in)</td>
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</tr>
<tr>
<td>Rate of strain (%)</td>
<td>2.0</td>
</tr>
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Notes:

Strain at 15%
Project Name: Olive Drive Bike and Ped Connection Davis
CAInc File No: 18-438.1
Date: 5/31/18
Technician: CAP
Sample ID: R-18-002-13A Depth (ft): 61.0
USCS Classification: CL

UN-confined Compression Test - D2166

Dry Density (pcf) 99.1
Water Content (%) 24.4
Unconfined Compressive Strength (psi) 21.1
Unconfined Compressive Strength (psf) 3038
Shear Strength (psf) 1519.2
Average Height (in) 6.002
Average Diameter (in) 2.412
Rate of strain (%) 2.0
Strain at Failure (%) 9.4

Notes:
Project Name: Olive Drive Bike and Ped Connection Davis
CAInc File No: 18-438.1
Date: 5/25/18
Technician: GL

200 Wash - ASTM D1140
Method A

<table>
<thead>
<tr>
<th>Max Particle Size (100% Passing)</th>
<th>Standard Sieve Size</th>
<th>Recommended Min Mass of Test Specimens</th>
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<tbody>
<tr>
<td>2 mm or less</td>
<td>No. 10</td>
<td>20 g</td>
</tr>
<tr>
<td>4.75 mm</td>
<td>No. 4</td>
<td>100 g</td>
</tr>
<tr>
<td>9.5 mm</td>
<td>3/8 &quot;</td>
<td>500 g</td>
</tr>
<tr>
<td>19.0 mm</td>
<td>3/4 &quot;</td>
<td>2.5 kg</td>
</tr>
<tr>
<td>37.5 mm</td>
<td>1 1/2 &quot;</td>
<td>10 kg</td>
</tr>
<tr>
<td>75.0 mm</td>
<td>3 &quot;</td>
<td>50 kg</td>
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Table from 6.2 of ASTM D1140

<table>
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<tr>
<th></th>
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<tr>
<td>USCS Symbol</td>
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<td>SC</td>
<td>CL</td>
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<tr>
<td>Depth (ft.)</td>
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<td>71</td>
<td>26</td>
<td>40.5</td>
</tr>
<tr>
<td>Tare No.</td>
<td>R8</td>
<td>P5</td>
<td>R5</td>
<td>R17</td>
</tr>
<tr>
<td>Tare (g)</td>
<td>130.8</td>
<td>131.8</td>
<td>126.6</td>
<td>130.3</td>
</tr>
<tr>
<td>Dry Soil + Tare (g)</td>
<td>393.5</td>
<td>297.3</td>
<td>342.8</td>
<td>419.4</td>
</tr>
<tr>
<td>Dry Mass before (g)</td>
<td>262.7</td>
<td>165.5</td>
<td>216.2</td>
<td>289.1</td>
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<tr>
<td>Dry Mass after (g)</td>
<td>25.3</td>
<td>58.1</td>
<td>160.8</td>
<td>41.7</td>
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<td>Percent Fines (%)</td>
<td>90</td>
<td>65</td>
<td>26</td>
<td>86</td>
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Notes:
**Project Name:** Olive Drive Bike and Ped Connection Davis  
**CAInc File No:** 18-438.1  
**Date:** 6/4/18  
**Technician:** CAP

### Plastic Index - ASTM D4318

<table>
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<tr>
<th>Sample ID</th>
<th>Depth (ft)</th>
<th>Liquid Limit</th>
<th>Plastic Limit</th>
<th>PI</th>
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<tbody>
<tr>
<td>R-18-001-5A</td>
<td>21</td>
<td>46</td>
<td>17</td>
<td>29</td>
</tr>
<tr>
<td>R-18-002-6A</td>
<td>26</td>
<td>45</td>
<td>17</td>
<td>28</td>
</tr>
</tbody>
</table>

![Plasticity Chart](image-url)

- CL or ML
- CH or OH
- ML or OL
- MH or OH

Liquid Limit (LL)  
Plasticity Chart

- A-18-001-5A
- A-18-002-6A
To: Hailey Wagenman  
Crawford & Associates, Inc.  
1100 Corporate Way STE. 230  
Sacramento, CA 95831-6120

From: Gene Oliphant, Ph.D. \ Randy Horney  
General Manager \ Lab Manager

The reported analysis was requested for the following location:  
Location : 18-438.1 OLIVE DR. Site ID : R-18-001-9A.  
Thank you for your business.

* For future reference to this analysis please use SUN # 77100-160932.

-------------------------------------------------------------
EVALUATION FOR SOIL CORROSION

Soil pH 7.62

Minimum Resistivity 1.42 ohm-cm (x1000)

Chloride 1.7 ppm 0.00017 %

Sulfate 32.9 ppm 0.00329 %

METHODS  
pH and Min.Resistivity CA DOT Test #643  
Sulfate CA DOT Test #417, Chloride CA DOT Test #422
To:  Hailey Wagenman
     Crawford & Associates, Inc.
     1100 Corporate Way STE. 230
     Sacramento, CA  95831-6120

From:  Gene Oliphant, Ph.D. \ Randy Horney\n        General Manager \ Lab Manager \\

The reported analysis was requested for the following location:
Location :  18-438.1 OLIVE DR.  Site ID : R-18-002-5A.
Thank you for your business.

* For future reference to this analysis please use SUN # 77100-160933.
---------------------------------------------------------------------------------------------------
EVALUATION FOR SOIL CORROSION

Soil pH        7.69

Minimum Resistivity  1.10 ohm-cm (x1000)

Chloride   0.4 ppm  00.00004 %
Sulfate     7.9 ppm  00.00079 %

METHODS
   pH and Min.Resistivity CA DOT Test #643
   Sulfate CA DOT Test #417,  Chloride CA DOT Test #422
### CONSOLIDATION TEST - ASTM D2435

#### STRESS VERSUS STRAIN

<table>
<thead>
<tr>
<th>Axial Load, psf</th>
<th>Void Ratio</th>
<th>Axial Strain, %</th>
<th>Measurement</th>
<th>Initial</th>
<th>Final</th>
</tr>
</thead>
<tbody>
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<td>initial</td>
<td>0.717</td>
<td>0.00</td>
<td>Height (in.)</td>
<td>0.750</td>
<td>0.694</td>
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<tr>
<td>100</td>
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<td>0.00</td>
<td>Moisture Content (%)</td>
<td>24.4</td>
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<tr>
<td>250</td>
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<td>0.12</td>
<td>Dry Density (pcf)</td>
<td>100.4</td>
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<td>500</td>
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<td>0.588</td>
<td>7.52</td>
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</table>

Note:
Gs = 2.76 (assumed)
### CONSOLIDATION TEST - ASTM D2435

#### STRESS VERSUS VOID RATIO

<table>
<thead>
<tr>
<th>Axial Load, psf</th>
<th>Void Ratio</th>
<th>Axial Strain, %</th>
<th>Measurement</th>
<th>Initial</th>
<th>Final</th>
</tr>
</thead>
<tbody>
<tr>
<td>initial</td>
<td>0.717</td>
<td>0.00</td>
<td>Height (in.)</td>
<td>0.750</td>
<td>0.694</td>
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<tr>
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Note:

Gs = 2.76 (assumed)
**CONSOLIDATION TEST - ASTM D2435**

Project Name: Crawford 18-438.1  
Project Number: S9763-05-128  
Sample Number: R-18-001 R-18-001-7A

**CONSOLIDATION TEST RESULTS**  
JOB S9763-05-128, BORING R-18-001, R-18-001-7A

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<th>Axial Load (psf)</th>
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<th>m_v, coef of vol Compres (in²/lb)</th>
<th>c_C, Comp Index</th>
<th>50% Consolidation t_50, Time to Consol (min)</th>
<th>C_v, Coeff of Consol (ft²/yr)</th>
<th>90% Consolidation t_90, Time to Consol (min)</th>
<th>C_v, Coeff of Consol (ft²/yr)</th>
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<td>0.603</td>
<td>6.64</td>
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<td>0.565</td>
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<td>2000</td>
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G_s = 2.76  
(assumed)

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<th>COND AT START OF TEST</th>
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<tr>
<td>HEIGHT (in.)</td>
<td>0.7500</td>
<td>0.6936</td>
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<td>MOISTURE CONTENT (%)</td>
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<td>21.2</td>
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<tr>
<td>DRY DENSITY (pcf):</td>
<td>100.4</td>
<td>108.5</td>
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<tr>
<td>SATURATION (%)</td>
<td>94.2</td>
<td>99.9</td>
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</table>
APPENDIX C

CIDH Pile Nominal Resistance
CAST-IN-DRILLED-HOLE (CIDH) PILE NOMINAL RESISTANCE

Pole Line Road/ Olive Drive Connection

Davis, CA

CAInc Project Number: 18-438.1

Support Location(s): Abutment 1

Pile Diameter = 24 inches

Pile Cut-Off Elevation = 36.11 feet

Socket Diameter = NA

Permanent Casing Tip Elevation = 19.11 feet

CAST-IN-DRILLED-HOLE (CIDH) PILE NOMINAL RESISTANCE

Pole Line Road/ Olive Drive Connection
Davis, CA
CAInc Project Number: 18-438.1

Support Location(s): Bent 2
Pile Diameter = 72 inches
Pile Cut-Off Elevation = 17.97 feet

Socket Diameter = NA
Permanent Casing Tip Elevation = 0.97 feet

<table>
<thead>
<tr>
<th>SERVICE LIMIT</th>
<th>REQUIRED NOMINAL RESISTANCE = 560 kips</th>
<th>SCOUR ELEVATION = NA feet</th>
<th>DESIGN PILE TIP ELEVATION = -16.0 feet</th>
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</thead>
<tbody>
<tr>
<td>STRENGTH LIMIT</td>
<td>REQUIRED NOMINAL RESISTANCE = 1070 kips</td>
<td>SCOUR ELEVATION = NA feet</td>
<td>DESIGN PILE TIP ELEVATION = -30.0 feet</td>
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<tr>
<td>EXTREME LIMIT</td>
<td>REQUIRED NOMINAL RESISTANCE = 480 kips</td>
<td>SCOUR ELEVATION = NA feet</td>
<td>DESIGN PILE TIP ELEVATION = -14.0 feet</td>
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CIDH PILE NOMINAL RESISTANCE
(AXIAL COMPRESSION)

Nominal Side Resistance with Permanent Casing

CAST-IN-DRILLED-HOLE (CIDH) PILE NOMINAL RESISTANCE

Support Location(s): Bent 3  
Pile Diameter = 72 inches  
Pile Cut-Off Elevation = 25.4 feet

<table>
<thead>
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<th>Required Nominal Resistance</th>
<th>Scour Elevation</th>
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<tbody>
<tr>
<td></td>
<td>580 kips</td>
<td>NA feet</td>
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<table>
<thead>
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<th>Strength Limit</th>
<th>Required Nominal Resistance</th>
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<td>1120 kips</td>
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<td>510 kips</td>
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CAST-IN-DRILLED-HOLE (CIDH) PILE NOMINAL RESISTANCE

Pole Line Road/ Olive Drive Connection
Davis, CA
CAInc Project Number: 18-438.1

Support Location(s): Bent 4
Pile Diameter = 72 inches
Pile Cut-Off Elevation = 32.55 feet

<table>
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<th>Service Limit</th>
<th>Strength Limit</th>
<th>Extreme Limit</th>
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<tr>
<td>REQUIRED NOMINAL RESISTANCE = 640 kips</td>
<td>REQUIRED NOMINAL RESISTANCE = 1230 kips</td>
<td>REQUIRED NOMINAL RESISTANCE = 570 kips</td>
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<td>SCOUR ELEVATION = NA feet</td>
<td>SCOUR ELEVATION = NA feet</td>
<td>SCOUR ELEVATION = NA feet</td>
</tr>
<tr>
<td>DESIGN PILE TIP ELEVATION = -14.0 feet</td>
<td>DESIGN PILE TIP ELEVATION = -37.0 feet</td>
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CIDH PILE NOMINAL RESISTANCE (AXIAL COMPRESSION)

ELEVATION (FEET)

NOMINAL RESISTANCE (KIPS)

CAST-IN-DRILLED-HOLE (CIDH) PILE NOMINAL RESISTANCE

Pole Line Road/ Olive Drive Connection
Davis, CA
CAInc Project Number: 18-438.1

Support Location(s): Bent 5
Pile Diameter = 72 inches
Pile Cut-Off Elevation = 39.69 feet

Socket Diameter = NA
Permanent Casing Tip Elevation = 22.69 feet

<table>
<thead>
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<th>Extreme Limit</th>
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<tr>
<td>Required Nominal Resistance = 390 kips</td>
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<td>Required Nominal Resistance = 380 kips</td>
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<tr>
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<td>Scour Elevation = NA feet</td>
<td>Scour Elevation = NA feet</td>
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<tr>
<td>Design Pile Tip Elevation = 9.0 feet</td>
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CIDH PILE NOMINAL RESISTANCE (AXIAL COMPRESSION)

APPENDIX D

LPile Parameters
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<th>Elevation</th>
<th>Soil Type</th>
<th>Total Unit Weight (lb/ft³)</th>
<th>Buoyant Unit Weight (lb/ft³)</th>
<th>Friction Angle, $\phi$ (degrees)</th>
<th>Cohesion, c (psf)</th>
<th>Strain Factor, $E50$ (dim.)</th>
<th>p-y Modulus, $K$ (lb/in³)</th>
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<tr>
<td>36 to 19</td>
<td>Soft Clay (Reese)</td>
<td>118</td>
<td>56</td>
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<td>750</td>
<td>0.010</td>
<td>500</td>
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<td>19 to -30</td>
<td>Stiff Clay w/o free water (Reese)</td>
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<td>68</td>
<td>xx</td>
<td>2500</td>
<td>0.005</td>
<td>1000</td>
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<td>-30 to -45</td>
<td>Stiff Clay w/o free water (Reese)</td>
<td>124</td>
<td>62</td>
<td>xx</td>
<td>1000</td>
<td>0.007</td>
<td>500</td>
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<tr>
<td>Elevation</td>
<td>Soil Type</td>
<td>Total Unit Weight (lb/ft³)</td>
<td>Buoyant Unit Weight (lb/ft³)</td>
<td>Friction Angle, ( \phi ) (degrees)</td>
<td>Cohesion, c (psf)</td>
<td>Strain Factor, ( E50 ) (dim.)</td>
<td>p-y Modulus, K (lb/in³)</td>
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<td>2000</td>
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<td>-35 to -45</td>
<td>Soft Clay (Reese)</td>
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<td>60</td>
<td>xx</td>
<td>750</td>
<td>0.010</td>
<td>500</td>
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<td>Total Unit Weight (lb/ft³)</td>
<td>Buoyant Unit Weight (lb/ft³)</td>
<td>Friction Angle, $\phi$ (degrees)</td>
<td>Cohesion, $c$ (psf)</td>
<td>Strain Factor, $E50$ (dim.)</td>
<td>$p$-y Modulus, $K$ (lb/in³)</td>
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<td>xx</td>
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<td>1000</td>
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<td>1000</td>
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<tr>
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<td>67</td>
<td>31</td>
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<td>xx</td>
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APPENDIX E

General Plan
APPENDIX F

Caltrans Review Comments
Responses to Caltrans Review Comments
### General Project Information
- **Dist:** 03  
- **EA:** 03-4H260  
- **Project #:** 0318000232  
- **Project:** 03-SAC-80-PM 0.90  
- **OSFP Liaison:** Saygunn Low  
- **Telephone:** (916) 227-8868  
- **e-mail:** Low, Saygunn@DOT <saygunn.low@dot.ca.gov>

### Review Phase
- PSR/PDS (Review No. )
- APS/PSR (Review No. )
- APS/PR (Review No. _1)
- **Type Selection**
- 65% PS&E Unchecked Details
- PS&E (Review No. )
- Construction Support
- **Other:** DRAFT

### Structure Information
- **Olive Dr. Bike Path /**  
- **Pole Line Road OC Ramp Connection**  
- **Bridge No:** 22-(New)

### Consultant Information (to be filled in by Consultant)
- **Consultant Structure Lead** (First and Last Name)  
- **Structure Consultant Firm**  
- **Phone Number**  
- **e-mail**  
- **Response Date**

### Review Comments

<table>
<thead>
<tr>
<th>#</th>
<th>Doc. See Note 1</th>
<th>Section Page #</th>
<th>Section Name</th>
<th>Review Comments</th>
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<tbody>
<tr>
<td>1</td>
<td>Cover Sheet</td>
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<td></td>
<td>-Please include Bridge number along with EA and Project numbers, on the cover sheet of the Foundation Report.</td>
</tr>
</tbody>
</table>
| 2 | FR | 4.1 3 | Field Exploration | -Please provide explanation that why only two borings were drilled for a five (5) support locations of the proposed structure? Caltrans usually recommends one borehole per support location for new structure for CIDH pile.  
-On what was the consistency of cohesive soil based on in the field classification? Please explain.  
-The exploration borings drilled by Taber Consultants in 1990s are okay for general study of the site geology but may not be a substitute for a new boring for the proposed structure. Please exclude. |
| 3 | FR | 4.3 4 | Ground | -Because of the large (7.2 feet) difference in Groundwater elevation, why piezometer was not installed? |

---

**Note 1: Abbreviations for Typical Documents**

- **P=Structure Plans**  
- **SP=Special Provisions**  
- **FR=Foundation Rpt**  
- **DC=Design Calcs**  
- **TS=Type Sel. Report**  
- **QCC=Quant. Check Calcs**  
- **RP=Road Plans**  
- **E=Estimate**  
- **H=Hydraulics Rpt**  
- **CC=Check Calcs**  
- **QC=Quant. Calcs**

- **_= Comment Resolved**  
- **(for Reviewer’s use)**
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<th>Section</th>
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<td>4</td>
<td>FR</td>
<td>7.1</td>
<td>What is the design Groundwater elevation (depth)? Please provide. Per Caltrans Memo To Designer 20-10, Fault Rupture studies should include both Alquist-Priolo Fault Zones and analysis of unzoned fault(s) that is Holocene or younger in age. Also, Caltrans online map is not to be used to obtain fault distances for fault rupture study.</td>
</tr>
<tr>
<td>5</td>
<td>FR</td>
<td>8.1 7</td>
<td>The design soil parameters provided in Table 5 thru 7 do not match with the laboratory test results as shown in appendix B. Please explain.</td>
</tr>
<tr>
<td>6</td>
<td>FR</td>
<td>8.2 9</td>
<td>-Please provide units for the given loads.</td>
</tr>
<tr>
<td>7</td>
<td>FR</td>
<td>9.1 10</td>
<td>-Table 10 represents both Abutment and Bents so it should be “Abutment and Bents Foundation Design Recommendations”. Please revise. -The resistance factors should be 0.7 for all cases? Please check.</td>
</tr>
<tr>
<td>8</td>
<td>FR</td>
<td>9.3.2 11</td>
<td>Please provide reference(s) to validate the sentence you mentioned “Also, the pile cap at Abutment 1 is expected to be in firm contact with the ground. Therefore, no pile group reduction is applied to the bent piles.”</td>
</tr>
<tr>
<td>9</td>
<td>FR</td>
<td>9.3.3 11</td>
<td>What did you mean by significant long-term settlement at this site (even for cohesive soil)? Please explain.</td>
</tr>
<tr>
<td>10</td>
<td>FR</td>
<td>9.3.4 12</td>
<td>-The lateral resistance analyses of the proposed pile are not included in Appendix D. Please include. -The Pile p-Multipliers, pm should be based on California Amendments to ASSHTO 10-7-2.4.</td>
</tr>
</tbody>
</table>

**Note 1: Abbreviations for Typical Documents (if Abbr. is not below, type in the document type)**

<table>
<thead>
<tr>
<th>P=Structure Plans</th>
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<tr>
<td>RP=Road Plans</td>
<td>E=Estimate</td>
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<td>CC=Check Calcs</td>
<td>QC=Quant. Calcs</td>
<td></td>
</tr>
</tbody>
</table>

✓ = Comment Resolved (for Reviewer’s use)
|   | FR  | 9.3.6 12 | Construction Consideration CIDH | -Please provide elevations along the designed depth of the proposed pile (graphical representation) piles.  
-Please provide calculation. |
|---|-----|---------|-----------------------------|-----------------------------------------------------------------------------------------------------------------------------------|
| 11 | FR  | Appendix A | Please ensure that the LOTB sheets conform to Caltrans’ Soil & Rock Logging, Classification, and Presentation Manual (SRLCPM), currently dated 2010.  
The LOTB attached to this report are not acceptable. |
| 12 | Plan | 4 | ARS-Curve | Please specify the methodology (deterministic or probabilistic) used to develop design response spectra. |
| 13 | FR  | Additional | -Please discuss alternative foundation types.  
-All required loads and geotechnical data should be provided according to LRFD Memo to Designer 3-1.  
-All responses to Caltrans review comments should be addressed in the report and be included as attachments. |
**Caltrans Review Comments** – Sections are updated to reflect current report (Sections in parentheses reflect Caltrans reviewed report – 2018)

1) No Bridge number reported on general plan (GP)  
   EA number not specified on GP  
   Included Crawford & Associates, Inc. (CAInc) project number on cover page

2) - 2 borings combined with the available information in the area are sufficient for evaluation of the project.  
   - The consistency of cohesive soil is based on the pocket penetrometer test. A sentence was added in Section 4.1: Field Exploration to address this.

3) - A piezometer was not installed.  
   - A design groundwater elevation of 16 feet (obtained from boring A-18-001) is being utilized as stated in Section 9.1 (Section 8.1): Soil Parameters.

4) - All faults in the project vicinity can be seen on Figure 2: Fault Activity Map; attached in the appendices. The closest fault <150 years in age, the Green Valley fault zone (Green Valley fault), is approximately 35.5 miles away and therefore not in the image.  
   - The Caltrans online map was not utilized to obtain fault distance. In Section 8.1 (Section 7.1): the United States Geologic Survey (USGS) Google Earth kmz file was utilized to obtain the closest active fault.  
   [https://earthquake.usgs.gov/hazards/qfaults/].

5) - Soil parameters were based off the LOTB and Laboratory results completed for the Pole Line/Olive Drive Connection Project.

6) - Units for the given loads are provided on the graph at the top.

7) - The title of Table 10 was updated to read **“Abutment and Bents Foundation Design Recommendations.”**  
   - Per Caltrans LRFD Memo to Designers, dated June 2014, Section 3-1: Deep Foundations Attachment 1 Design Procedure and Examples, Table 3-6 Foundation Recommendations on page 5 a resistance factor of 0.7 is used for the Strength and a resistance factor of 1.0 is used for the Extreme Event.  
   [https://dot.ca.gov/-/media/dot-media/programs/engineering/documents/memotodesigner/f0003026-3-1-a-1.pdf].

8) - Section 10.3.4 (Section 9.3.2) Pile Group Reduction was revised.

9) - Significant long term settlement for cohesive soils refers to the consolidation analysis. This portion was moved to Section 10.3.1. Long-term settlement of the embankments is also discussed in Section 11.2 (Section 10.2) Slope Geometry and Stability.
10) -Per Section 10.3.2 (Section 9.3.4) LPILE analysis results were not provided because Wood Rogers (bridge engineer) is performing their own LPile analysis for this project. P-multipliers could not be calculated due to the lack of pile layout. This information was since provided on the Olive Drive Path/Pole Line Rd OC Ramp Connection – Abutment Details No. 1 dated October 2019 and the report was updated. -Pile Cut-off elevations were added to the titles of the CIDH output graphs. Pile Cut-off elevations can also be found in Section 10.1 (Section 9.1), Table 10.

11) -Section 12.5 (Section 11.5) Vibration was added to the report to address the potential adverse effects of the proposed CIDH installation on the existing structures. If temporary casing cannot be installed using impact or vibratory hammers rotator/oscillator methods may be utilized instead.

12) -The LOTB was edited to conform to Caltrans current requirements.

13) -The probabilistic methodology was utilized to develop the design response spectra as discussed in Section 8.2 (Section 7.2) Seismic Ground Motions.

14) -Discussion of alternative foundation types were added to Section 9 (Section 8) conclusions. -Scour data as required per LRFD Memo to Designers 3-1 Attachment 1 Table 3-3 was missing from this report. A section was added to address that there are no scour considerations due to the lack of surface water. Subsequently all section after 6 were increased by one number. -All Caltrans review comments are reported as an attachment in Appendix F.