CGI TECHNICAL SERVICES, INC.

FUGRO CONSULTANTS, INC.

# REDWOOD CREEK LEVEE GEOTECHNICAL EVALUATION PROJECT ORICK, HUMBOLDT COUNTY, CALIFORNIA

**Prepared for:**
HUMBOLDT COUNTY,
PUBLIC WORKS DEPARTMENT

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Fugro Project No. 04.B3502002
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HUMBOLDT COUNTY PUBLIC WORKS DEPARTMENT
1106 Second Street
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Attention: Mr. Hank Seemann, Environmental Services Manager

Subject: Geotechnical Report, Redwood Creek Levee Geotechnical Evaluation Project, Orick, Humboldt County, California

Dear Mr. Seemann:

CGI Technical Services, Inc., in conjunction with Fugro Consultants, Inc., is pleased to present this revised draft Geotechnical Report for the Redwood Creek Levee Geotechnical Evaluation Project located in the Orick area of Humboldt County, California. This report is being submitted in accordance with the Agreement for Professional Services established between CGI and the County under County Project No. 251056.

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

The report is being submitted as a revised draft to allow the County the opportunity to review and comment on the findings, conclusions, and recommendations presented, herein. We will finalize this report after receiving your comments and, where appropriate, editing the report.

Please call if further information is required or if we can answer any questions.

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# LIST OF ACRONYMS AND ABBREVIATIONS

| | |
|---|---|
| ASTM | ASTM International |
| bgs | below ground surface |
| cfs | cubic feet per second |
| CGS | California Geologic Survey |
| CPT | Cone Penetration Test |
| CRR | Cyclic Resistance Ratio |
| CSR | Cyclic Stress Ratio |
| El. | Elevation |
| FS | Factor of Safety |
| GIS | Geographic Information System |
| h:v | horizontal:vertical |
| NAVD88 | North American Vertical Datum of 1988 |
| NCEER | National Center for Earthquake Engineering Research |
| PHGA | Peak Horizontal Ground Acceleration |
| Station | Survey Station |
| SPT | Standard Penetration Test |
| USA | Underground Service Alert |
| USACE | United States Army Corps of Engineers |
| USGS | United States Geological Survey |

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# 1.0 INTRODUCTION

## 1.1 PROJECT DESCRIPTION

CGI Technical Services, Inc. (CGI) has been authorized by Humboldt County Department of Public Works (County) to perform a geotechnical investigation of the Redwood Creek Levee system. This geotechnical evaluation has been performed in conjunction with Fugro Consultants, Inc. (Fugro), and our team is referred to as CGI/Fugro throughout this report. Funding for this study was provided by the California Department of Water Resources Local Levee Assistance Program and by a Department of Housing and Community Development Block Grant.

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, seepage, underseepage, or erosion to cause instability. Available information prior to this study was not sufficient to assess these risks for the Redwood Creek Flood Control Project. The results of the project will assist Humboldt County in assessing the public safety 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.

Redwood Creek is about 67 miles long and drains a watershed encompassing about 285 square miles (Redwood Creek Watershed Group, 2006). The Redwood Creek levee system extends from about one-half mile upstream of Orick for approximately 3.4 miles through the lower Orick Valley to the Pacific Ocean. The project site vicinity is shown on Plate 1-1. Construction of the levee system was completed in October 1968 by the Army Corps of Engineers (USACE) as part of the Redwood Creek Federal Flood Control Act of 1962 and responsibility for the levee system was transferred to the County on November 14, 1968. Prior to the levee construction, significant floods impacted the Orick area in 1861, 1890, 1915, 1953, 1955, and 1964, with the 1964 event having a particularly significant impact on the region (Redwood Creek Watershed Group; 2006; Harden, 1995). We understand that flooding during those events had flow volumes ranging from 45,000 to more than 50,000 cubic feet per second.

The Redwood Creek levee system is maintained and operated by Humboldt County. We understand that since the completion of construction of the levee, significant unanticipated aggradation of the channel bottom has occurred due to sediment influx from the upper Redwood Creek system (HDR, 2009; NHE, 2010b; USACE, 1966). According to HDR, an average of about 90,000 cubic yards of gravel are deposited within the levee system channel annually. We understand that removal of that sediment was not anticipated as normal maintenance during design of the project (USACE, 1966; USACE, 1969; NHE, 2010b) and that it has been a long-standing maintenance challenge for the County due to fiscal and regulatory restrictions and limitations, especially with Redwood Creek being a significant anadromous fishery. In addition, we understand that those same regulatory restrictions and limitations have hampered the ability of the County to remove some vegetative growth that has established along the waterside and landside flanks of the levee system, which has, to some degree, constrained the County's ability to maintain the levee system in accordance with the Operations and Maintenance Manual (USACE, 1969) for the levee system.

As part of the National Flood Insurance Program (NFIP), FEMA develops Flood Insurance Studies (FISs) and Flood Insurance Rate Maps (FIRMs) to depict the floodplain for the base flood (also known as the 1%-annual-chance-flow or 100-year recurrence interval flood). In order for a levee to be accredited by FEMA, the levee owner must provide data and information certified by a licensed engineer demonstrating that the levee meets, and will continue to meet, the minimum standards at 44 CFR Section 65.10. FEMA's standards address hydraulic, geotechnical, operational, and maintenance aspects of levee performance. When a levee is accredited by FEMA, the levee is recognized as providing flood protection for the base flood, and the Special Flood Hazard Area (also known as Zone A, the 100-year flood zone, or the regulated floodplain) on the FIRM will end at the levee centerline. If a levee does not meet FEMA standards, or if the technical documentation is insufficient or incomplete, then FEMA will not accredit the levee and the FIRM is modified to show the 100-year floodplain as if the levee does not exist.

In August 2009, FEMA released the preliminary updated FIRM and FIS for Humboldt County, including the community of Orick and the Redwood Creek levee system. It is our understanding that the Redwood Creek levee system is not accredited on this preliminary updated FIRM because FEMA has not received a complete documentation package demonstrating eligibility for accreditation. Therefore, portions of the Orick valley are expected to be mapped within the Special Flood Hazard Area when the updated FIRM is finalized. It is our understanding that the updated FIRM is currently undergoing revisions and is expected to be finalized in 2012. We understand that Humboldt County has been working since 2007 with available funding to perform the technical studies necessary to determine whether the Redwood Creek levee system meets the FEMA accreditation standards and whether a complete documentation package can be developed. This study is intended to assist Humboldt County in developing a documentation package and/or identifying repair or upgrade needs for the levee system to be eligible for accreditation.

The U.S. Army Corps of Engineers (USACE) administers a Rehabilitation and Inspection Program (RIP) under Public Law (PL) 84-99 for levees built by the USACE and operated by local sponsors. Continued eligibility in this program is contingent upon periodic inspections by the USACE and a determination that the levees are adequately maintained. Levees that have active status within this program are eligible for federal assistance following a damaging event, whereby the USACE is authorized to repair damages back to pre-flood conditions if the work is economically justifiable. It is our understanding that in January 2007 the USACE placed the Redwood Creek levee system on a national list of levees with maintenance deficiencies due to the presence of sediment within the channel and reduced flood capacity, and that in March 2008 the USACE disapproved Humboldt County's Maintenance Deficiency Correction Plan and placed the Redwood Creek levee system in inactive status under the RIP. As a result of its inactive status under the USACE RIP, the levee system was not eligible for preliminary accreditation status (effectively a two-year grace period) when FEMA was preparing the updated FIRM. Although currently inactive under the RIP, the Redwood Creek levee system remains a federal flood control project and continues to be inspected annually by the USACE.

## 1.2 PURPOSE AND 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 Redwood Creek levee system within the project extent to function in accordance with current FEMA/USACE performance criteria. To achieve this objective, CGI/Fugro performed subsurface exploration, laboratory testing, and engineering analyses.

Criteria for establishment of adequacy of levee systems to provide intended flood protection are set forth by Title 44 of the Code of Federal Regulations Section 65.10 of the National Flood Insurance Program (NFIP). FEMA is the federal agency responsible for administering the NFIP. Under these provisions, FEMA utilizes evaluation 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);
- 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:

### 1.2.1 Data Review, Site Reconnaissance, and Access Coordination

In preparation for the field exploration program, CGI/Fugro performed a 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 sonic cores, drill holes, and cone penetrometer test (CPT) soundings, CGI 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.

### 1.2.2 Field Exploration

Field exploration consisted of advancing a total of 91 subsurface explorations at selected locations along the levee alignment. In total, we performed the following explorations for this study:

**Explorations Performed for Study**

| Exploration Type | Total Advanced |
|---|---|
| Sonic Coring | 18 |
| Cone Penetrometer Soundings | 37 |
| Hollow-Stem-Auger Drilling | 36 |

The field exploration program was initiated in September 2010 and completed in October 2010. Locations of explorations are presented on Plate 5-3 and additional information regarding the field exploration program are presented in Section 3.0 and Appendix B.

### 1.2.3 Laboratory Testing

Laboratory testing was performed 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.

### 1.2.4 Geotechnical Analyses and Evaluation

Using data obtained from the data review, field exploration, and laboratory testing programs, CGI/Fugro performed geotechnical analyses, as specified in the scope of work, at 10 cross sections established across Redwood Creek and its associated levees. The analyses were consistent with geotechnical criteria specified 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)
- Evaluations of vegetative impacts and erosion on levee system (Sections 9.2.1, 9.2.2, and 9.2.3)

## 1.3 AUTHORIZATION

This study was authorized by the County in the Agreement for Professional Services under County Project No. 251056.

## 1.4 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/Fugro solely for the County for use in evaluating the ability of the existing Redwood Creek 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 Redwood Creek levee system extends from the Pacific Ocean upstream until the creek reaches the boundary with Redwood National Park, upstream of Orick. This approximately 3.4-mile alignment of earthen embankment levees extends through relatively flat terrain within the Orick Valley, as shown on Plates 2-1a through 2-1c. The waterside levee faces are rip-rap protected. The following sections describe the site conditions along segments of the levee system. Where the terms "right" and "left" are used to distinguish one levee from the other, the perspective is always as viewed looking downstream.

### 2.1 STATION NOS. 10+00 (DOWNSTREAM TERMINUS) TO 55+00

This lower reach of the levee system extends about 4,500 feet, from the ocean to the first major bend of Redwood Creek. This section of the levee system extends through the coastal estuary and bypasses a former oxbow (South Slough) of the creek system. It traverses a portion of Redwood National Park along the immediate coastal zone then transitions through relatively flat, privately-owned, agricultural grounds. The right levee is bordered by an unnamed dirt access road that leads from Hufford Road to the coastal fishing access area. Redwood Creek in this section of the levee system is subject to tidal influxes from the Pacific Ocean and is under varying stages of inundation throughout the day.

At approximate Station No. 32+00, a levee penetration, consisting of a gated concrete box structure, extends through the left levee system and into South Slough. In addition, a culvert with a flap-gate has been noted penetrating through the left levee system at about Station No. 44+00 (Humboldt County, 2008a); however, we could not locate that culvert during field studies. A drainage ditch is located adjacent to the right landside levee toe between about Station Nos. 43+00 and 55+00.

The landside banks along the levee system are covered with seasonal grasses, and locally with brambles and shrubs. Waterside banks are covered with seasonal vegetation, local brambles, and local shrubs. Trees and other larger vegetation are locally present along the waterside and landside toes of the levee system. The landside and waterside levee embankments are generally inclined at about 2h:1v and 3h:1v (horizontal to vertical), respectively; however local variances in the slope inclinations are present.

### 2.2 STATION NOS. 55+00 TO 85+00

This section of the levee system is oriented in a general north-south direction as it progresses through relatively flat, privately-held agricultural lands. In this area, the creek appears to be a braided drainage system with aggregate point bars, lateral bars, and mid-channel bars distributed across the channel. Adjacent lands are largely undeveloped and rangeland, with the exception of a barn, well house, and other structures adjacent to the left levee near Station No. 77+00.

Six existing relief wells were observed adjacent to the left levee between about Station Nos. 77+00 though 84+00. These relief wells are located at the landside levee toe and are adjacent to a drainage ditch that extends from about Station Nos. 75+00 to 85+00. Another drainage ditch is located along the right levee between about Station Nos. 55+00 and 62+00. In addition, a culvert with a flap-gate has been noted penetrating through the left levee system at about Station No. 76+00 (Humboldt County, 2008a).

The landside banks along the levee system are covered with seasonal grasses, brambles, and shrubs, with trees locally present at the levee toe. Waterside banks are covered with seasonal vegetation, brambles, and shrubs. Trees and other larger vegetation are present along the waterside toes of the levee. The landside levee embankments are generally inclined at about 2.5h:1v. Waterside levee slopes are generally inclined at about 2.8h:1v to 3h:1v. Local variances in the slope inclinations are present.

## 2.3 STATION NOS. 85+00 TO 135+00

Redwood Creek changes to a generally easterly direction in this segment. It progresses through relatively flat, privately-held agricultural lands along its right embankment for the westerly two-thirds of this section. The remaining easterly third of the right levee is bordered by residential improvements, a commercial establishment, and a segment of Hufford Road. The left embankment is bordered by residential and commercial improvements from about Station Nos. 85+00 to 95+00 and 115+00 to 127+00. At about Station 128+00 to 132+00, the left levee is located adjacent to Orick School. At about Station No. 127+00, the levee is crossed by the Highway 101 Bridge spanning Redwood Creek. The remainder of the alignment is adjacent to privately owned, relatively flat agricultural or fallow lands. In this area, the creek appears to be a braided drainage system with aggregate point bars, lateral bars, and mid-channel bars distributed across the channel.

Two existing relief wells were observed adjacent to the right levee at about Station Nos. 110+00 and 111+50. A drainage ditch is located along the right levee between about Station Nos. 111+00 and 118+00. In addition, culverts with flap-gates have been noted penetrating through the left levee system at about Station Nos. 76+00 and 291+00 (Humboldt County, 2008a).

The landside banks along the levee system are covered with seasonal grasses, brambles, and shrubs, with trees locally present at the levee toe, especially adjacent to residential developments on the left levee between Station Nos. 114+00 and 120+00 and on the right levee between Station Nos. 124+00 and 127+00. Between about Station Nos. 86+00 and 89+00, aggregate extraction from Redwood Creek has resulted in a gravel stockpile along the left levee. Waterside banks are covered with seasonal vegetation, brambles, and shrubs. Trees and other larger vegetation are present along the waterside toes of the levee. Trees and shrubs are also present on mid-channel bars within Redwood Creek. The landside levee embankments are generally inclined at about 2.5h:1v. Waterside levee slopes are generally inclined at about 2.8h:1v to 3h:1v. Local variances in the slope inclinations are present.

## 2.4 STATION NOS. 135+00 TO 177+00

This section of the levee system is oriented in a general north-northeast direction through relatively flat, privately-held agricultural lands along its left embankment and through a portion of the community of Orick adjacent to its right embankment. Drydens Road extends along the southerly about three-quarters of this segment along the left levee system. The Orick Chamber of Commerce Rodeo Grounds extend along the approximately northern quarter of the left levee. In this area, the creek appears to be a braided drainage system with aggregate point bars, lateral bars, and mid-channel bars distributed across the channel. The right levee extends to about Station 173+00 then terminates, whereas the left levee continues upstream.

Twenty existing relief wells were observed adjacent to the right levee between about Station Nos. 135+00 and 144+00. An additional relief well is located in an access ramp on the right levee at about Station 159+00. A drainage ditch is located along the left levee along Drydens Road, between about Station Nos. 135+00 and 160+00. Between about Station Nos. 138+00 and 144+00, aggregate extraction from Redwood Creek has resulted in a gravel stockpile along the left levee. A USGS gaging station (No. 11482500) is present on the right levee at about Station No. 171+50. In addition, culverts with flap-gates have been noted penetrating through the left levee system at about Station Nos. 142+00 and 159+00 and through the right levee at about Station Nos. 166+00 and 170+00 (Humboldt County, 2008a). An additional culvert penetration with a gate valve is located at the right levee at about Station No. 170+00.

The landside banks along the levee system are covered with seasonal grasses, brambles, and shrubs, with trees locally present at the right levee toe. Waterside banks are covered with seasonal vegetation, brambles, and shrubs. Trees and shrubs are present along the waterside toes of the levee, and are also present on mid-channel bars within Redwood Creek. The landside levee embankments are generally inclined at about 2h:1v. Waterside levee slopes are generally inclined at about 3h:1v. Local variances in the slope inclinations are present.

## 2.5 STATION NOS. 177+00 TO 193+00 (UPSTREAM TERMINUS)

Only the left levee is present within this generally easterly-oriented section. It extends through relatively flat, privately-held agricultural lands to its upstream terminus at Redwood National Park. An active gravel extraction and processing plant is present north of the creek, between about Station Nos. 181+00 to 187+0. The creek in this segment appears to be a braided drainage system with aggregate point bars, lateral bars, and mid-channel bars distributed across the channel.

The landside banks along the levee system are predominantly covered with seasonal grasses. Waterside banks are covered with seasonal vegetation, brambles, and shrubs. Trees and other larger vegetation are present along the waterside toe, especially where a vegetated bench up to about 120 feet wide is present above the active creek channel. The landside levee embankments are generally inclined at about 2h:1v. Waterside levee slopes are generally inclined at about 3h:1v. Local variances in the slope inclinations are present.

# 3.0 FIELD EXPLORATION

## 3.1 FIELD EXPLORATION PLANNING

### 3.1.1 Review of Previous Reports

The County provided us with the following documents for use in the geotechnical investigation:

- Report of soils tests for foundation materials at the Redwood Creek levee project (USACE, 1960);
- A survey report for the Redwood Creek levee system (USACE, 1961);
- Report of soils tests for undisturbed foundation materials for the Redwood Creek levee project (USACE, 1965a);
- General Design Memorandum for the project (USACE, 1966);
- Redwood Creek Local Flood Protection Project Operations and Maintenance Manual (USACE, 1969);
- Annual USACE inspection reports for the levee system between 2006, 2007, 2008a, 2009a, and 2009b;
- A geotechnical report for the South Slough drainage structure (Geo/Resource Consultants, 1986);
- Subsurface exploration logs and a map of exploration locations for the Orick wastewater treatment facility (SHN, 2009); and
- A recent geotechnical report for a bridge crossing Strawberry Creek, in the vicinity of the levee system (Galli Group, 2011).

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

Relevant geotechnical information from the reports listed above was reviewed during the investigation, including site and subsurface descriptions, historical groundwater data, laboratory testing results, drill hole logs, test pit logs, and as-built plans. Data from previous reports reviewed for the Redwood Creek Levee study are presented in Appendix A of this report.

According to the USACE (1966), field exploration programs were conducted during the following periods:

- August 1956;
- October 1958;
- March 1960;
- October through November 1964; and
- August through September 1965.

In total, those exploration programs advanced 85 explorations to depths ranging from 3 to 100 feet. The explorations ranged from test pits, hand-auger drill holes, bucket-auger drill holes, push-tube (Shelby tube) holes, a trench, and disturbed-sample holes. Table 3-1 presents information regarding explorations performed by the USACE (1966) prior to construction of the levee system:

### Table 3-1. USACE Pre-Levee Construction Explorations

| Exploration Period | Exploration No. | Type of Exploration | Depth Range (ft) |
|---|---|---|---|
| August 1956 | 2F-1 - 2F-4 | 4 Hand-Augers | 3' - 17' |
| October 1958 | 4B-1 | 1 Trench | Unknown |
| March 1960 | 2F-5 - 2F-18 | 6 Hand-Augers<br>8 Bucket-Augers | 6' - 19'<br>12' - 25' |
| October-November 1964 | 2F-19 - 2F-41<br>2F-42<br>7F-1 - 7F-5 & 7F-2A | 29 Bucket-Augers (24" - 36")<br>1 Hand-Auger<br>6 Push-Tubes | 10' - 40'<br>5.5'<br>6' - 12' |
| August-September 1965 | 2F-43 - 2F-71<br>4F-2<br>7F-6 - 7F-13 | 29 Disturbed-Holes<br>1 Test Pit<br>8 Push-Tube | 11' - 100'<br>Unknown<br>8' - 15' |

In all of the explorations reported by USACE, alluvium was encountered for the full depth of the exploration. The alluvium was not fully penetrated by any of their explorations. Alluvial soils range from clay to sandy gravel, with interbedded silt, fine silty sand, coarse silty sand, and gravelly sand. Layers of organic-rich silty clay to clay are reported in the exploration logs. In addition, pieces of wood and logs are reported at depths ranging from 2 to 28 feet in 12 of the explorations.

The USACE (1966) noted that fine-grained, impervious soil layers ranging in depth from 0 to 25 feet and 6.5 to 19 feet for the right and left levee, respectively, are present. The impervious soils are described as silt, sandy silt, and silty sand with some sandy clay. Below the impervious layers are pervious materials consisting of sand and sandy gravel with cobbles and boulders. The pervious layer is reported to be greater than 100 feet thick and could be 200 feet or more in thickness (USACE, 1966).

Stabilized groundwater depths were not established in the USACE explorations (USACE, 1966). The explorations were backfilled immediately upon completion of exploration; therefore, the stabilized water elevations were not measured. Depth to groundwater within the exploration programs advanced by USACE ranged from 2.5 to 19 feet below the existing ground surface at the time of exploration. Based on those explorations, the USACE (1966) states:

*"In general, the free water surfaces encountered during drilling of the holes near the creek correspond with the water level in the channel."*

### 3.1.2 Exploration Plan

CGI/Fugro 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 a spreadsheet and maps forwarded to the County on July 28, 2010. The work plan, spreadsheet, and maps detailed the following:

Exploration numbers, locations and methods, including:

- Sonic core locations;
- CPT locations; and
- Hollow-stem auger locations;
- Station number for each exploration;
- Relative locations on the levee, levee toe, or offsets for each exploration;
- Target depths of each exploration;
- The Assessor's Parcel Number and landowner name for proposed exploration locations;
- Dates or date ranges that each exploration were to be performed;
- Anticipated permits required for the explorations; and
- Methods of destruction of each exploration following completion.

This information was used for landowner notification by the County and also for permitting, 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 and the California Department of Transportation (Caltrans). Private lands owned by nine individuals or entities were located in areas of our exploration along proposed cross section lines. Per our contractual relationship, the County secured or attempted to secure permission to access and explore private properties at selected locations along the levee system. Of the nine private landholders, the County was able to secure permission for access and exploration from seven of the property owners. Only the properties at the offset location at the left side of the levee at about Station No. 185+00 and at the offset location on the right side of the levee at about Station No. 132+00 were inaccessible to us during this study.

Encroachment and/or environmental permits were required to perform exploration on public and private properties for this study. An encroachment permit was obtained from the California Department of Transportation (Caltrans) to perform exploration at the offset location on the right side of the levee at about Station 158+50. Environmental permits for all exploration locations for this project were obtained by CGI/Fugro from Humboldt County Department of Environmental Health (HCDEH). Prior to our field exploration program, the well permits were issued and documented in the project file.

### 3.1.4 Site Visit and Utility Clearance

Prior to performing drill holes, sonic core holes, and CPTs, each exploration location was visually inspected for the presence of overhead and underground utilities, and then marked with white paint and wood lathe, as required by 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 ten 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. In addition, during field exploration, it was necessary to relocate one drill hole due to a landowner conflict, thus, necessitating contact with USA for an additional ticket number.

No utility conflicts were identified at the site of any of CGI/Fugro's exploration locations and no manmade subsurface obstructions were encountered during our field exploration program. However, it should be noted that other buried utilities and structures not located by USA or observed by CGI/Fugro may be present along the project extent.

The proximity to overhead utilities was evaluated at each exploration location. In general, a clearance of at least 10 feet was maintained between drill rig mast and active overhead utilities (power lines).

## 3.2 EXPLORATION PROGRAM

The field exploration program completed by CGI/Fugro for the Redwood Creek Levee Geotechnical Evaluation project consisted of sounding and observations of existing relief wells, and subsurface exploration using sonic core holes, hollow-stem-auger drill holes, and CPTs. The following sections discuss those exploration services.

### 3.2.1 Relief Wells

As part of this study, existing, accessible relief wells located at various locations along the levee system were observed and sounded to estimate the depth to water and the existing bottom depth of each well. Prior to CGI/Fugro accessing the wells, the County removed vegetation where known wells were obscured. CGI/Fugro then opened well covers on each of those wells, photographed the well casings, sounded the wells (where possible), and down-hole videotaped portions of each well.

The relief wells were observed to consist of 12-inch internal diameter corrugated metal pipe (CMP) capped with a locking, hinged, plate-steel cover. As indicated by the down-hole camera, the CMP extended to varying depths then transitioned to a screened pipe near the groundwater table. The screened pipe is noted on the project plans as consisting of a wood-stove screen that extends to the bottom of the relief well (USACE, 1969). It is anticipated that the relief wells were installed contemporaneously with or shortly after the construction of the levee system.

According to USACE (1969), a total of 54 relief wells were installed by 1967 and should be present along the levee system. During this study, we located a total of 39 relief wells for which the locations are noted on Plate 5-3. Table 3-2 presents the information gathered from our observations and sounding of relief wells during this study.

## Table 3-2. Relief Well Information

| Relief Well No. | Date Observed/ Sounded | Sounded Depth to Relief Well Bottom | Sounded Depth of Well in 1966[1] | Depth to Water | Comments |
|---|---|---|---|---|---|
| 1 | 9/28/10 | NA | NA | 15.1 | |
| 2 | 9/28/10 | NA | NA | 14.5 | |
| 4 | 9/8/10 | 62.4 | 63.0 | 15.4 | |
| 5 | 9/8/10 | NA | 64.1 | NA | Obstructed at depth of 15.4 feet. |
| 6 | 9/8/10 | 91.0 | 92.9 | 14.3 | |
| 7 | 9/8/10 | 91.5 | 92.6 | 14.8 | |
| 8 | 9/8/10 | 86.3 | 87.3 | 14.2 | |
| 9 | 9/8/10 | 92.5 | 93.1 | 14.4 | |
| 10 | 9/8/10 | 91.1 | 91.6 | 14.7 | |
| 11 | 9/8/10 | 98.0 | 98.3 | 14.4 | |
| 12 | 9/8/10 | NA | 100.9 | NA | Obstructed at a depth of 7.6 feet. |
| 13 | 9/8/10 | 79.3 | 81.9 | 14.0 | |
| 15[3] | 9/8/10 | 100.3 | 87.9[3] | 13.4 | |
| 16 | 9/8/10 | NA | 96.1 | NA | Could not open lock on casing. |
| 17 | 9/8/10 | 96.3 | 98.1 | 13.8 | |
| 18 | 9/8/10 | 96.2 | 97.8 | 14.2 | |
| 19 | 9/8/10 | 95.5 | 97.2 | 14.2 | Wood or pipe obstruction at 7.5 feet. |
| 20 | 9/8/10 | 92.4 | 92.2 | 14.7 | |
| 21 | 9/8/10 | 84.6 | 85.2 | 16.3 | |
| 23 | 9/8/10 | 96.6 | 98.1 | 16.5 | |
| 24 | 9/8/10 | 95.1 | IL | 16.4 | Wood obstruction at a depth of 6.8 feet |
| 25 | 9/8/10 | NA | IL | NA | Rock and wood obstructions at 9.5 feet |
| 26 | 9/8/10 | 84.3 | 95.4 | 16.2 | Obstruction at a depth of 72 feet. |
| 27 | 9/8/10 | 94.2 | IL | 15.8 | |
| 28 | 9/8/10 | 80.7 | 95.4 | 15.1 | Obstruction observed at a depth of 73 feet. |
| 29 | 9/8/10 | NA | 82.2 | NA | Within well house. Pump and piping obstructed sounder and downhole camera. |
| 30 | 9/8/10 | NA | 96.1 | 23.5 | Debris obscuring well at depth of 28.3 feet. |
| 31 | 9/8/10 | 90.5 | 91.1 | 15.4 | |
| 32 | 9/8/10 | 90.2 | 90.8 | 15.9 | |
| 33 | 9/8/10 | 87.8 | 88.7 | 16.1 | Top of well deformed, partially obscuring opening |
| 34 | 9/8/10 | 48.8 | 50.3 | 16.1 | |

## Table 3-2. Relief Well Information (Continued)

| Relief Well No. | Date Observed/ Sounded | Sounded Depth to Relief Well Bottom | Sounded Depth of Well in 1966[1] | Depth to Water | Comments |
|---|---|---|---|---|---|
| 35 | 9/8/10 | NA | 49.1 | NA | Cobbles & debris obstructions at 3.4 feet. |
| 36[4] | 9/14/10 | NA | 51.0 | NA | Cobbles & debris obstructions at 4.5 feet. |
| 49 | 9/8/10 | 48.7 | 50.7[2] | 11.7 | PVC pipeline extending into well casing and cap to offsite well house. |
| 50 | 9/8/10 | 52.0 | IL[2] | 12.8 | Heavy algal growth of water surface. Obstruction at a depth of 13.9 feet. |
| 51 | 9/8/10 | 54.0 | 55.3[2] | 13.3 | Heavy algal growth on water surface. |
| 52 | 9/8/10 | 54.8 | 55.3[2] | 13.1 | Heavy algal growth on water surface. |
| 53 | 9/8/10 | 48.5 | 51.0[2] | 13.2 | Heavy algal growth on water surface. |
| 54 | 9/8/10 | 50.2 | 51.4[2] | 13.6 | Heavy algal growth on water surface. |

NA - Not available; IL - Illegible

[1] - per USACE (1969); [2] - 1967 measurement from USACE (1969); [3] - Well No. 15 likely corresponds to Well No. 14 from USACE (1969); [4] - Well No. 36 likely corresponds to USACE (1969) Well No. 48.

USACE (1969) records show that Relief Well Nos. 3, 14, and 37 through 48 at one time were present along the alignment. Levee stationing presented in USACE (1969) appears very close to stationing used within this study and using historical relief well station locations, we can approximate where the missing relief wells should be present. Relief Well No. 3 should be present at about Station No. 128+50 on the right levee, immediately downstream from the Highway 101 Bridge. That area is moderately overgrown with brambles and a relief well was not observed while we were making measurements; however, it was observed at another time during this study and is present. Relief Well No. 14 might correspond with Relief Well No. 15 noted in this study. The missing relief well (14 or 15) is likely present beneath fill material located adjacent to the toe of the levee. Relief Well Nos. 37 through 48 are noted as being present along the right levee between about Station Nos. 144+50 and 158+50. Those wells were not observed during this study and could be obscured by vegetation along the toe of the levee. Based on the historical data, it is likely that our Relief Well No. 36 corresponds to Relief Well No. 48 of USACE (1969).

As noted on Table 3-2, a number of relief wells were observed to have debris, rocks, and other obstructions present at varying depths. In addition, at Relief Well Nos. 29 and 49, PVC or metal piping has been installed into the wells apparently to be used for extraction of water. Heavy algal growth is present on the water surface within Relief Well Nos. 50 through 54.

Comparing the bottom depths of the wells sounded during this study to wells sounded in 1966 or 1967 (USACE, 1969) indicates that some sanding of the wells has occurred during that 44-year period. In general, approximately 0.3 to 14.7 feet of sanding were measured in the accessible wells. Some of that sanding could be the result of debris at or near the bottom of the wells.

### 3.2.2 Subsurface Exploration

The locations of core holes, drill holes, and CPTs performed for this investigation are shown on Plate 5-3. Information for each exploration including latitude and longitude, ground surface elevations, and total exploration depth are tabulated in Tables 3-3a and 3-3b.

## 3.3 REFERENCE DATUM

The following reference datums were used for this project:

**Horizontal Datum.** The horizontal datum reference for this project is 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/Fugro during the week of November 1, 2010 using an Ashtech (Magellan Professional) MobileMapper 6 global positioning system receiver. Lidar data were provided by the National Park Service and Humboldt County for CGI/Fugro's use in engineering analyses.

**Table 3-3a. Subsurface Explorations Summary - HSA and Sonic Drill Holes**

| Drill Hole | Estimated Ground Surface Elevation (feet, NAVD88) | Completion Depth (feet) | Start Date | Completion Date | NAD83 Coordinates | |
|---|---|---|---|---|---|---|
| | | | | | Northing | Easting |
| **HSA Drill Holes** | | | | | | |
| H057LW | 15.7 | 26.5 | 10/15/2010 | 10/15/2010 | 2361058 | 5991233 |
| H057RO | 19.2 | 26.5 | 10/13/2010 | 10/13/2010 | 2361477 | 5991575 |
| H057RW | 19.2 | 26.5 | 10/12/2010 | 10/12/2010 | 2361355 | 5991407 |
| H073LL | 29.6 | 26.5 | 10/15/2010 | 10/15/2010 | 2359772 | 5991199 |
| H073LW | 17.1 | 26.5 | 10/15/2010 | 10/15/2010 | 2359813 | 5991248 |
| H073RW | 20.5 | 26.5 | 10/12/2010 | 10/12/2010 | 2359816 | 5991545 |
| H087LW | 18.9 | 26.5 | 10/15/2010 | 10/15/2010 | 2358415 | 5991851 |
| H087RW | 17.9 | 26.5 | 10/8/2010 | 10/8/2010 | 2358638 | 5992081 |
| H098LW | 21.6 | 26.5 | 10/14/2010 | 10/14/2010 | 2358561 | 5993174 |
| H098RW | 22.6 | 26.5 | 10/7/2010 | 10/7/2010 | 2358816 | 5993027 |
| H114LW | 23.2 | 26.5 | 10/14/2010 | 10/14/2010 | 2358970 | 5994614 |
| H114RL | 20.1 | 26.5 | 10/7/2010 | 10/7/2010 | 2359387 | 5994624 |
| H114RW | 24.9 | 26.5 | 10/7/2010 | 10/7/2010 | 2359258 | 5994611 |
| H132LL | 32.0 | 26.5 | 10/6/2010 | 10/6/2010 | 2359230 | 5996539 |
| H132LO | 31.5 | 26.5 | 10/6/2010 | 10/6/2010 | 2359169 | 5996543 |
| H132LW | 25.3 | 26.5 | 10/5/2010 | 10/5/2010 | 2359307 | 5996464 |
| H132RL | 28.7 | 26.5 | 9/29/2010 | 9/29/2010 | 2359656 | 5996257 |
| H132RO | 30.6 | 26.5 | 10/13/2010 | 10/13/2010 | 2359973 | 5996361 |
| H132RW | 21.4 | 26.5 | 9/29/2010 | 9/29/2010 | 2359538 | 5996325 |
| H146LL | 33.4 | 26.5 | 10/5/2010 | 10/5/2010 | 2360462 | 5997220 |
| H146LO | 32.1 | 26.5 | 10/7/2010 | 10/7/2010 | 2360425 | 5997319 |
| H146LW | 26.6 | 26.5 | 10/5/2010 | 10/5/2010 | 2360481 | 5997118 |
| H146RL | 33.8 | 26.5 | 10/13/2010 | 10/13/2010 | 2360699 | 5996761 |

## Table 3-3a. Subsurface Explorations Summary - HSA and Sonic Drill Holes (Continued)

| Drill Hole | Estimated Ground Surface Elevation (feet, NAVD88) | Completion Depth (feet) | Start Date | Completion Date | NAD83 Coordinates | |
|---|---|---|---|---|---|---|
| | | | | | Northing | Easting |
| H146RO | 33.1 | 26.5 | 10/14/2010 | 10/14/2010 | 2360739 | 5996706 |
| H146RW | 17.4 | 26.5 | 9/29/2010 | 9/29/2010 | 2360638 | 5996889 |
| H158LL | 36.5 | 26.5 | 10/1/2010 | 10/1/2010 | 2361622 | 5997652 |
| H158LO | 31.8 | 26.5 | 10/6/2010 | 10/6/2010 | 2361597 | 5997720 |
| H158LW | 29.7 | 26.5 | 10/4/2010 | 10/4/2010 | 2361648 | 5997554 |
| H158RL | 34.0 | 26.5 | 9/28/2010 | 9/28/2010 | 2361793 | 5997186 |
| H158RO | 34.6 | 26.5 | 10/14/2010 | 10/14/2010 | 2361819 | 5997132 |
| H158RW | 21.7 | 26.5 | 9/28/2010 | 9/28/2010 | 2361746 | 5997315 |
| H171LL | 36.4 | 26.5 | 10/1/2010 | 10/1/2010 | 2362531 | 5998205 |
| H171LO | 35.9 | 26.5 | 10/1/2010 | 10/1/2010 | 2362514 | 5998246 |
| H171LW | 32.3 | 26.5 | 9/30/2010 | 9/30/2010 | 2362592 | 5998137 |
| H185LL | 41.1 | 26.5 | 9/30/2010 | 9/30/2010 | 2362619 | 5999862 |
| H185LW | 31.5 | 26.5 | 9/30/2010 | 9/30/2010 | 2362793 | 5999709 |
| **Sonic Drill Holes** | | | | | | |
| S057LC | 27.8 | 50 | 9/24/2010 | 9/24/2010 | 2361009 | 5991202 |
| S057RC | 28.7 | 50 | 9/18/2010 | 9/18/2010 | 2361390 | 5991452 |
| S073LC | 29.4 | 50 | 9/23/2010 | 9/23/2010 | 2359809 | 5991200 |
| S073RC | 28.0 | 50 | 9/17/2010 | 9/17/2010 | 2359829 | 5991609 |
| S087LC | 32.2 | 50 | 9/23/2010 | 9/23/2010 | 2358356 | 5991852 |
| S087RC | 32.8 | 50 | 9/17/2010 | 9/17/2010 | 2358703 | 5992130 |
| S098LC | 33.2 | 50 | 9/23/2010 | 9/23/2010 | 2358522 | 5993213 |
| S098RC | 33.7 | 50.5 | 9/16/2010 | 9/16/2010 | 2358875 | 5993028 |
| S114LC | 35.2 | 50 | 9/21/2010 | 9/21/2010 | 2358917 | 5994632 |
| S114RC | 35.2 | 50 | 9/16/2010 | 9/16/2010 | 2359318 | 5994607 |
| S132LC | 40.4 | 50 | 9/21/2010 | 9/21/2010 | 2359249 | 5996502 |
| S132RC | 41.1 | 50 | 9/15/2010 | 9/15/2010 | 2359615 | 5996289 |
| S146LC | 39.9 | 50 | 9/20/2010 | 9/20/2010 | 2360460 | 5997184 |
| S146RC | 41.6 | 50 | 9/15/2010 | 9/15/2010 | 2360683 | 5996813 |
| S158LC | 41.6 | 50 | 9/20/2010 | 9/20/2010 | 2361631 | 5997624 |
| S158RC | 42.5 | 51.5 | 9/14/2010 | 9/14/2010 | 2361775 | 5997209 |
| S171LC | 42.3 | 50 | 9/18/2010 | 9/18/2010 | 2362553 | 5998192 |
| S185LC | 48.6 | 50 | 9/20/2010 | 9/20/2010 | 2362728 | 5999676 |

## Table 3-3b. Subsurface Explorations Summary - Cone Penetration Tests

| CPT | Estimated Ground Surface Elevation (feet, NAVD88) | Completion Depth (feet) | Start Date | Completion Date | NAD83 Coordinates | |
|---|---|---|---|---|---|---|
| | | | | | Northing | Easting |
| C-57LC | 27.7 | 50.2 | 9/24/2010 | 9/24/2010 | 2361026 | 5991192 |
| C-57LL | 20.3 | 25.1 | 9/23/2010 | 9/23/2010 | 2360999 | 5991145 |
| C-57LO | 20.5 | 25.1 | 9/23/2010 | 9/23/2010 | 2361017 | 5991145 |
| C-57RC | 28.6 | 50.0 | 9/22/2010 | 9/22/2010 | 2361403 | 5991435 |
| C-57RL | 27.1 | 50.0 | 9/22/2010 | 9/22/2010 | 2361423 | 5991486 |
| C-57RO | 19.4 | 25.1 | 9/22/2010 | 9/22/2010 | 2361467 | 5991572 |
| C-73LC | 29.3 | 44.6 | 9/24/2010 | 9/24/2010 | 2359820 | 5991194 |
| C-73LL | 21.6 | 25.1 | 9/24/2010 | 9/24/2010 | 2359817 | 5991166 |
| C-73LO | 22.0 | 25.1 | 9/23/2010 | 9/23/2010 | 2359816 | 5991144 |
| C-73RC | 28.6 | 50.0 | 9/21/2010 | 9/21/2010 | 2359828 | 5991605 |
| C-73RL | 23.6 | 25.1 | 9/21/2010 | 9/21/2010 | 2359823 | 5991634 |
| C-73RO | 21.3 | 25.1 | 9/22/2010 | 9/22/2010 | 2359826 | 5991658 |
| C-87LC | 32.5 | 31.3 | 9/23/2010 | 9/23/2010 | 2358357 | 5991844 |
| C-87LL | 27.5 | 25.1 | 9/23/2010 | 9/23/2010 | 2358322 | 5991833 |
| C-87LO | 23.7 | 25.4 | 9/23/2010 | 9/23/2010 | 2358275 | 5991805 |
| C-87RC | 32.6 | 25.6 | 9/21/2010 | 9/21/2010 | 2358710 | 5992107 |
| C-87RL | 26.9 | 25.1 | 9/21/2010 | 9/21/2010 | 2358731 | 5992124 |
| C-87RO | 26.8 | 25.1 | 9/22/2010 | 9/22/2010 | 2358740 | 5992128 |
| C-98LC | 33.4 | 38.7 | 9/24/2010 | 9/24/2010 | 2358517 | 5993202 |
| C-98LL | 26.8 | 25.1 | 9/23/2010 | 9/23/2010 | 2358470 | 5993221 |
| C-98LO | 25.6 | 25.1 | 9/23/2010 | 9/23/2010 | 2358386 | 5993274 |
| C-98RC | 33.7 | 50.5 | 9/21/2010 | 9/21/2010 | 2358872 | 5993013 |
| C-98RL | 28.2 | 25.4 | 9/21/2010 | 9/21/2010 | 2358899 | 5993005 |
| C-98RO | 28.3 | 25.1 | 9/21/2010 | 9/21/2010 | 2358923 | 5992999 |
| C-114LC | 35.3 | 26.2 | 9/24/2010 | 9/24/2010 | 2358918 | 5994641 |
| C-114LL | 28.6 | 25.3 | 9/22/2010 | 9/22/2010 | 2358841 | 5994627 |
| C-114LO | 28.5 | 25.1 | 9/22/2010 | 9/22/2010 | 2358786 | 5994643 |
| C-114RC | 34.2 | 50.0 | 9/21/2010 | 9/21/2010 | 2359329 | 5994587 |
| C-114RO | 26.7 | 50.2 | 9/24/2010 | 9/24/2010 | 2359428 | 5994591 |
| C-132LC | 40.1 | 50.0 | 9/24/2010 | 9/24/2010 | 2359243 | 5996497 |
| C-132RC | 40.9 | 50.2 | 9/20/2010 | 9/20/2010 | 2359612 | 5996278 |
| C-146LC | 40.2 | 39.9 | 9/24/2010 | 9/24/2010 | 2360459 | 5997183 |
| C-146RC | 41.1 | 50.0 | 9/20/2010 | 9/20/2010 | 2360675 | 5996804 |
| C-158LC | 41.4 | 50.5 | 9/24/2010 | 9/24/2010 | 2361622 | 5997627 |
| C-158RC | 41.4 | 50.0 | 9/20/2010 | 9/20/2010 | 2361764 | 5997202 |
| C-171LC | 41.6 | 42.0 | 9/23/2010 | 9/23/2010 | 2362561 | 5998199 |
| C-185LC | 47.0 | 50.0 | 9/23/2010 | 9/23/2010 | 2362725 | 5999672 |

The sonic cores were advanced through the levee crest at about where each cross section crosses the levee system. Core depths of 50 feet were generally obtained during this study. Continuous core samples were obtained to the depths explored in each sonic core hole, with the exception of isolated depth increments where no recovery occurred. Samples were bagged in about 3-foot depth increments and delivered to CGI's laboratory in Redding, California, for testing and evaluation.

Thirty-seven CPTs were advanced as part of this study. Generally, one CPT sounding was advanced adjacent to each sonic core hole within the crown of the levee system. Those CPTs were targeted to a depth of 50 feet; however, Table 3-4 presents CPTs that encountered practical refusal at depths of less than 50 feet:

**Table 3-4. CPT Refusal Depths**

| CPT Sounding | Refusal Depth (feet) |
|---|---|
| C73LC | 44.5' |
| C87LC | 31.5' |
| C98LC | 38.5' |
| C114LC | 26' |
| C146LC | 40' |
| C171LC | 42' |

In addition to CPT soundings in the levee crown, a number of CPT soundings were performed at the landside toe and offset locations along the levees downstream of the Highway 101 bridge. The toe and offset CPTs were targeted to depths of 25 feet.

Thirty-six drill holes were advanced for this study using a track-mounted, hollow-stem auger drill rig. The drill holes were advanced in the following locations:

- At all waterside toe locations at the cross section sites;
- At all accessible landside and offset locations upstream of the Highway 101 bridge;
- At selected landside toe and offset locations downstream from the Highway 101 bridge.

The target depth of the drill holes was 25 feet (up to 26.5 feet following sampling at the bottom of the drill hole), which was achieved by each of the drill holes. Soil samples were obtained at selected depth intervals within the drill holes using Shelby tubes, California modified split-spoon, and Standard Penetration Test (SPT) samplers. A CGI engineering geologist logged the drill holes as they were advanced. All Shelby tube and California Modified split-spoon samples were delivered to Fugro's San Luis Obispo office for testing. All SPT samples were transmitted to CGI's Redding laboratory for testing.

All explorations were destroyed using cement grout in accordance with HCDEH permit conditions. The grout was placed through the sonic core barrel, hollow-stem auger, or CPT rods, which were extracted as the grout was being placed to reduce the potential for bridging or collapse of the holes prior to grout placement.

## 3.4 INVESTIGATION DERIVED CUTTINGS DISPOSAL

CPT and sonic cores did not generate cuttings that required disposal during the course of this study. The CPT soundings displace underlying sediments and, therefore, do not generate cuttings that require disposal. The sonic cores were collected during the sampling process and returned to our laboratory, thus, few cuttings were generated.

Hollow-stem-auger drill holes did generate cuttings during the exploration process. Because those holes were backfilled using cement grout, the cuttings required disposal. With the exception of explorations on private properties along Drydens Road, the cuttings were dispersed adjacent to the drill hole locations, as discussed in our exploration plan and as reviewed and approved of by HCDEH during permit application. Those cuttings generated on the private properties adjacent to Drydens Road were disposed of at the aggregate stockpile locations along the levee system.

## 4.0 LABORATORY TESTING PROGRAM

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 3.0-inch diameter thin-wall push (i.e., Shelby) samples, 2.4-inch diameter drive (i.e., ring) samples, SPT samples, and bulk samples. Density evaluations, consolidation tests, permeability tests and strength tests were typically performed only on relatively undisturbed Shelby and ring samples.

The numbers of the various tests conducted for the Redwood Creek 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] |
|---|---|---|
| Moisture Content | 146 | ASTM D2216 |
| Density | 27 | ASTM D2937 |
| Sieve Analysis with #200 Wash | 114 | ASTM D422 |
| Sieve and Hydrometer | 13 | ASTM D422 |
| Atterberg Limits | 18 | ASTM D4318 |
| Modified Proctor | 2 | ASTM D1557 |
| Consolidation (Incremental Load Control) | 3 | ASTM D2435 |
| Flexible Membrane Permeability | 15 | ASTM D5084 |
| Constant Head Permeability | 1 | ASTM D2434 |
| Direct Shear | 14 | ASTM D3080 |
| Consolidated Undrained (CU) Triaxial Compression Test on Cohesive Soils with R-Pore Pressure Measurement | 1 | ASTM D4767 |

1) ASTM International (2005)

Laboratory test results are tabulated or presented graphically in Appendix C. A tabular summary of laboratory test results for the Redwood Creek Levee Evaluation is presented on Plates C-1a through C-1e. Various laboratory test results are also tabulated versus depth on the individual drill hole logs (Plates B-2 through B-55). 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 test results, triaxial test results, 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 (Hines, 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 (i.e., the North American, Pacific, and Gorda plates). North of Cape Mendocino, the Gorda plate is being actively subducted beneath North America, forming 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 Redwood Creek region consists of the Cretaceous-Jurassic Franciscan Formation (Cashman et al., 1995). 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 Redwood Creek basin is situated on the Eastern Belt and is juxtaposed to the relatively older Klamath Mountains province to the northeast (Harden et al., 1982). In the project region, it is composed of northwest-trending units of schist, sandstone, and mudstone, each separated by faults (Harden et al., 1995).

## 5.2 LOCAL GEOLOGIC SETTING

Coastal valleys, such as Orick Valley, represent sediment-filled estuaries that 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 all coastal streams would be graded. During these low sea levels, streams within the coastal valleys around Humboldt County would be incised. Subsequent sea level fluctuations would result in cycles of filling and incision in these coastal valleys, depending on the relative base level (i.e., the ocean shoreline). 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 sediment filling the Orick Valley would be anticipated to be mid-Holocene in age, or younger.

The sediment filling Orick Valley is alluvium derived from Redwood Creek, Prairie Creek, and other upstream sources, and it is relatively unconsolidated and locally contains high concentrations of organics and woody debris. Associated with the alluvium are colluvial soils along the margins of the valley and concentrations of artificial fill materials, such as the levee system and approach ramps for the Highway 101 bridge crossing Redwood Creek. A geologic map of the project area and a regional fault map are shown on Plates 5-1 and 5-2., respectively.

## 5.3 SUBSURFACE CONDITIONS

As previously noted, the project field exploration program included sonic cores, CPT soundings, and hollow-stem auger drill holes along the crest of the Redwood Creek levees, 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 50 feet below ground surface (see Section 3.0 for details of the field exploration program). Subsurface conditions are described and illustrated in the sonic core, drill hole, and CPT 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 5-3 for a map showing the location of the cross-sections and Plates 5-4 and 5-5 for the cross sections. Ten transverse cross sections (57, 73, 87, 98, 114, 132, 146, 158, 171 [left levee only], and 185 [left levee only]) were created by projecting exploration logs onto regional topographic data provided by the County and by LIDAR surveys performed for Redwood National Park.

### 5.3.1 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 both sides of the levee system. Foundation soils showed some variability. Earth materials encountered within the levee embankment and in the foundation soils are discussed in the following sections.

**Levee Embankment Materials.** Levee embankment thicknesses appear to range from about 6 feet to over 24 feet, as estimated from cross sections presented on Plates 5-4 and 5-5. A total of 36 explorations, consisting of sonic cores and CPT soundings, 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 observed that about 1 to 4 feet of granular soils are present below the crown of the levee. Those granular soils consist of sand, gravelly sand, sandy gravel, and gravel, some of which appears to be aggregate base materials. The granular soils were generally dry to damp, dense to very dense, with fine to coarse sand and fine to coarse gravel ranging from subrounded to angular. Tested samples of these granular soils indicate that they have a moisture content ranging from 3- to 8-percent, 10- to 16-percent soil fraction passing the No. 200 sieve, and plasticity indices (PI) of about 4. In addition, the near-surface levee granular soils have angles of internal friction (Ø) ranging from about 28 to 36 degrees and hydraulic conductivity values ranging from $1.6 \times 10^{-4}$ to $1.3 \times 10^{-3}$ centimeters per second (cm/sec). Unit dry weights of these materials ranged from 110 to 144 pounds per cubic foot (pcf). Locally, the upper few inches of the granular soils were loose. Local silt to sandy silt fill layers are present within these granular soils.

Silty clay, clay, and clay with gravel, clayey silt, and silt was encountered beneath the surficial granular soils within the levee. Those materials generally constitute the "impervious fill" noted on the USACE plans (1966) and extend to the bottom of the levee embankment soils. Occasionally, thin granular interbeds are present within the impervious fill, as discerned from the CPT logs. The impervious fill materials were damp, stiff to very stiff, and locally slightly plastic. In-situ moisture contents ranged from 8- to 15-percent and unit dry weights ranged from 111 to 117 pcf. These materials had PIs ranging from about 5 to 8, and hydraulic conductivities ranging from $3 \times 10^{-7}$ to $2.8 \times 10^{-5}$ cm/s. The impervious fill materials tested had Ø values ranging from 30 to 34 degrees.

**Foundation Soils.** The USACE (1966) notes, on Plate A-11, that imperious blanket materials and pervious foundation soils are present beneath the levee embankment. The following sections discuss foundation soils encountered during this study relative to generalized soil descriptions presented by USACE (1966).

**Impervious Blanket Materials.** The USACE (1966) report indicates the presence of impervious blanket materials consisting of silty sand, sandy silt, silty, clayey silt, silty clay, and clay varying in thickness from nonexistent to about 25 feet within the upper portion of the foundation soils. The USACE projected that the thickness of the impervious blanket materials decreases significantly beneath the right levee, especially between Station Nos. 85+00 to 105+00 and 133+00 to 140+00, and slightly beneath the left levee as the levees progresses upstream.

CGI/Fugro encountered similar materials during this study as described by the USACE as "impervious blanket materials." Those materials predominantly consisted of clayey silt, silty clay, sandy silt, with lesser amounts of silty clay to clay and silty sand to sand, as noted on Plates 5-4 and 5-5. We encountered thicker sections and relatively more abundant silty clay to clay between about Station Nos. 57+00 and 100+00 beneath the left levee and 57+00 and 70+00 beneath the right levee, relative to the remainder of the levee system explored. Minor, thin silty clay to clay interbeds were encountered in CPT soundings upstream from Station No. 158+00 on the left levee and upstream from about Station 114+00 on the right levee.

Thicknesses of those materials varied from a few feet to about 25 feet, with an average thickness of about 18.4 and 14.5 feet for the right and left levee, respectively. For the right levee, the impervious blanket materials encountered had a maximum and minimum thickness of about 25 and 13 feet, respectively. The maximum and minimum thickness of those materials for the left levee was estimated to be about 18 and 11 feet, respectively. Table 5-1 presents some material properties for the impervious blanket materials encountered during our study:

### Table 5-1. Impervious Blanket Material Properties

| Soil Property Ranges | Average Thickness (ft) | Approximate Amounts Silty Clay & Clay (%) | Dry Unit Weight (pcf) | Moisture Content (%) | Fines (%) | Plasticity Index | Angle of Internal Friction (degrees) | Hydraulic Conductivity (cm/sec) |
|---|---|---|---|---|---|---|---|---|
| **Left Levee** | | | | | | | | |
| Station 57+00 to 98+00 | 13 | 44 | 125 | 32 | 25 | NA | 36 | 9.4x10$^{-4}$ |
| Station 98+00 to 185+00 | 17 | 53 | 75-103 | 5-58 | 8-27 | 7-12 | 33-34 | 6.4x10$^{-3}$ - 1.8x10$^{-6}$ |
| **Right Levee** | | | | | | | | |
| Station 57+00 to 73+00 | 23 | 74 | 78-95 | 27-33 | 60 | 1-15 | 36-39 | 3.3x10$^{-6}$ - 9.3x10$^{-6}$ |
| Station 73+00 to 87+00 | 13 | 42 | NA | NA | NA | NA | NA | NA |
| Station 87+00 to 114+00 | 10 | 57 | NA | NA | NA | NA | NA | NA |
| Station 114+00 to 185+00 | 16 | 57 | NA | 10-19 | 10-27 | NA | NA | NA |

NA - Not Available.

***Pervious Foundation Soils*.** The USACE (1966) encountered pervious foundation soils underlying the impervious blanket materials in all explorations. Those materials were reported as silty sand, sand, sand with gravel, gravel with sand, and gravel, with few thin interbeds of sandy silt, silty clay, and clay. The USACE explorations did not fully penetrate the thickness of the pervious foundation materials to depths of 100 feet.

CGI/Fugro encountered similar pervious foundation materials as reported by the USACE (1966). In general, the pervious foundation materials encountered during our study consisted of loose to very dense, moist to wet, poorly- to well-graded sand, sand with gravel, and gravel with sand. The pervious foundation soils were not fully penetrated by explorations performed for this study.

Relatively fine-grained interbeds, consisting of silty sand, sandy silt, silty clay, and clay, were encountered locally within the pervious explorations. Those interbeds were encountered predominantly below an elevation of about -23 to -28 feet (NAVD88) at Station 57+00 at both levees, and between elevations -14 and -21 feet (NAVD88) at Station 114+00 in the right levee. Additional relatively fine-grained interbeds ranging in thickness from about 1 to 3 feet were encountered within other explorations, as noted on Plates 5-4 and 5-5.

The pervious foundation soils tested as part of this study had dry, in-situ unit weights ranging from 83 to 106 pcf, in-situ moisture contents ranging from 3 to 30 percent, Ø values ranging from 36 to 44 degrees, and hydraulic conductivity values ranging from 1.3x10$^{-2}$ to 7.9x10$^{-3}$ cm/sec.

### 5.3.2 Groundwater

Depths to groundwater were measured in existing relief wells, hollow-stem auger drill holes, and sonic coring holes advanced as part of this study. Depths to groundwater cannot be measured directly using CPT soundings and dissipation tests were not performed because those soundings were generally placed proximal to the drill holes or sonic core holes.

The observed groundwater levels varied from about 5 to 32 feet below ground surface, with elevations ranging from about -2 to 26 feet (NAVD88). Groundwater levels observed in the field are summarized on Table 5-2 and presented graphically on Plate 5-9.

**Table 5-2. Groundwater Level Observations in Study Explorations**

| Station & Measurement Date | Groundwater Elevations (NAVD88) | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| | Left Levee | | | | Right Levee | | | |
| | Offset | Landside Toe | Crest | Water-side Toe | Water-side Toe | Crest | Landside Toe | Offset |
| Station 57+00 | | | 2 | 8 | 8 | 9 | | 14 |
| Date | | | 9/24/10 | 10/15/10 | 10/12/10 | 9/18/10 | | 10/13/10 |
| Station 73+00 | | 14 | -2 | 5 | 11 | -1 | | |
| Date | | 10/15/10 | 9/23/10 | 10/15/10 | 10/12/10 | 9/17/10 | | |
| Station 87+00 | | | 6 | 5 | 9 | 5 | | |
| Date | | | 9/23/10 | 10/15/10 | 10/8/10 | 9/17/10 | | |
| Station 98+00 | | | 5 | 6.5 | 12.5 | 6 | | |
| Date | | | 9/23/10 | 10/14/10 | 10/7/10 | 9/16/10 | | |
| Station 114+00 | | | 11 | 10 | 14 | 6 | 9 | |
| Date | | | 9/21/10 | 10/14/10 | 10/7/10 | 9/16/10 | 10/7/10 | |
| Station 132+00 | 15 | 15 | 10.5 | 15 | 12 | 11 | | 13 |
| Date | 10/6/10 | 10/6/10 | 9/21/10 | 10/5/10 | 9/29/10 | 9/15/10 | | 10/13/10 |
| Station 146+00 | 15 | | 12 | 16.5 | 13 | 14 | 15 | 8 |
| Date | 10/7/10 | | 9/20/10 | 10/5/10 | 9/29/10 | 9/15/10 | 10/13/10 | 10/14/10 |
| Station 158+00 | 19 | 19 | 11.5 | 19 | 17 | 18.5 | 19 | 16 |
| Date | 10/6/10 | 10/1/10 | 9/20/10 | 10/4/10 | 9/28/10 | 9/14/10 | 9/28/10 | 10/14/10 |
| Station 171+00 | 19 | 19 | 14.5 | 18 | | | | |
| Date | 10/1/10 | 10/1/10 | 9/18/10 | 9/30/10 | | | | |
| Station 185+00 | | 26 | 21 | 24 | | | | |
| Date | | 9/30/10 | 9/20/10 | 9/30/10 | | | | |

In addition, groundwater levels in the project area were researched through the California Department of Water Resources (2011) and through the California State Water Resources Control Board Geotracker database (CSWRCB, 2011). Based on information obtained from those sources and from data provided by the County at the start of this project, we have compiled historical groundwater levels in the project area. Those data are presented in Table 5-3. Groundwater levels were also measured for relief wells and were presented in Table 3-2.

### Table 5-3. Groundwater Data Reported by Others

| Source | General Location | | | Groundwater Elevation Range (NAVD 88) | Monitoring Period Range |
|---|---|---|---|---|---|
| | Site | Longitude | Latitude | | |
| SHN (2006b) | 121444 Highway 101 | 41° 17' 39" | 124° 3' 21" | 11.75 to 16.35 NGVD29 | 2/2004 to 2/2006 |
| SHN (2006a) | Shoreline Market | 41° 17' 6" | 124° 4' 24.5" | 7.36 to 14.33 NGVD29 | 1/2004 to 4/2006 |
| HCDEH (2008) | 120784 Highway 101 | 41° 17' 5" | 124° 3' 48" | >14' bgs | NA |
| SHN (2009) | Orick WWTF | 41° 17' 33" | 124° 3' 31" | 17.2 to 22.25 datum unknown | 10/2008 to 3/2009 |
| Galli Group (2011) | Strawberry Creek Bridge | 41° 17' 13" | 124° 4' 43" | 3.0 to -2.5 datum unknown | 2/18/2011 |
| DWR (2011) | 10N01E04D004H | 41° 17' 6" | 124° 4' 20" | 5.8 to 6.7 datum unknown | 10/2010 to 4/2011 |
| DWR (2011) | 10N01E04C001H | 41° 17' 6" | 124° 3' 17" | 4.6 to 14.4 datum unknown | 3/1983 to 10/2002 |

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. In addition, some variations in groundwater depths could be related to on site waste disposal systems present at residential developments located adjacent to the levee system, such as between Station Nos. 132+00 and 146+00. 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 estimations of seismic hazards using three-dimensional earthquake sources. The USGS hazard model includes ground motions up to 2 standard deviations above or below the median for each earthquake considered. EC 1110-2-6067 (2010a) specifies that a Statistical Return Period of 100 years (1 percent annual chance of exceedance) be used when performing seismic evaluations; however, at the time of our analyses, the USGS website did not have the capability to provide estimations for a 100-year return period. Consequently, we utilized the next longest return period available on the website of 134 years (0.7 percent annual chance of exceedance) to determine spectral accelerations needed for the probabilistic seismic hazard evaluation.

The probabilistic spectral accelerations were calculated by the USGS website using equally-weighted ground motion values from three (3) attenuation relationships: Boore and Atkinson (2008), Campbell and Bozorgnia (2008), and Chiou and Youngs (2008). In addition, documentation provided by USGS in support of their website application indicates that, due to the site's proximity to the Cascadia subduction zone, weighted ground motion values from the following three attenuation relationships were also included in the USGS hazard model: Zhao and others (2006), Youngs and others (1997), and Atkinson and Boore (2003). For the purposes of our evaluation, the site coordinates (latitude and longitude) for the Redwood Creek project site were estimated to be 41.289 degrees north latitude and 124.059 degrees west longitude.

The estimated values of spectral acceleration ($S_a$) for the probabilistic response spectrum at a damping ratio of 5 percent are shown graphically on Plate 6-1. According to the USGS website, the peak horizontal ground acceleration (PHGA) for the project site is estimated to be approximately 0.2 g, corresponding to deaggregated modal and mean magnitudes of 9.0 and 6.85, respectively. According to the USGS (2008), this PHGA value is, on average, approximately 1.3 standard deviations below the median value and 0.1 standard deviations above the median value for the modal and mean earthquake sources, respectively.

Based on correlations developed by Sykora (1987), Mayne (2007), Dickenson (1994), and Mayne and Rix (1995), 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 180 meters per second (m/s), based on data collected from explorations C57RC and H146RO.

## 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 (Harden et al., 1982; 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 Grogan fault, located about 0.8 miles northeast of the northeast terminus of the levee system, as shown on Plate 5-2. The Grogan fault is a north-west trending, northeast-dipping thrust or reverse fault (Smith, 1981), or possibly a dip-slip/strike-slip fault (Cashman et al., 1995). The Grogan fault separates the schist of Redwood Creek from incoherent and coherent Franciscan sandstone and mudstone (Harden et al., 1982). Some authors report the fault as low angle (Smith, 1981); however, Harden et al. (1982) indicate that the straightness of the fault implies the fault is vertical to near-vertical. According to Smith (1981), the Grogan fault is poorly exposed near Orick except locally where clear cuts are present, and exhibits no geomorphic features that would imply relative recency in movement of the fault. Jennings & Bryant (2010) indicate the Grogan fault has failed during the Quaternary but do not differentiate when, during the last 1.6-million years, the fault last ruptured. Cashman et al. (1995) report that exposures of Franciscan Formation rocks juxtaposed against Pliocene or Pleistocene sediments were at one time exposed near Prairie Creek, thus, further constraining the age of movement.

The potentially active Bald Mountain fault is located about 3.2 miles southwest of the westerly terminus of the levee system, as shown on Plate 5-2. The Bald Mountain fault system juxtaposes unmetamorphosed Franciscan Formation and the schist of Redwood Creek (Smith, 1981; Harden et al., 1982). It is reported as a low-angle east-dipping thrust fault based on its sinuous and discontinuous exposures (Harden et al., 1982). Geomorphic features of the fault are locally well expressed; however, there is no evidence that the fault ruptures Holocene or late Pleistocene rocks or sediments along its length (Smith, 1981). Thus, the State (Smith, 1981) and Jennings & Bryant (2010) have labeled the fault as potentially active.

The closest zoned active fault is the Trinidad fault, located about 16 miles south of the project site, as shown on Plate 5-2. The Trinidad fault is part of the larger Mad River fault zone, which includes the Trinidad, McKinleyville, Mad River, Fickle Hill, and related faults (Smith, 1982), which extends from south of Arcata to west and offshore of Trinidad. The fault is northwest trending, east dipping, and offsets a number of marine terraces along the coastal zone (Carver, 1982, Smith 1982).

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

As part of our geotechnical investigation, ten cross sections were analyzed for seepage and stability. The locations of the cross sections are shown on Plate 5-3. The cross section locations generally conform to the locations presented in the County's Work Plan developed for the project (Humboldt County Department of Public Works, 2010), with the exception of sections located at about Station Nos. 171+00 and 185+00. The original work plan proposed Cross Section 171+00 to extend from east of Redwood Creek to west of Redwood Creek; however, the presence of formational rock materials on the west side of the creek precluded the need to extend the section to the west side. Instead, a new cross section encompassing the levee located south of Redwood Creek at about Station No. 185+00 was included in this study.

The cross sections analyzed are located between 1,100 and 1,800 feet apart, with an average of about 1,425-foot separations. They are generally equally distributed along the levee system, with five cross sections located upstream and five cross sections located downstream of the Highway 101 bridge crossing Redwood Creek.

The cross sections evaluated, herein, have generalized station number names to indicate the approximate locations of the cross section. Table 7-1 presents the stationing where the cross sections cross the stationing line established along the approximate centerline of Redwood Creek.

### Table 7-1. Cross Section Station Numbers

| Cross Section | Station Number |
|---|---|
| 57+00 | 57+60 |
| 73+00 | 72+55 |
| 87+00 | 86+90 |
| 98+00 | 98+25 |
| 114+00 | 114+25 |
| 132+00 | 132+55 |
| 146+00 | 145+80 |
| 158+00 | 157+90 |
| 171+00 | 169+25(east side only) |
| 185+00 | 189+30(south side only) |

CGI/Fugro approximated cross section topography on the basis of Lidar data collected by the National Park Service and provided by Humboldt County. In order to minimize the effect of vegetation on Lidar-based elevations, we compared the topography at each cross section location to measurements approximated in the field.

Our idealized seepage and stability analysis models were based on both the northern (right) levee and southern (left) levee of each cross section. For the purposes of our analyses, the subsurface conditions were modeled as levee fill overlying fine- and coarse-grained alluvial deposits. Based on our review of the USACE General Design Memorandum (1966), the

Operations and Maintenance Manual (USACE, 1969), and field exploration logs, we modeled the levee as a core of impervious fill material covered by a shell of pervious fill material. Additionally, we modeled the alluvium as an impervious top stratum overlying a pervious substratum.

Estimated model dimensions were based on our review of USACE (1966, 1969) and field exploration logs. In general, the landside and waterside slope inclinations of our models were estimated to be about 2.5h:1v and 3h:1v, respectively; the impervious core landside and waterside slope inclinations of our models were estimated to be 2h:1v and 1h:1v, respectively.

As a measure of conservatism, the 12- to 24-inch-thick layer of riprap slope protection was not included in our stability models. Based on our experience, including the riprap slope protection in our models would likely result in a relatively small increase in estimated factors of safety.

For existing and flood-stage conditions, creek water surfaces were modeled to represent the median annual and 100-year water surface elevations, respectively, as defined by Northern Hydrology & Engineering (NHE, 2010a). Groundwater elevations were estimated based on data collected during our field exploration efforts, between September 9 and October 15, 2010; groundwater data are presented in Table 5-2 and on Plate 5-9.

# 8.0 GEOTECHNICAL ANALYSES

## 8.1 LIQUEFACTION

Liquefaction triggering analyses were performed for all 37 CPTs located in the project area. Proprietary software programmed into ArcGIS was used to: 1) perform the CPT-based liquefaction triggering analyses according to the 1997 National Center for Earthquake Engineering Research (NCEER) guidelines (NCEER, 1997), and 2) to estimate seismic settlement using Ishihara and Yoshimine (1992). The results of the analyses are presented on individual CPT plots in Appendix D as Plates D-2 through D-38. A key to the liquefaction logs is presented on Plate D-1. In addition, individual logs are projected onto cross sections as Plates D-39 and D-40.

Liquefaction analyses were performed as follows:

1. In accordance with EC 1110-2-6067 (USACE, 2010a), groundwater depths corresponding to the median annual water surface elevation were assumed to approximate existing phreatic conditions.
2. Liquefaction triggering was evaluated for a PHGA of 0.2 g and a moment magnitude of 9.0 (greater of the deaggregated modal and mean magnitudes), corresponding to a statistical return period of 134 years.
3. 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 versus depth was calculated using 1997 NCEER guidelines (NCEER, 1997). CRR values versus depth for the CPT-based liquefaction triggering procedure were estimated using the 1997 NCEER guidelines (NCEER, 1997).
4. Seismic settlement resulting from potential liquefaction was estimated for each CPT using procedures presented in Ishihara and Yoshimine (1992).

### 8.1.1 Liquefaction Triggering Analyses per NCEER (1997)

NCEER (1997) guidelines (Youd and Idriss, 2001) present deterministic liquefaction triggering procedures based on CPT data. The CRR function to determine liquefaction potential is estimated by the following equation:

If $(q_{c1N})_{CS} < 50$, $CRR_{7.5} = 0.833[(q_{c1N})_{CS}/1{,}000] + 0.05$

If $50 \leq (q_{c1N})_{CS} < 160$, $CRR_{7.5} = 93[(q_{c1N})_{CS}/1{,}000]_3 + 0.08$

where:

$(q_{c1N})_{CS}$ = clean-sand cone penetration resistance normalized to approximately 100 kPa (1 atm)

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 CPT plots (Plates D-2 through D-40). The red lines on the plots are the estimated CPT tip resistance that is needed to resist liquefaction for the seismic conditions considered. A blue zone between the red line and the CPT tip resistance indicates a zone of potentially liquefiable soil.

As noted above, discontinuous, interbedded layers of silty sand and clayey sand were encountered within predominantly fine-grained subsurface stratigraphic units. Depending on the thickness of sandy layers relative to overlying and underlying fine-grained materials, the full cone tip resistance in sandy layers may not be realized (termed the "thin layer" effect). For liquefaction evaluations presented herein, a thin layer correction was applied to the stiff sandy layers within the softer interbedded fine-grained materials.

### 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 CPT tip resistance (see Fig. 10 in Ishihara and Yoshimine, 1992). 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. Cumulative seismic settlement versus depth estimates are presented on the CPT plots and tabulated in Table 8-1.

### 8.1.3 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 very loose to very dense sandy silt, silty sand, sand, sand with gravel, gravelly sand, gravel with sand, sandy gravel, and gravel. The very loose to medium dense granular materials may be susceptible to dry seismic settlement. On the basis of our analysis, we estimate the potential magnitude of seismically induced dry settlement to be up to about 1 inch.

### 8.1.4 Results

The results of our seismically induced settlement analyses for the ten cross sections are summarized in Table 8-1 below, and presented graphically in Appendix D as Plates D-2 through D-40.

### Table 8-1. Estimated Seismically-Induced Settlement Analyses Results

| Section | Levee | Cumulative Liquefaction-Induced Settlement (inches) | Cumulative Seismically-Induced Dry Settlement (inches) |
|---|---|---|---|
| 57+00 | Left | 2½ - 4½ | ≤1 |
| | Right | 2½ - 5 | ≤1 |
| 73+00 | Left | 1½ - 3½ | ≤1 |
| | Right | 2 - 5 | ≤1 |
| 87+00 | Left | ½ - 1½ | ≤1 |
| | Right | ½ - 1 | ≤1 |
| 98+00 | Left | ½ - 1 | ≤1 |
| | Right | 0 - 3½ | ≤1 |
| 114+00 | Left | ½ - 1 | ≤1 |
| | Right | 3 - 6½ | ≤1 |
| 132+00 | Left | 3½ - 4 | ≤1 |
| | Right | 0 - 1 | ≤1 |
| 146+00 | Left | 2 - 2½ | ≤1 |
| | Right | 2½ - 3 | ≤1 |
| 158+00 | Left | 4½ - 5 | ≤1 |
| | Right | 3½ - 4 | ≤1 |
| 171+00 | Left | 1 - 1½ | ≤1 |
| 185+00 | Left | 1½ - 2 | ≤1 |

## 8.2 PERMEABILITY AND SEEPAGE

Seepage analyses were performed for the ten cross sections along the project extent of Redwood Creek. The analyses were performed using the two-dimensional numerical modeling program SLIDE (Rocscience, 2010) to account for site-specific conditions, including irregular slope geometries and anisotropic permeabilities. This method of evaluation is generally consistent with the recommendations regarding seepage analysis computer programs in ETL 1110-2-569 (2005) and ETL 1110-2-555 (1997).

In general, SLIDE is a reputable program that has been favorably compared with similar modeling programs utilized within the geotechnical engineering industry (Pockoski and Duncan, 2000). The validity of our seepage and stability models was qualitatively verified by comparing the output estimated for existing conditions with conditions observed in the field. In addition, our models were reviewed internally by multiple team personnel for quality assurance purposes.

It should be noted that our idealized seepage and stability models represent the general site conditions approximated by our investigation and a review of historical data. Although localized, anomalous subsurface conditions are present within the levee system, the lateral extent of anomalous areas was not estimated as part of our evaluation. Therefore, the potential effects of anomalous conditions on seepage and stability were not included in our analyses.

As input to the seepage analyses, soil permeability values were selected based on a comparison of laboratory test results, well testing data collected by SHN (2006a, 2006b), values calculated using the empirical correlation presented in Chapuis (2004), a review of design documents, and limited sensitivity analyses. In addition, values were compared to typical permeability values presented in USACE EM-1110-2-1901 (1986), and Domenico and Schwartz (1998) before we selected the input value of saturated permeability for each material. Plates D-41 and D-42 show the estimated permeability values for the levee fill and foundation material, respectively.

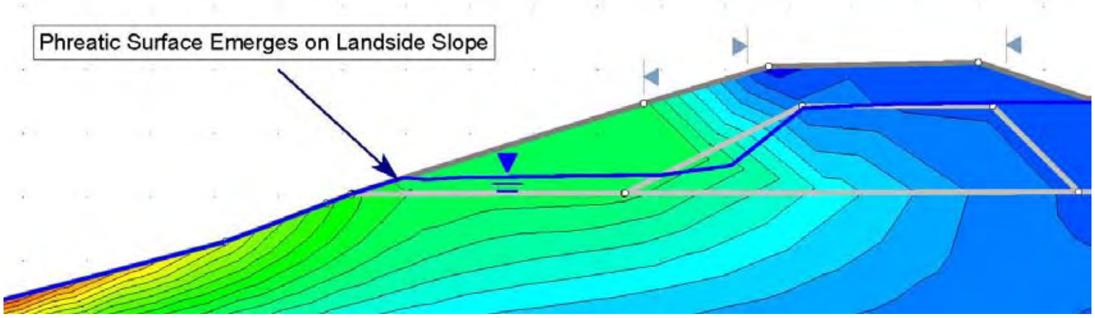
Both horizontal ($k_h$) and vertical permeability ($k_v$) values are required for modeling in SLIDE. Permeability values are assigned by inputting the saturated permeability in the horizontal direction and inputting the vertical to horizontal permeability ratio ($k_v/k_h$). Estimated $k_v/k_h$ ratios and additional material properties are presented with the results of our seepage and stability analyses in Appendix E.

In addition, curves representing the variation of permeability (unsaturated permeability) versus matric suction were estimated for our seepage analyses. Without direct measurements of unsaturated permeability, the curve estimates were based on data available for similar soils.

As input to our slope stability analyses and in accordance with EM 1110-2-1913 (USACE, 2000a), seepage analyses were performed for both existing conditions (in lieu of the 'End of Construction' case), and steady-state seepage during a full flood event. It should be noted that the effects of relief wells were not included in our seepage analyses.

As noted above, creek water surfaces for existing and flood-stage conditions were modeled to represent the median annual and 100-yr water surface elevations, respectively, as defined by Northern Hydrology & Engineering (NHE, 2010a). Groundwater elevations were estimated on the basis of data collected during our field exploration.

The potential for through-seepage was estimated at each section by reviewing the seepage analyses output to determine if the phreatic surface emerges on the landside slope during the design flood event. As noted in USACE (2000), 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. An example of output illustrating through-seepage is presented below.



### 8.2.1 USACE Acceptable Seepage Criteria

Acceptance criteria have been established by the USACE to reduce the potential impacts of levee underseepage (USACE, 2005; USACE, 2000a). 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).

### 8.2.2 Steady-State Seepage

For the full flood steady-state seepage analyses, the landside extent of our models was defined by the lesser of the perpendicular horizontal distance to the valley basin limits (bedrock slope) and a distance of 2,000 feet from the creek centerline.

To simulate the general hydrogeologic conditions we anticipate during a flood event, a no-flow condition was assigned as the landside boundary condition if the landside model extent was defined by the valley basin limits (i.e. relatively impermeable bedrock). Alternatively, the phreatic surface was modeled at the ground surface if the landside model extent was defined by a distance of 2,000 feet from the creek centerline.

### 8.2.3 Transient Seepage

Based on the results of our steady-state analyses, we identified cross sections likely to develop through-seepage, or underseepage at the landside toe with exit gradients in excess of 0.5 and corresponding factors of safety that do not comply with USACE performance criteria. Steady-state evaluations assume that base flood elevations are maintained for a sufficient time to allow steady-state seepage flow conditions to develop through and beneath the levee system. Our evaluations of exit gradients under steady-state conditions were estimated to range up to 2.7, which significantly exceeds the allowable exit gradient of 0.5, per ETL 110-2-569 (USACE, 2005).

It is our opinion that steady-state conditions are generally not representative of the anticipated base flood conditions at Redwood Creek. Redwood Creek is about 67 miles long and drains a watershed encompassing about 285 square miles. It is, thus, a relatively small drainage basin that, in our discussions with NHE (the County's project hydrologist), is relatively responsive to storm events. Accordingly, it is our understanding that the water elevations within Redwood Creek rise and fall relatively rapidly during and following storm events and do not have sustained peak crests as observed along rivers such as the Sacramento, Mississippi, and Missouri rivers. With responsive drainages such as Redwood Creek, water elevations typically are not elevated for durations sufficient to establish steady-state seepage flow through the embankment and foundation materials. Thus, steady-state evaluations of seepage flow paths and exit gradients may represent improbable, excessively conservative geotechnical conditions.

In an effort to model the development of seepage forces and phreatic surfaces during a full flood event that is representative of the Redwood Creek drainage, we conducted transient seepage and stability analyses of the select cross sections likely to develop through-seepage or sections with estimated exit gradients in excess of 0.5 under steady-state conditions. Because steady-state conditions represent a worst-case scenario, those cross sections unlikely to develop through-seepage *and* with estimated exit gradients below 0.5 were not evaluated in our transient analyses, having already satisfied performance criteria. The purpose of our transient analyses was to estimate if the conditions evaluated in our steady-state analyses are likely to develop during the projected duration of a 100-yr flood event.

Input parameters applied to the transient seepage models were identical to those applied to the steady-state models, with the exception of the initial boundary conditions. For the transient seepage analyses, initial boundary conditions included water surface elevations corresponding to an assumed base flow of 5,000 cubic feet per second (cfs), and groundwater levels estimated on the basis of data collected during our field exploration efforts. In addition, volumetric water content values were estimated on the basis of laboratory results and assigned to subsurface materials to model the initial pore pressure conditions. Volumetric water content estimates are presented in Appendix D.

To model the changes in creek water surface elevations during the 100-year storm event, we applied the hydrograph data defined by NHE (NHE, 2011b). Along the project extent, the estimated duration of flow above the base flow (5,000 cfs) during the 100-year flood event is approximately 8 days. Following limited sensitivity analyses, the transient seepage analyses were conducted with the following time stages:

**Table 8-2. Time Stages of Transient Analyses**

| Transient Analysis Stage (hours) | Redwood Creek Water Surface Elevation Trend |
|---|---|
| 0 | Rising |
| 12 | Rising |
| 24 | Rising |
| 36 | Rising |
| 45.5 | Maximum |
| 48 | Falling |
| 60 | Falling |
| 72 | Falling |
| 84 | Falling |
| 96 | Falling |
| 108 | Falling |
| 120 | Falling |
| 132 | Falling |
| 144 | Falling |
| 156 | Falling |
| 168 | Falling |
| 180 | Falling |
| 192 | Falling |

### 8.2.4 Results

The results of our seepage analyses for the 10 cross sections are summarized below, and presented graphically in Appendix E. The results are discussed in section 9.4.

### Table 8-3. Summary of Results for Seepage Analyses

| Section | Potential for Through-Seepage | | Steady-State | | Transient | |
|---|---|---|---|---|---|---|
| | Steady-State | Transient | Exit Gradient at Landside Toe | Factor of Safety | Exit Gradient at Landside Toe | Factor of Safety |
| 57+00 Left | Yes* | No | 0.3** | 2.54 | < 0.1*** | > 7.62 |
| 57+00 Right | No | -- | 0.2 | 3.81 | -- | -- |
| 73+00 Left | No | No | 0.6 | 1.27 | 0.2 | 3.81 |
| 73+00 Right | Yes* | No | 0.3** | 2.54 | < 0.1*** | > 7.62 |
| 87+00 Left | No | -- | 0.1 | 7.62 | -- | -- |
| 87+00 Right | Yes* | No | 1.2 | 0.63 | 0.6 | 1.27 |
| 98+00 Left | No | No | 0.3** | 2.54 | 0.2 | 3.81 |
| 98+00 Right | No | No | 0.8 | 0.95 | < 0.1*** | > 7.62 |
| 114+00 Left | Yes* | No | 0.4** | 1.90 | < 0.1*** | > 7.62 |
| 114+00 Right | Yes* | No | 2.7 | 0.28 | 0.7 | 1.08 |
| 132+00 Left | No | -- | 0.5 | 1.52 | -- | -- |
| 132+00 Right | Yes* | No | 1.2 | 0.63 | < 0.1*** | > 7.62 |
| 146+00 Left | Yes* | No | 0.3** | 2.54 | 0.3 | 2.54 |
| 146+00 Right | Yes* | No | 0.5 | 1.52 | 0.4 | 1.90 |
| 158+00 Left | No | No | 0.7 | 1.08 | 0.4 | 1.90 |
| 158+00 Right | Yes* | No | 0.4** | 1.90 | 0.3 | 2.54 |
| 171+00 Left | Yes* | No | 0.3** | 2.54 | 0.2 | 3.81 |
| 185+00 Left | Yes* | No | 0.4** | 1.90 | 0.2 | 3.81 |

* If column response is "yes", this indicates that the phreatic surface is estimated to encounter the landside levee slope face above the adjacent natural ground surface under the conditions evaluated. If the response is "No", then it is estimated that the phreatic surface will not daylight on the landside levee face.

** Transient analyses were performed on sections with estimated exit gradient gradients less than 0.5 if the section was likely to develop through-seepage or the estimated FS for Case III (refer to Section 8.3.1) was less than 1.4.

*** Phreatic surface does not infiltrate impervious foundation stratum in transient analysis; therefore, underseepage forces at the landside toe are anticipated to be minimal.

## 8.3 SLOPE STABILITY

Slope stability analyses were performed for the ten cross sections along Redwood Creek. The analyses were performed using the numerical modeling program SLIDE

(Rocscience, 2010). Stability analyses were computed by SLIDE using the Spencer method (Spencer, 1967) and the GLE/Morgenstern and Price method (Fredlund and Krahn, 1977; Morgenstern and Price, 1965). These methods are briefly described below.

**Spencer Method.** The Spencer Method assumes that the normal forces are located at the center of the base of each slice and that all side forces are parallel. The result is an equation that satisfies complete moment and force equilibrium. Although the Spencer Method was directly applicable to a circular shear surface, the procedure may be readily extended to slip surfaces of a general shape.

Because of the complexity of the procedure, the Spencer Method is suitable only for computer-aided slope stability analyses. Although the Spencer Method typically yields a relatively accurate estimate of the factor of safety for a slope, its solution requires several iterations.

**GLE/Morgenstern and Price's Method.** GLE/Morgenstern and Price's Method as originally formulated took a somewhat different approach to the solution of complete slice equilibrium. While Spencer considered overall moment equilibrium, Morgenstern and Price have considered only the moment equations of individual slices. Each method satisfies all conditions of equilibrium but Spencer's Method requires about half the computation time. When static equilibrium is satisfied, there is little practical difference among Spencer's and Morgenstern and Price's procedures.

### 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 > 1 implies a stable slope, a FS < 1 implies a slope that is failing, and a FS = 1 implies a slope that 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.

**Under Case I**, soils are modeled to simulate engineered 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 in an undrained condition, whereas, granular soils (pervious soils) are modeled in a drained condition. 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 where a prolonged flood stage saturates the major portion of the waterside face of the levee 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 landslide slope stability is evaluated.

**Case IV** includes the effects of earthquake forces on the stability of a levee slope.

The USACE (2000) has established minimum FS values for each levee slope stability case evaluated. Those allowable minimum FS values, presented in Table 8-4 below, were referenced in our interpretation of slope stability analyses discussed herein.

**Table 8-4 - Minimum Acceptable Factor of Safety Values**

| Case | Minimum Factor of Safety |
|---|---|
| Case I - End of Construction | 1.30 |
| Case II - Rapid Drawdown | 1.00 to 1.20[1] |
| Case III - Steady State Seepage | 1.40 |
| Case IV - Earthquake Loading | 1.01 |

[1] 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

As input to the stability analyses, parameters including soil unit weight, and drained and undrained shear strength parameters were estimated based on laboratory test results, CPT correlations, and review of design documents. As noted above, consolidated drained direct shear and consolidated, undrained triaxial strength testing was performed on relatively undisturbed samples collected during our field exploration program. The results of our direct shear and triaxial testing are presented in Appendix C. Correlations relating CPT tip resistance to friction angle were also reviewed in our estimation of drained shear strength parameters. Friction angle estimates based on CPT data are presented on the CPT logs in Appendix B.

Geotechnical design values for levee fill and foundation materials were also included in the *General Design Memorandum* (USACE, 1966) and *Survey Report for Flood Control and Allied Purposes [Appendices]* (USACE, 1961). Listed values include dry unit weights, moist weights, saturated weights, buoyed weights, shear strength parameters and permeability. These values are presented in Appendix A.

For our stability analyses of existing conditions, pseudo-static conditions, and landside slope conditions during a full flood event, drained shear strength parameters were assigned to the subsurface materials. The results of direct shear and triaxial testing conducted by the USACE (1966, 1965a, and 1965b) are presented with our drained strength estimates presented on Plates D-43 through D-46.

For our stability analyses of rapid drawdown, we conducted three-stage computations for the impervious levee fill and foundation material. Input to the three-stage computations included both drained and undrained shear strength parameters, which were estimated according to the methods outlined in Appendix G of EM 1110-2-1902 (USACE, 2003) for rapid drawdown analyses. The results of triaxial testing conducted by Fugro and USACE (1965a) are presented with our estimate of the consolidated undrained shear strength [CU(R)] envelope presented on Plate D-47.

Phreatic surfaces and pore pressures modeled in our stability analyses of full flood and rapid drawdown conditions were based on the results of our steady-state seepage analyses. Based on the results of our stability analyses of steady-state full flood conditions, we identified cross sections with estimated factors of safety that do not meet or exceed the minimum FS of 1.4 defined for long-term (steady-state seepage) flood conditions in USACE (2000a).

In an effort to model the development of phreatic surfaces and pore pressures during a full flood event, we conducted transient seepage and stability analyses of the select cross sections discussed in the preceding paragraph. The purpose of our transient analyses was to estimate if the conditions evaluated in our steady-state analyses are likely to develop during the projected duration of a 100-yr flood event.

For post-earthquake 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 in accordance with the empirical correlations proposed by Seed and Harder (1990), and Idriss and Boulanger (2007); undrained strength estimates are presented on Plates D-2 through D-38, and on Plates D-39 and D-40. Our evaluation of potentially liquefiable strata is discussed in Section 8.1.

If the post-earthquake FS was estimated to be between 1.2 and 1.0, we performed a seismic deformation analysis to estimate the levee's performance during the design seismic event. Slopes with estimated factors of safety less than 1.0 in our post-earthquake stability analyses are considered unstable under static conditions and generally susceptible to flow failure following the design seismic event. Large displacements are typically associated with flow failure of slopes, and a seismic deformation analysis method is not suitable for estimation of static flow failure deformation.

### 8.3.3 Results

The results of our slope stability analyses for the ten cross sections are summarized below, and presented graphically in Appendix E.

# Table 8-5. Summary of Slope Stability Analyses Results

| Section | Levee Slope | Factor of Safety For Each Case Evaluated | | | | | |
|---|---|---|---|---|---|---|---|
| | | Existing Conditions (USACE min. 1.3) | Pseudostatic (USACE min. undefined) | Seepage During a Full Flood Event (USACE min. 1.4) | | Rapid Drawdown (USACE min. 1.0) | Post-Earthquake (USACE min. undefined) |
| | | | | Steady-State | Transient | | |
| 57+00 Left | Landside | 1.76 | 1.04 | 1.15 | 1.81 | -- | 1.75 |
| | Waterside | 2.19 | 1.20 | ------------- | | 2.19 | 1.91 |
| 57+00 Right | Landside | 3.56 | 1.64 | 2.25 | -- | -- | 3.56 |
| | Waterside | 2.10 | 1.06 | ------------- | | 1.10 | 0.54 |
| 73+00 Left | Landside | 1.83 | 1.11 | 1.55 | 1.83 | -- | 1.83 |
| | Waterside | 2.10 | 1.08 | ------------- | | 1.07 | 1.85 |
| 73+00 Right | Landside | 2.30 | 1.30 | 1.37 | 2.30 | -- | 2.27 |
| | Waterside | 1.90 | 1.05 | ------------- | | 0.86 | 1.1 |
| 87+00 Left | Landside | 2.18 | 1.26 | 1.64 | -- | -- | ** |
| | Waterside | 1.91 | 1.09 | ------------- | | 0.99 | ** |
| 87+00 Right | Landside | 2.69 | 1.44 | 1.75 | 2.69 | -- | ** |
| | Waterside | 2.15 | 1.25 | ------------- | | 1.28 | ** |
| 98+00 Left | Landside | 1.38 | 0.97 | 1.07 | 1.38 | -- | 1.38 |
| | Waterside | 1.80 | 1.12 | ------------- | | 0.90 | 1.65 |
| 98+00 Right | Landside | 2.72 | 1.46 | 2.59 | 2.68 | -- | 2.72 |
| | Waterside | 1.79 | 1.06 | ------------- | | 0.86 | 1.32 |
| 114+00 Left | Landside | 2.05 | 1.22 | 1.29 | 2.05 | -- | 2.05 |
| | Waterside | 2.00 | 1.19 | ------------- | | 1.18 | 2.00 |
| 114+00 Right | Landside | 1.86 | 1.08 | 0.40 | 1.75 | -- | 0.69 |
| | Waterside | 2.21 | 1.10 | ------------- | | 1.10 | 0.47 |

### Table 8-5. Summary of Slope Stability Analyses Results (Continued)

| Section | Levee Slope | Factor of Safety For Each Case Evaluated | | | | | |
|---|---|---|---|---|---|---|---|
| | | Existing Conditions (USACE min. 1.3) | Pseudostatic (USACE min. undefined) | Seepage During a Full Flood Event (USACE min. 1.4) | | Rapid Drawdown (USACE min. 1.0) | Post-Earthquake (USACE min. undefined) |
| | | | | Steady-State | Transient | | |
| 132+00 Left | Landside | 2.22 | 1.28 | 1.42 | -- | -- | 2.15 |
| | Waterside | 2.09 | 1.14 | ------------ | | 1.22 | 1.09 |
| 132+00 Right | Landside | 1.54 | 1.07 | 0.88 | 1.73 | -- | ** |
| | Waterside | 2.47 | 1.31 | ------------ | | 1.34 | ** |
| 146+00 Left | Landside | 1.96 | 1.18 | 1.43 | 1.96 | -- | 1.87 |
| | Waterside | 2.64 | 1.31 | ------------ | | 1.23 | 1.22 |
| 146+00 Right | Landside | 1.52 | 1.00 | 1.12 | 1.57 | -- | 1.42 |
| | Waterside | 1.85 | 1.06 | ------------ | | 0.91 | 0.91 |
| 158+00 Left | Landside | 2.50 | 1.38 | 1.80 | 2.46 | -- | 2.28 |
| | Waterside | 1.98 | 1.07 | ------------ | | 1.00 | 0.93 |
| 158+00 Right | Landside | 1.35 | 0.88 | 1.14 | 1.46 | -- | 1.36 |
| | Waterside | 2.02 | 1.15 | ------------ | | 1.14 | 1.76 |
| 171+00 Left | Landside | 1.98 | 1.17 | 1.66 | 2.20 | -- | 1.97 |
| | Waterside | 2.01 | 1.15 | ------------ | | 1.18 | 1.47 |
| 185+00 Left | Landside | 1.87 | 1.22 | 1.48 | 2.07 | -- | 2.00 |
| | Waterside | 2.10 | 1.08 | ------------ | | 1.01 | 1.42 |

** Soils are not considered to be potentially liquefiable during the design seismic event, or potentially liquefiable strata are localized and unlikely to affect the strength of materials along estimated failure planes.

## 8.4 SEISMIC DEFORMATION

In accordance with EC 1110-2-6067 (USACE, 2010a), we performed deformation analyses for levee cross sections with factors of safety between 1.2 and 1.0 estimated in our post-earthquake stability analyses, which are discussed in sections 8.3 and 9.5 of this report. Slopes with factors of safety less than 1.0 are considered unstable under static conditions and generally susceptible to flow failure following the design seismic event. Therefore, a seismic deformation analysis method is not suitable for estimation of flow failure deformation.

The analyses were performed according to procedures presented in Bray and Travasarou (2007) to evaluate seismic displacement potential, and estimate displacements corresponding to specified probabilities. The seismic displacement model requires the following input parameters (see Equations 5 and 6 in Bray and Travasarou, 2007):

- $T_s$ = initial fundamental period of the sliding mass in seconds
- $S_a(1.5T_s)$ = spectral acceleration of the input ground motion at a period of 1.5Ts

- MW = moment magnitude of design earthquake event
- $k_y$ = yield coefficient (defined as the seismic coefficient corresponding to an estimated FS equal to 1.0 in a pseudostatic stability analysis.)

In order to estimate a yield coefficient and initial fundamental period of each sliding mass for input to our seismic deformation analyses, pseudo-static stability analyses were performed with post-earthquake undrained residual strengths assigned to potentially liquefiable strata.

Following the procedure proposed by Marcuson et al. (1983), a seismic coefficient (k) equal to half of the PHGA (approximately 0.1g) was initially applied in pseudo-static analyses. If the corresponding FS was estimated to be less than 1.0, the seismic coefficient was iteratively decreased until a yield coefficient was determined.

As noted above, we applied spectral accelerations estimated to have approximately 0.7 percent annual chance of exceedance (Statistical Return Period ≈ 134 years). Additionally, a moment magnitude of 9.0 (greatest of the deaggregated modal and mean magnitudes for the spectral periods evaluated) was applied in all of the seismic deformation analyses.

### 8.4.1 Results

In the absence of a displacement threshold defined by the USACE, the results of our analyses are presented in terms of the median displacement estimate (i.e. the displacement with a 50 percent probability of exceedance). The results of the analyses are tabulated below and the calculations are presented in Appendix D as Plates D-49.

**Table 8-6. Summary of Results for Seismic Deformation Analyses**

| Representative Cross Section | Slope Location | Yield Coefficient | Approximate Estimated Median Displacement (i.e. displacement with a 50% probability of exceedance) | |
|---|---|---|---|---|
| | | | Centimeters | Feet |
| 73+00 Right | Waterside | 0.022 | 91.1 | 3.0 |
| 132+00 Left | Waterside | 0.025 | 114.3 | 3.8 |
| 146+00 Left | Waterside | 0.035 | 81.2 | 2.7 |

## 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 levees (on the order of 42 years for most of the project site), 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 GENERAL

The purpose of our investigation was to explore subsurface conditions at the project site to aid the County in evaluating the ability of the Redwood Creek levee system within the project extent to function in accordance with current FEMA/USACE performance criteria. To achieve this objective, CGI/Fugro performed subsurface exploration, laboratory testing, and engineering analyses in accordance with the standards of FEMA (2008), and USACE (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 addressed the geotechnical aspects of flood control protection for the 100-year recurrence interval (or 1% annual chance exceedance) flood event within the project extent at Redwood Creek. Our evaluation did not address the hydrologic, hydraulic, or any other aspects of flood protection for Redwood Creek.

**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 6-1/2 inches and 1 inch, respectively. For post-earthquake stability analyses, undrained residual strength parameters were estimated for potentially liquefiable strata in accordance with published empirical correlations.

**Permeability and Seepage.** For our initial evaluation, the potential for underseepage and through-seepage was evaluated for steady-state flood stage conditions. Following identification of sections considered to be deficient under steady-state conditions, we conducted transient seepage analyses to determine if the seepage forces and phreatic surfaces estimated by our steady-state analysis are likely to develop during the 100-year flood event. Based on our analyses, only two sections do not meet performance criteria: the estimated vertical exit gradients at the landside toe of the right levee at about Station No. 87+00 and the right levee at about Station No. 114+00 are 0.6 and 0.7, respectively, which exceed the USACE minimum allowable exit gradient of 0.5. However, as discussed below, the synthetic hydrograph applied to our analyses may be considered conservative.

**Slope Stability.** Our slope stability analyses indicated that the slopes evaluated are generally stable for each of the conditions discussed in section 8.3.1, with the exception of the post-earthquake condition. Based on our post-earthquake stability analyses, five of the slopes evaluated were estimated to have a FS less than 1.0. In addition, three of the slopes evaluated were estimated to have a FS between 1.2 and 1.0. The anticipated instability of these slopes is primarily attributable to the undrained residual strengths applied to our models.

**Seismic Deformation.** Displacements ranging between 2.7 and 3.8 feet were estimated for the three waterside levee slopes identified by our post-earthquake stability analysis to have an estimated FS between 1.2 and 1.0. Slopes with estimated factors of safety less than 1.0 in our post-earthquake stability analyses are considered unstable under static conditions and generally susceptible to flow failure following the design seismic event. Large displacements are typically associated with flow failure of slopes.

**Settlement.** Based on our qualitative evaluation, the potential for static settlement to impact the levees is considered to be minimal under most present conditions. However, the potential for settlement due to gravel stockpiles located on the levee crest is discussed below.

### 9.1.1 Summary of Findings and Recommendations

On the basis of field observations, subsurface exploration, laboratory testing and engineering analyses performed for this investigation, it is our opinion that the Redwood Creek levee system, within the limits of the project extent as defined in the scope of work, is generally capable of providing flood control protection to adjacent improvements during static, stable conditions.

However, based on our evaluation, the ability of the levee to provide flood protection will likely be compromised during and/or following a seismic event equal to or greater than the design seismic event discussed in Section 6.0 of this report. Specifically, the loss of soil strength due to liquefaction of very loose to medium dense granular soils encountered at the site is likely to compromise the stability of up to eight of the levee slopes we evaluated.

Relative to performance standards, FEMA (2008) suggests that an embankment stability analysis should demonstrate that a levee system is stable under Case IV (earthquake) loading conditions, as defined by USACE (2000a). As previously noted, portions of the levee alignments have been evaluated to be susceptible to liquefaction and, therefore, are likely not stable under Case IV conditions. While this is a deficiency in the stability of the levee system, it should be noted that the likelihood of a large earthquake occurring at the same time as a large flow event in Redwood Creek is, in the opinion of CGI/Fugro and USACE (2000a), relatively low. A more likely scenario is that a large earthquake event will occur at a time when the levee system is not retaining peak flows and that there will be sufficient time for emergency repairs to be made to the levee prior to a peak flow occurring.

While a number of mitigation measures to reduce the potential for liquefaction are discussed in Section 9.3.1 of this report, each of those ground modification measures is expensive to implement. It is our opinion that an evaluation of the probability of both a large earthquake and peak flood event occurring at the same time and a cost-benefit analysis of mitigating liquefaction potential will indicate that implementing ground modification measures will not be justifiable. Such an analysis will likely indicate that establishment of an emergency response plan (ERP) be performed that can address assessment and response procedures for the levee following seismic events. The ERP should identify the potential for liquefaction and seismic hazards to impact the levee, and designate high hazard areas that should be inspected for damage following an earthquake. ERPs typically include elements addressing potential damage scenarios, storage locations for supplies and equipment associated with repair, and evacuation plans for any communities potentially affected by the hazard.

### 9.1.2 Opinion Regarding Geotechnical Aspects of Levee Certification

Based on the results of our study, the levee system was evaluated to be stable under static conditions for all conditions evaluated at the ten cross section locations, with the exception of two locations with marginally high vertical exit gradients. Additional transient analyses at those locations using data-generated hydrographs will likely reduce those exit gradients from the values reported herein. The only additional deficiencies identified during this study occur under dynamic (earthquake) conditions. Those deficiencies should be addressed using ERPs outlining procedures to be implemented following a seismic event.

Based on the results of our evaluations, it is our opinion that geotechnical stability aspects of the levee system are likely certifiable under FEMA and USACE standards provided the ERPs are prepared and additional transient analyses are performed with results conforming to USACE thresholds.

## 9.2 OPERATION AND MAINTENANCE

During our site inspection and mapping efforts, we identified a number of potentially detrimental geotechnical conditions that are typically addressed by an operation and maintenance program. These site conditions are briefly discussed below.

### 9.2.1 Encroachment of Levee Landside Toe

At the time of our field studies, the northern (right) levee landside toe between approximately STA 30+00 to STA 40+00 had been excavated to accommodate an adjacent unpaved road. In our opinion, that nearly vertical toe, measuring approximately 1 to 2 feet in height, could have destabilized the slope. We recommend the slope be restored to the original 2.5h:1v inclination specified in USACE (1966). We understand that during January and May, 2011, the County (Humboldt County, 2011) restored the landside slope toe to the original 2.5h:1v inclination specified in USACE (1966) and corrected the slope modification, thus, mitigating the potentially destabilizing condition.

### 9.2.2 Vegetation on Levee Embankment and within 15 Feet of Levee Toes

According to USACE (2000b), landscape within 15 of the levee should be free of vegetation. Based on our observations, there is generally dense vegetation on segments of the levee slopes and within 15 feet of both the landside and waterside levee toes along the entire project extent.

Vegetation on the waterside slopes of the levee generally consists of seasonal grasses, berry vines, and other low shrubs. Trees and shrubs were observed in narrow bands at the lower approximate 5 feet of the waterside levee slope face. We understand that the presence of these trees is the result of the County's inability to remove them due to restrictions placed on such activities by outside regulatory agencies.

Similarly, seasonal grasses, berry vines, and low shrubs were observed along the landside levee face; however, relatively large trees have been established along the left landside levee face between Station Nos. 115+00 and 125+00, and on the right landside levee face between Station Nos. 122+00 and 127+00, and at about Station No. 150+00. Relatively dense vegetation and trees are also present within 15 feet of the landside levee toe along much of the levee extent.

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). The County has made considerable effort to control vegetation along the waterside levee slopes and toes but it is constrained on vegetative removal due to regulatory restrictions and limitations, especially along the waterside portions of the levee system. 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, it also appears to have helped anchor foundation soils along the waterside levee toes and helps stabilize channel banks, thus reducing erosion within these areas.

While the vegetation treatments should attempt to address recommendations presented in the most recent inspection reports (USACE, 2009a and 2009b), it is a much more complex issue that we understand is currently under consideration and development at the national and state level (personal communication with Humboldt County). Until updated guidelines are developed, we recommend that the County continue efforts to reduce vegetation along the waterside and landside levee faces and toes. We also recommend that trees on the landside levee face be removed because they can form seepage conduits through the embankment materials and potentially compromise the levee at those locations. Root balls from those trees should also be removed and the resulting void should be backfilled with impermeable engineered fill materials placed in a manner that meets USACE standards.

### 9.2.3 Erosion and Erosional Impact of Culverts

In general, we did not observe signs of significant erosion occurring along the levee system, with the exception of erosion occurring at specific culvert discharge locations. We observed culverts daylighting at about mid-slope or higher near Station Nos. 142+00, 160+00, 166+00, and 170+00. Erosion observed at waterside culvert discharge locations appears to have been the result of water discharge from the culverts and not scour from elevated flows within Redwood Creek. In our opinion, heavy discharge from the culverts may potentially erode and impact the stability of the slope. Though we did not observe evidence of active erosion at the time of our site inspection and mapping efforts, we recommend an assessment of the slope protection adequacy below the culverts. Additionally, slopes below the culverts should be inspected for any evidence of erosion following major discharge episodes.

### 9.2.4 Rutting

We noted scattered depressions/ruts measuring approximately 1 foot or less in depth on the levee crest along the entire project. In addition, scattered depressions/ruts measuring approximately 1 foot or less in depth were observed on the unpaved road adjacent to the landside toe approximately between Station Nos. 25+00 and 45+00. In our opinion, water may pond in the depressions/ruts and adversely impact the levee stability. We recommend filling the voids with well-compacted impervious material to facilitate proper drainage.

### 9.2.5 Rip-rap Slope Protection

Historical occurrences of slumping and subsequent repair of the left levee waterside slope near approximately Station No. 19+00 and Station No. 25+00 are noted in the most recent Inspection report (USACE, 2009b) and the 2009 Levee Operation & Maintenance Annual Report (County of Humboldt), respectively. Moreover, removal of rip-rap stones is noted in the 2008 Inspection report (USACE, 2008a), and fracture/fragmentation of rip-rap stones at the downstream end of the project is noted in the 2006 Inspection report (USACE). Though we did not observe evidence of slope protection deficiencies at the time of our site inspection and mapping efforts, we recommend regular inspection of the slope protection, especially following high-flow events.

### 9.2.6 Gravel stockpiles

We understand the gravel stockpiles observed between about Station Nos. 86+00 and 89+00 and approximately between about Station Nos. 138+00 and 144+00 have been placed by the County of Humboldt as part of the County's gravel extraction efforts. We understand the County requires storage sites in close proximity to Redwood Creek in order to effectively remove aggregated sediment and improve the hydraulic capacity of the creek channel (Humboldt County, 2011).

In our opinion, the additional surcharge of the stockpiles will likely cause some settlement of the levee and foundation material, and increase the potential for instability of the levee slopes. Although the impact of increased settlement and instability may be minimal relative to the impact of improved channel capacity, we recommend the County consider removing stockpiles whenever possible in accordance with the recommendations presented in the most recent inspection report (USACE, 2009b).

## 9.3 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 settlement is approximately 6½ inches and 1 inch, respectively.

In our opinion, the need for any remedial action should be based on an assessment of risk posed to adjacent improvements. 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. If levee slopes settle during or after the design seismic event, water levels in the creek will likely be lower than the adjacent land, and the consequences of settlement may be relatively minimal. At a minimum, it is recommended that the levees be inspected for signs of settlement, cracking, or failure as soon as possible following an earthquake and repaired as soon as conditions will allow. Typical mitigation measures for liquefaction hazard and seismically induced dry settlement are detailed below.

### 9.3.1 Mitigation of Liquefaction and Seismically Induced Dry Settlement

Ground improvement consisting of deep compaction can be employed to mitigate the potential for liquefaction and dry seismic settlement of the pervious foundation material encountered at the site. It is our opinion that deep compaction by vibroflotation, vibro-replacement, compaction grouting, permeation grouting, or jet grouting are the most suitable methods for the site conditions. Advantages and disadvantages of each method relative to performance, entitlement, construction, maintenance, and estimated implementation costs are noted in Table 9-1 below.

Although existing relief wells may provide drainage and, thus, some measure of liquefaction mitigation, based on our observations and review of historical documents, the ability of relief wells to facilitate dissipation of earthquake-induced pore pressures is undocumented and very likely limited by obstructive debris and sediment.

## Table 9-1. Mitigation Methods, Advantages and Disadvantages

| Mitigation Method | Advantages | Disadvantages |
|---|---|---|
| Vibroflotation | Least expensive relative to other mitigation methods presented herein | Effectiveness may be limited by the clay content of potentially liquefiable materials within the project limits |
| Vibro-replacement | May provide additional mitigation by facilitating dissipation of both earthquake-induced pore pressures and seepage forces during high-flow events | Generally more expensive, but more effective than vibroflotation in fine-grained soils. Typically requires more space for equipment access than vibroflotation |
| Compaction grouting | Generally suitable for all soil types; for treatment of soils at sufficient depth, grouting may be performed at greater grid intervals than vibroflotation and vibro-replacement. | Generally more expensive and requires more space for equipment access than vibroflotation and vibro-replacement; chemical grout may not be compliant with environmental regulations |
| Permeation grouting | Generally suitable for all soil types; minimal space required for equipment access relative to other mitigation methods presented herein | Expensive relative to other mitigation methods presented herein; determination of treatment extent and effectiveness is typical challenge; chemical grout may not be compliant with environmental regulations |
| Deep Soil Mixing | Generally suitable for all soil types; strength of soil-cement mixture typically greater than strength of native, untreated soil | Assessing the appropriate treated soil to untreated soil ratio is a challenge, and the cost of treatment is high relative to the other options described herein |

Vibroflotation consists of advancing a crane-suspended vibratory probe to the depth of the potentially liquefiable material by a combination of vibration and water or air jetting. Surrounding soil is densified in successive layers as the vibrating probe is extracted incrementally. This method is generally most effective in clean granular sands with less than about 5-percent clay fines, and has been used successfully up to depths of about 200 feet (Seed et al. 2003). Vibroflotation is typically performed in a grid pattern until the targeted area of the site has been suitably densified. Subsidence of the ground surface above the probe may be a consequence of vibroflotation, and would require backfill and compaction. Pre-drilling with an auger may be required to more easily penetrate dense or cohesive material that may be encountered.

In general, vibroflotation is the least expensive mitigation method described herein, and based on the results of our gradation testing, the fines content of the pervious foundation material is generally 5-percent or less. However, it should be noted that the gradation of alluvial deposits is typically variable, and the effectiveness of vibroflotation may be limited if soils with higher fines content are encountered.

Vibro-replacement (stone columns) is similar in concept to vibroflotation, except that gravel or crushed stone is added to the ground to assist with the densification. Deep compaction with stone columns typically consists of advancing a vibratory probe similar to vibroflotation, except that the probe is hollow to allow aggregate to be placed at the base of the probe. As the probe is extracted, the resulting void is backfilled with compacted gravel to densify the surrounding granular soil layers. Some surface subsidence may occur, particularly in untreated areas along the perimeter of the deep compaction, however, subsidence in the treated area is somewhat offset by the addition of gravel or crushed stone to the subsurface.

In general, vibro-replacement with stone columns is slightly more expensive and requires more space for equipment access than vibroflotation. However, the presence of relatively permeable stone columns may provide additional mitigation by facilitating dissipation of both earthquake-induced pore pressures and seepage forces during high-flow events.

Compaction grouting typically consists of driving a steel casing to the depth of potentially liquefiable material and then injecting a low slump grout into the casing under pressure to form a bulb within the ground around the casing. The bulb is expanded to densify the ground around the casing; the casing is then extracted in stages and the grouting process is repeated. Compaction grouting is typically performed in a grid pattern until the targeted area of the site has been suitably densified. For treatment of soils at sufficient depth, higher grout pressures may be used, and grouting may be performed at greater grid intervals.

Compaction grouting is generally more expensive and requires more space for equipment access than vibroflotation and vibro-replacement. However, compaction grouting is generally suitable for all soil types.

Permeation grouting involves injection of a low-viscosity grouting agent into voids within the potentially liquefiable material. Grouted soil is improved by strengthened bonds between individual soil grains, and minimal void space. Typical grouting agents include particulate grouts and chemical grouts, which are generally lower viscosity and capable of penetrating fine-grained soils. The permeation potential of particulate grouts is generally limited in soils with greater than about 6- to 10-percent fines (Seed et al. 2003). Typically, grout injection pipes are installed in boreholes spaced in a grid pattern covering the targeted area of the site. Drawbacks of this method include the relatively high cost of treatment, and the uncertainty in determining the volume of treated soil. Additionally, testing the effectiveness of the treatment usually requires relatively expensive drill holes because the grout impedes penetration of relatively inexpensive CPTs.

Deep Soil Mixing is a technique whereby cementitious material is mixed with potentially liquefiable soils by a hollow stem auger and rotating paddles. This method is generally applicable to any type of inorganic soil, and has been used up to depths of about 200 feet. The cementitious material is pumped through the auger stem to the open auger tip and mixed with the surrounding soil as the borehole is advanced to the target depth. The resulting soil-cement columns are generally uniform in diameter, and typically spaced to maximize the ratio of treated to untreated soil volume.

In addition, the strength of the resulting soil-cement mixture is typically greater than the strength of the native, untreated soil. However, assessing the appropriate treatment ratio is a challenge, and the cost of treatment is high relative to the other options described herein (Seed et al. 2003).

### 9.4 PERMEABILITY AND SEEPAGE

The steady-state seepage analyses resulted in estimated vertical exit gradients ranging from 0.13 to 2.71 at the levee landside toe, for the conditions analyzed at each of the ten cross sections. According to USACE (2005), the minimum allowable FS for underseepage corresponds to a maximum exit gradient of 0.5.

Additionally, the steady-state seepage analyses indicated through-seepage could develop at 11 of the 18 slopes evaluated, which are identified in Table 8.3 of this report. As noted in section 8.2, the potential for through-seepage was estimated at each section by reviewing the seepage analyses output to determine if the phreatic surface emerges on the landside slope during the design flood event. Excessive seepage forces acting on the landward slope could potentially decrease the stability of the slope as a result of sloughing or internal erosion of the slope.

In an effort to model the development of seepage forces and phreatic surfaces during a full flood event that is representative of the Redwood Creek drainage, we conducted transient seepage and stability analyses of the select cross sections likely to develop through-seepage or sections with estimated exit gradients in excess of 0.5 under steady-state conditions. Because steady-state conditions represent a worse-case scenario, those cross sections unlikely to develop through seepage *and* with estimated exit gradients below 0.5 were not evaluated in our transient analyses because they already satisfy performance criteria. The purpose of our transient analyses was to estimate if the conditions evaluated in our steady-state analyses are likely to develop during the projected duration of a 100-yr flood event. The transient seepage analyses for those cross sections estimated vertical exit gradients ranging from less than 0.1 to 0.7 at the levee landside toe, and indicated through-seepage is unlikely to develop at any of the cross sections during the 100-year flood event.

Based on our transient seepage analyses, the estimated vertical exit gradients at the landside toe of the right levee at about Station No. 87+00 and the right levee at about Station No. 114+00 are 0.6 and 0.7, respectively, for the 100-year flood event.

Based on the Lidar data provided and our field exploration, the high exit gradient at the Station No. 114+00 right levee is likely attributable to the interior drainage ditch abutting the levee, and the relatively thin impermeable foundation stratum underlying the ditch. Existing relief wells #1 and #2 may provide drainage and, thus, some measure of seepage mitigation at this location. However, based on our observations and review of historical documents, the functionality of relief wells is undocumented and very likely limited by obstructive debris and sediment.

In our opinion, the high exit gradient at the Station No. 87+00 right levee is likely attributable to the relatively thin impermeable foundation estimated on the basis of our field exploration logs.

Although these two estimated exit gradients exceed the allowable exit gradient defined by ETL 110-2-569 (USACE, 2005), it should be noted that the synthetic hydrograph applied to our analyses modeled a flood event with a greater duration than the typical flood event for Redwood Creek (NHE, 2011a, 2011b and 2011c), and may be considered conservative as a result. Prior to developing any remedial actions to address underseepage at the deficient locations, we recommend additional engineering analyses to better evaluate the anticipated hydrologic conditions and refine the seepage model.

We note that existing relief wells scattered along the project extent are expected to provide some measure of seepage mitigation. However, to ensure relief wells are capable of mitigating seepage forces during a high-flow event, we recommend wells be inspected annually (preferably prior to the wet season), desanded, and pump tested every five years, in accordance with the operation and maintenance manual (USACE, 1969) and EM 1110-2-1914 (USACE, 1992).

According to the 2006 (USACE) inspection report, relief wells have been tested, although test data were not included with the report. As noted in EM 1110-2-1914 (USACE, 1992), pump test data should be documented for future reference.

## 9.5 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.

### 9.5.1 Existing

Our slope stability analyses of existing levee conditions estimated factors of safety ranging from approximately 1.4 to 3.6 and 1.8 to 2.6 for the landside and waterside slopes, respectively. Based on our review of historical information and observations during field exploration, 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 meet or exceed the minimum end-of-construction FS for a new levee, defined as 1.3 (USACE, 2000a).

### 9.5.2 Pseudostatic

Our slope stability analyses of existing levee conditions subjected to pseudostatic (i.e. earthquake) loading estimated factors of safety ranging from approximately 0.9 to 1.6 and 1.0 to 1.3 for the landside and waterside slopes, respectively. A minimum FS is not defined for pseudostatic loading conditions in USACE (2000a).

Based on our analyses, the estimated failure surfaces of potentially unstable slopes are generally surficial. As noted in Section 7.0, the 12- to 24-inch thick layer of riprap slope protection was not included in our stability models. However, we anticipate the riprap would likely reduce the potential for slope instability during a seismic event.

Based on our review of historical information, we understand the levee slopes have not experienced instability during a seismic event. However, according to limited historical seismicity data, the levee system has not been subjected to a horizontal ground acceleration greater than or equal to 0.2g, which corresponds to the estimated design seismic event with an approximately 1 percent annual chance of exceedance (Statistical Return Period ≈ 100 years).

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. If levee slopes failed during or after the design seismic event, water levels in the creek will likely be lower than the adjacent land, and the consequences of failure may be relatively minimal.

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.

### 9.5.3 Full Flood Event

Our slope stability analyses of existing landside levee slopes, based on steady-state full flood conditions, estimated factors of safety ranging from approximately 0.4 to 2.6. A minimum FS of 1.4 is defined for long-term (steady-state seepage) flood conditions in USACE (2000a).

As noted in Section 8.3, we performed transient seepage and stability analyses of select sections that did not meet or exceed the minimum FS defined by USACE (2000a), based on our steady-state analyses. The purpose of our transient analyses was to estimate if the conditions evaluated in our steady-state analyses are likely to develop during the projected duration of a 100-year flood event. Our slope stability analyses of select landside levee slopes, based on transient full flood conditions, estimated factors of safety ranging from approximately 1.38 to 2.69.

Based on our review of historical information, we understand the landside levee slopes have not experienced instability during previous flood events. However, according to USACE (2010b), the USGS gauge located at the upstream end of the project extent has not recorded historical flows greater than or equal to the estimated design flood flow of 58,868 cubic feet per second (cfs).

On the basis of our analyses results, no remedial action is considered necessary at this time. 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.

### 9.5.4 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 0.86 to 2.19. According to USACE (2000a), a minimum FS of 1.0 is defined for rapid drawdown conditions where water levels are unlikely to persist for long periods preceding drawdown.

Although the estimated factors of safety at seven of the analyzed cross section slopes are about 1.0 or less, we note that phreatic surfaces and pore pressures modeled in our analyses were based on the results of our steady-state seepage analyses. Based on our transient seepage analyses of select sections discussed above, we anticipate the phreatic surfaces and pore pressures estimated by our steady-state analyses are unlikely to develop during the estimated duration of a 100-year flood event.

Therefore, the potential for waterside slope failures due to rapid drawdown during the 100-year flood event is considered to be low. 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.

### 9.5.5 Post-Earthquake

As noted in Section 8.3, we performed post-earthquake stability analyses for select levee cross sections with soils considered to be potentially liquefiable during the design seismic event. Our post-earthquake slope stability analyses of select levee sections estimated factors of safety ranging from approximately 0.69 to 3.56 and 0.47 to 2.00 for the landside and waterside slopes, respectively.

A minimum FS is not defined for post-earthquake conditions in USACE (2000a). However, USACE (2010a) states "if post-earthquake factors of safety are greater than 1.2, then no further evaluation is necessary." If the post-earthquake FS was estimated to be between 1.2 and 1.0, we performed a seismic deformation analysis to estimate the levee's performance during the design seismic event.

Based on our post-earthquake stability analyses, five of the slopes evaluated were estimated to have a FS less than 1.0. Slopes with estimated factors of safety less than 1.0 in our post-earthquake stability analyses are considered unstable under static conditions and generally susceptible to flow failure following the design seismic event. Large deformations are typically associated with flow failure of slopes.

Estimated factors of safety below 1.2 in our post-earthquake stability analyses are primarily attributable to the undrained residual strengths applied to our models, discussed in section 8.3. We note that although our methodology in estimating undrained residual strengths is consistent with the recommendation presented in USACE (2010a), the number of published empirical correlations is relatively limited and the current standard of practice is poorly defined. In addition, it should be noted that, to our knowledge, a specific guidance document for seismic stability analysis has yet to be published by the USACE.

The Redwood Creek levee is infrequently subjected to water loading. Therefore, the probability of a high-water flood event and design earthquake event occurring simultaneously is considered to be relatively low. If levee slopes failed during or after the design seismic event, water levels in the creek will likely be lower than the adjacent land, and the consequences of failure may be relatively minimal.

To minimize the potential for post-earthquake instability, flow failure deformations, and the general impacts of liquefaction, typical remedial actions are discussed in Section 9.3.1 of this report. 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.

## 9.6 SEISMIC DEFORMATION

As stated in Section 8.4 of this report, we performed deformation analyses for levee slopes with factors of safety between 1.2 and 1.0 estimated in our post-earthquake analyses.

As shown in Table 8.2, displacements ranging between 2.7 and 3.8 feet were estimated for the three waterside levee slopes evaluated.

Based on our review of historical information, we understand the levee slopes have not experienced instability or deformation following an earthquake event. Following a magnitude 6.5 earthquake on January 9, 2010, no indications of earthquake damage were observed by Humboldt County inspectors, and no reports of damage were received by the County (County of Humboldt, 2010). According to the USGS' Earthquake Hazards Program website, the 2010 earthquake's epicenter was located approximately 70 miles southwest of Orick. Additionally, the project site is estimated to have been subjected to a PHGA between 0.05g and 0.1g during the 2010 event (USGS).

To reduce the magnitude of seismically induced displacements and the general impacts of liquefaction, typical remedial actions are discussed in Section 9.3.1 of this report. At a minimum, we recommend the levees be inspected for signs of excessive deformation or failure as soon as possible following an earthquake and, if necessary, repaired as soon as conditions will allow.

## 9.7 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 for the majority of the project extent are considered necessary at this time.

However, as noted in Section 9.2, to minimize the potential for static settlement of the levee and foundation material between about Station Nos. 86+00 and 89+00 and between about Station Nos. 138+00 and 144+00, we recommend the observed gravel stockpiles be removed in accordance with the recommendations presented in the most recent inspection report (USACE, 2009b).

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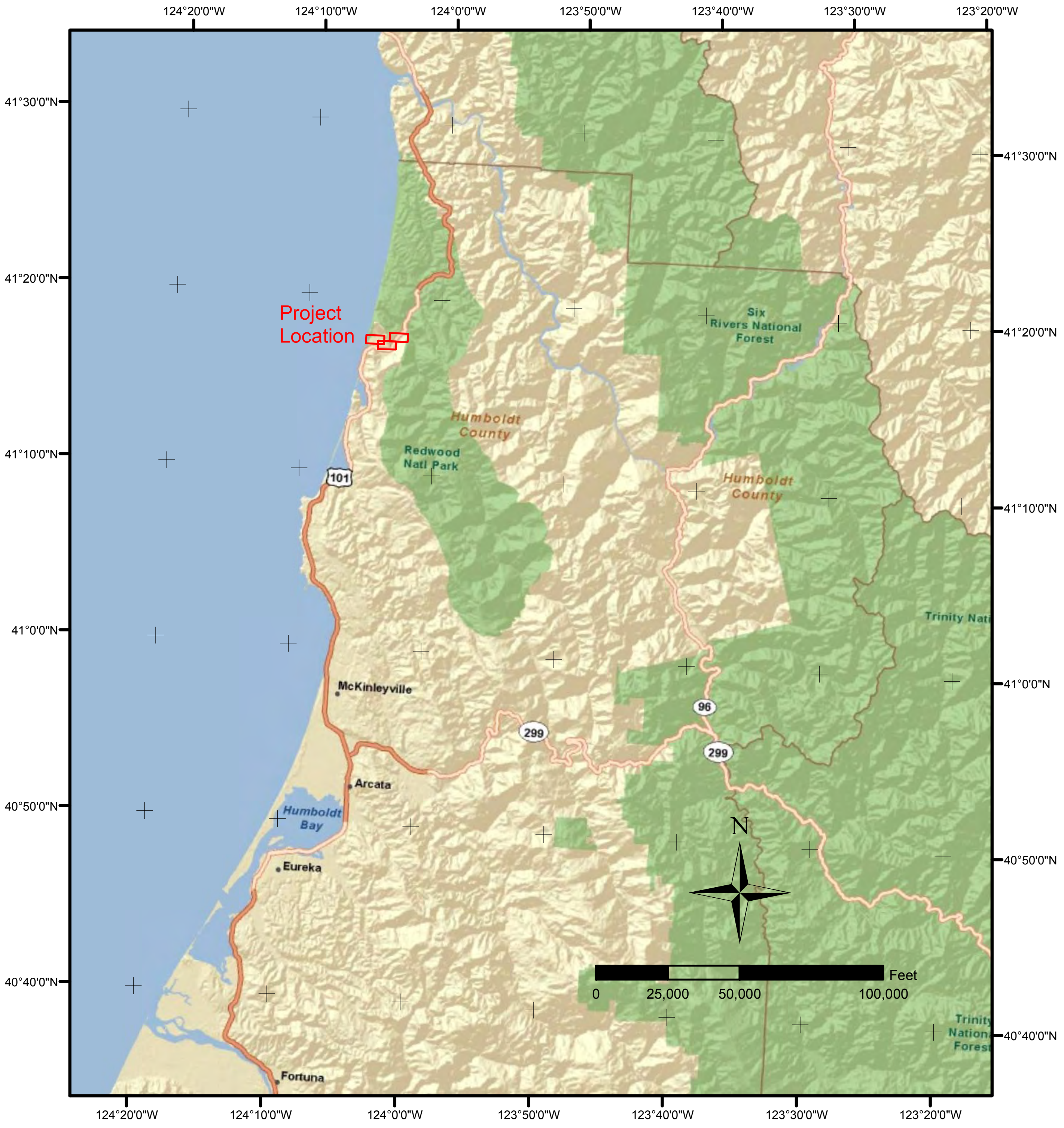
# PLATES



# VICINITY MAP

## Redwood Creek Levee Geotechnical Evaluation Project

Orick, Humboldt County, California

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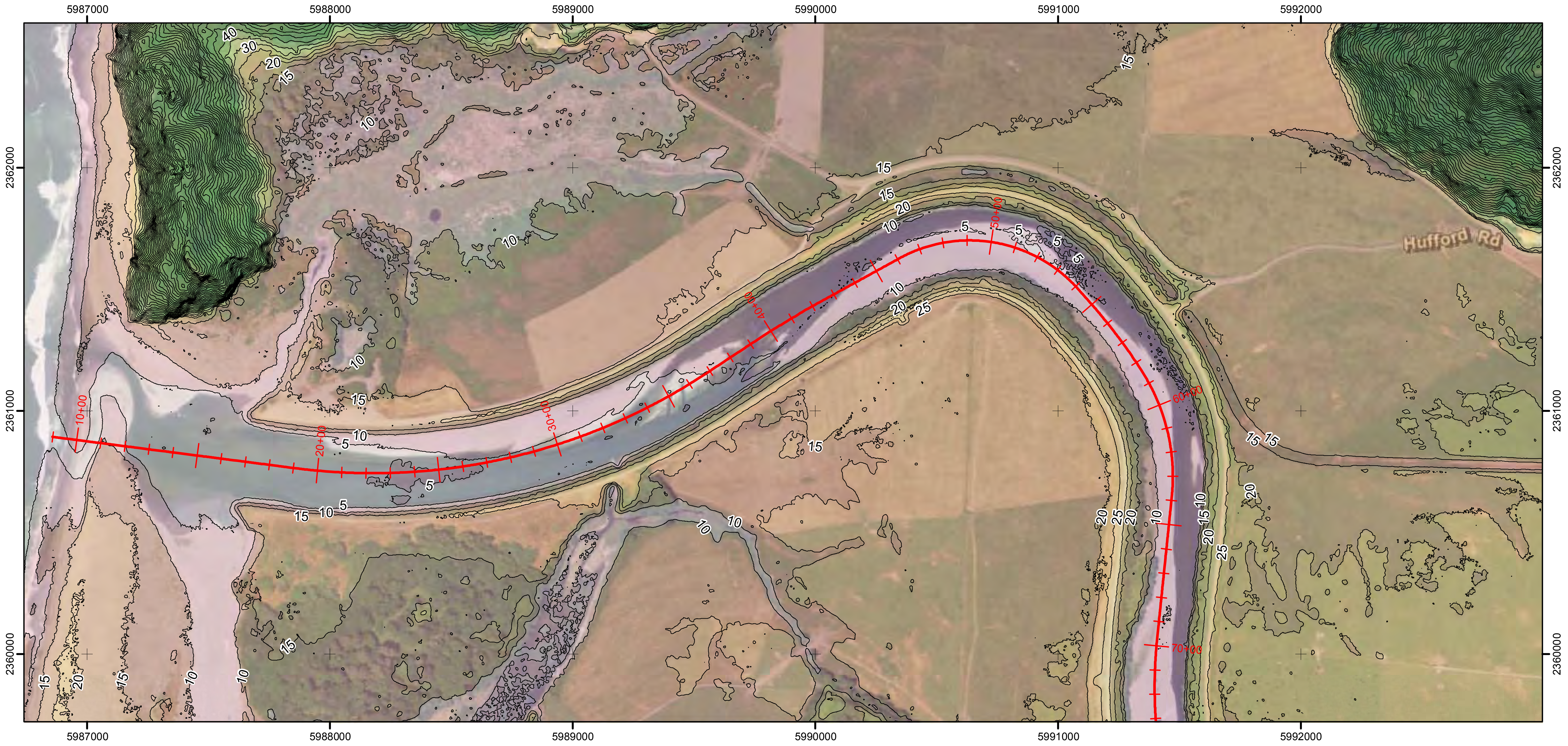
Humboldt County Public Works Department
Fugro Project No. 04.B3502002
CGI Project No. 10-1949.01

CGI Technical Services Inc.

FUGRO



**Legend**

Approximate River Centerline with Stations (2+00, 4+00)

LIDAR Contour, interval = 5 ft

Coordinate System: CA State Plane, Zone 1, NAD83, Feet

2007 LIDAR Topography, Feet (re. NAVD88)

| | |
|---|---|
| ▨ < 5 | ▨ 25 - 30 |
| ▨ 5 - 10 | ▨ 30 - 35 |
| ▨ 10 - 15 | ▨ 35 - 40 |
| ▨ 15 - 20 | ▨ > 40 |
| ▨ 20 - 25 | |

N

1:4,800

0 200 400 800 Feet

## SITE MAP

Redwood Creek Levee
Geotechnical Evaluation Project
Orick, Humboldt County, California

PLATE 2-1a

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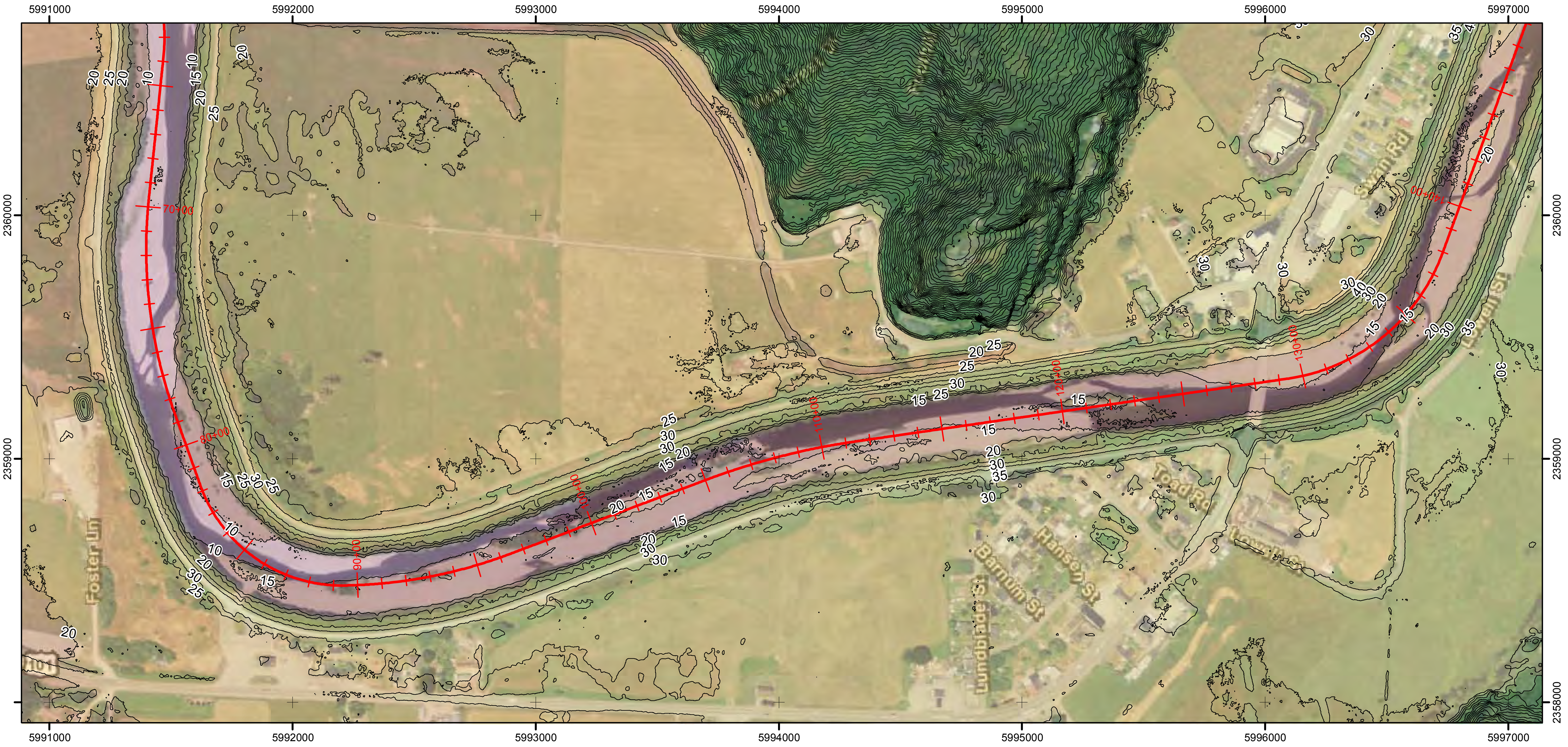
Humboldt County Public Works Department
Fugro Project No. 04.B3502002
CGI Project No. 10-1949.01

CGI Technical Services Inc.

Fugro



**Legend**

Approximate River Centerline with Stations

LIDAR Contour, interval = 5 ft

Coordinate System: CA State Plane, Zone 1, NAD83, Feet

2007 LIDAR Topography, Feet (re. NAVD88)

- < 5
- 5 - 10
- 10 - 15
- 15 - 20
- 20 - 25
- 25 - 30
- 30 - 35
- 35 - 40
- \> 40

N

1:4,800

0 200 400 800 Feet

## SITE MAP

Redwood Creek Levee
Geotechnical Evaluation Project
Orick, Humboldt County, California

PLATE 2-1b

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Humboldt County Public Works Department
Fugro Project No. 04.B3502002
CGI Project No. 10-1949.01

CGI TECHNICAL SERVICES INC.

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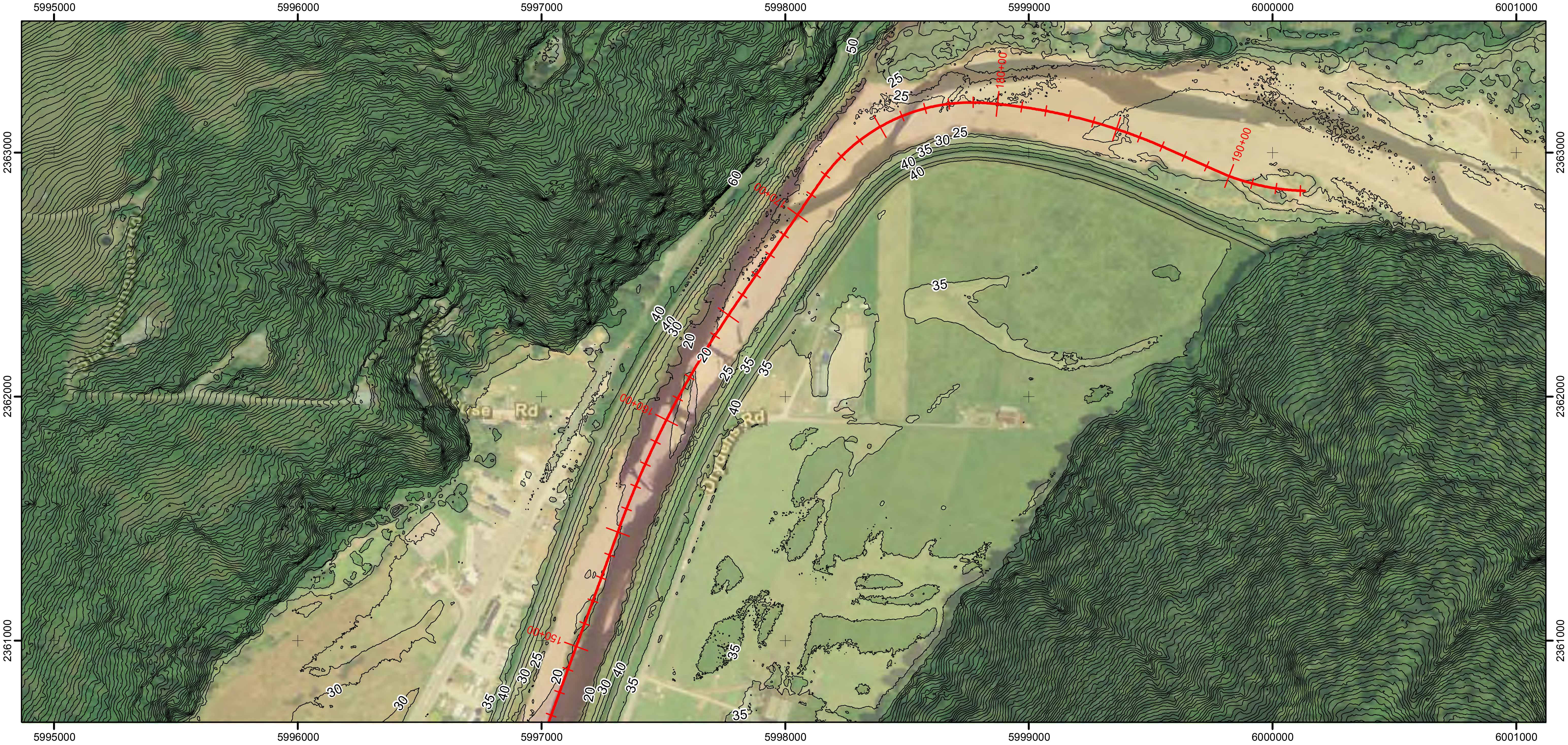

**Legend**

Approximate River Centerline with Stations (2+00, 4+00)

LIDAR Contour, interval = 5 ft

Coordinate System: CA State Plane, Zone 1, NAD83, Feet

2007 LIDAR Topography, Feet (re. NAVD88)

| | |
|---|---|
| ■ | < 5 |
| ■ | 5 - 10 |
| ■ | 10 - 15 |
| ■ | 15 - 20 |
| ■ | 20 - 25 |
| ■ | 25 - 30 |
| ■ | 30 - 35 |
| ■ | 35 - 40 |
| ■ | > 40 |

N

1:4,800

0 200 400 800 Feet

## SITE MAP
Redwood Creek Levee
Geotechnical Evaluation Project
Orick, Humboldt County, California

PLATE 2-1c

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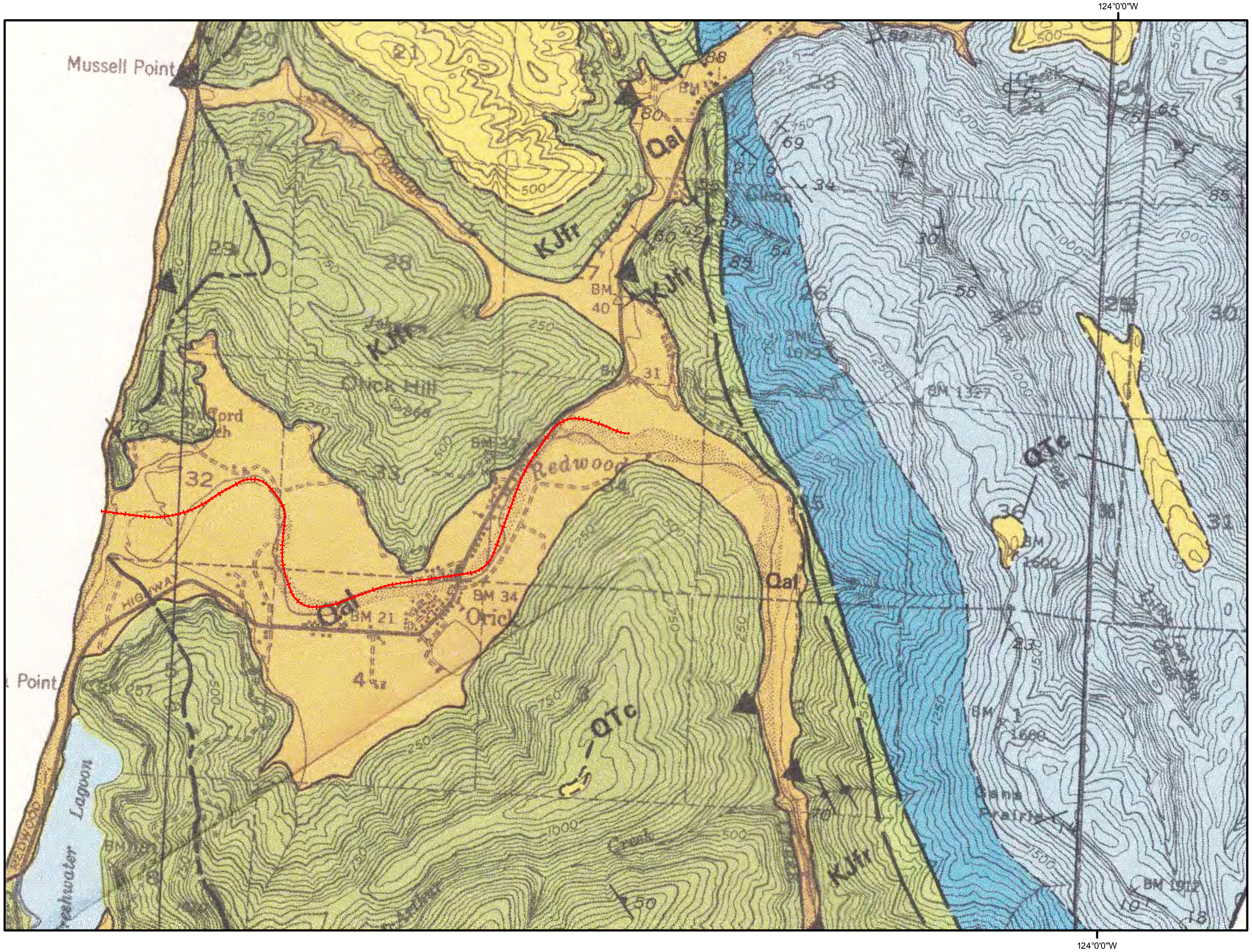


**Legend**

- Redwood Creek Centerline
- Drainage divide
- Contact—dashed where approximately located, short dashes where gradational and/or sheared, dotted where concealed
- Fault—dashed where approximately located, short dashes where inferred, dotted where concealed
- Thrust fault—sawteeth on upper plate, dashed where approximately located
- Photo-geological lineaments
- Area of common greenstone, chert, or sandstone outcrops
- Outcrops of metabasalt and metatuff

Strike and dip of beds
- 32 Inclined
- Vertical
- 53 Vertical; dot indicates top of bed
- 49 Overturned

Strike and dip of foliation
- Inclined
- Vertical

- Crenulated bedding; arrow shows trend of minor fold axes
- Minor folds, showing plunge of axes

**Geologic Units**

- Qal - Modern Alluvium
- QTc - Coastal Plain Sediments
- KJfl - Coherent Unit of Lacks Creek
- KJfc - Incoherent Unit of Coyote Creek
- KJfr - Schist of Redwood Creek

SOURCE: Water-Resources Investigations Open-File Report 81-496, Department of the Interior, United States Geological Survey.

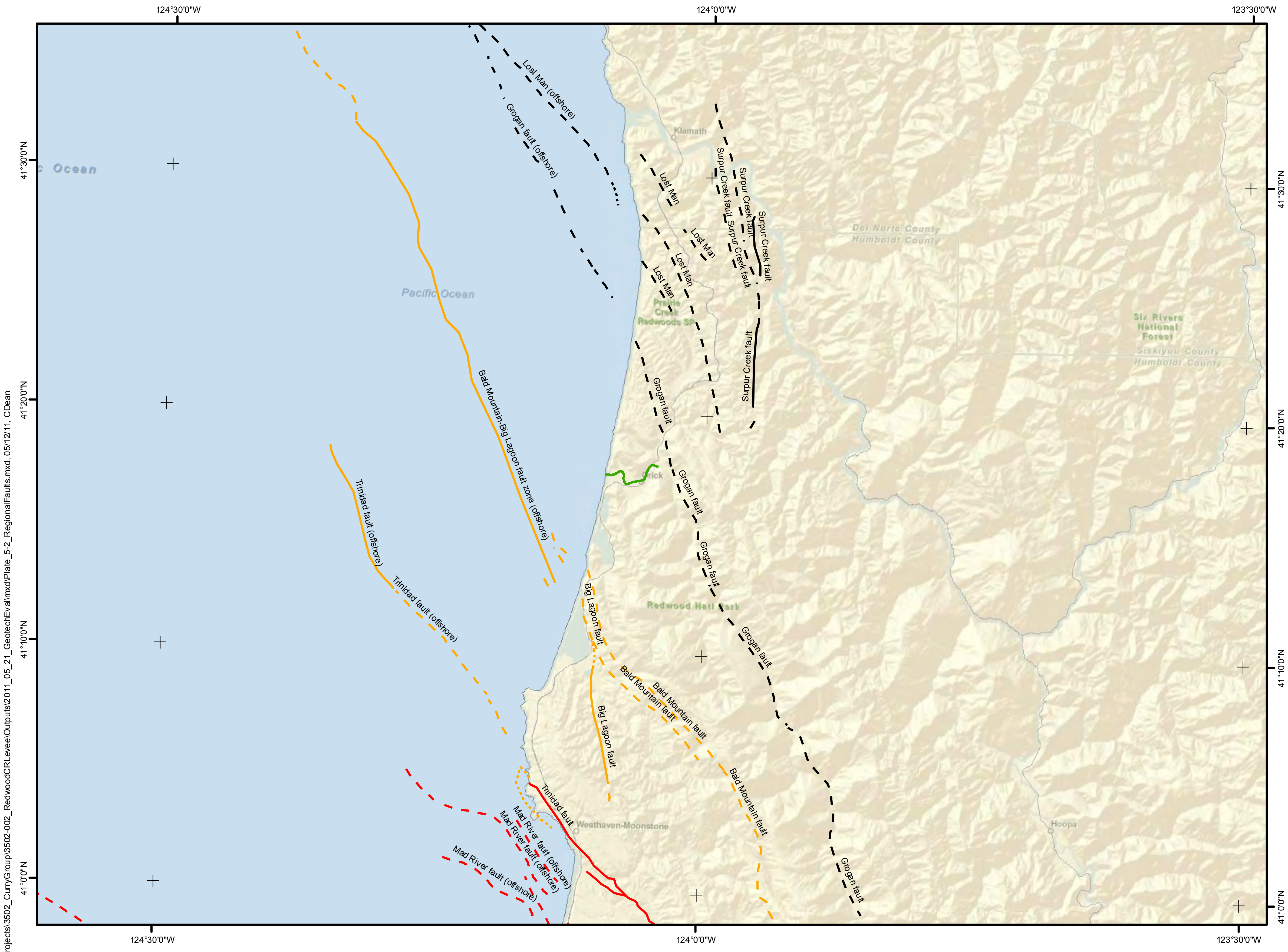
N

0 1,500 3,000 6,000 Feet

0 0.25 0.5 1 Miles

# GEOLOGIC MAP
Redwood Creek Levee
Geotechnical Evaluation Project
Orick, Humboldt County, California

N:\Projects\3502_CurryGroup\3502-002_RedwoodCRLevee\Outputs\2011_05_21_GeotechEval\mxd\Plate_5-1_RegionalGeologicMap.mxd, 05/12/11, CDean



**Legend**

- 🟩 Site
- Highways

**Faults**

- 🟥 Active Fault
- 🟧 Potentially Active Fault
- ⬛ Inactive Fault

Dashed where approximate,
dotted where concealed

SOURCES:
Faults:

1) Bryant, W.A. (compiler), 2005, Digital Database of Quaternary and Younger Faults from the Fault Activity Map of California, version 2.0: California Geological Survey Web Page, http://www.consrv.ca.gov/CGS/information/publications/QuaternaryFaults_ver2.htm>; (2005).

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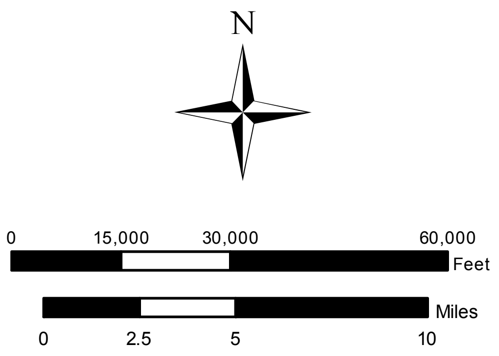
3) Bing Yen & Associates, Inc., Earthquake Faults, (2000).



## REGIONAL FAULTS

Redwood Creek Levee
Geotechnical Evaluation Project
Orick, Humboldt County, California

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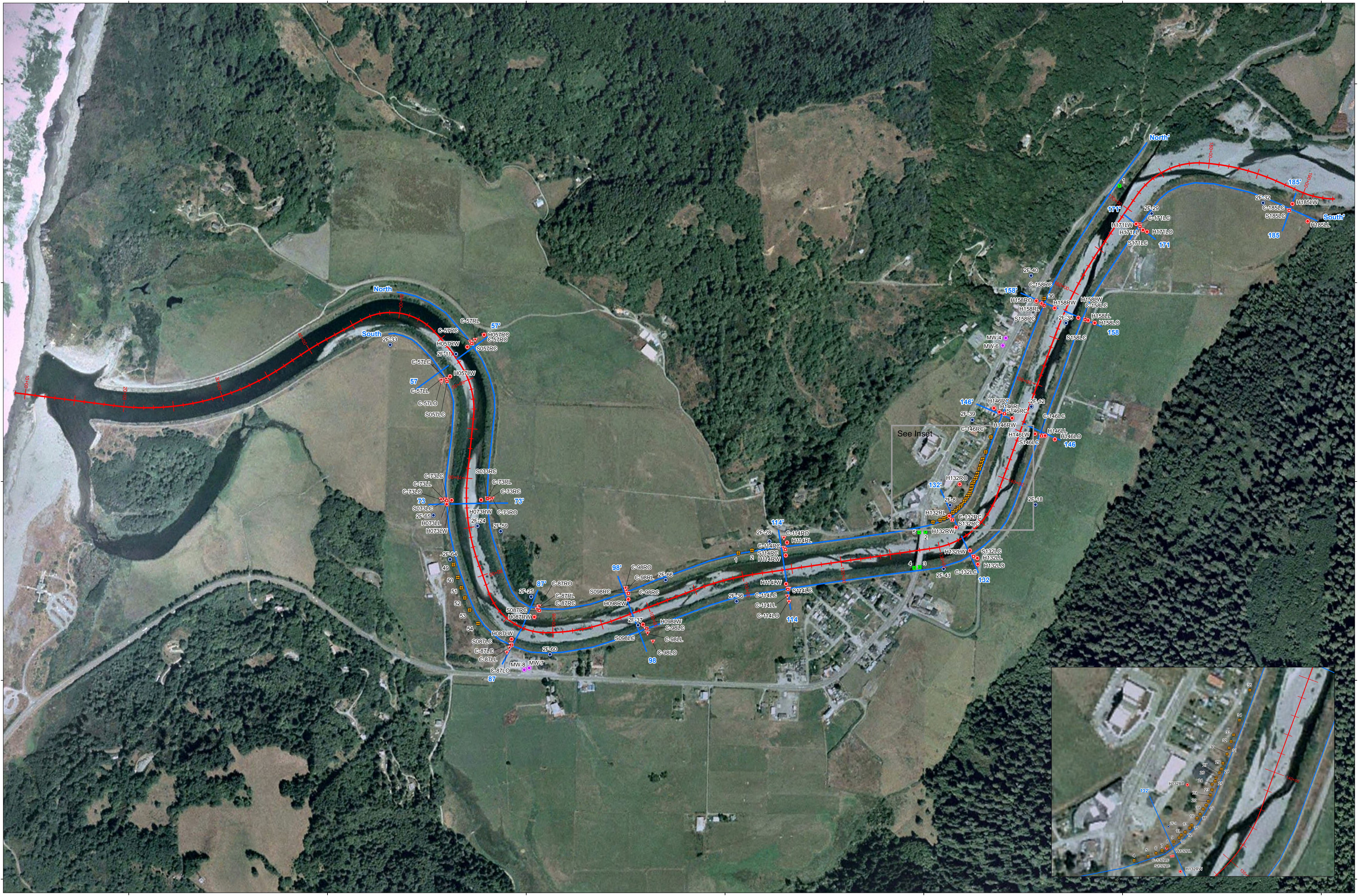
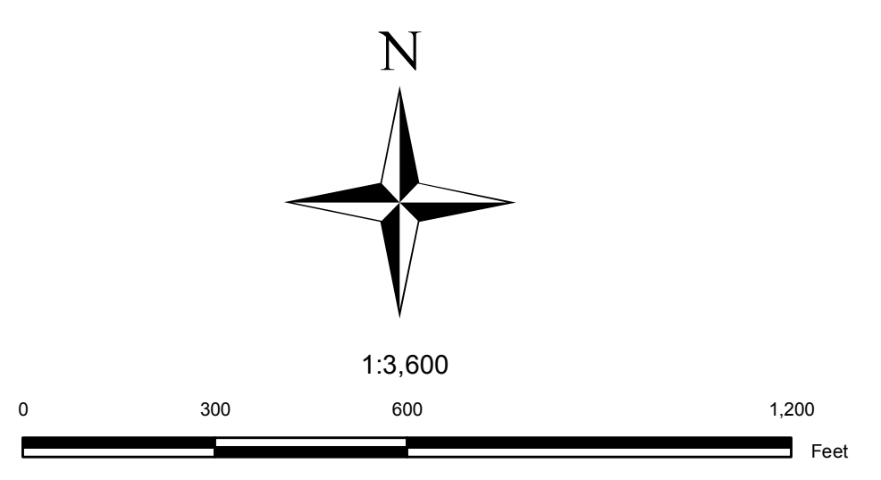


## Legend

| | |
|---|---|
| 🟩 | Control Point |
| ⊕ | Relief Well |
| — | Cross Section Line |
| ┼ 2+00 ┼ 4+00 | Approximate River Centerline with Stations |

## Explorations

| | |
|---|---|
| ◉ | CGI/Fugro Hollow Stem Auger |
| ▣ | CGI/Fugro Vibracore |
| ▽ | CGI/Fugro CPTs |
| ◉ | Historical SHN Borings |
| ◉ | Historical USACE Borings |



Humbolt County Public Works Department
Fugro Project No. 04.B3502002
CGI Project No. 10-1949.01

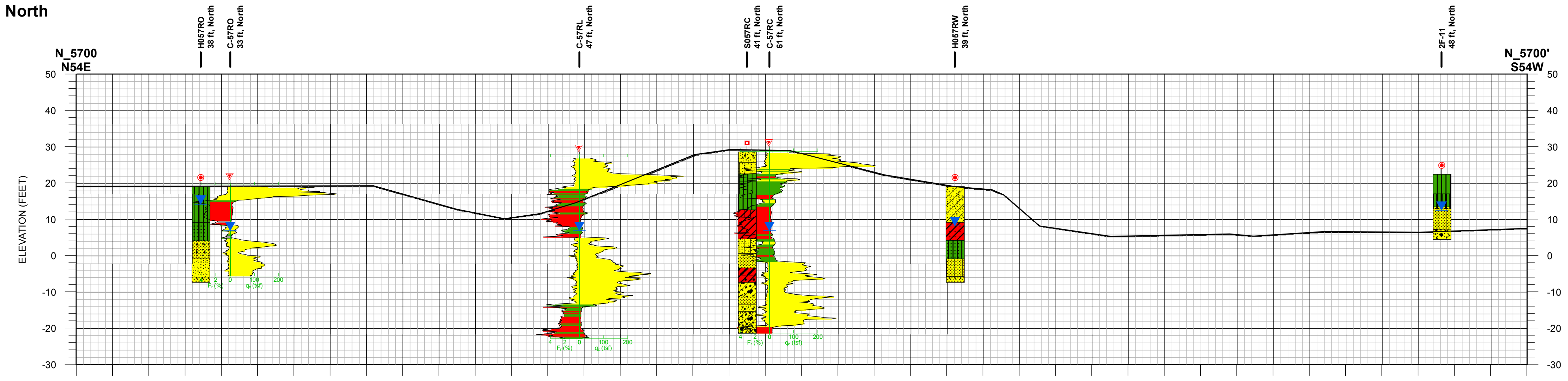
CGI Technical Services Inc.
Fugro

# EXPLORATION LOCATION PLAN
Redwood Creek Levee Geotechnical Evaluation Project
Orick, Humboldt County, California

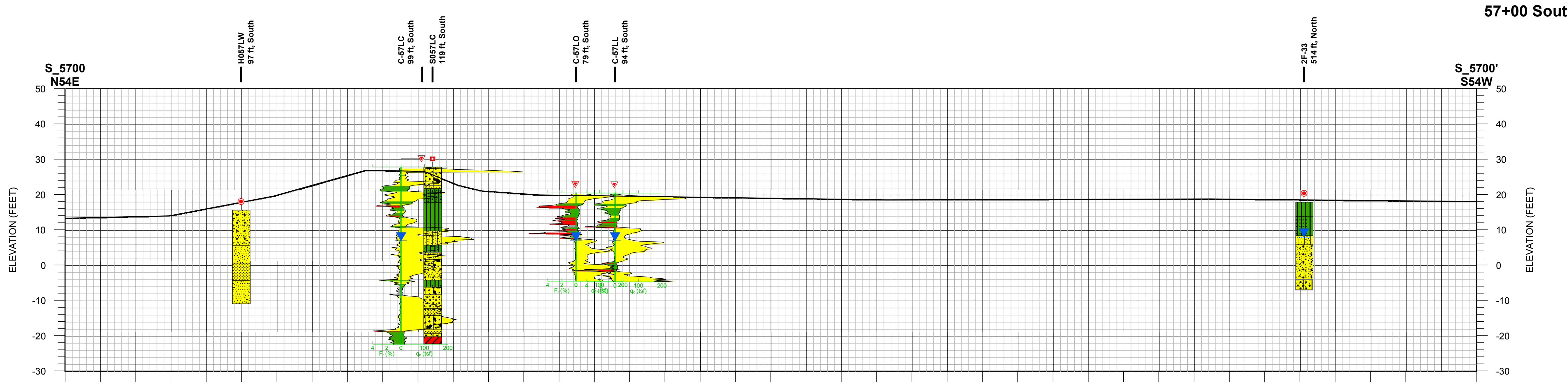
| NO: | DATE: | DESCRIPTION: | DRAWN: | CHKD: | APPR: |
|---|---|---|---|---|---|
| 1 | May 2011 | Exploration Plan | DRP | CBD | GE |

PLATE 5-3

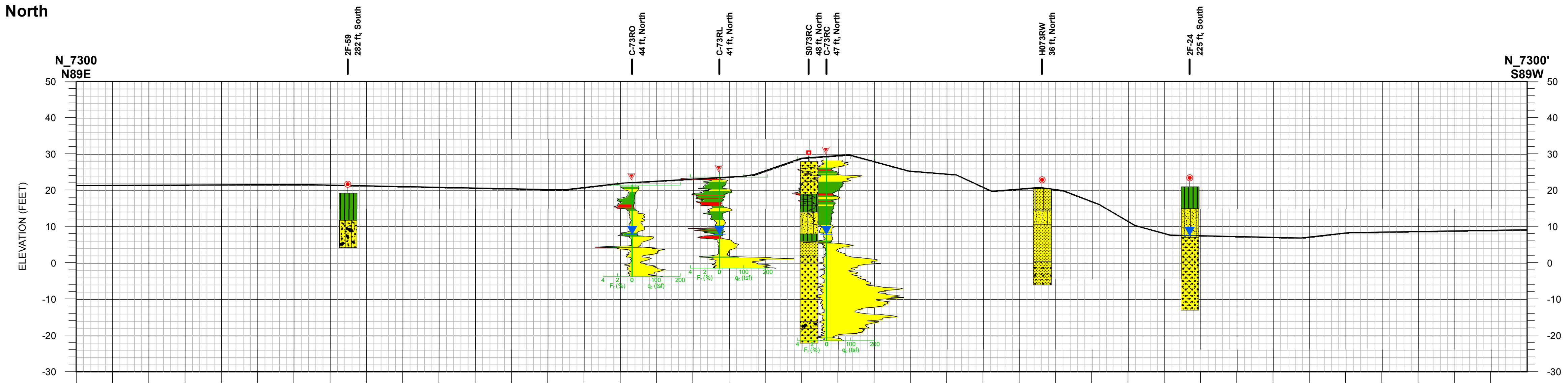
57+00 North



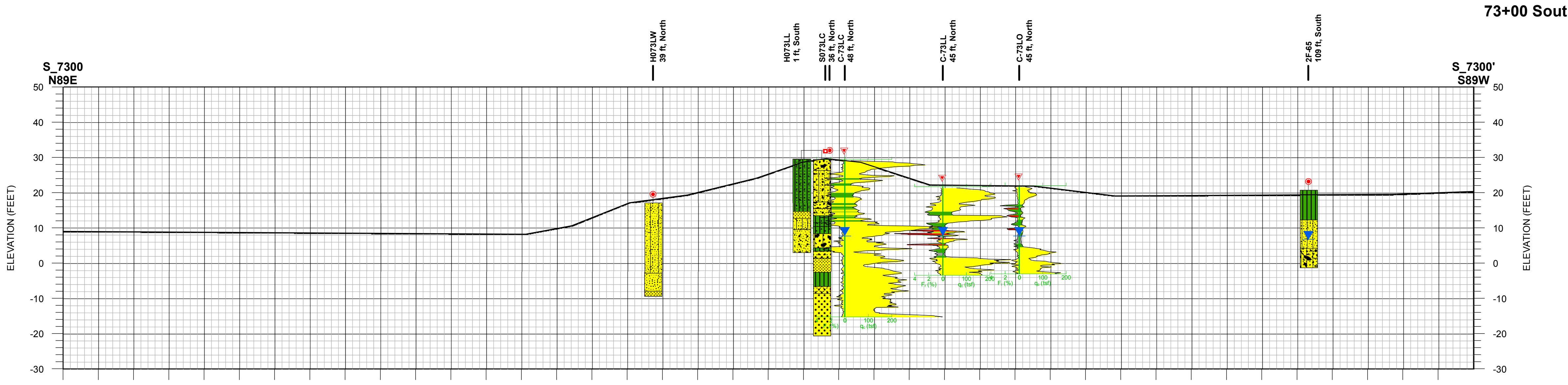
57+00 South



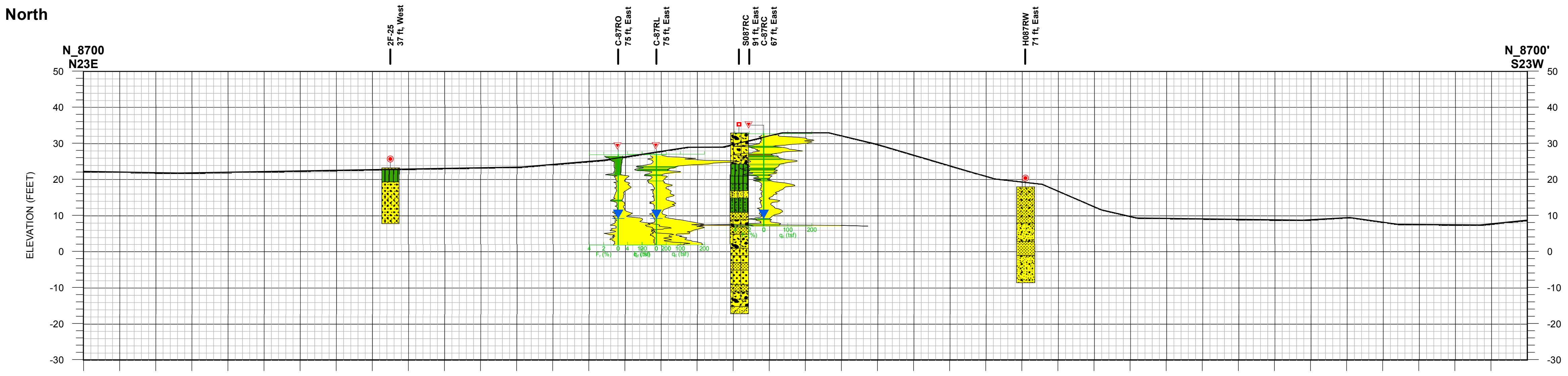
73+00 North



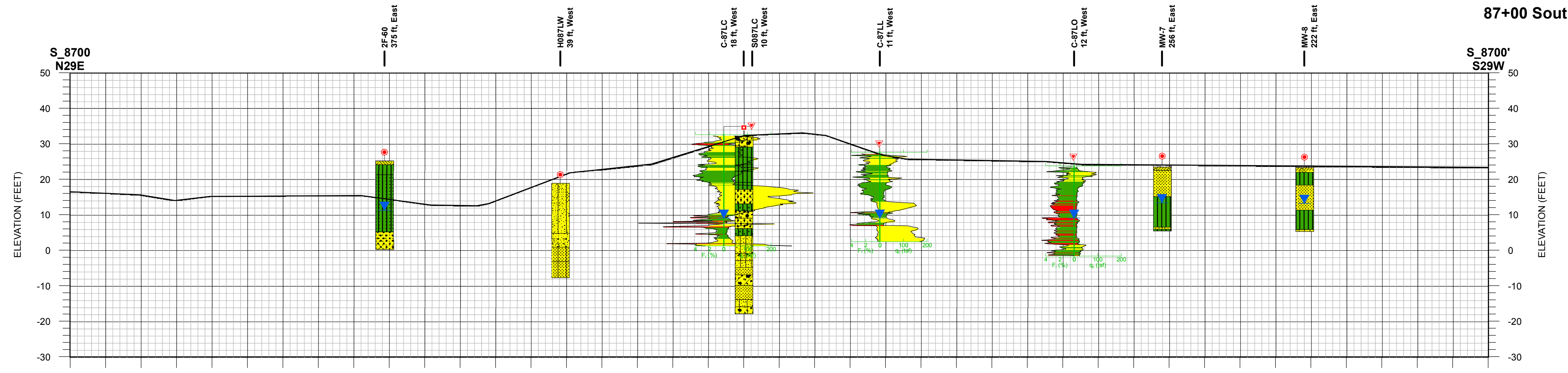
73+00 South



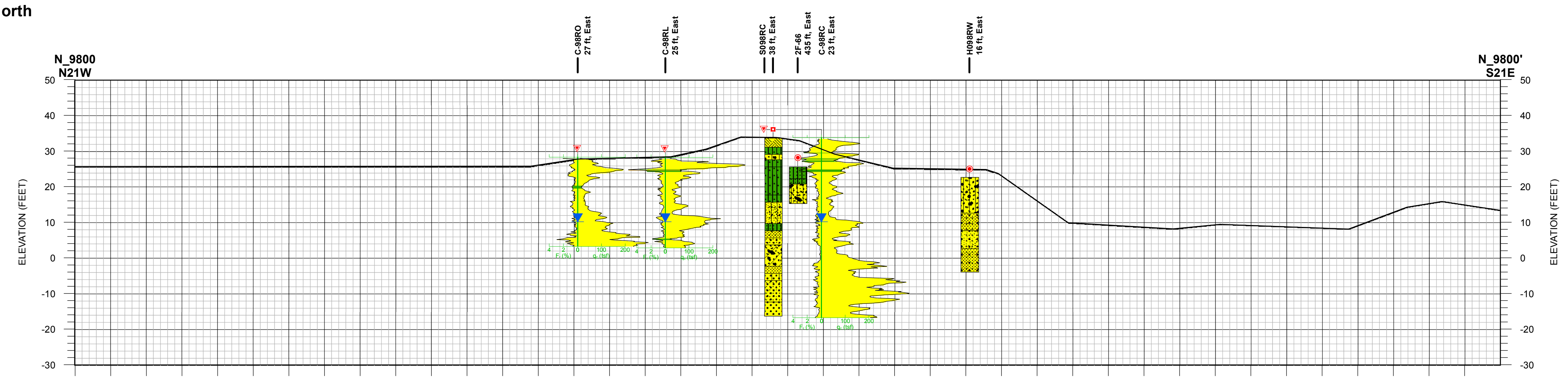
87+00 North



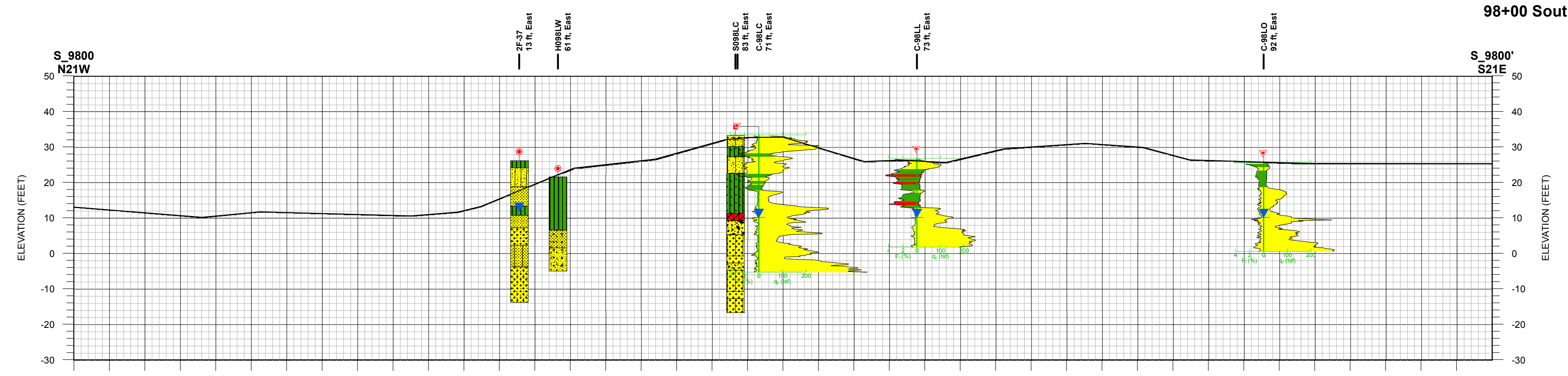
87+00 South



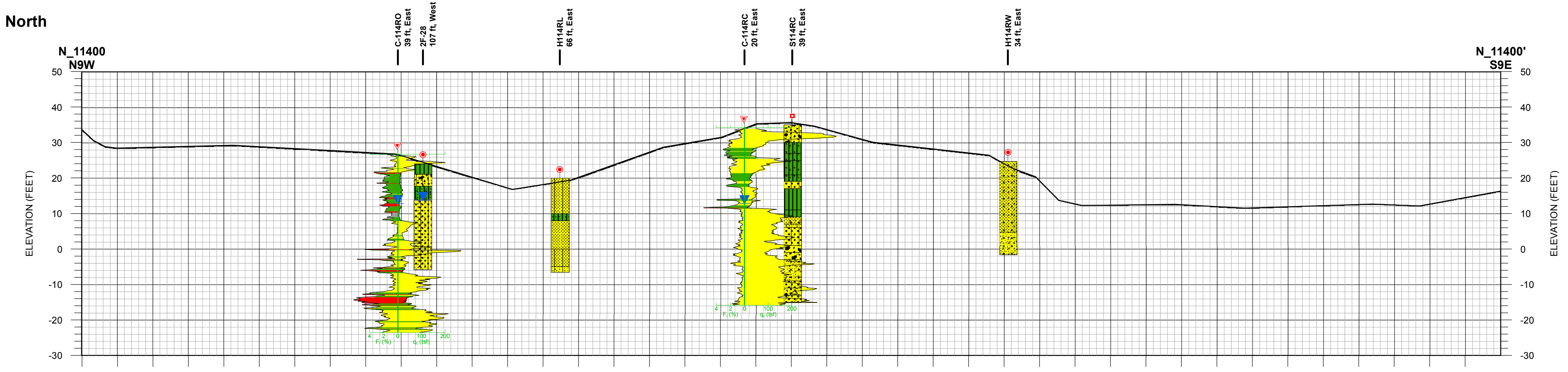
98+00 North



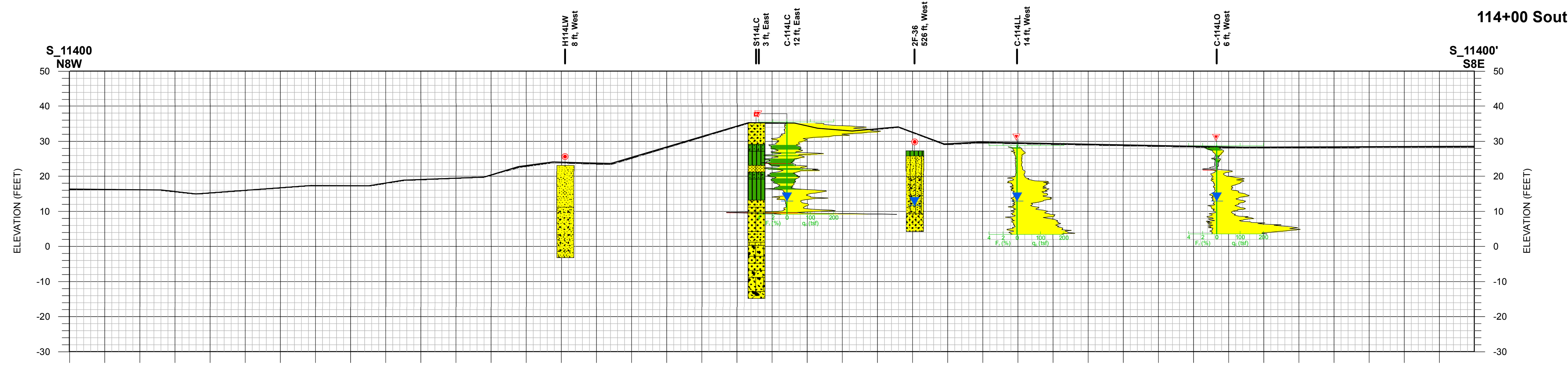
98+00 South

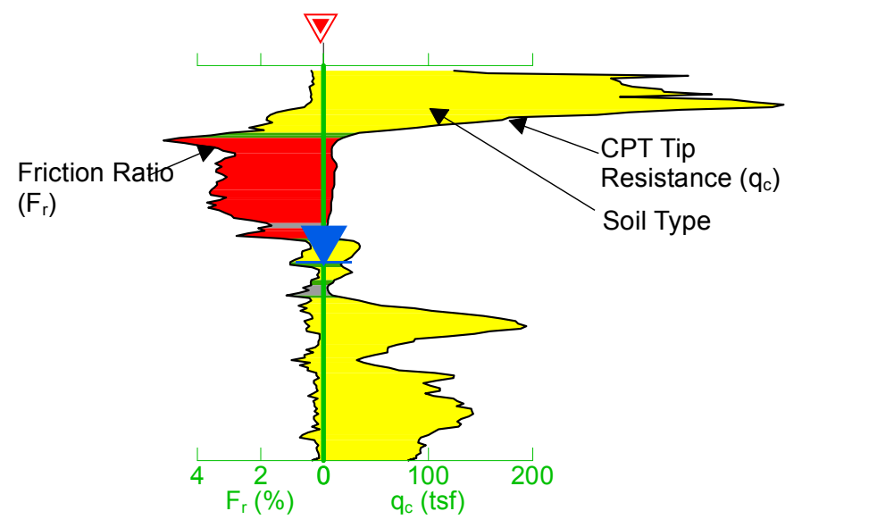


114+00 North

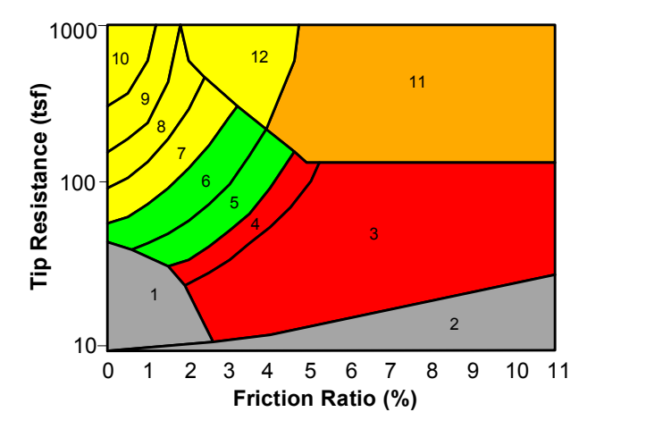


114+00 South





**COLOR LEGEND FOR FRICTION RATIO TRACES**

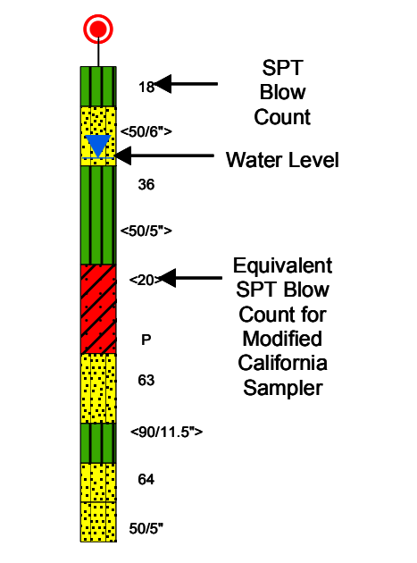


(Robertson and Campanella, 1988)

| Zone | Soil Behavior Type | U.S.C.S. |
|---|---|---|
| 1 | Sensitive Fine-grained | OL-CH |
| 2 | Organic Material | OL-OH |
| 3 | Clay | CH |
| 4 | Silty Clay to Clay | CL-CH |
| 5 | Clayey Silt to Silty Clay | MH-CL |
| 6 | Sandy Silt to Clayey Silt | ML-MH |
| 7 | Silty Sand to Sandy Silt | SM-ML |
| 8 | Sand to Silty Sand | SM-SP |
| 9 | Sand | SW-SP |
| 10 | Gravelly Sand to Sand | SW-GW |
| 11 | Very Stiff Fine-grained * | CH-CL |
| 12 | Sand to Clayey Sand * | SC-SM |

*overconsolidated or cemented

**CPT CORRELATION CHART**



**SOIL BORING LITHOLOGY**

- 🟥 Lean CLAY (CL)
- 🟥 Silty CLAY (CL-ML)
- 🟥 Sandy Lean Clay (CL)
- 🟥 Sandy, Gravelly Lean CLAY (CL)
- 🟥 Gravelly Lean CLAY (CL)
- 🟩 Silt (ML)
- 🟩 Sandy, Gravelly SILT (ML)
- 🟩 Sandy SILT (ML)
- 🟩 Gravelly SILT (ML)
- 🟨 Poorly-Graded SAND (SP)
- 🟨 Poorly-Graded SAND with Silt (SP-SM)
- 🟨 Gravelly Poorly-Graded SAND with Silt (SP-SM)
- 🟨 Gravelly Poorly-Graded SAND (SP)
- 🟨 Well-Graded SAND (SW)
- 🟨 Well-Graded SAND with Clay (SW)
- 🟨 Well-Graded SAND with Silt (SW-SM)
- 🟨 Gravelly Well-Graded SAND (SW)
- 🟨 Clayey SAND (SC)
- 🟨 Gravelly, Clayey SAND (SC)
- 🟨 Silty SAND (SM)
- 🟨 Gravelly Silty SAND (SM)
- 🟨 Poorly-Graded GRAVEL (GP)
- 🟨 Poorly-Graded GRAVEL with Clay (GP-GC)
- 🟨 Poorly-Graded GRAVEL with Silt (GP-GM)
- 🟨 Sandy GRAVEL with Silt (GP-GM)
- 🟨 Sandy GRAVEL (GP)
- 🟨 Well-Graded GRAVEL with Clay (GW-GC)
- 🟨 Well-Graded GRAVEL with Silt (GW-GM)
- 🟨 Sandy Well-Graded GRAVEL (GW)
- 🟨 Sandy, Clayey GRAVEL (GC)
- 🟨 Silty Gravel (GM)
- 🟨 Sandy, Silty GRAVEL (GM)
- ▨ Fill
- ▨ Asphaltic Concrete

20 ft
20 ft
Vertical Exaggeration = 1

Humbolt County Public Works Department
Fugro Project No. 04.B3502002
CGI Project No. 10-1949.01

CGI TECHNICAL SERVICES INC.
FUGRO

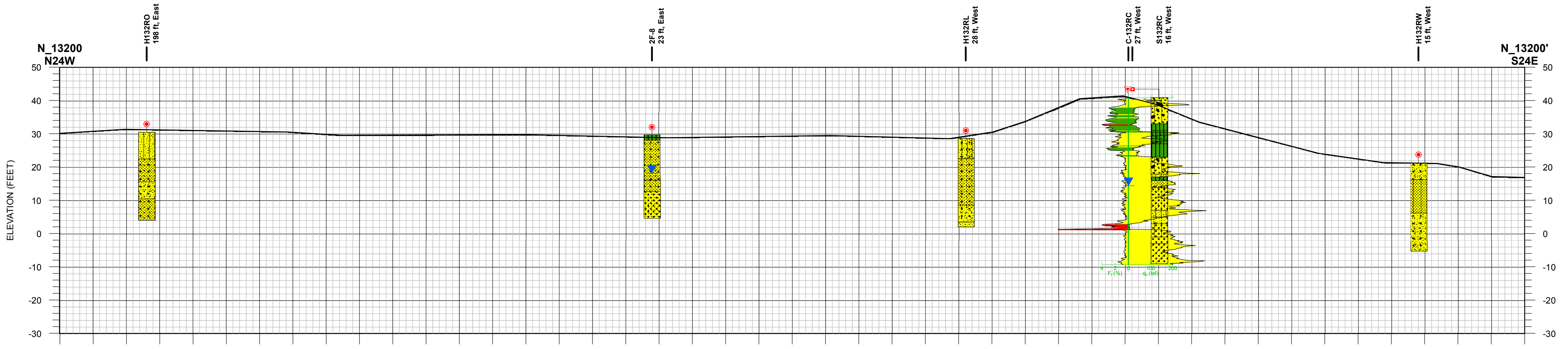
## CROSS SECTIONS
## STATION 57+00, 73+00, 87+00, 98+00 AND 114+00
Redwood Creek Levee Geotechnical Evaluation Project
Orick, Humboldt County, California

| NO: | DATE: | DESCRIPTION: | DRAWN: | CHKD: | APPR: |
|---|---|---|---|---|---|
| 1 | May 2011 | Levee Cross Sections | DRP | CBD | GE |

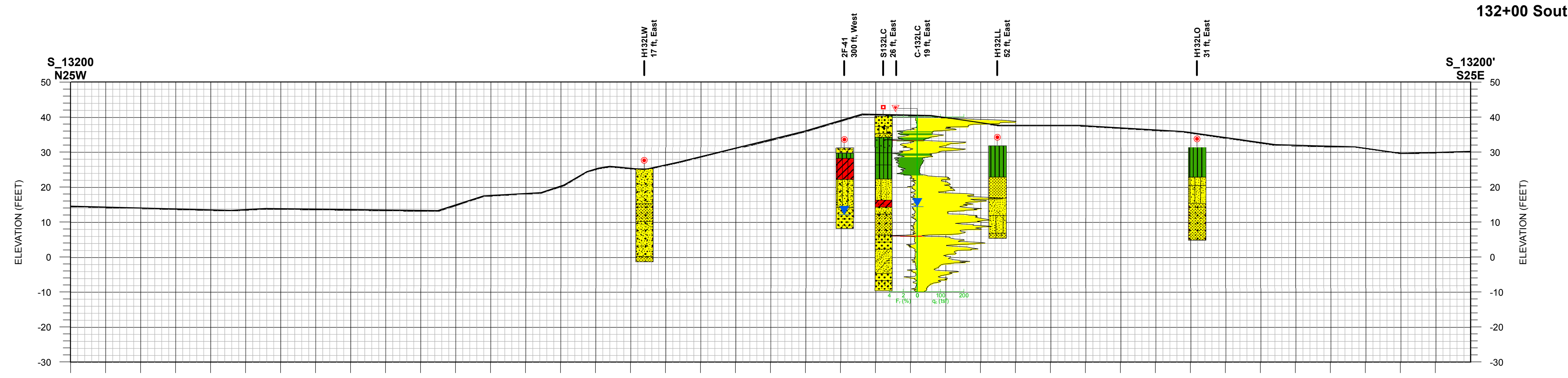
PLATE 5-4

N:\Projects\3502_CurryGroup\3502-002_RedwoodCRLevee\Outputs\2011_05_21_GeotechEval\mxd\Plate_5-5_CrossSection_Page2.mxd, 05/11/11, CDean

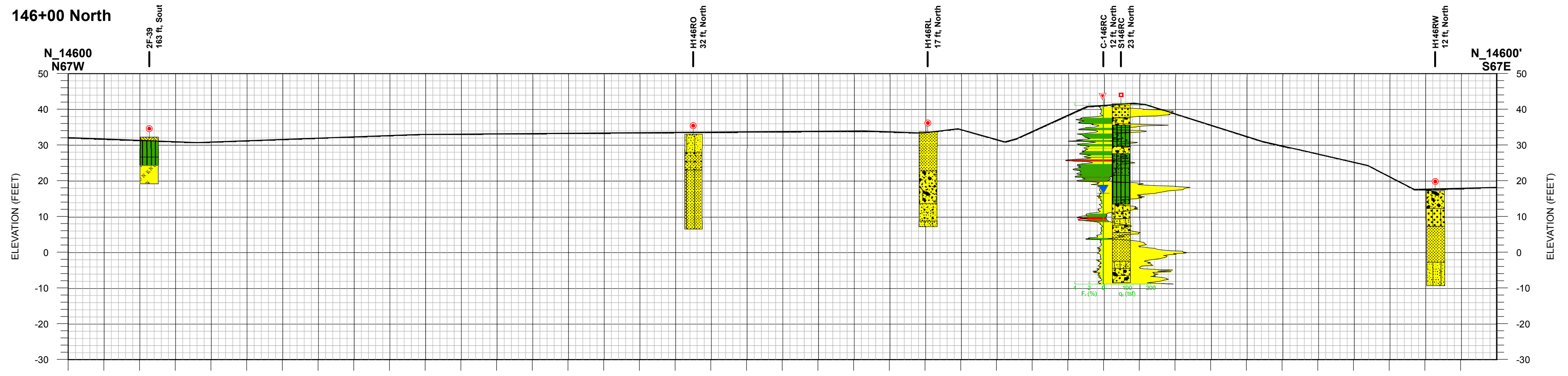
## 132+00 North



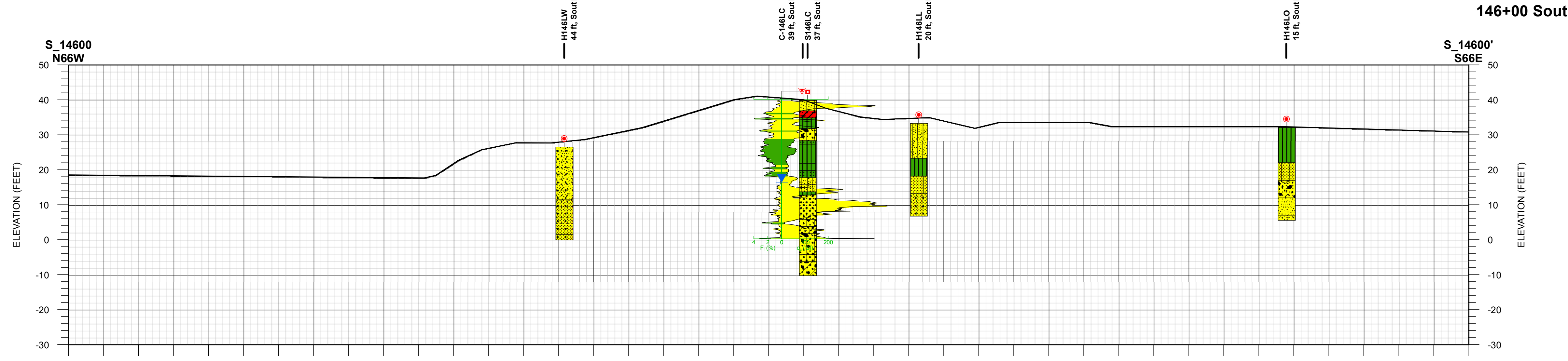
## 132+00 South



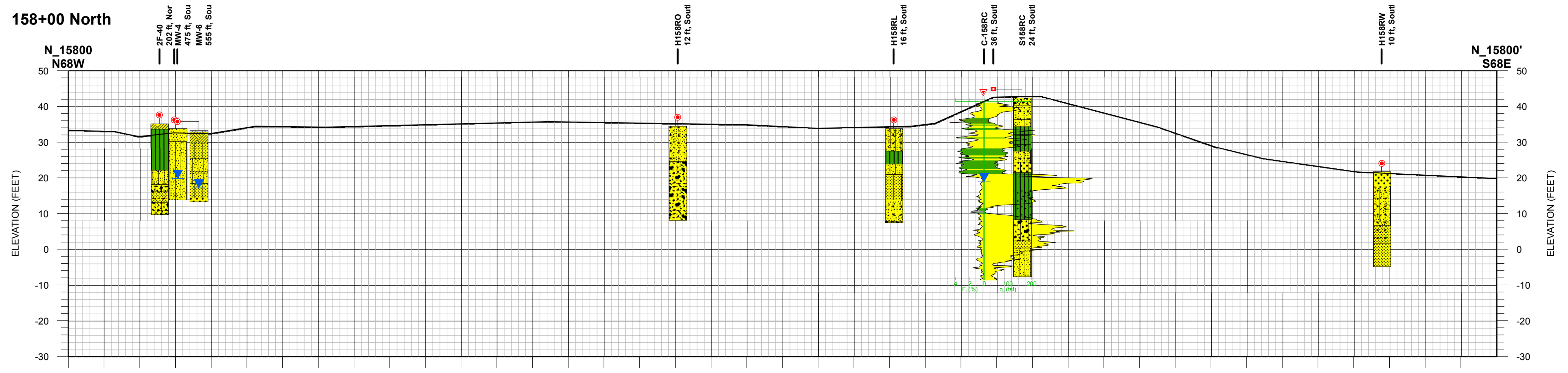
## 146+00 North



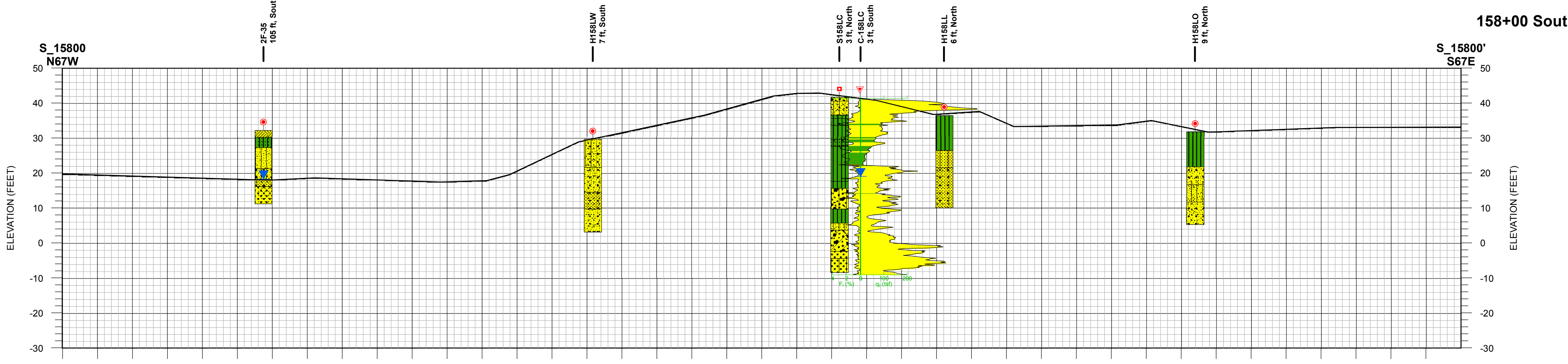
## 146+00 South

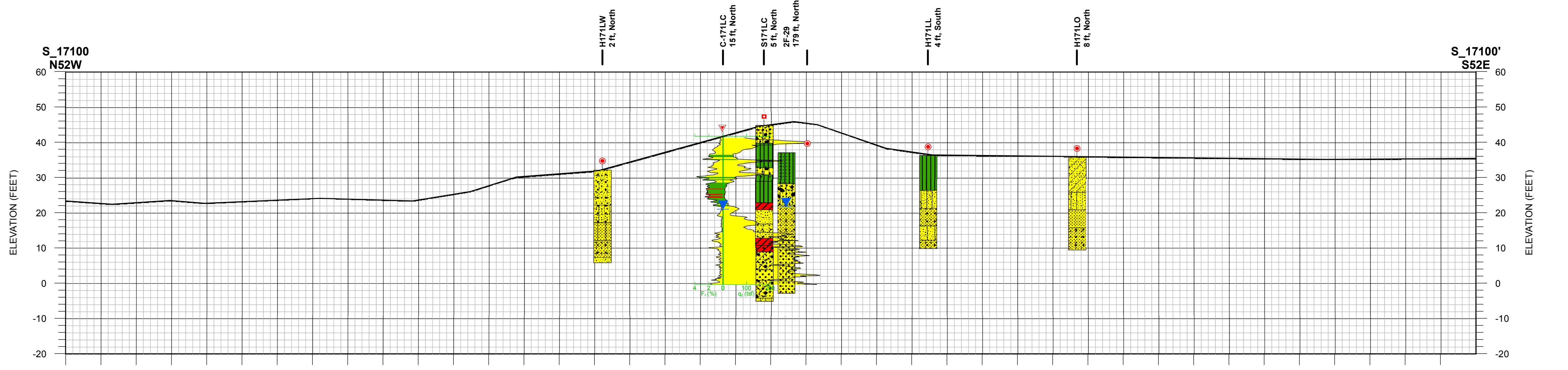


## 158+00 North

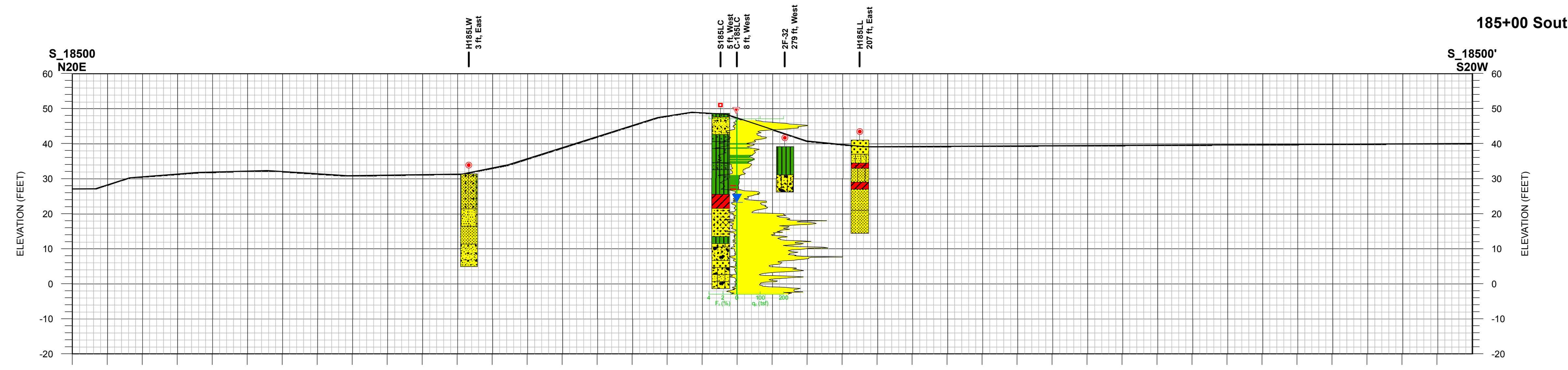


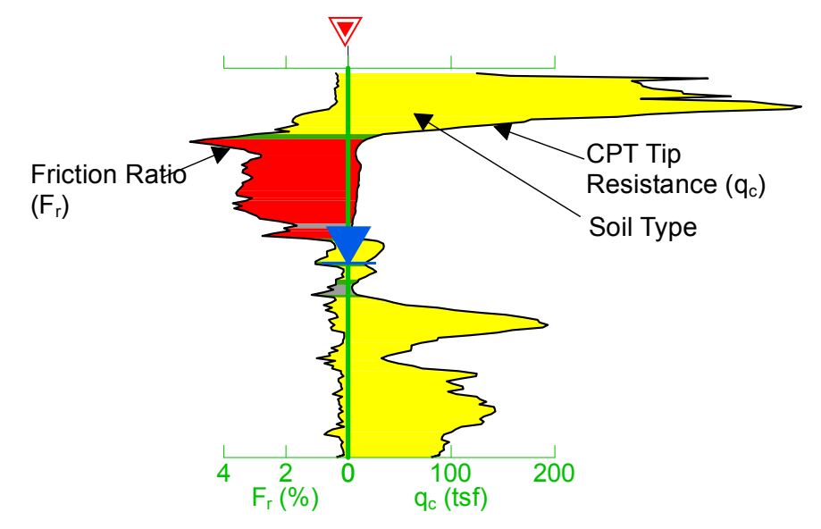
## 158+00 South



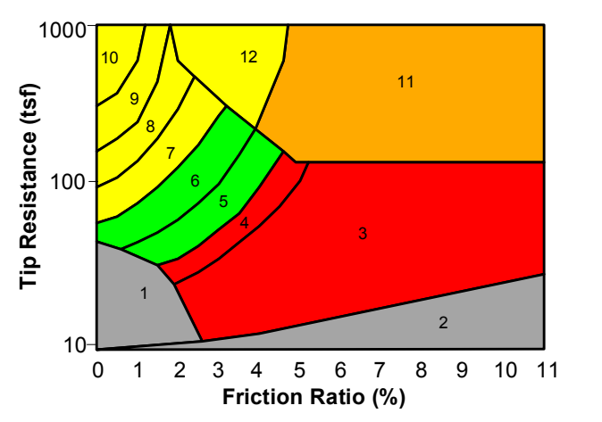


## 185+00 South





### COLOR LEGEND FOR FRICTION RATIO TRACES

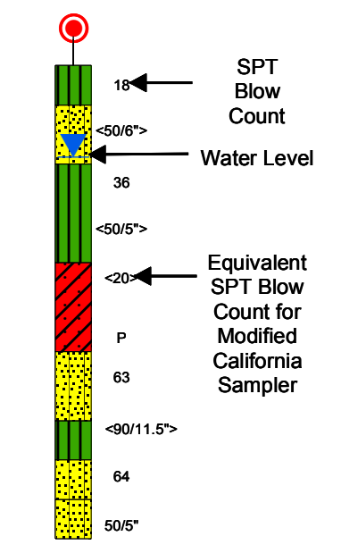


| Zone | Soil Behavior Type | U.S.C.S. |
|---|---|---|
| 1 | Sensitive Fine-grained | OL-CH |
| 2 | Organic Material | OL-OH |
| 3 | Clay | CH |
| 4 | Silty Clay to Clay | CL-CH |
| 5 | Clayey Silt to Silty Clay | MH-CL |
| 6 | Sandy Silt to Clayey Silt | ML-MH |
| 7 | Silty Sand to Sandy Silt | SM-ML |
| 8 | Sand to Silty Sand | SM-SP |
| 9 | Sand | SW-SP |
| 10 | Gravelly Sand to Sand | SW-GW |
| 11 | Very Stiff Fine-grained * | CH-CL |
| 12 | Sand to Clayey Sand * | SC-SM |

*overconsolidated or cemented

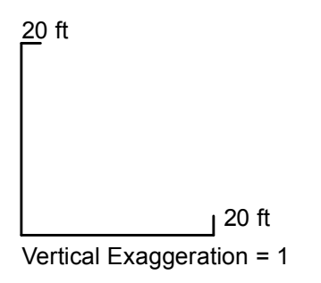
**CPT CORRELATION CHART**

**(Robertson and Campanella, 1988)**



**SOIL BORING LITHOLOGY**

- 🟥 Lean CLAY (CL)
- 🟥 Silty CLAY (CL-ML)
- 🟥 Sandy Lean Clay (CL)
- 🟥 Sandy, Gravelly Lean CLAY (CL)
- 🟥 Gravelly Lean CLAY (CL)
- 🟩 Silt (ML)
- 🟩 Sandy, Gravelly SILT (ML)
- 🟩 Sandy SILT (ML)
- 🟩 Gravelly SILT (ML)
- 🟨 Poorly-Graded SAND (SP)
- 🟨 Poorly-Graded SAND with Silt (SP-SM)
- 🟨 Gravelly Poorly-Graded SAND with Silt (SP-SM)
- 🟨 Gravelly Poorly-Graded SAND (SP)
- 🟨 Well-Graded SAND (SW)
- 🟨 Well-Graded SAND with Clay (SW)
- 🟨 Well-Graded SAND with Silt (SW-SM)
- 🟨 Gravelly Well-Graded SAND (SW)
- 🟨 Clayey SAND (SC)
- 🟨 Gravelly, Clayey SAND (SC)
- 🟨 Silty SAND (SM)
- 🟨 Gravelly Silty SAND (SM)
- 🟨 Poorly-Graded GRAVEL (GP)
- 🟨 Poorly-Graded GRAVEL with Clay (GP-GC)
- 🟨 Poorly-Graded GRAVEL with Silt (GP-GM)
- 🟨 Sandy GRAVEL with Silt (GP-GM)
- 🟨 Sandy GRAVEL (GP)
- 🟨 Well-Graded GRAVEL with Clay (GW-GC)
- 🟨 Well-Graded GRAVEL with Silt (GW-GM)
- 🟨 Sandy Well-Graded GRAVEL (GW)
- 🟨 Sandy, Clayey GRAVEL (GC)
- 🟨 Silty Gravel (GM)
- 🟨 Sandy, Silty GRAVEL (GM)
- ▨ Fill
- ▨ Asphaltic Concrete



Vertical Exaggeration = 1

Humbolt County Public Works Department
Fugro Project No. 04.B3502002
CGI Project No. 10-1949.01

CGI TECHNICAL SERVICES INC.
FUGRO

# CROSS SECTIONS
# STATION 132+00, 146+00, 158+00, 171+00 AND 185+00

Redwood Creek Levee Geotechnical Evaluation Project
Orick, Humboldt County, California

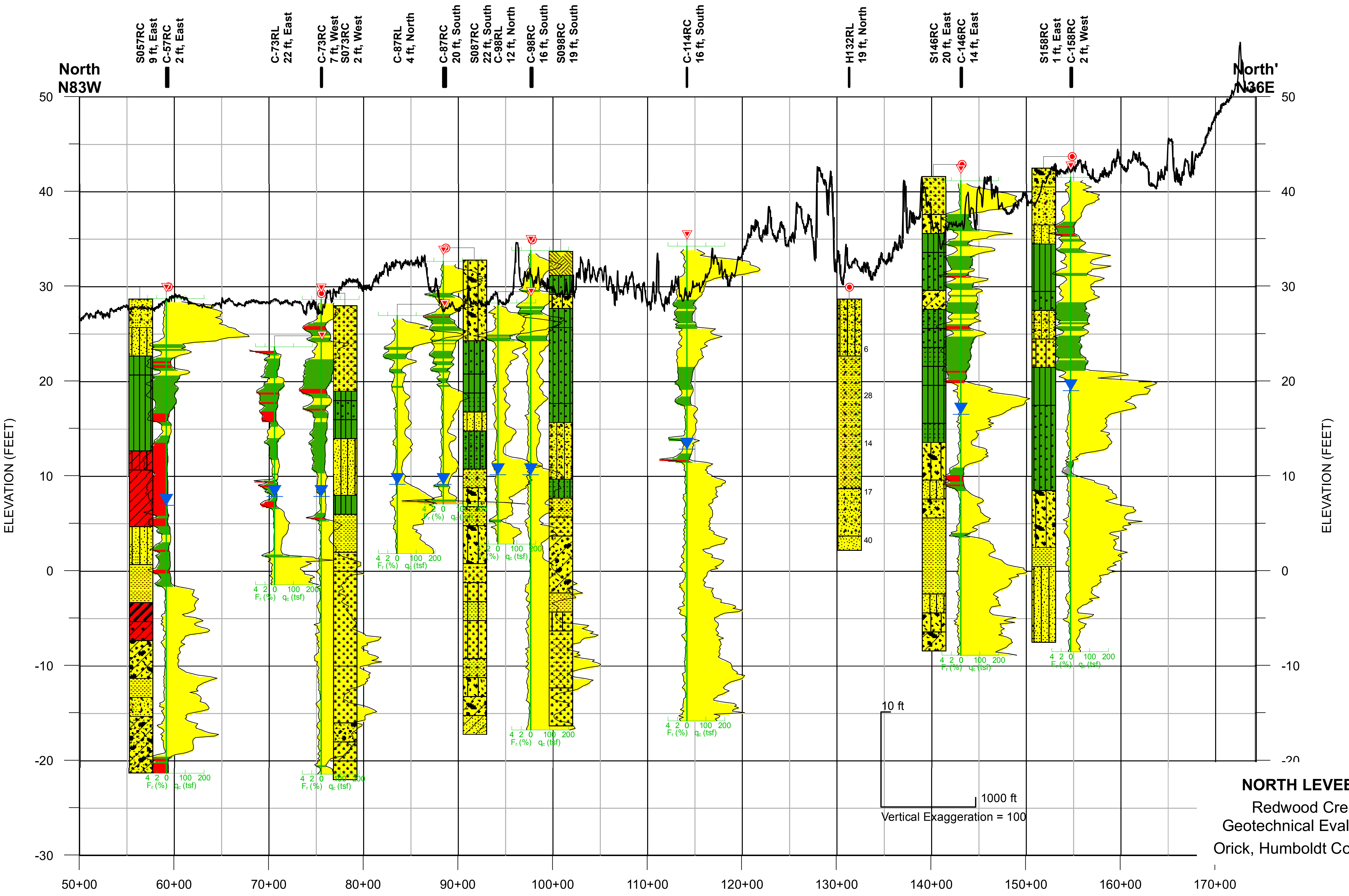
| NO: | DATE: | DESCRIPTION: | DRAWN: | CHKD: | APPR: |
|---|---|---|---|---|---|
| 1 | May 2011 | Levee Cross Sections | DRP | CBD | GE |

PLATE 5-5

Humboldt County Public Works Department
Fugro Project No. 04.B3502002
CGI Project No. 10-1949.01

CGI Technical Services Inc.

FUGRO



## NORTH LEVEE PROFILE

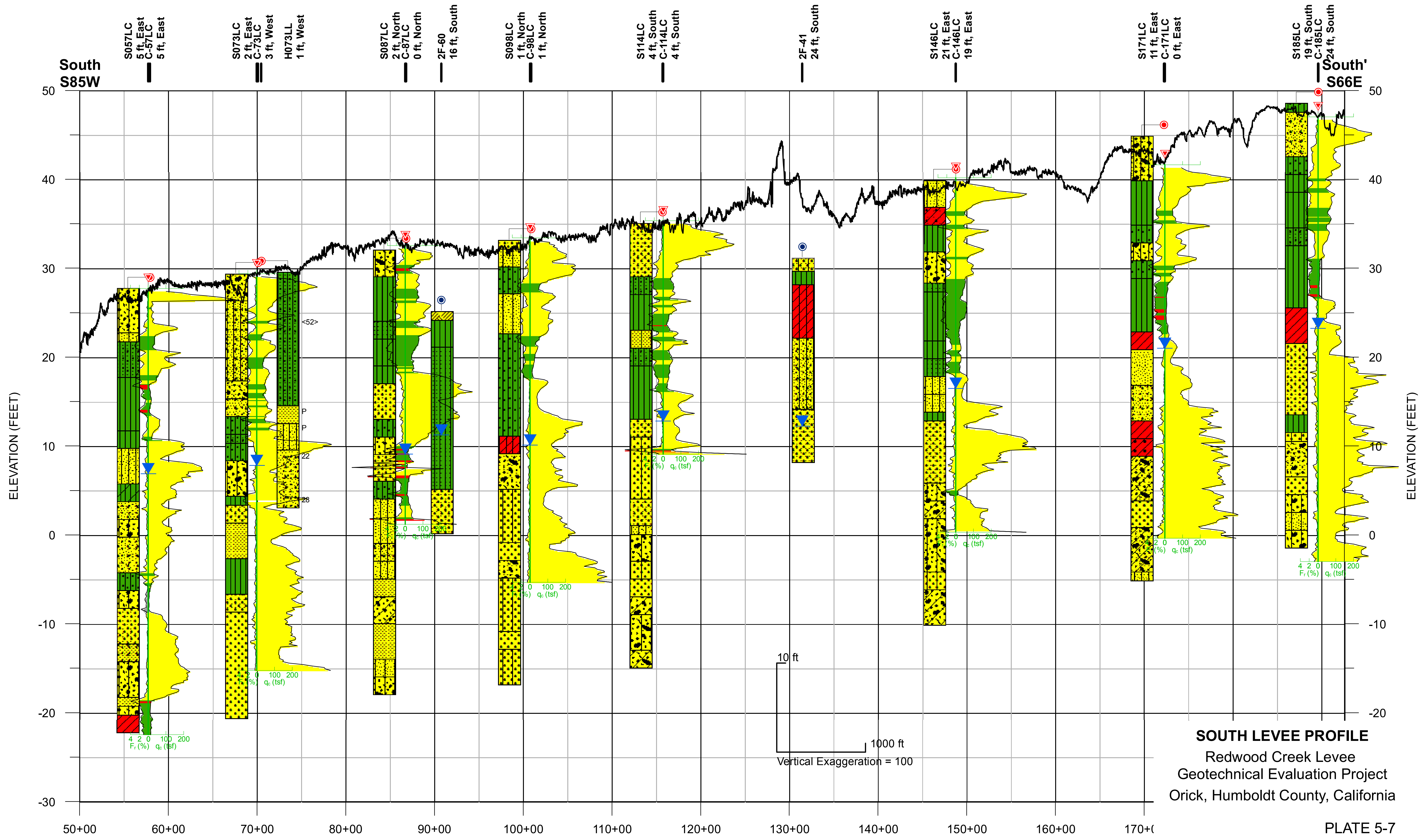
Redwood Creek Levee
Geotechnical Evaluation Project
Orick, Humboldt County, California

PLATE 5-6

N:\Projects\3502_CurryGroup\3502-002_RedwoodCRLevee\Outputs\2011_05_21_GeotechEval\mxd\Plate_5-6_Profile_North.mxd, 05/11/11, CDean

Humboldt County Public Works Department
Fugro Project No. 04.B3502002
CGI Project No. 10-1949.01

CGI Technical Services Inc.



## SOUTH LEVEE PROFILE

Redwood Creek Levee
Geotechnical Evaluation Project
Orick, Humboldt County, California

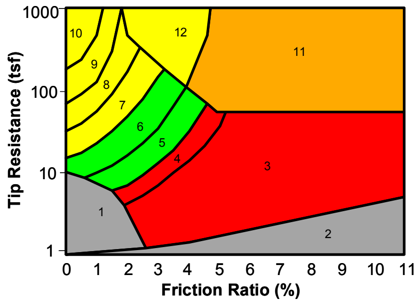
PLATE 5-7

N:\Projects\3502_CurryGroup\3502-002_RedwoodCRLevee\Outputs\2011_05_21_GeotechEval\mxd\Plate_5-7_Profile_South.mxd, 05/11/11, CDean

Humboldt County Public Works Department
Fugro Project No. 04.B3502002
CGI Project No. 10-1949.01

## SOIL TYPES

- Lean CLAY (CL)
- Silty CLAY (CL-ML)
- Sandy Lean Clay (CL)
- Gravelly Lean CLAY (CL)
- Lean to Fat CLAY (CL-CH)
- Fat CLAY (CH)
- Silt (ML)
- Clayey SILT (ML)
- Sandy, Gravelly SILT (ML)
- Sandy SILT (ML)
- Gravelly SILT (ML)
- Poorly-Graded SAND (SP)
- Poorly-Graded SAND with Silt (SP-SM)
- Gravelly Poorly-Graded SAND (SP)
- Well-Graded SAND (SW)
- Well-Graded SAND with Clay (SW)
- Well-Graded SAND with Silt (SW-SM)
- Gravelly Well-Graded SAND (SW)
- Clayey SAND (SC)
- Clayey SAND to Lean CLAY (SC-CL)
- Gravelly, Clayey SAND (SC)
- Silty SAND (SM)
- Gravelly Silty SAND (SM)
- Poorly-Graded GRAVEL (GP)
- Poorly-Graded GRAVEL with Clay (GP-GC)
- Poorly-Graded GRAVEL with Silt (GP-GM)
- Sandy GRAVEL (GP)
- Well-Graded GRAVEL (GW)
- Well-Graded GRAVEL with Clay (GW-GC)
- Well-Graded GRAVEL with Silt (GW-GM)
- Sandy Well-Graded GRAVEL (GW)
- Clayey GRAVEL (GC)
- Sandy, Clayey GRAVEL (GC)
- Silty Gravel (GM)
- Sandy, Silty GRAVEL (GM)
- Fill

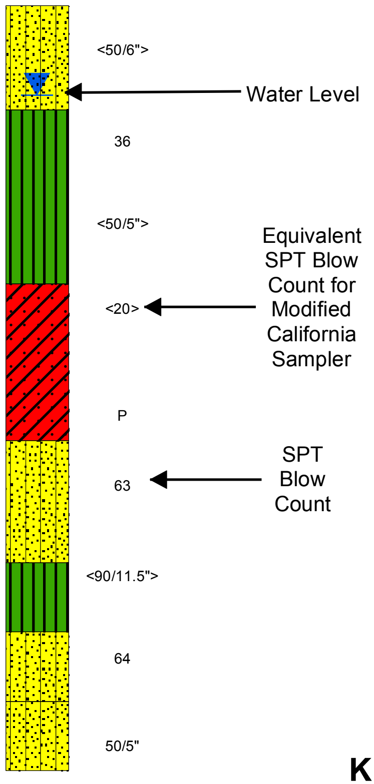


| Zone | Soil Behavior Type | U.S.C.S. |
|---|---|---|
| 1 | Sensitive Fine-grained | OL-CH |
| 2 | Organic Material | OL-OH |
| 3 | Clay | CH |
| 4 | Silty Clay to Clay | CL-CH |
| 5 | Clayey Silt to Silty Clay | MH-CL |
| 6 | Sandy Silt to Clayey Silt | ML-MH |
| 7 | Silty Sand to Sandy Silt | SM-ML |
| 8 | Sand to Silty Sand | SM-SP |
| 9 | Sand | SW-SP |
| 10 | Gravelly Sand to Sand | SW-GW |
| 11 | Very Stiff Fine-grained * | CH-CL |
| 12 | Sand to Clayey Sand * | SC-SM |

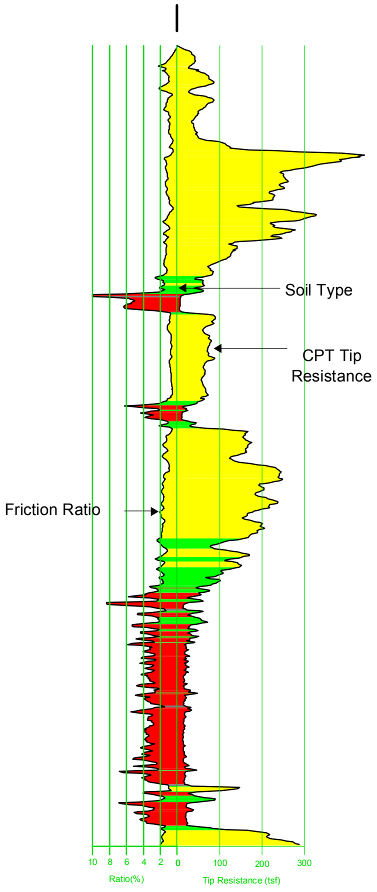
*overconsolidated or cemented

**CPT CORRELATION CHART**

**(Robertson and Campanella, 1988)**



**SOIL BORING LITHOLOGY**

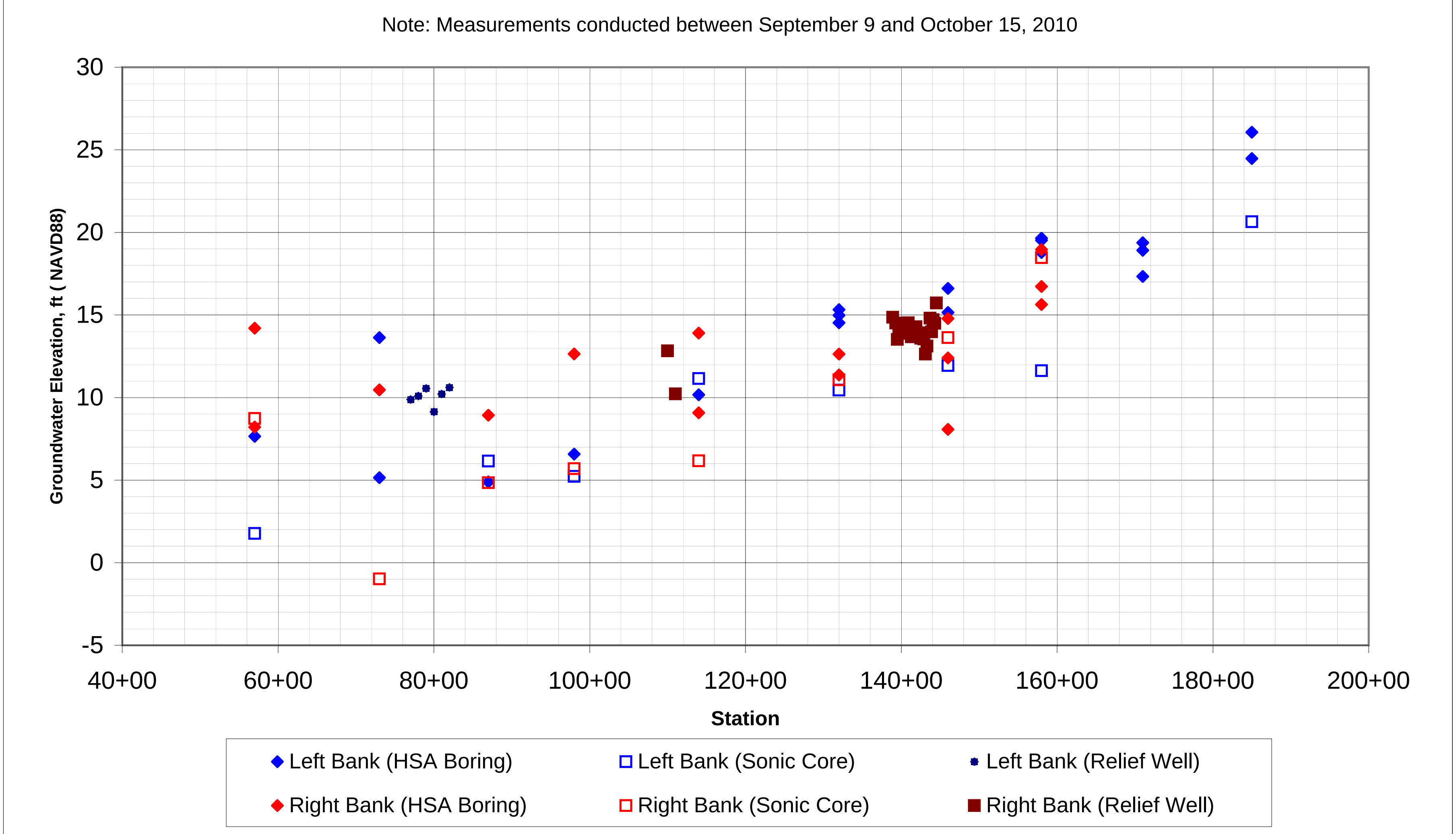


**CPT SOUNDING WITH INTERPRETED LITHOLOGY**

# KEY TO CROSS SECTIONS
Redwood Creek Levee
Geotechnical Evaluation Project
Orick, Humboldt County, California

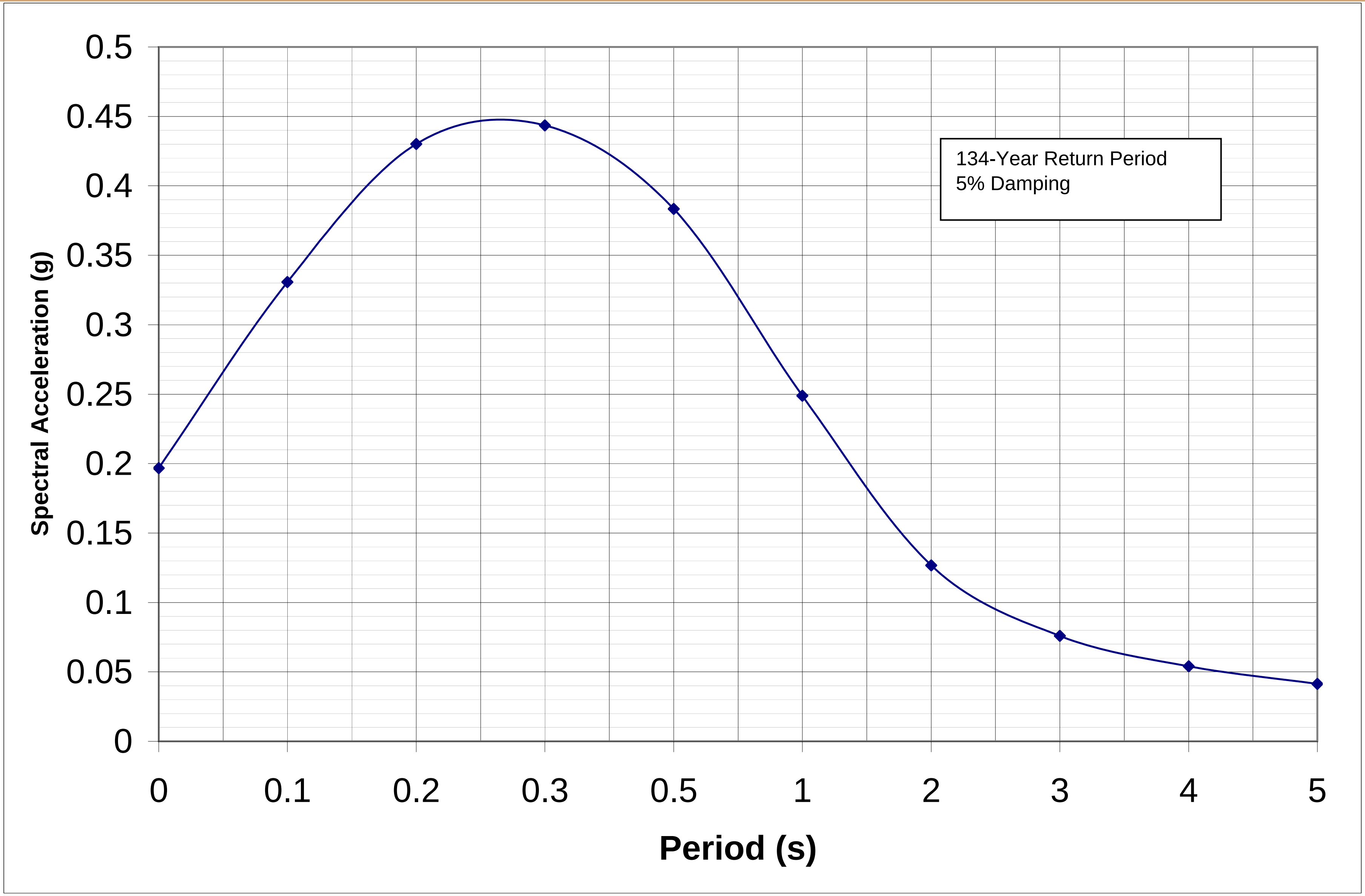
PLATE 5-8

N:\Projects\3502_CurryGroup\3502-002_RedwoodCRLevee\Outputs\2011_05_21_GeotechEval\mxd\Plate_5-8_Xsect_Key.mxd, 05/12/11, CDean



**GROUNDWATER MEASUREMENTS**
Redwood Creeek Levee Geotechnical Evaluation Project
Orick, Humboldt County, California

PLATE 5-9



**PROBABILISTIC SEISMIC HAZARD RESPONSE SPECTRUM**
Redwood Creeek Levee Geotechnical Evaluation Project
Orick, Humboldt County, California

PLATE 6-1