GEOTECHNICAL REPORT

JACOBS AVENUE LEVEE
EVALUATION PROJECT
CITY OF EUREKA, CALIFORNIA

Prepared For:

Humboldt County Public Works Department

CGI TECHNICAL SERVICES INC.

COUNTY OF HUMBOLDT The Home of the Redwoods

CGI TECHNICAL SERVICES INC.

July 25, 2016
CGI: 15-1949.03

Mr. Hank Seemann
Deputy Director - Environmental Services
HUMBOLDT COUNTY PUBLIC WORKS DEPARTMENT
1106 Second Street
Eureka, California 95501

Subject: **Geotechnical Report**
Jacobs Avenue Levee Evaluation Project
Eureka, California

Dear Mr. Seemann:

CGI Technical Services, Inc. (CGI), is pleased to submit this geotechnical report to Humboldt County Public Works Department for the Jacobs Avenue Levee Evaluation Project, located in Eureka, California. This report is being submitted in accordance with the Agreement for Professional Services established between CGI and the County.

This report summarizes the geotechnical engineering analyses performed to evaluate the levee and its ability to function in accordance with current United Stated Army Corps of Engineers performance guidelines and presents remedial-concept recommendations, where appropriate. In addition, this report also discusses field explorations performed, laboratory test results, and general subsurface conditions encountered at the site during this study.

We appreciate the opportunity to perform this study. If you have any questions pertaining to this report, or if we may be of further service, please contact us at (530) 244-6277 at your earliest convenience.

Regards,

CGI TECHNICAL SERVICES, INC.

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EXECUTIVE SUMMARY

The purpose of our investigation was to explore subsurface conditions at the project site to aid Humboldt County, City of Eureka, and affected private landowners in evaluating the ability of the Jacobs Avenue Levee system to function in accordance with current FEMA and U.S. Army Corps of Engineers (USACE) performance criteria. To achieve this objective, CGI performed subsurface exploration, laboratory testing, and engineering analyses in accordance with the standards of FEMA (2008), and USACE (2013, 2010a, 2008, 2005, 2003, and 2000a).

The following paragraphs summarize the analyses that were performed to assess the geotechnical aspects of the levee system and include brief summaries of our findings. It should be noted that our analyses were limited to addressing the geotechnical aspects of flood control protection for the 100-year recurrence interval (or 1% annual chance exceedance) coastal stillwater flood event for the Jacobs Avenue Levee system. Information prepared by Northern Hydrology & Engineering regarding the hydrologic and hydraulic aspects of the levee system was provided to us by Humboldt County utilized as applicable.

On the basis of field observations, subsurface exploration, laboratory testing and engineering analyses performed for this investigation, the existing Jacobs Avenue Levee system is marginally capable of providing flood control protection to adjacent improvements. Based on our analyses, all of the sections evaluated were susceptible to seepage-related failure or instability under steady state conditions. Furthermore, seismically-induced settlement values were estimated to be relatively small; however, they would impinge upon the minimum USACE specified freeboard along the levee.

SUMMARY OF FINDINGS AND RECOMMENDATIONS

Levee Prism Geometry & Freeboard. The landside and waterside slope inclinations were observed to exceed inclinations recommended by the USACE for levee systems. Those inclinations are to be at 2:1 and 3:1 or flatter for landside and waterside slopes, respectively. In addition, with the exception of local areas, the crown width of the levee was deficient of the minimum 12 feet specified by the USACE. Furthermore, minimum freeboard along the levee was deficient for much of the alignment (freeboard is the vertical distance between the top of the levee and the water surface elevation for the 1% annual chance exceedance flood). Mitigation measures to correct the levee geometry and freeboard consist of widening, raising, and grading the levee to meet minimum USACE specifications.

Vegetation and Erosion. Vegetation is present on the waterside and landslide slopes and locally on the crest of the levee. The vegetation includes seasonal grasses, shrubs, brambles, and trees that are locally dense. Mitigation for vegetation consists of implementation of a vegetation reduction program.

Erosion was locally observed along the waterside slope of the levee in areas where rip-rap is not present, in gaps within rip-rap, in areas where rip-rap is relatively thin, and in areas where high tides extend above rip-rap. The erosion appears to be caused by wind-generated waves lapping against the slope and generally creates over-steepened slopes. Mitigation for the erosion is to remove rip-rap in areas of erosion, restore the inclinations to meet USACE standards, place filter fabric on the eroded area, and replace the rip-rap with sufficient amounts of properly sized rip-rap that extends to a minimum height of mean high water.

Liquefaction and Seismic Settlement Potential. Based on our analysis, there is a potential for liquefaction and seismically induced dry settlement of the pervious foundation material during the design seismic event. The maximum estimated liquefaction and seismically induced dry settlements are approximately 4 inches and less than 1 inch, respectively. Also, there is a high risk of lateral spreading to occur with estimated lateral displacement of up to 63 inches. Mitigation options for liquefaction include stone columns, rammed aggregate piers, and compaction grouting.

Permeability and Seepage. For our seepage evaluation, the potential for underseepage and through-seepage was evaluated for steady-state flood stage conditions. Based on the results of our seepage analyses, all of the cross sections have underseepage exit gradients that are lower than 0.5, which is the maximum threshold established by the USACE . However, through-seepage was projected to intersect the landside slope of the levee above the landside toe at all cross sections, which does not meet UASCE requirements. Mitigation options for through-seepage include construction of a cutoff trench and/or a landside seepage berm.

In addition, penetrations of the levee should be assessed to evaluate composition of backfill materials surrounding the penetrations. If those materials do not consist of relatively fat clay or a Portland cement concrete (PCC) material, then unacceptable seepage forces are likely to occur along the penetrations.

Slope Stability. Our slope stability analyses indicated that the slopes evaluated are generally stable to marginally stable for most of the conditions discussed in section 8.3 under static conditions but predominately unstable under pseudo-static (seismic) conditions. For pseudo-static slope stability analyses, undrained residual strength parameters were estimated for potentially liquefiable strata. However, a number of conditions do not meet the threshold established by USACE. Many of those conditions are anticipated to be mitigated if the geometry of the levee is reconstructed to meet USACE standards.

Seismic Deformation. Based on the results of pseudo-static slope stability analyses, at least 36 inches of seismic displacements are anticipated for the design seismic event. If mitigations for the levee geometry and liquefaction potential are implemented, then seismic deformation will be mitigated. Otherwise, mitigation options include increasing the strength of soils within the levee and/or construction of stability berms.

OPINION REGARDING GEOTECHNICAL ASPECTS OF LEVEE CERTIFICATION

Based on the results of our study, the levee system's ability to provide flood protection in its present state is not sufficient considering nearly all conditions evaluated at the five cross section locations.

Based on the results of our evaluation, it is our opinion that the geotechnical aspects of the levee system are unlikely to be certifiable relative to FEMA and USACE standards.

1 INTRODUCTION

CGI Technical Services, Inc. (CGI), is pleased to present this report providing the results of studies performed for the Jacobs Avenue Levee Evaluation Project, located in the City of Eureka, California. This study was performed for the Humboldt County Department of Public Works (County). Funding for this study was provided by the California Department of Water Resources Local Levee Assistance Program, Humboldt County, City of Eureka, and affected private landowners, with laboratory testing provided as a cost-share by Caltrans.

The purpose of the project was to collect and analyze selected geotechnical information in order to estimate the stability of the levee foundation and embankment, and to evaluate the potential for settlement, through-seepage, underseepage, or erosion to cause instability. Available information prior to this study was not sufficient to assess these risks for the Jacobs Avenue Levee. The results of the project will assist the County, City of Eureka, and private landowners in assessing the risks associated with the levee system and in determining whether the levee meets the embankment and foundation stability criteria of FEMA's National Flood Insurance Program standards at 44 CFR 65.10.

The following sections present our understanding of the project, the purpose of this study, and the findings, conclusions, and recommendations of this study. Our services were performed in general conformance with our proposal dated April 24, 2015.

1.1 PROJECT UNDERSTANDING

The Jacobs Avenue levee system extends along the northern margin of the Eureka Slough from Highway 101 east for about 5,100 feet to a tie-in point near Airport Road. The location of the levee is shown on Plate 1 – Site Location Map. The levee is an earthen embankment. Prior to this study, it was unknown whether the levee embankment was constructed with an impervious core, and whether levee foundation soils included an impervious layer. The levee is slightly to heavily vegetated, predominately with seasonal weeds, shrubs, and few trees. Intertidal mud flats are present at low tide at the waterside toe of the levee.

As shown on Plate 2 – Levee Details, the levee currently protects 31 properties from the Eureka Slough, located south of the levee. Most of those properties operate as commercial businesses; however, one property is a County corporate yard, one is a mini-storage facility, and another as a residential trailer park. The Eureka Slough is a wetland area that connects Freshwater Creek with Humboldt Bay and is, thus, subject to tidal influences from the bay and floodwaters from the creek.

The area behind Jacobs Avenue Levee was given a Flood Zone "C" rating by FEMA on the 1986 flood insurance rate map ([FIRM]; FEMA, 1986), even though technical studies documenting the stability and freeboard of the levee had not been performed. This rating designated the area behind the levee as having minimal flood hazards under the base flood condition (the "1% annual chance exceedance probability" or "100-year recurrence interval" flood level). We understand that FEMA's current policies require levee owners to provide technical studies documenting the stability and freeboard of the levee to support certification and accreditation in accordance with FEMA's National Flood Insurance Program standards in 44 CFR 65.10. Based on the lack of sufficient documentation for the Jacobs Avenue Levee, FEMA rezoned the area protected by the levee as "Zone A – Special Hazard Area" on preliminary FIRM maps (FEMA, 2015a; FEMA, 2015b), which are expected to become effective in 2016.

The geotechnical evaluation of the Jacobs Avenue levee system is a subtask of work being performed for the overall evaluation of the levee system. Hydrologic, hydraulic, bathymetric, and topographic services were performed by others.

1.2 PROJECT HISTORY

We understand that the Jacobs Avenue levee was a privately built embankment constructed by the initial landowner, Frank Herrick (McKamee et al., 2014). The actual date that the levee was constructed is uncertain with some sources claiming it occurred prior to 1931 (Walters, 2011).

Once the levee was constructed, we understand that the landside property was initially utilized predominately for agricultural purposes. Aerial photographs taken up until early 1948 show the lands as fallow aside from fences, eight billboards adjacent to Highway 101, and a small structure at the western end of the property near the bridge crossing the slough. In addition, a drainage ditch is present adjacent and parallel to the 1948 alignment of Highway 101. That drainage ditch was emptied into the slough by a ditch extending across property where the Lazy J Mobile Home Park currently is located. A concrete penetration extended through the levee connecting the ditch to the Eureka Slough. In late 1948, construction had begun on a livestock auction yard and facility sited about where The Farm Store is currently located.

We understand that the property was subdivided by Harold Hilfiker in 1949 (Walters, 2011) to create 31 individual parcels on about 45 acres. Aerial photographs from 1950 show three pads had been developed onto the landside properties but no structures had been established on those pads. In addition, the 1950 photographs show the auction yard as completed and an unpaved access road present approximately along the same alignment as Jacobs Avenue. By 1953, about 8 of the parcels show construction of structures or other development.

Between 1953 and 1956, the previously described drainage ditch extending across the Lazy J Mobile Home Park had been relocated to east of the auction yard, to where the current drainage channel is present adjacent to Airport Road. The bridge at Airport Road and the concrete penetration both have 1954 marked in their concrete, likely indicating the year in which those structures were constructed. However, the concrete levee penetration for the new channel cannot be discerned in aerial photographs from 1956 and 1972 but that might be due to the relatively poor resolution of the 1956 photograph. The 1956 photographs do show that the former drainage channel has been filled and the mobile home park constructed. It is unclear how the concrete penetration was abandoned.

By 1957, about half of the parcels had been developed with commercial structures. By 1990, the Jacobs Avenue area had been almost fully developed and appeared to be similar to how the area looks today.

Because of the subdivided lots, levee maintenance became the responsibility of the individual landowners of the parcels abutting the levee. No comprehensive program to maintain the entire levee has been established (Mattson, 2011). Maintenance records for the levee were not available (and likely don't exist). However, we understand that in the 1980s, a number of the parcel owners worked with the City of Eureka to improve sections of the eastern portion of the levee (McNamee et al., 2014). It appears that those improvements included raising a portion of the levee, increasing its width, and locally placing rip-rap and rock armoring to improve protection along the waterside levee slope.

Limitations of the parcel-by-parcel maintenance of the levee by the individual landowners have been recognized, prompting discussions of establishing a levee district (Mattson, 2011). In 2012, the Humboldt County Board of Supervisors authorized the Public Works Department to work with the parcel owners protected by the levee to explore formation of a levee district in accordance with Water Code Section 70000. We understand that after further consideration, formation of a new levee district was problematic and that the existing Humboldt County Flood Control District was found capable as serving as the funding recipient for grant funds from DWR to assess the stability of the levee system (Mattson, 2014). The results of the technical studies will help inform future discussion regarding potential district formation.

1.3 PURPOSE & SCOPE OF SERVICES

The purpose of our investigation was to explore subsurface conditions at the project site to aid the County in evaluating the ability of the Jacobs Avenue Levee system to function in accordance with current USACE performance criteria. To achieve this objective, CGI performed subsurface exploration, laboratory testing, and engineering analyses.

Levee stability evaluations utilize standards established by the USACE's National Flood Risk Management Program through various policy letters, engineering circulars, and engineering manuals (USACE, 2010a). The primary USACE standards utilized during this study consist of the following:

- Engineering Circular 1110-2-6067, Certification of Levee Systems for the NFIP (2010a);
- REF10L0 Rev 2, Geotechnical Levee Practice (2008);
- ETL 1110-2-569, Design Guidance for Levee Underseepage (2005);
- EM 1110-2-1902, Slope Stability (2003);
- EM 1110-2-1913, Design and Construction of Levees (2000a);
- EM 1110-2-301, Guidelines for Landscape Planting and Vegetation Management at Floodwalls, Levees, and Embankment Dams (2000b);
- EM 1110-2-1914, Design, Construction, and Maintenance of Relief Wells (1992); and
- EM 1110-2-1901, Determination of Permeability of Soil (1986).

References for those standards are included in Section 10.0 of this report. The scope of work for this study included the following:

1.1.1 Data Review, Site Reconnaissance, and Access Coordination

In preparation for the field exploration program, CGI performed site reconnaissance, and reviewed historic data, aerial photographs, and other documents, maps, and existing geologic and geotechnical literature relevant to the site. In addition, we met with County representatives at the site to discuss exploration and cross section locations in the field.

Relevant geotechnical data from previous reports prepared by the USACE and local consultants, including site and subsurface descriptions, groundwater data, laboratory test results, drill hole logs, and as-built plans, were reviewed during the study. Further details regarding previous reports are presented in Section 3.0 and in Appendix A.

Prior to performing drill holes and cone penetrometer test (CPT) soundings, CGI obtained drilling permits from the County and contacted Underground Service Alert (USA) to help identify potential buried utility conflicts at the proposed exploration locations, as required by California law. In addition, we marked all exploration locations and worked with the County to coordinate access onto private properties.

1.1.2 Field Exploration

Field exploration consisted of advancing a total of 18 subsurface explorations at selected locations along the levee alignment. In total, we performed the following explorations for this study:

Table 1-1 – Explorations Performed for Study

Exploration Type	Total Advanced
Cone Penetrometer Soundings	11
Rotary Mud Drilling	7

The field exploration program was performed in November 2015 and January 2016. Locations of explorations are presented on Plates 3.1 through 3.4 – Cross Section & Exploration Locations. Additional information regarding the field exploration program are presented in Section 3.0 and Appendix B.

1.1.3 Laboratory Testing

Laboratory testing was performed for CGI by Caltrans on selected soil samples obtained during the field exploration program. The types and numbers of tests were chosen to help classify and characterize selected subsurface soil characteristics and engineering properties. Further details regarding the laboratory testing program are presented in Section 4.0 and in Appendix C.

1.1.4 Geotechnical Analyses and Evaluation

Using data obtained from the data review, field exploration, and laboratory testing programs, CGI performed geotechnical analyses, as specified in the scope of work, at 5 cross sections established along the length of the Jacobs Avenue Levee system. The analyses were consistent with geotechnical criteria specified by the USACE in EM 1110-2-1913 (USACE, 2000a), REFP10L0 (USACE, 2008) and EM 1110-2-569 (USACE, 2005). This report summarizes the geotechnical analyses and presents recommendations for remedial action where applicable. The following analyses were performed:

- Seismic Design Criteria (Section 6.0)
- Liquefaction Potential (Section 8.1)
- Permeability and Seepage (Section 8.2)
- Slope Stability (Section 8.3)
- Seismic Deformation (Section 8.4)
- Settlement (Section 8.5)

1.2 *Authorization*

This study was authorized by the County in the Agreement for Professional Services under County Project No. 251059.

1.3 *Limitations*

The factual findings for this investigation, described herein, are applicable for the project description as contained in Section 1.1. The factual information presented in this report was documented by CGI solely for the County for use in evaluating the ability of the existing Jacobs Avenue Levee system within the project extent to function in accordance with current USACE performance criteria. Although information contained in this report may be of some use for other purposes, it may not contain sufficient information for other parties or other uses.

The scope of services did not include any environmental assessments for the presence or absence of hazardous/toxic materials in surface water, groundwater, or atmosphere. Any statements, or absence of statements, in this report or data presented herein regarding odors, unusual or suspicious items, or conditions observed are strictly for descriptive purposes and are not intended to convey engineering judgment regarding potential hazardous/toxic assessment.

In performing our professional services, we have used generally accepted geologic and geotechnical engineering principles and have applied that degree of care and skill ordinarily exercised under similar circumstances, by reputable geotechnical engineers and engineering geologists currently practicing in this or similar localities. No other warranty, either express or implied, is made.

2.0 SITE DESCRIPTION

The Jacobs Avenue Levee system is approximately 5,100 feet long and is located entirely on the right (north) side of the Eureka Slough, as shown on Plate 2. The levee system consists of one contiguous embankment located south of Jacobs Avenue, within the City of Eureka. We understand from the County that the eastern terminus of the levee intersects a former railroad embankment situated parallel to Airport Road. The starting point for our stationing is located beyond this intersection, near a bridge on Airport Road crossing a drainage ditch. Based on discussion with the County, we understand that the levee begins at approximately Station 10+30. The western terminus of the levee intersects Highway 101 at Station 61+00. The approximate center point of the levee has the following coordinates:

TABLE 2.1 – PROJECT LOCATION		
	Degrees, Minutes. Seconds	Decimal Degrees
Latitude	40° 48' 12.3"	40.803403°
Longitude	-124° 7' 45.9"	-124.129414°

Those coordinates were utilized, where necessary, for analyses within this report.

The overall levee configuration varies between the western and eastern segments, as discussed below. Refer to Plates 8.1 and 8.2 – Levee Topography, for elevations and topographic relief of the site

2.1 *EASTERN LEVEE SEGMENT (STATION 10+30 TO 42+00)*

The eastern section of the levee, extending from Stations 10+30 to about 42+00 (see Plate 2) ranges in height from about 6 to 9 feet above ground surface and has a crown width ranging from about 7 to 12 feet. The waterside slopes are inclined at about 1:1 (horizontal:vertical) to 2.5:1, with local areas, especially near the toe of slope, being vertical to near vertical. Landside slopes have slope inclinations ranging from about 1.5:1 to about 2:1.

Rip rap and concrete rubble have been placed at the toe of the waterside slope from about Station 10+50 to 35+90. An interior drainage ditch is located along the landside toe of slope from about Station 24+00 to 42+00. Chain link fences extend along most of the levee crest, with some intermittent gaps, such as at about Station 32+00.

Two penetrations are present along this segment of the levee. At about Station 12+00, a concrete gate structure is present that penetrates the levee and controls inland drainage into the Eureka Slough. We understand that Caltrans services and operates that gate at this time. At about Station 18+50 is another concrete penetration that at one time served the inland drainage control now being served by the penetration at Station 12+00, but that was abandoned sometime between 1953 and 1956. The methods used to abandon this penetration are unknown but, based on the waterside facing of the structure, appears to have consisted of placing concrete to reduce the potential of flows through the penetration.

The eastern levee segment described herein, is generally moderately to heavily vegetated with seasonal grasses, shrubs, brambles and local trees. Locally, brush is sufficiently thick to limit access or observation of the levee. Some property owners have maintained the levee relatively free of vegetation, such as between about Stations 10+00 and 20+00.

2.2 *WESTERN LEVEE SEGMENT (STATION 42+00 TO 61+00)*

The western section of the levee extends from Stations4 2+00 to the western levee terminus at Highway 101. The western levee ranges in height from about 5 feet to 8 feet above ground surface. The levee crown in this section ranges in width from about 3 to 5 feet. Landside and waterside slopes range in inclinations from about 1:1 to 3:1. At lower tides, an intertidal bench is visible at the toe of the waterside slope.

An interior drainage ditch is located along the landside toe of slope along this entire levee segment. Chain link fences extend along most of the levee crest or landside toe of levee along the entire segment.

Two penetrations are reportedly present along this segment of the levee, as shown on Plate 2. One penetration is located at about Station 46+00 and the other near the western terminus of the levee. The penetrations were not observed during this study, even at relatively low tides, and we have no record on how they were constructed. They may be obscured by intertidal sediments. Both penetrations likely drain the inland ditch that extends along the landside toe of levee slope along the entire length of this levee segment and a portion of the eastern levee segment.

The western levee segment described herein, is generally moderately to heavily vegetated with seasonal grasses, shrubs, brambles and local trees. Locally, brush is sufficiently thick to limit access or observation of the levee. Some property owners have maintained the levee relatively free of vegetation, such as at about Station 42+00, where the County has periodically reduced vegetation thicknesses.

3.0 FIELD EXPLORATION

3.1 *FIELD EXPLORATION PLANNING*

3.1.1 Review of Previous Reports

No previous geotechnical reports or information specific to construction of the levee were available for review during the course of this study. A feasibility study was performed by Winzler and Kelly Consulting Engineers (1985) to repair and/or reconstruct the Jacobs Avenue Levee. Current hydrologic analyses were performed for the levee system by Northern Hydrology & Engineering (NHE) and made available during this study.

Those data consisted of:

- Technical Memorandum, Jacobs Avenue Levee Bathymetric, Hydrologic and Hydraulic Study (NHE, 2016);
- Feasibility Study, Jacobs Avenue Dike, Winzler and Kelly Consulting Engineers, (Winzler, 1985)

These documents and additional reports are referenced in Section 10 of this report.

3.1.2 Exploration Plan

CGI prepared an exploration plan for performing and executing the subsurface exploration for this study. The plan was described, in general, in our proposal to provide geotechnical services for this study, and in greater detail in maps forwarded to the County on November 3, 2015. The work plan, spreadsheet, and maps detailed the following:

- Exploration numbers, locations and methods, including:
- Station number for each exploration;
- Relative locations on the levee, levee toe, or offsets for each exploration;
- Target depths of each exploration;
- Dates or date ranges that each exploration were to be performed; and
- Methods of destruction of each exploration following completion.

This information was used for landowner notification by the County, as discussed in Section 3.1.3. In addition, a health and safety work plan was prepared for field personnel during exploration.

3.1.3 Site Access & Permitting

Exploration for this project was performed on private and public properties. Public properties included lands or easements controlled by the County. Private lands owned by five individuals/companies were located in areas of our exploration along proposed cross section lines. The County secured permission to access and explore private properties at selected locations along the levee system.

Well and Boring permits from Humboldt County Division of Environmental Health were required for the exploration performed for this study. Two Well and Boring permits were acquired for this project: one for cone penetrometer test soundings (CPT) and one for drilling. In addition, permits from the California Coastal Commission, California Fish & Wildlife Service, California Regional Water Quality Control Board (North Coast Region), State Water Resources Control Board, and the USACE were obtained for this study. The County, with some assistance from CGI, acquired all permits for this project.

3.1.4 Site Visit and Utility Clearance

Prior to mobilizing exploration equipment to the site, each exploration location was visually inspected for the presence of overhead and underground utilities, and then marked with white paint, as required by Underground Service Alert (USA). Following the marking of the exploration locations, USA was contacted a minimum of 48 hours prior to subsurface investigation of the site. A total of five USA ticket numbers, as well as clearance dates, expiration dates, and call-back-to-extend dates, were obtained for the project and documented in the project file.

No utility conflicts were identified at proposed exploration locations. No manmade subsurface obstructions were encountered during our field exploration program. It should be noted that other buried utilities and structures not located by USA or observed by CGI may be present along the project extent.

3.2 *Exploration Program*

The field exploration program completed by CGI for the Jacobs Avenue Levee Evaluation project consisted of advancing subsurface explorations using CPT and rotary-wash drill holes. The following sections discuss those exploration services.

3.2.1 Subsurface Exploration

The locations of drill holes advanced for this investigation are shown on Plate 3.1 through 3.4. Information for each exploration including total exploration depth are tabulated in Table 3.1.

3.3 *Reference Datum*

The following reference datums were used for this project:

Horizontal Datum. The horizontal datum reference for this project is California State Plane, Zone 1, North American Datum 83 (NAD83, feet).

Vertical Datum. The vertical datum reference for this project is the North American Vertical Datum of 1988 (NAVD88, feet).

Coordinates and elevations for the exploration locations were estimated by CGI during exploration performed in November 2015 and January 2016, using a Garmin *etrex* hand-held global positioning system receiver.

TABLE 3.1 – SUBSURFACE EXPLORATION SUMMARY			
Exploration No.	Exploration Depth (feet)	Start Date	Completion Date
CPT-10L	50.4	11/19/15	11/19/15
CPT-10off	25.6	11/19/15	11/19/15
CPT-20L	25.5	11/19/15	11/19/15
CPT-30C	75.4	11/20/15	11/20/15
CPT-30off	25.6	11/19/15	11/19/15
CPT-30W	22.3	11/22/15	11/22/15
CPT-42L	50.9	11/20/15	11/20/15
CPT-42off	25.6	11/20/15	11/20/15
CPT-56C	20.1	11/21/15	11/21/15
CPT-56L	23.7	11/20/15	11/20/15
CPT-56W	16.7	11/21/15	11/21/15
DH-10L	26.5	1/14/16	1/14/16
DH-20off	26.5	1/13/16	1/16/16
DH-30C	51.5	1/12/16	1/12/16
DH-30L	26.5	1/13/16	1/13/16
DH-42L	26.5	1/12/16	1/12/16
DH-56L	26.5	1/11/16	1/11/16
DH-56off	26.5	1/14/16	1/14/16

One drill hole was advanced through the levee crest (Station 32+00) to depths of 50 feet. Limited clearance and access, limited crown width, and vegetation cover prevented drilling though the levee crest at the other four cross sections.

Soil samples were obtained at selected depth intervals within the drill holes using California modified split-spoon, Standard Penetration Test (SPT), and Shelby tube samplers. A CGI engineering geologist logged the drill holes as they were advanced. All samples were transmitted to Caltrans District 1 laboratory for testing, with some samples later being delivered to Caltrans Translab facility in Sacramento, California.

Eleven CPT soundings were performed for this study. The depths of the CPT soundings ranged from about 17 to 75 feet. Planned depths of CPT soundings in the levee crown was a minimum of 50 feet, which was achieved during this study. Planned depths of CPT soundings in explorations aside from the levee crown was 25 feet, which was achieved in those CPT sounding except at CPT-30W, CPT-56L, CPT-56W, which extended to depths ranging from about 17 to 24 feet.

3.4 *Investigation Derived Cuttings Disposal*

All drill holes were destroyed using cement-grout. The grout was placed through tremie tubes extended to the hole bottom and extracted as the grout was being placed to reduce the potential for bridging or collapse of the holes prior to grout placement. Because those holes were backfilled using cement grout, the cuttings required disposal. The cuttings were barreled for the County's disposal.

CPT soundings displace soil materials during the sounding process and do not generate cuttings that require disposal.

4.0 LABORATORY TESTING PROGRAM

All laboratory testing for this project was performed by Caltrans, District 1 based on testing schedules prepared by CGI. The majority of the testing was performed in the District 1 laboratory with some samples forwarded to Caltrans Translab facility located in Sacramento.

The purpose of the laboratory testing program was to supplement field classification of soils and provide relevant physical indices and engineering properties of the subsurface materials. The primary objectives of the program were to:

- Classify and characterize sampled subsurface materials;
- Evaluate the existing in situ conditions; and
- Develop relevant consolidation, strength, and permeability estimates of selected subsurface materials.

To meet these objectives, various tests were performed on selected samples. Test types are generally grouped into the following categories: classification/index tests, moisture content/density evaluations, consolidation tests, permeability tests, relevant strength tests, and subgrade characterization tests. Classification/index and subgrade characterization tests were performed on both disturbed and relatively undisturbed samples, including 2.4-inch diameter drive (i.e., ring) samples, 3-inch diameter Shelby tubes, Standard Penetration Test (SPT) samples, and bulk samples. Density evaluations, consolidation tests, permeability tests and strength tests were typically performed only on relatively undisturbed ring and/or Shelby tube samples.

The numbers of the various tests conducted for the Jacobs Avenue Levee Evaluation are listed in Table 4.1.

TABLE 4.1 – SUMMARY OF LABORATORY TESTS PERFORMED ON SELECTED SAMPLES

Laboratory Testing	Number of Tests	ASTM Test Designation[1]
In-Situ Moisture Content	16	ASTM D2216
In-Situ Dry Density	24	ASTM D7263b
Sieve Analysis with #200 Wash	28	ASTM D422
Atterberg Limits	9	ASTM D4318
Modified Proctor	1	ASTM D1557
Consolidation (Incremental Load Control)	2	ASTM 2435
Flexible Membrane Permeability	3	ASTM D5084
Direct Shear	5	ASTM D3080

[1] ASTM International , latest edition.

Laboratory test results are tabulated or presented graphically in Appendix C. Various laboratory test results are tabulated versus depth on the individual drill hole logs (Plates B-2.1 through B-2.7). Test results that cannot be conveniently tabulated or plotted versus depth on logs are also provided in Appendix C. Test results in this category include: grain-size curves, plasticity charts, direct shear, permeability, consolidation, and compaction test results.

5.0 GEOLOGIC AND SUBSURFACE CONDITIONS

5.1 *Regional Setting*

The project site is located in the Coast Ranges Geologic/Geomorphic Province of Northern California. The Coast Ranges province consists of an approximately 50-mile wide range of mountains extending from Santa Barbara County approximately 400 miles northward into Shasta and Humboldt Counties (Hinds, 1952). It is bounded to the north and east by the Klamath Mountains province, to the south by the Transverse Ranges province, to the southeast by the Great Valley province, and to the west by the Pacific Ocean.

The project region occupies a complex geologic environment characterized by high rates of active tectonic deformation and seismicity. The area lies immediately north of the Mendocino Triple Junction, which is the location of intersection of three crustal plates (e.g., the North American, Pacific, and Gorda plates). North of Cape Mendocino, the Gorda plate is being actively subducted beneath North America, along what is commonly referred to as the Cascadia subduction zone, which is located west and offshore of the study area. In Humboldt County, stresses from the offshore subduction zone are manifested on-land as a series of northwest-trending, southeast-vergent thrust faults, and intervening folds (i.e., "fold and thrust belt"). The geomorphic landscape of the project region is largely a manifestation of the active tectonic processes and the setting in this dynamic coastal environment.

Basement rock within the Eureka region consists of the Cretaceous-Jurassic Franciscan Formation (McLaughlin et al., 2000). The Franciscan Formation is a sequence of metasedimentary, metavolcanic, metamorphic, and ultramafic rocks that have been and are continuing to be accreted to the North American plate during subduction of the Pacific plate. As a consequence, the Franciscan Formation has been mapped as three broad belts (Eastern, Central, and Coastal belts) that become younger from east to west (McLaughlin et al., 2000).

The project is situated on Neogene-age overlapping sediments that rest on the Yager Terrane of the Coastal Belt (McLaughlin et al., 2000). The Yager Terrane is composed of Late Cretaceous argillite, sandstone, and conglomerate (McLaughlin et al., 2000). The Neogene-age materials consist of sediments of the Plio-Pleistocene-age Hookton and Yager Formations.

In turn, the region is juxtaposed to the relatively older Central Belt of the Franciscan Formation, located east of the site. In the project region, it is composed of intercalated blocks of sandstone and shale turbidites, chert, limestone, and metabasalts (McLaughlin et al., 2000), often in a matrix of sheared argillite.

5.2 *Local Geologic Setting*

Coastal margins in the project area reflect the late Quaternary history of sea level changes and tectonic deformation. During most of the late Quaternary, sea level was lower than its present position, resulting in a shoreline located farther to the west, and a lower fluvial base level to which coastal wave-cut platforms and all coastal streams would be graded. During these low sea levels, wave-cut platforms and streams within the coastal valleys around Humboldt County would be incised. In addition, wave-cut platforms would be exposed and uplifted during the low sea level stands, creating marine terraces along the coastal zone. It is thought that sea level reached its current high level in the mid-Holocene, about 6,000 years ago. As such, most of the near-surface sediments filling the Eureka Slough and Humboldt Bay would be anticipated to be mid-Holocene in age, or younger. A geologic map of the project area is shown on Plate 5 – Regional Geologic Map.

The Jacobs Avenue Levee is itself underlain by a mixture of Holocene-age intercalated bay muds, estuarian deposits, and alluvial deposits. The bay muds and estuarian deposits consist of marine and terrestrial fat clay, organic clays, and elastic silts. Interspersed with those fine-grained sediments are silts, sands, and trace fine gravel alluvial layers associated with high flow events from Freshwater Creek and other unnamed tributaries confluencing with the Eureka Slough. In addition, some littoral granular deposits associated with tsunamis that have affected Humboldt Bay might also be present within the quaternary near-surface sediments.

Directly underlying the near-surface sediments are Quaternary-age marine terrace deposits and the Hookton Formation. McLaughlin et al. (2000) map marine terrace deposits projecting beneath the Holocene-age sediments in the project area but Kelley (1984) and Kilbuorne et al. (1980) indicate that the Hookton Formation directly underlies the younger sediments.

5.3 *Geomorphology*

The Eureka Slough is 20.8 miles long with 80 percent of its shoreline composed of artificial structures mainly composed of dikes/levees with a minor component of roadway (Laird et al, 2013). Thus, the slough is largely fortified and contained by artificial structures. Prior to fortifying the shoreline, the slough consisted of a number of intertidal channels draining into the slough, which discharges into Humboldt Bay. This is illustrated on Plate 6 – Former Tidal Channels and Improvements Circa 1950. As can be seen on that plate, at least 10 intertidal channels in the project area were truncated by construction of the levee, yet remained on the landside of the impoundment.

The slough is a sinuous channel fed by moderately to highly sinuous drainages. At high tide, the slough is a single channel but at low tide, the channel becomes bifurcated exposing intertidal bars south of the western two-thirds of the levee alignment.

Comparison of historical topographic maps dated between 1921 and 2015 indicate that the slough channel has remained relatively stable during that period of time with slight channel variations in the eastern third of the study area. Some of this change was likely due to erosion prior to construction of the levee.

5.4 *Subsurface Conditions*

As previously noted, the project field exploration program included CPT soundings and rotary-wash drill holes along the crest of the levee, at the landside and waterside toes, and at landside levee offset locations within the project extent. Subsurface conditions were explored to a depth of up to approximately 75 feet below ground surface (see Section 3.0 for details of the field exploration program). Subsurface conditions are described and illustrated in the drill hole logs (see Appendix B). The logs have also been projected onto cross sections to depict subsurface conditions across and along the levee system. See Plate 2 shows cross section locations and those cross sections are presented on Plates 4.1 through 4.5 – Geologic Cross Sections. Five transverse cross sections were created by projecting exploration logs onto the topography along the cross sections.

5.4.1 Earth Materials

Subsurface conditions are generally similar throughout the project site, although some variability is seen along the levee system. In general, the embankment materials within the levee were relatively similar along the length of the levee system, whereas foundation soils are relatively variable. Earth materials encountered within the levee embankment and in the foundation soils are discussed in the following sections. Logs of the explorations for this study are presented in Appendix B. Laboratory data for earth materials are presented in Appendix C.

Levee Embankment Materials. Levee embankment thicknesses appear to range from about less than 3 feet to up to about 9 feet, as estimated from cross sections presented on Plates 4.1 through 4.5. A total of 3 explorations, consisting of rotary-wash drill holes (1) and CPT soundings (2), were advanced through the levee embankment materials to sample and characterize selected geotechnical conditions within the embankments.

Subsurface conditions within levee embankment materials were observed to be generally consistent throughout the levee system. The explorations encountered gravelly silt, to silt within the levee at all sections. The embankment materials were generally moist to wet with depth and ranged from slightly stiff to stiff. The embankment was not constructed with an impervious core (normally clay), which is standard practice for an engineered levee. The lack of a low-permeability core allows the potential for water to seep through the embankment more readily and can lead to piping, through seepage, and other factors which can lead to destabilization of a levee. An impermeable core will typically eliminate or significantly reduce the potential for seepage forces to adversely affect a levee, which is why impervious cores are design and constructed into levee systems.

Foundation Soils. Prior to this study, it was uncertain whether impervious blanket materials and pervious foundation soils are present beneath the levee embankment. It was found that the levee foundation is composed of an impervious blanket layer overlying pervious material. The impervious blanket materials reduce the ability of seepage movement beneath the levee from occurring quickly or with great volume, thus, inhibiting seepage forces from destabilizing the foundation beneath the levee. Impervious blanket materials typically reduce the seepage exit gradients, which, in turn reduce the potential for piping of sediments from the foundation layer underlying the levee.

The following sections discuss foundation soils encountered during this study.

Impervious Foundation Blanket Materials. Fine-grained materials consisting of sandy silt to clayey silt are present beneath the levee embankment soils. These soils vary in thickness from 5 to 17 feet. As shown on Plates 4.1 through 4.5, impervious materials were encountered on both the landside and waterside of the levee at each location explored.

Pervious Foundation Materials. Subsurface exploration indicated pervious foundation soils consisting of silty sand to sand are present beneath the impervious blanket materials. These pervious foundation materials consisted of loose to dense, wet, poorly- to well-graded sand, silty sand, sand with silt, sand with silt. The pervious foundation soils were not fully penetrated by explorations performed for this study.

5.4.2 Groundwater

Depths to groundwater were measured in CPT soundings advanced during this study. The observed groundwater levels varied from about 1.8 to 7.9 feet below ground surface at crown locations and 0.0 to 3.7 below ground surface at other locations. Because drilling was performed using rotary-wash methods, groundwater depths were not observable during the time-frame available for exploration at each drill hole. Table 5.1 summarizes the groundwater levels observed in the field.

TABLE 5-1 – GROUNDWATER LEVEL OBSERVATIONS IN STUDY EXPLORATIONS

Station & Measurement Date	Groundwater Elevations (feet below ground surface)			
	Offset	Landside Toe	Crest	Waterside Toe
Station 10+75	3.7	2.7		
Date	11-19-15	11-19-15		
Station 21+50		2.0		
Date		11-19-15		
Station 32+00	1.8		7.9	1.7
Date	11-19-15		11-20-15	11-22-15
Station 42+00	2.5	2.5		
Date	11-20-15	11-20-15		
Station 56+00		2.9	1.8	0.0
Date		11-20-15	11-21-15	11-21-15

Groundwater levels in the project area were also researched through the California Department of Water Resources (2015) and through the California State Water Resources Control Board Geotracker database (2015). Neither database had groundwater data sources located within 1,000 feet of the study site.

In general, groundwater levels were measured to be at relatively consistent depths beneath the ground surface along the levee system. Some variations in depths to water were observed; however, in a depositional environment that has the potential to have significant changes in soil types over relatively short distances, variations in groundwater depths should be anticipated. None of the variations measured during this study imply the presence of a significant aquitard or aquiclude impeding horizontal groundwater flow through sediments in the vicinity of the levee system. Variations in groundwater levels and soil moisture conditions can occur as a result of rainfall, runoff, and other factors. Therefore, groundwater conditions should be assumed to fluctuate.

6.0 SEISMIC DESIGN CRITERIA

6.1 *Strong Ground Motion*

As part of our geotechnical analyses, a probabilistic seismic hazard evaluation was conducted with the aid of the United States Geological Survey (USGS) 2008 Interactive Deaggregation (beta) website. The USGS website provides probabilistic estimates of seismic hazards using three-dimensional earthquake sources. In accordance with ETL 1110-2-580 (USACE, 2013), we utilized the website to determine the peak ground acceleration estimated to have approximately 1 percent annual chance of exceedance (Statistical Return Period ≈ 100 years).

Based on correlations developed by Sykora (1987), shear wave velocity data were estimated for the subsurface conditions encountered at the site. The average shear wave velocity for the upper 100 feet was estimated to be approximately 155 meters per second (m/s), based on data collected from exploration CPT-30C.

The peak ground acceleration was calculated by the USGS website using equally-weighted averages of three (3) attenuation relationships: Boore and Atkinson (2008), Campbell and Bozorgnia (2008), and Chiou and Youngs (2008). For the purposes of our evaluation, the site coordinates (latitude and longitude) for the Jacobs Avenue Levee project site were estimated to be 40.803 degrees north latitude and 124.129 degrees west longitude.

According to the USGS website, the peak horizontal ground acceleration (PHGA) for the project site is estimated to be approximately 0.25g, corresponding to deaggregated modal and mean magnitudes of 6.99 and 6.91, respectively. Based on the deaggregation graph generated by the USGS website, a magnitude 9.0 event also contributed a relatively large percentage to the estimated seismic hazard. Therefore, we considered a magnitude 9.0 earthquake with a PHGA of 0.25g as the design seismic event for our evaluation of the levee system.

6.2 *Surface Fault Rupture*

The State of California designates faults as active, potentially active, and inactive depending on the recency of surface fault rupture that can be substantiated for a fault. Table 6.1 presents the current California fault activity ratings.

TABLE 6.1 – FAULT ACTIVITY RATINGS

Fault Activity Rating	Geologic Period of Last Rupture	Time Interval (Years)
Active	Holocene	Within last 11,000 Years
Potentially Active	Quaternary	>11,000 to 1.6 Million Years
Inactive	Pre-Quaternary	Greater than 1.6 Million Years

The California Geologic Survey (CGS) evaluates the activity rating of a fault in fault evaluation reports (FER). FERs compile available geologic and seismologic data, and evaluate if a fault should be zoned as active, potentially active, or inactive. If an FER evaluates a fault as active, then it is typically incorporated into a Special Studies Zone in accordance with the Alquist-Priolo Earthquake Hazards Act (AP). AP Special Studies Zones require site-specific evaluation of fault location and require a structure setback if the fault is found traversing a project site.

The site is not located within an Alquist-Priolo Earthquake Fault Zone and no active faults are known to pass through the project site (McLaughlin et al. 2000; Kelley et al., 19804; Kilbourne et al., 1980; Jennings, 1994; Hart & Bryant, 1997; Jennings & Bryant, 2010). However, a number of regional and local faults traverse the project region. The closest of these faults is the potentially active Freshwater fault located about 2.2 miles east of the site. A number of active faults are located relatively nearby: the Faylor fault (4.7 miles northeast); Mad River fault (5 miles northeast); Little Salmon fault (5.7 miles southwest); and Fickle Hill fault (6.2 miles northeast), as shown on Plate 7 – Regional Fault Map.

In addition to the continental faulting noted above, the project area rests above the Cascadia subduction zone. West of the site, off the coast of California, the oceanic crust of the Gorda plate is being subducted beneath the continental crust of the Pacific Plate, in an area known as the Gorda Escarpment. The descending ramp caused by that subduction, called the Cascadia Subduction zone, extends beneath the project area at a depth of about 9 to 12 miles (McLaughlin et al., 2000). That ramp is capable of storing elastic stress that periodically causes earthquakes that could affect the project area.

7.0 CROSS SECTIONS FOR ANALYSES

Five cross sections were selected for seepage and stability analysis, as shown on Plates 3.1 through 3.4. Cross sections are shown on Plates 4.1 through 4.5. Selected cross sections are located an average of about 1,000 feet apart. Cross sections are named based upon their approximate station number along the levee alignment. Those stationings are shown on Plate 2. Table 7.1 presents cross section station names and the corresponding stations established during this study along the Jacobs Avenue Levee crown and corresponding stationing for hydrologic cross sections presented by NHE (2016).

TABLE 7.1 – CROSS SECTION STATION NUMBERS

Cross Section Name	Station Number	
	Actual Station No.	NHE (2016)
10+75	11+50	8228
21+50	21+50	7477
32+00	31+80	6323
42+00	42+40	5262
56+00	56+10	3572

Cross section topography was estimated based on LiDAR, site specific survey, and bathymetry data provided by the County for the study area. Subsurface conditions were modeled as levee fill overlying fine- and coarse-grained alluvial deposits, which were designated as impervious and pervious foundation materials, respectively. The following paragraphs describe levee subsurface conditions modeled in seepage and slope stability analyses. Beneath the levee, alluvium was modeled as pervious material or as impervious material (blanket) overlying a pervious substratum.

Levee Embankment. No project plans and specifications were available to show details of how the levee was constructed. The levee appears to be constructed from material derived locally, possibly through dredging or drag lining sediments from Eureka Slough. As previously discussed, the levee embankment does not appear to have an impervious core or a zoned embankment having low-permeability soils placed on the waterside levee face, both of which are utilized consistently in levee construction. For our modeling of seepage and stability, we used no core and a uniform embankment with engineering properties obtained from testing materials sampled during this study.

Impervious Blanket. Soils encountered directly beneath the levee embankment materials consisted of sandy to clayey silt. These materials have relatively low hydraulic conductivity and range in thickness from 5 to about 17 feet.

Pervious Material. Pervious materials consisting of silty sand and sand are present beneath the impervious blanket materials. We modeled these materials as extending unobstructed beneath the impervious materials at each cross section.

Rip-rap & Cobble Armoring. Armoring consisting of concrete fragments, cobbles and boulders were observed mainly at the eastern half of the levee waterside face; however they were not included in cross sections modeled during this study due to the relatively minimal influence those materials will have on slope stability and seepage.

Design Water Surfaces. For existing conditions, creek water surfaces were modeled to generally correspond with groundwater measurements collected during our field exploration program, field observations and mean high elevations provided by NHE (2016). Groundwater elevations were estimated based on data collected during our field exploration efforts, between November 19 and January 14, 2016; groundwater data are presented in Table 5.1 and on Plates 4.1 through 4.5.

For flood-stage conditions, water surfaces were modeled to represent 100-yr coastal stillwater flood as defined by NHE (2016).

8.0 GEOTECHNICAL ANALYSES

8.1 *Liquefaction*

Liquefaction triggering analyses were performed for CPT soundings advanced through the levee crest. The computer program LiqIT (Geologismiki, 2006) was used to:

1) Perform the liquefaction triggering analyses according to the 1998 National Center for Earthquake Engineering Research (NCEER) guidelines (NCEER, 1998), and
2) Estimate seismic settlement using Ishihara and Yoshimine (1992).

The results of the analyses are presented on individual plots in Appendix D.

Liquefaction analyses were performed as follows:

- In accordance with ETL 1110-2-580 (USACE, 2013), groundwater depths corresponding to the median annual water surface elevation were assumed to approximate existing phreatic conditions.
- Liquefaction triggering was evaluated for a PHGA of 0.25g and a moment magnitude of 9.0, corresponding to a statistical return period of 108 years. As noted in Section 6.1, a magnitude 9.0 event is estimated to be the modal event for the deaggregated seismic hazard and contributed a relatively large percentage to the estimated seismic hazard. It should be noted that the minimum magnitude scaling factor defined by NCEER (1998) was applied in our analyses because values are not defined for magnitudes greater than 8.5.
- Liquefaction analyses are based on estimates of two values: the cyclic stress ratio (CSR), which characterizes the seismic demand on the soil, and the cyclic resistance ratio (CRR), which characterizes the soil's capacity to resist liquefaction. CSR and CRR versus depth were calculated using 1998 NCEER guidelines (NCEER, 1998).
- Seismic settlement resulting from potential liquefaction was estimated for each drill hole advanced through the levee crest using procedures presented in Ishihara and Yoshimine (1992).

8.1.1 Liquefaction Triggering Analyses per NCEER (1998)

NCEER (1998) guidelines (Youd and Idriss, 2001) present deterministic liquefaction triggering procedures based on SPT and CPT data. The factor of safety (FS) against liquefaction is computed as the ratio of CRR to CSR. The results of the analyses are presented on the liquefaction plots (Appendix D). Those analyses found that the factor of safety against liquefaction was less than 1.0 in the drill holes evaluated. At location 30C the FS was less than 1.0 between the depth increments of about 31 to 36 feet and 40 to 42 feet beneath the levee crown. At location 56C the FS was less than 1.0 between the depth increments of about 5 to 7 feet and 21 to 25 feet beneath the levee crown.

8.1.2 Seismic Settlement per Ishihara and Yoshimine (1992)

Seismic settlement resulting from liquefaction was estimated using procedures presented in Ishihara and Yoshimine (1992). The procedure relates volumetric strain to the FS against liquefaction and normalized N values. Estimated volumetric strains are multiplied by layer thickness to obtain incremental settlement in each liquefiable layer. The increments are summed from the bottom of the exploration to the ground surface. On the basis of our analysis, we estimate the potential magnitude of seismically induced settlement to be up to about 3.42 to 4.36 inches. Cumulative seismic settlement versus depth estimates are presented in Appendix D.

8.1.3 Lateral Spreading

Lateral spreading is defined as lateral earth movement of liquefied soils, or soil riding on a liquefied soil layer, down slope toward an unsupported slope face, such as a creek bank, or an inclined slope face. In general, lateral spreading has been observed on low to moderate gradient slopes, but has been noted on slopes inclined as flat as one degree.

Evaluations using methods of NCEER (1997) in the program LIQit, lateral spreading magnitudes were estimated for the site. Those estimates indicate that up to 63 inches of lateral displacement could occur during the modeled seismic event.

8.1.4 Seismically Induced Dry Settlement per Pradel (1998)

Seismically induced dry settlement can occur in association with liquefaction, or in soils not prone to liquefaction (above the water table) that are loose to medium dense. Seismically induced dry settlement differs from settlement resulting from liquefaction of saturated granular materials, discussed above.

We estimated the potential for seismically induced dry settlement to occur at the site using procedures presented in Pradel (1998). In general, the granular materials encountered in our field explorations ranged from loose to very dense sandy silt, silty sand, sand, sand with gravel, gravelly sand, gravel with sand, sandy gravel, and gravel. The loose to medium dense granular materials may be susceptible to dry seismic settlement. Because the high fine content in the upper layers and the relatively limited thicknesses of material above modeled groundwater, we estimate the potential magnitude of seismically induced dry settlement to be less than one inch.

8.1.5 Results

The results of our seismically induced settlement analyses are summarized in Table 8.1 below, and presented graphically in Appendix D.

TABLE 8.1 – SEISMICALLY INDUCED SETTLEMENT ANALYSES RESULTS		
Section Station Number	**Liquefaction-Induced Settlement (inches)**	**Seismically-Induced Dry Settlement (inches)**
10+75	Not Evaluated	<1
21+50	Not Evaluated	<1
32+00	3.42	<1
42+00	Not Evaluated	<1
56+00	4.36	<1

8.2 *Permeability and Seepage*

The potential for seepage to influence the stability of a levee is dependent upon a number of factors that include:

1. The geometry of the levee;
2. The composition and engineering properties of the levee and foundation soils; and
3. The elevation of the water being retained by the levee.

The geometry and composition of the levee evaluated during this study are discussed in Sections 5.0 and 7.0 of this report. The water surface elevations of the 100-year flood event are discussed below. Estimated water surface elevations along the Jacobs Avenue Levee correspond to freeboards ranging from 0.70 to 2.55.

Two potential seepage pathways were evaluated:

1. Through-seepage, where water flows through the levee embankment and daylights on the landside face of the levee; and
2. Underseepage, where water flows through the levee foundation soils and daylights at the landside ground surface.

Analyses were performed using the finite element modeling program SLIDE (Rocscience, 2016).

Our approach to seepage analyses utilized generalized parameters that could be applied to all of the sections evaluated. Those parameters were derived from site observation and laboratory testing.

In accordance with EM 1110-2-1913 (USACE, 2000a), seepage analyses were performed for steady-state seepage during a full flood event. As noted above, water surfaces for coastal stillwater flood-stage conditions were modeled to represent the 100-yr water surface elevations, as defined by NHE (2016). Groundwater elevations for existing conditions were estimated on the basis of data collected during our field exploration, which are presented in Table 5.1 and on Plates 4.1 through 4.5. and water elevation representing mean high water (MHW) provided by NHE (2016)

8.2.1 Through-Seepage Analyses

The potential for through-seepage was estimated at each section by review of seepage analyses output to determine if the estimated phreatic surface emerges on the landside slope during the design flood event. As noted in USACE (2000a), seepage exiting on the landside slope could potentially decrease the stability of the slope as a result of sloughing or internal erosion of the slope.

8.2.2 Underseepage Analyses

Acceptance criteria have been established by the USACE to reduce the potential impacts of levee underseepage (USACE, 2005). Those criteria specify that underseepage beneath an existing levee is acceptable if the exit gradient is 0.5 or less, which would provide a factor of safety against seepage-related failures of at least about 1.6 (USACE, 2005).

If cross sections are modeled with landside impervious foundation (i.e. blanket) material, average exit gradients were estimated by dividing the difference in total head across the thickness of the blanket material by the thickness of the stratum. Factors of safety for underseepage estimated below are based on an assumed critical gradient value of approximately 0.8 for impervious and pervious foundation materials.

For steady-state seepage analyses, the landside extent of the seepage models was defined by the lesser of the perpendicular horizontal distance to the valley basin limits (bedrock slope), a distance of 2,000 feet from the levee centerline, or established groundwater condition (Arcata Bay).

To simulate the general hydrogeologic conditions we anticipate during a flood event, estimated groundwater elevations were applied as boundary conditions at the landside boundary to model the Arcata Bay groundwater levels which are the defining hydrologic limit conditions.

8.2.3 Results

The results of our seepage analyses for the five cross sections are summarized below.

TABLE 8.2 – SUMMARY OF RESULTS FOR SEEPAGE ANALYSES				
Section	Potential for Through-Seepage Estimated	Approximate Daylight Height Above Landside Toe (ft)	Exit Gradient at Landside Toe	
			Steady-State	Factor of Safety
10+75	Yes	2.1	0.2	3.8
21+50	Yes	2.5	0.3	2.5
32+00	Yes	1.9	0.4	1.9
42+00	Yes	2.9	0.2	3.8
56+00	Yes	1.4	0.2	3.8

8.3 *Slope Stability*

Slope stability analyses were performed for the five cross sections along the Jacobs Avenue Levee. The analyses were performed using the limit equilibrium program SLIDE (Rocscience, 2016). Our approach to slope stability analyses assumed generalized parameters for all of the sections evaluated. In addition, slope stability analyses focused on failure surfaces that intersected the levee crest. Shallow failure surfaces typically cut only a small portion of the levee slope and typically do not pose substantial threat to the integrity and safety of the levees (DWR, 2013).

8.3.1 USACE Acceptable Slope Stability Criteria

The USACE has established minimum factor of safety (FS) thresholds for levee slope stability conditions (USACE, 2000a). The FS against slope failure is estimated by calculating the forces resisting slope failure divided by the forces causing slope failure. Thus, a FS of greater than 1 implies a stable slope, a FS less than 1 implies a failing slope, and a FS equal to 1 implies that a slope is on the verge of failure.

The conditions for analysis specified by the USACE consist of four slope stability cases:

- Case I – End of Construction;
- Case II – Sudden Drawdown;
- Case III – Steady Seepage from Full Flood Stage; and
- Case IV – Earthquake.

Case I represents approximate as-constructed embankment conditions that would be present at the time when levee construction is just completed. Under this scenario, fine-grained soils (impervious embankment and foundation soils) are modeled using undrained (more likely unsaturated) shear strength parameters, whereas granular soils (pervious soils) are modeled using drained shear strength parameters. Slope stability analyses for this case are performed for both the landside and waterside slopes. Case I is typically not performed for existing levees.

Case II represents the scenario when a prolonged flood stage saturates the levee embankment and then recedes faster than the soil can drain (rapid drawdown). Slope stability analyses for this case are performed on the waterside slopes.

Conditions for **Case III** occur when a prolonged flood stage allows steady state seepage flow to occur through the levee. Under this scenario, the landside slope stability is evaluated.

Case IV represents the effects of earthquake forces on the levee stability.

The USACE (2000, 2013) has established minimum FS values for each levee slope stability case evaluated. Those allowable minimum FS values, presented in Table 8.3 below, were referenced in our interpretation of slope stability analyses discussed herein.

TABLE 8.3 – MINIMUM ACCEPTABLE FACTOR OF SAFETY VALUES

Case	Minimum Factor of Safety
Case I – End of Construction	1.30 to 1.50*
Case II – Rapid Drawdown	1.00 to 1.20**
Case III – Steady State Seepage	1.40
Case IV – Earthquake (pseudo-static)	1.00

* Frequently loaded levee
** FS of 1.0 applies to pool levels prior to drawdown for conditions where these water levels are unlikely to persist for long periods preceding drawdown. FS of 1.2 applies to pool levels likely to persist prior to drawdown.

8.3.2 Methodology

Shear strength parameters applied to each slope stability case noted above were derived from observation site condition, laboratory test results and applicable correlations. For our stability analyses of existing conditions and landside slope conditions during a full flood event, drained shear strength parameters were assigned to the subsurface materials. Phreatic surfaces and pore pressures used for slope stability analyses of full flood and rapid drawdown conditions were based on steady-state seepage results.

Rapid drawdown slope stability evaluations used the three-stage method, as described in Appendix G of EM 1110-2-1902 (USACE, 2003), and programmed into SLIDE. The three-stage method uses both drained and consolidated-undrained shear strength parameters, estimated as described in Appendix G of EM 1110-2-1902 (USACE, 2003) and DWR (2015).

For pseudo-static stability analyses of slopes with soils considered to be potentially liquefiable during the design seismic event, undrained residual strength parameters were estimated for potentially liquefiable strata according to the empirical correlation proposed by Seed and Harder (1990). Potential liquefaction is discussed in Section 8.1 and estimated undrained residual strength values for potentially liquefiable strata are presented in Appendix E.

We performed screening-level pseudo-static slope stability analyses by applying a horizontal seismic coefficient equal to approximately half of the estimated PGHA [i.e. 0.5×0.25g = 0.127g (rounded up to 0.13g]. If the estimated FS for the initial pseudo-static slope stability analyses was less than 1.0, we performed a seismic deformation analysis to estimate the levee's performance during the design seismic event. If the FS was estimated to be equal to or greater than 1.0, minimal or negligible seismic displacements are anticipated for the design seismic event and post-earthquake slope stability analyses were not performed (DWR, 2013).

8.3.3 Results

The results of our slope stability analyses for the three cross sections evaluated are summarized in Table 8.4 and presented graphically in Appendix E.

TABLE 8.4 – SUMMARY OF SLOPE STABILITY ANALYSES RESULTS

Section	Levee Slope	Factor of Safety For Each Case Evaluated			
		Existing Conditions	Seepage During a Full Flood Event	Rapid Drawdown	Pseudo-static
10+75	Landside	1.36	1.08		1.10
	Waterside	1.04		0.98	0.80
21+50	Landside	1.78	1.27		1.34
	Waterside	1.32		1.34	0.92
32+00	Landside	1.47	1.16		1.10
	Waterside	1.36		1.36	1.03
42+00	Landside	1.74	1.33		1.23
	Waterside	1.04		1.04	0.83
56+00	Landside	1.90	1.47		1.36
	Waterside	1.75		1.75	1.31

8.4 *Seismic Deformation*

As shown in Table 8.4, the estimated factor of safety for the screening-level pseudo-static slope stability analyses were lower than 1.0 for three of the sections evaluated. Therefore, slope stability analyses were performed to estimate seismic deformation. That displacement is estimated to be higher than 3 feet.

8.5 *Settlement*

In addition to liquefaction-induced settlement analyses, as discussed above, a qualitative evaluation of static settlement was also performed. The potential for static settlement can arise through several cases, including consolidation of underlying soils due to fill or structural loads at the ground surface; subsidence due to removal of groundwater or other subsurface fluids, such as petroleum; and hydroconsolidation due to saturation of collapsible soils.

Due to the age of the levee (on the order of 84 to 114 years or more for most of the project site and about 35 years for the eastern levee section where modifications were made in the 1980s), it is very likely that all of the consolidation of underlying soils has already taken place in response to the weight of the levee fill material. Additional settlements are possible if fill or structural loads are placed on or in close proximity to the levees.

The project site is not in an area where the withdrawal of groundwater or other subsurface fluids is known to have caused subsidence. There is a potential for settlement to occur under the levees if sufficient lowering of the groundwater table were to occur in close proximity to the project site.

Two scenarios that could induce such settlement are construction dewatering and large-scale pumping of groundwater for industrial, agricultural, or municipal purposes.

Hydroconsolidation, or hydrocollapse, is a phenomenon associated with soils that are prone to relatively rapid settling when subjected to wetting or saturation. Near-surface deposits of dry, porous soils where the particles are cemented with soluble salts are particularly vulnerable to hydrocollapse. The results of our subsurface exploration and laboratory testing did not indicate the presence of such soils within or below the levees.

9.0 CONCLUSIONS AND RECOMMENDATIONS

9.1 *Identified Deficiencies and Relative Severity*

As noted in the sections below, a number of deficiencies have been identified for the Jacobs Avenue Levee. The sections below provide details of the deficiencies and conceptual mitigation measures for reducing the flood protection risks associated with those deficiencies. The following table lists the identified deficiencies in decreasing order of relative corrective urgency, based on our opinion of risks posed by the deficiencies.

TABLE 9.1 – LEVEE DEFICIENCIES IN ORDER OF PERCEIVED URGENCY FOR MITIGATION

Urgency	Deficiency
Most Urgent	Levee Prism Geometry and Freeboard
↕	Slope Stability
	Seepage
	Erosion
	Vegetation Management
	Liquefaction
Least Urgent	Seismic Deformation

In addition, it should be noted that our evaluations are based upon hydrologic conditions as they exist at this time. Sea levels will continue to rise over time, which will adversely affect many aspects of levee stability including freeboard, seepage, and slope stability. These possible future changes could have additional future economic impacts on landowners located adjacent to the levee due to future mitigation measures needed to keep the levee stable.

9.2 *Levee Prism Geometry & Freeboard*

The geometry of the levee prism does not meet USACE standards for most of the levee alignment. These standards require a minimum of 3:1 waterside slopes, 2:1 landside slopes, and a 12-foot wide levee crown. For most of the levee, the waterside and landside levee slopes are too steep and the crown varies from about 12 feet wide at Station 10+50 to less than 3 feet wide at about Station 56+00. Many of the deficiencies noted for erosion, seepage, and slope stability are likely related to the lack of adequate levee prism geometry.

In addition, the freeboard is locally deficient along the levee, based on the hydrologic evaluations provided to us. A minimum of one-foot of freeboard is required above the maximum wave run-up or height of the one-percent wave, whichever is greater, and under no circumstances should freeboard be less than 2 feet above the 100-year stillwater surge elevation.

Mitigation for establishing an acceptable levee prism and freeboard consists of grading the levee to establish an acceptable geometry. Most likely this would consist of leaving the levee waterside toe in place and grading towards the north to avoid environmental impacts and permitting issues associated with grading work in the Eureka Slough. It is anticipated that soil materials meeting applicable specifications would need to be imported to the site to accomplish the grading. Impacts to some landowner improvements, such as buildings currently close to the levee landside toe, would occur and could require construction of retaining walls or demolition of those structures.

Many of the defects noted in the sections below are related to the lack of appropriate levee prism geometry and would likely be improved or eliminated if the levee geometry was reconstructed to meet USACE standards. Thus, we have ranked implementation of mitigations for levee prism geometry as the relatively most urgent mitigation associated with Jacobs Avenue Levee.

9.3 *Vegetation and Erosion*

According to USACE (2000b), terrain within 15 feet of the levee should be free of vegetation. Based on our observations, there is vegetation along portions of the waterside and landside levee slope areas. This vegetation includes seasonal grasses, shrubs, brambles, and trees.

According to the USACE, vegetation in these areas has the potential to compromise the functionality of the levee and limit access for operations and maintenance (USACE, 2009a and 2009b). In our opinion, based on observations at the site, the waterside vegetation, while not desirable according to the USACE, might not be an adverse condition to the stability of the levee system. The vegetation could induce more turbulent water and cause some mounding effect during high flows; however, the slough generally has relatively slack or low velocity waters, thus mounding and turbulent waters pose relatively low risks to the project. In addition, the vegetation appears to have helped anchor foundation soils along the waterside levee toes and likely helps stabilize channel banks, thus reducing erosion within these areas.

While the vegetation treatments should attempt to address specifications developed by the USACE, it is a much more complex issue that we understand is currently under consideration and development at the national and state level. Until updated guidelines are developed, we recommend that the levee owner initiate reasonable efforts to reduce vegetation along the waterside and landside levee faces and toes, but avoid treatments that would remove the vegetation that provides slope stability benefits.

We noted localized areas of erosion along the waterside levee slope along the levee. The erosion is discontinuous and occurs in areas where rip-rap is absent, and also in rip-rapped areas containing gaps in the rip-rap, insufficient amounts of rip-rap, or insufficient height of rip-rap. The erosion appears to be occurring due to wind generated waves lapping onto the waterside face of the levee are relatively high tides. In general, the erosion, where it occurs, is on the order of about a foot deep and creates a near-vertical face in the levee face embankment. In general, it is our opinion that the severity of the erosion is relatively low based on the localized nature of the erosion. However, regular monitoring for changing conditions is recommended.

Mitigation of the erosion would consist of removal of existing rip-rap in eroded areas, grading of the levee on the waterside to meet geometric standards, placement of a geotextile filter fabric, then placement of properly sized rip-rap onto the filter fabric. The rip-rap should extend to sufficient height to be above mean high tides.
The current state of erosion and vegetation on the levee slopes does not pose an imminent threat to the stability of the levee. Thus, in Section 9.1, they are ranked relatively low in regards to perceived urgency, based on the information available to us at this time.

9.4 *Liquefaction and Seismically Induced Dry Settlement*

There is a likely potential for liquefaction and seismically induced dry settlement of the pervious foundation material at the project site during the design seismic event, as described in Section 8.1 of this report. The maximum estimated liquefaction and seismically induced dry settlements are approximately 4.36 inches and less than 1 inch, respectively. In addition, there is a risk of lateral displacement in the order of 63 inches due to lateral spreading.

In our opinion, the need for remedial action should be based on an assessment of risk posed to adjacent improvements. The estimated levee settlement due to liquefaction and seismically induced dry settlement is anticipated to result in freeboards of less than 2 feet at any cross section evaluated.

It should be noted that the probability of a high-water flood event and design seismic event occurring simultaneously is considered to be relatively low. However, due to the tidal nature of the adjacent slough, the levee could be exposed to high water levels relatively soon if levee slopes settle during or after the design seismic event.

Based on these conclusions, it is our opinion that mitigations measures for liquefaction are necessary for the Jacobs Avenue Levee system. However, in the event of an earthquake prior to any mitigation measure, we recommend that the levees be inspected for signs of settlement, cracking, or failure as soon as possible following the earthquake and repaired as soon as conditions will allow.

Mitigation measures for liquefaction risks vary in scope and cost for settlement and lateral spreading. In general, mitigation of liquefaction-induced settlement is relatively less expensive and simpler than mitigation measures for lateral spreading. Both liquefaction phenomenon are mitigated using similar techniques; however, settlement mitigations are performed more locally, whereas, lateral spreading requires a regional mitigation which would encompass much of the landside areas located between the levee and Jacobs Avenue.

Mitigation measures for liquefaction (both settlement and lateral spreading) consist of ground modification methods, such as the installation of stone columns, rammed aggregate piles (geopiers), and injection grouting. Deep dynamic compaction (DDC) is an additional method but due to the presence of existing structures and buried infrastructure that would be damaged by DDC, this method has been excluded.

Ground modification methods typically involve the penetration of the liquefiable zone with materials that lateral displace and densify those liquefiable soils to reduce the potential for liquefaction to occur. For stone columns and geopiers, this typically consists of the drilling of a pilot hole to a target depth, then the placement of gravel within the drill hole. For stone columns, the gravel is vibrated into place as it is placed. For geopiers, the aggregate is rammed as it is placed. With displacement or compaction grouting, a pilot hole is advanced to a target depth then grout is injected under pressure to displace the adjacent soils. In all cases, the materials placed within the holes displace and densify the adjacent soils.

Typically, the locations of each of the above noted mitigations are made on a grid spacing. The distance between each mitigation point and the depth of mitigation are typically determined by the specialty subcontractor based on subsurface information developed for the area of concern. For mitigating settlement, the grid spacing would likely occur along the levee alignment to a nominal distance on the landside and waterside of the levee. For lateral spreading, the grid spacing would encompass the same area as for settlement and also much of the area between the levee and Jacobs Avenue.

Liquefaction has a high potential to destabilize the levee, potentially rendering it ineffective as a flood control improvement. However, liquefaction only occurs when there is a relatively strong near-field earthquake, which happens relatively infrequently. So, while the consequence to the levee from liquefaction is high, the probability of occurrence is low for any given year. Thus, we have ranked the urgency for mitigation as relatively low because the likelihood of liquefaction to occur soon is considered low.

9.5 *Permeability and Seepage*

9.5.1 Underseepage

The steady state seepage analyses resulted in estimated vertical exit gradients ranging from about 0.2 to 0.4 at the levee landside toe, for the conditions analyzed at each of the five cross section locations. According to USACE (2005), the allowable FS for underseepage corresponds to a maximum exit gradient of 0.5. Therefore, each of the sections evaluated have a FS greater than that required by the USACE and are considered stable. Based on these conclusions, it is our opinion that mitigations measures for underseepage are not necessary for the Jacobs Avenue Levee system.

That being said, there are no data for the backfill surrounding the penetrations that cross the levee or the former channels that were present prior to levee construction and that cross the levee alignment, as shown on Plate 6. That backfill in these areas should be exposed to assess the composition and characteristics of the materials. If relatively plastic clay or PCC materials were not used to backfill around the penetrations and former channels then mitigations to reduce the potential for underseepage along the backfill need to be developed. Those mitigations can include:

- Removal and replacement of the backfill using highly plastic clay or controlled low strength material (lean cement slurry); or
- Place low permeability trench plugs at selected location

9.5.2 Through-Seepage

The steady state seepage analyses indicated through-seepage develops within each of the five cross sections evaluated. All sections resulted in the phreatic surface encountering the landside levee slope face.

If the levee prism is reconstructed to USACE standards, as discuss in Section 9.2, then daylight height above landside may be reduced or through-seepage may not continue to be a deficiency, depending on the characteristics of the fill materials being placed. Soil material criteria should be established prior to grading to reduce the potential for through seepage.

If recommendations in Section 9.2 are not implemented, then through-seepage can be mitigated using a cutoff trench or seepage berm.

Cutoff Trenches. A cutoff trench reduces the potential for through-seepage by blocking seepage paths through the embankment and foundation materials. A cutoff trench is typically a minimum of three feet wide and extends to a depth of about 1.5 times the levee height below the bottom of the levee; however, the configuration of the cutoff trench should be designed for site-specific conditions. The cutoff trench is filled with low permeable materials such as bentonite, cement slurry, or low permeability soils.

Landside Seepage Berm. Landside berms are constructed on the landward side of embankments to reduce the potential for through seepage by lengthening seepage paths. The berms also reduce the potential for sloughing of the landside face and can be used as an emergency source of borrow materials for repair of the levee. Berms are composed of impervious or semi-pervious material, sand, or free-draining gravel. However, the configuration (thickness and width) of berms should be designed for site-specific conditions and may require significant space, which locally along the Jacobs Avenue Levee, could impinge upon landside improvements.

Seepage can pose significant risks to the stability of a levee and is an on-going condition at

Jacobs Avenue due to tidal fluctuations. It is also one of the relatively easiest conditions to mitigate, although those mitigations are relatively expensive. Thus, we have ranked seepage relatively high regarding urgency of mitigation in Section 9.1.

9.6 *Slope Stability*

As discussed in Section 8.3 of this report, slope stability analyses were performed at each cross section for various conditions. The results are discussed relative to each condition below.

Slope instability poses a significant risk to the levee, especially in those areas where the levee width is deficient, such as from about Station 46+00 to the western terminus of the levee at Highway 101. For this reason, we have ranked slope stability as relatively high in urgency for mitigation, as noted in Section 9.1.

9.6.1 Existing

Our slope stability analyses of existing levee conditions estimated factors of safety ranging from approximately 1.36 to 1.90 and 1.04 to 1.75 for the landside and waterside slopes, respectively. Based on our review of historical information and observations during the field exploration program, we understand the levee slopes have not experienced past instability, which is consistent with our estimated factors of safety.

Although a minimum FS is not defined for existing levees in USACE (2000a), our estimated factors of safety for existing levee conditions marginally meet or exceed the minimum end-of-construction FS for a new levee, defined as 1.3 (USACE, 2000a).

9.6.2 Pseudostatic

Our slope stability analyses of existing levee conditions subjected to pseudostatic (i.e. earthquake) loading estimated factors of safety ranging from approximately 1.10 to 1.36 and 0.80 to 1.31 for the landside and waterside slopes, respectively. A minimum FS is not defined for pseudo-static loading conditions in USACE (2000a). There is high risk of lateral spreading in the event of a strong earthquake (see Section 9.4). Based on our review of historical information, there is no information indicating that levee slopes have experienced instability during a seismic event.

At a minimum, it is recommended that the levees be inspected for signs of instability or slope failure as soon as possible following an earthquake and, if necessary, repaired as soon as conditions will allow. If recommendations for reconstruction of the levee prism are implemented as discussed in Section 9.2, it is likely that pseudostatic factors of safety for waterside slopes will exceed 1.0.

9.6.3 Full Flood (Steady State) Event

Our slope stability analyses of existing landside levee slopes, based on steady-state full flood conditions, estimated factors of safety ranging from approximately 1.08 to 1.47.

As noted above, phreatic surfaces and pore pressures modeled in slope stability analyses of full flood conditions were based on the results of our steady-state seepage analyses. It is our opinion that steady state phreatic surfaces are unlikely to develop during the projected duration of a 100-yr flood event. Slope stability analyses of sections with phreatic surfaces more likely to develop during the design flood event duration are anticipated to result in higher estimated factors of safety than the FS values shown in Section 8.3.3, considering full flood conditions.

Therefore, the potential for landside slope failures during the 100-yr flood event is considered to be low to medium. At a minimum, however, it is recommended that the levees be inspected for signs of instability or slope failure during a flood event and, if necessary, repaired as soon as conditions will allow.

9.6.4 Rapid Drawdown

Our slope stability analyses of existing waterside levee slopes following a rapid drawdown of full flood water levels estimated factors of safety ranging from approximately 0.98 to 1.75.

As noted above, phreatic surfaces and pore pressures modeled in slope stability analyses of rapid drawdown conditions were based on the results of our steady-state seepage analyses. It is our opinion that steady state phreatic surfaces are unlikely to develop during the projected duration of a 100-yr flood event. Furthermore, slope stability analyses with phreatic surfaces likely to develop during the design flood event are anticipated to result in higher estimated factors of safety than the FS values shown in Section 8.3.3.

Therefore, the potential for waterside slope failures due to rapid drawdown during the 100-yr flood event is considered to be low to medium except for sections 10+75 and 42+00 where it is considered high (FS=0.98 and 1.04). If recommendations for reconstruction of the levee prism are implemented as discussed in Section 9.2, it is likely that rapid drawdown factors of safety will increase and possibly meet the minimum threshold of 1.4. Following flood events, we recommend that the levees be inspected for signs of instability or slope failure and, if necessary, repaired as soon as conditions will allow.

9.7 *Seismic Deformation*

As noted in Section 8.3.3, the estimated factors of safety for the screening-level pseudostatic slope stability analyses ranged from 0.80 to 1.36 for all of the sections evaluated. Therefore, seismic displacements are anticipated for the design seismic event and post-earthquake slope was estimated to be at least 36 inches.

If mitigations discussed in Section 9.2 and 9.4 are implemented, then the seismic displacement noted above would be eliminated. If those mitigations are not implemented, then mitigation measures to reduce the seismic deformation include modifying the levee prism geometry and reinforcing the levee embankment so that it is stable under pseudostatic conditions. This would involve possibly reinforcing the levee embankment with geogrid, removal and replacement of the levee embankment materials with strong soils, construction of stability berms, etc. Each would involve extensive grading and modification to the levee along its entire length. Utilization of stronger soils and stability berms would require importation of soils. Construction of stability berms would potentially impinge upon landside property improvements and could be required on the waterside slope of the levee, too.

Seismic deformation has a high potential to destabilize the levee, potentially rendering it ineffective as a flood control improvement. However, seismic deformation only occurs when there is a relatively strong near-field earthquake, which happens relatively infrequently. So, while the concequence to the levee from seismic deformation is high, the probability of occurrence is low for any given year. Thus, we have ranked the urgency for mitigation as relatively low because the likelihood of seismic deformation to occur soon is considered low.

9.8 *Static Settlement*

As discussed in Section 8.5, the potential for static settlement to impact the levees is considered to be minimal under the present conditions. Therefore, no remedial actions are considered necessary at this time.

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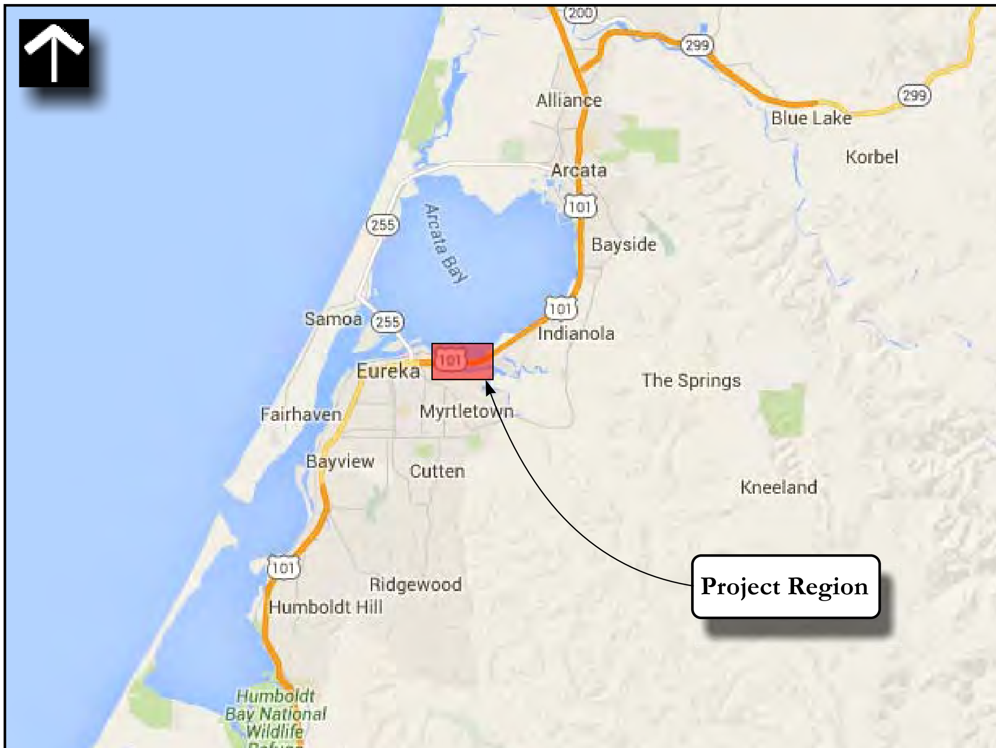
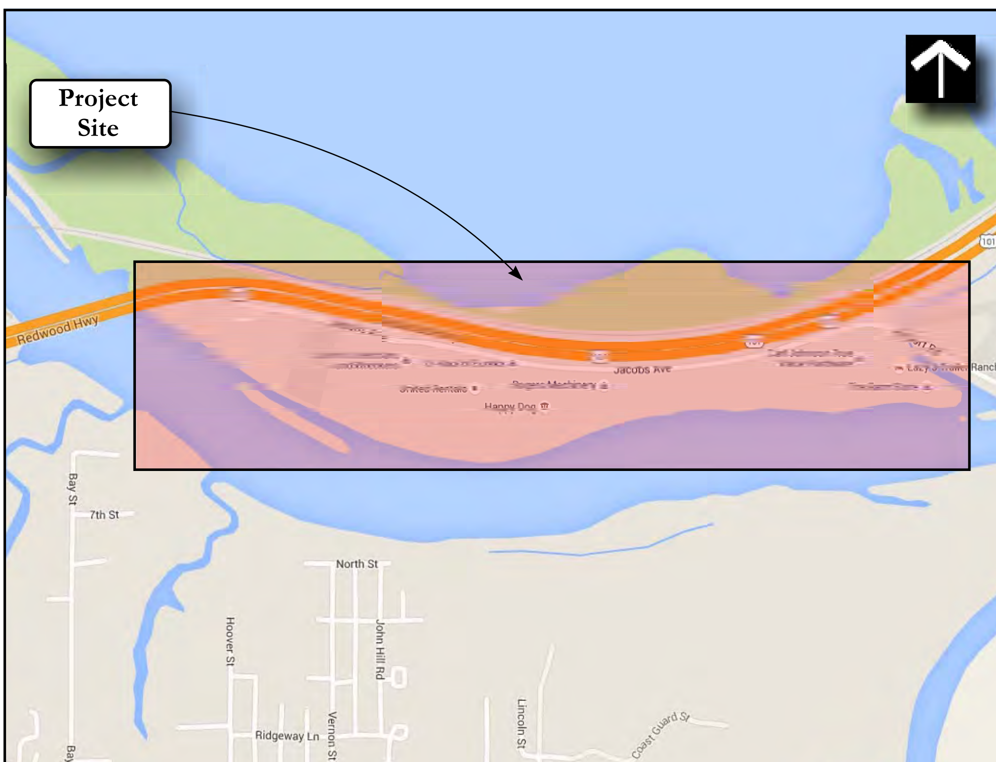




Base maps obtained from Google Maps

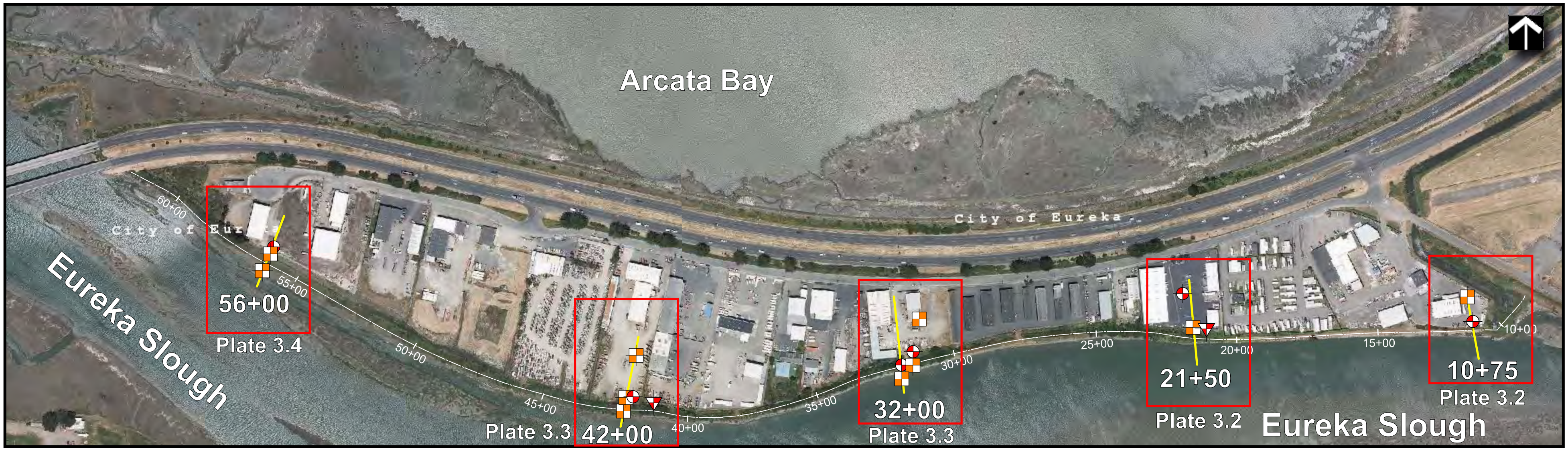
Scale undetermined

CGI TECHNICAL SERVICES INC.	SITE LOCATION MAP JACOBS AVENUE LEVEE EVALUATION PROJCT HUMBOLDT COUNTY PUBLIC WORKS DEPARTMENT CITY OF EUREKA, CALIFORNIA	Plate 1
Project No.:15-1949.03		

Base maps obtained from Google Earth

Scale undetermined

CGI Technical Services Inc.	LEVEE DETAILS JACOBS AVENUE LEVEE EVALUATION PROJCT HUMBOLDT COUNTY PUBLIC WORKS DEPARTMENT CITY OF EUREKA, CALIFORNIA	Plate 2
Project No.:15-1949.03		



DH-10L — Drill Hole

CPT-30C — Cone Penetrometer Test Sounding

B-2 — Taber Consultants (1985) Boring (very approximate)

Cross Section Location

Plate 3.4 — Plate number showing greater detail regarding exploration locations

Image from Google Earth

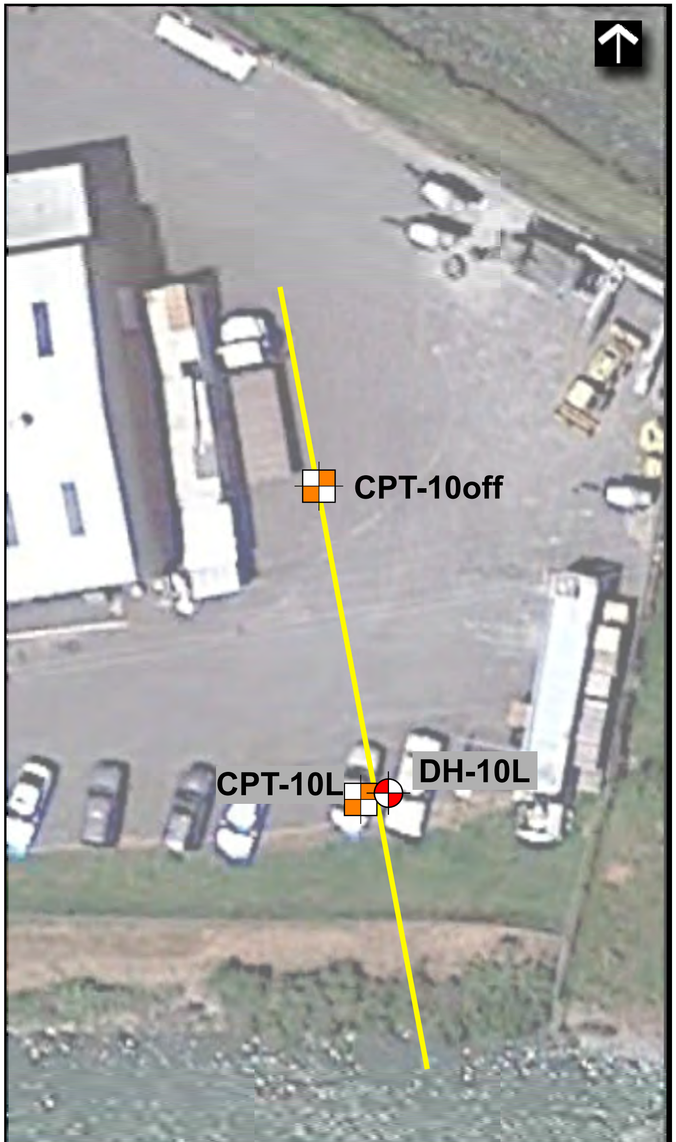
Scale Undetermined

CGI Technical Services Inc.

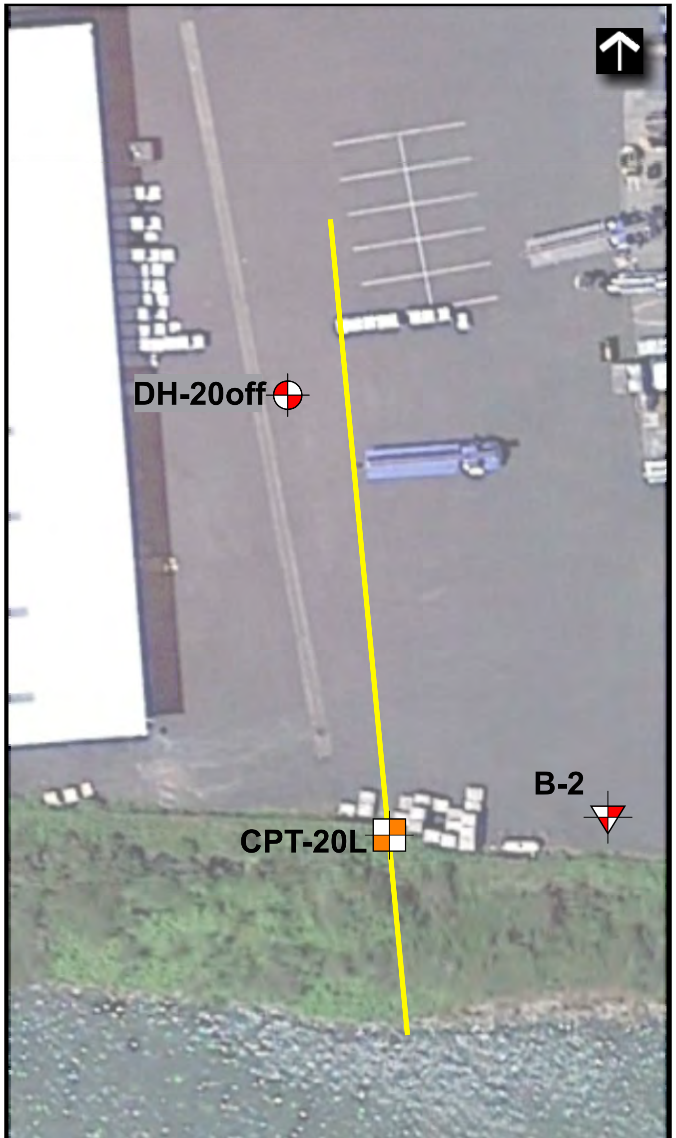
Project No.:15-1949.03

CROSS SECTION & EXPLORATION LOCATIONS
JACOBS AVENUE LEVEE EVALUATION PROJECT
HUMBOLDT COUNTY PUBLIC WORKS DEPARTMENT
CITY OF EUREKA, CALIFORNIA

Plate 3.1



CROSS SECTION 10+75



CROSS SECTION 21+50

DH-10L — Drill Hole

CPT-30C — Cone Penetrometer Test Sounding

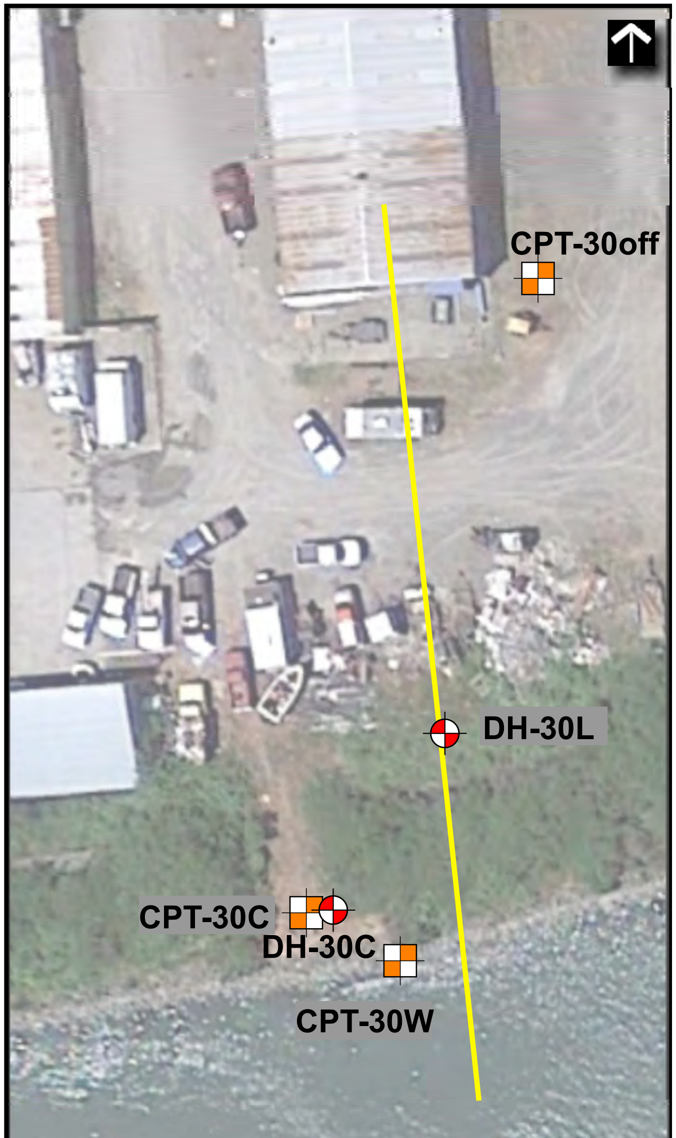
B-2 — Taber Consultants (1985) Boring (very approximate)

Cross Section Location

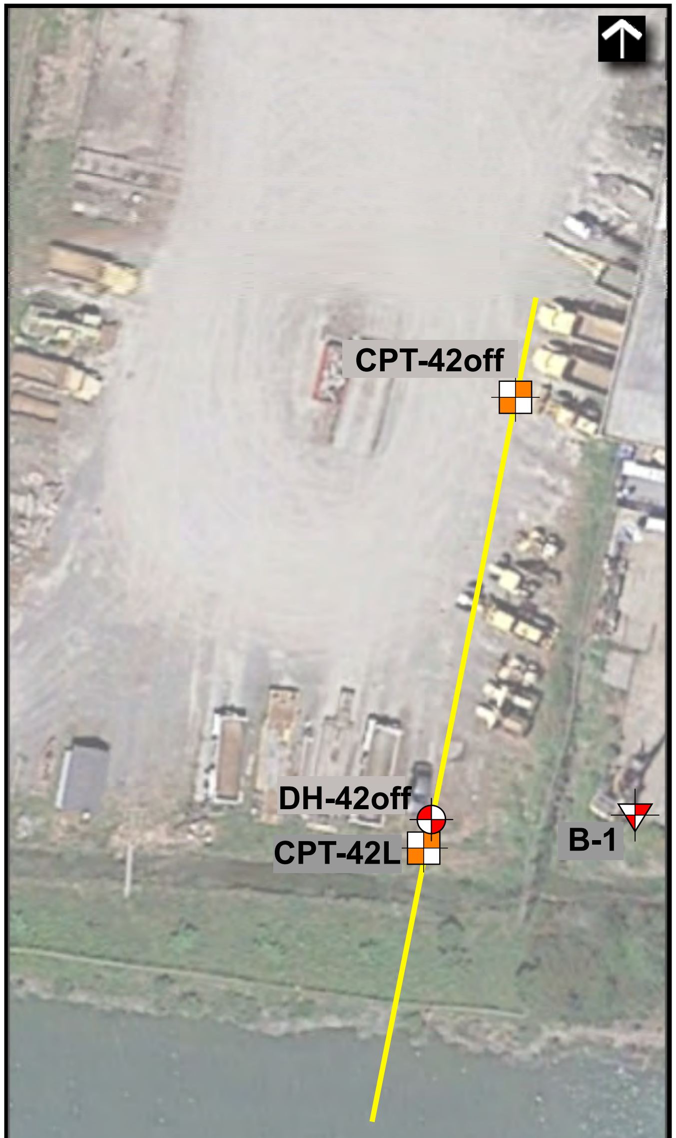
Image from Google Earth

Scale Undetermined

CGI TECHNICAL SERVICES INC.	CROSS SECTION & EXPLORATION LOCATIONS JACOBS AVENUE LEVEE EVALUATION PROJECT HUMBOLDT COUNTY PUBLIC WORKS DEPARTMENT CITY OF EUREKA, CALIFORNIA	Plate 3.2
Project No.:15-1949.03		



CROSS SECTION 32+00



CROSS SECTION 42+00

DH-10L	Drill Hole
CPT-30C	Cone Penetrometer Test Sounding
B-2	Taber Consultants (1985) Boring (very approximate)
	Cross Section Location

Image from Google Earth

Scale Undetermined

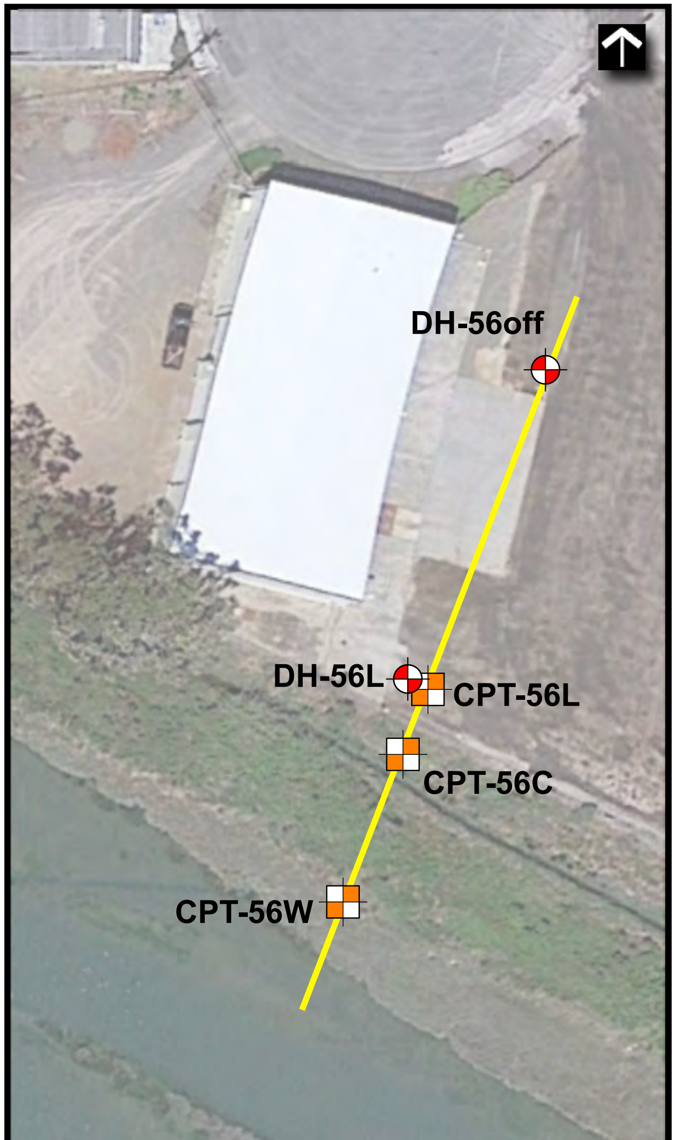
CGI TECHNICAL SERVICES INC.

Project No.:15-1949.03

CROSS SECTION & EXPLORATION LOCATIONS
JACOBS AVENUE LEVEE EVALUATION PROJECT
HUMBOLDT COUNTY PUBLIC WORKS DEPARTMENT
CITY OF EUREKA, CALIFORNIA

Plate

3.3



CROSS SECTION 56+00

DH-10L Drill Hole

CPT-30C Cone Penetrometer Test Sounding

B-2 Taber Consultants (1985) Boring (very approximate)

Cross Section Location

Image from Google Earth

Scale Undetermined

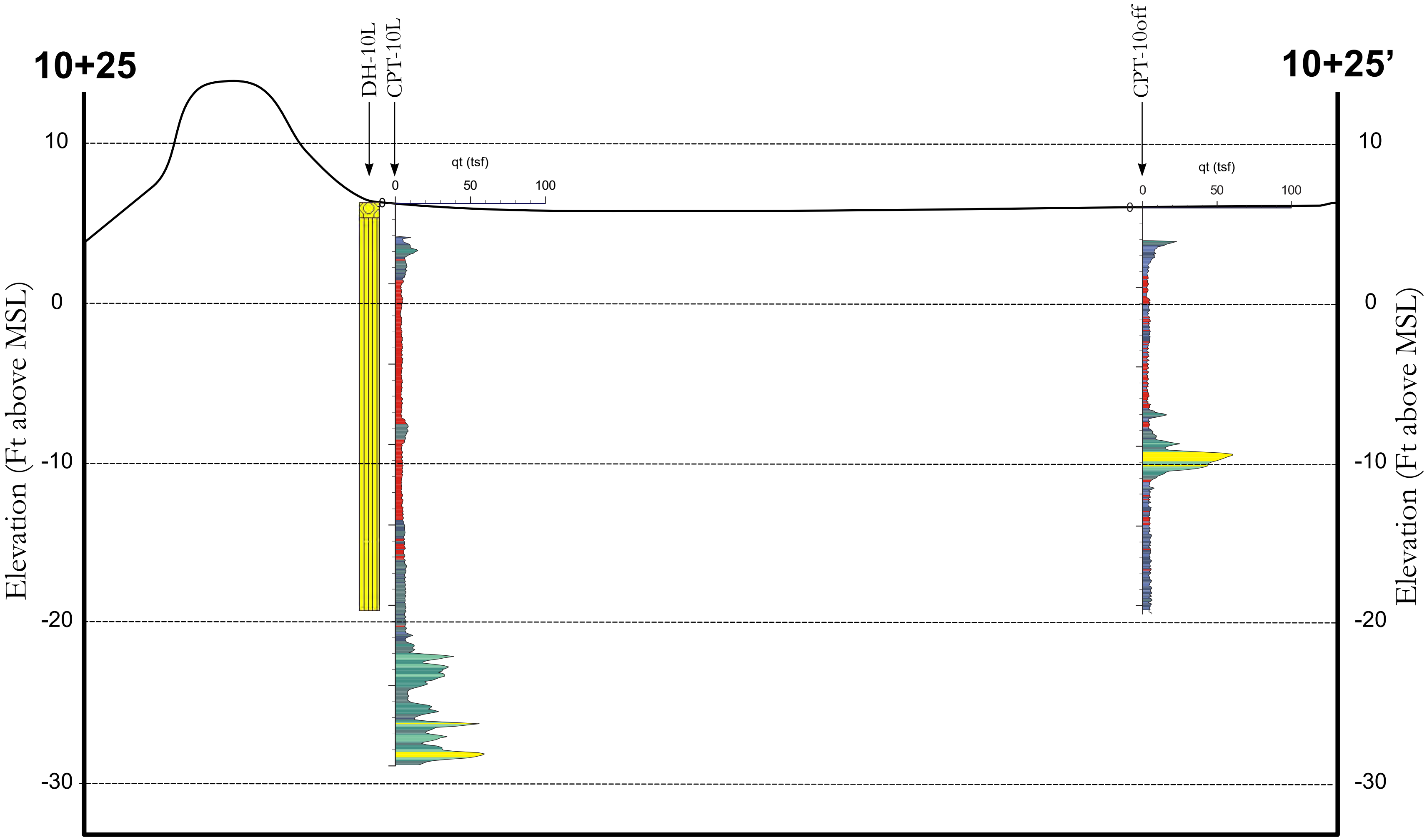
CGI TECHNICAL SERVICES INC.

Project No.:15-1949.03

CROSS SECTION & EXPLORATION LOCATIONS
JACOBS AVENUE LEVEE EVALUATION PROJECT
HUMBOLDT COUNTY PUBLIC WORKS DEPARTMENT
CITY OF EUREKA, CALIFORNIA

Plate

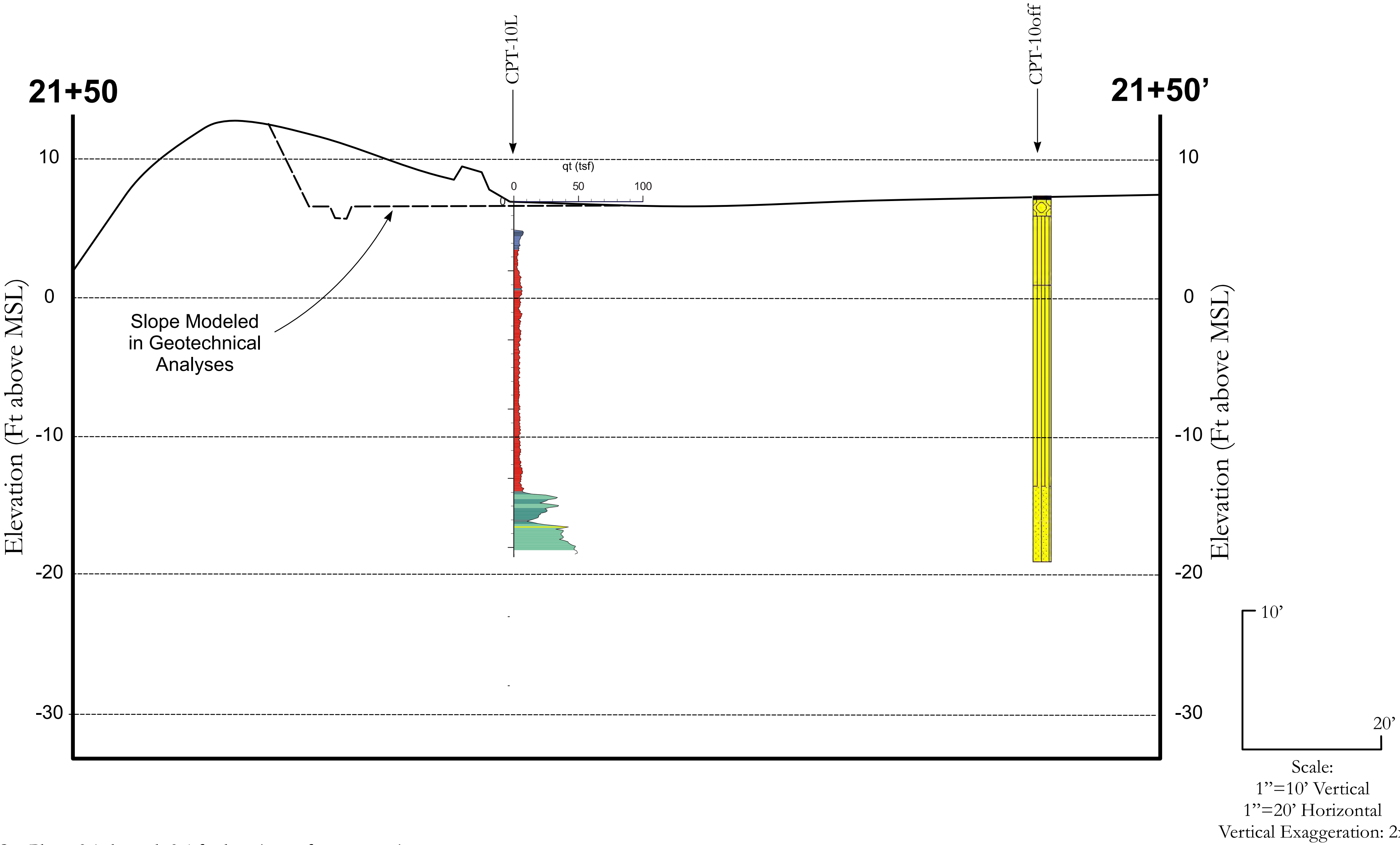
3.4



Scale:
1"=10' Vertical
1"=20' Horizontal
Vertical Exaggeration: 2x

See Plates 3.1 through 3.4 for locations of cross sectrions
See Plate 4.6 for legend to colors and symbols

CGI Technical Services Inc.	CROSS SECTION 10+25 JACOBS AVENUE LEVEE EVALUATION PROJECT HUMBOLDT COUNTY PUBLIC WORKS DEPARTMENT CITY OF EUREKA, CALIFORNIA	Plate 4.1
Project No.:15-1949.03		



See Plates 3.1 through 3.4 for locations of cross sectrions
See Plate 4.6 for legend to colors and symbols

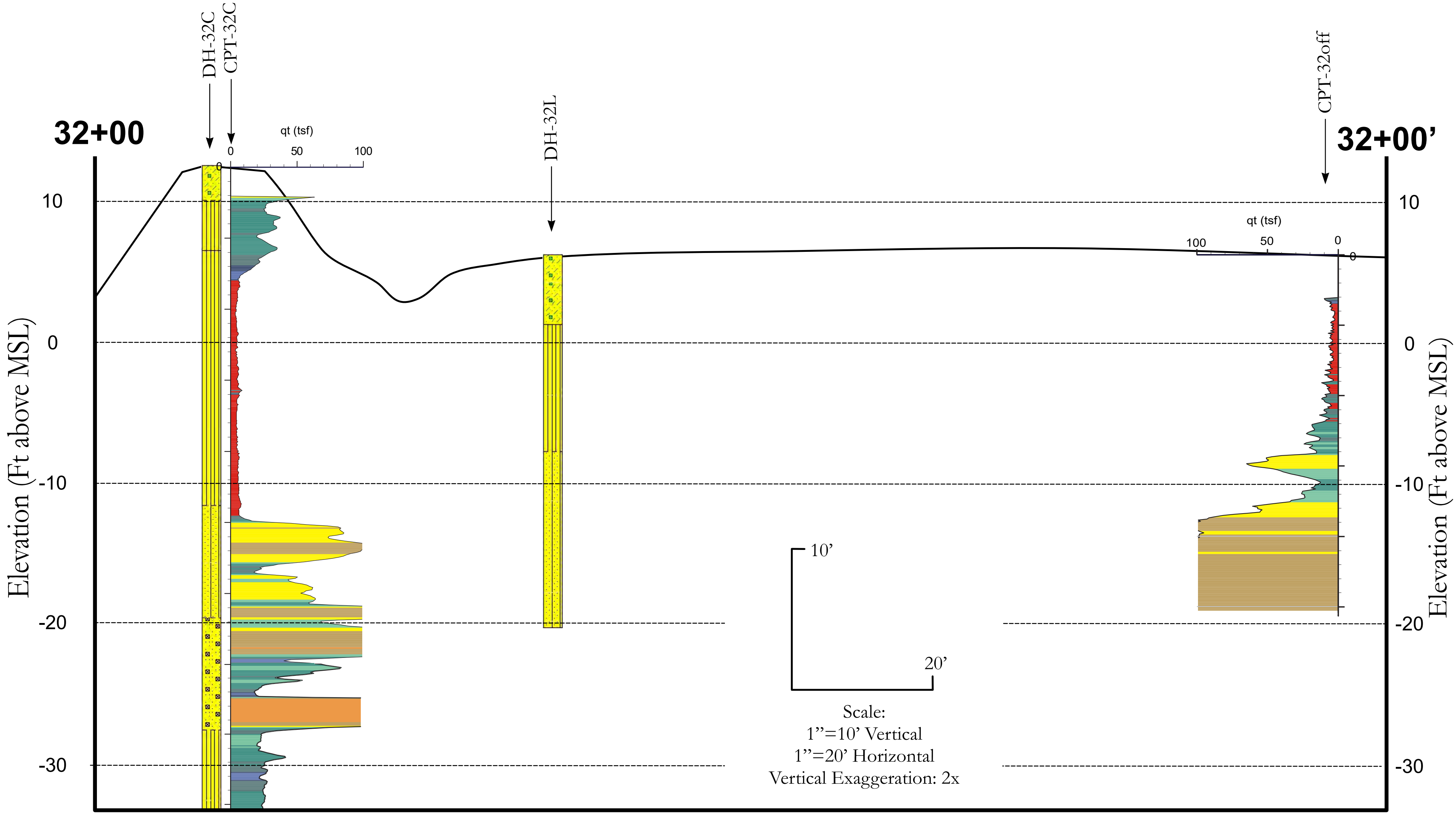
CGI Technical Services Inc.

Project No.:15-1949.03

CROSS SECTION 21+50
JACOBS AVENUE LEVEE EVALUATION PROJECT
HUMBOLDT COUNTY PUBLIC WORKS DEPARTMENT
CITY OF EUREKA, CALIFORNIA

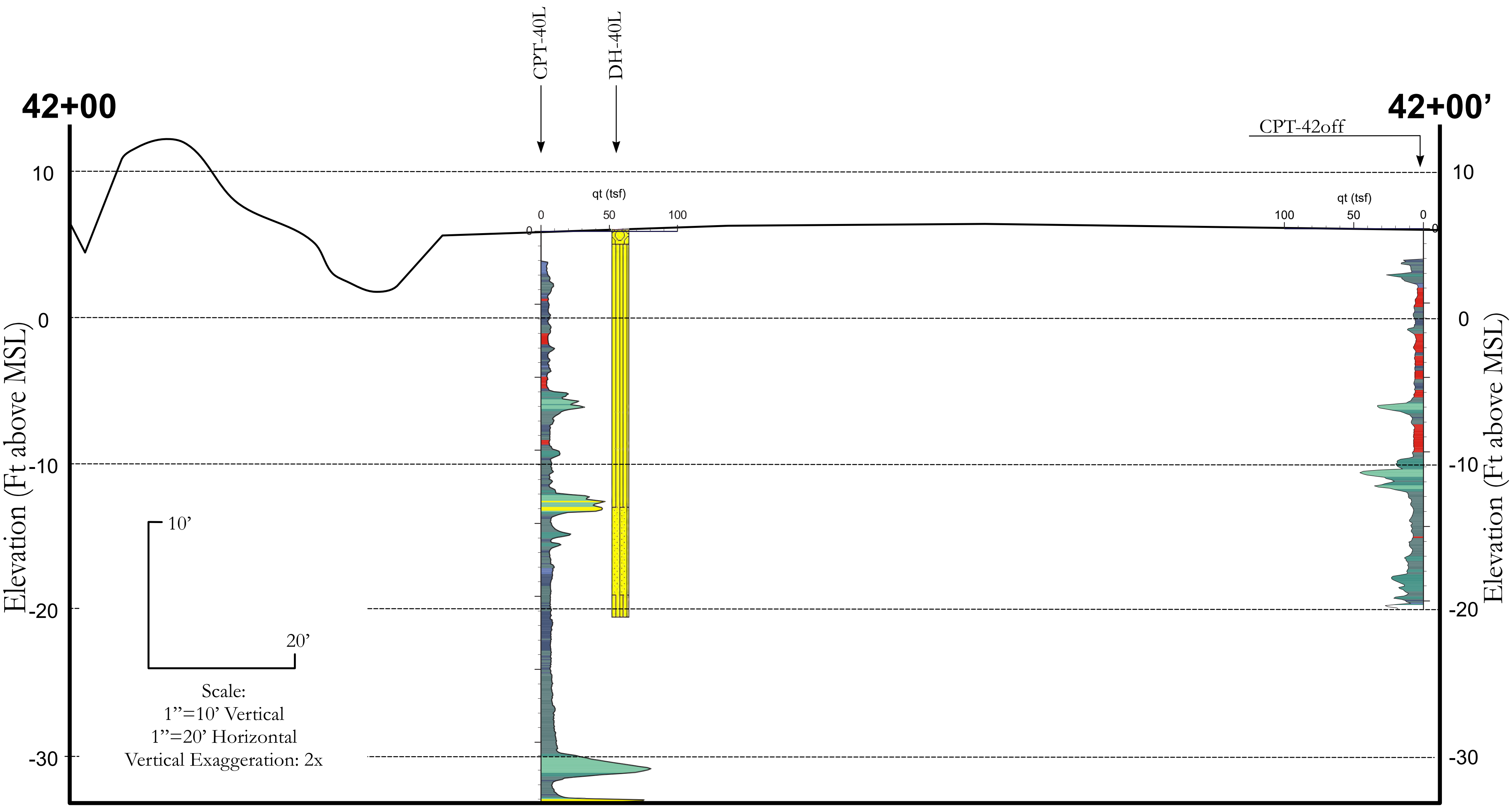
Plate

4.2



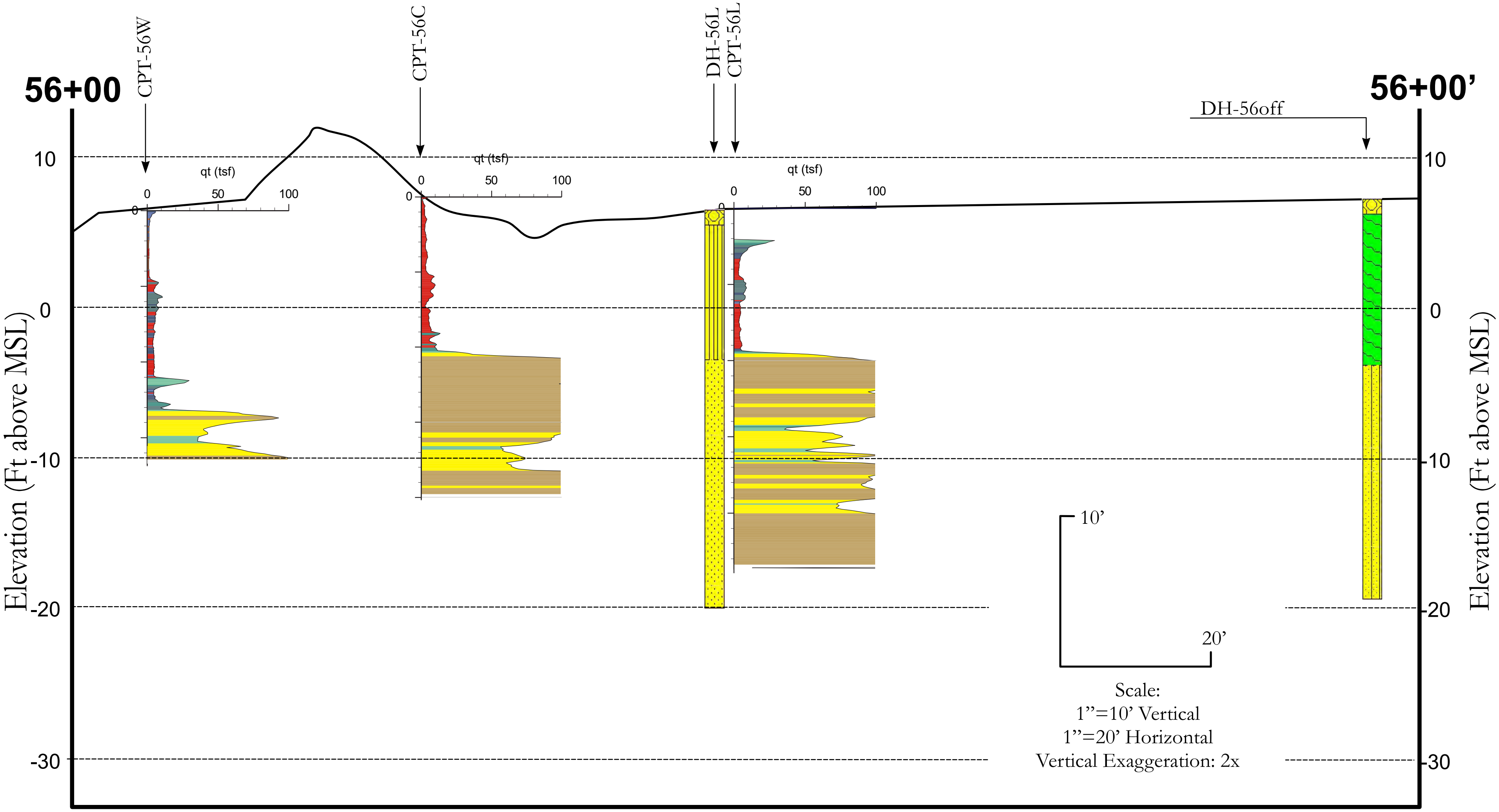
See Plates 3.1 through 3.4 for locations of cross sections
See Plate 4.6 for legend to colors and symbols

CGI Technical Services Inc.	CROSS SECTION 32+00 JACOBS AVENUE LEVEE EVALUATION PROJECT HUMBOLDT COUNTY PUBLIC WORKS DEPARTMENT CITY OF EUREKA, CALIFORNIA	Plate 4.3
Project No.:15-1949.03		



See Plates 3.1 through 3.4 for locations of cross sections
See Plate 4.6 for legend to colors and symbols

CGI Technical Services Inc. Project No.:15-1949.03	CROSS SECTION 42+00 JACOBS AVENUE LEVEE EVALUATION PROJECT HUMBOLDT COUNTY PUBLIC WORKS DEPARTMENT CITY OF EUREKA, CALIFORNIA	Plate 4.4



See Plates 3.1 through 3.4 for locations of cross sectrions
See Plate 4.6 for legend to colors and symbols

CGI Technical Services Inc.

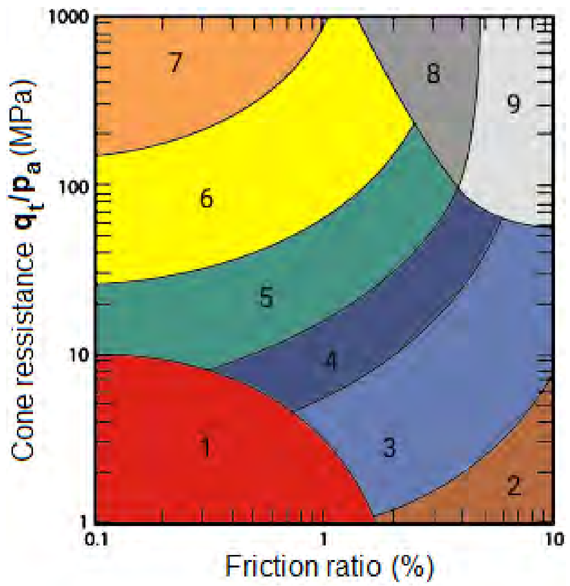
Project No.:15-1949.03

CROSS SECTION 56+00
JACOBS AVENUE LEVEE EVALUATION PROJECT
HUMBOLDT COUNTY PUBLIC WORKS DEPARTMENT
CITY OF EUREKA, CALIFORNIA

Plate

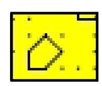
4.5

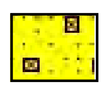
CONE PENETROMETER LEGEND

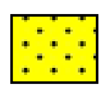


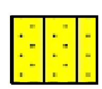
Zone	Soil Behavior Type (SBT)
1	Sensitive, fine grained
2	Organic soils - clay
3	Clay - silty clay to clay
4	Silt mixtures - clayey silt to silty clay
5	Sand mixtures - silty sand to sandy silt
6	Sands - clean sand to silty sand
7	Gravelly sand to dense sand
8	Very stiff sand to clayey sand *
9	Very stiff fine grained *

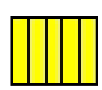
DRILL HOLE LEGEND

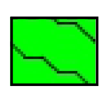

Aggregate Base


Gravel


Sand


Silty Sand


Silt


Clay

Surface elevations noted on cross sections based on survey data provided by Humboldt County unless noted otherwise on individual cross sections.

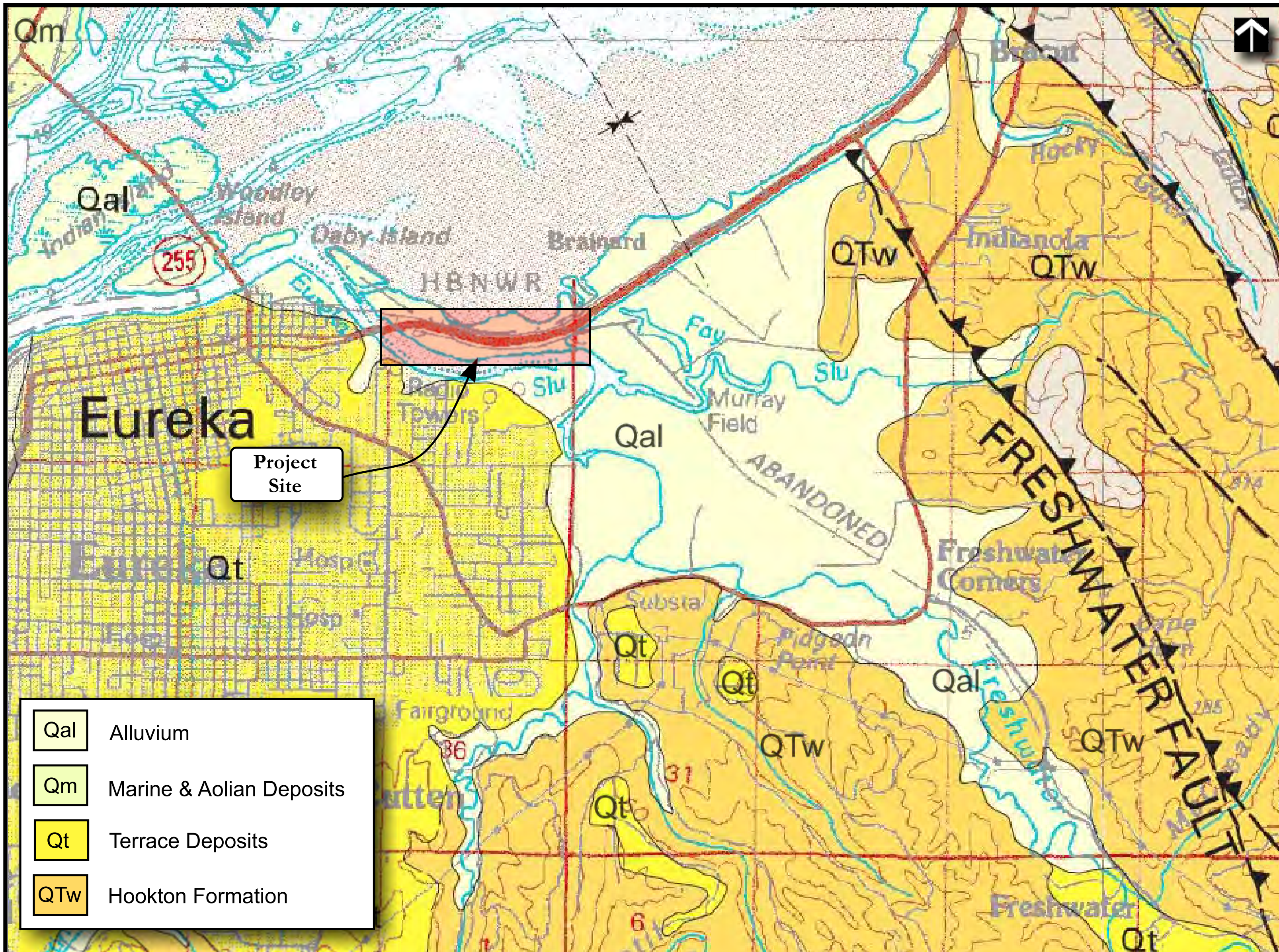
CGI TECHNICAL SERVICES INC.

Project No.:15-1949.03

LEGEND TO CROSS SECTIONS
JACOBS AVENUE LEVEE EVALUATION PROJECT
HUMBOLDT COUNTY PUBLIC WORKS DEPARTMENT
CITY OF EUREKA, CALIFORNIA

Plate

4.6



Base map from McLaughlin et al. (2000)

Scale Undetermined

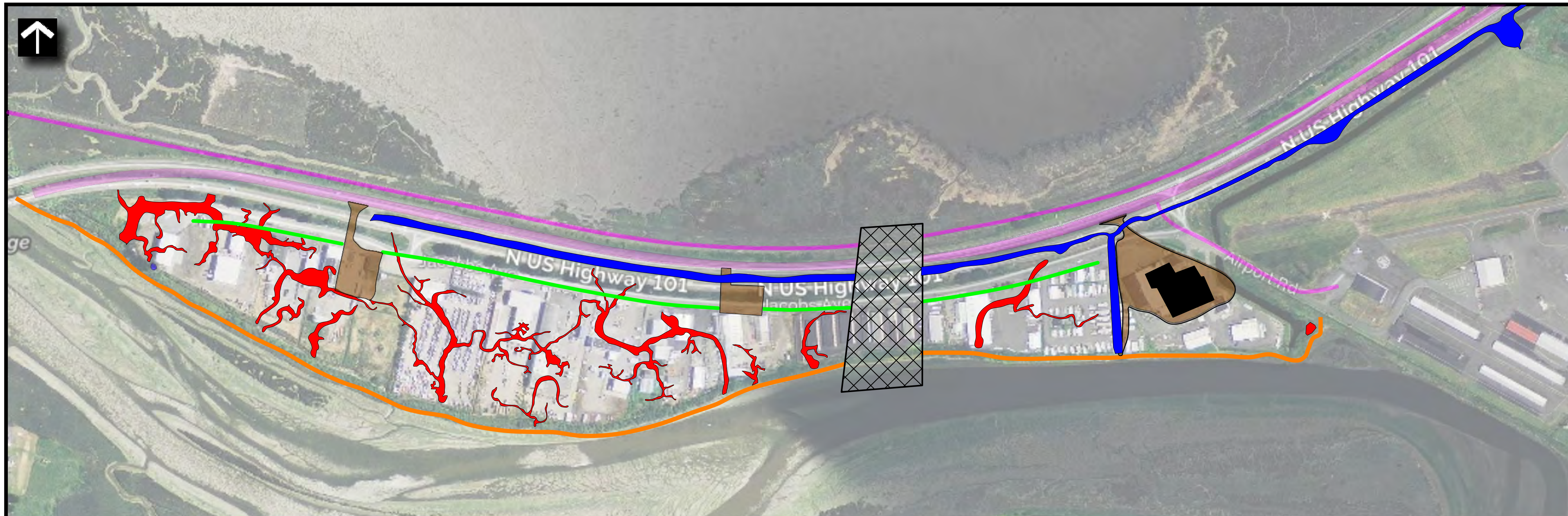
CGI Technical Services Inc.

Project No.:15-1949.03

REGIONAL GEOLOGIC MAP
JACOBS AVENUE LEVEE EVALUATION PROJECT
HUMBOLDT COUNTY PUBLIC WORKS DEPARTMENT
CITY OF EUREKA, CALIFORNIA

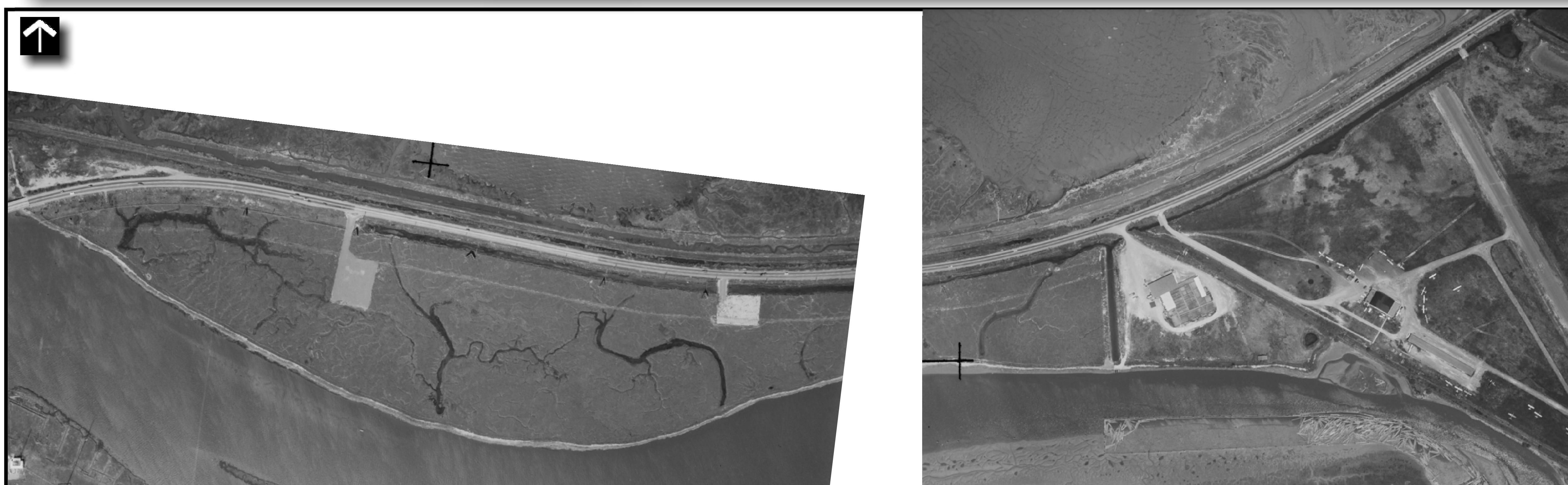
Plate

5



FEATURES NOTED ON 1950 AERIAL PHOTOGRAPHS

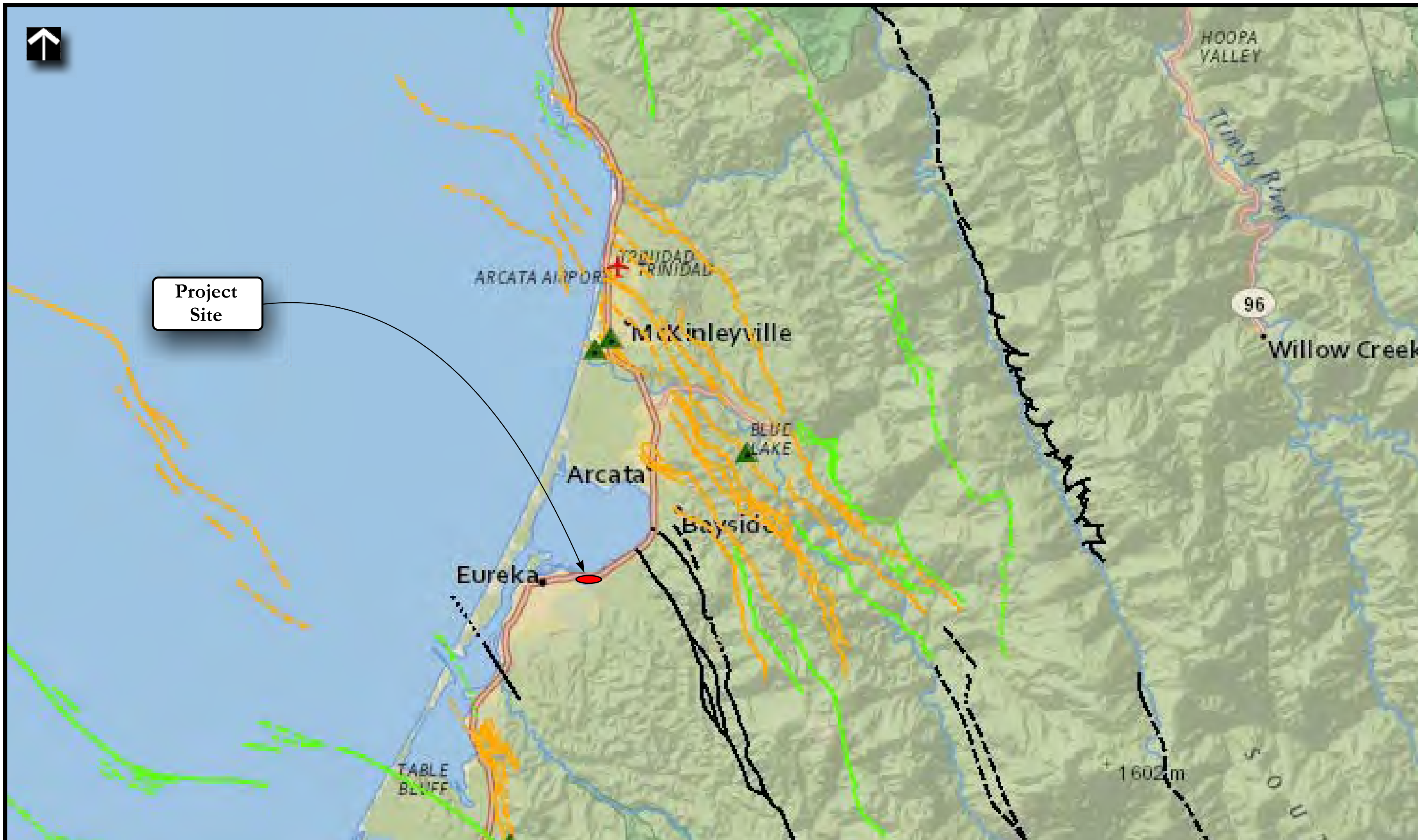
- Former Tidal Channel
- Former Drainage Channel (easterly segment, east of Airport Road may still be present)
- Former Auction Yard Building
- Road & Railroad Alignment
- Jacobs Avenue Road Alignment
- Levee Alignment
- Graded Building Pad
- Gap in Aerial Photographs



Images: Top - Google Earth; Bottom - from Shuster collection at Humboldt State University

Scale Undetermined

CGI Technical Services Inc.	FORMER TIDAL CHANNELS AND IMPROVEMENTS CIRCA 1950 JACOBS AVENUE LEVEE EVALUATION PROJECT HUMBOLDT COUNTY PUBLIC WORKS DEPARTMENT CITY OF EUREKA, CALIFORNIA	Plate 6
Project No.:15-1949.03		



Historic Displacement (last 200 years)

Holocene Displacement (last 11,700 years)

Late Quaternary Displacement (last 700,000 years)

Quaternary Fault (last 1.6 million years)

Map from USGS Interactive Fault Map

Scale undetermined

CGI Technical Services Inc.

Project No.:15-1949.03

REGIONAL FAULT MAP
JACOBS AVENUE LEVEE EVALUATION PROJECT
HUMBOLDT COUNTY PUBLIC WORKS DEPARTMENT
CITY OF EUREKA, CALIFORNIA

Plate

7

Base maps obtained from Google Earth. Topographic information obtained from Humboldt County.

Scale undetermined

CGI Technical Services Inc.

Project No.:15-1949.03

LEVEE TOPOGRAPHY - WESTERN
JACOBS AVENUE LEVEE EVALUATION PROJCT
HUMBOLDT COUNTY PUBLIC WORKS DEPARTMENT
CITY OF EUREKA, CALIFORNIA

Plate

8.1

Base maps obtained from Google Earth. Topographic information obtained from Humboldt County.

Scale undetermined

CGI TECHNICAL SERVICES INC.

Project No.:15-1949.03

LEVEE TOPOGRAPHY - EASTERN
JACOBS AVENUE LEVEE EVALUATION PROJCT
HUMBOLDT COUNTY PUBLIC WORKS DEPARTMENT
CITY OF EUREKA, CALIFORNIA

Plate

8.2