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Geotechnical
Environmental and
Water Resources
Engineering

Upper Sand Creek Detention Basin

Design Report

Submitted to:
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1. Introduction

1.1 Background and Purpose

The Upper Sand Creek Detention Basin Project is a key component of flood protection improvements planned by the Contra Costa County Flood Control and Water Conservation District (District) for the Marsh Creek watershed in east Contra Costa County (Figure 1). The Marsh Creek watershed encompasses portions of the cities of Brentwood and Antioch (Figure 2). Historically, the land use in the watershed has been predominantly cattle ranching and farming. However, since the early 1990's the cities of Brentwood and Antioch have rapidly urbanized. The District is collaborating with the cities on infrastructure needs associated with urbanization to improve flood protection for the residents and businesses in east Contra Costa County.

In 1992 the District developed the Marsh Creek Watershed Plan in order to address a deficiency in flood storage capacity in Marsh Creek. The funding mechanism to implement the Watershed Plan was the formation of Drainage Area 104 (DA 104). New development within the DA 104 boundary is required to contribute to the DA 104 fund, which is earmarked for construction of infrastructure improvements in the watershed. As part of the Marsh Creek Watershed Plan, two sites within the Sand Creek watershed (a tributary of Marsh Creek), named the Upper and Lower Sand Creek Detention Basins, were selected by the District for construction of regional detention facilities.

The Upper Sand Creek Detention Basin is the subject of this report. The project location is on Sand Creek, along the south margin of Lone Tree Valley in Antioch. The proposed basin will be located east of the intersection of Sand Creek and Deer Valley Road. The Sand Creek watershed generally drains from west to east and contributes approximately 15 square miles of drainage area to the Marsh Creek watershed. At the site, the terrain consists of rolling hills along the south and east that give way to flat lands to the west and north. The incised channel of Sand Creek traverses the southern and eastern portions of the proposed basin. The relatively steep creek banks are 15 to 20 feet high. Ground elevations range from a low of about elevation 160 feet (El. 160) at the creek bottom to a high of about El. 330 atop a hill to the east of the basin. The land to the north and west of the creek typically ranges from El. 180 to El. 200.

An existing interim detention basin, built in 1994, occupies part of the site. It is formed by an excavation with the floor at about El. 176 surrounded by a berm at about El. 194. The interim basin is an off-stream facility with a capacity of 123 acre-feet. It receives stormwater from the local area to the north and discharges into Sand Creek through two 36-inch-diameter corrugated metal pipes. Sand Creek currently flows south of the basin, unattenuated.

The Upper Sand Creek Detention Basin will encompass approximately 62 acres and will be formed by construction of a dam across Sand Creek, construction of a saddle dike across a low area along the southern edge of the proposed basin, and excavation within the reservoir to create the necessary storage capacity. A plan of the dam and basin is shown on Drawing C-1. Excavation depths will range from 0 to about 60 feet below existing grade in the basin area.

The basin will be a normally dry reservoir that will attenuate peak runoff by containing (except for the limited release through the low-level outlet) the Sand Creek flows up to the 100-year storm event. During normal periods and smaller rain events, the creek flows will be conveyed through the bottom of the basin in a low-flow channel and will discharge through the ungated (and therefore permanently open) outlet works under the dam. More severe storms will result in creek flows that exceed the inlet-controlled discharge capacity of the outlet works. Creek water in excess of the capacity of the outlet works will then flow into and pond in the basin, and the basin stage will rise. After the peak of the storm has passed, and once the creek flow becomes smaller than the outlet discharge, the water stored in the basin will be passively released back to Sand Creek. For storms greater than the 100-year design storm event, the basin will fill completely and excess flows will pass over the emergency spillway and follow a controlled route to enter the creek downstream of the basin.

The purpose of this Design Report is to summarize the concepts, basis, criteria, and analyses for the design of the Upper Sand Creek Detention Basin. The design is being led by the District, with GEI Consultants Inc. as prime consultant, and Cal Engineering & Geology Inc. and Terra Mater Engineering Services Inc. as subconsultants.

1.2 Integration with City of Antioch Facilities

The Upper Sand Creek Detention Basin is being planned as a multi-purpose project. In addition to its primary flood protection objective, the basin site is designed to be compatible with recreational and environmental purposes. Pursuant to planning discussions between the City of Antioch (City) and the District, the basin floor is proposed to have two primary tiers. The lowest portion of the basin, where the creek will normally flow, will provide an opportunity for environmental enhancement. The higher portion of the basin floor may be developed by the City for sports fields and other recreational uses. While the higher portion of the basin floor would become inundated during large storms, the majority of the time the detention basin will be available for recreational use.

The recreational facilities will be constructed and maintained by the City. The City's General Plan identifies the basin site as a public facilities and sports complex, potentially including several baseball fields, with hard structures such as tennis courts, basketball courts, and parking located along the upper rim of the basin and downstream of the dam.

Based on changes in the City's traffic circulation for the area, the City is currently revisiting the park configuration and it is possible that the recreational facilities may be constructed at a later date.

The City has asked that the dam structure be constructed so as not to preclude the construction of a road to the southern adjacent property (the Camera/Albers properties). An alignment for the possible future access road to properties south of the site has been identified for the portion of road along the basin rim as shown on Drawing C-1. It is anticipated that rough grading for the portion of road that coincides with the basin perimeter maintenance road will be constructed as part of the basin construction.

1.3 Permits and Agreements

Development of permits and agreements is the responsibility of the District. The District is the project sponsor and Lead Agency. The District will construct, own and maintain the detention basin as a flood protection facility. The City of Antioch will design, develop, maintain and operate the park uses within the basin under a license agreement with the District.

The District is in the process of securing additional land rights from property owners to the east (the Williamson property) and south (the Camera/Albers property), as needed to cover the footprint of the ultimate basin.

1.3.1 California Division of Safety of Dams (DSOD) Permit

The basin will constitute a jurisdictional dam and its design and operation will fall under the review and jurisdiction of the California Division of Safety of Dams (DSOD). The District will need to submit an application for construction of the basin to DSOD. The application will include plans, specifications, supplemental technical information, and filing fees. The District has already paid the first installment of the required filing fees. DSOD will independently analyze and evaluate the plans and specifications and will need to grant approval in writing before the District may proceed with construction. DSOD will inspect and evaluate the dam and basin during construction to verify compliance with the approved plans and specifications and to assure that changes or unforeseen foundation conditions are recognized and the design is modified as necessary. Upon completion of construction, DSOD will issue a Certificate of Approval upon finding that the dam and reservoir are safe to impound water within the limitations prescribed in the certificate. Subsequently, DSOD will continue to be involved through the life of the facility through their authority to perform annual inspections and to monitor and evaluate the dam as necessary to assure safety.

DSOD generally requires evidence of water rights before they can approve a construction application. The District has received a determination from the State Water Resources

Control Board that, because of the short-term retention provided by this facility, no water rights permit will be required. This information will be shared with DSOD as part of the formal application. DSOD will also require information to enable it to comply with the California Environmental Quality Act and regulations as a responsible agency. A copy of the adopted Final Environmental Impact Report (EIR) will need to be submitted. The EIR was prepared for the Sand Creek watershed in 1992 and the District is updating CEQA documentation to address any changes to the design since that time.

1.3.2 Environmental Permits

The District will obtain environmental permits for the project. Participation by the District in the East Contra Costa County Habitat Conservation Program (ECCHCP) is anticipated to streamline the permitting process. A Stream Alteration Agreement with the Department of Fish and Game will be needed. Demonstration of compliance with California Endangered Species Act (CESA), Federal Endangered Species Act (ESA) and Section 402 of the Clean Water Act through a National Pollution Discharge Elimination System (NPDES) permit will also be needed. Additionally, the District will secure a Section 404 Corps of Engineers permit (including a Section 7 consultation for California Red Legged Frog following the guidelines of the ECCHCP) and Section 401 Water Quality Certification.

The District intends to recreate creek habitat within the basin along the low-flow section of the creek, including a geomorphic design for the stream with compatible landscaping for restoring creek habitat. The environmental enhancement will provide impact mitigation, valuable habitat for local wildlife, an aesthetic feature for the sports complex, and a restoration project that can be used by the District to determine sustainable creek concepts.

1.3.3 Other Permits

Local permits may include a County or City grading permit, as well as haul permits from the City of Antioch for any major haul operations on City roadways.

1.4 Earlier Studies

1.4.1 Design Basis Memorandum and Field Exploration Work Plan

In May 2009, the project team prepared a Draft Design Basis Memorandum to document the design basis and concepts for the detention basin and to present the criteria and standards to be used in the design of the dam. The project team also prepared a Draft Work Plan for Foundation Investigations to outline supplemental geotechnical investigations required to support the geotechnical analyses and detailed design studies for the dam and basin. Both of these documents were submitted to DSOD for review and comment. DSOD provided review comments which are included in Appendix 1.1. DSOD's review comments were incorporated in subsequent design activities. Proposed responses to DSOD's comments were prepared and are also included in Appendix 1.1.

1.4.2 Optimization of Historic Design Concepts

The conceptual design for the basin was initially described in the following draft reports:

- Contra Costa County Flood Control and Water Conservation District, *Hydrology and Hydraulics, Sand Creek Watershed, Study of Upper Sand Creek Basin, Lower Sand Creek Basin, and Sand Creek Hydraulics*, January 8, 2008.
- Fugro West, Inc., *Geotechnical Study, Upper Sand Creek Detention Basin, Old Sand Creek Road, Antioch, California*, June 2004.
- William Lettis & Associates (WLA), *Sand Creek Floodwater Detention Basin Engineering Geologic Investigation*, March 2004.

At the District's request, DSOD performed an initial review of the conceptual design described in the Fugro and WLA 2004 reports and provided draft review comments. DSOD's review comments are included in Appendix 1.2. As part of the current design effort, the project team reviewed the basis for the conceptual design of the flood control basin before starting the field investigations and detailed design activities. The general design criteria established in prior District studies were verified or updated in coordination with the District, and proposed responses to DSOD's comments were prepared. These are also included in Appendix 1.2.

As part of the initial optimization effort, the project team reviewed the conceptual design and identified design refinements to improve the basin's function, reduce the ultimate cost of the facility, and address the expressed DSOD concerns. The following paragraphs summarize the project team's recommendations that were implemented as part of the optimization of the Upper Sand Creek Basin design:

(1) The basin site and design capacity presented in the conceptual documents have remained essentially unchanged. However, refinements were proposed in the arrangement of the dam and appurtenant structures. The conceptual design for the dam considered an alignment along the route of a possible future access road to adjacent properties located south of the site. The dam/road was aligned in a southeast direction along the creek. The alignment straddled bedrock knobs and repeatedly crossed the creek channel and young alluvial sediment deposits on the creek bottom. (The site geology, foundation conditions and anticipated foundation treatment needs are described in Sections 4 and 5 of this report, respectively.) It

was judged that this variable foundation conditions along this alignment would introduce the potential for differential settlements that would need to be addressed through increased foundation excavation and backfilling requirements. Also, the foundation excavation for the eastern reach of the embankment would be nearly parallel to the general trend of the hillside and would tend to undermine it. The foundation excavation and treatment for the conceptual alignment would likely entail larger excavation and embankment volumes and present geotechnical concerns. Consequently, the alignment of the dam was revised based on geotechnical considerations. The main embankment alignment will be nearly perpendicular to the creek and will abut the hillside just to the southeast of the creek crossing (see Drawing C-1). A small saddle dike will be needed at the southeast corner of the Basin to avoid loss of containment to the southeast, but the proposed dam and dike alignment are expected to reduce the required foundation excavation and filling and to reduce geotechnical concerns related to the adjacent hill. The proposed embankment design is described in Section 6 of this report.

(2) The conceptual design for the basin originally included three outflow structures located along the eastern side of the basin:

- A low-level outlet, consisting of 1,700 feet of 42-inch-diameter reinforced concrete pipe (RCP), located under the right abutment of the dam (east of the creek) (referred to as the "Primary Spillway" in the January 2008 Hydraulics and Hydrology Report).

- A mid-level service spillway located in the left abutment adjacent to the creek channel consisting of an ungated weir/ vault connected to two 275-foot-long 96-inch-diameter RCPs passing under the dam and discharging to the creek (referred to as the "Emergency Spillway" in the January 2008 Hydraulics and Hydrology Report).

- A high-level emergency overflow spillway weir formed at a future parking area and road along the east side of the basin, discharging along the downstream slope and toe of the embankment and along a future road and into the creek (referred to as the "Basin Overflow Emergency Spillway" in the January 2008 Hydraulics and Hydrology Report).

It was anticipated that the excavation for the low-level outlet as originally proposed would cut into the alluvial terrace at the toe of an ancient landslide and possibly into the slide mass itself. In addition, the length of pipe (about 1,700 feet) would make construction very costly. Therefore, as described later in this report, the low-level outlet was relocated to the left abutment to reduce the length of pipe to approximately 350 feet and to reduce the potential for undermining and reactivating the ancient slide by the pipeline excavation. Additionally, the mid-level service spillway was eliminated and its functions were split between the low-level outlet and the emergency overflow spillway. Lastly, the emergency overflow spillway was relocated to the left abutment, with the same crest length and elevation as the original conceptual design. The proposed design for the outlet works and spillway is described in Section 7.

The results of the above-described initial optimization effort were incorporated in the design and were described in the above referenced May 2009 Design Basis Memorandum.

1.5 Project Schedule

In summary, planned project activities include the following:

- Design and Permitting: Through November 2010
- Bidding and Award: Winter 2010/11
- Construction of Dam and Appurtenant Structures: Late Spring and Summer 2011
- Completion of Park Facilities: As needed by City of Antioch

Project costs depend on the District's ability to partner with third-party entities to excavate and remove soil material from the basin for the third parties' construction needs, thereby reducing the volume and cost of excavation required to complete the grading of the basin. If third-party needs and demand for borrow fill decrease or do not materialize due to general economic conditions, the District could temporarily defer some of the excavation in the basin until the demand for borrow fill materials picks up again. Any deferred or phased excavation would be located away from the embankment dam.

2. Hydrology

The hydrologic basis for the project design was described in the draft report entitled *Hydrology and Hydraulics, Sand Creek Watershed, Study of Upper Sand Creek Basin, Lower Sand Creek Basin, and Sand Creek Hydraulics*, dated January 8, 2008, prepared by the Contra Costa County Flood Control and Water Conservation District. This report is incorporated by reference. Key conclusions from the report included the following:

- The drainage area of the Upper Sand Creek Detention Basin is 11.0 square miles.

- The reservoir storage capacity was selected to store the runoff from the 100-year 12-hour storm with no or minimal spillway overflow. Based on the assumption that the low-level outlet works is ungated and will remain open throughout the storm, hydrologic modeling indicated that the most critical storm duration for both peak discharge and runoff volume would be the 12-hour storm. Accordingly, the proposed design was based on the 12-hour storm.

- For the 100-year 12-hour storm, the total rainfall was estimated to be 3.34 inches, the total loss was estimated to be 1.64 inches, and the total excess was estimated to be 1.70 inches. The peak inflow was estimated to be 2,818 cfs. The runoff volume was estimated to be 1,324 acre-feet.

The 100-year design storm was routed through the reservoir using the updated stage-storage curve for the reservoir and discharge rating curve for the outlet. The key results are as follows:

- A reservoir capacity of approximately 900 acre-feet below the emergency spillway crest at El. 191.0 is needed to route the flow from the 100-year storm event through the outlet works and limit the peak reservoir stage to the level of the spillway crest thus avoiding significant overflow. The reservoir was assumed to be empty at the beginning of the storm.

- The peak outflow from the outlet works is 134 cfs with maximum reservoir level at El. 191.0.

The key considerations for emergency spillway design are described as follows:

- Based on initial studies and coordination between the District and DSOD, the District selected the 1,000-year 12-hour-storm event as the initial spillway design flood. For this storm event, the total rainfall was estimated at 5.11 inches, the total loss was estimated at 1.74 inches, and the total excess was estimated at 3.37

inches. The runoff volume was calculated to be 2,305 acre-feet. The peak inflow from the 1,000-year 12-hour-storm event was calculated to be 4,635 cfs. This hydrograph was routed through the reservoir using the updated stage-storage curve for the reservoir and discharge rating curves for the outlet and spillway. The discharge rating curve for the spillway was developed assuming a broad-crested weir. A residual freeboard not less than 1.5 feet was required. The peak outflow from the emergency spillway resulting from the 1,000-year storm event was approximately 2,500 cfs. The reservoir was assumed to be empty at the beginning of the storm.

- Based on further review by DSOD, as communicated to GEI verbally on July 2, 2010, DSOD also has the following additional requirements. Based on DSOD's longstanding hydrologic policy (Calzascia and Fitzpatrick, undated) and a Total Class Weight of 26, the 1,000,000-year 72-hour storm needs to be routed through the reservoir without overtopping the dam. For the selected spillway crest length of 225 feet the estimated peak reservoir stage is El. 195.5. As a minimum the dam crest needs to be at an elevation of 195.5 feet to pass this design storm without overtopping. Residual freeboard is not required for this storm event. The peak inflow and outflow were computed to be approximately 5,700 cfs.

- In summary, the emergency spillway hydraulic design criteria based on the above requirements are as follows:

    - Spillway invert: El. 191.0
    - Peak stage for the 1,000,000-year 72-hour storm: El. 195.5
    - Spillway width: 225 feet
    - Peak outflow: 5,700 cfs at a peak stage of El. 195.5

3. Detention Basin Geometry

The basin geometry and boundaries are based on storage requirements, land ownership constraints, geotechnical considerations, and coordination with the City of Antioch regarding objectives for the recreational facilities and property access. A plan of the basin is shown on Drawing C-1, and cross-sections are shown on Drawings C-3, C-4, and C-5. Key criteria for design of the basin are as follows:

(1) The storage capacity of the basin is 900 acre-feet below the crest of the emergency spillway to provide routing of the 100-year 12-hour storm event with minimal or no discharge over the emergency spillway. Total storage capacity at impoundment crest elevation (El. 195.5) is 1,106 acre-feet. The reservoir area-capacity curves are shown on Drawing C-1.

(2) Two distinct areas are planned for the basin:

- A lower area, where the creek flows will normally occur, will provide environmental enhancement opportunities. This area is located along the southern and southeast portions of the basin and includes a low-flow channel to carry flow from the creek through the basin during normal flow periods.
- A slightly higher area is proposed in the central and northern portions of the basin to potentially include playing fields and other future recreational uses.

Including the volume within the low-flow channel, the storage capacity below the rim of the lower area, above El. 166, is approximately 35 acre-feet. This corresponds to approximately a 2-year storm level (50% annual chance of occurrence storm event), which represents the expected frequency of inundation of the central and northern portions of the basin where the playing fields would be located.

(3) Within the lower area, the basin floor will be at elevations ranging from approximately El. 165 at the southwest (upstream) corner to El. 159 at the east (downstream) corner. A low-flow channel will be provided to carry creek flows, with channel bottom ranging from El. 162.5 at the basin inlet structure to El. 157.0 at the dam outlet works. The length of the low-flow channel is approximately 2,985 feet.

(4) The basin will be developed by excavation. Except for localized areas with unique needs, basin slope inclinations will generally be as follows:

- North of creek: 4 horizontal to 1 vertical [4(H):1(V)], for public safety and access considerations, or flatter if required by geotechnical considerations.

- South of creek: Slopes will be 3(H):1(V) except in the area adjacent to the basin inlet structure where the slopes are 2.5(H):1(V). Slopes are selected based on geotechnical considerations.

(5) A 20-foot-wide clearance has been provided between the western edge of the basin slope and the west property line. A maintenance road will be located within this 20-foot corridor.

(6) The north (downstream) edge of the dam crest is fixed along the north property line, and the dam downstream slope will extend onto the adjacent property. As stated above, the dam crest will be at El. 195.5. It is anticipated that an extension of Sand Creek Road will eventually be constructed by others to the north of and adjacent to the property line, at approximately El. 195.

(7) In addition to the roadway along the dam crest, the District will have a continuous perimeter access roadway along the perimeter of the basin at approximately El. 195. Portions of the perimeter roadway will be shared with public roadways, as well as with internal circulation supporting future park users. Ramps to the basin bottom, as well as to the primary spillway inlet, will be provided at locations most compatible with public and maintenance access to the park. Ramp locations are shown on Drawing C-1.

(8) The portion of the perimeter access roadway along the east side of the basin will be shared with a future roadway providing access to properties south of the basin. The rough-graded bench for the future roadway will be constructed as part of the basin. The City has indicated that a 40- to 60-foot-wide roadway with a design speed of 35 miles per hour is contemplated. This requires a minimum curve radius of 475 feet and superelevation of 2 percent. For rough-grading purposes, a 60-foot-wide bench at approximately El. 195 will be constructed as part of the basin, with the District's perimeter access road located along the basin-side of the bench. Fine grading and superelevation of the future roadbed will be addressed by the City at the time the future roadway is installed.

4. Geology, Seismicity, and Foundation Conditions

4.1 General

Available information on the geology at the project site is described in the following reports which are incorporated by reference:

- Cal Engineering & Geology (CE&G), draft *Geotechnical Data Report, Upper Sand Creek Detention Basin, Deer Creek Road, Antioch, California*, Contra Costa County Flood Control & Water Conservation District, May 2010.

- Fugro West, draft *Geotechnical Study, Upper Sand Creek Detention Basin, Old Sand Creek Road, Antioch, California*, June 2004.

- William Lettis & Associates (WLA), *Sand Creek Floodwater Detention Basin Engineering Geologic Investigation*, March 2004 (published as an appendix to the Fugro 2004 report).

Fugro, in conjunction with WLA, performed a preliminary geotechnical engineering study which included geology mapping and a field exploration program consisting of 7 cone penetrometer tests (CPTs), 15 exploratory borings (8 hollow-stem auger and mud-rotary borings by Fugro and 7 mud-rotary core borings by WLA), 5 test pits, and 3 exploratory trenches. The Fugro report incorporates the findings from the WLA investigation.

CE&G performed supplemental geotechnical explorations which included geology mapping and a field exploration program consisting of 14 exploratory borings, 33 test pits, and one exploratory trench. CE&G's May 2010 *Geotechnical Data Report* encompasses the geologic and geotechnical data obtained from earlier studies. Selected figures from the CE&G report are attached herein for easy reference. A geologic plan with subsurface investigation locations and geologic cross-sections are presented in CE&G's Figures 8 through 15. Subsurface profiles showing simplified representations of soils encountered in the soil borings performed along the dam alignment are presented in Drawings S-2 through S-10, also attached herein.

Key findings from these studies are summarized in this section.

4.2 Regional Geology

The project site is located in the Coast Ranges geomorphic province of California. The Coast Range geomorphic province is characterized by northwest-southeast trending mountain ranges and intervening valleys. The right-lateral strike-slip San Andreas Fault system controls the northwest-southeast structural grain of the Coast Ranges and the Bay Area. The San Andreas Fault system includes the Hayward-Rodgers Creek, Calaveras, Concord-Green Valley, and Greenville-Marsh Creek faults, among others. The fault system marks the major boundary between two of earth's tectonic plates, the Pacific Plate on the west and the North American Plate on the east. The Pacific Plate is moving north relative to the North American plate at approximately 32 mm/yr in the Bay Area (USGS, 2003). The plate movement is being accommodated mostly by the San Andreas Fault, but it is also being distributed along the other faults within the San Andreas system. Fault movements have been ongoing for about the last 25 million years.

In general, the Coast Ranges are composed of sedimentary bedrock interspersed with volcanic rocks. Nearer the surface, layers of alluvium fill the intervening valleys. The project site is situated within the Diablo Range of the Coast Ranges geomorphic province (Page, 1992). This portion of Contra Costa County is comprised of a complex sequence of Mesozoic and Cenozoic age sedimentary and volcanic rocks. Along the northeastern edge of the Diablo Mountains, the bedrock consists of Tertiary marine sedimentary rocks of the Kreyenhagen Formation including sandstones, shales and mudstones. The bedrock materials comprising this portion of the Diablo Range have been extensively folded and faulted as a result of regional tectonic forces. As a consequence, geological relationships are often complex, and individual bedrock units are locally tightly folded, faulted, sheared, and overturned.

4.3 Site Geology

The geology of the area has been mapped by Dibblee (2005), Crane (1995), Nilsen (1975), and Helley and Lajoie (1979). Dibblee and Crane map the site as alluvium in the lower reaches and Markley sandstone in the hillside portions (Figure 6, CE&G, 2010). Helley and Lajoie map the lower reaches as containing late Pleistocene alluvium described as weakly consolidated, poorly sorted, and irregular interbedded clay, silt, sand, and gravel. The deposit ranges between 35,000 and 70,000 years old with an unknown thickness.

Based on field mapping by CE&G, Fugro and WLA, surficial geologic units at the project site consist of quaternary alluvial terrace deposits and recent alluvium, underlain by the Markley Sandstone formation (see geologic map on Figure 8 of CE&G and Drawing S-1). Two distinct alluvial terrace deposits have been mapped. A higher late

Pleistocene-early Holocene terrace is coincident with the floor of Lone Tree Valley (assigned unit Qt_2), and a discontinuous series of lower inset Holocene terraces (assigned unit Qt_1) is apparent along Sand Creek. The terraces do not show evidence of fault displacement or deformation.

The active Holocene channel of Sand Creek is incised between about 15 and 25 feet below the top of the terrace surface, and in places is bounded by discontinuous Qt_1 Holocene terraces that are about 3 to 6 feet above the floor of the channel. Recent alluvium deposits (assigned unit Q_{al}) form a thin ribbon within and immediately adjacent to the active Sand Creek channel.

Bedrock underlies the rounded hills that rise above the south side of Sand Creek and is exposed in the creek banks. South of the creek, bedrock is covered by a veneer of topsoil and colluvial soil, and was encountered in the exploratory trenches and core holes at depths of between about 5 and 15 feet. North of the creek, a planated bedrock surface underlies the alluvial terrace deposits at depths ranging between about 40 and 75 feet below the terrace surface (based on Fugro and CE&G borings). Bedrock at the site consists of Markley Formation interbedded sandstone, shale, and claystone that consistently dips at shallow inclinations between about 10 and 30 degrees to the north/northeast. Most of the exposed bedrock consists of shale and claystone, with a lower percentage of sandstone. Sandstone beds tend to be harder and more competent, and generally form steep resistant ledges. In general the rock is relatively soft and weak and is expected to break down into a fine-grained soil when excavated, handled, and compacted with heavy earth-moving equipment.

Several small displacement bedrock faults were mapped by WLA in the stream bank exposures along the south margin of Sand Creek. All of these faults appear to be confined to bedrock and do not visibly offset overlying colluvial soils in natural exposures or in the exploratory trenches excavated by WLA.

The U.S. Geological Survey and the California Geological Survey have developed regional landslides features maps for a significant portion of the greater San Francisco Bay Area. The landslide features map for the greater Antioch area (Nilsen, 1975) shows a moderate size ancient landslide feature near the eastern boundary of the project site (Figure 7, CE&G, 2010). This mapping is consistent with observations of the site and the information developed from CE&G (2010) and WLA exploratory borings and test pits. Other geologic features mapped in the site vicinity include shallow active landslides on the bedrock slope above Sand Creek that forms the southern edge of the Basin.

4.4 Surficial Soils

The surficial soils in Contra Costa County have been mapped by U.S. Department of Agriculture Natural Resources Conservation Service (NRCS) (2008). The NRCS map indicates that the project site is underlain by four types of surficial soils as shown on the attached Figure 5 of CE&G (2010). All of them are clay soil types. The soil distribution and key characteristics described in the referenced NRCS report are summarized as follows:

- The Capay Clay occurs on 0 to 2 percent slopes located in the northwest portion of the site. The Capay Clay Series soils classify as low plasticity clays (CL) and have moderate shrink-swell potential. These soils are described with a liquid limit range of 30 to 50 percent and a plasticity index range of 10 to 30 percent. Soil permeability is very slow and the soil presents little to no hazard of erosion.

- The Altamont-Fontana Complex occurs on 30 to 50 percent slopes located in the southwestern and eastern portions of the site. The Altamont-Fontana Complex Series soils classify as low plasticity clays (CL) and have low to moderate shrink-swell potential. These soils are described with a liquid limit range of 30 to 50 percent and a plasticity index range of 10 to 30 percent. Soil permeability is very slow to moderate and the soil presents a low hazard of erosion.

- The Pescadero Clay Loam is mapped on the floor of the alluvial valley in the southeastern corner of the basin. The Pescadero Clay Series soils classify as low to high plasticity clays (CL/CH) with a low to moderate shrink-swell potential. The soils are described with a liquid limit range of 30 to 65 percent and a plasticity index range of 10 to 40 percent. Soil permeability is very slow and the soil presents little to no hazard of erosion.

- The Rincon Clay Loam located in the remaining portions of the site and covers the majority of the site area. The Rincon Clay Series soils classify as low to high plasticity clays (CL/CH) and silty clay loam (SC) and have moderate shrink-swell potential. The Rincon Series soils have been described with a liquid limit range of 25 to 60 percent and a plasticity index range of 10 to 35 percent. Soil permeability is very slow to slow and the soil presents little to no hazard of erosion.

This mapping is consistent with the results of previous and current supplemental exploration and testing.

4.5 Potential Geologic Hazards

4.5.1 Antioch-Davis fault

Two segments of the Antioch-Davis fault are mapped as crossing the western portion of the site. The fault is a prominent bedrock fault that is mapped with a north-south trend across the footprint of the detention basin. Although formerly zoned as an active fault by the California Geologic Survey (CGS), the fault has been reevaluated by CGS and dezoned on the basis of a lack of evidence for Holocene activity. Additional recent studies, including three exploratory trenches by WLA in September 2002, support the conclusion that the fault has not been active during the Holocene era and likely has not experienced Quaternary activity. Based on the Fugro and WLA evaluation of the Antioch-Davis fault, the fault is not considered an active structure that could pose a surface rupture or earthquake source hazard to the detention basin.

4.5.2 Landslides

Based on the available information from the field investigations made by WLA and CE&G, an inactive, deep-seated ancient bedrock landslide exists on the hillslope above and under the east (right) abutment of the proposed main dam, see Figure 8 of CE&G (2010). This slide is an ancient translational failure to the north along a bedding plane. It is interpreted to have occurred along a claystone bed when the creek incised a deep channel into bedrock before the Qt_2 alluvial terrace was deposited. The geologic studies by WLA and CE&G indicate that subsequently the slide has been substantially modified by erosion, and the toe has been buttressed by a portion of the Qt_2 alluvial terrace. The Qt_2 terrace is undeformed by the slide, and shows that this slide has not been active for many thousands of years.

In 2002, WLA trenched along the west margin of the ancient landslide and near the toe of the ancient landslide in the proposed dam's right abutment area (Trenches T-2 and T-3 respectively on Figure 8 of CE&G). No failure planes were observed in the trench wall at the margin of the landslide, and the base of the slide appeared to be below the bottom of trench T-2. No evidence of recent-appearing movement of the ancient landslide was observed in the trench. In trench T-3 near the dam's right abutment and toe of the slide, the colluvium and alluvium appeared to be massive without significant structure, and showed no evidence of landslide displacement or deformation.

According to the WLA and CE&G studies, the post-failure profile of the slide is at a relatively low angle (about 10 degrees), and at a stable inclination. The recovered core and borehole televiewer log from a boring drilled by WLA in the center of the slide mass showed that the landslide failure plane is an irregular, apparently partly-bonded rock-rock contact that at least locally does not contain softened gouge. This suggests that the slide failure plane is partly "healed." Boring B-5-09 drilled by CE&G in the right abutment of the dam showed a similar contact at a depth of 50.5 feet, and boring B-6-09, drilled above the right abutment, showed a thin zone of silty sand above the sandstone bedrock at the interpreted contact at a depth of 47 feet. Based on the available borings, the overlying alluvium and disturbed rock mass is tight. The bedrock below the slide plane is sound and massive with tight fractures. A stability evaluation by CE&G (Appendix 2.1) using a residual friction angle of 10 degrees confirms that, in its current configuration buttressed by the Qt_2 alluvial terrace, the abutment has an ample safety factor and is thus less susceptible to sliding displacements than at the time when the slide was triggered.

Several smaller and shallower, more active landslides are present in the hillslope above Sand Creek in the area of the proposed cut slopes bounding the south margin of the detention basin (Figure 8 of CE&G). These slides include shallow erosion gullies, rotational slumps and translational slides in colluvial soils and weathered bedrock that are up to about 15 feet deep and typically toe-out above the creek channel. Also, areas of shallow raveling were observed in steep bedrock cuts and slopes adjacent to the creek channel, partly in response to creek incision. The proposed detention basin grading, shown on Drawings C-1 in plan and C-3, C-4 and C-5 in section involves the placement of a fill slope, the top of which will be near the toe of the existing landslides, and which will buttress the existing slope below the landslide. A 20-foot-wide bench at El. 195 and a shallow 3(H):1(V) cut above the bench will trim the face of the existing landslide and colluvium slope above the landslide. Any slide mass that remains under the cut slope will be overexcavated and replaced with drained, compacted fill. An evaluation and stability analysis of these shallow features is in Appendix 2.1.

4.5.3 Liquefaction

The site is underlain by predominantly fine-grained alluvium consisting of stiff to hard clay and silty clay layers. In general, the clayey soils are typically not susceptible to soil liquefaction. Relatively thin layers of medium dense silty sand and clayey sand exist within the dam foundation. The depositional environment for the alluvial plain region of the Lone Tree Valley is one of small streams (like Sand Creek) draining eastward from the Coast Range foothills to the west. Flood flows of these streams spread out from the hills depositing fine-grained sediments. Stream flows deposited intermittent sand beds that tended to stack upon each other causing the few occurrences of thicker sand beds (The Source Group, 1999).

GEI, and earlier Fugro (2004), have conducted liquefaction evaluations for soil borings where deposits of primarily cohesionless materials were logged. The thickness of potentially liquefiable layers ranged between 1 and 12 feet. The liquefiable layers are typically interbedded with non-liquefiable soils and do not correlate well between borings. GEI's analysis is included in Appendix 2.5. Based on SPT test data from borings, the magnitude of the dam's earthquake-induced settlement is estimated to be less than 1 inch for a 50th percentile earthquake event and on the order of 1.5 to 6 inches for a 84th percentile earthquake event.

4.5.4 Expansive Soils

The project site is located on alluvial soils primarily consisting of stiff to hard clay of low to medium plasticity. The soils are likely to have moderate shrink-swell potential. The potential for soil volume changes has been considered in the design of the project structures.

4.6 Seismicity

4.6.1 Active Faults

Seismicity at the project site was originally described in the Fugro 2004 report. As requested by DSOD (Appendix 1.1, Comment 9), GEI has updated the seismicity study performed by Fugro to review and incorporate nearby fault sources identified as potentially active based on information in the document titled, *Delta Risk Management Study (DRMS), Phase 1, Seismology Memorandum*, dated June 2007, prepared by URS and Jack R. Benjamin Associates, Inc. (URS, 2007). The *Delta Risk Management Study (DRMS), Phase 1, Seismology Memorandum* will be referred to as the "2007 Seismology Memorandum."

The deterministic fault sources having the greatest contribution to site seismicity, along with their source magnitude and distance from the site, were selected by reviewing seismic hazard deaggregation results for the project site using the USGS's Interactive Deaggregation computer program website (http://eqint.cr.usgs.gov/deaggint/2008). The Next Generation Attenuation (NGA) models were then applied to these fault sources to determine the 84th and 50th percentile peak ground motions for each source. In addition to these sources, a zone of seismicity, referred to as the Montezuma Source Zone was evaluated based on findings from the 2007 Seismology Memorandum, as discussed further below.

The site is located in a seismically active area of the San Francisco Bay Area. The locations of major active faults are shown on the fault map (Figure 3). The regional seismicity is dominated by potential seismic activity of the major faults related to the San Andreas fault zone.

The San Andreas fault zone includes numerous faults found by the California Geologic Survey (CGS) under the Alquist-Priolo Earthquake Fault Zoning Act to be "active", i.e., Holocene seismic activity (past 10,000 years). The latest maps issued by CGS in conformance with the Alquist-Priolo Earthquake Fault Zoning Act do not indicate there are active or potentially active faults at, or adjacent to, the project site.

The known major active faults near the site are shown on Figure 3 and summarized in Table 1 below. These fault sources were deemed to have the greatest potential influence on the seismicity at the site.

The updated seismic evaluation includes the Montezuma Hills Source Zone as a potential contributing source to the site seismicity. The findings of the June 2007 Seismology Memorandum suggest there is evidence that the Quaternary Montezuma Hills uplift is associated with a reverse fault extending from Rio Vista southward towards the site. The Sherman Island Fault is assumed to be the southern extension of this reverse fault. The June 2007 Seismology Memorandum indicates that the Montezuma Hills Source Zone encompasses possible active fault structures including the Sherman Island Fault. For purposes of the updated seismic evaluation, the Sherman Island Fault was used as a potential deterministic fault source to determine distance from the site to the Montezuma Hills Source Zone, as shown on Figure 3.

As discussed in Section 4.4.1, two segments of the Antioch-Davis fault are mapped as crossing the western portion of the site. The Antioch fault was originally classified as an Alquist-Priolo Earthquake Fault Zoning Act fault; however, in the early 1990's further study concluded that there was no evidence that an active, surface fault existed on the previously mapped Antioch alignment. Subsequently, the current version of the Alquist-Priolo Earthquake Fault map no longer indicates the Antioch Fault, and it is no longer classified as an active fault. The Davis fault runs north-south and crosses Lone Tree Valley near the project site. No geomorphic features suggesting Holocene displacements along the fault have been found, and hence, the Davis fault is not classified as active. Recent fault studies by WLA and others (WLA, 2004) support the conclusion that the fault has not been active during the Holocene era and likely has not experienced Quaternary activity. The fault is not considered an active structure that could pose a surface rupture or earthquake source hazard to the detention basin.

4.6.2 Design Ground Motions

Relevant fault and seismic parameters were used in conjunction with the Next Generation Attenuation (NGA) models by Boore and Atkinson (2008), Campbell and Bergonia (2008), and Chiou and Young (2008) to determine the 50^{th} and 84^{th} percentile ground motions. These parameters, presented in Table 1 below, include moment magnitude, fault length, rupture width, dip, sense of slip and direction of dip. The NGA models include shear wave velocity input for the top 30 meters of the site profile. Site specific shear wave velocity tests were performed by NORCAL Geophysical Consultants in October 2009 as presented in Appendix G of the Geotechnical Data Report (CE&G, 2010). An average shear wave velocity of 300 meters per second was used in the NGA models based on the shear wave velocity test results.

Based on the results of the NGA models shown in Appendix 2.2, the Greenville fault will produce the largest potential horizontal peak ground acceleration (PGA) at the site. Based on an upper bound magnitude of M_w 6.9 for the Greenville Fault at a distance of 9 km from the site, the median and 84th percentile horizontal PGA were estimated to be 0.30g and 0.50g, respectively.

Because the basin is designed to operate as a dry reservoir for flood control purposes, earthquake and flood are not assumed to occur concurrently. Under non-flood conditions the reservoir storage will be less than 35 acre-feet. Under non-flood conditions, earthquake damage, if any were to occur, would not result in a sudden increase in creek flow and flooding downstream of the dam. Downstream property damage is not anticipated, and there would not be people that would be placed in peril and need to be evacuated from downstream facilities, provided that damage repairs can be pursued in a timely manner. The dam height is small (approximately 40 feet). Based on these considerations, the basin's Total Class Weight, a damage-potential parameter historically used by DSOD to represent the range of failure consequences, is estimated to be very low (with a value less than 7) for seismic risk purposes. Accordingly, the consequence of seismic damage is judged to be low and, based on DSOD guidelines explained by Fraser and Howard (2002), the appropriate statistical level of acceleration for deterministic hazard analyses is judged to be the 50th percentile PGA values presented in Table 1. However, an analysis using the 84th percentile PGA value has also been performed for the record. The seismic stability analysis is included as Appendix 2.5.

Table 1: Summary of Nearby Active Faults

Fault	Fault Parameters [1]			Approx Distance from Site[3] (km/miles)	General Direction from Site	Moment Magnitude M_w[1]	Peak Ground Acceleration (g)[4]	
	Type[2]	Rupture Width / Length (km)	Dip (Direction)				50th percentile (Mean)	84th percentile
Great Valley 5 (Pittsburg-Kirby Hills)	R (HW)	25/24	80° (E)	14.5/9.0 *10.0 /6.2*	Northwest	6.7	0.22	0.38
Greenville	RL-SS	18/58	90° (N/A)	9.0/5.6	Southwest	6.9	**0.30**	**0.50**
Mt. Diablo Blind Thrust	R (HW)	19/31	45° (NE)	19.5 /12.0 *7.0/4.3*	Southwest	6.7	0.29	0.48
Concord-Green Valley	RL-SS	16/56	90° (N/A)	20.7 /12.8	West	6.7	0.18	0.30
Montezuma Source Zone (Sherman Island Fault)	RO (HW)	20 /Not Applicable	70° (W)	10/6.2 *10/6.2*	North	6.5	0.26	0.44

1. Fault parameters and moment magnitude based on information obtained from Delta Risk Management Study (DRMS), Phase 1, Seismology Memorandum, dated June 2007, prepared by URS and Jack R. Benjamin Associates, Inc (June 2007 Seismology Memorandum).
2. RL-SS- = right lateral strike slip, R= reverse, RO= reverse oblique, HW= site is assumed to be on the hanging wall side of fault (for normal or reverse faults), FW= site is on the footwall side of the fault (for normal or reverse faults).
3. Distance of Fault to project site obtained from Quaternary Fault and Fold Database by USGS (http://earthquake.usgs.gov/hazards/qfaults/) or the 2002 California Fault Parameters Database by the California Geological Survey (www.consrv.ca.gov/cgs/rghm/psha/). For reverse faults, the closest estimated distance to the surface projection of coseismic rupture determined based on the dip angle of the fault and orientation to the site (value shown in italics).
4. Based on an average shear wave velocity of 300 meters per second for the upper 30 meters of the site subsurface profile.

4.7 Summary Assessment of Subsurface Conditions

Based on available geologic mapping (CE&G Figure 8), the majority of the detention basin will be excavated in alluvial deposits. Along the southern rim, excavation will be in surficial colluvium and bedrock. The embankment will be founded primarily on alluvial terrace deposits that form the floor of Lone Tree Valley. At the east (right) abutment, where the dam will abut the hillside east of the creek, the embankment also will be founded on stiff clay colluvial and terrace deposits overlying an ancient landslide mass (disturbed bedrock), which is in turn underlain by undisturbed bedrock.

4.7.1 Dam

The geology along the approximate dam axis northwest of the creek is shown on cross-section A-A on CE&G Figure 9. Based on the available information, the dam will be founded on Qt_2 terrace deposits consisting of stiff to very stiff, lean to fat clay soils underlain by interbedded shale/claystone/sandstone bedrock. Interbedded lenses or layers of sandy alluvium exist above the bedrock surface. Depth to rock varies across the site, ranging from over 75 feet under the left abutment to over 20 feet at the creek channel. Under the right abutment, the disturbed bedrock of the landslide mass ranges in depth from a few feet to about 20 feet, and the undisturbed bedrock is at a depth of approximately 40 to 45 feet.

Based on the information provided in the CE&G (2010) report, we expect that the Qt_2 terrace deposits will provide a satisfactory foundation for the dam given the low height and flat slopes of the embankment. Isolated lower terrace deposits (Qt_1) and alluvial stream deposits (Qal) have been mapped in and adjacent to the channel. The Qt_1 materials were observed in boring B-8-09 and included layers of soft to very soft clay. The alluvial stream deposits mainly consist of recently deposited silt and silty sand. As discussed in Section 5, these isolated deposits of recent alluvium will need to be removed from the dam foundation but could potentially be processed and reused in an appropriate zone of the compacted embankment.

On the right (east) side, the dam will abut the existing hillslope that descends to the southeast bank of Sand Creek. This area has been investigated with an exploratory program consisting of seven core borings extended into bedrock (four by WLA and three by CE&G) and three exploratory trenches (two by WLA and one by CE&G) to characterize the depth of surficial deposits and type of underlying bedrock, assess conditions in the slide mass above the abutment and along the slide plane, and assess slide stability.

The available data and interpretation indicate that the ancient slide has been inactive for many thousands of years and is buttressed by the Qt_2 alluvial terrace that has now been incised by Sand Creek. Stability analyses have been performed through the slide mass and dam foundation excavation zone to verify the stability of the slide mass. The analysis is presented in Appendix 2.1. The limited foundation excavation for the dam has been evaluated and should not undermine the slide. The results of the investigations and analyses indicate that the ancient slide will not adversely affect the performance of the dam or pose a high risk of reactivation during dam construction.

4.7.2 Outlet Works and Spillway

The proposed outlet pipeline and spillway alignments have been investigated with three test borings along the outlet works and four test pits along the spillway. Based on the site conditions it is anticipated that the spillway alignment will generally be excavated in terrace deposits consisting of stiff to hard lean clay. The outlet works will generally be excavated in terrace deposits consisting of stiff to hard lean clay and in localized deposits of alluvial stream deposits adjacent to the creek channel. The alluvial stream deposits will be removed as described in Section 5.

4.7.3 Saddle Dike

As shown on the geologic map and cross-section (CE&G Figures 8 and 14 respectively), a tributary (hanging) valley to Sand Creek exists at the southeast corner of the basin. A saddle dike with maximum height of approximately 13 feet will be needed to provide containment of the reservoir in this area. Based on three borings and four test pits along the alignment, the foundation is expected to consist of Pleistocene alluvial deposits consisting of very stiff to hard clay with sand, with carbonate cementation, generally about 5 to 10 feet thick, underlain by bedrock of the Markley Formation. The saddle dike will be founded on the very stiff surficial clay.

4.7.4 Reservoir Basin and Rim

Based on available geologic mapping (CE&G Figure 8), the majority of the detention basin excavation is expected to be in alluvial deposits. Along the southern rim of the basin, excavation is expected to be in surficial colluvium deposits and bedrock.

Based on the results of the field investigations, the excavation within the basin north of and along the creek will generally encounter stiff to very stiff lean clay and sandy lean clay. Some medium dense clayey sand and recent alluvial deposits consisting of discontinuous lenses of loose sand, silty sand, and sandy silt will also be found primarily along the creek alignment.

The basin excavation south of the creek will be primarily in clay colluvium and claystone/shale bedrock. The cuts at the southwest corner of the basin and basin inlet structure area extend through a zone of shallow slides and areas of creeping colluvial soils. Slide materials that remain in the slope after the final grade is achieved will be overexcavated and replaced with compacted and drained fill.

4.8 Groundwater

A regional hydrologic evaluation for the broader Lone Tree Valley area where the project site is located was performed by the Source Group (1999). Regionally, the occurrence of groundwater is described as limited and laterally complex. Groundwater has been encountered both in the predominantly fine-grained alluvial materials and within the weathered portion of the bedrock. Regionally, groundwater is moving from west to east toward the San Joaquin River with an average groundwater gradient estimated at 0.008 ft/ft, with local flow to the northeast at the valley margin. Due to the fine-grained nature of the alluvium in the valley, the measured values of hydraulic conductivity reported by the Source Group were characteristically low, ranging from 0.044 feet per day to a maximum of 6.2 feet per day.

Based on the available information, it is anticipated that the groundwater table will be near the level of the water in the creek during the wet season. During the field explorations performed by CE&G in late summer 2009, groundwater was measured at El. 155 to 161. During the field explorations by Fugro (2004), groundwater was measured at El. 160 to 165.

Based upon the depth of excavation proposed in the basin design, groundwater and saturated soil conditions are likely to be encountered near the bottom of the excavation in the southern portion of the basin (along the low-flow channel), which is proposed at elevations ranging from El. 165 at the southwest corner to El. 159 adjacent to the upstream toe of the dam, within the low-flow channel itself, at the dam foundation excavation in the immediate vicinity of the creek channel, and along the excavation for the outlet works. Based on the available information and the proposed elevation for the future recreational athletic fields (above El. 166 feet), groundwater is expected to remain below the proposed fields except during wet periods.

5. Foundation Excavation and Treatment

5.1 Foundation Objective

The foundation objective is a descriptive tool to convey to construction engineering staff and the contractor the design intent for a dam foundation. It is a field-verifiable description of geologic or geotechnical conditions that meet foundation performance requirements. The approach to establishing the foundation objectives for the embankment dam and appurtenant structures is discussed below.

5.1.1 Dam Foundation

Based on the available information, the dam will be founded on Qt_2 terrace deposits consisting of stiff to very stiff, lean to fat clay soils underlain by shale/claystone bedrock. Isolated lower terrace deposits of younger terrace deposits (Qt_1), stream channel alluvium (Qal) and artificial fill (Qaf) have been mapped adjacent to the channel. These materials consist of soft to medium stiff clay and loose sandy soils. These deposits of recent alluvial materials will be excavated from the dam foundation. This foundation objective has been indicated in the construction drawings in the form of an excavation contour map (see Drawing C-11).

During construction, it will be necessary to confirm that geologic conditions exposed in the foundation at the predetermined excavation depths are consistent with this foundation objective. Detailed geologic mapping of the excavated foundation will be performed to confirm that stiff clay (with compressive strength greater than 2,000 psf) is exposed. Foundation conditions that are different will need to be evaluated to confirm that they are consistent with the assumptions made in the design analysis and that they meet the design requirements. Foundation conditions that invalidate the analysis would need to be evaluated in more detail prior to foundation acceptance.

The foundation objective for the saddle dike will be placement of the embankment on stiff clay as defined above.

5.1.2 Key Trench

A key trench will be provided under the dam crest to disrupt and lengthen the seepage path at the contact between foundation and embankment. The key trench will have a bottom width of 20 feet, approximately one half the height of the dam. The side slopes of the cutoff trench will be as needed for safety but not steeper than 1(H):1(V).

It is anticipated that the key trench also will be excavated and contained within the stiff clay of the Pleistocene-early Holocene terrace (unit Qt_2). The proposed foundation objective for the key trench will be the same as indicated above for the dam embankment. In addition, if during construction and due to changed conditions a portion of the foundation were accepted on a material other than a fine-grained soil, the key trench would still be required to extend to stiff clay of the Qt_2 unit or another impervious layer of soil that can be determined based on additional explorations and analyses to be of sufficient extent as to prevent excessive seepage beneath the dam.

The required minimum depths of the key trench are tabulated on Drawing C-12. The depth will be not less than 2 feet below the dam-foundation contact except west of Station 5+00 where the dam height will be less than 6 feet and a minimum depth of 1 foot will be allowed. The key trench will be backfilled with low-permeability core material obtained from basin excavation.

The lowest portion of the key trench, at the creek crossing, is expected to be below the groundwater table; therefore, dewatering measures will need to be implemented during construction (see Section 8).

5.1.3 Outlet

The outlet works system includes an intake structure, conduit and energy dissipator. The outlet works is a lightly loaded system. Due to the proposed depth of basin excavation along the outlet works, the intake and terminal structures apply no net load to their respective foundations above the loading of the excavated soils. Under the footprint of the dam, the foundation objective for the outlet conduit will be the same as that for the dam foundation, i.e., placement into the stiff clay of the Pleistocene - early Holocene terrace (unit Qt_2). Outside the dam footprint, the foundation objective for the outlet works will be placement on unit Qt_2 soils, although there will not be a need for the soil to classify as a clay. Soils encountered under the outlet conduit grade that do not meet the foundation objective (i.e., soils that are either unit Qt_1 or recent alluvium) will be excavated and replaced with compacted embankment material of the same type as specified for the adjacent embankment.

5.1.4 Emergency Spillway

As described more fully in Section 7.1 below, the emergency spillway will be a concrete structure where it passes through the dam abutment. The foundation objective for this section of the emergency spillway will be the same as for the dam foundation (see Section 5.1.1 above).

Beyond the downstream toe of the dam, the spillway chute will be lined to prevent erosion. Because of the stiff clayey nature of the soils, there is a potential for seasonal shrinkage and swelling of the spillway foundation and adjacent soils. Consequently, a flexible and plantable precast-concrete-block lining has been selected. In addition to its flexibility, the proposed lining offers adequate erosion resistance at the design flow velocity and will allow the establishment of vegetation between the blocks for a more natural appearance. The natural vegetated appearance is important given the secondary recreational purpose of the detention basin facilities. The foundation objective for the emergency spillway chute will be reaching soils of the Qt_2 unit.

The terminal riprap will be placed on the bed of the creek, excavated to the required grade, with a filter fabric placed to separate the riprap from the underlying soil.

5.2 Foundation Excavation and Treatment

In addition to the foundation objective stated above, surface treatment of the foundation under the embankment, cutoff trench, and saddle dike will be needed. Surface treatment will include the following requirements:

- Clearing, grubbing and stripping of the loose soil mantle, debris, topsoil, and vegetation prior to excavation. The minimum depth of stripping for the dam footprint shall be 12 inches, or deeper if needed to remove all roots and deleterious materials as directed by the Engineer. Stripped material and debris will be disposed of outside the dam footprint.

- Excavation slopes in the dam foundation shall be as follows:

  (a) key trench side slopes: 1(H):1(V) or flatter

  (b) cut slopes along dam upstream and downstream toes: 1(H):1(V) or flatter

  (c) foundation excavation slopes within dam footprint other than (a) and (b) above: 2(H):1(V) or flatter unless shown otherwise on the Drawings

- The foundation and abutment surfaces will be smoothed by removing high points and by further excavation as needed such that they will be not steeper than 1(H):1(V).

- Once the required excavation depth has been achieved, the foundation surface will be maintained free of water and cleaned of all loose, soft, disintegrated, rutted and/or erodible materials.

- Depressions and potholes in the foundation surface will be cleaned and filled with compacted fill of the same material as specified for the overlying embankment.

- The foundation surface will be scarified, moisture conditioned and re-compacted just prior to placement of the first layer of fill materials to achieve a suitable bond of the foundation surface with the overlying fill.

- Where the slope and orientation of the foundation surface is such that a sliver fill would be constructed, additional excavation will be made to obtain a minimum horizontal width of fill of 10 feet or greater.

- The excavated dam foundation will require approval by the Engineer and DSOD. The Engineer and DSOD will inspect and approve all foundation surfaces prior to placement of embankment material or concrete. Additional excavation may be required to obtain a satisfactory foundation and will be performed as directed in the field by the Engineer, with concurrence by DSOD.

- The following special foundation treatment will be applied over the Calpine gas pipeline that runs along the downstream toe of the dam (within the footprint) at the northwest corner:

    - In coordination with Calpine representatives, mark the location of the gas pipeline on the ground. Confirm by probing that the depth of the pipeline is approximately 4 feet.

    - Within 5 feet either side of the gas line, strip the foundation to a depth of 6 inches. Prior to placing fill, scarify and moisture-condition the stripped foundation surface to a depth of 6 inches.

6. Embankment Dam Design

6.1 Dam Design and Cross-Section

The key dimensions of the main embankment include:

- Maximum dam height (crest El. 195.5 to downstream toe El. 156): approximately 40 feet
- Crest length: 1,885 feet
- Crest width: 20 feet
- Upstream and downstream slopes: 4.0(H):1(V) below El. 191.0 (the reservoir normal maximum water surface), and ranging from 3.5(H):1(V) to 3.2(H):1(V) above El. 191.0 depending on camber requirements

The dam is designed to be safe and stable. Key design considerations include the following:

- The slopes must be stable and resistant to deformation under all operating conditions, including rapid reservoir drawdown.
- Seepage through the embankment and its abutments and foundation must be controlled so that piping and sloughing do not occur.
- The embankment must be safe from overtopping by both flood inflow and wave action.
- The embankment must be safe from catastrophic failure during reasonably expectable earthquakes at the site.
- The slopes must be safe from excessive damage from wave action or rain.

A typical cross-section showing the proposed internal zoning is shown on Drawing C-12. The dam is proposed to be a zoned embankment with internal drainage. The following zones are proposed:

- Zone 1: This zone will be constructed using low-permeability clayey soils from basin excavation and will form the impervious core of the dam.
- Zone 2: A zone of granular filter material will be located downstream of the core and extend the full length of the dam. This zone will serve as a "crack stopper."

- Zone 3: Highly pervious gravel will be placed along the bottom of the crack-stopper zone and in a base drain at the bottom of the creek in the dam's maximum section to convey seepage water and discharge it to the downstream toe. The base drain will consist of a pervious gravel core of Zone 3 enveloped by a filter layer of Zone 2 and will underlie the downstream portion of the embankment along the creek channel.

- Zone 4: Random material from basin excavation will be placed in Zone 4. The material from basin excavation south of the creek will likely still have low permeability but may contain remnant bedrock fragments from the Markley formation materials. This fill also may contain more sandy soils from excavation in recent alluvium deposits.

- Zone 5: Aggregate base placed in the crest surfacing and perimeter road.

As described in Section 5, a key trench will be provided under the central portion of the dam section to disrupt and lengthen the seepage path at the contact between foundation and embankment. The trench will be excavated into stiff clay of the late Pleistocene terrace, unit Qt_2. The key trench will have a bottom width of 20 feet, side slopes no steeper than 1(H):1(V), and will be backfilled with Zone 1 material. As discussed in Section 5, the embankment will be placed on a suitable foundation, with soft and/or loose recent alluvium materials anticipated to be removed.

The crest will be cambered (over-built) to account for future settlement. Based on the results of the settlement analysis presented in Appendix 2.6, a camber of 6 inches will be provided at the maximum section of the dam. A camber diagram is provided on Drawing C-15.

The crest of the embankment will be surfaced with 6 inches of aggregate base to provide a trafficable surface for occasional maintenance pickup truck traffic. The crest will peak at the centerline and will have a cross-slope of 2% draining to both sides. Even with cross-slopes the road will be subject to rutting under saturated conditions and will need periodic maintenance and repair.

6.2 Saddle Dike Design

A saddle dike with maximum height of approximately 13 feet and crest length of approximately 750 feet will be constructed at the southeast corner of the basin as shown on Drawing C-1. The saddle dike is proposed to be a homogeneous clay embankment with a crest width of 20 feet and 3(H):1(V) side slopes. The District anticipates that the landowners that own the property to the south of the saddle dike may desire placement of additional embankment to raise the ground level in the area south of the detention basin. If that is the case, the additional fill could be placed directly against the downstream slope of the saddle dike, so that the dike slope would become significantly flatter than 3(H):1(V) or even disappear. The saddle dike will be constructed from the same material as Zone 1 of the main dam.

6.3 Embankment Materials

The embankment Zones 1 and 4 will be constructed of material derived from required basin and embankment foundation excavations. Soils used for Zone 1 embankment fill will have a Liquid Limit (LL) less than 50, a Plasticity Index (PI) of at least 8 and less than 40, with 100 percent passing the 3-inch sieve and at least 30 percent passing the No. 200 sieve. Zone 4 will accept the same type of material. In addition, sandy soils (with less than 30 percent passing the No. 200 sieve) and gravelly soils, if any are excavated from the basin, will be routed to Zone 4. All fill will be free of organic materials, such as roots and other organic debris.

The materials for filter (Zone 2), drain (Zone 3), aggregate base (Zone 5), and rock slope protection (riprap) will be from commercial sources. Filter criteria have been evaluated in developing the gradation requirements for the internal drainage materials. Filter compatibility is required between the embankment (Zones 1 and 4) and the downstream filter zone (Zone 2) and between Zone 2 and the drain material (Zone 3). Filter compatibility has been verified using the filter criteria as detailed in USBR's Design Standards (USBR, 1987a). The evaluation is attached in Appendix 2.3.

Soil considered not suitable as embankment fill includes soil containing organic matter and highly plastic clays with PI greater than 40 and/or LL greater than 50, which would be considered to be overly expansive. These soils could be used as engineered fill outside the limits of the dam embankment.

6.4 Freeboard

As indicated in Section 2, the minimum normal freeboard (vertical distance from spillway crest to dam crest) will be 4.5 feet. The residual freeboard during the 1,000-year 12-hour storm, will be 2.1 feet. In addition, the spillway will pass the 1,000,000-year 72-hour storm. For this storm, while the calculated residual freeboard is zero (Section 2), several inches of freeboard are expected to exist due to the use of a conservatively low discharge coefficient for calculation of the design spillway discharge capacity over the concrete weir.

6.5 Erosion Protection

Many small dams with small reservoirs, constructed of cohesive soils, have performed well with no slope protection. With a basin normally dry, the embankments will not need upstream or downstream slope protection other than a good grass cover. The dam and basin will be vegetated with a mix of native grasses.

It is anticipated that eventually, as part of the development of recreation facilities, the City will choose to plant, irrigate and maintain turf on the upstream face of the dam. Trees will likely be planted in the detention basin, but not on the embankment dam. Because of the need for access at the toe areas by maintenance and construction equipment, a minimum 15-foot-wide zone will need to be maintained free of woody vegetation along the upstream and downstream toes of the dam.

6.6 Embankment and Foundation Seepage and Stability

The dam has been evaluated for seepage and stability. Due to the varying geometry of the dam foundation and basin grading, several sections were selected and analyzed to evaluate the critical safety factors. A description of the characterization of the foundation and embankment materials, selection of engineering properties, methodology, and results of the evaluations is presented in Appendix 2.4.

6.6.1 Embankment Loading

The reservoir levels used for stability analysis were as follows:

- Normal operating level: El.158.0 (one foot above upstream invert of outlet works)
- Reservoir level during the 100-year, 12-hour-storm (spillway crest): El. 191.0 (used in steady seepage analysis)

6.6.2 Embankment and Foundation Seepage Evaluation

Control of seepage beneath (underseepage) and through (through-seepage) the dam are critical to dam performance. Seepage was analyzed using the guidance in USACE EM 1110-2-1901, *Seepage Analysis and Control for Dams*. The basic assumption for the seepage analyses was that the design water level duration is sufficient for steady-state seepage conditions to develop. The principal criterion for evaluating steady-state seepage for this project is the exit gradient (estimated gradient at the point of exit). The maximum gradient allowed by USACE guidance is the gradient that yields a factor of safety of approximately 1.6 with respect to the critical gradient. For common mineral soils with a saturated unit weight of 112 pcf or greater, this factor of safety is commonly assumed to correspond to a seepage gradient of 0.5. Consequently, the maximum allowable seepage gradient assumed for this project is 0.5.

The seepage analyses were performed on generalized sections through the dam using the two-dimensional finite element program SEEP/W developed by GEO-SLOPE International Ltd. Sections were modeled assuming steady-state conditions for the 100-year, 12-hour-storm (water level at the spillway crest). The analysis results were used to:

- Estimate the steady-state phreatic levels and pore pressures in the dam embankment and foundation soils for use in stability analyses.
- Estimate exit gradients.
- Estimate flow rate into the graded filter for filter design.

Representative coefficients of permeability were selected based on laboratory tests, field data, typical values and correlations based on soil type, and correlations with grain size distribution. The values used in the analyses are described in Appendix 2.4.

Calculated exit gradients are summarized in Table 2. The estimated exit gradients are below 0.5.

Table 2: Summary of Estimated Seepage Gradients

Station	Exit Gradient
14+05	0.18
14+70	0.35
15+55	0.07

The estimated seepage flow into the base drain is estimated to be about 5 gallons per minute assuming steady-state seepage during the 100-year, 12-hour storm event (assumed reservoir elevation of 191 ft).

6.6.3 Embankment and Foundation Stability

Dam embankment and foundation stability were evaluated using the guidance in USACE EM 1110-2-1902, *Slope Stability* and U.S. Bureau of Reclamation *Design Standards No.*

13 – Embankment Dams, Chapter 4 – Static Stability Analyses. Stability analysis cases, minimum allowable factors of safety, basin water level assumptions, and soil strength assumptions are presented in Table 3.

Table 3: Dam Slope Stability Analysis Criteria

Stability Case	Minimum Factor of Safety	Reservoir Water Level	Shear Strengths(1)
Case I - End-of Construction *Upstream Slope* *Downstream Slope*	1.3(2)	Foundation Level	**Cohesive soils:** Total stress parameters. **Cohesionless soils:** Effective stress parameters
Case II - Steady-State Seepage with Variable Water Level *Upstream Slope*	1.5(2)	Between El. 158 (intake level) and El. 191 (spillway crest)	**Cohesive soils:** Effective stress parameters. **Cohesionless soils:** Effective stress parameters
Case III - Steady-state seepage with water level at spillway crest *Downstream Slope*	1.5(2)	El. 191 (spillway crest); tailwater level for the corresponding outlet discharge (5 ft above foundation level)	**Cohesive soils:** Effective stress parameters. **Cohesionless soils:** Effective stress parameters
Case IV - Seismic (pseudo-static coefficient = 0.15) for Cases I, II and III *Upstream Slope* *Downstream Slope*	1.1(3)	See Cases I, II and III	**Cohesive soils:** Total stress parameters. **Cohesionless soils:** Effective stress parameters
Case V - Rapid drawdown from maximum water level *Upstream Slope*	1.3(2)	From El. 191 to El. 158.	**Embankment Soils**: First stage-effective stress from CU tests with pore pressure measurements. Second Stage - τ_{ff} vs. σ'_{fc} envelope derived from CU tests. (Duncan et al, 1990) **Foundation Cohesive soils**: Total stress parameters. **Cohesionless soils**: Effective stress parameters

(1) CU = Consolidated Undrained
(2) From USBR, 1987b
(3) From Babbitt and Verigin, 1996

Slope stability was evaluated for generalized sections of the dam using the program SLOPE/W developed by GEO-SLOPE International Ltd. Parameters selected for the analyses included:

- Shear Strength: Shear strengths were estimated from available field and laboratory test data from the site. Estimates were based on laboratory strength data, correlations to SPT and CPT data, and published correlations to Atterberg limits. Selection of strength parameters is described in Appendix 2.4.

- Pore Water Pressure: Pore water pressures for Case I (end-of-construction) were estimated assuming a static groundwater level at the existing groundwater table. Pore water pressures for Cases II through IV were estimated using the steady-state phreatic surface from SEEP/W as described above. Pore water pressures for Case V (rapid drawdown) were estimated assuming the steady-state phreatic surface in the dam prior to drawdown.

A pseudo-static analysis was performed to evaluate stability for Case IV (Seismic). As required by DSOD, a pseudo-static coefficient of 0.15 was used for this case.

The analyses are presented in Appendix 2.4, and the results are provided in Table 4 below. The analyses indicate that the cross-section geometry meets the minimum required factors of safety for stability.

Table 4: Dam Slope Stability Analysis Results

Stability Case	*Required Minimum Factors of Safety*	Estimated Factors of Safety		
		Station 14+05	Station 14+70	Station 15+55
Case I End-of-Construction (U) – Upstream Slope (D) Downstream Slope	***1.3***	3.0 (U) 2.0(D)	2.9 (U) 2.2 (D)	3.0 (U) 2.2 (D)
Case II Steady State Seepage with Variable Water Level (Upstream Slope)	***1.5***	2.6	2.4	3.0
Case III Steady State Seepage with Water Level at Spillway (Downstream Slope)	***1.5***	2.5	2.6	2.6
Case IV Pseudo-Static (U) – Upstream Slope (D) Downstream Slope	***1.1***	1.7 (U) 1.2 (D)	1.7 (U) 1.3 (D)	1.7 (U) 1.3 (D)
Case V Rapid Drawdown	***1.3***	2.1	1.7	1.9

6.7 Post-Seismic Stability and Deformation Analysis

As discussed in Section 6.6 of this report, a pseudo-static seismic stability analysis was performed for the dam assuming undrained, pre-seismic shear strengths. The dam was also evaluated for post-seismic stability and deformation. Selection of the engineering properties, analysis methodology, and analysis results for the post-seismic evaluations are presented in Appendix 2.5.

To determine the post-seismic shear strengths, we evaluated the foundation soils to determine if they were susceptible to strength reduction due to seismic shaking from the controlling fault (Section 4.6) with estimated 50th and 84th percentile bedrock horizontal peak ground accelerations at the dam site of 0.30g and 0.50g, respectively. For susceptible soils, post-seismic residual shear strengths were estimated. Stability analyses were performed for three generalized cross sections using the estimated post-seismic shear strengths and assuming the basin was empty. The factors of safety (Table 5) for both the downstream and upstream slopes are still higher than the required minimum factors of safety specified in Section 6.6 of this report, indicating adequate dam embankment stability with post-seismic shear strengths. The factors of safety are the same for both the 50th and 84th percentile events.

Table 5: Dam Post-Seismic Slope Stability Analysis Results

Section	Upstream Factor of Safety	Downstream Factor of Safety
14+05	2.2	1.6
14+70	2.5	2.4
15+55	3.0	2.0

In addition to stability, an analysis was performed to estimate post-seismic deformations using Makdisi and Seed's (1977) modified Newmark method. Vertical displacements of the dam due to foundation liquefaction were estimated to be less than 1 inch for a 50^{th} percentile earthquake event and on the order of 1.5 to 6 inches for an 84^{th} percentile earthquake event.

6.8 Settlement Analysis

A settlement analysis of the dam was performed using the guidance in USACE EM 1110-1-1904, *Settlement Analysis*. Characterization of the dam foundation and embankment materials, selection of consolidation parameters, methodology, and analysis results are presented in Appendix 2.6. Consolidation parameters were estimated based on site-specific geotechnical data along the alignment. The commercial program WINSAF-I was used to model the embankment loading and to estimate the total settlement at the existing ground surface from the embankment loading.

A total settlement of 6 inches was estimated under the tallest part of the proposed dam. A corresponding amount of camber (overbuilding of the embankment crown) is provided in the design to compensate for the anticipated settlement and maintain the required freeboard.

The estimation of foundation settlements is far from exact due to the variability of foundation conditions, the limitations and simplifications of the analysis, and other factors. Conservative overbuilding of the embankment would entail a cost, and the additional overbuilding would not necessarily provide an increased level of protection against flooding because it would likely not be uniform along the crest. Survey monuments will be installed along the dam crest to allow for long-term monitoring of crest elevations. Periodic monitoring of the monuments will be included in the Operation and Maintenance Plan. If, over the long term, areas of the embankment are detected where the crest has settled below the design elevation, the addition of fill to the crown would be necessary to restore the design freeboard. This would be implemented as a maintenance action.

6.9 Basin Rim

The cut slope stability along the southern rim of the basin was evaluated. The methodology and results of the analysis are presented in Appendix 2.1. Basin cut slopes will generally have a slope of 3(H):1(V). The analysis indicates a static safety factor of 1.5 assuming conservative shear strengths.

6.10 Placement and Compaction Specifications

Embankment fill will be spread in uniform layers not exceeding 8 inches in loose thickness prior to compaction. The fill will be free of lenses, pockets, and/or layers of materials differing substantially in texture or gradation from surrounding materials. Layers of Zones 1 and 4 will be compacted using a sheepsfoot compactor in a uniform and systematic manner. Layers of Zones 2 and 3 will be compacted using a smooth-wheel vibratory roller. Compaction will be conducted parallel to the axis of the embankment whenever possible. The embankment will be constructed in layers such that the surface of each layer across the length and width of the embankment is nearly level.

Each layer of Zone 1 and Zone 4 fill will be moisture conditioned, as necessary, to obtain moisture content between optimum minus 2 percent and optimum plus 2 percent as determined by ASTM D 698. If, in the opinion of the Project Geotechnical Engineer, the contact surface of the fill layer is too wet, or too dry or smooth to permit suitable bonding of the surface and subsequent fill layers, it will be necessary to moisture condition, scarify, and re-compact the surface prior to placement of succeeding layers of fill material. All fill within Zones 1 and 4 of the embankment will be compacted to at least 100 percent relative compaction as determined by ASTM D 698.

Upon completion of embankment construction, the slope will be rolled or cut back to provide a firm, uniform slope.

Temporary cut slopes will be excavated to a slope as required for safety but not steeper than 1(H):1(V).

7. Appurtenant Structures

7.1 Emergency Spillway

The emergency spillway will be located in the left abutment of the dam and will consist of a broad-crested overflow-type spillway with a side-channel arrangement. The spillway crest will be at El. 191.0. The crest length is 225 feet. The spillway arrangement is shown on Drawing C-30, and a profile and cross-sections are shown on Drawing C-31. The overflow section will form a depression in the basin service road and will consist of a reinforced-concrete slab founded on non-erodible cement-treated soil with a thickness ranging from 2 to 4 feet.

The hydrologic design basis for the spillway is summarized in Section 2. The peak outflow for design is approximately 5,700 cfs. The rating curve is shown on Drawing C-30. The reservoir area-capacity curve is shown on Drawing C-1.

The basin service road will cross over the spillway crest. At the ends of the spillway crest, the road will slope upward to rejoin the crest of the dam at El. 195.5. A longitudinal slope of 10 percent will be used at both ends of the spillway crest.

A concrete sill will form the control section at the spillway crest. The 4(H):1(V) slope downstream of the spillway crest will be lined with a reinforced-concrete slab to prevent erosion of the foundation along the downstream toe. Underdrains are provided to prevent uplift pressures in the downstream concrete apron. Flows passing over the spillway crest will discharge over the lined 4(H):1(V) slope and collect in a lined side-channel that will turn the flow and convey the water to the creek beyond the toe of the dam.

A wide trapezoidal channel is provided to avoid high flow depths and velocities. A maximum velocity of approximately 25 fps at design conditions is provided in the spillway channel above the tailwater level in the creek. It is anticipated that the spillway discharge channel could be integrated with parking facilities for the recreational fields in the future.

The spillway discharge channel will be lined to minimize the potential for erosion. Because of the stiff clayey nature of the soils, there is a potential for seasonal shrinkage and swelling of the spillway foundation and adjacent soils. Where the spillway passes through the dam abutment, the spillway will be a concrete structure. Beyond the downstream toe of the dam, a flexible and plantable precast-concrete-block lining has been selected. In addition to its flexibility, the proposed lining offers erosion resistance at the design flow velocity and will allow the establishment of vegetation between the blocks for a more natural appearance.

The spillway lined side-slopes and the downstream discharge channel beyond the dam have been designed to prevent over-topping during the spillway design flood. Hydraulic calculations for the emergency spillway are provided in Appendix 3.1.

The spillway discharge channel will join the creek downstream of the dam toe. Riprap will be provided to reduce the potential for erosion at the terminal end of the spillway and along the creek banks at the section where the spillway flow rejoins the creek channel. The riprap size has been selected taking into consideration that the spillway flows would result in an elevated creek level, which would dissipate some of the energy of the incoming flow. However, under rapidly rising high flows some damage to the creek bed at the downstream end of the spillway may occur.

7.2 Outlet Works

7.2.1 Design Criteria

The outlet works will pass flows under the dam. The outlet works will be constructed from reinforced concrete and includes a low-level inlet structure, a gated shaft inlet structure, a conduit, and an outlet structure. The two inlets to the conduit are provided for the following purposes:

- To reduce operational risk, the District desires passive management of creek flows through the range of normal operation up to the 100-year storm event. An ungated low-level (and therefore always open) inlet will be provided to pass low-level flows including seasonal creek flows. No operation will be needed, other than periodic removal of trash that will collect at the trashrack. The flow through this ungated opening will be inlet controlled throughout the range of normal operation up to the 100-year storm event.

- A higher-level gated inlet will be provided for very rare occasions when the District decides to more quickly regain storage capacity in the event of back-to-back storm events or under emergency conditions. The gated inlet will consist of a concrete shaft covered with a sluice gate located near the upstream toe of the dam and will be normally closed. Gate operation will be manual and locally controlled.

The arrangement of the outlet works is shown on Drawing C-20. Design criteria are as follows:

- The discharge through the low-level, ungated inlet will be approximately 134 cfs with reservoir stage at El. 191.0. This means that under normal operating conditions (up to the 100-year storm), discharge out of the basin will be metered at a maximum flow rate of approximately 134 cfs.

- A peak discharge capacity of approximately 500 cfs has been targeted for the outlet works with the gated inlet open and reservoir stage at El. 191.0.
- The outlet conduit needs to be self-cleaning to minimize sedimentation and associated maintenance requirements. The District defines self-cleaning conduits as conduits where the uniform flow velocity calculated assuming a flow depth of 1 foot (for conduits with diameter greater than 4 feet) or 25% of conduit diameter (for conduits with diameter less than 4 feet) equals or exceeds 3 fps.
- Under normal conditions (i.e., with the gated inlet closed), flow through the outlet conduit will be primarily open-channel flow.
- A trash rack will be provided for the ungated inlet.
- The outlet works design has ample capacity to discharge at least 50% of the reservoir capacity by gravity in a period of seven days or less.

Hydraulic calculations for the outlet works are provided in Appendix 3.2. The selected outlet dimensions are as follows:

- Conduit diameter: 60 inches
- Ungated inlet opening: 26 inches by 26 inches
- Gated inlet: 60 inches by 60 inches

7.2.2 Location, Alignment and Grade

The length of the outlet works will be approximately 400 feet, sloping uniformly between El. 157.0 feet at the inlet and El. 155.5 feet at the outlet. The conduit will cross the dam foundation under the left abutment (see Drawing C-20). The outlet conduit is positioned such that the full reservoir capacity can be discharged by gravity.

7.2.3 Outlet Works Inlet

At the inlet, two openings will be provided in accordance with the design criteria stated above. The design of the inlet is shown on C-22. An ungated opening with invert at El. 157.0 and protected with a trash rack will allow passage of the creek flows. The invert of the opening will match the invert of the conduit.

A separate, gated, opening will be provided for emergency operation, with invert at approximately El. 171.0. This inlet will have an upstream control device (sluice gate) designed for opening under an unbalanced seating load of 25 feet. A locked manual operator will be provided at the crest of the dam. A rack will be provided over the opening for safety.

A service ramp into the basin will be provided to access the two inlets. This road will afford access to clean the trashrack from above and to service the gate.

If absolutely necessary, the gated inlet could be used for maintenance access to the outlet conduit from the upstream slope of the dam (when the reservoir is empty). Ladder rungs will be cast into the inlet wall to enable access. Entry will be a confined space entry and will require fall protection equipment and other necessary precautions under the District's Safety Plan.

7.2.4 Conduit

The outlet conduit consists of a precast reinforced-concrete pipe encased in concrete. Typical cross-sections are shown on Drawing C-21. The conduit will be constructed in a trench excavated in the stiff to hard clay of the late Pleistocene-early Holocene alluvial terrace that forms the left abutment. Concrete will be cast against the bottom and sides of the trench and at least 1 foot above the top of the conduit.

Except for a short reach near the downstream toe of the dam where the conduit will cross the creek channel, the entire length of the conduit under the dam will be placed in a trench and bedded on native foundation materials of relatively uniform density and consistency (stiff clay). At the creek crossing, recent alluvium deposits will be excavated and replaced with compacted embankment material. An analysis of settlements along the conduit is presented in Appendix 2.6. Anticipated settlements along the conduit range from 0.2 to 1.9 inches. The maximum estimated differential settlement along the conduit is 0.4 percent. To accommodate these small potential differential settlements, provisions will be made for the conduit to be capable of deforming without cracking. To accomplish this, contraction joints with water stops will be provided at short intervals to allow flexibility of movement to the structure in response to the foundation settlements.

The final alignment of the outlet conduit will be verified in the field after site stripping operations, and after the embankment foundation and key trench are excavated.

7.2.5 Discharge Structure and Energy Dissipator

A discharge structure with an energy dissipator, concrete apron and riprap erosion protection will be provided at the point of discharge into the creek. Its purpose is to reduce erosive forces created by the concentration of flow at the outlet of the pipeline and to prevent excessive scour from occurring at the end of the structure. The design, shown on Drawing C-24, consists of an impact-type energy dissipator, contained in a boxlike structure. The design follows guidelines given in USBR (1978) and is often used for pipe outlets since it requires no tailwater for successful performance. The structure has been designed for the flow which would occur at the maximum normal reservoir level assuming that the high-level gated inlet is open.

7.3 Basin Inlet Structure

A basin inlet structure will be required to drop the Sand Creek flows from the channel bed upstream of the basin into the basin floor. The basin inlet structure will consist of a drop structure with energy dissipator baffle blocks, downstream stilling and desilting basin, and riprap erosion protection. Concrete wing walls and a bridge deck will provide a crossing for the perimeter maintenance road. Riprap armoring is provided at the transitions between the concrete structure and the stream channel upstream and downstream. The design of the structure, developed by the District, is shown on Drawings C-40, C-41, and C-42.

8. Control and Diversion of Water

The following features of the proposed project, listed from upstream to downstream, will be constructed in the area now occupied by the Sand Creek channel and banks:

- Basin inlet structure
- Basin lower-tier excavation and creek low-flow channel
- Outlet works inlet, portion of conduit, and discharge structure
- Dam foundation along creek bottom
- Spillway terminal end and riprap lining

Control and diversion of Sand Creek water during construction will be a responsibility of the construction Contractor. Control measures will depend heavily on the time of year and amount of runoff in the creek during construction, and on the Contractor's proposed construction schedule. The District has indicated that the creek dries up during summer/fall; however, local storm drains have year round base flow that will need to be bypassed. Creek water and/or storm drain flows could be handled by one or a combination of the following measures:

- Constructing a cofferdam and routing the flows into a temporary pipeline around the construction area.
- Constructing a cofferdam with a sump and pumping the flows through a temporary pipe around the construction area.
- Constructing the outlet works before performing all of the dam foundation excavation across the creek, and routing the flows through the outlet conduit in order to dewater the dam foundation.

A water diversion and control plan will be identified in the construction contract package as a required preconstruction submittal requiring approval by the District before construction operations can commence in the stream.

Additionally, construction water will need to be handled and discharged in accordance with the applicable NPDES permits and other applicable environmental permits as described in Section 1.3.

9. Operation and Maintenance Considerations

Monitoring of the dam condition and performance is required and customary for a water-impounding earthen structure newly constructed on an untested foundation. Particular attention will need to be paid to dam and basin condition and performance during the first several years of life and initial high-water events. Detailed inspections should be performed at regular intervals, during and after each storm that results in the basin water level rising 10 feet or more above the invert of the outlet, and after each felt earthquake. Conditions to monitor and record will include but not be limited to the following:

- Erosion: Condition of dam and basin slopes, vegetation cover, erosion-induced sloughing, erosion at discharge areas of outlet works, spillway, and basin inlet structure, erosion of gravel surfacing along perimeter maintenance road.
- Human and animal activity: Unauthorized excavations or encroachments, animal burrows, rodent activity along dam slopes and/or crest
- Settlement: Crest settlement, dips, depressions, and observation of settlement-related distress, such as transverse cracking across the crest.
- Vegetation control: Trees or woody vegetation on dam slopes or immediately adjacent to the toe.
- Seepage: Excessive seepage at or near the downstream toe during high-water events, wet areas on the downstream slope, boils.
- Stability: Raveling, sloughing or slumping on the dam slopes, slides or ground creep on the basin slopes, bulging, depressions, lateral deformation (weave in alignment of the crest).
- Trash and Sedimentation: Excessive accumulation of trash or sediment in the low-flow channel just upstream of the outlet pipe intake and/or obstructing the trashrack.
- Outlet Works Condition: Trashrack condition, debris accumulation, gate condition and operability, condition of lifting rod and operator, condition of interior of outlet pipe, condition of pipe joints (separation, movement, infiltration of water or soil).
- Concrete Structures: Condition of concrete, deterioration, spalling, excessive cracking, displaced slabs or walls, tilting of walls.

- Emergency Spillway: Unobstructed overflow weir and discharge channel, displacement or irregularities in the articulated concrete block lining, vegetation cover, riprap condition.

- Perimeter Access Road: Slope raveling or sliding that obstructs road, plugged drop inlets, plugged culverts, plugged ditches, erosion or other damage to road surfacing.

The dam will be constructed with the locally available plastic clays. As such, the slopes will be susceptible to desiccation cracking that could extend several inches or more into the dam slopes. This condition should be expected and could cause local nuisance sloughing along the slope when heavy rains saturate the slope that was previously desiccated and had surficial cracking. Periodic maintenance will be required to maintain the dam and basin slopes so that their condition does not deteriorate. Routine maintenance activities to minimize the impacts of desiccation cracking and water action would include maintaining healthy vegetation growth on the slopes, rodent control, and dragging and local grading of the slopes when needed to repair minor surface erosion or irregularities and prevent more serious erosion. Maintenance will need to be conducted with the realization that increased protection must be implemented expeditiously if and when actual problems develop in localized areas.

Periodic monitoring and visual observation for evidence of settlement and cracking, and prompt initiation of repairs, should be included in the dam operation and maintenance activities during the initial few years after construction, and particularly immediately after the occurrence of flood events and felt earthquakes. Minor cracks would be repaired by overexcavating the crack with a backhoe and backfilling the trench with compacted material from the excavation. In addition to such routine maintenance actions, observations that lead to a concern or suggest the potential for a future problem will need to be brought to the immediate attention of a qualified engineer for evaluation and implementation of remedial measures if appropriate.

As discussed earlier, the estimation of foundation settlements is only approximate. Settlement monuments are provided along the dam crest to address the risk that actual settlement, whether resulting from static foundation consolidation or seismic ground motion, could exceed the camber provided along the crest. Periodic surveys of the dam crest should be included in the operation and maintenance activities. If, over the long term, areas of the dam are detected where the crest has settled below the design elevation, the addition of fill to the crown would be necessary to restore the design elevation. This would be implemented as a maintenance action. Estimated settlements are presented on Table 6. Also included are maximum settlement values which, if exceeded, should trigger a prompt consultation with a qualified dam engineer. The accuracy of surveys performed for this purpose is typically in the range of ¼ to ½ inch.

Data from inspections and settlement surveys should be reviewed as it is collected and evaluated by District staff trained in dam safety concepts. In reviewing the data, particular attention should be given to changing conditions such as increases in seepage (during periods when the basin is impounding water) or settlement rate. Such changing conditions may be indicative of safety issues that could require urgent attention. All records and documentation of dam safety evaluations should be maintained in the project file.

Table 6: Settlement Monuments

Monument No.	Estimated Settlement (ft)	Maximum Settlement (ft)
SM-1	0.0	0.1
SM-2	0.1	0.1
SM-3	0.1	0.2
SM-4	0.2	0.3
SM-5	0.2	0.4
SM-6	0.3	0.5
SM-7	0.2	0.3
SM-8	0.0	0.1

10. References

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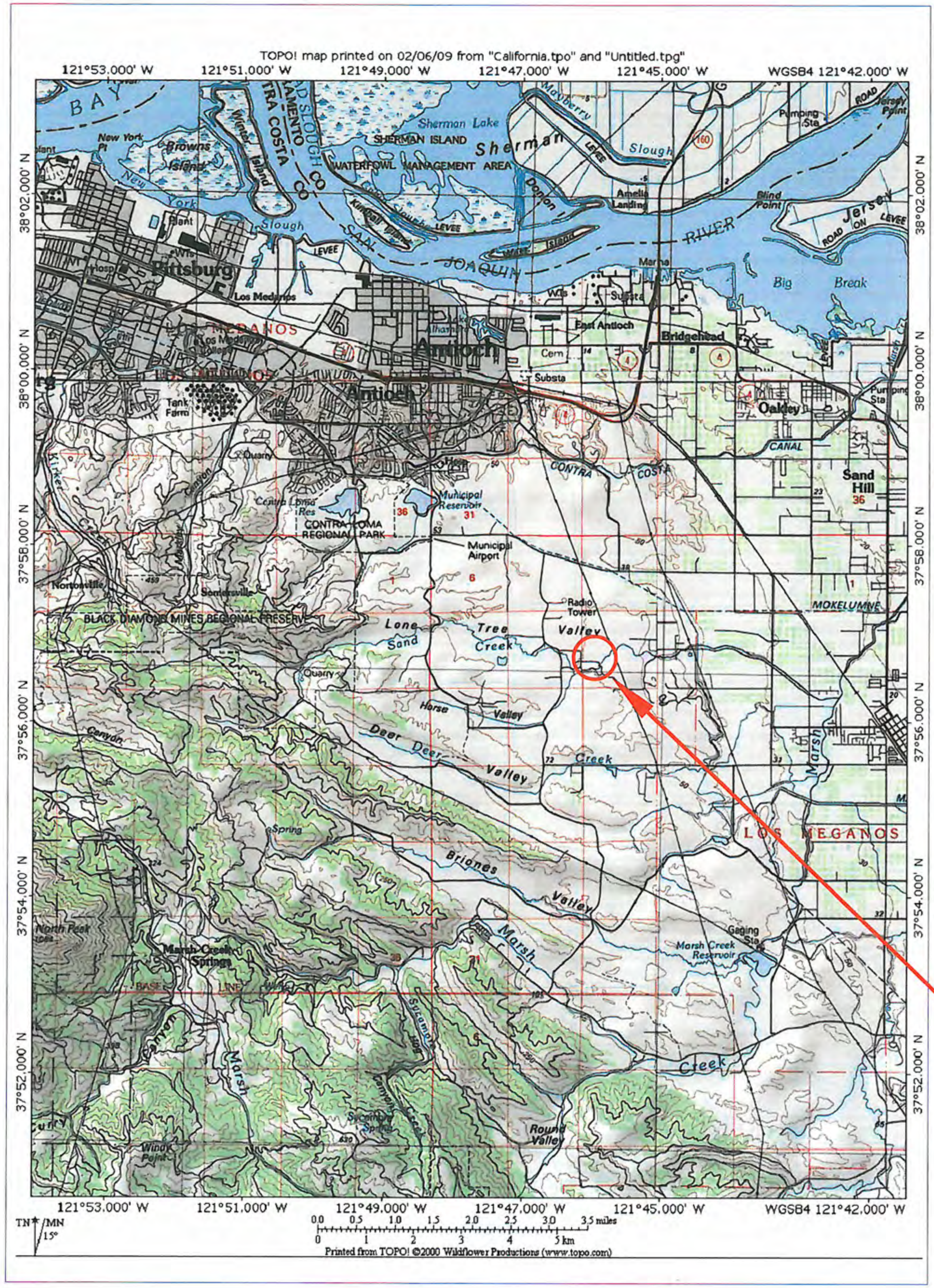
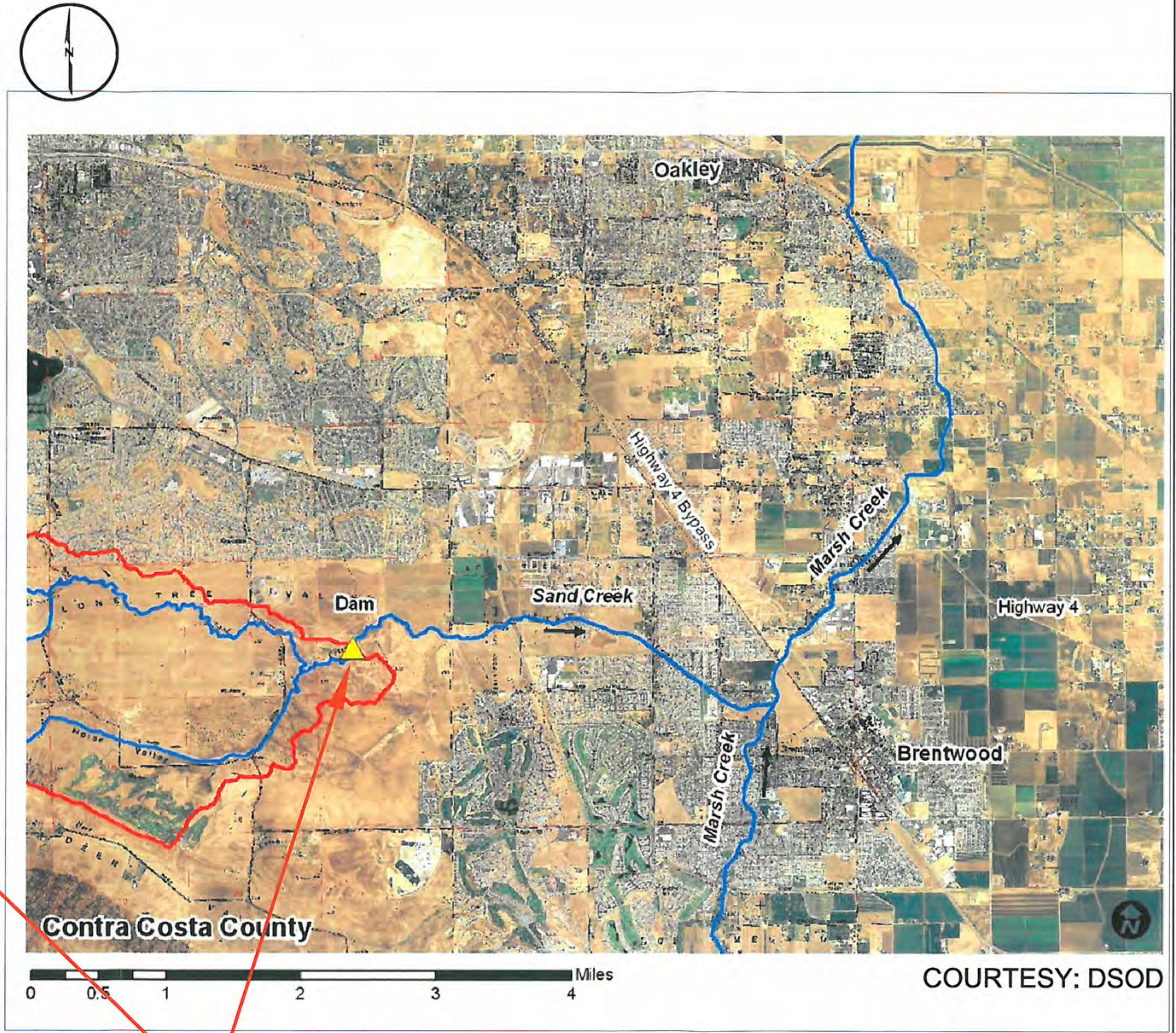
Figures

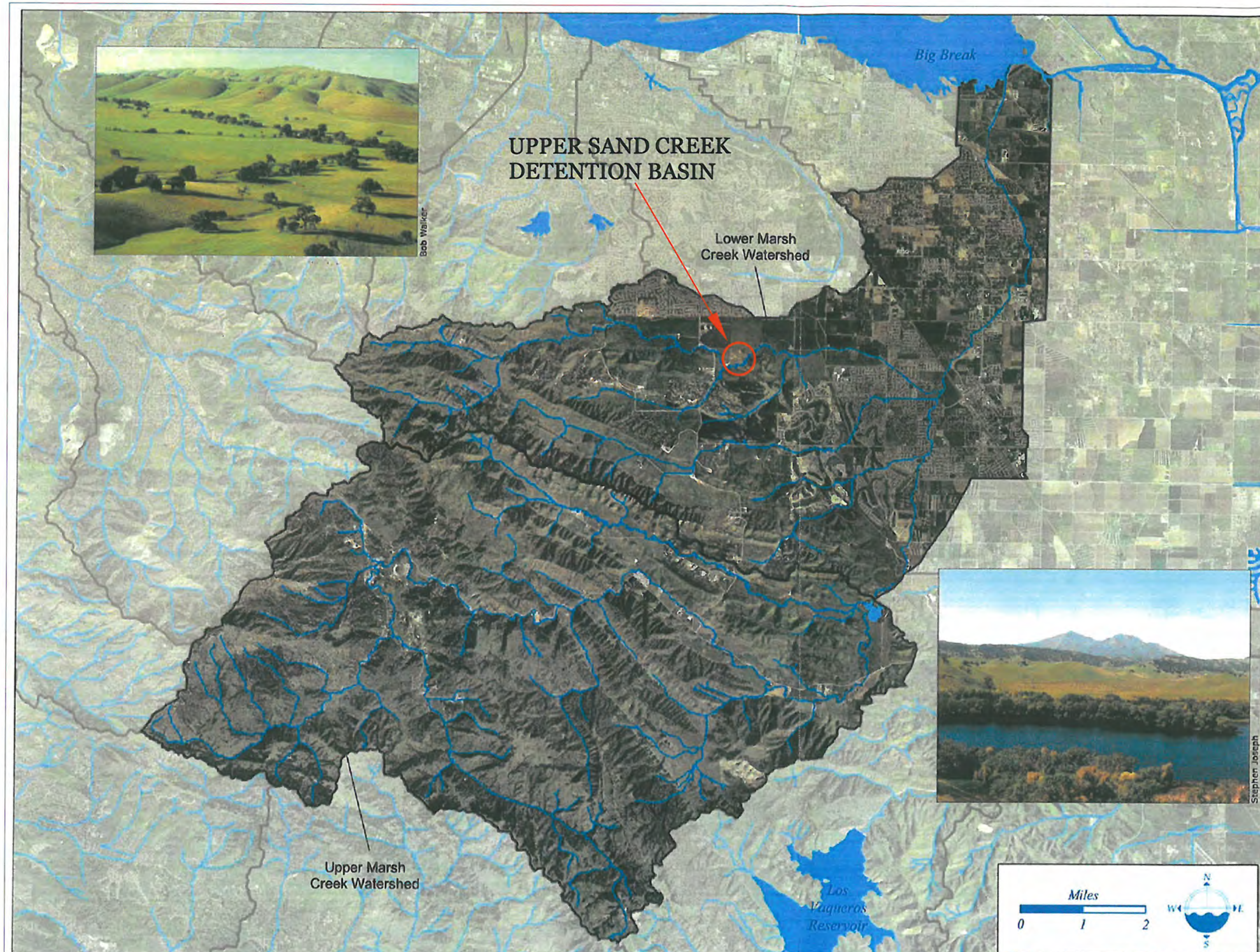




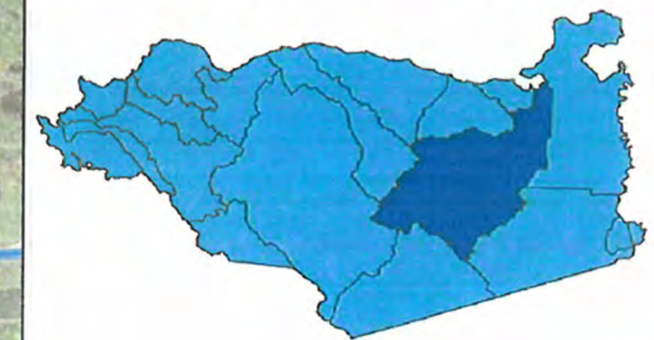
UPPER SAND CREEK
DETENTION BASIN

GEI Consultants	CONTRA COSTA COUNTY PUBLIC WORKS DEPARTMENT 255 GLACIER DRIVE MARTINEZ, CALIFORNIA 94553 GEI Project 083180	UPPER SAND CREEK DETENTION BASIN SITE VICINITY MAP	FIGURE 1

Figure 1 07-12-10 MP



Chapter 13
Marsh Creek Watershed



With its headwaters in the Morgan Territory (Mount Diablo foothills) on the eastern and flanks of Mount Diablo, Marsh Creek flows 34.57 miles before exiting into the San Joaquin River Delta at Big Break. The second largest watershed in the County, it encompasses 60,066 acres in eastern Contra Costa County. Tributaries in the upper watershed include Curry Canyon Creek (5.8 miles) and Sycamore Creek (4 miles) and lastly, Briones Creek (13 miles), which flows into the Marsh Creek Reservoir. Tributaries entering the middle portion of the main stem near and in the City of Brentwood include, Dry Creek (5.8 miles), Sand Creek (18.74 miles) and Dear Creek (9 miles).

Marsh Creek Watershed Vital Statistics	
Watershed Size	60,066 acres
Length of Longest Branch of Creek	34.57 miles
Total Channel Length in Watershed	167.18 miles
Average Annual Rainfall	17 inches
Estimated Mean Daily Flow	28.3 cfs
Estimated 100-Year Flood Flow	5,740 cfs*
Highest Elevation in Watershed	3849 feet
Population (estimated)	38,500 people
Estimated Percent Impervious	15 %
Recognized Pollutants of Concern	Mercury & Metals**

*Above Marsh Creek Reservoir (34,900 acres upstream, or 77% of watershed). Downstream, 100yr flow is: 1,490 cfs below dam; and 2,720 cfs at mouth.

**Marsh Creek Reservoir, Dunn Creek and portions of Marsh Creek are listed as an Impaired Water Bodies in the State's 303(d) list. Pollutants of Concern are the following: Mercury (Reservoir, Dunn Creek and Marsh Creek below Reservoir) and Metals (Dunn Creek and Marsh Creek below Dunn).

SOURCE: CONTRA COSTA COUNTY WATERSHED ATLAS, 2004

GEI Consultants	CONTRA COSTA COUNTY PUBLIC WORKS DEPARTMENT 255 GLACIER DRIVE MARTINEZ, CALIFORNIA 94553	UPPER SAND CREEK DETENTION BASIN	
	GEI Project 083180	MARSH CREEK WATERSHED	FIGURE 2

Figure 2 07-12-10 MP

=Surficial faults evaluated for the site

=Blind faults evaluated for the site

0 10 20 30 Miles

0 10 20 30 40 50 Kilometers

Legal Delta Boundary V. 2002-4

Surficial faults

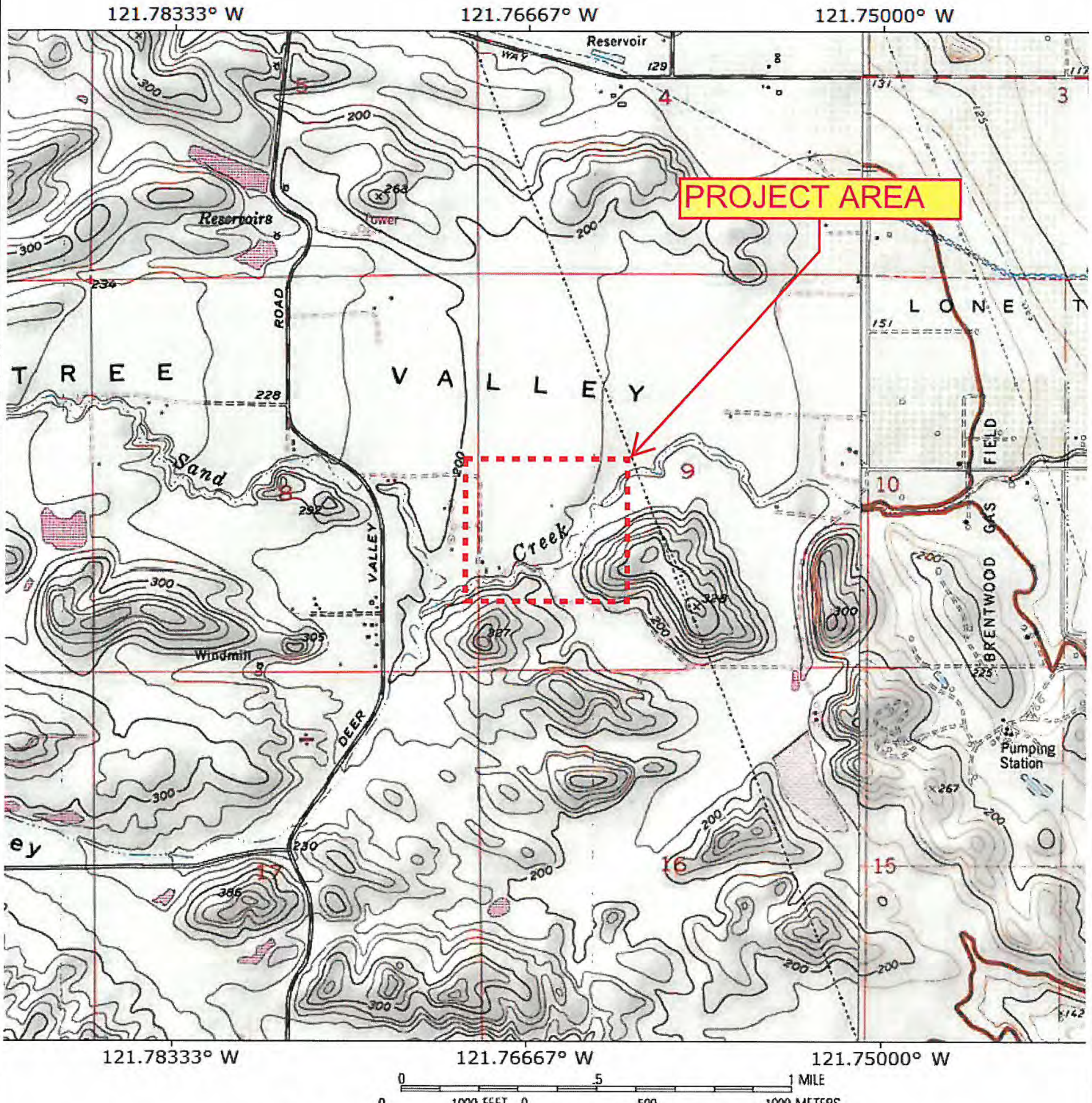
Blind faults

Bounds of delta islands

CRSB - Coast Range Sierran Block

GEI Consultants	CONTRA COSTA COUNTY PUBLIC WORKS DEPARTMENT 255 GLACIER DRIVE MARTINEZ, CALIFORNIA 94553	UPPER SAND CREEK DETENTION BASIN	
	GEI Project 083180	NEARBY ACTIVE FAULTS	FIGURE 3

Selected Figures from CE&G's Geotechnical Data Report (CE&G, May 2010)



Map created with TOPO!® ©2003 National Geographic (www.nationalgeographic.com/topo)

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UPPER SAND CREEK BASIN
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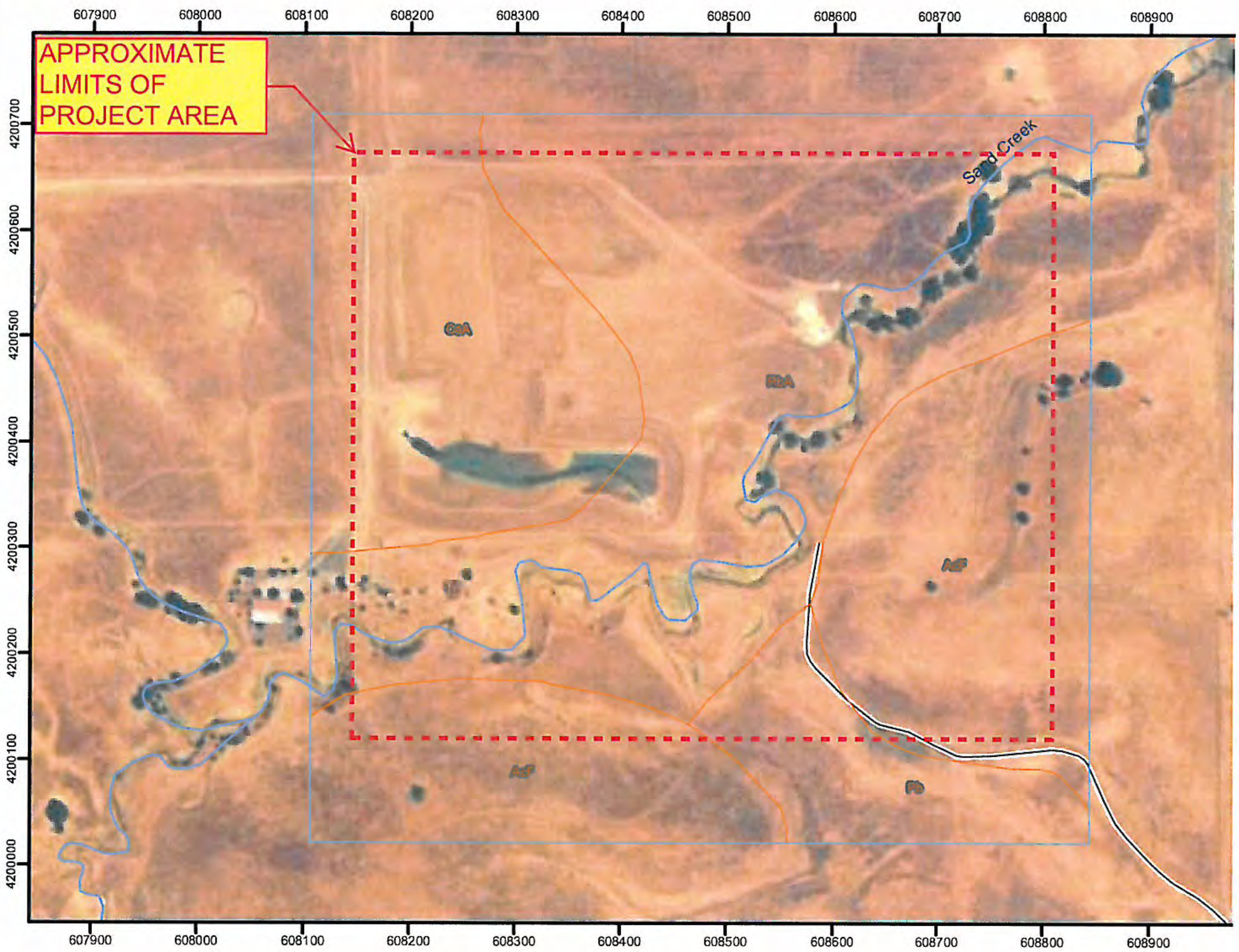
MARTINEZ, CALIFORNIA

TOPOGRAPHY

JOB NO. 081070	APRIL 2010	FIGURE 4



121° 46' 21"

N

Map Scale: 1:6,020 if printed on A size (8.5" x 11") sheet.

0 50 100 200 300 Meters

0 300 600 1,200 1,800 Feet

USDA Natural Resources Conservation Service

Web Soil Survey
National Cooperative Soil Survey

Map Unit Symbol	Map Unit Name
AcF	ALTAMONT-FONTANA COMPLEX, 30 TO 50 PERCENT SLOPES
CaA	CAPAY CLAY, 0 TO 2 PERCENT SLOPES
Pb	PESCADERO CLAY LOAM
RbA	RINCON CLAY LOAM, 0 TO 2 PERCENT SLOPES

FROM NRCS WEB SOIL SURVEY, http://websoilsurvey.nrcs.usda.gov (2010)

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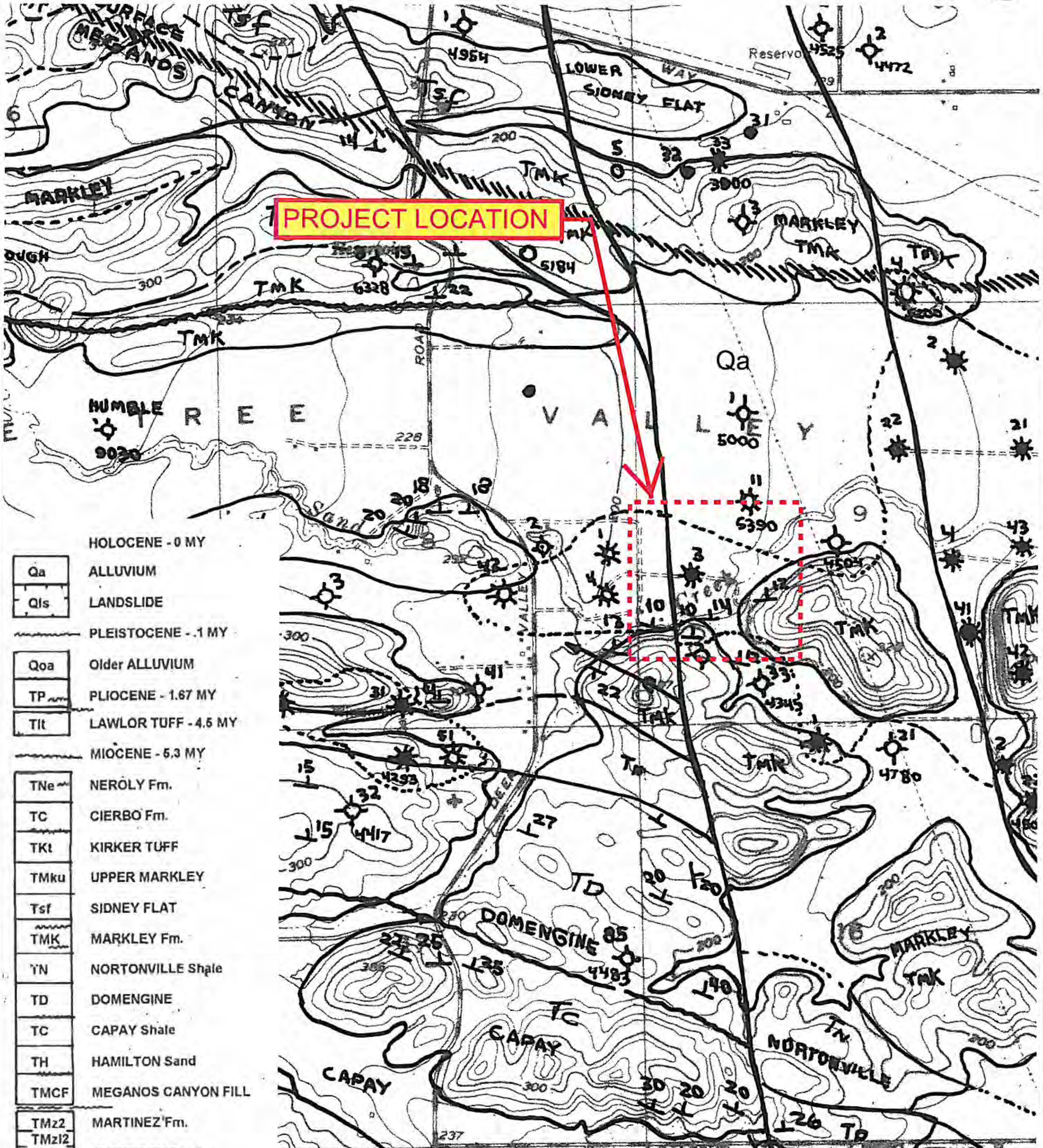
SURFICIAL SOILS

JOB NO. 081070	APRIL 2010	FIGURE 5



	HOLOCENE - 0 MY
Qa	ALLUVIUM
Qls	LANDSLIDE
	PLEISTOCENE - .1 MY
Qoa	Older ALLUVIUM
TP	PLIOCENE - 1.67 MY
Tlt	LAWLOR TUFF - 4.5 MY
	MIOCENE - 5.3 MY
TNe	NEROLY Fm.
TC	CIERBO Fm.
TKt	KIRKER TUFF
TMku	UPPER MARKLEY
Tsf	SIDNEY FLAT
TMK	MARKLEY Fm.
TN	NORTONVILLE Shale
TD	DOMENGINE
TC	CAPAY Shale
TH	HAMILTON Sand
TMCF	MEGANOS CANYON FILL
TMz2 / TMzl2	MARTINEZ Fm.
TA	ANDERSON Sand
TMz	
TMzl	

NO SCALE

FROM CRANE (1995)

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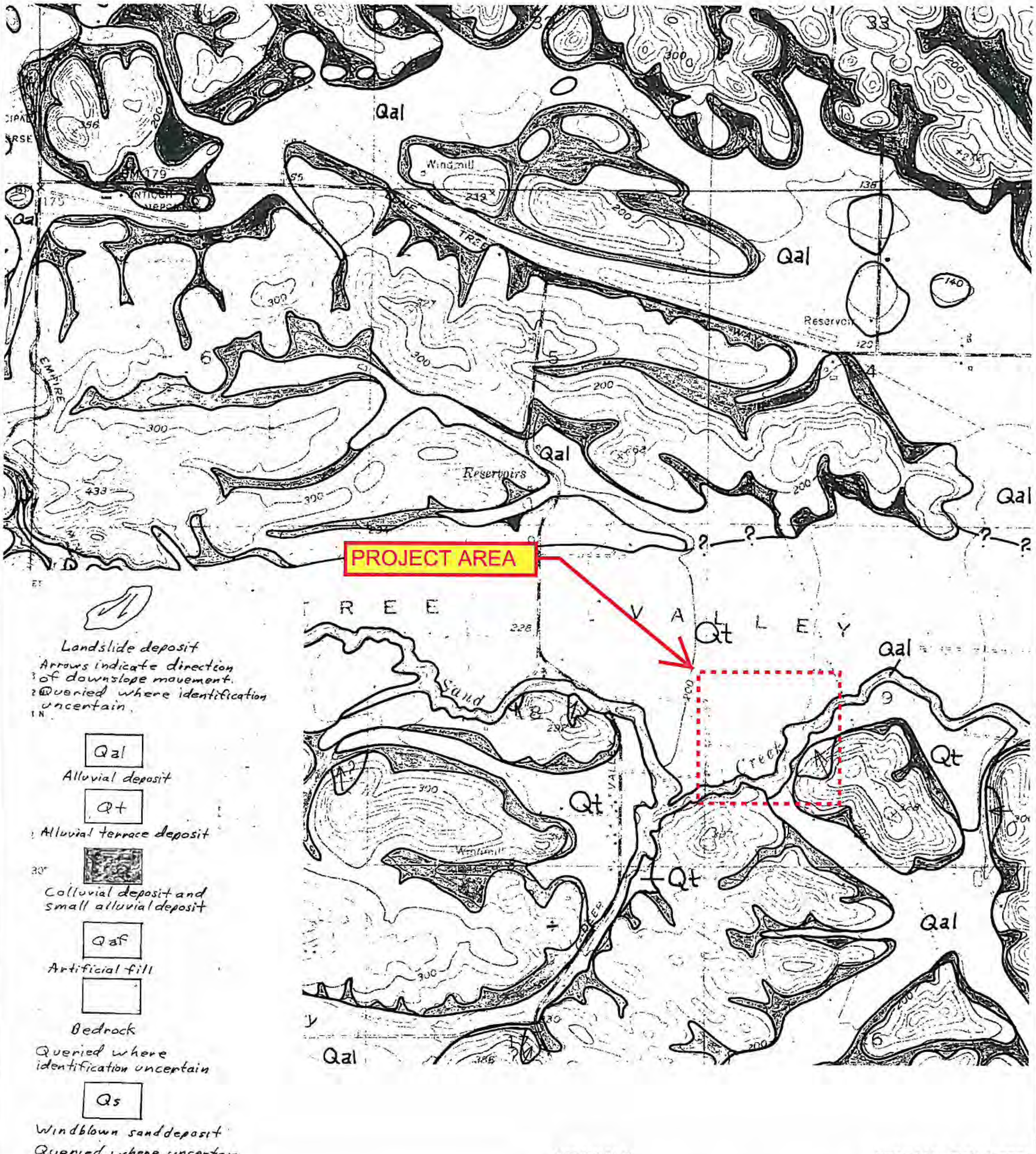
REGIONAL GEOLOGY

JOB NO. 081070	APRIL 2010	FIGURE 6



NO SCALE

FROM NILSEN (1975)

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UPPER SAND CREEK BASIN
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MARTINEZ, CALIFORNIA

LANDSLIDE FEATURES

JOB NO. 081070	APRIL 2010	FIGURE 7

BASEMAP REFERENCE

BASEMAP DEVELOPED FROM PLAN TITLED, "UPPER SAND CREEK DETENTION BASIN SITE PLAN," PREPARED BY GEI CONSULTANTS, DATED 8 OCTOBER 2009, NO JOB NUMBER. PREVIOUS EXPLORATION LOCATIONS DEVELOPED FROM REPORT TITLED, "GEOTECHNICAL STUDY, UPPER SAND CREEK DETENTION BASIN," PREPARED BY FUGRO WEST, DATED JUNE 2004.

LEGEND

- APPROXIMATE LOCATION OF TEST BORING (FUGRO, 2004)
- APPROXIMATE LOCATION OF CONE PENETROMETER TEST (FUGRO, 2004)
- APPROXIMATE LOCATION OF TEST PIT (FUGRO, 2000)
- APPROXIMATE LOCATION OF ROCK CORE (WLA, 2004)
- APPROXIMATE LOCATION OF TEST TRENCH (WLA, 2004)
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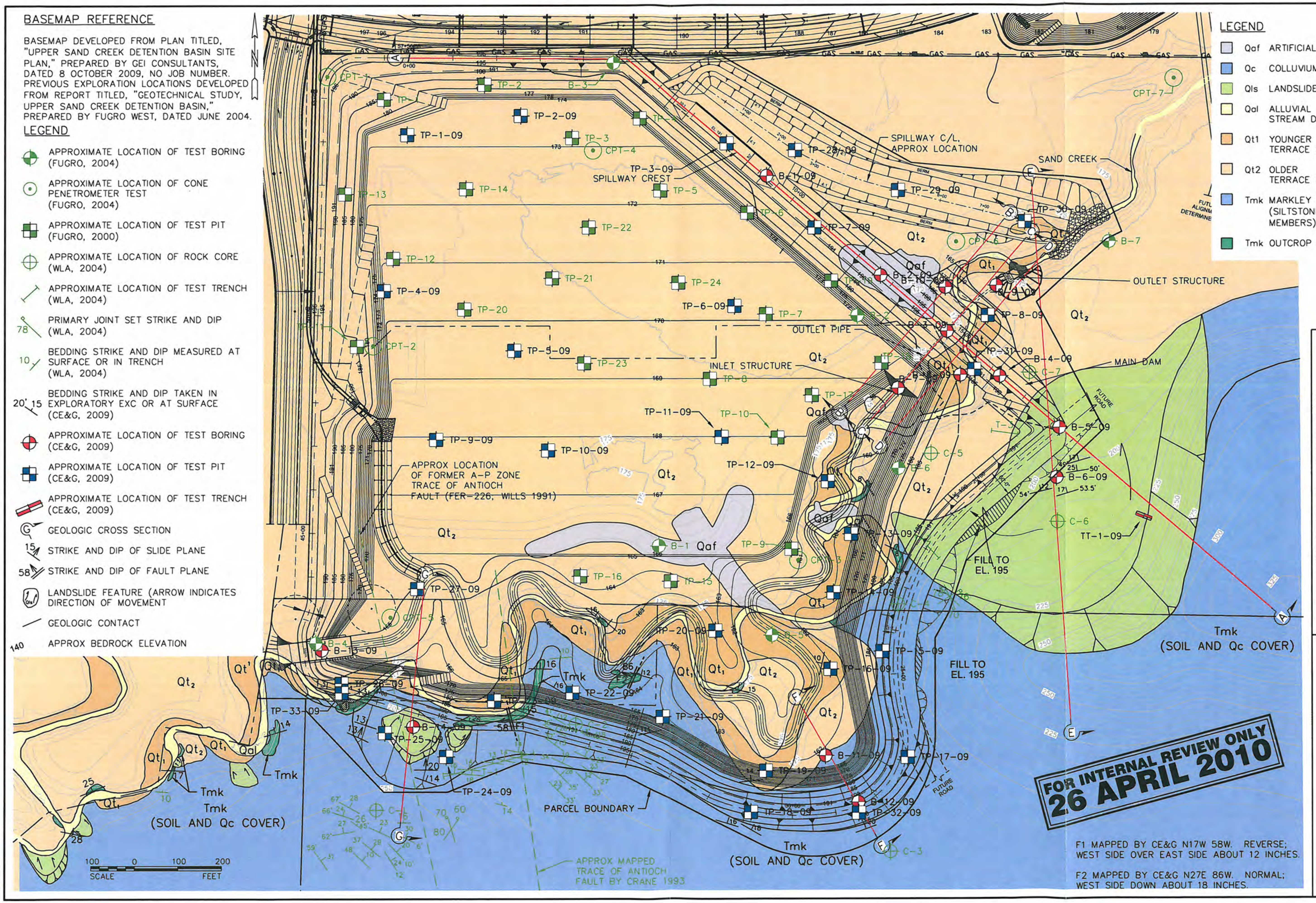
- 10 BEDDING STRIKE AND DIP MEASURED AT SURFACE OR IN TRENCH (WLA, 2004)
- 20' 15 BEDDING STRIKE AND DIP TAKEN IN EXPLORATORY EXC OR AT SURFACE (CE&G, 2009)
- APPROXIMATE LOCATION OF TEST BORING (CE&G, 2009)
- APPROXIMATE LOCATION OF TEST PIT (CE&G, 2009)
- APPROXIMATE LOCATION OF TEST TRENCH (CE&G, 2009)
- G GEOLOGIC CROSS SECTION
- 15 STRIKE AND DIP OF SLIDE PLANE
- 58 STRIKE AND DIP OF FAULT PLANE
- LANDSLIDE FEATURE (ARROW INDICATES DIRECTION OF MOVEMENT)
- GEOLOGIC CONTACT
- 140 APPROX BEDROCK ELEVATION



LEGEND

Qaf	ARTIFICIAL FILL
Qc	COLLUVIUM
Qls	LANDSLIDE DEBRIS
Qal	ALLUVIAL STREAM DEPOSIT
Qt1	YOUNGER TERRACE DEPOSIT
Qt2	OLDER TERRACE DEPOSIT
Tmk	MARKLEY FORMATION (SILTSTONE, SANDSTONE MEMBERS)
Tmk	OUTCROP FEATURE

F1 MAPPED BY CE&G N17W 58W. REVERSE; WEST SIDE OVER EAST SIDE ABOUT 12 INCHES.

F2 MAPPED BY CE&G N27E 86W. NORMAL; WEST SIDE DOWN ABOUT 18 INCHES.

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UPPER SAND CREEK BASIN
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SUBSURFACE EXPLORATION & GEOLOGIC MAP

FIGURE 8

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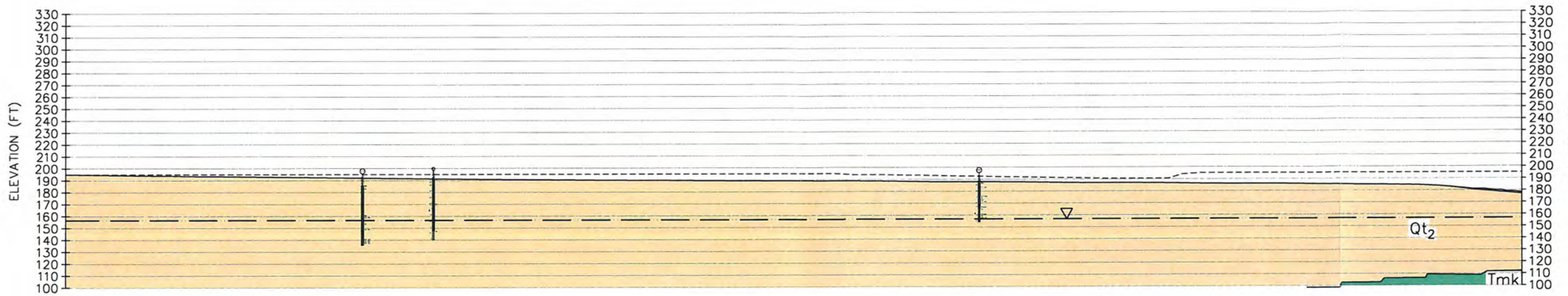

SECTION A

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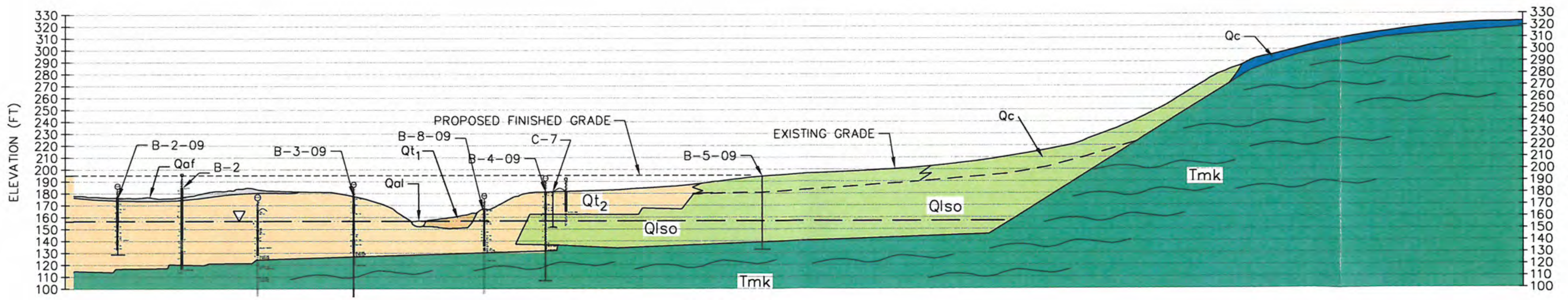
SCALE FEET



SECTION A

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SCALE FEET

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GEOLOGIC SECTION A

FIGURE 9

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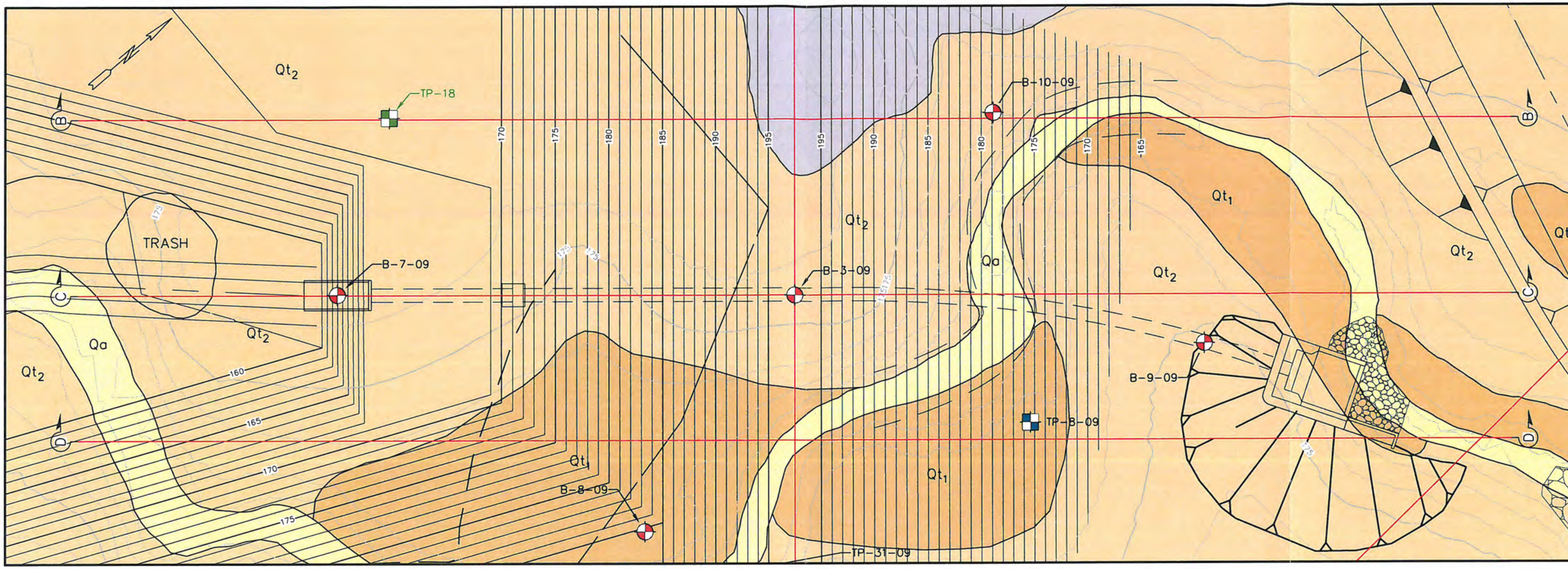

GEOLOGIC SECTIONS B, C, AND D

20 0 20 40
SCALE FEET

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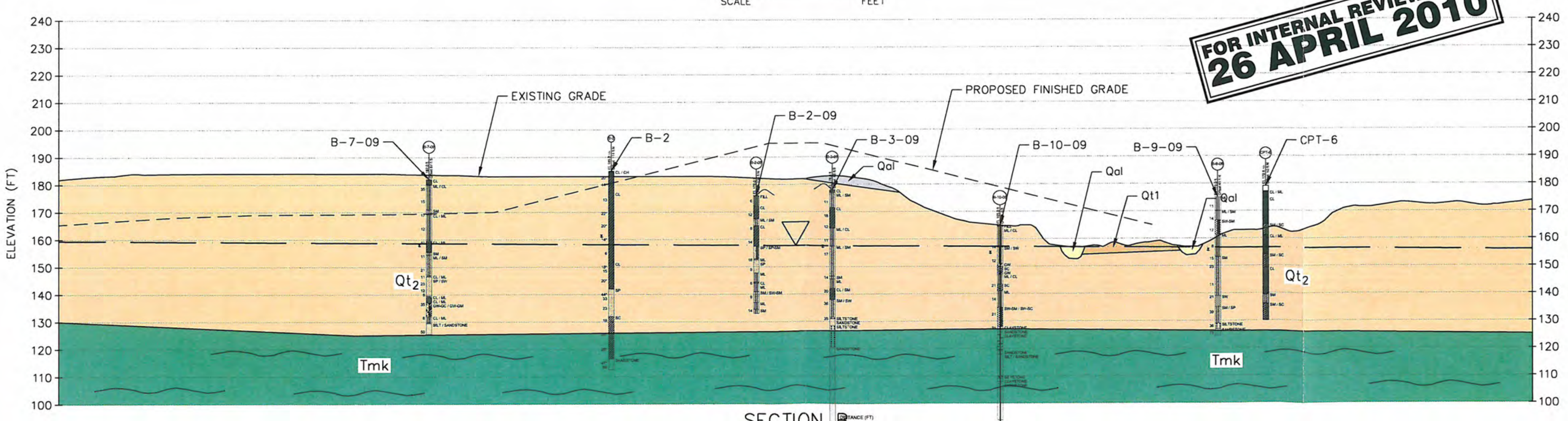

SECTION B

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SCALE FEET

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GEOLOGIC SECTION B

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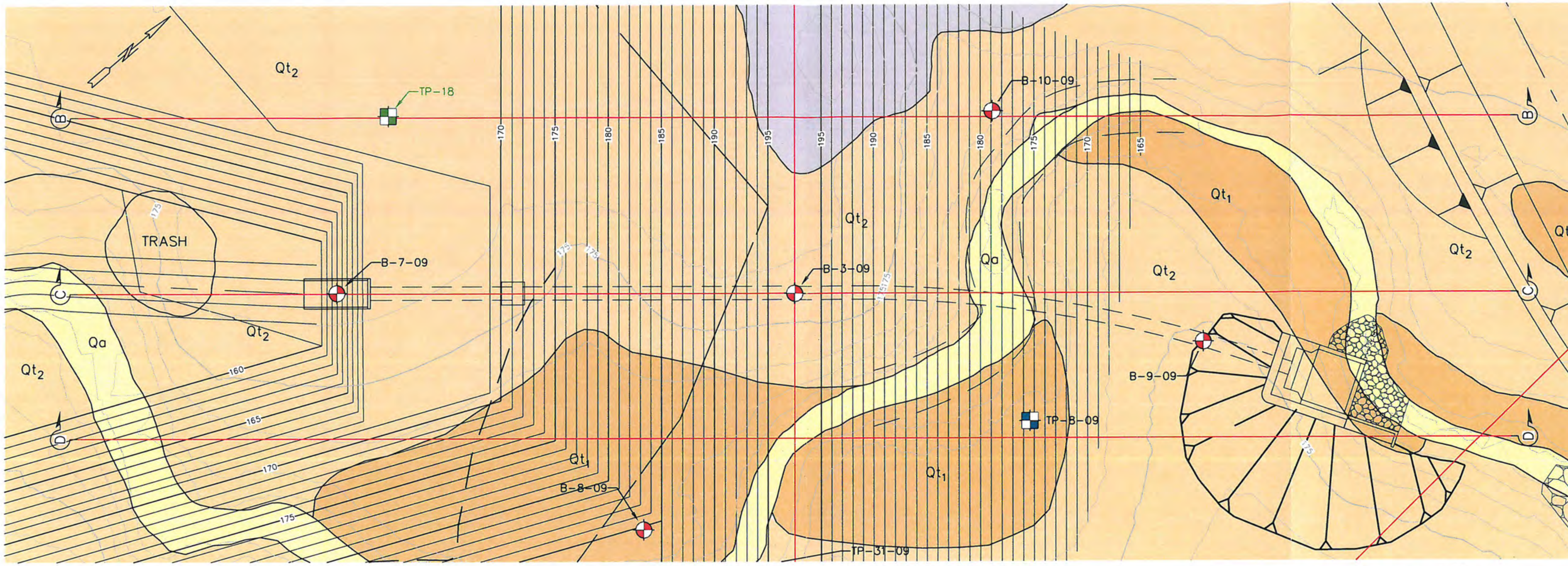

GEOLOGIC SECTIONS B, C, AND D

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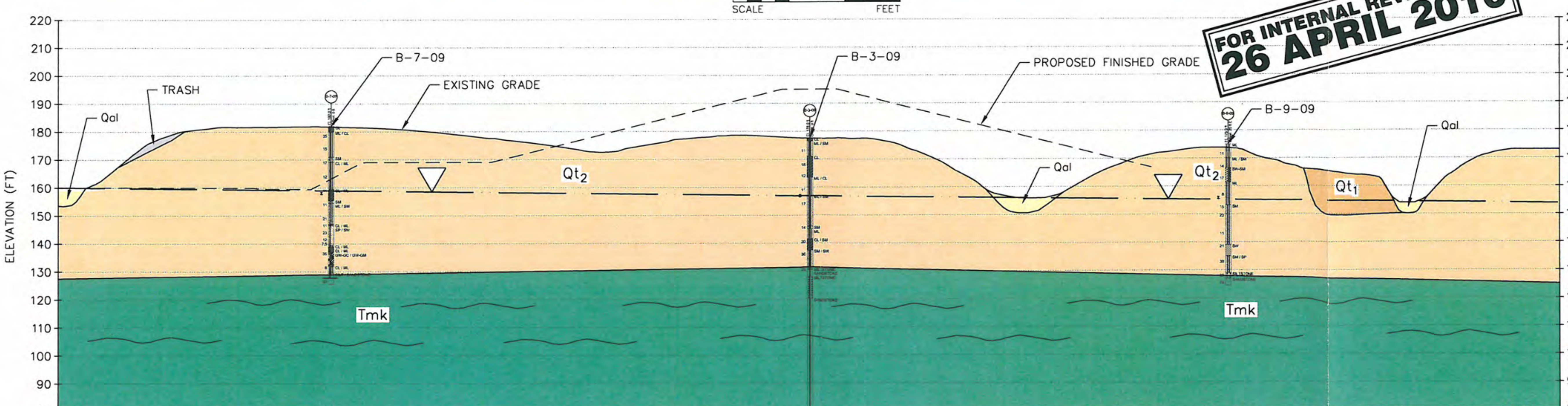
SCALE FEET



SECTION C

20 0 20 40
SCALE FEET

UPPER SAND CREEK BASIN
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GEOLOGIC SECTION C

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FIGURE 11

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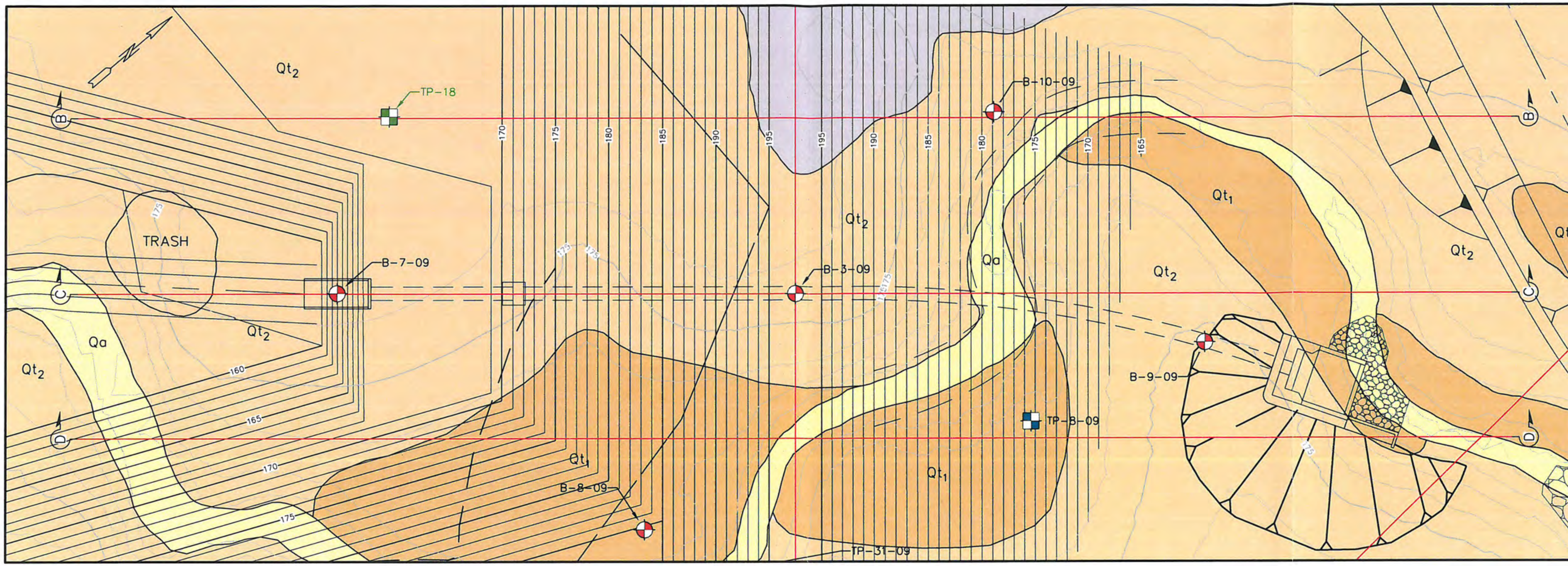

GEOLOGIC SECTIONS B, C, AND D

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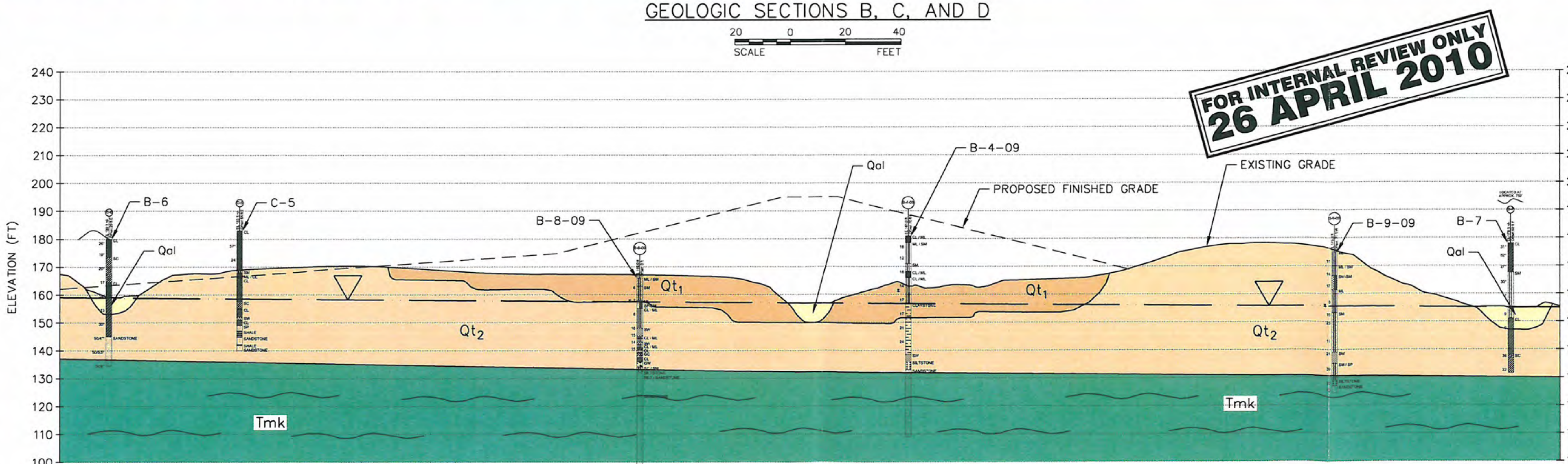
SCALE FEET



SECTION D

20 0 20 40
SCALE FEET

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GEOLOGIC SECTION D

FIGURE 12

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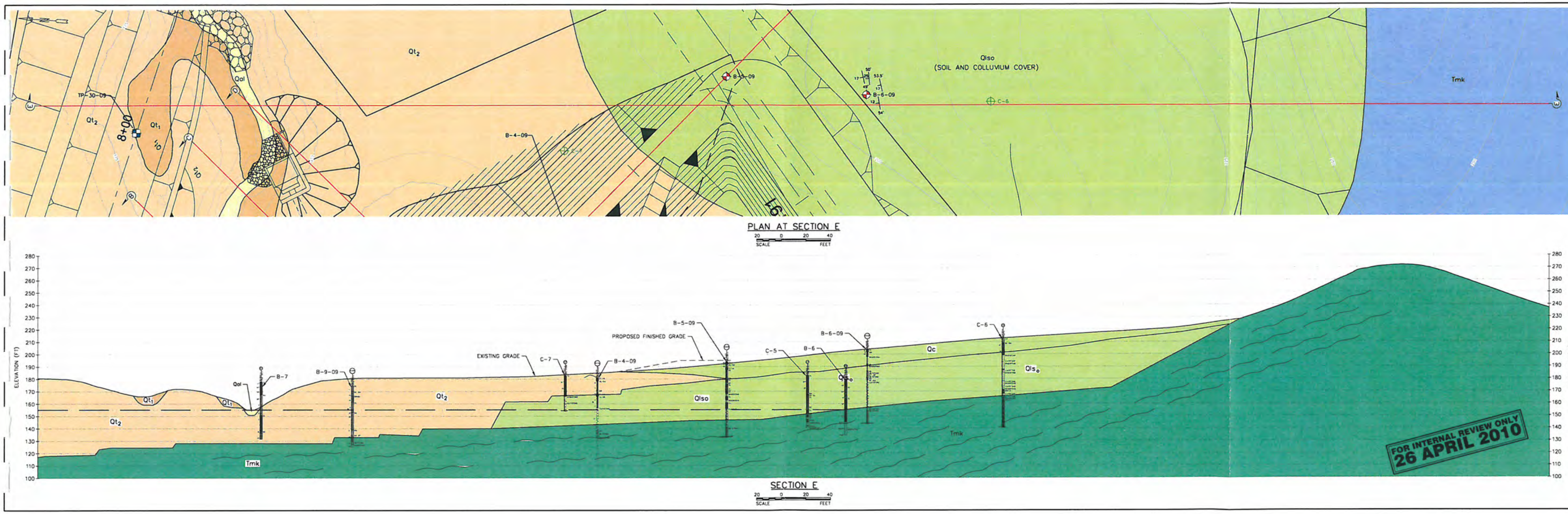

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GEOLOGIC CROSS SECTION E

081070 | 26 APRIL 2010 | FIGURE 13

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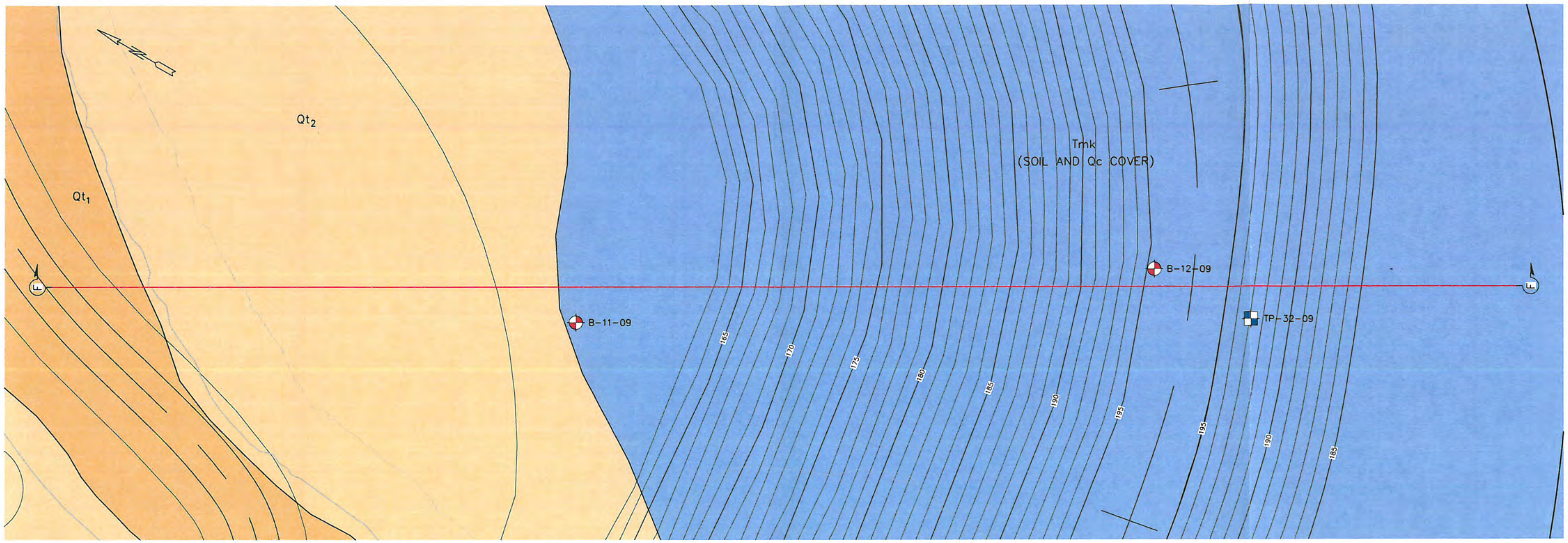
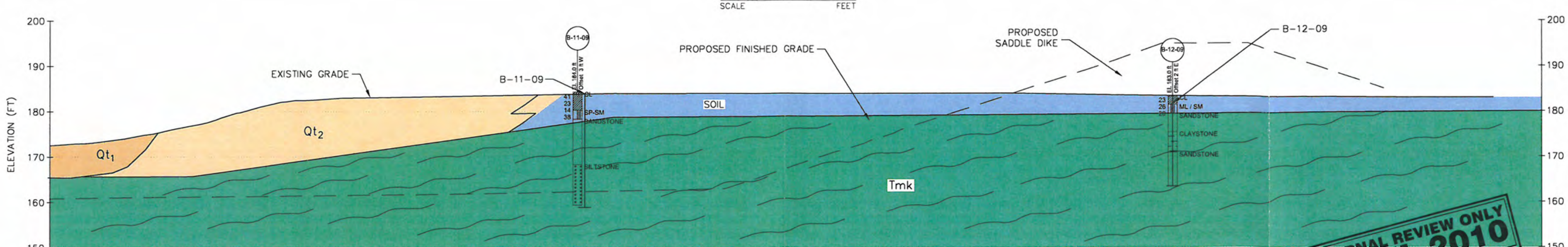


PLAN AT SECTION F



SECTION F

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UPPER SAND CREEK BASIN
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GEOLOGIC CROSS SECTION F

081070	26 APRIL 2010	FIGURE 14

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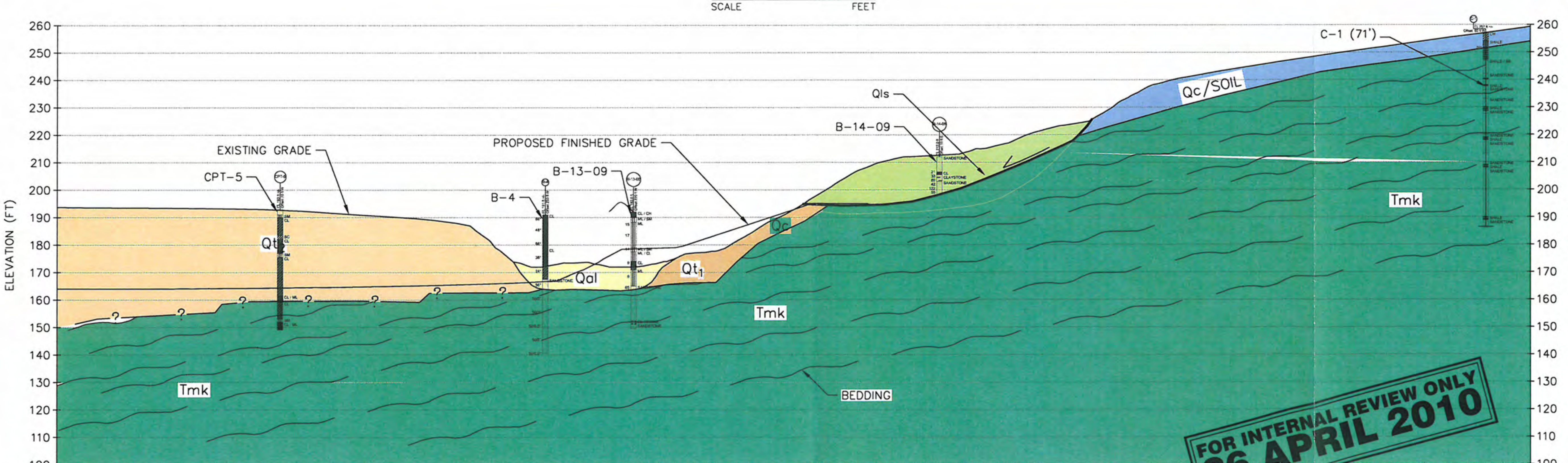
PLAN AT SECTION G

20 0 20 40
SCALE FEET



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26 APRIL 2010

SECTION G

20 0 20 40
SCALE FEET

UPPER SAND CREEK BASIN
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GEOLOGIC CROSS SECTION G

081070 | 26 APRIL 2010 | FIGURE 15

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Drawings

BASEMAP REFERENCE

BASEMAP DEVELOPED FROM PLAN SHEET C-1 TITLED, "UPPER SAND CREEK DETENTION BASIN, SITE PLAN," PREPARED BY GEI CONSULTANTS, DATED 3 AUGUST 2010, FILE NO. FD-3210. PREVIOUS EXPLORATION LOCATIONS DEVELOPED FROM REPORT TITLED, "GEOTECHNICAL STUDY, UPPER SAND CREEK DETENTION BASIN," PREPARED BY FUGRO WEST, DATED JUNE 2004.

LEGEND

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- GEOLOGIC CONTACT
- 140 APPROX BEDROCK ELEVATION

LEGEND

- Qaf ARTIFICIAL FILL
- Qc COLLUVIUM
- Qls LANDSLIDE DEBRIS
- Qal ALLUVIAL STREAM DEPOSIT
- Qt1 YOUNGER TERRACE DEPOSIT
- Qt2 OLDER TERRACE DEPOSIT
- Tmk MARKLEY FORMATION (SILTSTONE, SANDSTONE MEMBERS)
- Tmk OUTCROP FEATURE

F1 MAPPED BY CE&G N17W 58W. REVERSE; WEST SIDE OVER EAST SIDE ABOUT 12 INCHES.

F2 MAPPED BY CE&G N27E 86W. NORMAL; WEST SIDE DOWN ABOUT 18 INCHES.

DRAFT - 90% REVIEW

S-1

REVISIONS			
NO.	DESCRIPTION	BY	DATE

DES.: S. WATRY
DRAWN: C. HOCKETT
CHKD.: P. GREGORY
DATE: 08-16-10
SCALE: 1" = 100'
FLD. BK. XXXX

PROJECT ENGINEER

PLANS APPROVAL DATE

CE&G
CAL ENGINEERING & GEOLOGY

CONTRA COSTA COUNTY
PUBLIC WORKS DEPARTMENT
255 GLACIER DRIVE
MARTINEZ, CALIFORNIA 94553

0 100 200
SCALE, FEET

FOR REDUCED PLANS ORIGINAL SCALE IS IN INCHES
0 1 2 3

BASE MAP	EAST COORD.	NORTH COORD.
L23	1635	528

UPPER SAND CREEK DETENTION BASIN

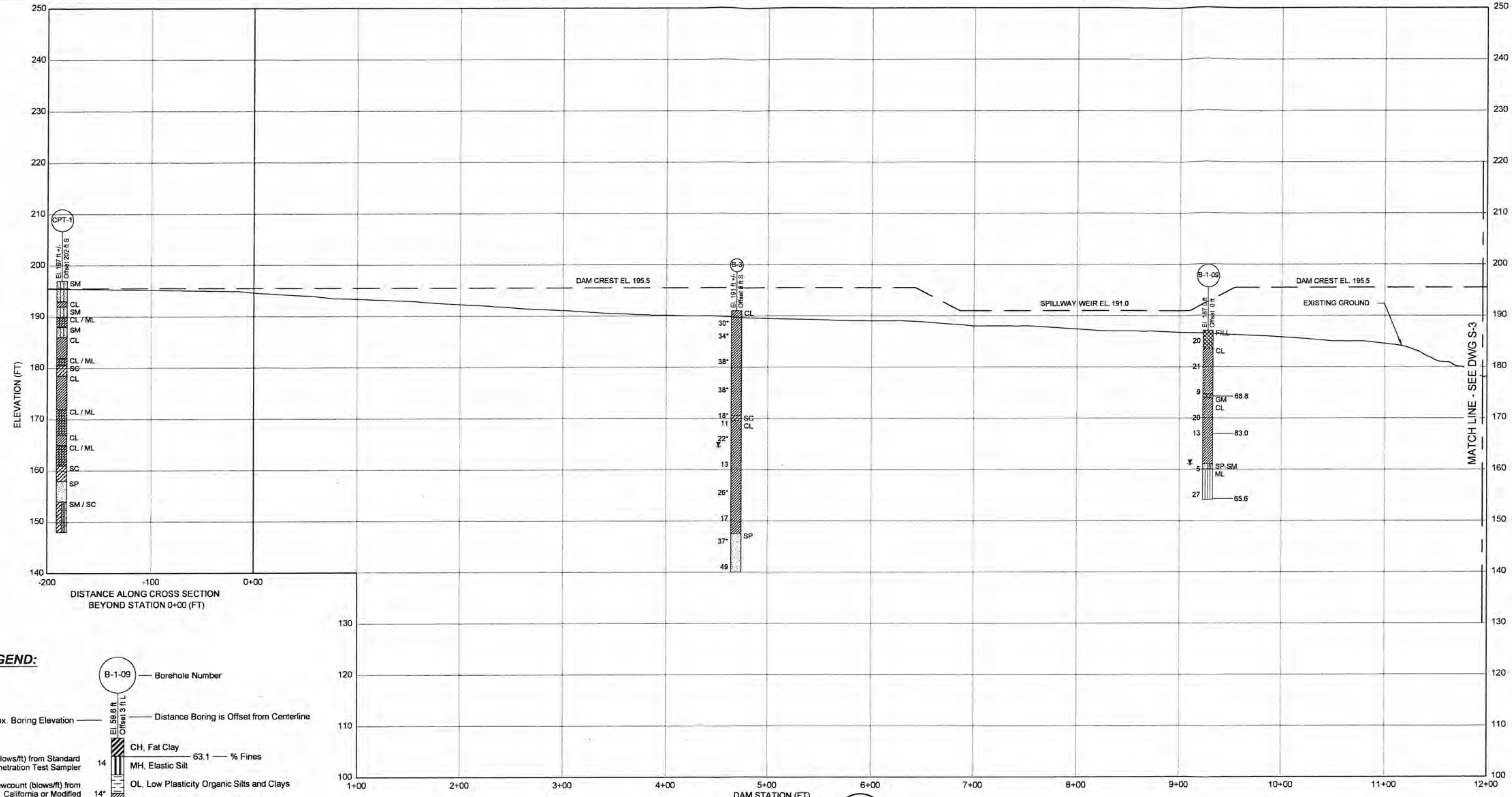
GEOLOGIC PLAN AND EXPLORATION LOCATIONS

FILE NO. FD-3210

SHEET XX OF YY

CADD FILE# S-1.dwg PEN TBL: GEIWALKER.ctb

S-1 08-16-10 CAH



SECTION A (1 of 2) S-1

LEGEND:

B-1-09 — Borehole Number

Approx. Boring Elevation — El. 59.6 ft

Offset 3 ft L — Distance Boring is Offset from Centerline

63.1 — % Fines

Blowcount (blows/ft) from Standard Penetration Test Sampler — 14

Blowcount (blows/ft) from California or Modified California Sampler — 14*

Represents refusal of the sampler — REF

Water Level Reading at time of drilling.

Water Level Reading after drilling.

Weight of Hammer — WOH

- CH, Fat Clay
- MH, Elastic Silt
- OL, Low Plasticity Organic Silts and Clays
- CL, Lean Clay or Sandy Lean Clay
- ML, Silt or Sandy Silt
- SC, Clayey SAND
- SM, Silty Sand, Silty Sand with Gravel
- SP, Poorly Graded Sand, Poorly Graded Sand with Gravel
- SW, Well Graded Sand, Well Graded Sand with Gravel
- GC, Clayey Gravel
- GM, Silty Gravel, Silty Gravel with Sand
- GP, Poorly Graded Gravel, Poorly Graded Gravel with Sand
- GW, Well Graded Gravel, Well Graded Gravel with Sand

NOTES:

1. BLOW COUNTS ON CE&G BORINGS (B-1-09 ETC.) ARE CORRECTED BASED ON TABER DRILLING ENERGY CALIBRATION IN ACCORDANCE WITH DWR AUGUST 2008 MANUAL.
2. BLOW COUNTS ON HISTORIC BORINGS (B-# AND C-#) ARE UNCORRECTED.
3. OFFSET DISTANCES SHOWN FOR EACH BORING REPRESENT THE PERPENDICULAR DISTANCE OF THE BORING TO THE SECTION LINE. OFFSET BORINGS ARE PROJECTED HORIZONTALLY (MAINTAINING ELEVATION).
4. LOGS SHOWN REPRESENT SUMMARIZED AND GENERALIZED SUBSURFACE CONDITIONS. REFER TO BORING LOGS FOR SPECIFIC SUBSURFACE CONDITIONS.
5. SUBSURFACE CONDITIONS SHOWN REPRESENT OBSERVATIONS AT THE SPECIFIC BORING LOCATIONS AT THE TIME THE EXPLORATIONS WERE COMPLETED. SUBSURFACE CONDITIONS BETWEEN BORING LOCATIONS AND SAMPLES ARE UNKNOWN. GROUNDWATER LEVELS ARE LIKELY TO VARY WITH TIME.

DRAFT - 90% REVIEW

S-2

REVISIONS			
NO.	DESCRIPTION	BY	DATE

DES.: DD
DRAWN: DD
CHKD.: JN
DATE: 08-16-10
SCALE: VARIES
FLD. BK.: XXXX

REGISTERED PROFESSIONAL ENGINEER ALBERTO PUJOL-RIOS No. 37051 EXP. CIVIL STATE OF CALIFORNIA

PROJECT ENGINEER

PLANS APPROVAL DATE

GEI Consultants

CONTRA COSTA COUNTY PUBLIC WORKS DEPARTMENT
255 GLACIER DRIVE
MARTINEZ, CALIFORNIA 94553

0 10 20 VERTICAL SCALE, FEET

0 50 100 HORIZONTAL SCALE, FEET

FOR REDUCED PLANS ORIGINAL SCALE IS IN INCHES
0 1 2 3

BASE MAP	EAST COORD	NORTH COORD
L23	1635	528

UPPER SAND CREEK DETENTION BASIN

GEOLOGIC PROFILE ALONG DAM AXIS (1 OF 2)

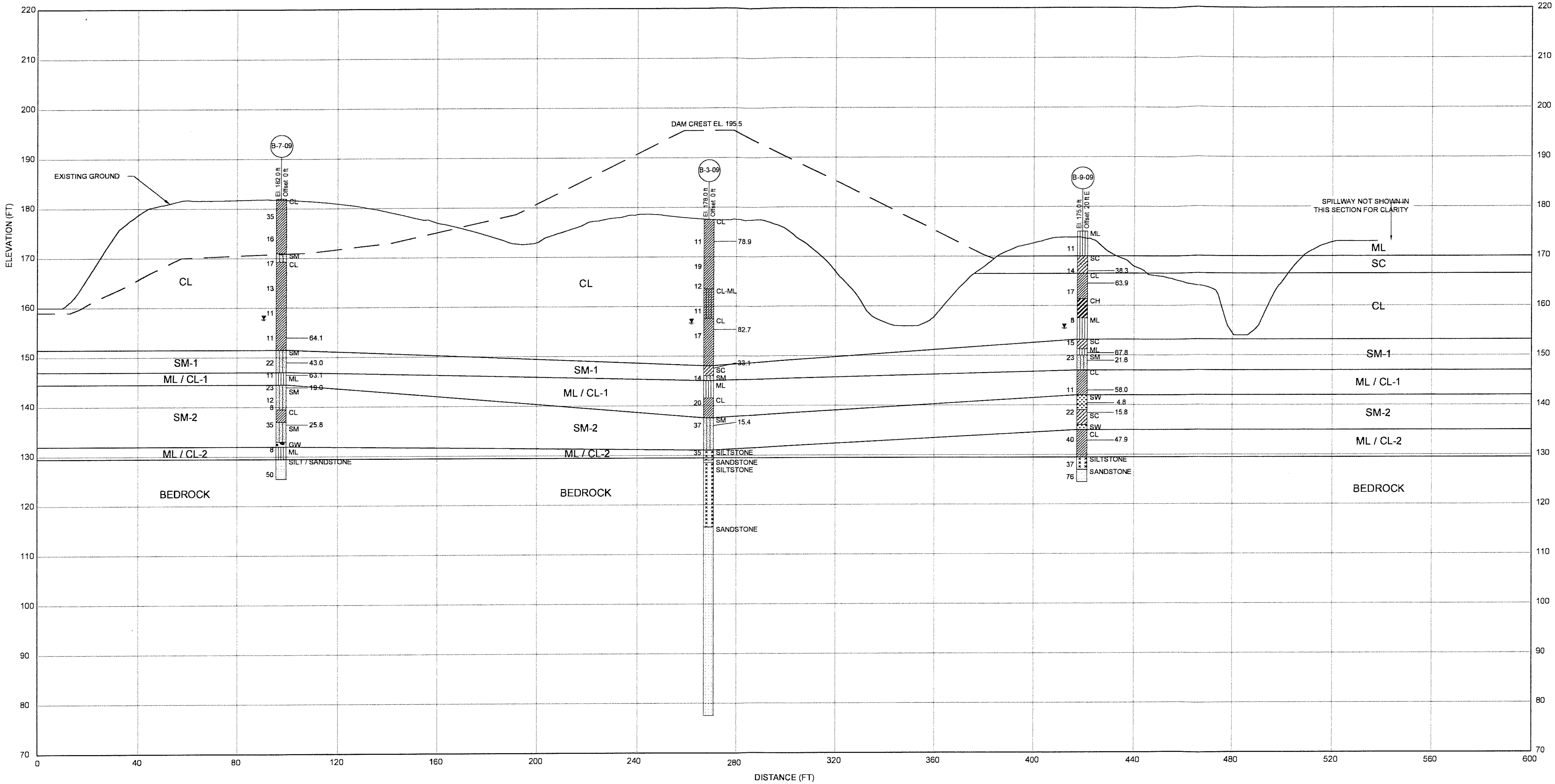
FILE NO. FD-3210

SHEET XX OF YY

CADD FILE# S-2.dwg

PEN TBL: GEI Boston Standard 2009.ctb

S-2 08-16-10 D.DeCesaris



NOTES:

1. BLOW COUNTS ON CE&G BORINGS (B-1-09 ETC.) ARE CORRECTED BASED ON TABER DRILLING ENERGY CALIBRATION IN ACCORDANCE WITH DWR AUGUST 2008 MANUAL.
2. BLOW COUNTS ON HISTORIC BORINGS (B-# AND C-#) ARE UNCORRECTED.
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SECTION C / S-1
STATION 14+70

LEGEND:

SEE DRAWING S-2.

DRAFT - 90% REVIEW

S-5

REVISIONS			
NO.	DESCRIPTION	BY	DATE

DES.: DD
DRAWN: DD
CHKD.: JN
DATE: 08-16-10
SCALE: VARIES
FLD. BK.: XXXX

REGISTERED PROFESSIONAL ENGINEER — ALBERTO PUJOL-RIUS — No. 37051 — EXP — CIVIL — STATE OF CALIFORNIA

PROJECT ENGINEER

PLANS APPROVAL DATE

GEI Consultants

CONTRA COSTA COUNTY
PUBLIC WORKS DEPARTMENT
255 GLACIER DRIVE
MARTINEZ, CALIFORNIA 94553

VERTICAL SCALE, FEET: 0 10 20
HORIZONTAL SCALE, FEET: 0 20 40

FOR REDUCED PLANS ORIGINAL SCALE IS IN INCHES: 0 1 2 3

BASE MAP	EAST COORD.	NORTH COORD.
L23	1635	528

UPPER SAND CREEK DETENTION BASIN

GEOLOGIC SECTIONS (2 OF 7)

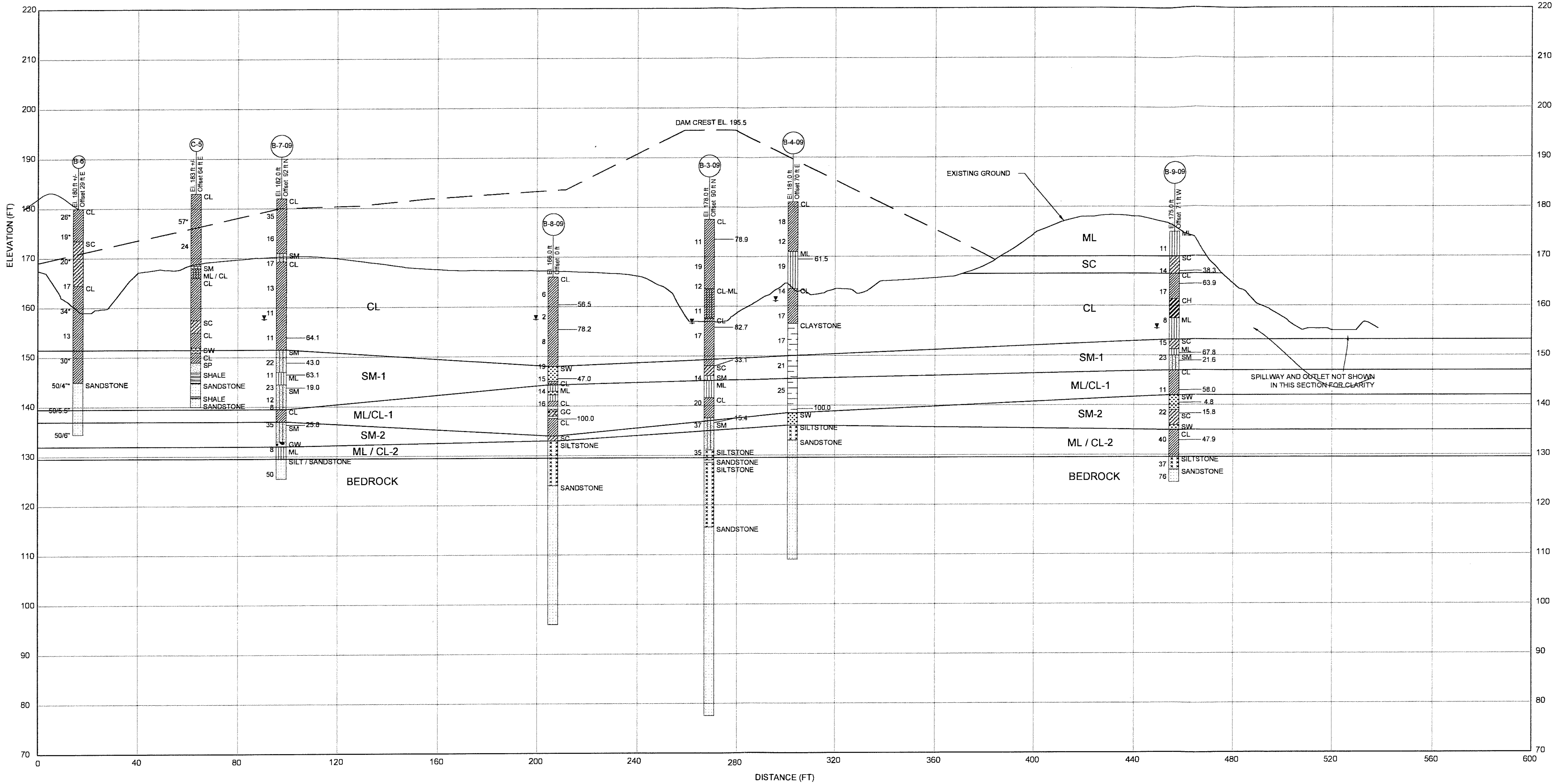
FILE NO. FD-3210

SHEET XX OF YY

CADD FILE# S-5.dwg

PEN TBL. GEI Boston Standard 2009.ctb

S-5 08-16-10 D.DeCesaris



NOTES:

1. BLOW COUNTS ON CE&G BORINGS (B-1-09 ETC.) ARE CORRECTED BASED ON TABER DRILLING ENERGY CALIBRATION IN ACCORDANCE WITH DWR AUGUST 2008 MANUAL.
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SECTION D
STATION 15+55 S-1

LEGEND:

SEE DRAWING S-2.

DRAFT - 90% REVIEW

S-6

REVISIONS			
NO.	DESCRIPTION	BY	DATE

DES.: DD
DRAWN: DD
CHKD.: JN
DATE: 08-16-10
SCALE: VARIES
FLD. BK. XXXX

REGISTERED PROFESSIONAL ENGINEER — ALBERTO PUJOL-RIUS — No. 37051 — EXP — CIVIL — STATE OF CALIFORNIA

PROJECT ENGINEER

PLANS APPROVAL DATE

GEI Consultants

CONTRA COSTA COUNTY
PUBLIC WORKS DEPARTMENT
255 GLACIER DRIVE
MARTINEZ, CALIFORNIA 94553

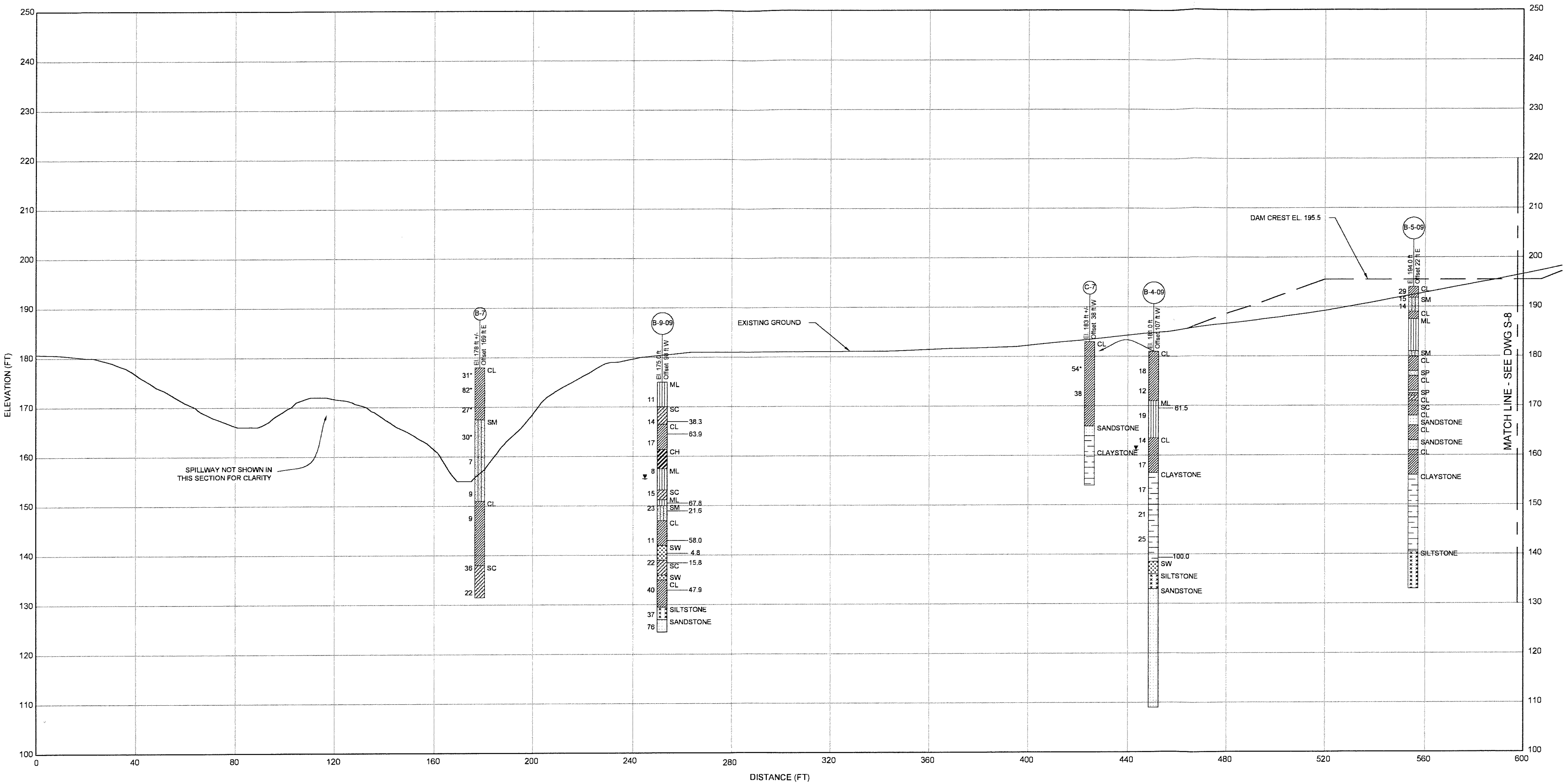
VERTICAL SCALE, FEET: 0 10 20
HORIZONTAL SCALE, FEET: 0 20 40

FOR REDUCED PLANS ORIGINAL SCALE IS IN INCHES: 0 1 2 3

BASE MAP	EAST COORD.	NORTH COORD
L23	1635	528

UPPER SAND CREEK DETENTION BASIN
GEOLOGIC SECTIONS (3 OF 7)
FILE NO. FD-3210
SHEET XX OF YY
CADD FILE# S-6.dwg
PEN TBL. GEI Boston Standard 2009.ctb

S-6 08-16-10 D.DeCesaris



NOTES:

1. BLOW COUNTS ON CE&G BORINGS (B-1-09 ETC.) ARE CORRECTED BASED ON TABER DRILLING ENERGY CALIBRATION IN ACCORDANCE WITH DWR AUGUST 2008 MANUAL.
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SECTION E (1 of 2) S-1

LEGEND:

SEE DRAWING S-2.

DRAFT - 90% REVIEW

S-7

REVISIONS			
NO.	DESCRIPTION	BY	DATE

DES.: DD
DRAWN: DD
CHKD.: JN
DATE: 08-16-10
SCALE: VARIES
FLD. BK. XXXX

REGISTERED PROFESSIONAL ENGINEER ALBERTO PUJOL-RIUS No. 37051 EXP CIVIL STATE OF CALIFORNIA

PROJECT ENGINEER

PLANS APPROVAL DATE

GEI Consultants

CONTRA COSTA COUNTY
PUBLIC WORKS DEPARTMENT
255 GLACIER DRIVE
MARTINEZ, CALIFORNIA 94553

VERTICAL SCALE, FEET 0 10 20

HORIZONTAL SCALE, FEET 0 20 40

FOR REDUCED PLANS ORIGINAL SCALE IS IN INCHES 0 1 2 3

BASE MAP	EAST COORD	NORTH COORD
L23	1635	528

UPPER SAND CREEK DETENTION BASIN

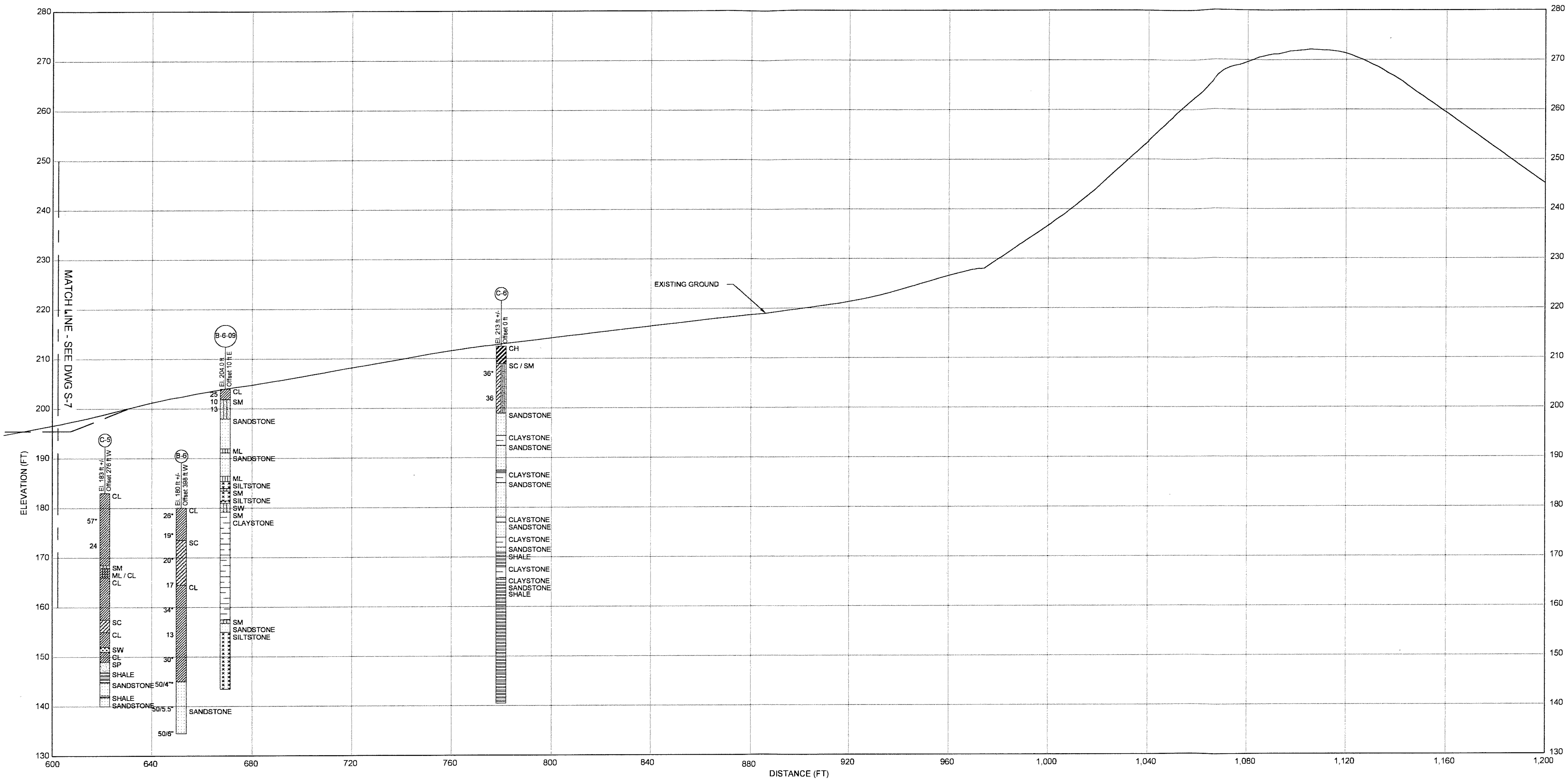
GEOLOGIC SECTIONS (4 OF 7)

FILE NO. FD-3210

SHEET XX OF YY

CADD FILE# S-7.dwg PEN TBL: GEI Boston Standard 2009.ctb

S-7 08-16-10 D.DeCesaris



NOTES:

1. BLOW COUNTS ON CE&G BORINGS (B-1-09 ETC.) ARE CORRECTED BASED ON TABER DRILLING ENERGY CALIBRATION IN ACCORDANCE WITH DWR AUGUST 2008 MANUAL.
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SECTION E (2 of 2) S-1

LEGEND:

SEE DRAWING S-2.

DRAFT - 90% REVIEW

S-8

REVISIONS			
NO.	DESCRIPTION	BY	DATE

DES.: DD
DRAWN: DD
CHKD.: JN
DATE: 08-16-10
SCALE: VARIES
FLD. BK. XXXX

REGISTERED PROFESSIONAL ENGINEER ALBERTO PUJOL-RIOS No. 37051 EXP CIVIL STATE OF CALIFORNIA

PROJECT ENGINEER

PLANS APPROVAL DATE

GEI Consultants

CONTRA COSTA COUNTY
PUBLIC WORKS DEPARTMENT
255 GLACIER DRIVE
MARTINEZ, CALIFORNIA 94553

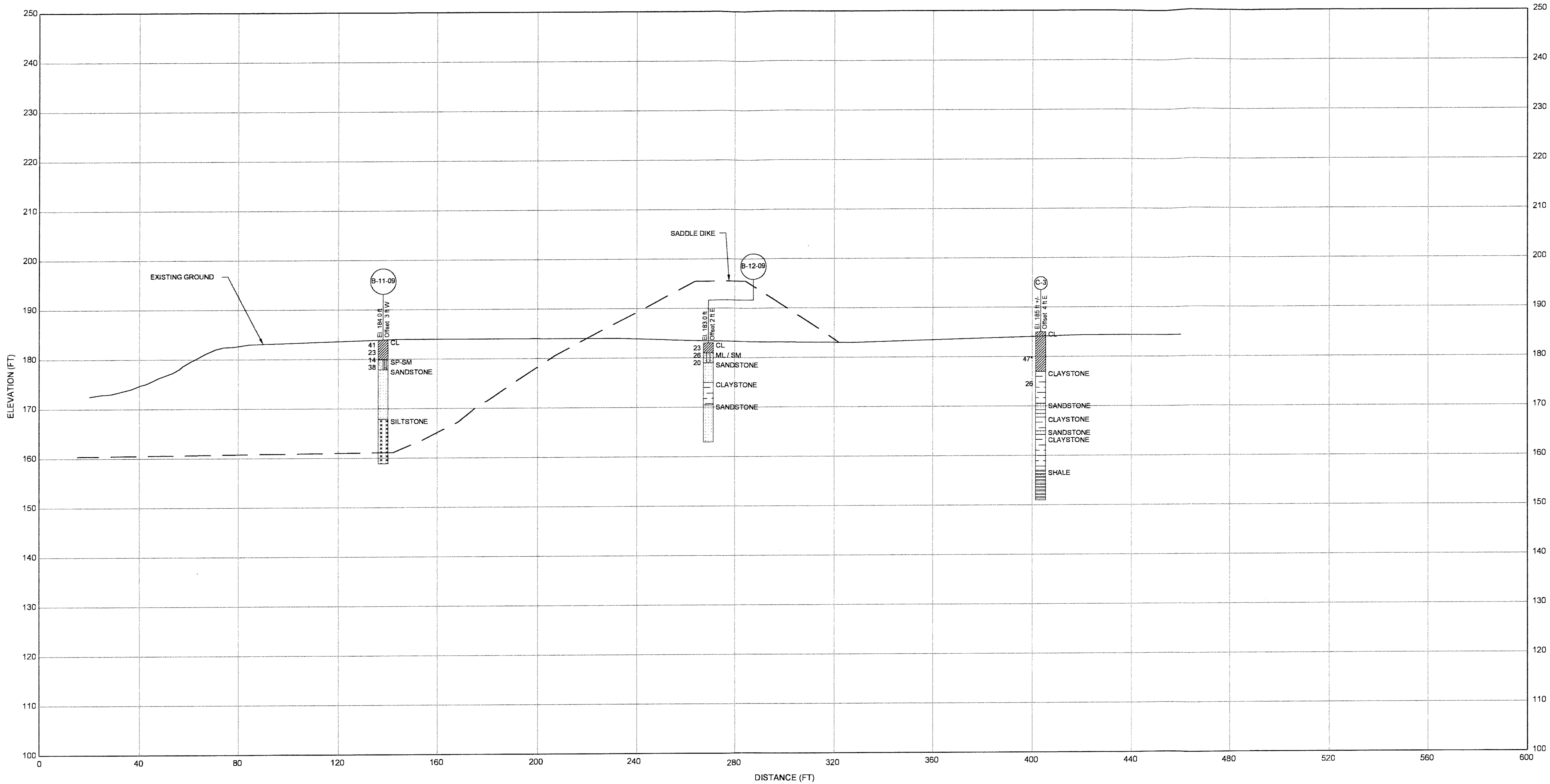
0 10 20
VERTICAL SCALE, FEET
0 20 40
HORIZONTAL SCALE, FEET

FOR REDUCED PLANS ORIGINAL SCALE IS IN INCHES
0 1 2 3

BASE MAP	EAST COORD.	NORTH COORD.
L23	1635	528

UPPER SAND CREEK DETENTION BASIN
GEOLOGIC SECTIONS (5 OF 7)
FILE NO. FD-3210
SHEET XX OF YY
CADD FILE# S-8.dwg
PEN TBL. GEI Boston Standard 2009.ctb

S-8 08-16-10 D.DeCesaris



NOTES:

1. BLOW COUNTS ON CE&G BORINGS (B-1-09 ETC.) ARE CORRECTED BASED ON TABER DRILLING ENERGY CALIBRATION IN ACCORDANCE WITH DWR AUGUST 2008 MANUAL.
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SECTION F
STATION 28+50 S-1

LEGEND:

SEE DRAWING S-2.

DRAFT - 90% REVIEW

S-9

REVISIONS			
NO.	DESCRIPTION	BY	DATE

DES.: DD
DRAWN: DD
CHKD.: JN
DATE: 08-16-10
SCALE: VARIES
FLD. BK.: XXXX

REGISTERED PROFESSIONAL ENGINEER ALBERTO PUJOL-RIUS No. 37051 EXP CIVIL STATE OF CALIFORNIA

PROJECT ENGINEER

PLANS APPROVAL DATE

GEI Consultants

CONTRA COSTA COUNTY
PUBLIC WORKS DEPARTMENT
255 GLACIER DRIVE
MARTINEZ, CALIFORNIA 94553

0 10 20
VERTICAL SCALE, FEET
0 20 40
HORIZONTAL SCALE, FEET

FOR REDUCED PLANS ORIGINAL SCALE IS IN INCHES
0 1 2 3

BASE MAP	EAST COORD.	NORTH COORD.
L23	1635	528

UPPER SAND CREEK DETENTION BASIN

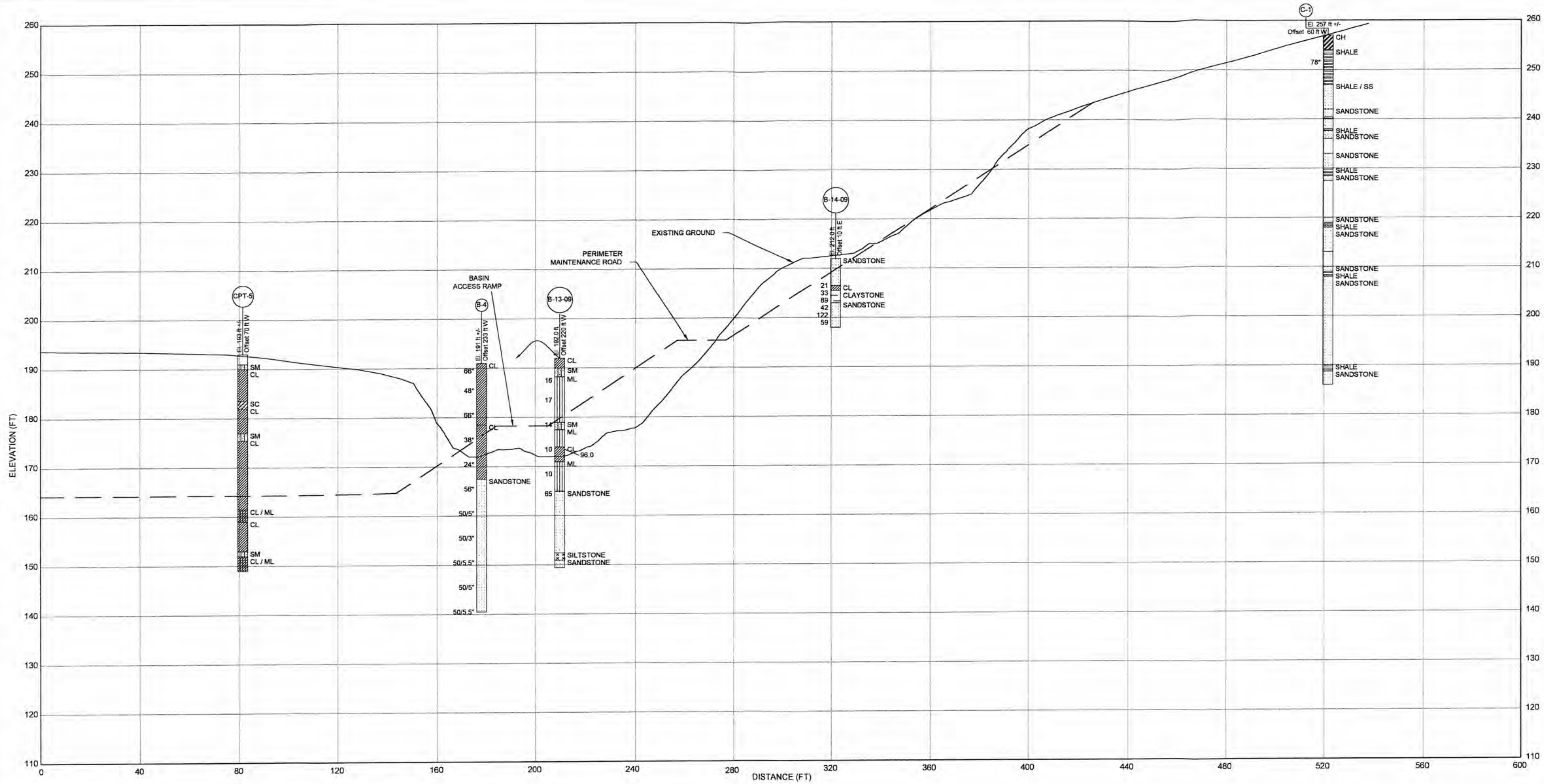
GEOLOGIC SECTIONS (6 OF 7)

FILE NO. FD-3210

SHEET XX OF YY

CADD FILE# S-9.dwg
PEN TBL: GEI Boston Standard 2009.ctb

S-9 08-16-10 D.DeCesaris



SECTION G
STATION 39+10
S-1

NOTES:

1. BLOW COUNTS ON CE&G BORINGS (B-1-09 ETC.) ARE CORRECTED BASED ON TABER DRILLING ENERGY CALIBRATION IN ACCORDANCE WITH DWR AUGUST 2008 MANUAL.
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LEGEND:

SEE DRAWING S-2.

DRAFT - 90% REVIEW

S-10

REVISIONS

NO.	DESCRIPTION	BY	DATE

DES.: DD
DRAWN: DD
CHKD.: JN
DATE: 08-16-10
SCALE: VARIES
FLD. BK. XXXX

REGISTERED PROFESSIONAL ENGINEER ALBERTO PUJOL-RIOS No. 37051 EXP CIVIL STATE OF CALIFORNIA

PROJECT ENGINEER

PLANS APPROVAL DATE

GEI Consultants

CONTRA COSTA COUNTY
PUBLIC WORKS DEPARTMENT
255 GLACIER DRIVE
MARTINEZ, CALIFORNIA 94553

VERTICAL SCALE, FEET (0 10 20)
HORIZONTAL SCALE, FEET (0 20 40)

FOR REDUCED PLANS ORIGINAL SCALE IS IN INCHES

BASE MAP	EAST COORD	NORTH COORD
L23	1635	528

UPPER SAND CREEK DETENTION BASIN

GEOLOGIC SECTIONS (7 OF 7)

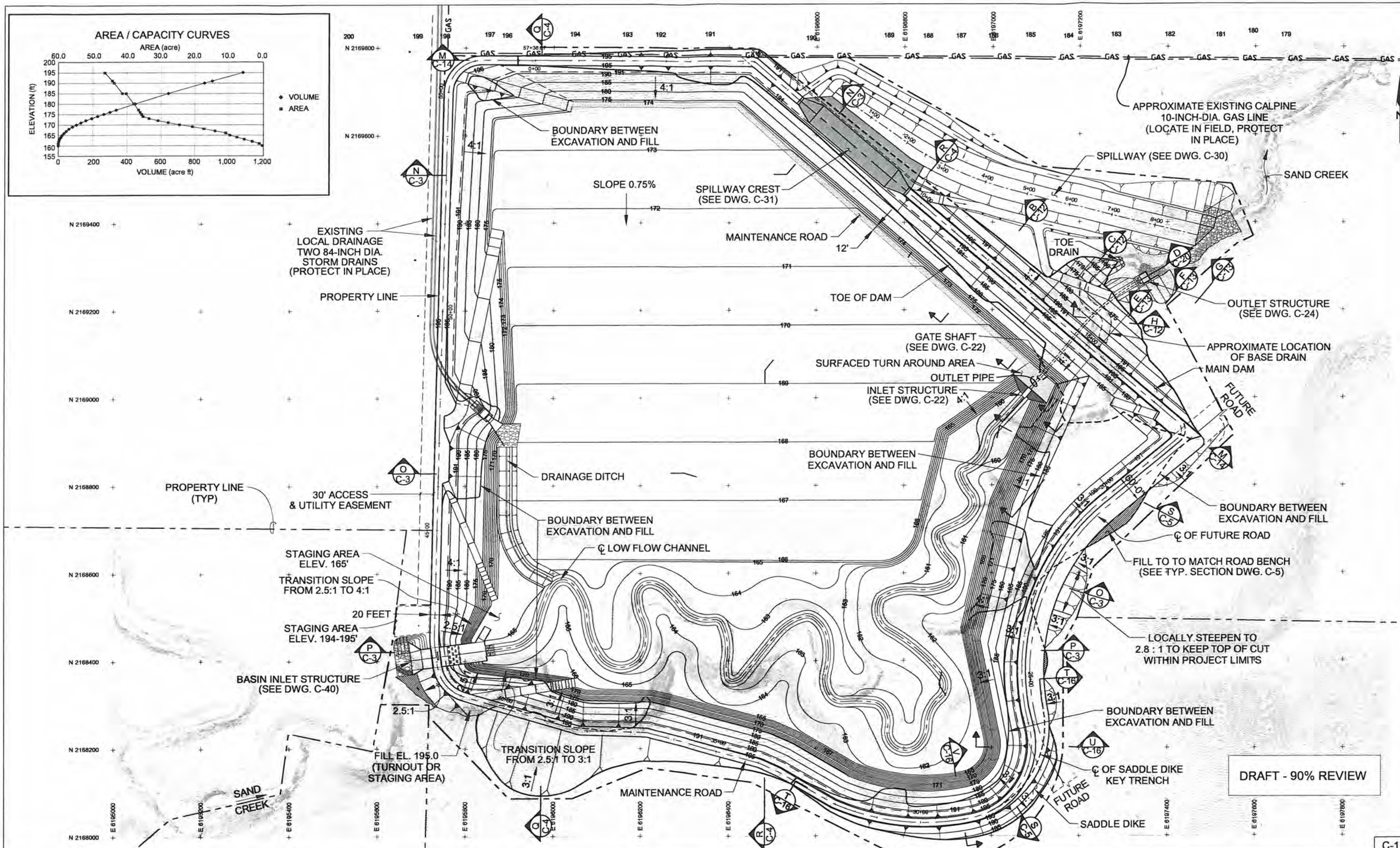
FILE NO. FD-3210

SHEET XX OF YY

CADD FILE# S-10.dwg

PEN TBL: GEI Boston Standard 2009.ctb

S-10 08-16-10 D.DeCesaris



REVISIONS			
NO.	DESCRIPTION	BY	DATE

DES.: K.V.S.
DRAWN: P.Y.M.
CHKD.: A.T.
DATE: 08-16-10
SCALE: 1" = 100'
FLD. BK. XXXX

REGISTERED PROFESSIONAL ENGINEER ALBERTO PUJOL-RIOS No. 37051 CIVIL STATE OF CALIFORNIA

PROJECT ENGINEER

PLANS APPROVAL DATE

GEI Consultants

CONTRA COSTA COUNTY
PUBLIC WORKS DEPARTMENT
255 GLACIER DRIVE
MARTINEZ, CALIFORNIA 94553

0 100 200
SCALE, FEET

FOR REDUCED PLANS ORIGINAL SCALE IS IN INCHES
0 1 2 3

BASE MAP	EAST COORD.	NORTH COORD.
L23	1635	528

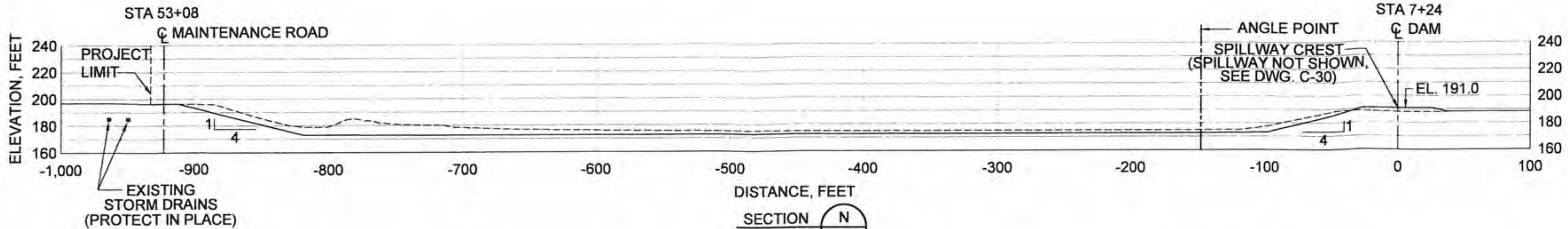
UPPER SAND CREEK DETENTION BASIN

BASIN SITE PLAN

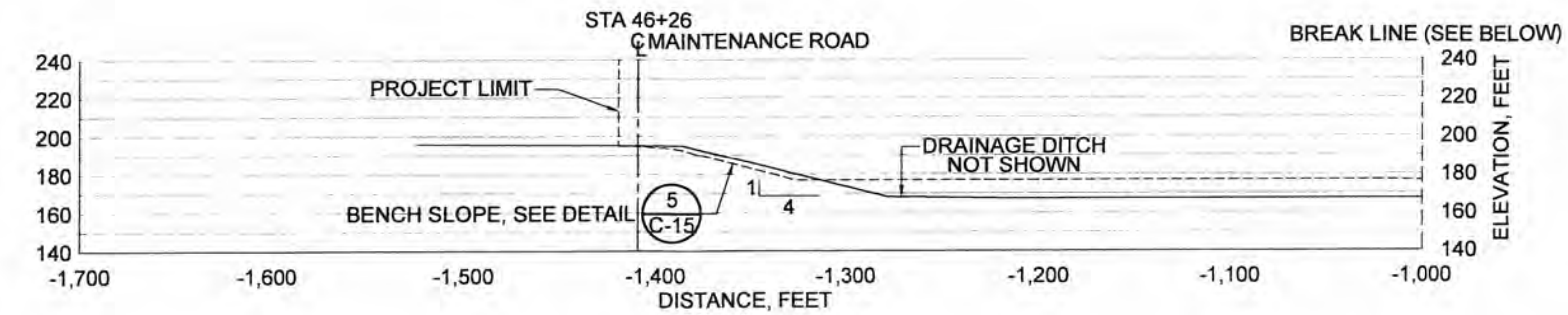
FILE NO. FD-3210

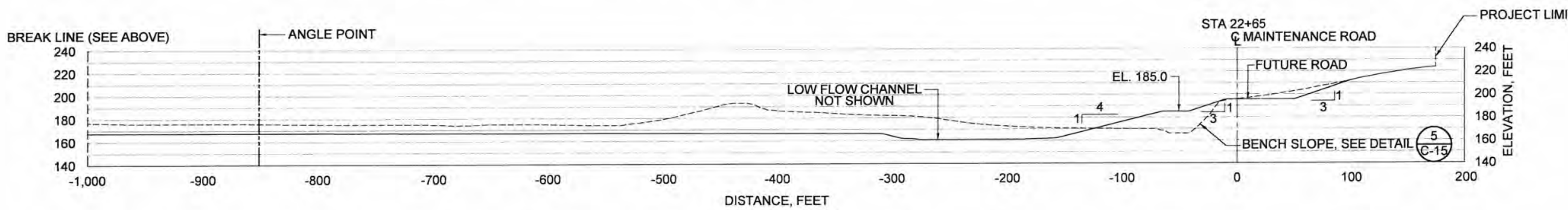
SHEET XX OF YY

C-1 08-16-10 KVS/PYM

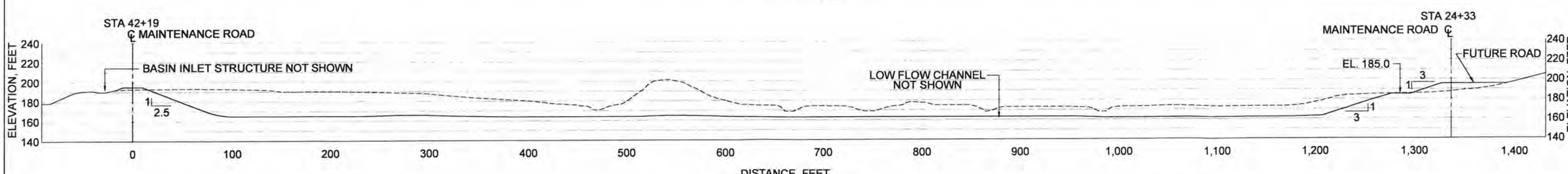


SECTION N / C-1





SECTION O / C-1



SECTION P / C-1

LEGEND

- FINISH GRADE (EMBANKMENT AND EXCAVATION)
- EXISTING GROUND TO REMAIN
- EXISTING GROUND TO BE EXCAVATED

DRAFT - 90% REVIEW

NOTES:

1. THE PURPOSE OF THE CROSS-SECTIONS IS TO ILLUSTRATE EXISTING GROUND LINE, BASIN EXCAVATION, AND CUT AND FILL SLOPES. CROSS-SECTIONS DO NOT SHOW DAM EMBANKMENT ZONING OR GRADING DETAILS.

C-3

REVISIONS

NO.	DESCRIPTION	BY	DATE

DES.: K.V.S.
DRAWN: P.Y.M.
CHKD.: A.T.
DATE: 08-16-10
SCALE: 1" = 50'
FLD. BK. XXXX

REGISTERED PROFESSIONAL ENGINEER ALBERTO PUJOL-RIO No. 37051 EXP CIVIL STATE OF CALIFORNIA

PROJECT ENGINEER

PLANS APPROVAL DATE

GEI Consultants

CONTRA COSTA COUNTY PUBLIC WORKS DEPARTMENT 255 GLACIER DRIVE MARTINEZ, CALIFORNIA 94553

0 50 100
SCALE, FEET

FOR REDUCED PLANS ORIGINAL SCALE IS IN INCHES
0 1 2 3

BASE MAP	EAST COORD.	NORTH COORD.
L23	1635	528

UPPER SAND CREEK DETENTION BASIN

BASIN GRADING - SECTIONS & DETAILS (Sheet 1 of 3)

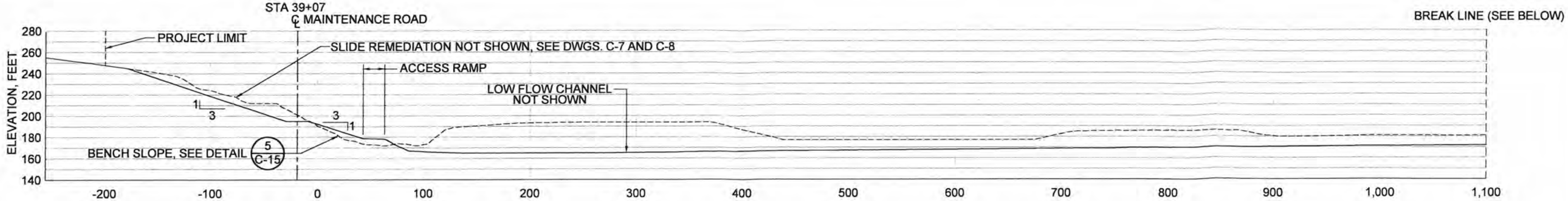
FILE NO. FD-3210

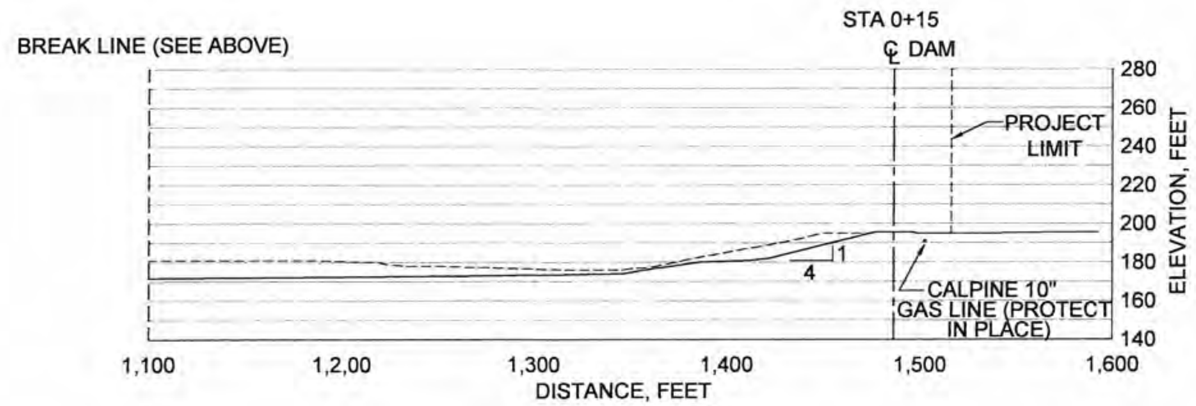
SHEET XX OF YY

CADD FILE# C-3.dwg

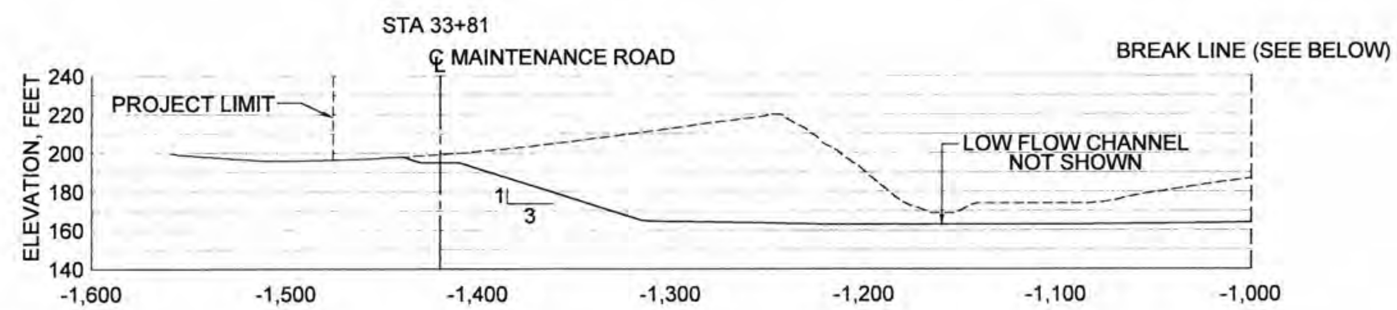
PEN TBL. GEIWALKER.ctb

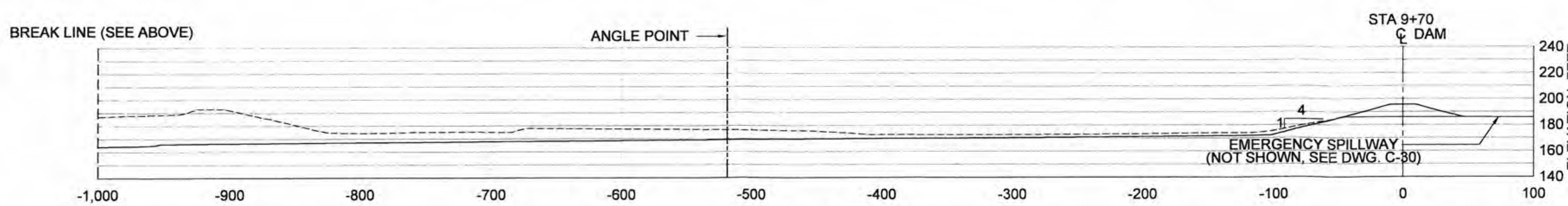
C-3 08-16-10 KVS





SECTION Q / C-1





SECTION R / C-1

LEGEND

- FINISH GRADE (EMBANKMENT AND EXCAVATION)
- EXISTING GROUND TO REMAIN
- EXISTING GROUND TO BE EXCAVATED

DRAFT - 90% REVIEW

NOTES:

1. THE PURPOSE OF THE CROSS-SECTIONS IS TO ILLUSTRATE EXISTING GROUND LINE, BASIN EXCAVATION, AND CUT AND FILL SLOPES. CROSS-SECTIONS DO NOT SHOW DAM EMBANKMENT ZONING OR GRADING DETAILS.

C-4

REVISIONS			
NO.	DESCRIPTION	BY	DATE

DES.: K.V.S.
DRAWN: P.Y.M.
CHKD.: A.T.
DATE: 0884610
SCALE: 1" = 50'
FLD. BK. XXXX

REGISTERED PROFESSIONAL ENGINEER · ALBERTO PUJOL-RUIS · No. 37051 · EXP · CIVIL · STATE OF CALIFORNIA

PROJECT ENGINEER

PLANS APPROVAL DATE

GEI Consultants

CONTRA COSTA COUNTY
PUBLIC WORKS DEPARTMENT
255 GLACIER DRIVE
MARTINEZ, CALIFORNIA 94553

0 50 100
SCALE, FEET

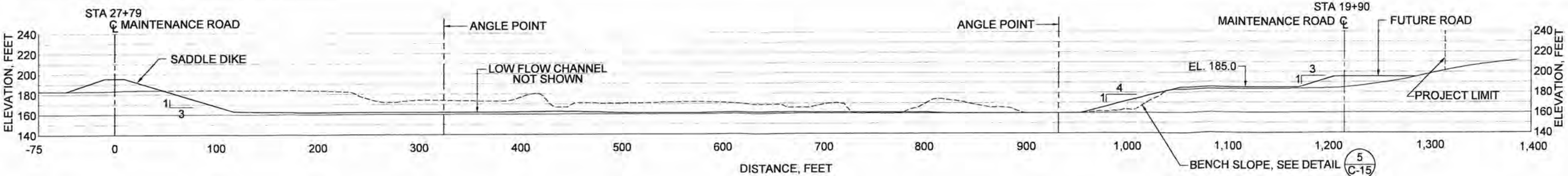
FOR REDUCED PLANS ORIGINAL SCALE IS IN INCHES
0 1 2 3

BASE MAP	EAST COORD.	NORTH COORD.
L23	1635	528

UPPER SAND CREEK DETENTION BASIN
BASIN GRADING - SECTIONS & DETAILS (Sheet 2 of 3)

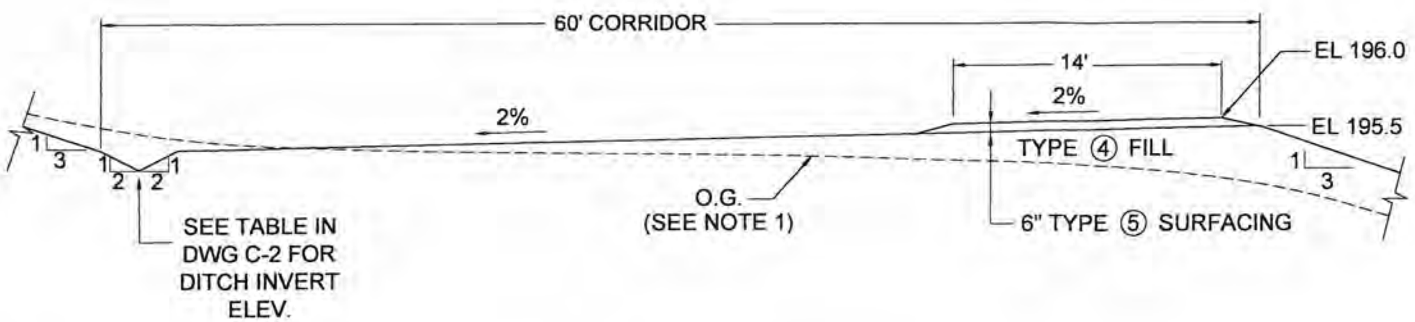
FILE NO. FD-3210
SHEET XX OF YY
CADD FILE# C-4.dwg
PEN TBL GEIWALKER.ctb

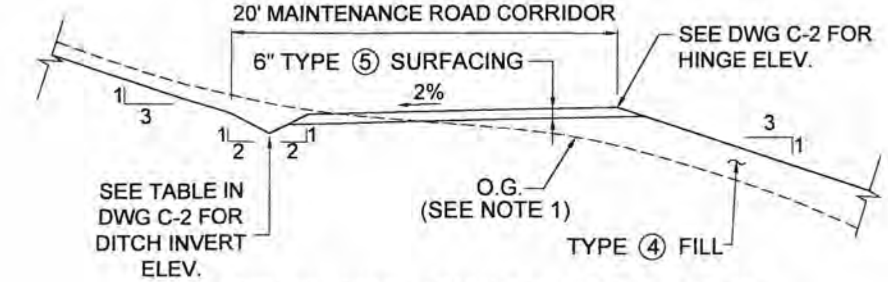
C-4 08-16-10 KVS



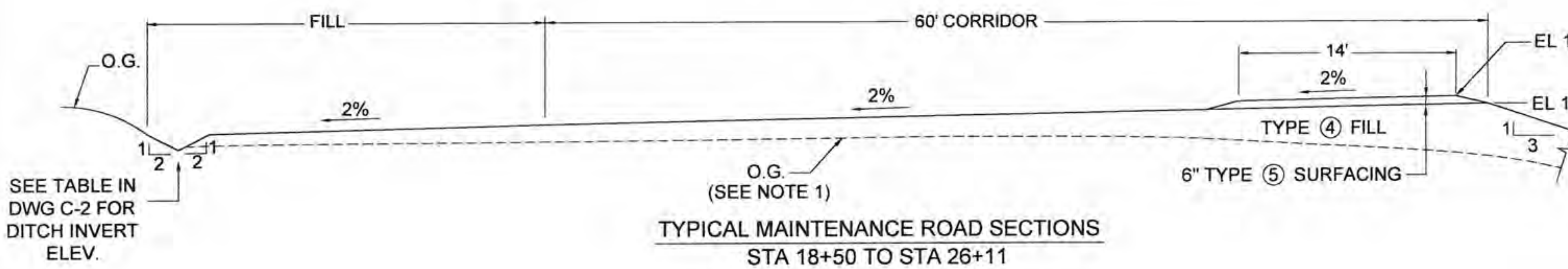
SECTION S C-1

SCALE, FEET



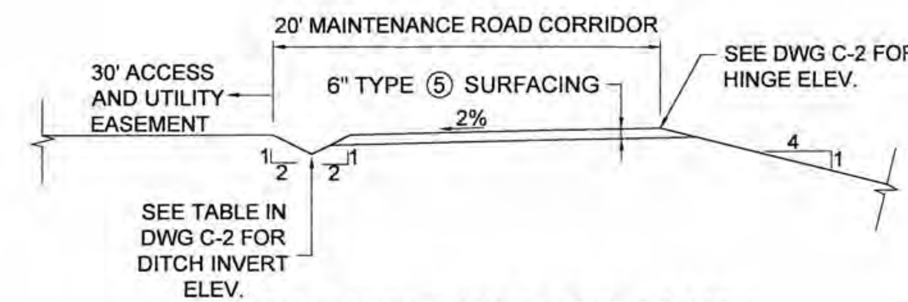


TYPICAL MAINTENANCE ROAD SECTION
STA 33+05 TO STA 40+45

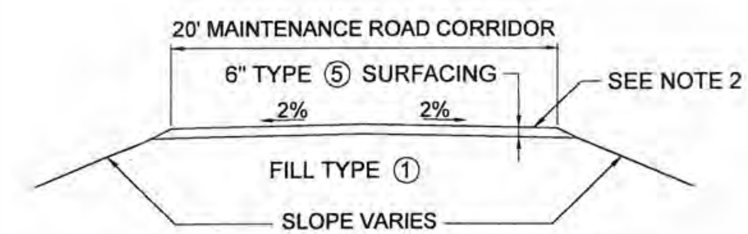


TYPICAL MAINTENANCE ROAD SECTIONS
STA 18+50 TO STA 26+11

SCALE, FEET



TYPICAL MAINTENANCE ROAD SECTION
STA 43+30 TO STA 55+25



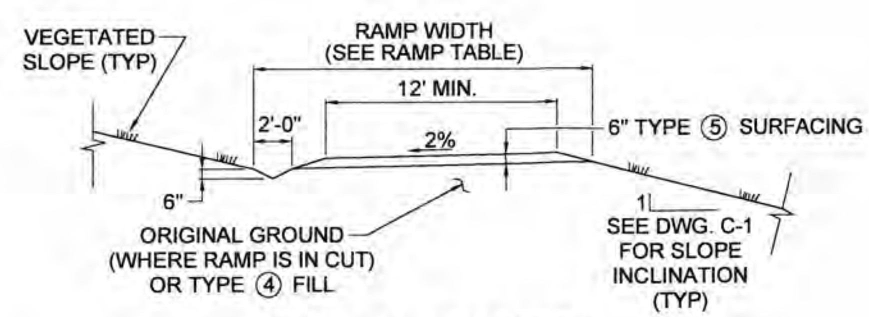
TYPICAL MAINTENANCE ROAD SECTION
STA 40+45 TO STA 41+70 AND STA 42+44 TO 43+30

SCALE, FEET

RAMP TABLE			
PERIMETER ROAD STA.	RAMP WIDTH (FT)	APPROX. RAMP SLOPE (%)	AB SURFACING
40+50	20	10	YES
46+10	15	10	YES
47+50	28	5	YES
55+40	24	9	YES
6+30	15	VARIES *	NO
10+00	20	VARIES *	YES**
17+30	20	10	YES

* SEE DWG. C-30 & C-31

** BETWEEN DAM CREST AND PIPE OUTLET



TYPICAL RAMP SECTION

SCALE, FEET

DRAFT - 90% REVIEW

NOTES:

1. STRIP ORGANIC AND LOOSE SOILS DOWN TO COMPETENT MATERIAL UNDER ALL BASIN FILLS.
2. PROVIDE SMOOTH TRANSITIONS AT BRIDGE APPROACHES AND AT JUNCTIONS WITH OTHER ROAD SECTIONS.

C-5

REVISIONS			
NO.	DESCRIPTION	BY	DATE

DES.: K.V.S.
DRAWN: P.Y.M.
CHKD.: A.T.
DATE: 08-16-10
SCALE: AS INDICATED
FLD. BK. XXXX

REGISTERED PROFESSIONAL ENGINEER ALBERTO PUJOL-RIOS No. 37051 EXP CIVIL STATE OF CALIFORNIA

PROJECT ENGINEER

PLANS APPROVAL DATE

GEI Consultants

CONTRA COSTA COUNTY
PUBLIC WORKS DEPARTMENT
255 GLACIER DRIVE
MARTINEZ, CALIFORNIA 94553

SCALE AS INDICATED

FOR REDUCED PLANS ORIGINAL SCALE IS IN INCHES

BASE MAP	EAST COORD.	NORTH COORD.
L23	1635	528

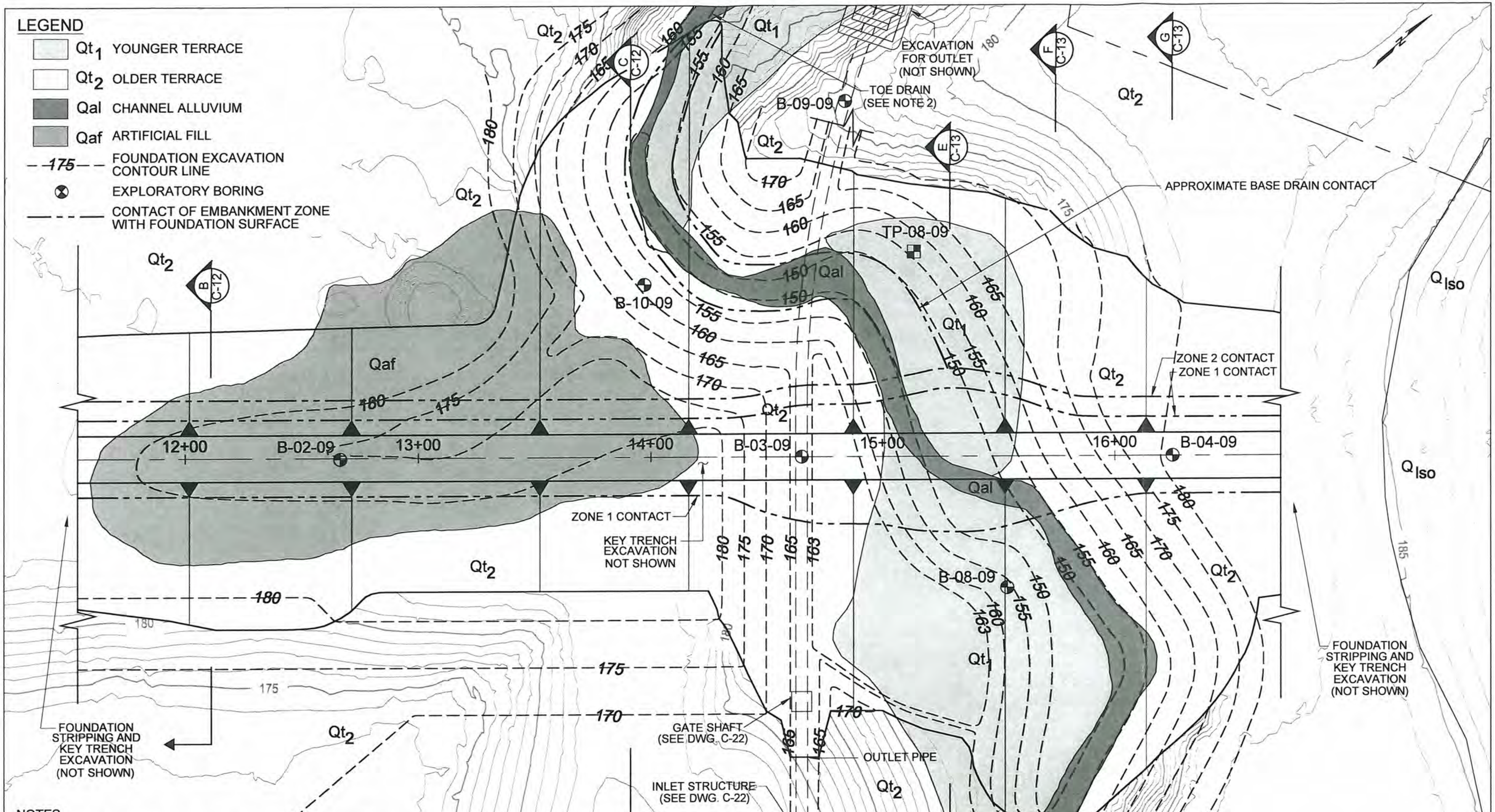
UPPER SAND CREEK DETENTION BASIN
BASIN GRADING - SECTIONS AND DETAILS (Sheet 3 of 3)

FILE NO. FD-3210

SHEET XX OF YY

CADD FILE# C-5.dwg PEN TBL: GEIWALKER.ctb

C-5 08-16-10 KVS



NOTES

1. SEE SPECIAL PROVISIONS FOR DAM FOUNDATION EXCAVATION AND TREATMENT REQUIREMENTS. FINAL FOUNDATION EXCAVATION SHALL BE DETERMINED BY THE ENGINEER DURING CONSTRUCTION.
2. EXTEND EXCAVATION AT DOWNSTREAM TOE OF DAM TO FORM A DRAIN CHANNEL WITH 5-FT BOTTOM WIDTH, 2H (MIN): 1V SIDE SLOPES, AND INVERT EL. 155.0, EXTEND DOWNSTREAM TO DAYLIGHT AT EL. 155.0 IN EXISTING CREEK CHANNEL.

REVISIONS			
NO.	DESCRIPTION	BY	DATE

DES.: K.V.S.
DRAWN: P.Y.M.
CHKD.: A.P.
DATE: 08-16-10
SCALE: 1" = 20'
FLD. BK. XXXX

REGISTERED PROFESSIONAL ENGINEER ALBERTO PUJOL-RIUS No. 37051 EXP CIVIL STATE OF CALIFORNIA

PROJECT ENGINEER

PLANS APPROVAL DATE

GEI Consultants

CONTRA COSTA COUNTY PUBLIC WORKS DEPARTMENT
255 GLACIER DRIVE
MARTINEZ, CALIFORNIA 94553

0 20 40 SCALE, FEET

FOR REDUCED PLANS ORIGINAL SCALE IS IN INCHES

BASE MAP	EAST COORD.	NORTH COORD.
L23	1635	528

UPPER SAND CREEK DETENTION BASIN

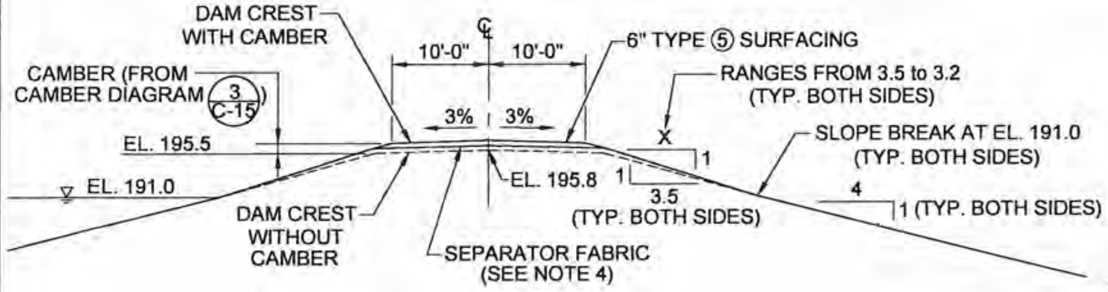
DAM EXCAVATION PLAN

FILE NO. FD-3210

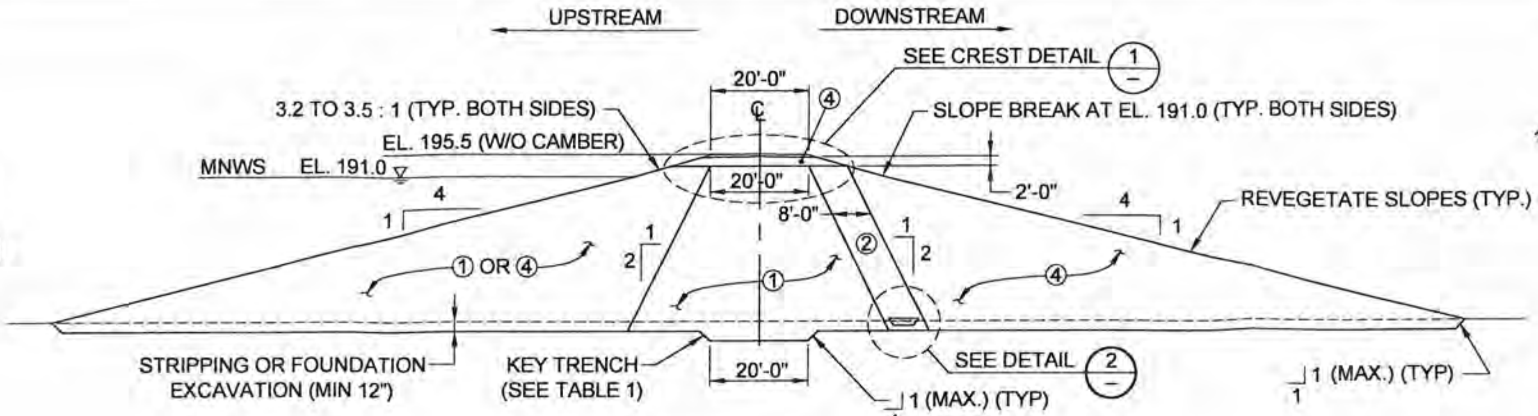
SHEET XX OF YY

CADD FILE# C-11.dwg PEN TBL: GEIWALKER.ctb

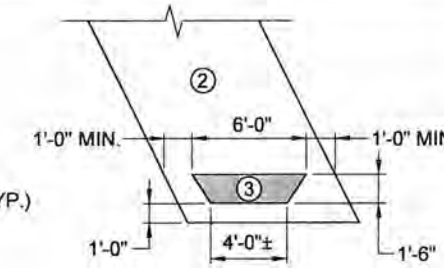
C-11 08-16-10 PYM



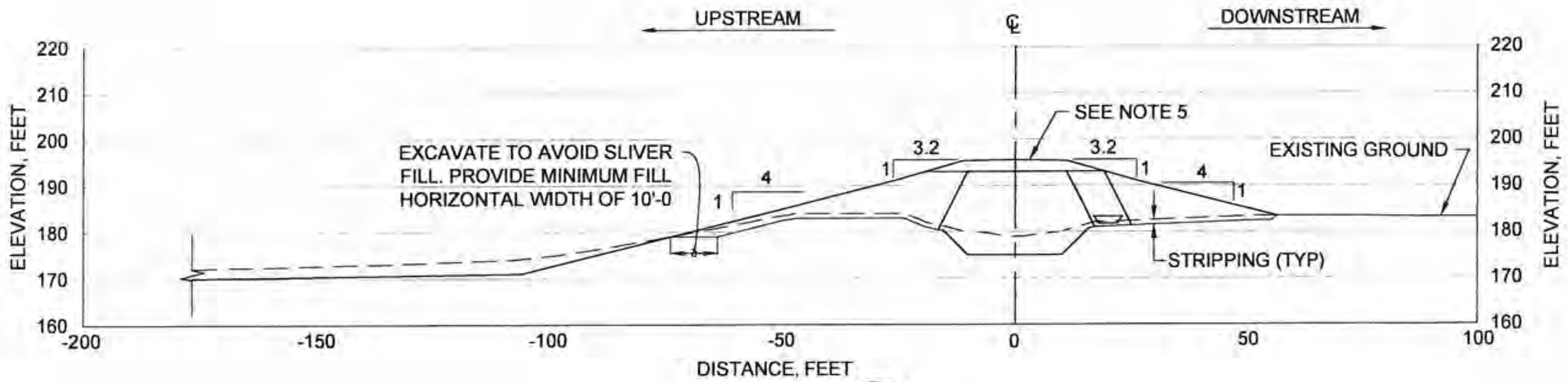
CREST DETAIL 1 / NTS



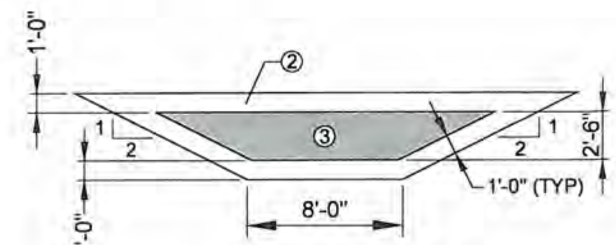
TYPICAL DAM SECTION A / (SEE NOTE 1)



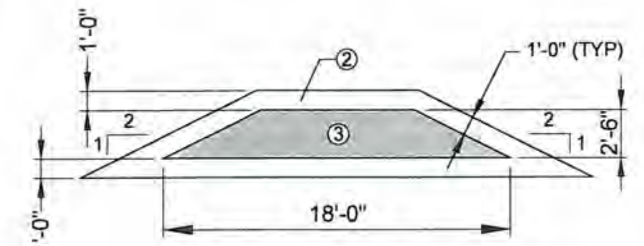
DETAIL 2 / 1" = 5'-0"



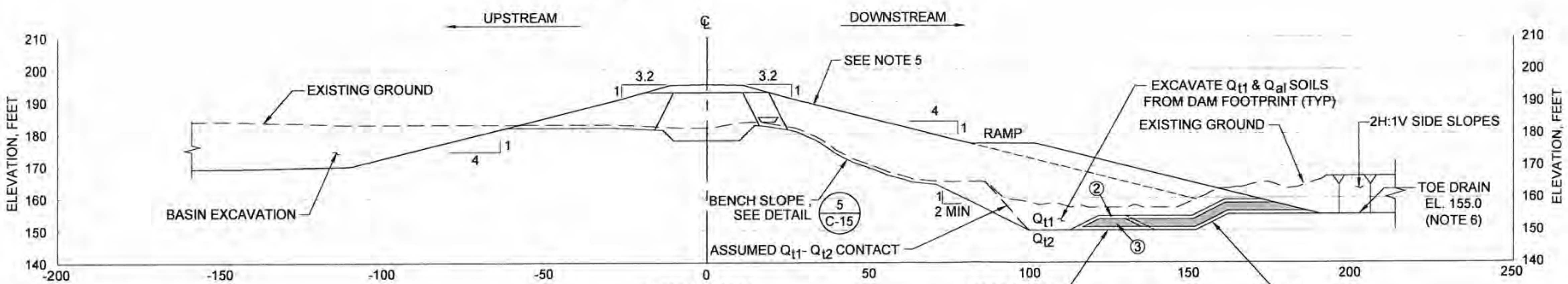
SECTION B / STA 12+11 / C-1



SECTION H / (SEE NOTE 3) / C-1 / 1" = 5'-0"



SECTION (ALT.) H / (SEE NOTE 3) / C-1 / 1" = 5'-0"



SECTION C / STA 13+97 / C-1

MATERIAL TYPE

1. CLAY
2. FILTER SAND
3. DRAIN GRAVEL
4. RANDOM FILL
5. AGGREGATE BASE
6. STRUCTURAL FILL

LEGEND

- FINAL GRADE (EMBANKMENT AND EXCAVATION)
- EXISTING GROUND THAT REMAINS
- ORIGINAL GROUND TO BE EXCAVATED
- BOUNDARY BETWEEN DAM FILL AND BASIN FILL

TABLE 1
KEY TRENCH DEPTH BELOW STRIPPED GROUND SURFACE

STATION	DEPTH (FT)
0+00 TO 5+00	1.0
5+00 TO 10+00	2.0
10+00 TO 17+00	3.0
17+00 TO 18+30	2.0

DRAFT - 90% REVIEW

NOTES

1. SECTION A IS A GENERALIZED CROSS-SECTION TO ILLUSTRATE THE EMBANKMENT CONFIGURATION AND INTERNAL ZONING. THE DOWNSTREAM FILTER (ZONE 2 AND ZONE 3) SHALL EXTEND BETWEEN STA. 9+61 AND STA. 17+80.
2. KEY TRENCH DEPTHS SHOWN IN TABLE 1 ARE THE MINIMUM REQUIRED AND MAY BE INCREASED BY THE ENGINEER TO SUIT FIELD CONDITIONS EXPOSED BY THE FOUNDATION EXCAVATION.
3. THE DIMENSIONS OF ZONE 3 ARE APPROXIMATE AND CAN BE ADJUSTED TO FIT THE BOTTOM OF THE VALLEY. HOWEVER, THE MINIMUM ALLOWABLE CROSS-SECTION AREA OF ZONE 3 SHALL BE 30-FT ^{2} AT ALL SECTIONS. ZONE 3 SHALL BE ENVELOPED BY ZONE 2 WITH A MINIMUM THICKNESS OF 1'-0" ALL AROUND.
4. SEPARATOR FABRIC SHALL BE NONWOVEN-TYPE ROCK SLOPE PROTECTION FABRIC TYPE B.
5. FOR DAM ZONING AND CREST DETAILS SEE SECTION A AND DETAIL 1.
6. TOE DRAIN SHALL BE AN EXCAVATED CHANNEL WITH BOTTOM WIDTH OF 5 FT (MIN.), SIDE SLOPES OF 2(MIN):1, AND INVERT EL. 155.0.

C-12

REVISIONS			
NO.	DESCRIPTION	BY	DATE

DES.: K.V.S.
DRAWN: P.Y.M.
CHKD.: A.T.
DATE: 08-16-10
SCALE: 1" = 20'
FLD. BK. XXXX

REGISTERED PROFESSIONAL ENGINEER ALBERTO PUJOL-RIUS No. 37051 EXP CIVIL STATE OF CALIFORNIA

PROJECT ENGINEER

PLANS APPROVAL DATE

GEI Consultants

CONTRA COSTA COUNTY
PUBLIC WORKS DEPARTMENT
255 GLACIER DRIVE
MARTINEZ, CALIFORNIA 94553

0 20 40
SCALE, FEET

FOR REDUCED PLANS ORIGINAL SCALE IS IN INCHES
0 1 2 3

BASE MAP	EAST COORD.	NORTH COORD.
L23	1635	528

UPPER SAND CREEK DETENTION BASIN

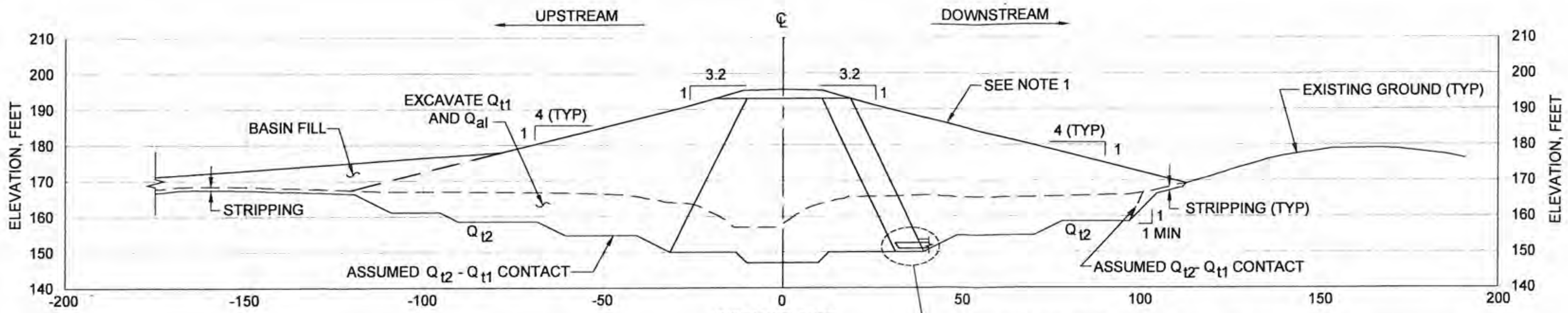
DAM EMBANKMENT - CROSS SECTIONS (Sheet 1 of 2)

FILE NO. FD-3210

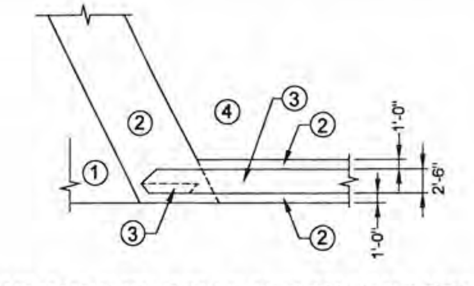
SHEET XX OF YY

CADD FILE# C-12.dwg PEN TBL. GEIWALKER.ctb

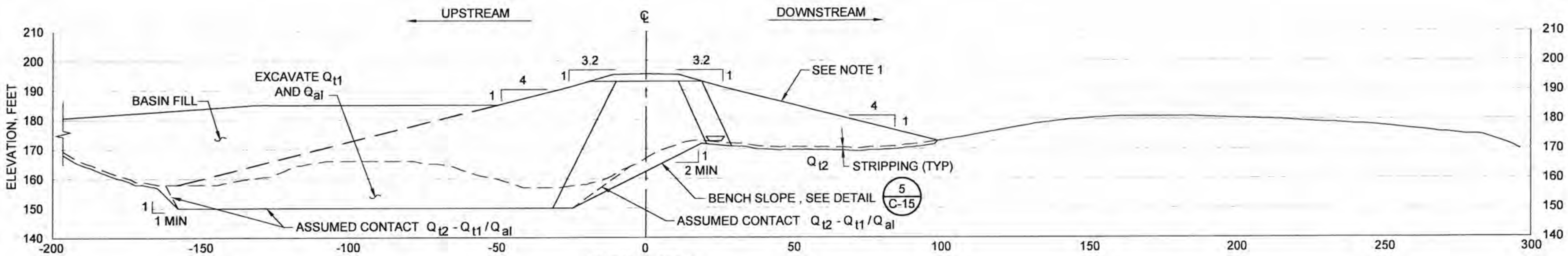
C-12 08-16-10 PYM



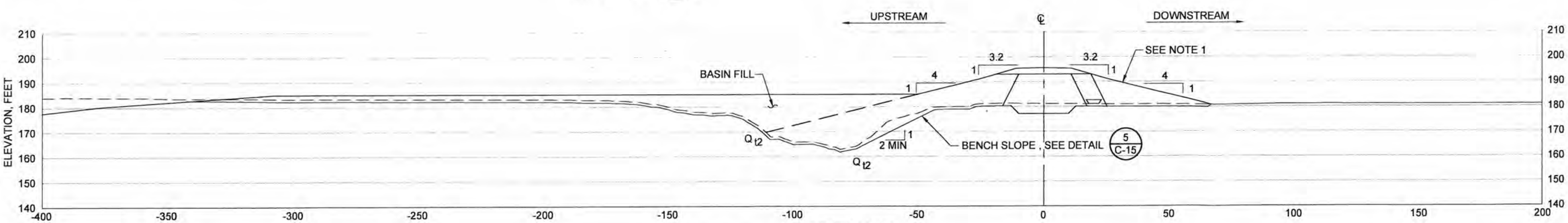
SECTION E/C-1
STA 15+29



CONNECTION OF D/S FILTER TO BASE DRAIN
DETAIL 1/–



SECTION F/C-1
STA 15+74



SECTION G/C-1
STA 16+25

LEGEND

- FINAL GRADE (EMBANKMENT AND EXCAVATION)
- EXISTING GROUND THAT REMAINS
- ORIGINAL GROUND TO BE EXCAVATED
- BOUNDARY BETWEEN DAM FILL AND BASIN FILL

NOTES

1. FOR DAM ZONING AND CREST DETAILS SEE SECTION A AND DETAIL 1 OF DWG. C-12.

DRAFT - 90% REVIEW

C-13

REVISIONS			
NO.	DESCRIPTION	BY	DATE

DES.: K.V.S.
DRAWN: P.Y.M.
CHKD.: A.T.
DATE: 08-16-10
SCALE: 1" = 20'
FLD. BK. XXXX

REGISTERED PROFESSIONAL ENGINEER ALBERTO PUJOL-RIUS No. 37051 EXP CIVIL STATE OF CALIFORNIA

PROJECT ENGINEER
PLANS APPROVAL DATE

GEI Consultants

CONTRA COSTA COUNTY
PUBLIC WORKS DEPARTMENT
255 GLACIER DRIVE
MARTINEZ, CALIFORNIA 94553

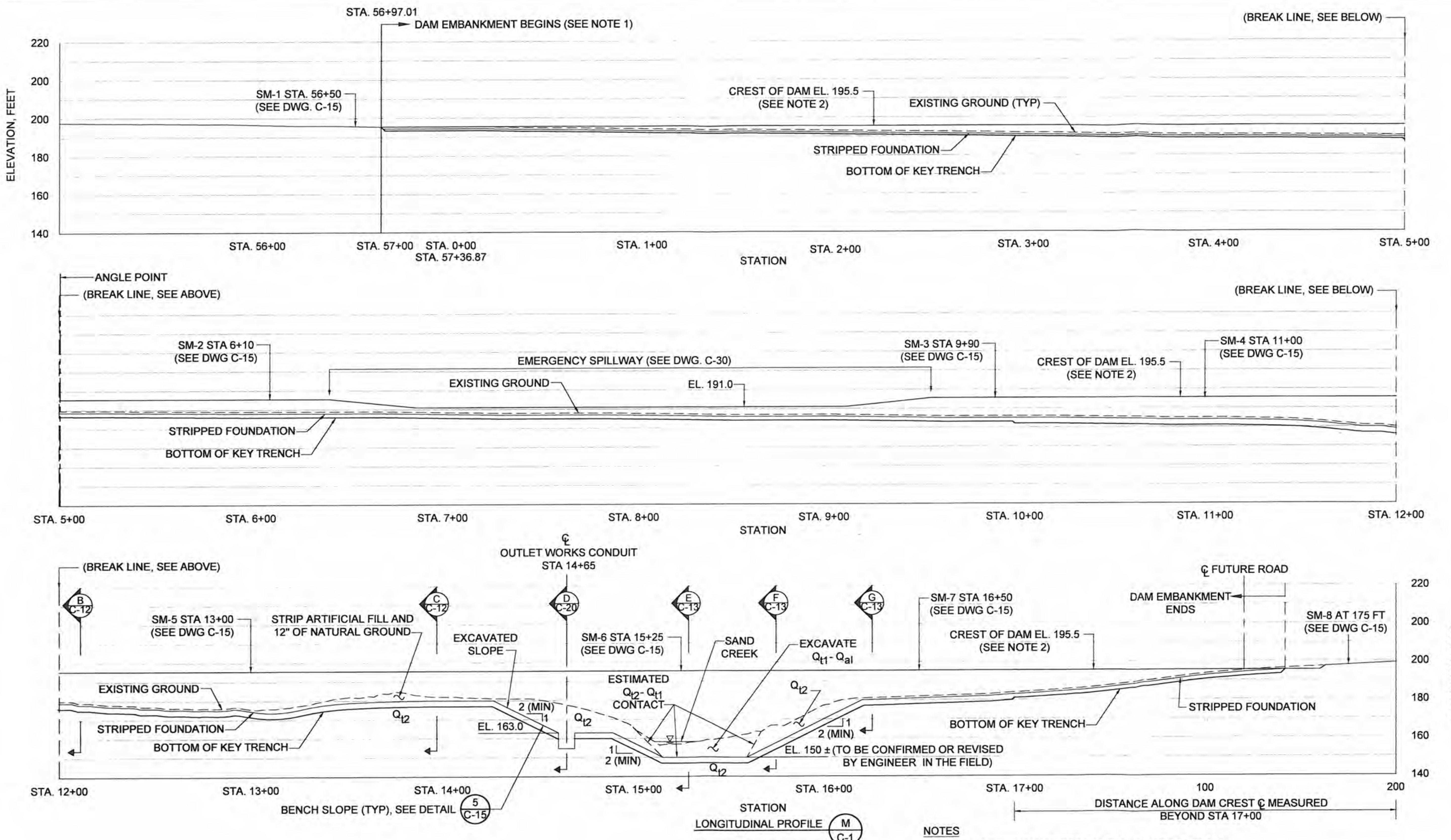
0 20 40
SCALE, FEET

FOR REDUCED PLANS ORIGINAL SCALE IS IN INCHES
0 1 2 3

BASE MAP	EAST COORD.	NORTH COORD
L23	1635	528

UPPER SAND CREEK DETENTION BASIN
DAM EMBANKMENT - CROSS-SECTIONS
FILE NO. FD-3210
SHEET XX OF YY
CADD FILE# C-13.dwg
PEN TBL GEIWALKER.ctb

C-13 08-16-10 PYM



LONGITUDINAL PROFILE M C-1

NOTES

1. FOR DAM ZONING AND CREST DETAILS SEE SECTION A AND DETAIL 1 OF DWG. C-12.
2. DAM CREST EL. 195.5 DOES NOT INCLUDE CAMBER. FOR CAMBER PROFILE SEE DWG. C-15.

DRAFT - 90% REVIEW

C-14

REVISIONS			
NO.	DESCRIPTION	BY	DATE

DES.: K.V.S.
DRAWN: P.Y.M.
CHKD.: A.T.
DATE: 08-16-10
SCALE: 1" = 25'
FLD. BK. XXXX

REGISTERED PROFESSIONAL ENGINEER ALBERTO PUJOL-RIOS No. 37051 EXP CIVIL STATE OF CALIFORNIA

PROJECT ENGINEER

PLANS APPROVAL DATE

GEI Consultants

CONTRA COSTA COUNTY
PUBLIC WORKS DEPARTMENT
255 GLACIER DRIVE
MARTINEZ, CALIFORNIA 94553

0 25 50
SCALE, FEET

FOR REDUCED PLANS ORIGINAL SCALE IS IN INCHES
0 1 2 3

BASE MAP	EAST COORD.	NORTH COORD.
L23	1635	528

UPPER SAND CREEK DETENTION BASIN

DAM EMBANKMENT - LONGITUDINAL SECTION

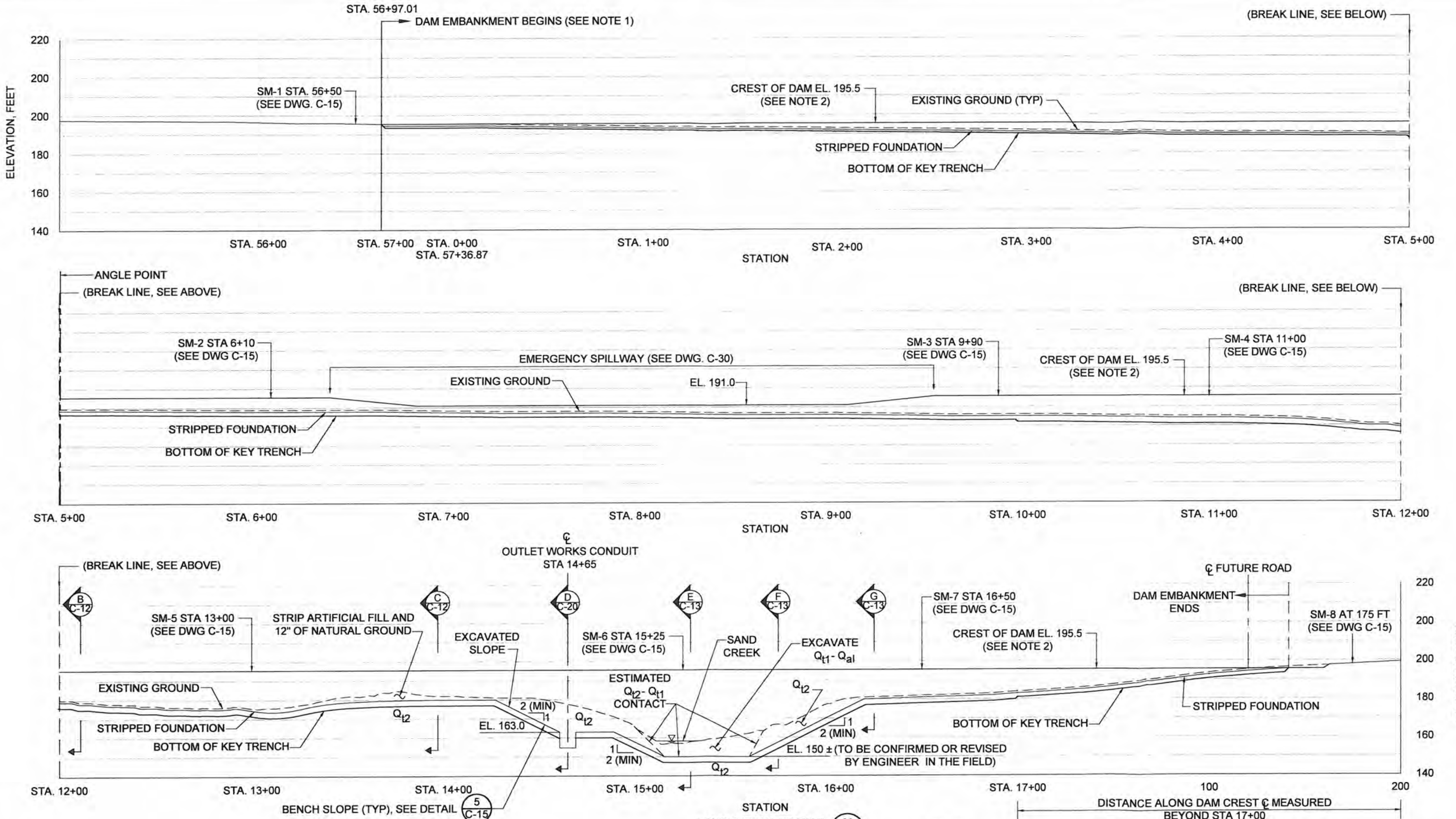
FILE NO. FD-3210

SHEET XX OF YY

CADD FILE# C-14.dwg

PEN TBL: GEIWALKER.ctb

C-14 08-16-10 PYM



LONGITUDINAL PROFILE M C-1

NOTES

1. FOR DAM ZONING AND CREST DETAILS SEE SECTION A AND DETAIL 1 OF DWG. C-12.
2. DAM CREST EL. 195.5 DOES NOT INCLUDE CAMBER. FOR CAMBER PROFILE SEE DWG. C-15.

DRAFT - 90% REVIEW

C-14

REVISIONS			
NO.	DESCRIPTION	BY	DATE

DES.: K.V.S.
DRAWN: P.Y.M.
CHKD.: A.T.
DATE: 08-16-10
SCALE: 1" = 25'
FLD. BK. XXXX

REGISTERED PROFESSIONAL ENGINEER ALBERTO PUJOL-RIOS No. 37051 EXP CIVIL STATE OF CALIFORNIA

PROJECT ENGINEER

PLANS APPROVAL DATE

GEI Consultants

CONTRA COSTA COUNTY
PUBLIC WORKS DEPARTMENT
255 GLACIER DRIVE
MARTINEZ, CALIFORNIA 94553

0 25 50
SCALE, FEET

FOR REDUCED PLANS ORIGINAL SCALE IS IN INCHES
0 1 2 3

BASE MAP	EAST COORD.	NORTH COORD.
L23	1635	528

UPPER SAND CREEK DETENTION BASIN

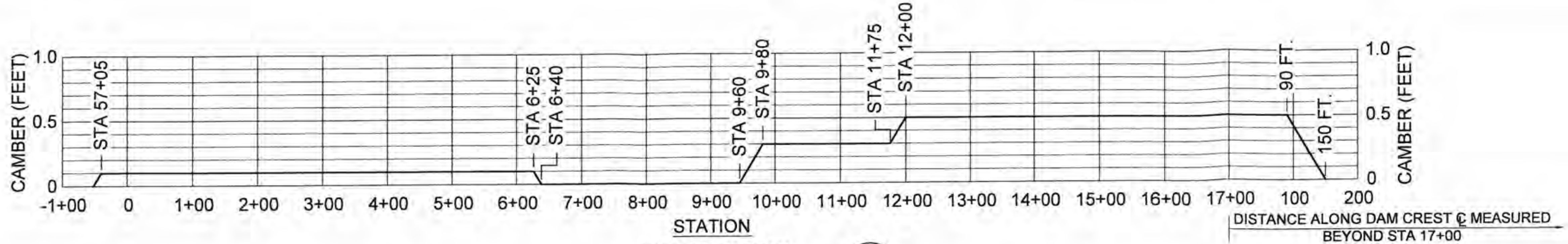
DAM EMBANKMENT - LONGITUDINAL SECTION

FILE NO. FD-3210

SHEET XX OF YY

CADD FILE# C-14.dwg PEN TBL: GEIWALKER.ctb

C-14 08-16-10 PYM



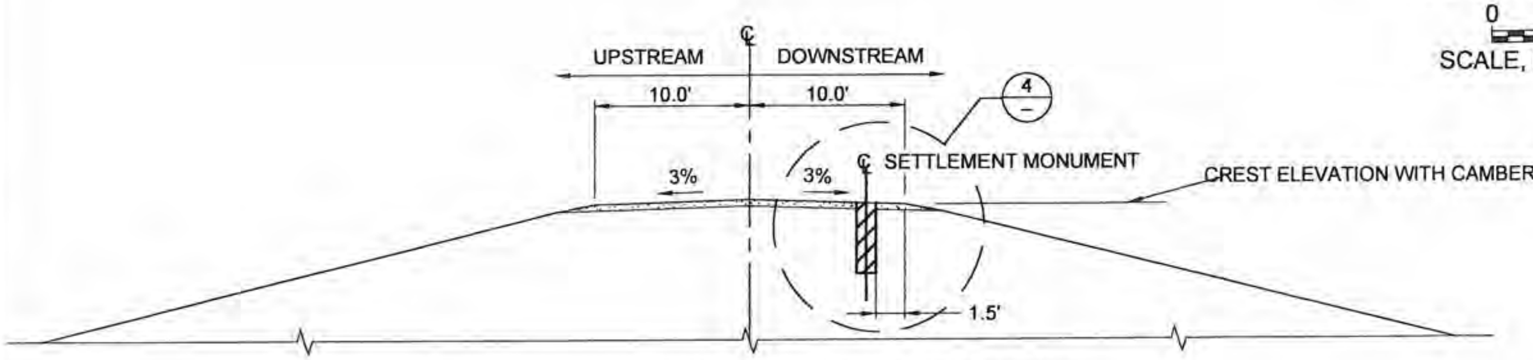
CAMBER DIAGRAM (3 / C-12)

0 100' 200'

SCALE, FEET (HORIZONTAL)

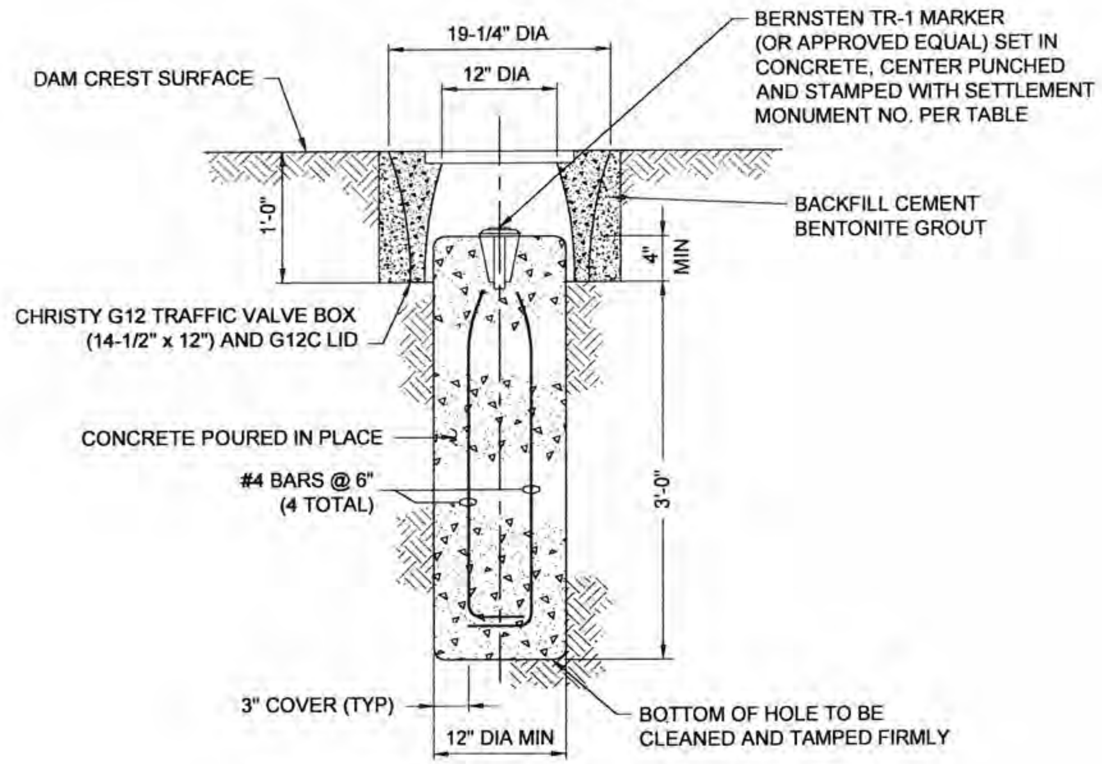
0 0.50' 1.00'

SCALE, FEET (VERTICAL)



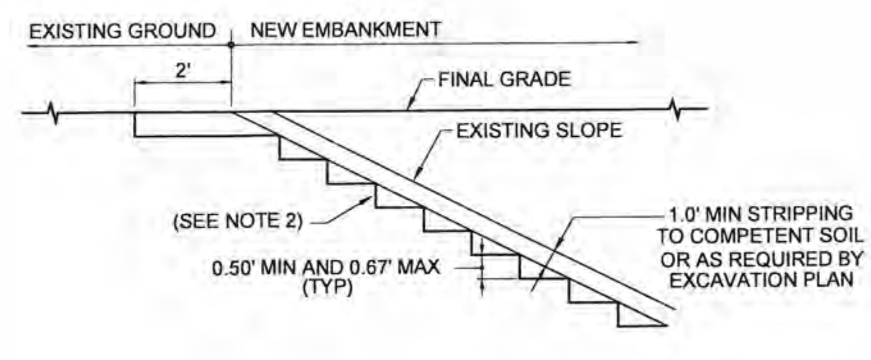
SETTLEMENT MONUMENT PLACEMENT CROSS-SECTION

(NTS)



SETTLEMENT MONUMENT TYPICAL DETAIL (4 / –)

(NTS)



TYPICAL SLOPE BENCHING DETAIL (5 / –)

(NTS)

SETTLEMENT MONUMENT CHART

SETTLEMENT MONUMENT NO.	℄ STATION	AS-BUILT ℄ STATION	AS-BUILT OFFSET	AS-BUILT MONUMENT ELEVATION (SEE NOTE 1)	CREST ELEVATION ADJACENT TO MONUMENT (FT)
SM-1	56+50				195.5
SM-2	6+10				195.6
SM-3	9+90				195.8
SM-4	11+00				195.8
SM-5	13+00				196.0
SM-6	15+25				196.0
SM-7	16+50				196.0
SM-8	175 FT*				198.0

* DISTANCE ALONG DAM CREST CENTERLINE MEASURED BEYOND STA 17+00

NOTES

1. COORDINATES AND ELEVATIONS OF EACH SETTLEMENT MONUMENT SHALL BE SURVEYED IN ACCORDANCE WITH SPECIAL PROVISION 10-1.23. SETTLEMENT MONUMENT SURVEY SHALL BE CONDUCTED NOT EARLIER THAN TWO WEEKS AFTER INSTALLATION.
2. PLACE FILL IN HORIZONTAL LIFTS AGAINST VERTICAL FACES CUT INTO EXISTING MATERIAL. AVOID SLIVER FILL BY EXCAVATING SO AS TO PROVIDE A MINIMUM FILL HORIZONTAL WIDTH OF 10.0'.

DRAFT - 90% REVIEW

C-15

REVISIONS			
NO.	DESCRIPTION	BY	DATE

DES.: K.V.S. / A.P.
DRAWN: P.Y.M.
CHKD.: A.T.
DATE: 08-16-10
SCALE: AS INDICATED
FLD. BK. XXXX

REGISTERED PROFESSIONAL ENGINEER ALBERTO PUJOL-RIUS No. 37051 EXP CIVIL STATE OF CALIFORNIA

PROJECT ENGINEER

PLANS APPROVAL DATE

GEI Consultants

CONTRA COSTA COUNTY
PUBLIC WORKS DEPARTMENT
255 GLACIER DRIVE
MARTINEZ, CALIFORNIA 94553

SCALE, AS INDICATED

FOR REDUCED PLANS ORIGINAL SCALE IS IN INCHES
0 1 2 3

BASE MAP	EAST COORD.	NORTH COORD.
L23	1635	528

UPPER SAND CREEK DETENTION BASIN

DAM EMBANKMENT - CAMBER, SETTLEMENT MONUMENTS & MISCELLANEOUS DETAILS

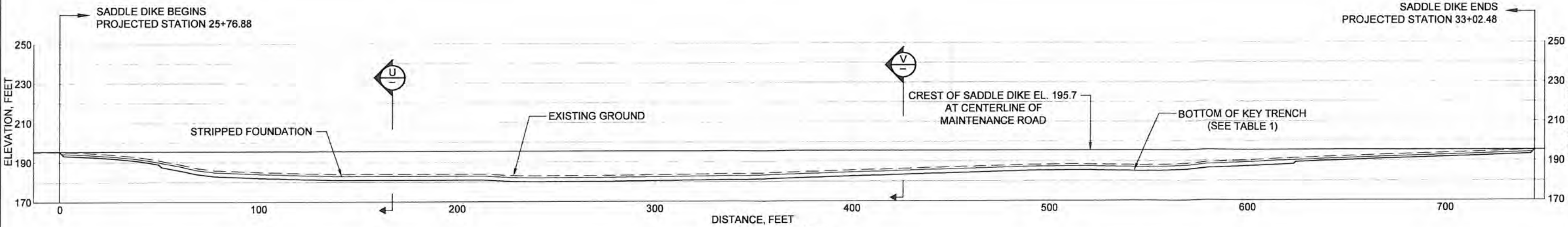
FILE NO FD-3210

SHEET XX OF YY

CADD FILE# C-15.dwg

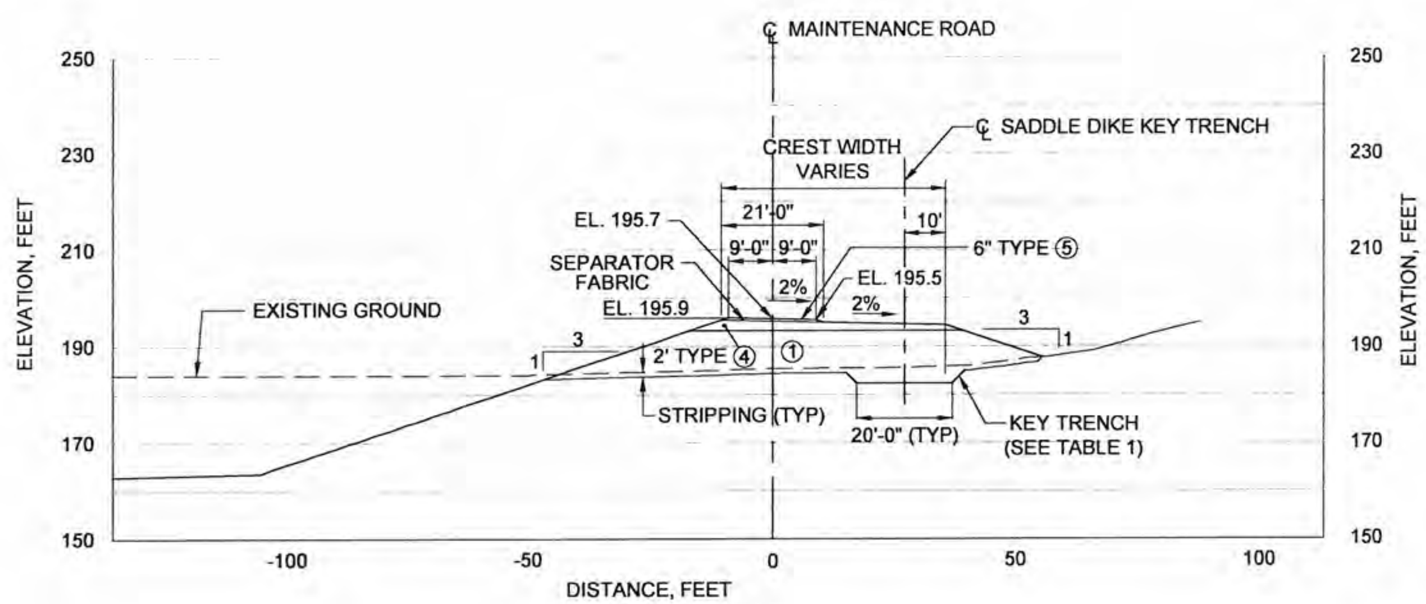
PEN TBL. GEIWALKER.ctb

C-15 08-16-10 PYM

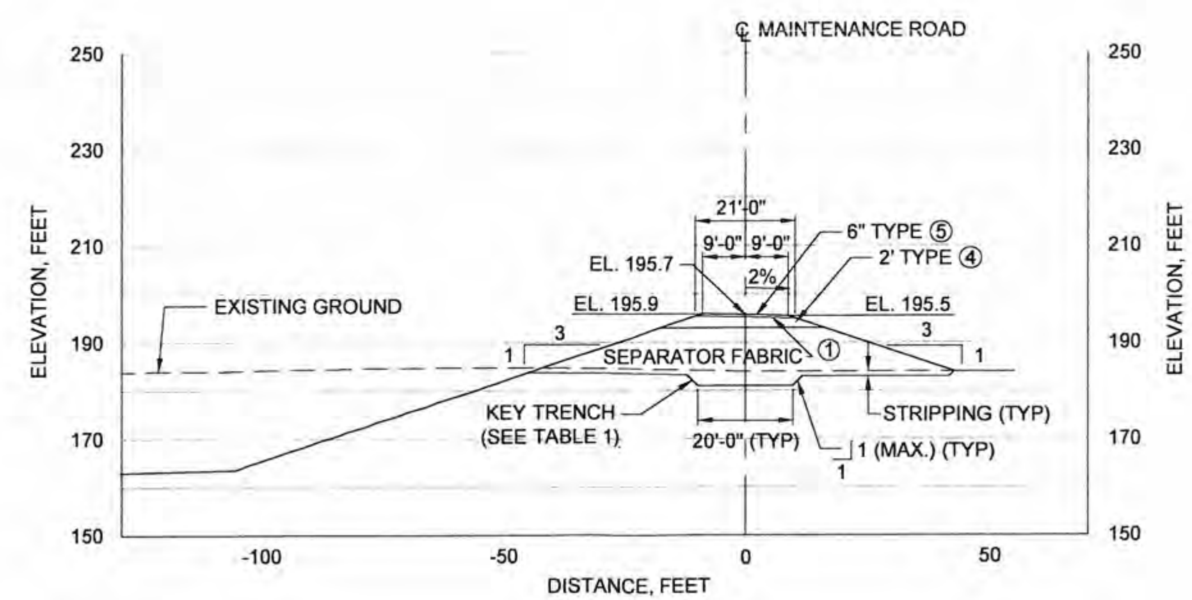


SECTION T / C-1

0 25 50
SCALE, FEET



SECTION U / C-1



SECTION V / C-1

0 20 40
SCALE, FEET

LEGEND

- PROPOSED FINAL GRADE (EMBANKMENT AND EXCAVATION)
- EXISTING GROUND TO REMAIN
- EXISTING GROUND TO BE EXCAVATED

TABLE 1
KEY TRENCH DEPTH BELOW STRIPPED GROUND SURFACE

DISTANCE (FT)	DEPTH (FT)
0 TO 50	1
50 TO 625	2
625 TO 746	1

DRAFT - 90% REVIEW

NOTES

1. KEY TRENCH DEPTHS SHOWN IN TABLE 1 ARE THE MINIMUM REQUIRED AND MAY BE INCREASED BY THE ENGINEER TO SUIT FIELD CONDITIONS EXPOSED BY THE FOUNDATION EXCAVATION.

C-16

REVISIONS			
NO.	DESCRIPTION	BY	DATE

DES.: K.V.S.
DRAWN: P.Y.M.
CHKD.: A.T.
DATE: 08-16-10
SCALE: AS INDICATED
FLD. BK. XXXX

REGISTERED PROFESSIONAL ENGINEER ALBERTO PUJOL-RIUS No 37051 EXP. CIVIL STATE OF CALIFORNIA

PROJECT ENGINEER

PLANS APPROVAL DATE

GEI Consultants

CONTRA COSTA COUNTY
PUBLIC WORKS DEPARTMENT
255 GLACIER DRIVE
MARTINEZ, CALIFORNIA 94553

SCALE AS INDICATED

FOR REDUCED PLANS ORIGINAL SCALE IS IN INCHES
0 1 2 3

BASE MAP	EAST COORD.	NORTH COORD.
L23	1635	528

UPPER SAND CREEK DETENTION BASIN

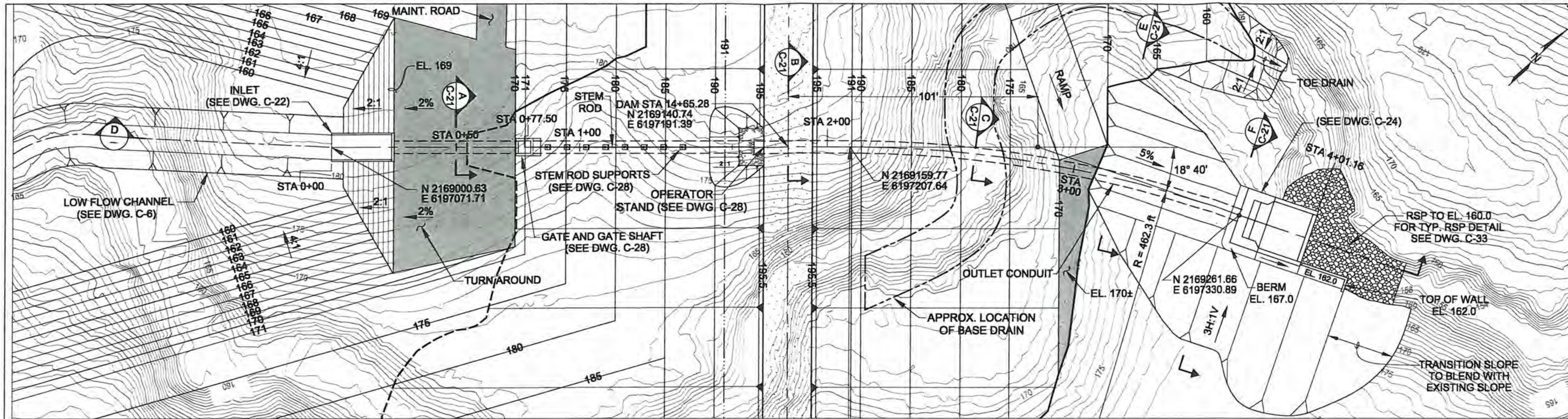
SADDLE DIKE PROFILE & SECTIONS

FILE NO. FD-3210

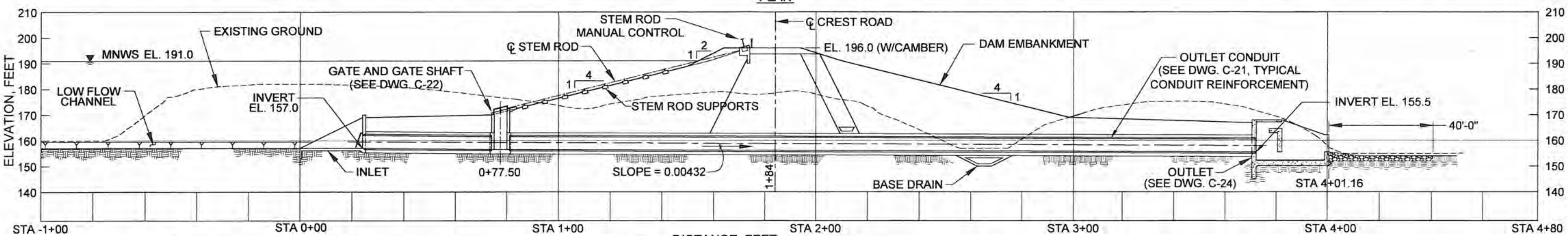
SHEET XX OF YY

CADD FILE# C-16.dwg PEN TBL. GEIWALKER.ctb

C-16 08-16-10 PYM



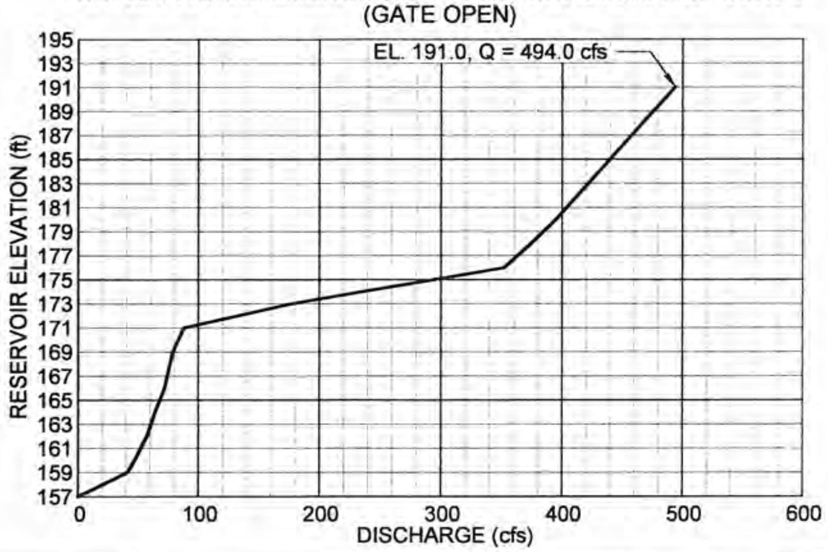
PLAN



DISTANCE, FEET

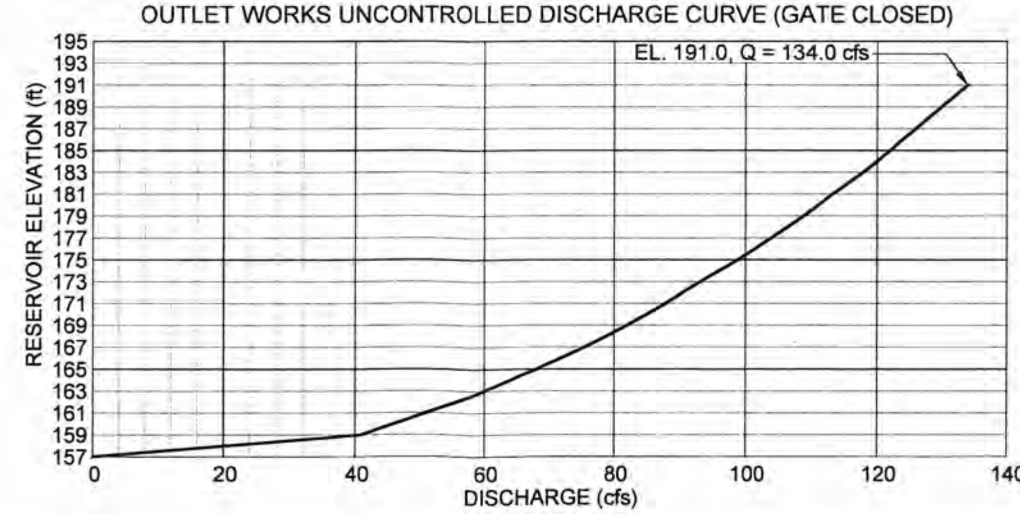
PROFILE ALONG ℄ OUTLET CONDUIT (D / C-1)

OUTLET WORKS EMERGENCY OPERATION DISCHARGE CURVE (GATE OPEN)



DRAFT - 90% REVIEW

OUTLET WORKS UNCONTROLLED DISCHARGE CURVE (GATE CLOSED)



C-20

REVISIONS			
NO.	DESCRIPTION	BY	DATE

DES.: A.T.
DRAWN: P.Y.M.
CHKD.: R.S.
DATE: 08-16-10
SCALE: AS SHOWN
FLD. BK. XXXX

REGISTERED PROFESSIONAL ENGINEER ALBERTO PUJOL-NUIN No. 37051 EXP. CIVIL STATE OF CALIFORNIA

PROJECT ENGINEER

PLANS APPROVAL DATE

GEI Consultants

CONTRA COSTA COUNTY
PUBLIC WORKS DEPARTMENT
255 GLACIER DRIVE
MARTINEZ, CALIFORNIA 94553

0 20 40
SCALE, FEET

FOR REDUCED PLANS ORIGINAL SCALE IS IN INCHES
0 1 2 3

BASE MAP	EAST COORD	NORTH COORD
L23	1635	528

UPPER SAND CREEK DETENTION BASIN

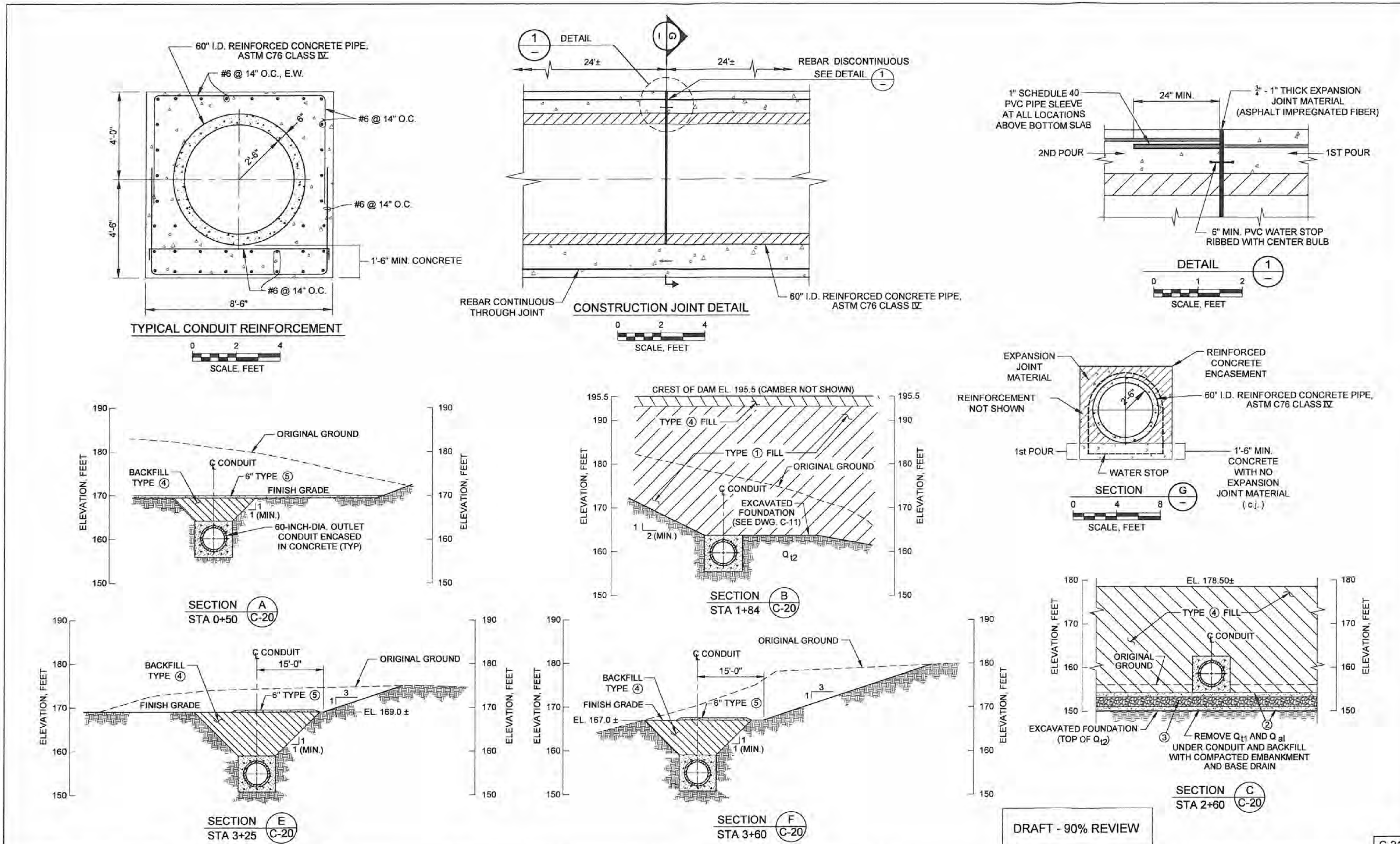
OUTLET WORKS - PLAN, PROFILE & CROSS-SECTIONS

FILE NO. FD-3210

SHEET XX OF YY

CADD FILE# C-20.dwg
PEN TBL. GEIWALKER.ctb

C-20 08-16-10 PYM



DRAFT - 90% REVIEW

C-21

REVISIONS			
NO.	DESCRIPTION	BY	DATE

DES.: AT
DRAWN: PYM
CHKD.: KVS
DATE: 08-16-10
SCALE: 1" = 10'
FLD. BK: XXXX

REGISTERED PROFESSIONAL ENGINEER ALBERTO PUJOL-RIUS No. 37051 EXP CIVIL STATE OF CALIFORNIA

PROJECT ENGINEER

PLANS APPROVAL DATE

GEI Consultants

CONTRA COSTA COUNTY PUBLIC WORKS DEPARTMENT 255 GLACIER DRIVE MARTINEZ, CALIFORNIA 94553

0 10 20
SCALE, FEET (AND NOTED)

FOR REDUCED PLANS ORIGINAL SCALE IS IN INCHES
0 1 2 3

BASE MAP	EAST COORD.	NORTH COORD.
L23	1635	528

UPPER SAND CREEK DETENTION BASIN

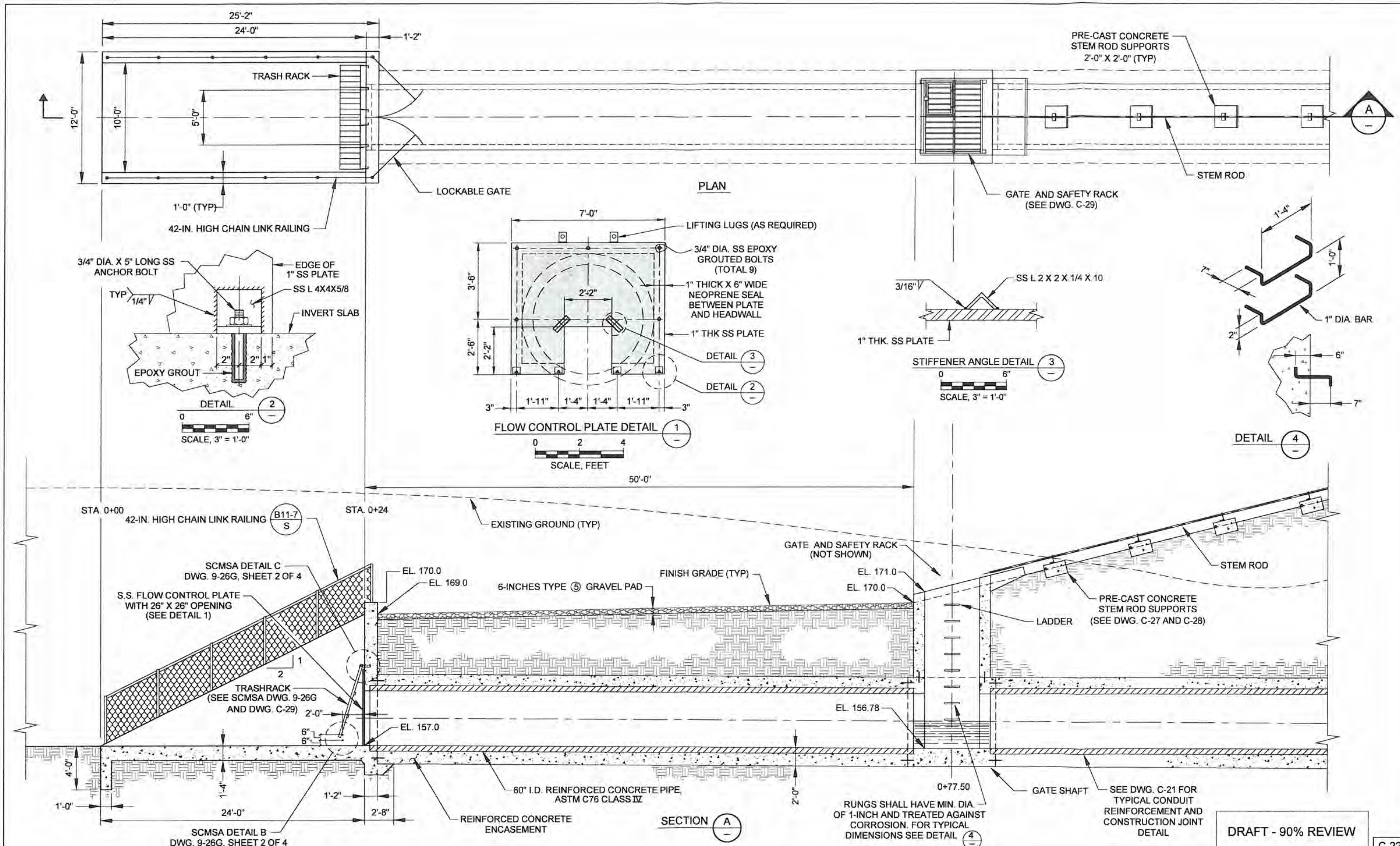
OUTLET WORKS - SECTIONS & DETAILS

FILE NO. FD-3210

SHEET XX OF YY

CADD FILE# C-21.dwg
PEN TBL: GEIWALKER.ctb

C-21 08-16-10 PYM



DRAFT - 90% REVIEW

C-22

REVISIONS			
NO.	DESCRIPTION	BY	DATE

DES.: A.T.
DRAWN: P.Y.M.
CHKD.: R.S.
DATE: 08-16-10
SCALE: 1" = 4' AND NOTED
FLD. BK. XXXX

REGISTERED PROFESSIONAL ENGINEER, ALBERTO PUJOL-RIUS, No. 37051, EXP, CIVIL, STATE OF CALIFORNIA

PROJECT ENGINEER

PLANS APPROVAL DATE

GEI Consultants

CONTRA COSTA COUNTY
PUBLIC WORKS DEPARTMENT
255 GLACIER DRIVE
MARTINEZ, CALIFORNIA 94553

0 4' 8'
SCALE, FEET AND NOTED

FOR REDUCED PLANS ORIGINAL SCALE IS IN INCHES
0 1 2 3

BASE MAP	EAST COORD	NORTH COORD
L23	1635	528

UPPER SAND CREEK DETENTION BASIN

OUTLET WORKS - INLET STRUCTURE DETAILS

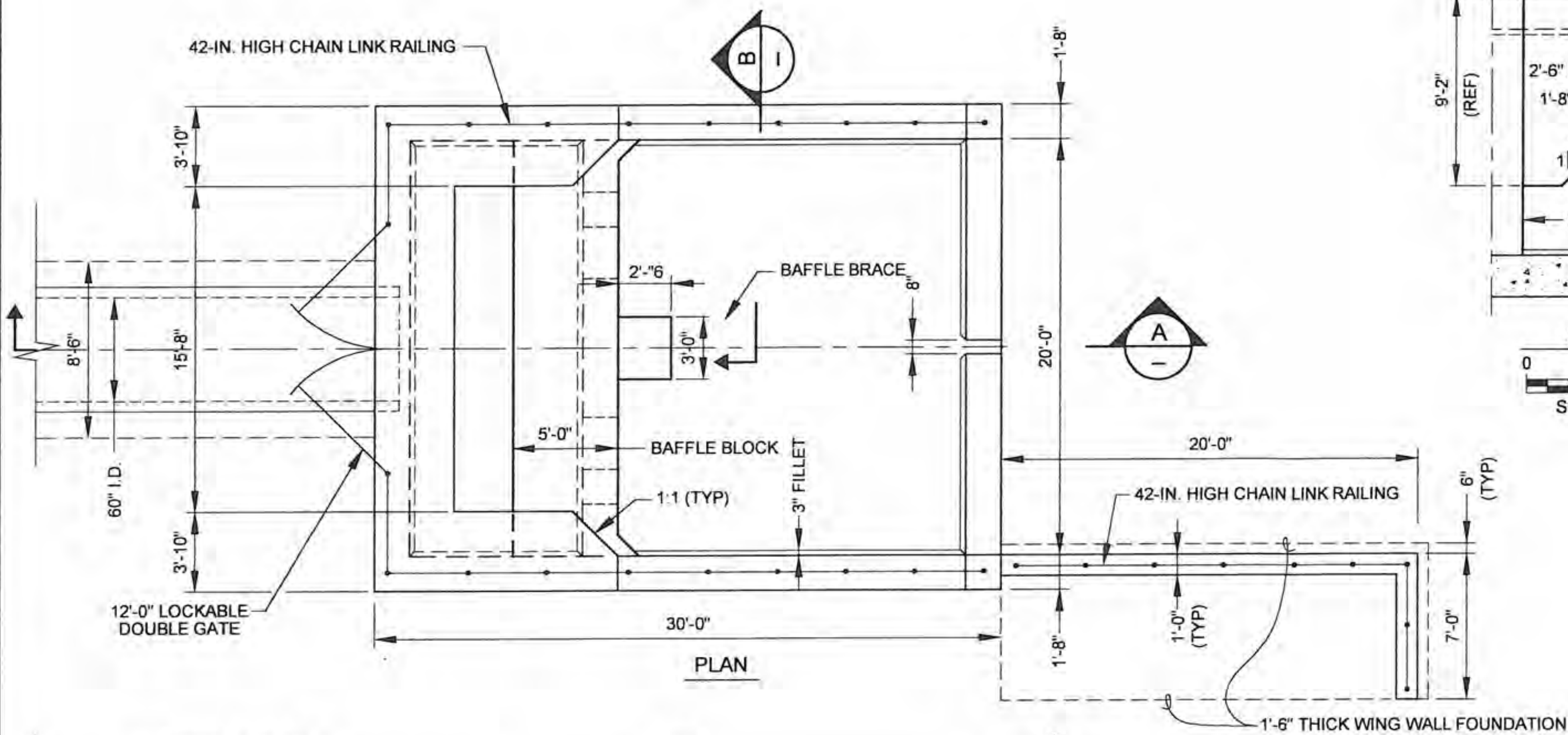
FILE NO. FD-3210

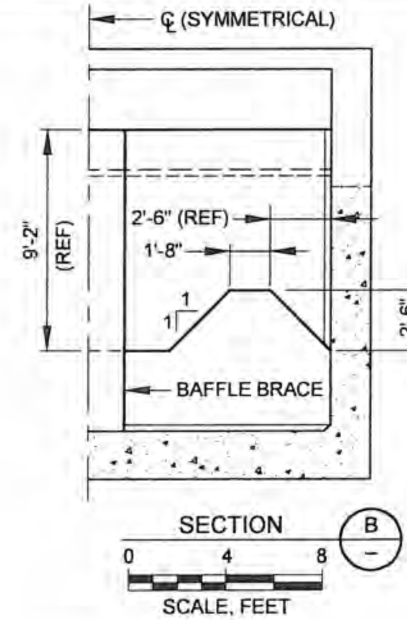
SHEET XX OF YY

CADD FILE# C-22.dwg

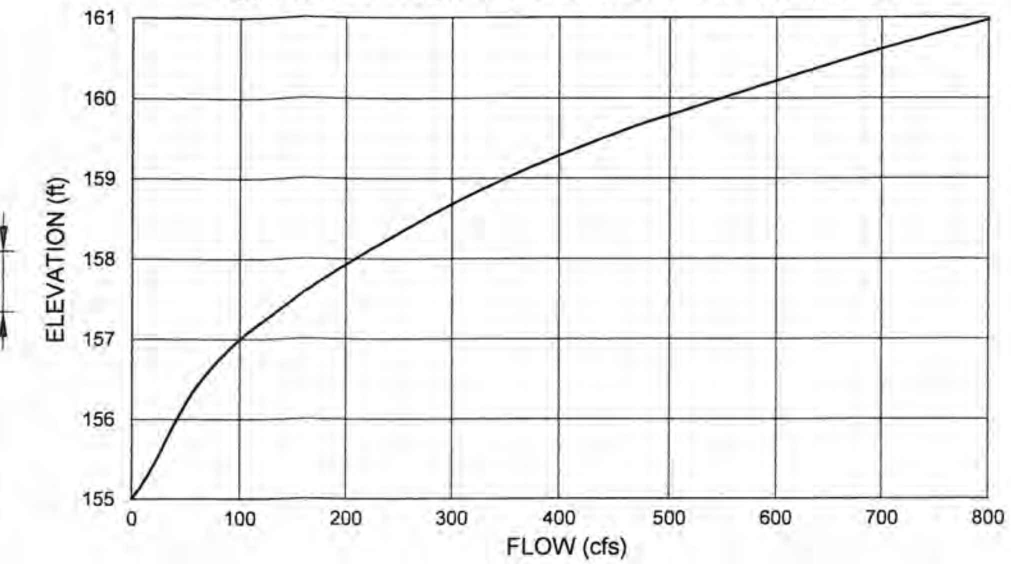
PEN TBL: GEIWALKER.ctb

C-22 08-16-10 PYM



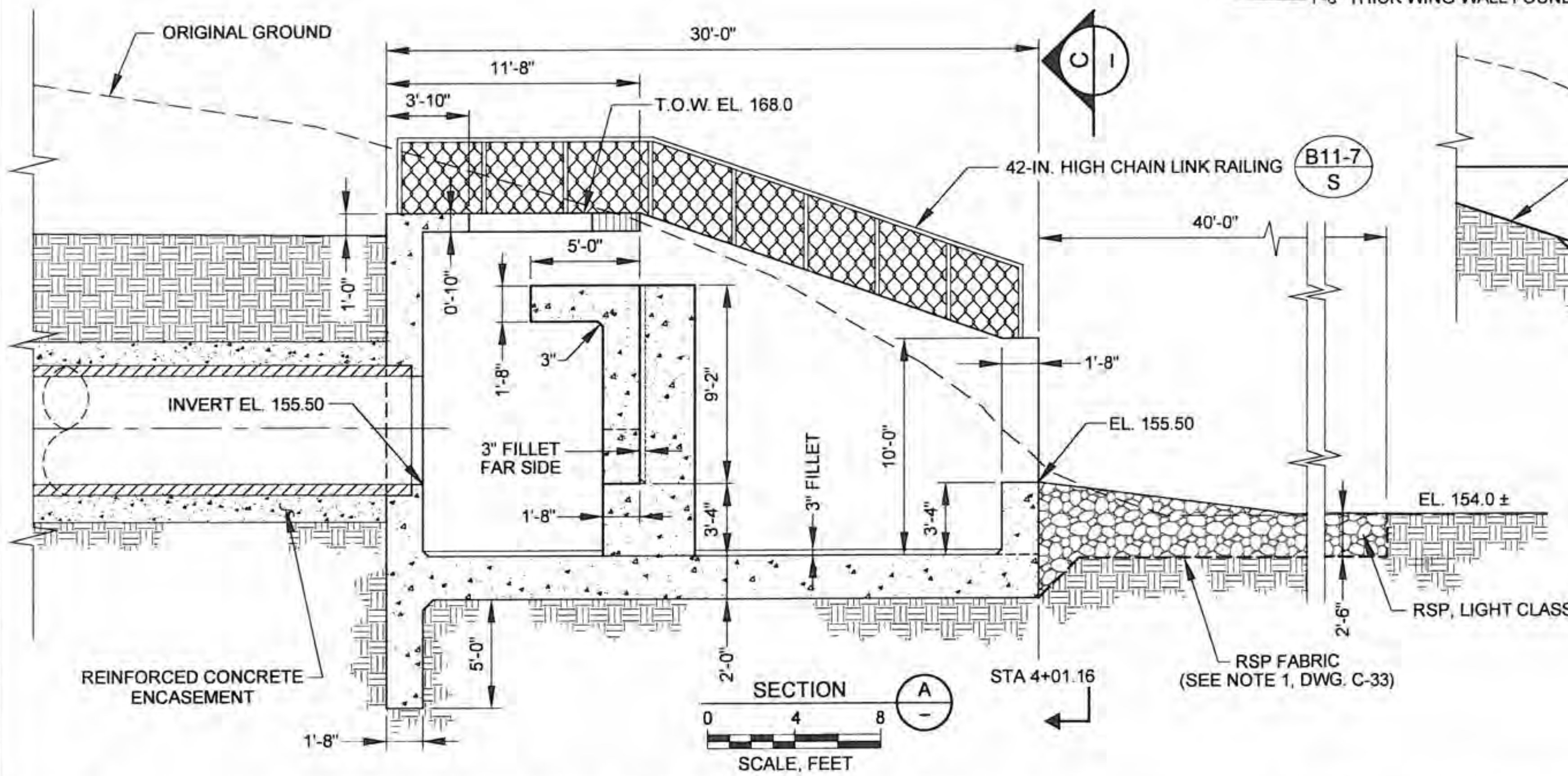


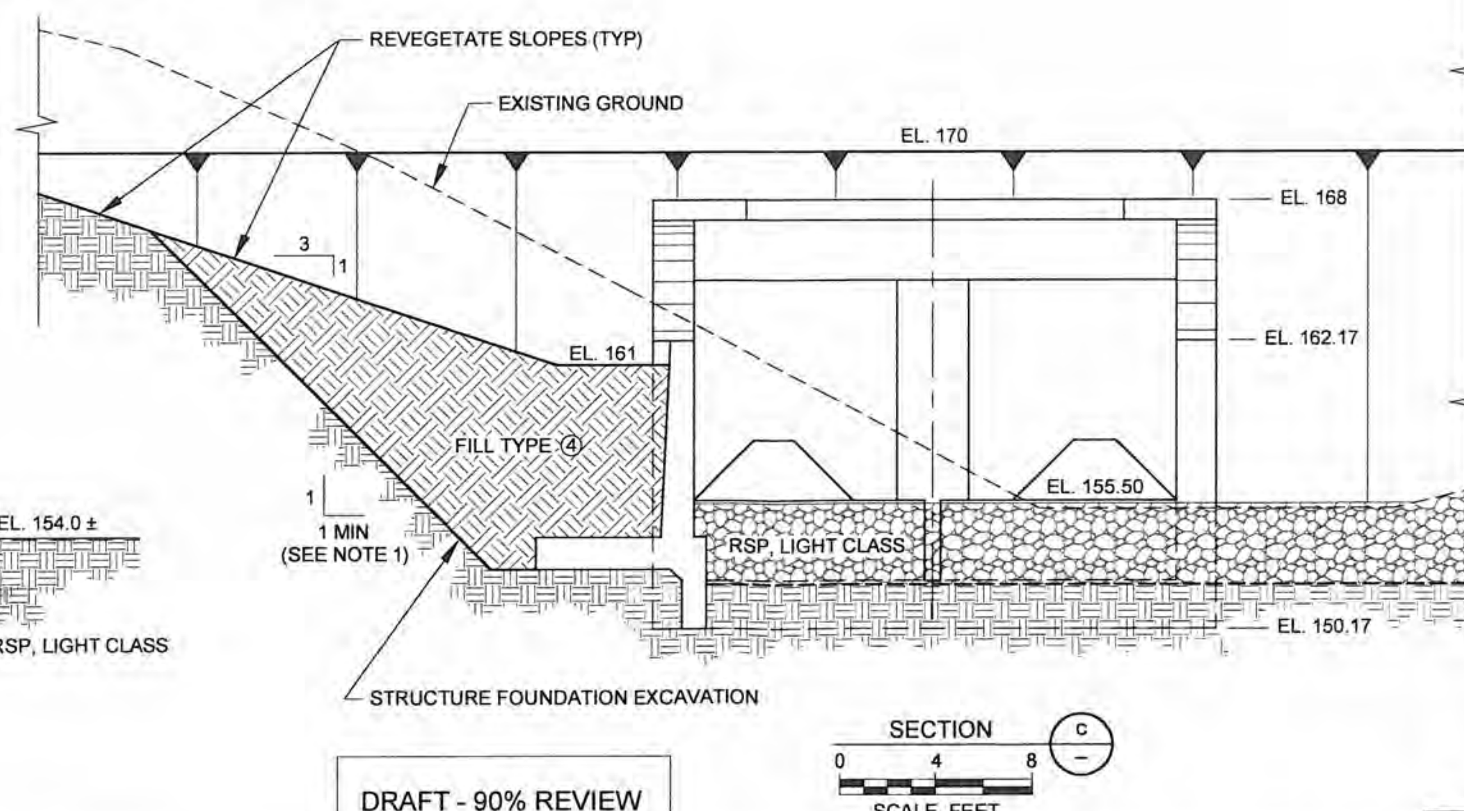
ESTIMATED TAILWATER CURVE AT BOTTOM OUTLET



NOTE

1. EXCAVATED SLOPES SHOWN ARE STEEPEST ALLOWED. CONTRACTOR SHALL EXCAVATE AT FLATTER SLOPES IF REQIRED FOR SAFETY OF TEMPORARY EXCAVATION.





DRAFT - 90% REVIEW

C-24

REVISIONS			
NO.	DESCRIPTION	BY	DATE

DES.: A.T.
DRAWN: P.Y.M.
CHKD.: R.S.
DATE: 08-16-10
SCALE: 1" = 4'
FLD. BK. XXXX

REGISTERED PROFESSIONAL ENGINEER ALBERTO PUJOL-RIUS No. 37051 EXP CIVIL STATE OF CALIFORNIA

PROJECT ENGINEER

PLANS APPROVAL DATE

GEI Consultants

CONTRA COSTA COUNTY
PUBLIC WORKS DEPARTMENT
255 GLACIER DRIVE
MARTINEZ, CALIFORNIA 94553

0 4' 8'
SCALE, FEET

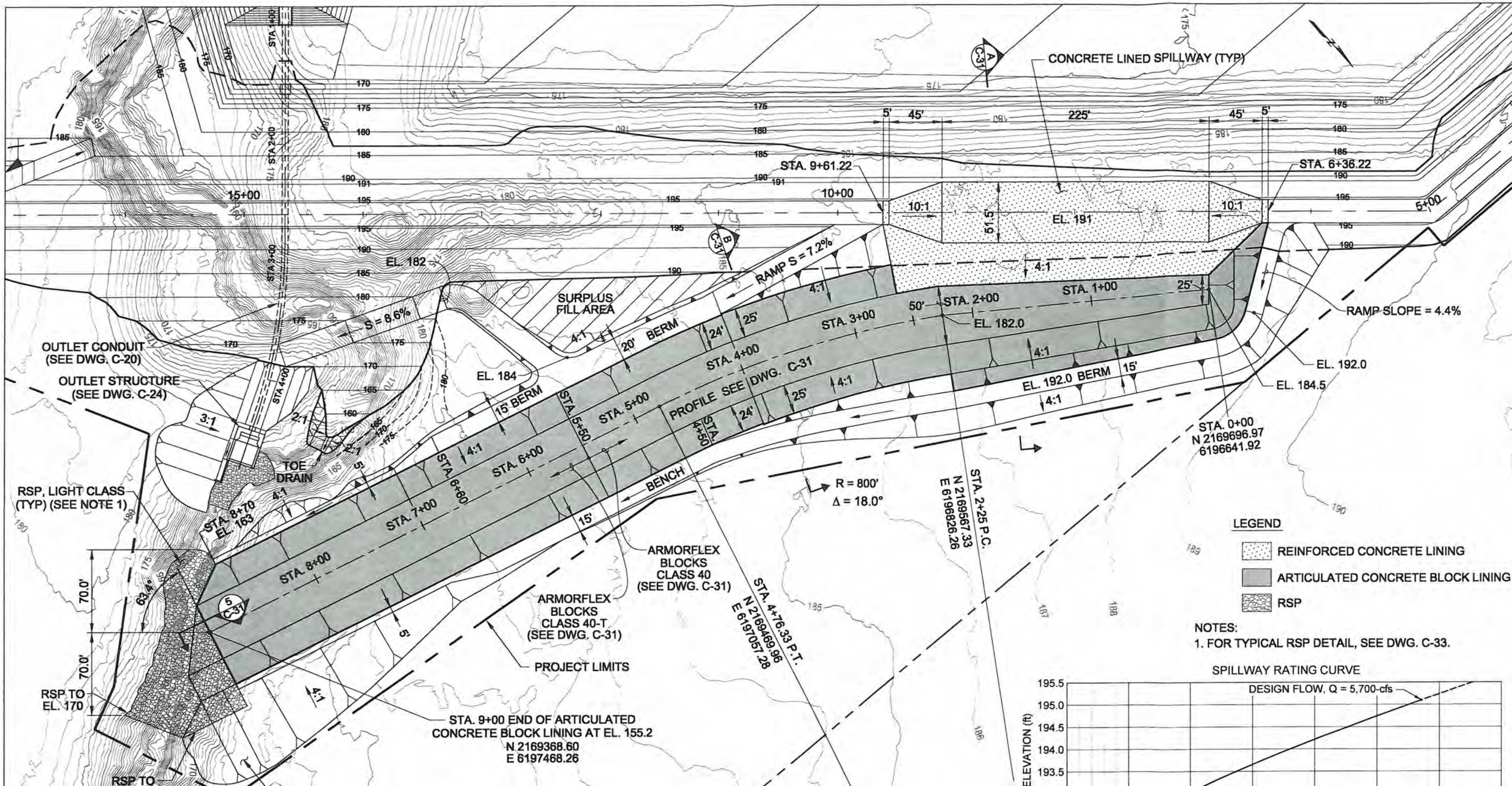
FOR REDUCED PLANS ORIGINAL SCALE IS IN INCHES
0 1 2 3

BASE MAP	EAST COORD.	NORTH COORD.
L23	1635	528

UPPER SAND CREEK DETENTION BASIN
OUTLET WORKS -
OUTLET ENERGY DISSIPATOR SECTIONS & DETAILS
(Sheet 1 of 2)

FILE NO. FD-3210
SHEET XX OF YY
CADD FILE# C-24.dwg
PEN TBL. GEIWALKER.ctb

C-24 08-16-10 PYM



LEGEND

REINFORCED CONCRETE LINING

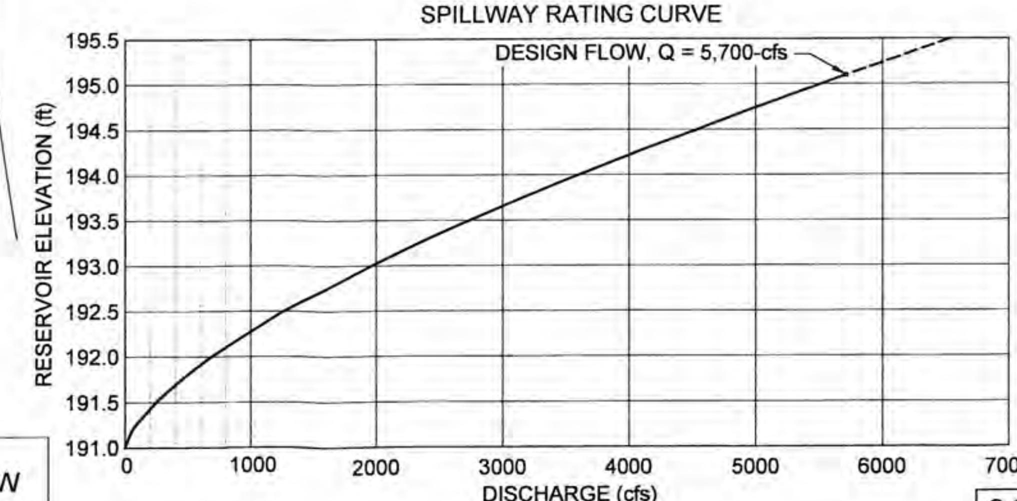
ARTICULATED CONCRETE BLOCK LINING

RSP

NOTES:

1. FOR TYPICAL RSP DETAIL, SEE DWG. C-33.

SPILLWAY RATING CURVE



DRAFT - 90% REVIEW

C-30

REVISIONS			
NO.	DESCRIPTION	BY	DATE

DES.: A.T.
DRAWN: P.Y.M.
CHKD.: R.S.
DATE: 08-16-10
SCALE: 1" = 40'
FLD. BK. XXXX

REGISTERED PROFESSIONAL ENGINEER — ALBERTO PUJOL-RIUS — No. 37051 — EXP — CIVIL — STATE OF CALIFORNIA

PROJECT ENGINEER

PLANS APPROVAL DATE

GEI Consultants

CONTRA COSTA COUNTY
PUBLIC WORKS DEPARTMENT
255 GLACIER DRIVE
MARTINEZ, CALIFORNIA 94553

0 40 80
SCALE, FEET

FOR REDUCED PLANS ORIGINAL SCALE IS IN INCHES
0 1 2 3

BASE MAP	EAST COORD.	NORTH COORD.
L23	1635	528

UPPER SAND CREEK DETENTION BASIN

EMERGENCY SPILLWAY - PLAN & RATING CURVE

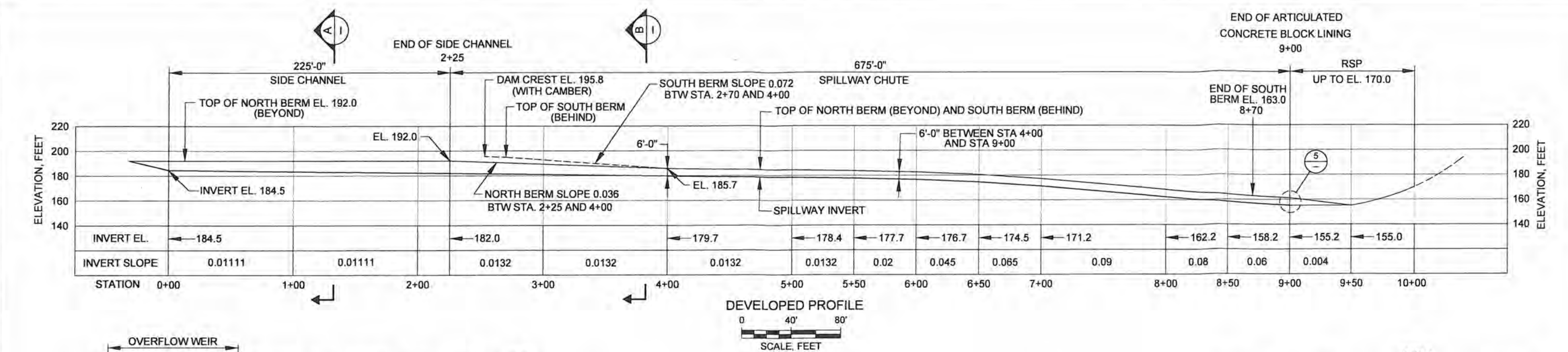
FILE NO. FD-3210

SHEET XX OF YY

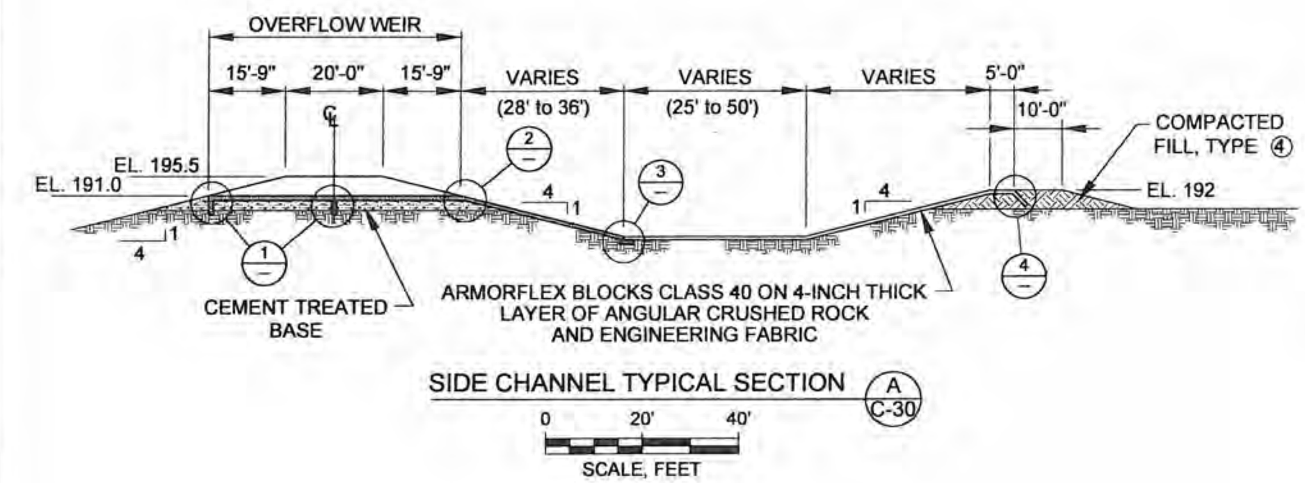
CADD FILE# C-30.dwg

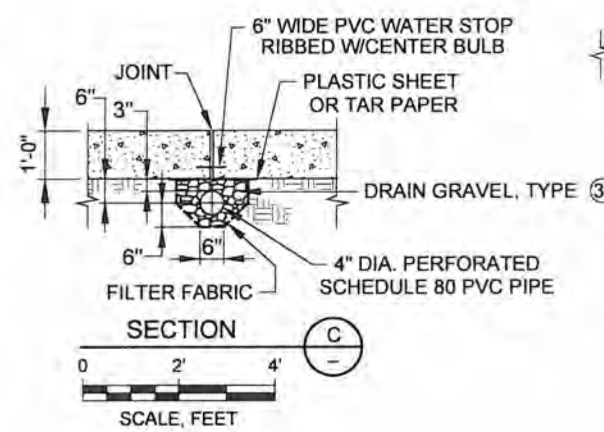
PEN TBL. GEIWALKER.ctb

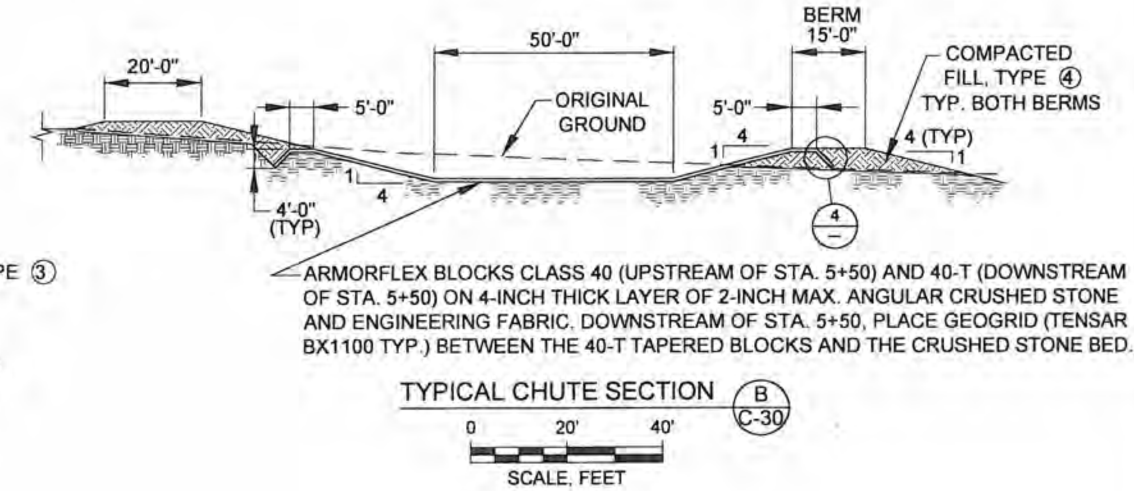
C-30 08-16-10 PYM



STATION	INVERT EL.	INVERT SLOPE
0+00	184.5	0.01111
1+00		0.01111
2+25	182.0	0.0132
3+00		0.0132
4+00	179.7	0.0132
5+00	178.4	0.0132
5+50	177.7	0.02
6+00	176.7	0.045
6+50	174.5	0.065
7+00	171.2	0.09
8+00	162.2	0.08
8+50	158.2	0.06
9+00	155.2	0.004
9+50	155.0	
10+00		

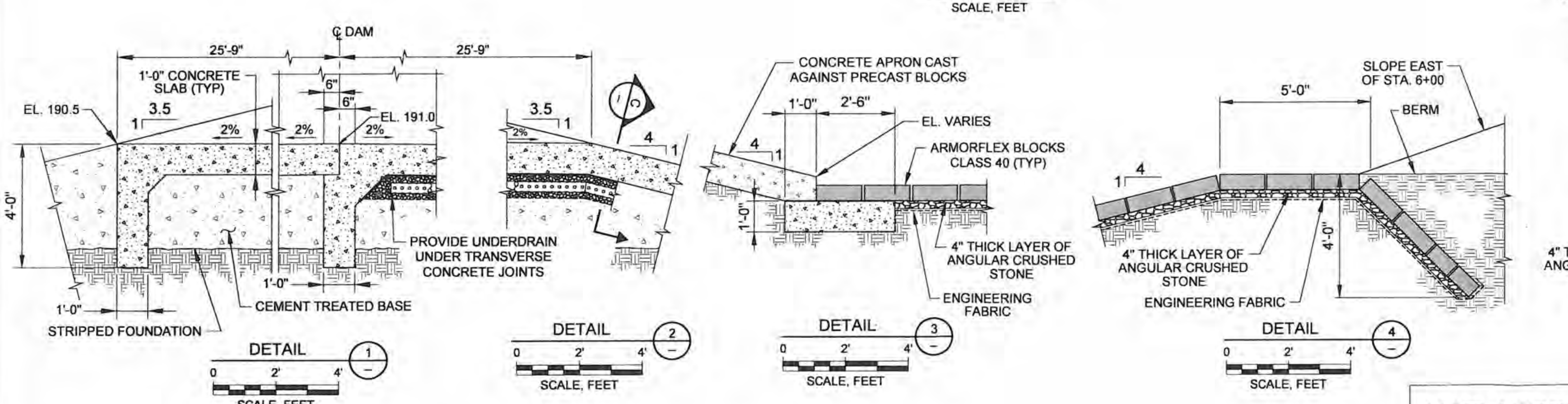


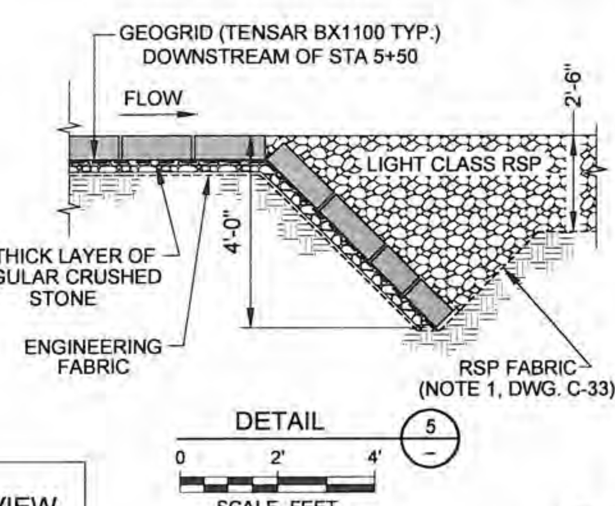




NOTES:

1. FOR TYPICAL RSP DETAIL SEE DWG. C-33.





DRAFT - 90% REVIEW

C-31

REVISIONS			
NO.	DESCRIPTION	BY	DATE

DES.: A.T.
DRAWN: P.Y.M.
CHKD.: R.S.
DATE: 08-16-10
SCALE: 1" = 40'
FLD. BK.: XXXX

REGISTERED PROFESSIONAL ENGINEER No. 37051 CIVIL STATE OF CALIFORNIA

PROJECT ENGINEER

PLANS APPROVAL DATE

GEI Consultants

CONTRA COSTA COUNTY
PUBLIC WORKS DEPARTMENT
255 GLACIER DRIVE
MARTINEZ, CALIFORNIA 94553

SCALE, FEET

FOR REDUCED PLANS ORIGINAL SCALE IS IN INCHES

BASE MAP	EAST COORD.	NORTH COORD.
L23	1635	528

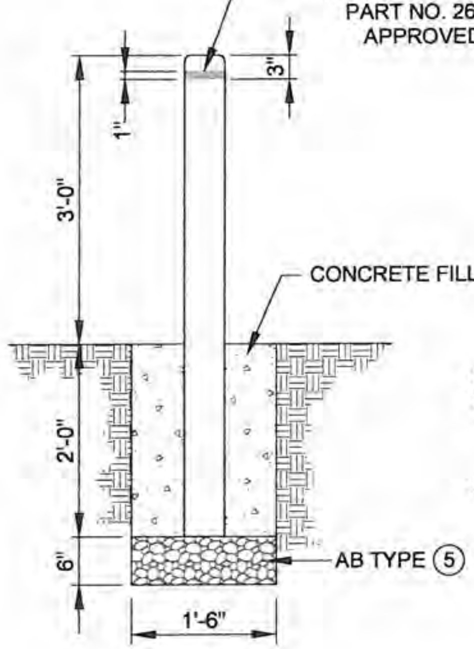
UPPER SAND CREEK DETENTION BASIN

EMERGENCY SPILLWAY - PROFILE, SECTIONS & DETAILS

FILE NO. FD-3210

SHEET XX OF YY

CADD FILE# C-31.dwg
PEN TBL. GEIWALKER.ctb

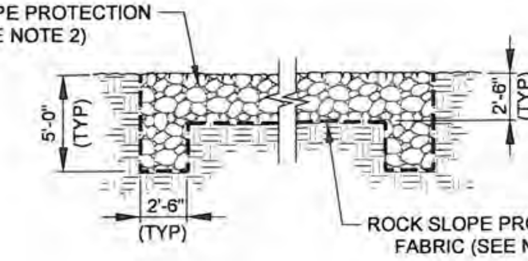


BOLLARD DETAIL

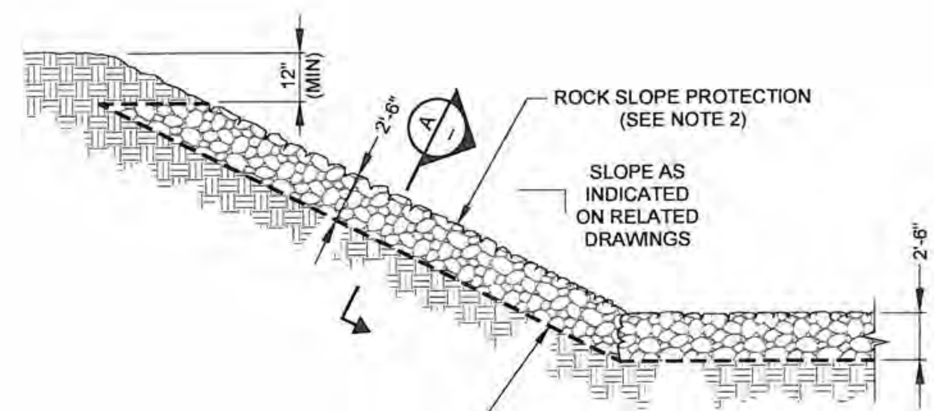
0 1 2
SCALE, FEET

BOLLARD NOTES:

1. BOLLARDS SHALL BE FABRICATED FROM 5" OUTER DIAMETER SCHEDULE 80 STEEL PIPE WITH DOMED TOP.
2. BOLLARDS SHALL BE POWDER COATED AS PER MANUFACTURER'S RECOMMENDATIONS AND SHALL BE BROWN IN COLOR.



SECTION A/–
N.T.S.



TYPICAL DETAIL
ROCK SLOPE PROTECTION
N.T.S.

NOTES

1. ROCK SLOPE PROTECTION FABRIC SHALL BE TYPE B IN ACCORDANCE WITH SECTION 88 AND PLACED IN ACCORDANCE WITH SECTION 72-2 OF THE STATE STANDARD SPECIFICATIONS.
2. ROCK SLOPE PROTECTION SHALL BE "LIGHT" CLASS IN ACCORDANCE WITH SECTION 72-2.02 OF THE STATE STANDARD SPECIFICATIONS.

DRAFT - 90% REVIEW

C-33

REVISIONS			
NO.	DESCRIPTION	BY	DATE

DES.: AT
DRAWN: PYM
CHKD.: KVS
DATE: 08-16-10
SCALE: 1" = 10'
FLD. BK.: XXXX

REGISTERED PROFESSIONAL ENGINEER ALBERTO PUJOL-KIUB No. 37051 EXP CIVIL STATE OF CALIFORNIA

PROJECT ENGINEER

PLANS APPROVAL DATE

GEI Consultants

CONTRA COSTA COUNTY
PUBLIC WORKS DEPARTMENT
255 GLACIER DRIVE
MARTINEZ, CALIFORNIA 94553

0 10 20
SCALE, FEET
(AND NOTED)

FOR REDUCED PLANS ORIGINAL SCALE IS IN INCHES
0 1 2 3

BASE MAP	EAST COORD.	NORTH COORD.
L23	1635	528

UPPER SAND CREEK DETENTION BASIN

SLOPE PROTECTION DETAILS

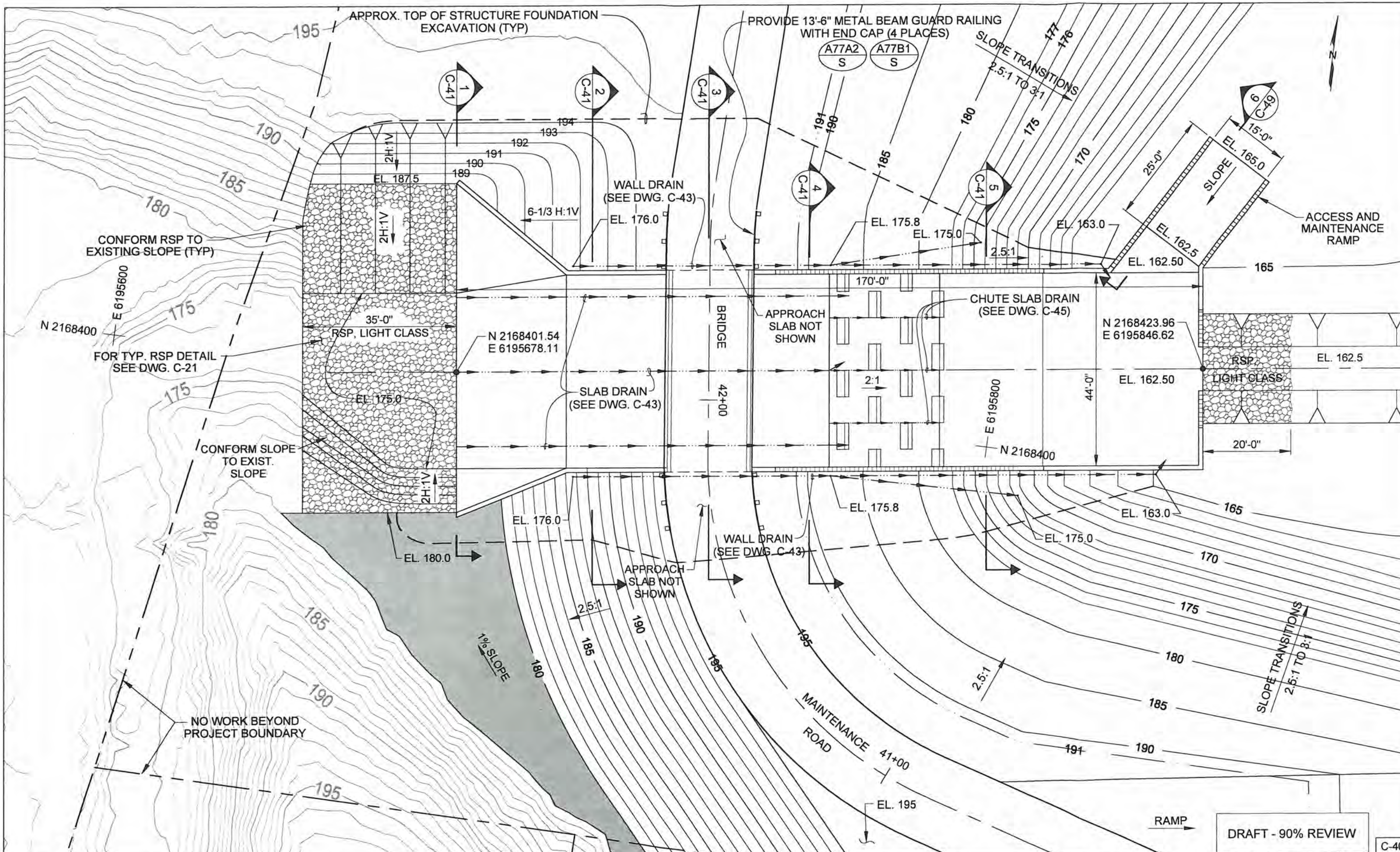
FILE NO. FD-3210

SHEET XX OF YY

CADD FILE# C-33.dwg

PEN TBL. GEIWALKER.ctb

C-33 08-17-10 PYM



REVISIONS			
NO.	DESCRIPTION	BY	DATE

DES.: K.V.S.
DRAWN: P.Y.M.
CHKD.: A.T.
DATE: 08-16-10
SCALE: 1" = 10'
FLD. BK. XXXX

REGISTERED PROFESSIONAL ENGINEER ALBERTO PUJOL-RIUS No. 37051 EXP CIVIL STATE OF CALIFORNIA

PROJECT ENGINEER

PLANS APPROVAL DATE

GEI Consultants

CONTRA COSTA COUNTY
PUBLIC WORKS DEPARTMENT
255 GLACIER DRIVE
MARTINEZ, CALIFORNIA 94553

0 10 20
SCALE, FEET

FOR REDUCED PLANS ORIGINAL SCALE IS IN INCHES
0 1 2 3

BASE MAP	EAST COORD	NORTH COORD
L23	1635	528

UPPER SAND CREEK DETENTION BASIN

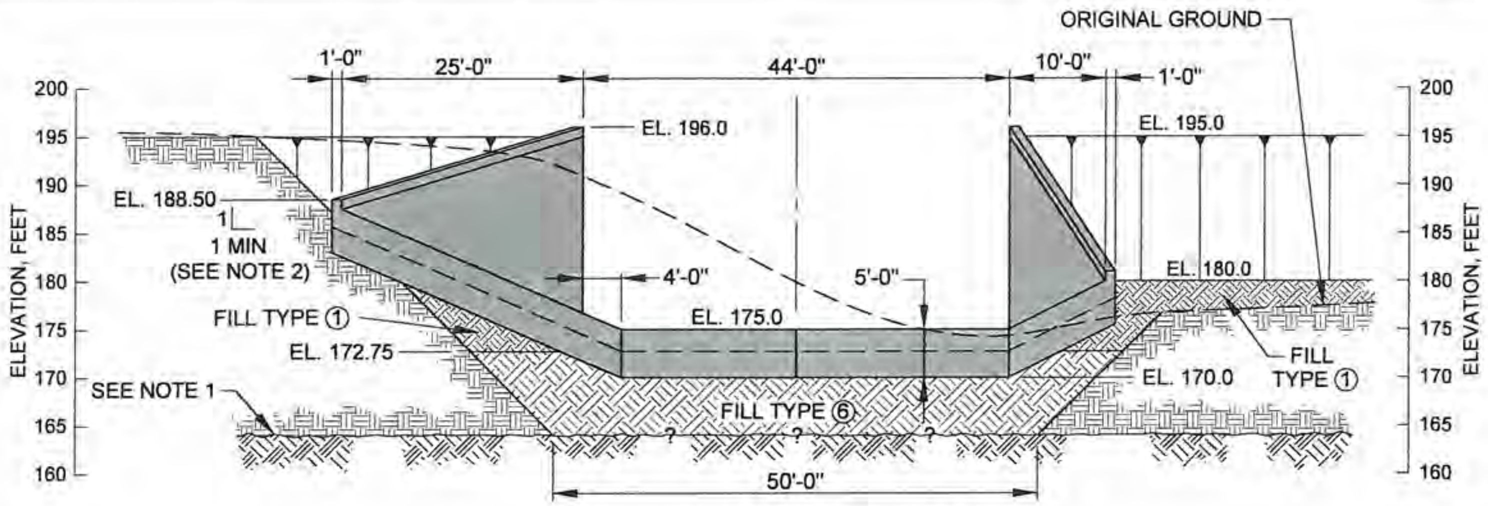
BASIN INLET STRUCTURE - GENERAL ARRANGEMENT

FILE NO. FD-3210

SHEET XX OF YY

CADD FILE# C-40.dwg
PEN TBL GEIWALKER.ctb

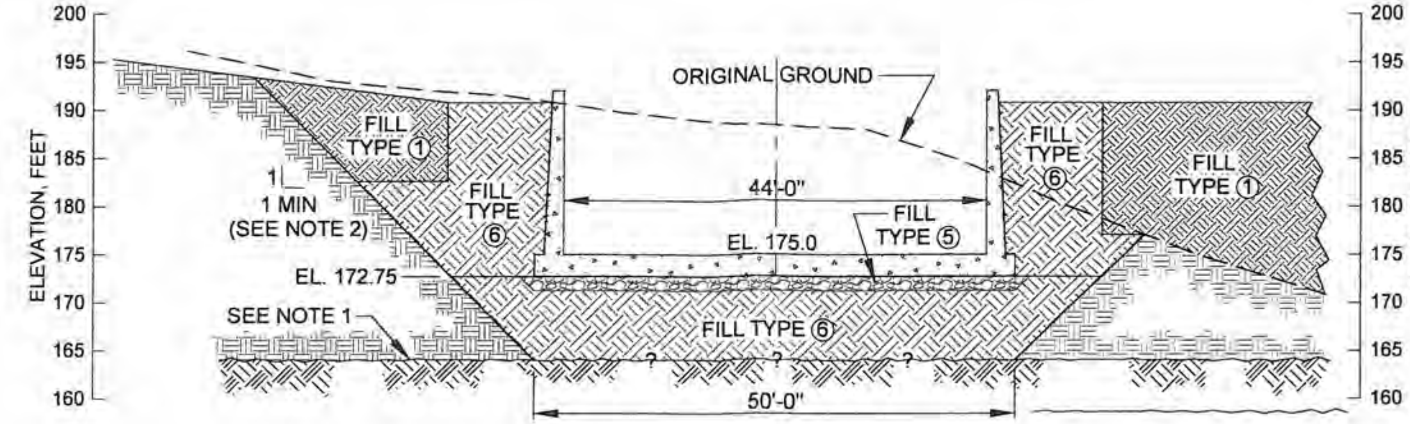
C-40 08-16-10 PYM



SECTION 1 C-40

0 10' 20'

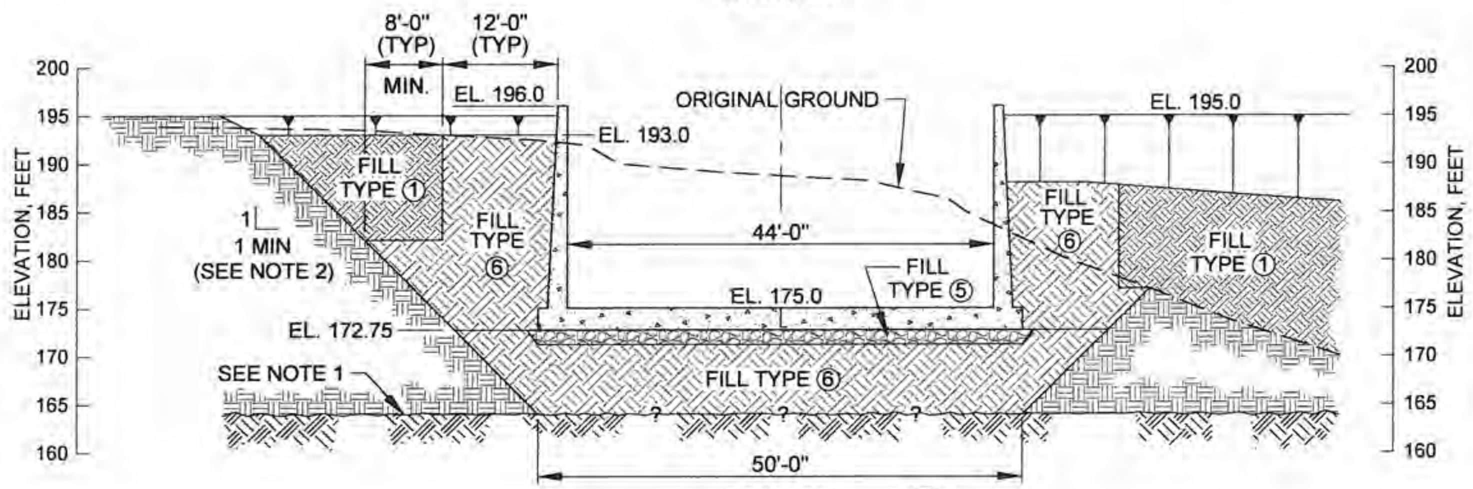
SCALE, FEET



SECTION 4 C-40

0 10' 20'

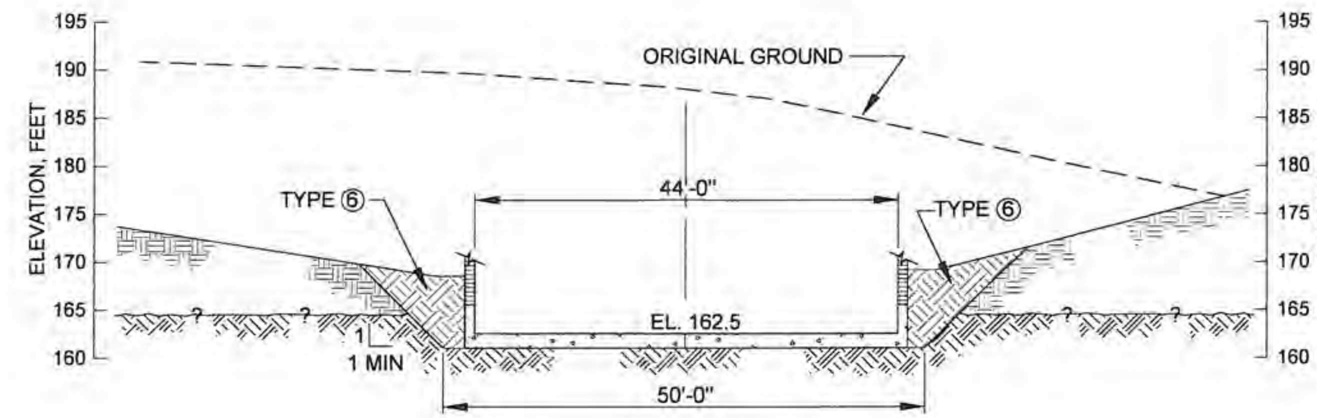
SCALE, FEET



SECTION 2 C-40

0 10' 20'

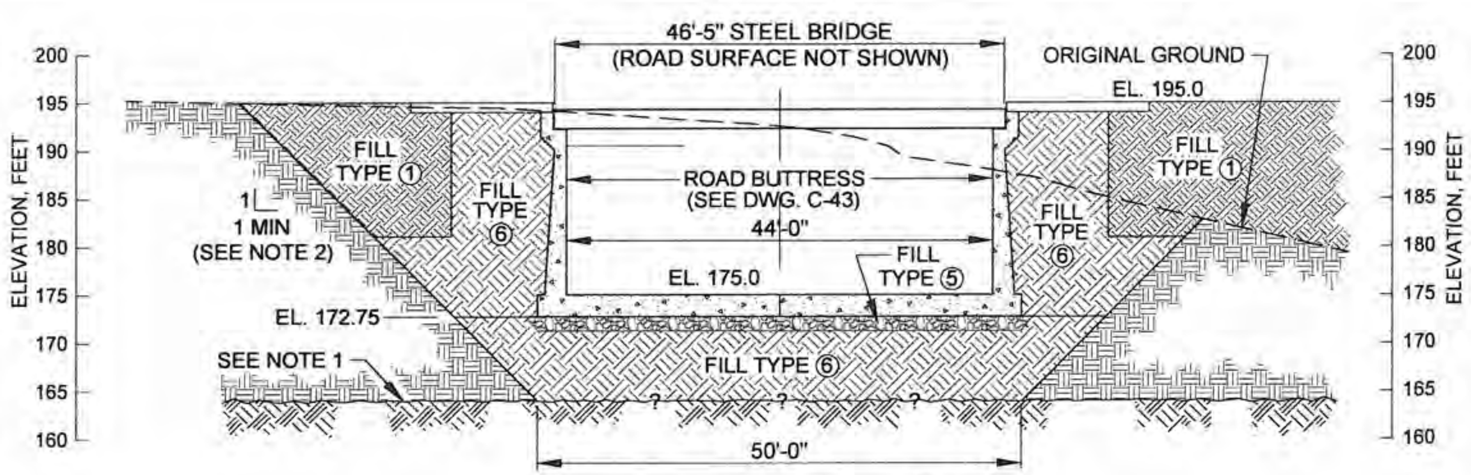
SCALE, FEET



SECTION 5 C-40

0 10' 20'

SCALE, FEET



SECTION 3 C-40

0 10' 20'

SCALE, FEET

NOTES:

1. STRUCTURE EXCAVATION SHALL EXTEND TO THE TOP OF WEATHERED BEDROCK (MARKLEY SANDSTONE), OR DEEPER IF REQUIRED BY THE STRUCTURE GEOMETRY (SUCH AS UNDER THE STILLING BASIN).
2. EXCAVATED SLOPES SHOWN ARE STEEPEST ALLOWED. CONTRACTOR SHALL EXCAVATE AT FLATTER SLOPES IF REQUIRED FOR SAFETY OF TEMPORARY EXCAVATION .
3. WALL DRAINS ARE NOT SHOWN.

DRAFT - 90% REVIEW

C-41

REVISIONS			
NO.	DESCRIPTION	BY	DATE

DES.: A.T.
DRAWN: P.Y.M.
CHKD.: K.V.S.
DATE: 08-16-10
SCALE: 1" = 10'-0"
FLD. BK. XXXX

REGISTERED PROFESSIONAL ENGINEER · ALBERTO PUJOL-RIUS · No. 37051 · EXP · CIVIL · STATE OF CALIFORNIA

PROJECT ENGINEER

PLANS APPROVAL DATE

GEI Consultants

CONTRA COSTA COUNTY
PUBLIC WORKS DEPARTMENT
255 GLACIER DRIVE
MARTINEZ, CALIFORNIA 94553

0 10' 20'
SCALE, FEET

FOR REDUCED PLANS ORIGINAL SCALE IS IN INCHES
0 1 2 3

BASE MAP	EAST COORD	NORTH COORD
L23	1635	528

UPPER SAND CREEK DETENTION BASIN

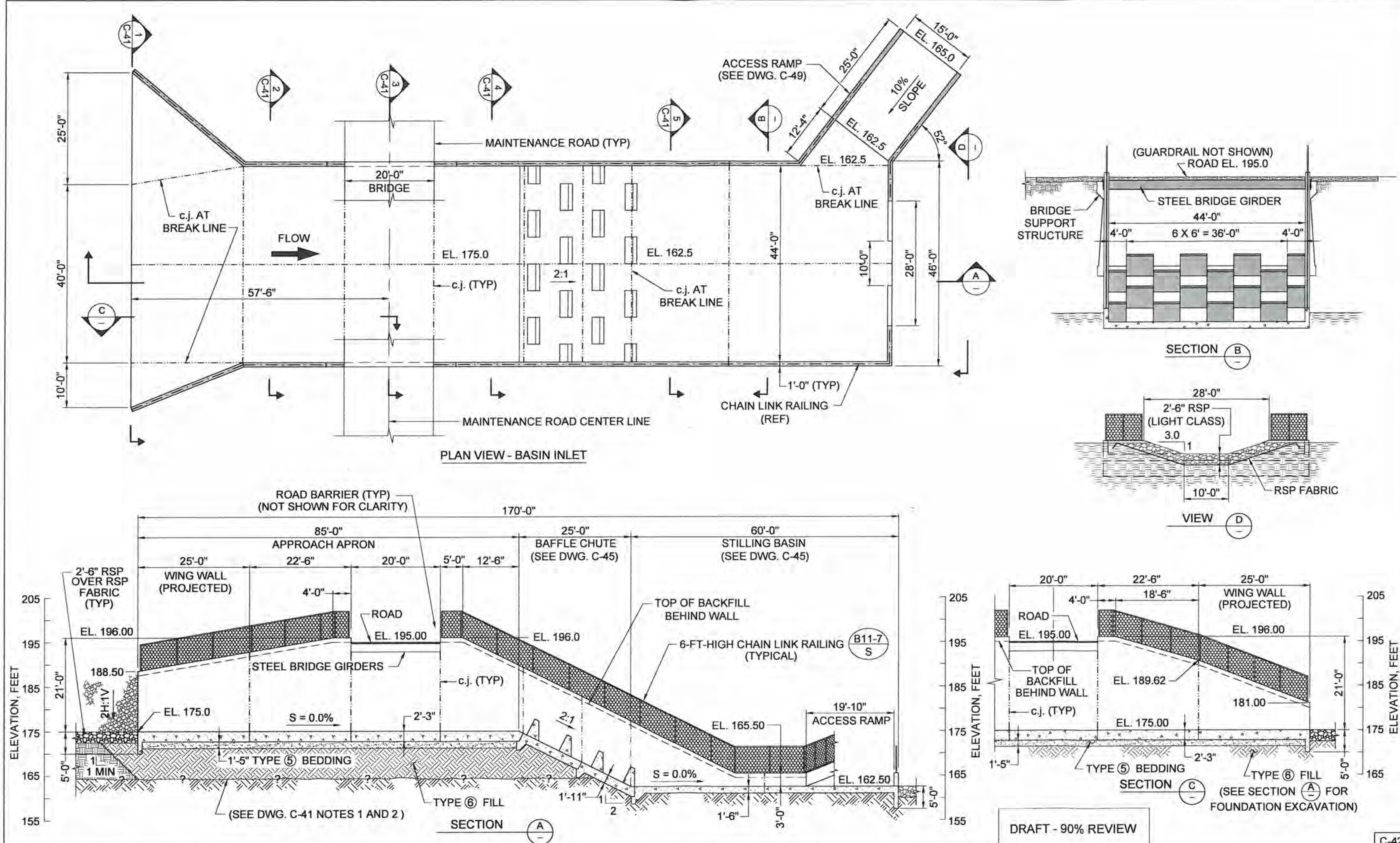
BASIN INLET STRUCTURE -
EXCAVATION AND BACKFILL -
PLAN, SECTIONS & DETAILS

FILE NO. FD-3210

SHEET XX OF YY

CADD FILE# C-41.dwg PEN TBL. GEIWALKER.ctb

C-41 08-16-10 PYM



C-42

REVISIONS			
NO.	DESCRIPTION	BY	DATE

DES.: A.T.
DRAWN: P.Y.M.
CHKD.: K.V.S.
DATE: 08-16-10
SCALE: 1" = 10'
FLD. BK. XXXX

PROJECT ENGINEER
PLANS APPROVAL DATE

GEI Consultants

CONTRA COSTA COUNTY
PUBLIC WORKS DEPARTMENT
255 GLACIER DRIVE
MARTINEZ, CALIFORNIA 94553

SCALE, FEET 0 10 20

FOR REDUCED PLANS ORIGINAL SCALE IS IN INCHES

BASE MAP	EAST COORD.	NORTH COORD.
L23	1635	528

UPPER SAND CREEK DETENTION BASIN
BASIN INLET STRUCTURE - PLAN & PROFILE

FILE NO. FD-3210
SHEET XX OF YY

Appendix 1 –DSOD Review Comments and Proposed Responses

- Appendix 1.1 - Draft Responses to DSOD Review Comments Dated July 31, 2009 on Draft Geotechnical Work Plan and Draft Design Basis Memorandum
- Appendix 1.2 - Draft Responses to DSOD Review Comments (undated) on Geotechnical Study Report Dated June 2004

Appendix 1.1

Draft Responses to:

Division of Safety of Dams Review Comments Dated July 31, 2009 on Draft Geotechnical Work Plan and Draft Design Basis Memorandum for Upper Sand Creek Detention Basin Dam, No. 1007-9 Contra Costa County

Reference:

1. GEI, Upper Sand Creek Detention Basin Design Basis Memorandum, May 2009.
2. CE&G, Upper Sand Creek Detention Basis Geotechnical Work Plan, May 2009.

DSOD Comments:

1) The proposed dam is located in a geologically complex setting; therefore, the proposed test pits should be excavated prior to drilling to become acquainted with the detailed Quaternary stratigraphy. Some boring locations may then need to be revised to target specific Quaternary units such as Qt1. The Division of Safety of Dams (DSOD) will inspect the conditions revealed in the test pits and subsequent drilling during the course of this investigation.

Design Team Response: Concur. The design team coordinated the test pit excavations and logging with DSOD personnel. The information obtained from the test pit excavations was utilized to refine test boring locations as necessary. In regards to ascertaining the extent of specific Quaternary units such as Qt1, previous work by others and our site observation indicate that unit Qt1 is largely confined to the edges of the active channel and forms a thin deposit of about 10 feet or less in thickness. It is anticipated that unit Qt1 will be largely removed from the dam footprint during foundation excavation and preparation activities.

2) An additional boring is needed to explore conditions important to cut slope stability in the landslide area near the Antioch Fault (southwest corner of the site).

Design Team Response: Acknowledged. The landslide area was explored with test pits and an additional boring was included in the program. It is anticipated that the landslide mass that extends below the proposed cut slope grade would be excavated and replaced with compacted fill to establish the desired slope grade.

3) ***DSOD will only accept triaxial test results of soil samples collected with Shelby Push or Pitcher Barrel thin-wall techniques for strength characterization of the foundation.***

Design Team Response: Concur. Triaxial tests on foundation materials have been performed on samples obtained as noted above.

4) ***The CU test report should include all index properties of the samples (Atterberg limits, gradation tests, etc.)***

Design Team Response: Concur.

5) ***If SPT blowcounts are to be used quantitatively in liquefaction analysis, then ASTM D 6066 is the appropriate guideline to be followed.***

Design Team Response: Concur.

6) ***The results of the SPT hammer energy calibration should be included in the geotechnical report. Also, the percent recovery of SPT samples and other logging details required by ASTM D6066 should be included in the geotechnical report.***

Design Team Response: Concur.

7) ***The borings should extend to no less than twice the height of the embankment at that location or five feet into the bedrock, whichever comes first.***

Design Team Response: Concur.

8) ***A temporary open standpipe piezometer or an observation well should be installed near the maximum section of the proposed dam to monitor changes in phreatic conditions through multiple seasons to determine the natural ground water level. This should be done in addition to documenting the saturation levels encountered during drilling.***

Design Team Response: Acknowledged. A temporary open standpipe piezometer was installed near the maximum section of the proposed dam, on boring B-08-09. Groundwater levels noted in borings previously drilled at the site and referenced in the Groundwater section on page 8 of the Fugro report of June 2004 ranged from elevation 161 to 165 feet. These elevations roughly correspond with the elevation of the channel of Sand Creek and the elevation of the southern portion of the proposed basin. Regional groundwater information is also available from the "Hydrogeologic Evaluation Report" for the Brentwood Oil and Gas Field Area, Lone Tree Valley, Brentwood and Antioch, by The Source Group dated 18 October 1999. The report indicates that groundwater elevations in the project area range from 155 to 162 feet; these elevations are at or slightly below the elevation of the southern portion of the basin and are consistent with our groundwater measurements.

9) GEI considers the Mount Diablo Blind Thrust Fault to be the primary seismic source at the site. DSOD believes that simultaneous rupture of the Rio Vista and Sherman Island faults is possible, resulting in high ground motions. This is based on the URS Corporation/Jack R. Benjamin & Associates, Inc., Technical Memorandum, Delta Risk Management Study, Phase 1; Draft Seismology, dated February 1, 2007. GEI should re-assess their understanding of the local faulting at the site based on the recent work.

Design Team Response: Acknowledged. The referenced URS Corporation / Jack R. Benjamin & Associates, Inc., technical memorandum has been reviewed and the understanding of the local faulting at the site has been refined accordingly.

10) Since GEI will use the NGA relations in the final design for this facility, and these relations are very dependent on shear wave velocities to a depth of 30 meters, we recommend at least two Vs30 measurements or potentially multiple shallow shear wave velocity measurements extrapolated to Vs30 be included in the investigation program.

Design Team Response: Concur. We performed the recommended shear wave velocity measurements.

Appendix 1.2

Draft Responses to:

Division of Safety of Dams Review Comments For Upper Sand Creek Detention Basin Dam (Proposed)

Geotechnical Study Report Dated June 2004

General

1) Update the dam details to the current values, including dam crest elevation, spillway crest, and embankment geometry.

Design Team Response: The current configuration of the basin, including embankment geometry, dam crest elevation and spillway crest, is presented in this Design Report.

2) Plate 1 in WLA report is missing, but can be found as Plate 4 in Fugro's main Report.

Design Team Response: Comment noted.

3) Missing CPT log for CPT No. 5 and extra CPT log for CPT 8. Possibly mislabeled the CPT nos.

Design Team Response: Comment noted.

4) Missing Table 3 (Borehole summary table) and Table 4 (Laboratory test data) in WLA report.

Design Team Response: Comment noted.

Foundation

5) The project layout is not well described. It is difficult to visualize where the cut areas and fill areas for this reservoir.

Design Team Response: Concur. See response to Comment No. 1 above.

6) The soil boring logs by Fugro should incorporate the five quaternary units that were identified in the geology mapping by WLA. Characterizing the soils with respect to these geologic units is important to better understanding the site.

Design Team Response: We concur that characterizing the soils with respect to the mapped geologic units is important to better understand the site. Since completion of the additional field explorations, geologic profile and cross-sections of the dam foundation have been prepared that delineate the estimated limits of the quaternary units identified in the geology mapping based on the available information and additional field data. Boring logs of the additional explorations delineate the units as observed in the core samples.

7) Additional field explorations are required to better characterize the foundation materials. The additional test borings and laboratory testing should aim at defining the foundation objectives, excavation depths, extent of liquefiable deposits and other poor soils associated with the Qt_1 to help out the excavation plan or foundation objectives. Entire southeast embankment alignment is parallel to the modern drainage and the suitability of this very young Qt_1 soils for embankment foundation is a major concern. Relatively thin (up to 5 ft) modern alluvium (Qal) within the stream channel is considered unsuitable for the foundation.

Design Team Response: We concur. A separate Work Plan for additional explorations was prepared and was submitted to DSOD for review prior to proceeding with the field work. The purpose of the exploration was to characterize the foundation materials, establish their engineering properties for use in design analyses, and define the foundation objective that appropriately determines the depth of required foundation excavation along the embankment footprint and appurtenant structures.

As described in the Design Report, the southeast embankment alignment has been realigned and no longer parallels the modern drainage. It is anticipated that the deposits of young Qt_1 soils will be excavated from the foundation footprint.

8) Clearly indicate what portion of the embankment will be founded on rock and what portion will be founded on alluvium. Additional exploration would help to fill the gaps along the alignment.

Design Team Response: It is anticipated that all of the embankment will be founded on older alluvial terrace deposits (Qt_2). Additional field explorations were performed that helped fill the gaps along the alignment.

9) The impact of the large landslide (on the east) should be carefully evaluated. Foundation excavation and spillway/outlet conduits may be located near or at the toe of the slide.

Design Team Response: We concur with DSOD's recommendation. This ancient bedrock landslide, located above the right abutment of the dam, has been modified by erosion and buttressed by an alluvial terrace. Excavation for the dam foundation may cut into the alluvial terrace at the toe of the slide and was evaluated to make sure that the slide is not reactivated by the project excavations. The toe of the ancient landslide and its relationship with respect to the dam embankment and outlet works excavation have been better defined based on the additional explorations. The outlet works has been realigned to reduce excavation at the toe of the slide. A stability analysis through the ancient landslide mass and dam foundation excavation zone has been performed to verify the stability of the slide mass for the proposed configuration.

10) Impact of the adverse bedding and sheared rock on the south side cut slope should be investigated by test pits to visually estimate shear strength and erosion potential. Test pits should be excavated in each Quaternary unit defined to provide DSOD staff exposure to the site geology.

Design Team Response: We agree. Test pits were located and excavated to develop an updated understanding of the shear strength and erosion potential characteristics of the bedrock formation. A field meeting was coordinated with DSOD geology staff while test pits were open for inspection.

11) Need additional laboratory tests. Use proper sampling method for triaxial shear strength tests of the clayey material. Undisturbed samples should be collected using thin wall Shelby push or Pitcher barrel sampler to quantify the strength of the foundation soil. Need more consolidated undrained (CU) triaxial test (no Bump test) data. For sandy material in addition to SPT values, some density values from undisturbed samples would be useful.

Design Team Response: Samples for foundation shear strength testing were collected using undisturbed sampling techniques. CU triaxial envelopes were developed by testing a set of three specimens, each consolidated to one different confining stress, instead of multi-stage testing on one specimen. Laboratory testing included density values of sandy soils for which undisturbed samples were recovered.

12) The seismic analysis should include Rio Vista-Sherman Island fault, which is the controlling fault for this site.

Design Team Response: Comment noted.

13) The impact of the proposed water quality pond on the east side of the dam should be evaluated.

Design Team Response: Concur. Please note that the proposed water quality ponds have been relocated farther from the dam, to a distance in excess of 1,000 feet from the downstream toe of the dam. It is expected that seepage from the ponds will drain to Sand Creek to the northeast of the dam.

14) Liquefaction analyses were done based on the existing water table. Groundwater table may increase due to the proposed water pond in downstream of the dam. Therefore, the maximum possible water table should be considered in the analysis. Example Boring B-6.

Design Team Response: Please see Comment No. 13 above. Please note that no potentially liquefiable layers have been observed in the boring logs above the water table.

15) In the liquefaction calculation the blow count used for the upper sand layer (from El. 139 to 142') at B-2 does not agree with the value in the boring log.

Design Team Response: Comment noted. The liquefaction calculations were updated based on the data from the relevant existing explorations and additional explorations performed along the dam footprint.

16) Provide back-up data for the strength values in Table 4.1 on page 13.

Design Team Response: Concur. Back-up data has been provided for the strength values used for the final stability analyses.

Embankment

17) Show the downstream slope and the toe of the embankment in the plan view.

Design Team Response: The current configuration of the embankment is presented in this Design Report.

18) Borrow sites need to be identified and characterized. Need additional laboratory test to quantify the strength of the embankment fill based on the compaction criteria.

Design Team Response: The materials from basin excavation are proposed for use in embankment construction. The borrow site (basin excavation) has been evaluated and delineated in drawings. The information from the 2004 Geotechnical Study has been incorporated in the borrow area evaluation, and additional test pits have been excavated to further characterize the engineering properties of the borrow material. Soil samples were taken, compacted to the specified embankment density (established in concurrence with DSOD), and tested for strength and deformation properties. These properties have been used in the final analysis of the dam.

19) The slope stability analysis is considered very preliminary as the embankment geometry, required foundation excavations, and detail geotechnical properties of soils are not sufficient.

Design Team Response: The slope stability analysis has been refined based on the final embankment geometry and considers the results of the available studies and additional foundation investigations, and the selected foundation objective. The geologic profile of foundation materials has been refined, and strength properties have been selected. Cross-sections have been defined at critical locations and shown on the drawings. The cross-sections extend sufficiently into the foundation materials to ensure that all critical deep failure surfaces have been analyzed. For the critical sections, the analysis covers all commonly accepted stability cases: end-of-construction, steady state with full reservoir, rapid drawdown, and seismic loading.

20) Slope stability analysis – show the analyzed cross-section locations in a plan.

Design Team Response: Concur. Please see response to Comment No. 19.

21) Slope stability analysis – some most critical slip surfaces go down to the bottom boundary. So need to include deeper layers for these analyses.

Design Team Response: Concur. Please see response to Comment No. 19.

22) Stability analysis should cover all the critical conditions/stages such as immediately after construction, rapid drawdown, etc for the most critical section(s).

Design Team Response: Concur. Please see response to Comment No. 19.

23) On page 35 (in Section 5.4.1 of WLA report) WLA recommended to do stability analysis through the ancient landslide mass and dam foundation excavation zone to verify the stability of the slide mass. It was not included in the Fugro report.

Design Team Response: Concur. As discussed under Comment No. 9, a stability analysis through the ancient landslide mass and dam foundation excavation zone has been performed to verify the stability of the slide mass for the proposed configuration.

24) Potential for toe of the ancient landslide (Q1sp) under the eastern side of the main dam. In WLA cross-section B-B' should show the proposed embankment and outlet pipe locations on this cross-section.

Design Team Response: Concur. The toe of the ancient landslide and its relationship with respect to the dam embankment and outlet works excavation has been better defined by additional explorations. The outlet works has been realigned to reduce excavation at the toe of the slide and for other reasons.

25) Section 5.1.1 indicates that the areas beneath the embankments and keyways should be excavated to a depth of 5 ft and replaced with the proper compacted fill. This is a general statement. Consultant should provide detail foundation objectives along the dam, including in situ density measurements (sand cone or ring test), percent of relative compaction or relative density of the foundation material, depths of excavation.

Design Team Response: Concur that the constant excavation depth of 5 feet for the entire dam footprint is too general. As discussed in this Design Report, the design team has developed detailed foundation objectives along the dam based on the results of the relevant explorations. The foundation objectives are consistent with the proposed design for the embankment (dam height, slopes, etc.).

26) The total freeboard depends on the deformation of the embankment. DSOD requires at least a 4 ft total freeboard and a 1.5 ft residual freeboard for the design storm.

Design Team Response: Concur. Small post-construction deformations are anticipated because of the firm nature of the dam foundation and low height and flat slopes of the dam. As indicated in Section 2 of the Design Report, the minimum normal freeboard (vertical distance from spillway crest to dam crest) will be 4.5 feet. The residual freeboard during the 1,000-year 12-hour storm, will be 2.1 feet. In addition, as established in coordination with DSOD, the spillway will pass the 1,000,000-year 72-hour storm.

27) Embankment fill material can have slightly lower PI (between 10 and 25) than what was indicated on page 19.

Design Team Response: Concur. The requirement for PI is unnecessarily restrictive. It will be important to design the embankment cross-section so that non-organic materials from basin excavation can be placed cost effectively in the embankment.

28) Section 5.1.3 in the main report (on page 19), the last paragraph first sentence should say "—to obtain moisture content between optimum minus 1 percent and optimum plus 4 percent." DSOD recommend the moisture content of the compacted embankment fill should be between optimum and plus 3 percent.

Design Team Response: For this project we would prefer to base the earthwork specifications on ASTM D 698 because its use leads to higher moisture contents and tends to result in a more flexible embankment. Typically, for clayey soils ASTM D

698 gives an optimum moisture content that is 2% to 3% (of the maximum dry weight of the soil) higher than the optimum moisture content from ASTM D 1557. Thus, the requested moisture content may be difficult to attain if using ASTM D 698. In principle we would prefer to specify a moisture content between optimum minus 2 percent and plus 2 percent) based on ASTM D 698.

29) Report says embankment should be compacted to at least 90% relative compaction as determined by ASTM D1557. DSOD requires min. 93% relative compaction using ASTM D1557.

Design Team Response: As noted in response to Comment No. 28 above, for this project we would prefer to base the earthwork density specification on ASTM D 698. We would suggest a minimum of 100% relative compaction as determined by ASTM D 698.

Outlet

30) Section 5.2.5 indicates that the concrete encasement is required where the pipeline penetrate the basin, especially adjacent to the outlet structures. DSOD recommends having concrete encasement underneath the embankment fill.

Design Team Response: Concur. The outlet pipe under the dam will be fully encased in concrete to minimize the potential for seepage and erosion along the pipe.

31) Trench backfill should be placed in lifts less than 8-inches instead of 12 inches (as indicated in Section 5.2.6) in un-compacted thickness and should be compacted to min. 93% relative compaction using ASTM D1557.

Design Team Response: Concur with the requested lift thickness. For this project we would prefer to base the earthwork specifications on ASTM D 698.

Spillways

32) In Section 5.4.2 when designing for lateral load on below grade walls the effect of groundwater level should be considered by assuming the reservoir in full capacity.

Design Team Response: Concur.

33) Need to characterize the borrow material and use the appropriate strength values for the soil with required compaction in the lateral load calculations.

Design Team Response: Concur.

34) In Section 5.4.3, lateral resistance – The frictional stress should be limited to the undrained strength of the base material when using a frictional coefficient of 0.3.

Design Team Response: Comment noted.

35) Section 5.5 indicates that the concrete encasement is required for pipe from a point at which it enters the embankment to where it crosses a vertical intercept with the top of the slope. DSOD recommends having concrete encasement for the full length of the embankment fill.

Design Team Response: Concur.

Appendix 2 – Geotechnical Analyses

Appendix 2.1

Landslide Stability Evaluation

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TECHNICAL MEMO

To:	Alberto Pujol, GE; Dan Wanket, PE
From:	Phillip Gregory, GE; Steve Watry, PG, GE
Date:	15 June 2010 DRAFT
Re:	CCFCWD Upper Sand Creek Basin Stability of South Rim Slopes and Ancient Landslide on Right Abutment

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Existing Landslides Near Southwest Corner of Basin

General Description

Relatively shallow landslides are present on the north-facing natural slope above the creek bank near the southwest corner of the basin. These landslides appear to be complex slides with both rotational and translational motion. Based on our field mapping and the excavation of test pits and a boring at the landslide locations, the landslides involve soil/colluvium (Qc) and weathered bedrock (see attached Figure 1 which includes an excerpt of the geologic map and a geologic cross-sections). The landslide has developed on the face of a gently ascending natural slope that is blanketed by a relatively thick veneer of soil / colluvium. The toe of the slide daylights near the top of a steep bank above Sand Creek.

Previous Work

WLA in their March 16, 2004 report (page 35) noted that the presence of the landslides could impact the stability of the basin margin and recommended that provisions for landslide stabilization (overexcavation and/or buttressing) be considered. WLA estimated that the depth of the larger landslide could be as great as 25 feet.

Current Investigation

Two test pits and one boring were excavated in the area of the landslides. These excavations along with field mapping were utilized to define the limits of the primary landslide. The proposed detention basin grading and landslide mitigation are shown on on Figure 2. The landslide involves soil/Qc and weathered bedrock. The toe of the landslide is in the bedrock slope that forms the steep southerly bank above Sand Creek and does not extend to the toe of the slope. Based on the boring the landslide debris consists of somewhat coherent blocks of sedimentary bedrock. It is estimated that the maximum depth of the landslide debris is on the order of 15 feet. A bulk sample of landslide debris near an apparent slip plane was obtained from Test Pit 25 to ascertain soil characteristics (LL, PL and hydrometer) to estimate residual shear strength parameters.

Proposed Upper Sand Creek Detention Basin Grading

Proposed detention basin grading in this area consists of the construction of a fill slope, the top of which will be near the toe of the existing landslides. The proposed fill will buttress the existing slope below the landslide. A shallow cut is proposed above the top of the fill slope. The cut will result in trimming of the face of the existing landslide and soil/Qc above the landslide.

Analysis

A back analysis of the primary landslide was performed to estimate residual shear on the apparent slide plane and to compare the results with published strength parameters for the Markley formation obtained from an October 12, 1991 field guide (Rogers, 1991) and values estimated from laboratory test correlations using the methods of Stark et al (2005). The landslide was "pushed back" to a configuration to eliminate the head scarp. Analyses were performed for both a rotational failure and a planar failure. For an estimated residual shear strength of 17 degrees, the factors of safety calculated were just above 1 indicating that the failure surface and shear strength are reasonable. The results of the back analysis are included on Figure 3.

Impact of Future Movement of Landslide

Without mitigation, future movement of the landslide would result in some debris falling into the basin and possibly some damage to the proposed rim maintenance road and/or the top of the fill slope. Headward enlargement of the landslide would be likely to occur over time. Neither the future movement of the landslide nor the headward enlargement of the landslide would pose a grave hazard to the basin, but may require maintenance to remove debris, repair damage to the graded slope and rectify concentrated drainage conditions around the perimeter of the landslide area.

Mitigation

Mitigation of the landslides would most reasonably be accomplished by their removal and replacement with drained, compacted fill. The extent of the removal beyond the perimeter of the landslides would be determined in the field, but for the purposes of planning it is recommended that removal extend a minimum of 5 horizontal feet beyond the head of the landslides. The extent of the remedial remove and replace grading for the landslide is shown on Figure 2. Results of a stability analyses for the typical "remove-and-replace" cross-section are attached as Figures 4 and 5.

Proposed Cut slope Near the South-central Portion of the Basin

General Description

A cut slope on the order of 30 feet high is proposed at a 3:1 inclination along the central portion of the south rim. This cut will expose shallow north-dipping interbedded shale and sandstone.

Previous Work

WLA in their March 16, 2004 report (page 35) noted that 2:1 cut slopes were proposed up to about 35 feet in height. Rock Core C-2, excavated for the Fugro West/WLA investigation of 2004, is located within the footprint of the currently proposed 3:1 cut slope near the top of the proposed cut slope. The WLA rock core encountered about 8 feet of soil over weathered shale and claystone bedrock. The finished grade will be about 25 feet below the top of the rock core so the soil will be entirely removed by the proposed cut. Bedding attitudes measured in the rock core C-2 by an optical televiewer ranged from 12 to 28 degrees. The majority of the beds (11 of 20) have a dip of 15 to 20 degrees while only three are flatter than 15 degrees and six are steeper than 20 degrees. WLA completed a Hoek-Brown rock mass classification which requires an unconfined compression strength, geologic strength index (GSI), and a material index (m_i). Unconfined compression tests (Uc) were performed on some intact bedrock samples, with the other parameters estimated by field tests and observations. WLA concluded that the friction angle of the bedrock samples tested ranges from about 27 to 35 degrees (page 13 of the WLA 2004 report) and so the rock slope would be stable in 2:1 cuts. WLA noted that planar bedrock failures in cuts steeper than 2:1 could occur in the event of an earthquake or heavy rainfall where perched water conditions develop above impermeable beds. They also opined that some of the deep-seated slides in the vicinity appear to involve sliding along bedding planes.

Current Investigation

Several test pits were excavated in the area of the proposed cut slope, and outcrops along the Sand Creek bank were mapped (see Figure 6 which includes an excerpt of the geologic map). A bulk sample of a weathered shale slip plane from Test Pit 25 was taken for an estimation of bedding plane shear strength parameters. There are some irregularities to the bedding as indicated by measurements in outcrops and to a greater degree in the variation in the bedding attitudes noted in Core C-2 drilled for the WLA investigation. The bedding variation noted in the rock core as measured by the televiewer is considered to be at least partially due to the slightly undulatory nature of much of the bedding. Based on larger exposures in outcrops the bedding appears to dip at about 15 to 16 degrees to the north/northeast in the area of the proposed 3:1 cut slope. The relatively steep banks south of Sand Creek in this area do not show even shallow bedrock failures. Portions of the creek banks in nearly a true down-dip direction are on the order of 1.25:1 to 1.5:1. The fact that the ground above and to the south of the top of the creek bank in this area slopes away from the Sand Creek bank may improve the natural slope stability by directing surface water away from the slope. It is also plausible that some of the bedrock is composed of sandstone that has a greater shear strength than the shale beds.

Proposed Upper Sand Creek Detention Basin Grading

Proposed grading in this area consists of the excavation of a 30-foot-high 3:1 cut slope. The existing and proposed grades are shown on Section H. A level road about 20 feet wide is planned near the top of the cut slope. This cut slope will expose north-dipping slightly undulatory bedding that generally dips about 15 to 16 degrees to the north.

Analysis

An analysis of the proposed cut slope was made using residual shear strength parameters estimated from a correlation from laboratory strength testing based on the Stark et al (2005) method and the published range of strengths for Markley formation bedrock (Rogers, 1991). The values utilized in the analyses are a residual friction angle of 17 degrees and cohesion of 200 psf for freshly cut bedrock. This resulted in a factor of safety of about 1.5. Utilizing a slightly lower bedrock strength of 17 degrees and no cohesion for weathered bedrock resulted in a factor of safety of about 1.0. Because a residual value is used for the friction angle, it is believed that the former analysis is a conservative representation of the condition after initial construction, while the latter analysis is a conservative representation of the exposed bedrock materials after several years of weathering.

Impact of Cut Slope Failure

Some sedimentary bedrock in the vicinity has developed planar, translation failures, along discontinuities, typically low-strength shale bedding planes. It is our opinion that in the proposed configuration it is unlikely that a large planar bedrock failure would occur on the proposed 3:1 cut slope, even in an extenuating circumstance such as a large seismic event. It is our opinion that prolonged exposure of the shale bedrock over time will likely result in some small scale slope failures as suggested by the low factor of safety calculated for the zero cohesion and residual friction angle condition. If a large planar (bedding plane) failure does occur it would not have a drastic impact on the utilization of the basin. Some debris would be shed into the basin and the rim road may require repair. It is our opinion that the proposed 3:1 cut slope is reasonable for the detention basin.

Mitigation

If desired, the performance of the proposed 3:1 cut slope can be improved by either laying the cut slope back to 4:1 (14 degrees) or construction of a stability fill on the face of the cut slope. Laying back the slope would result in a slope that is flatter than almost all of the bedding measured and so no unsupported condition would exist. A stability fill slope could be constructed over the face of the cut slope. This would serve to reduce the weathering depth of the underlying bedrock and reduce the potential for shallow failures.

Presence of Ancient Landslide Near the Right Abutment

General Description

An ancient landslide is present near the right (southeast) dam abutment. This landslide is subdued and is covered by a thick profile of soil/Qc. The lower portion of the landslide debris consists of fractured, weathered coherent blocks of sedimentary bedrock.

Previous Work

WLA in their March 16, 2004 report concluded that the ancient landslide had its toe removed by creek incision and was later buttressed by the deposition of an older terrace deposit, Qt2 (pages 9 and 34). WLA considered the ancient slide to be inactive. They described the landslide plane where exposed in one core as "healed"(page 34). WLA considered the ancient landslide to be rotational based on observation of reported south-dipping bedding in one trench (T-2). It is believed that this interpretation of the bedding dip is erroneous because the graphic trench log shows the bedding to actually be north dipping. A televiewer observation of the lower portion of Rock Core C-6 below the ancient slide plane, at about 47 feet, revealed some shallow bedding dipping to the north. The bedding recorded by the televiewer suggested dips of 18 degrees at about 51 and 53 feet flattening to 7 degrees at the last point measured, 55.5 feet. The rock core logs by WLA for C-6 indicate laminations directly below the slide plane of less than 5 degrees. It is our opinion that the bedding measurements in this area either are indicative of a small subtle fold resulting in a reversal of bedding or some of the televiewer interpreted bedding may have been along a contrast made between a silt and an irregular sand lens.

Current Investigation

A test trench and three borings were excavated in the area of the ancient landslide. The ancient landslide is shown on the excerpts from the geologic map and Sections A and E (Figures 7 and 8). One of the borings, Boring B-6-09, was logged with an optical and acoustic televiewer. Bedrock outcrops near the perimeter of the ancient landslide were mapped. Based on the results of the mapping, the boring logs, and geophysical data, it is our opinion that the ancient landslide mass is a translational failure along a bedding plane. We believe that a high steep bank was incised through more competent bedrock materials until a weak discontinuity (claystone bed) was encountered. Landslide debris consisting of weathered claystone was observed extending over a few feet of granular older terrace deposits in Boring B-4-09. The televiewer analyses of Boring B-6-09 revealed north-dipping bedding near and just below the base of the ancient landslide. It is our opinion that the ancient landslide failed to the north, rather than the northwest direction of failure hypothesized by WLA. Structural contouring of the bedrock surface also supports a more northerly dip direction for the ancient landslide basal surface.

Proposed Upper Sand Creek Detention Basin Grading

Proposed grading in this area consists of excavation for the right dam abutment and minor filling for construction of the rim road extending to the southwest from the dam abutment.

Analysis

A back analysis of the ancient landslide was conducted by reconstructing the pre-landslide geometry, i.e., by "pushing back" the landslide so that its surface was near the same elevation as the ridge to the south. The toe of the landslide was pushed back from its current position and a drainage course incised bluff assumed at the toe of the ancient landslide. The results of this back-analysis suggest that a friction angle of just under 10 degrees with no cohesion would be required to trigger the landslide (Figure 9). A large seismic event would have been required to mobilize the landslide utilizing the assumed values for the Markley formation of friction angle of 12 degrees and cohesion of 120 pounds per square foot (Figure 10). The back analysis is hampered by the fact that the pre-slide topography has been grossly modified over the years and so the accuracy of the pre-slide topography can not be considered quantitatively precise. However, from a qualitative sense, the results appear to reasonably explain the apparent geomorphology of the landslide.

Based on the strength values obtained from the back analysis, an analysis of the current stability of the existing landslide was completed. The current geometry of the landslide mass, buttressed by the older terrace deposit, was analyzed. This analysis (Figure 11) suggests the possibility for reactivation is very remote, as the calculated factor of safety is greater than 3 even without the benefit of the additional buttressing effect of the proposed dam.

An analysis was performed to ascertain the effect of foundation excavation for the embankment construction. For the purposes of the analyses it was assumed that 10 feet of overburden would be removed. The analysis (Figure 12) suggests that the ancient landslide mass will maintain a static factor of safety greater than 2 and will not experience movement during the temporary removal of earth materials to prepare the foundation for the embankment. Therefore, based on our review of the WLA report, our site observations and exploration, and the attached analyses, we consider the potential for reactivation of the ancient landslide is remote.

Mitigation

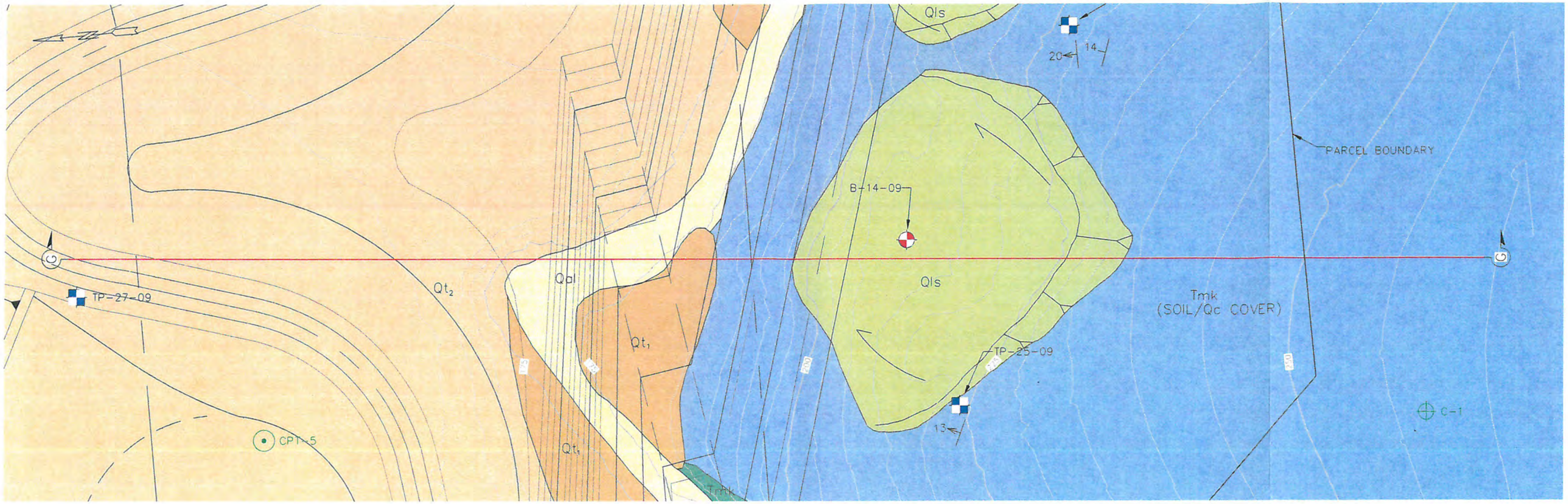
Reactivation of the landslide is considered remote. Some flatter temporary slopes may be required to excavate the dam abutment if shallow back cut failures occur.

References Cited

Rogers, J. David, (1991), Landslides in the Cordelia-Vallejo Area, Napa and Solano Counties, California, in Northern California Geological Society and Association Of Engineering Geologists, Filed Trip Guide to the Geology of Western Solano County, October 12, 1991.

Stark, Timothy D., Choi, H, and McCone, S., 2005, *Drained Shear Strength Parameters for Analysis of Landslides*, Journal of Geotechnical and Geoenvironmental Engineering, vol. 131, 575, ASCE Reston Virginia.

List of Figures



PLAN AT SECTION G

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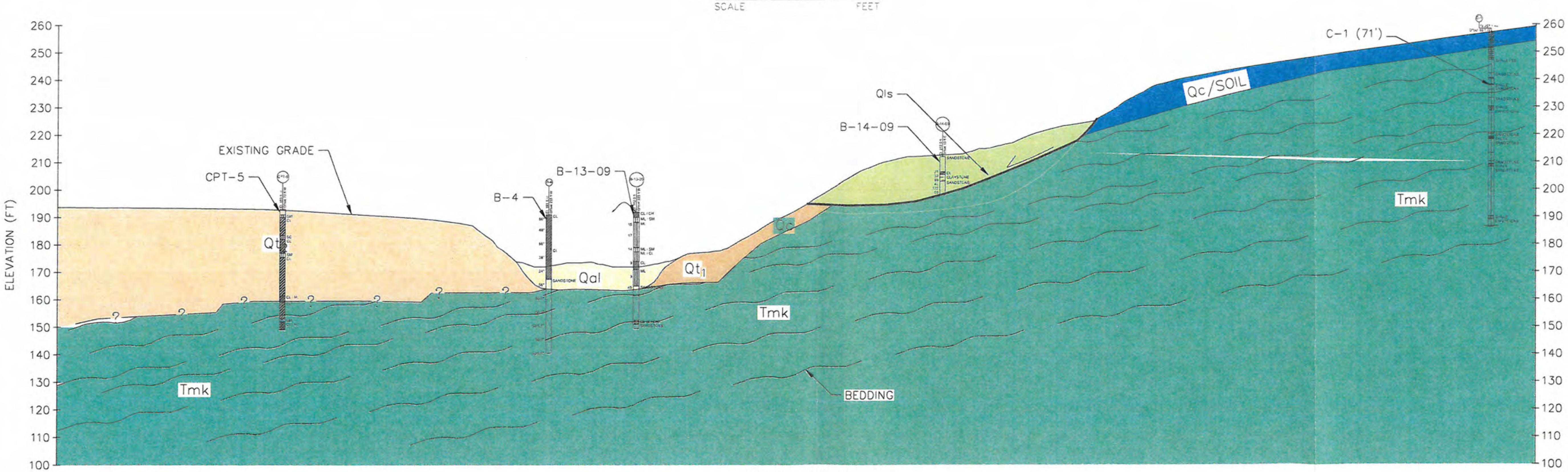
SCALE FEET



SECTION G

20 0 20 40
SCALE FEET

UPPER SAND CREEK BASIN
CCC FLOOD CONTROL & WATER CONSERVATION DISTRICT
MARTINEZ, CALIFORNIA

GEOLOGIC CROSS SECTION G

FIGURE 1

JUNE 2010

081070

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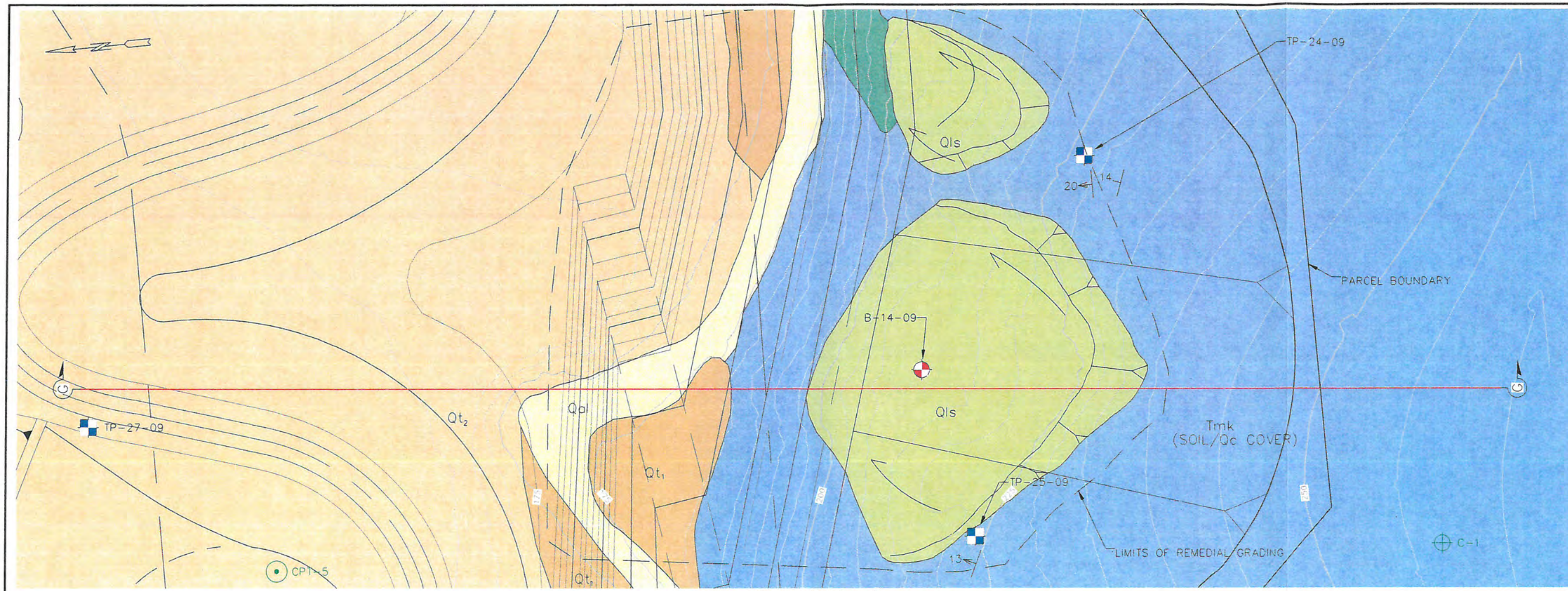

PLAN AT SECTION G

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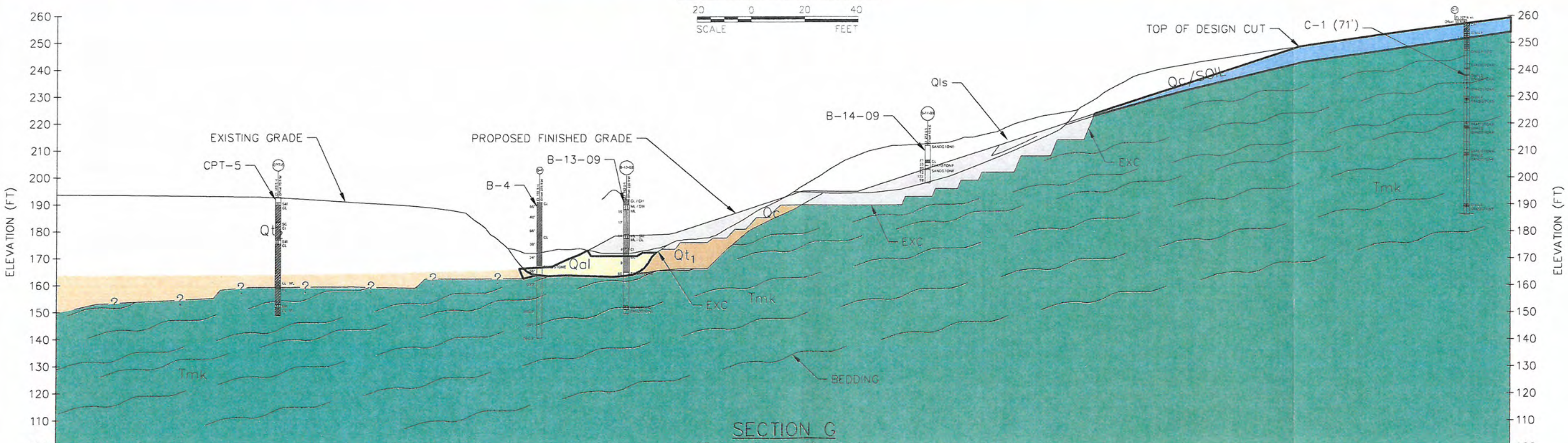
SCALE FEET



SECTION G

20 0 20 40
SCALE FEET

UPPER SAND CREEK BASIN
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PROPOSED GRADES & LANDSLIDE MITIGATION

081070 | JUNE 2010 | FIGURE 2

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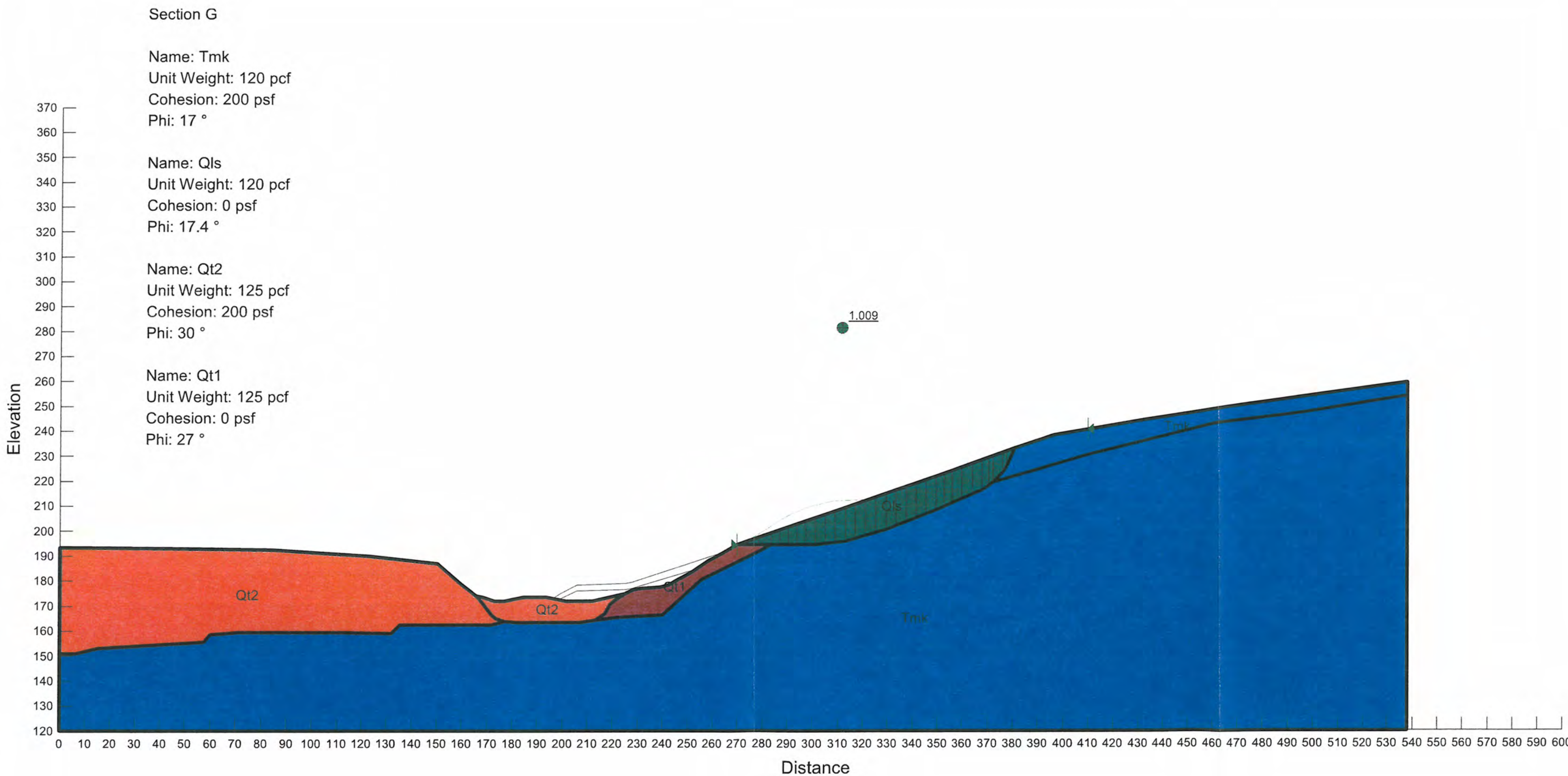

NO SCALE

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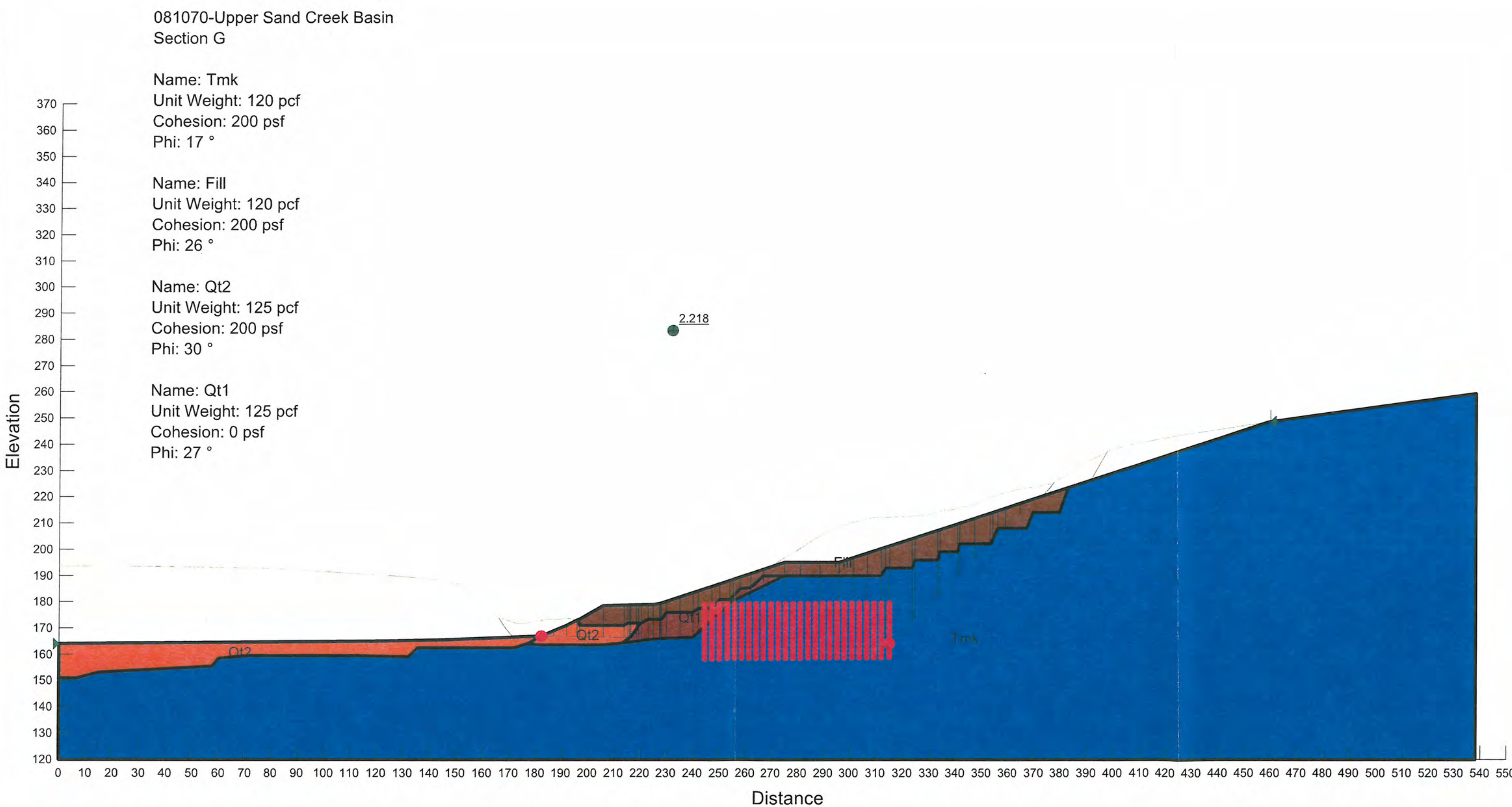
UPPER SAND CREEK BASIN

CCC FLOOD CONTROL & WATER CONSERVATION DISTRICT

MARTINEZ, CALIFORNIA

STABILITY ANALYSIS - BACK ANALYSIS

JOB NO. 081070	JUNE 2010	FIGURE 3



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STABILITY ANALYSIS - LOWER REPAIR

JOB NO. 081070	JUNE 2010	FIGURE 4

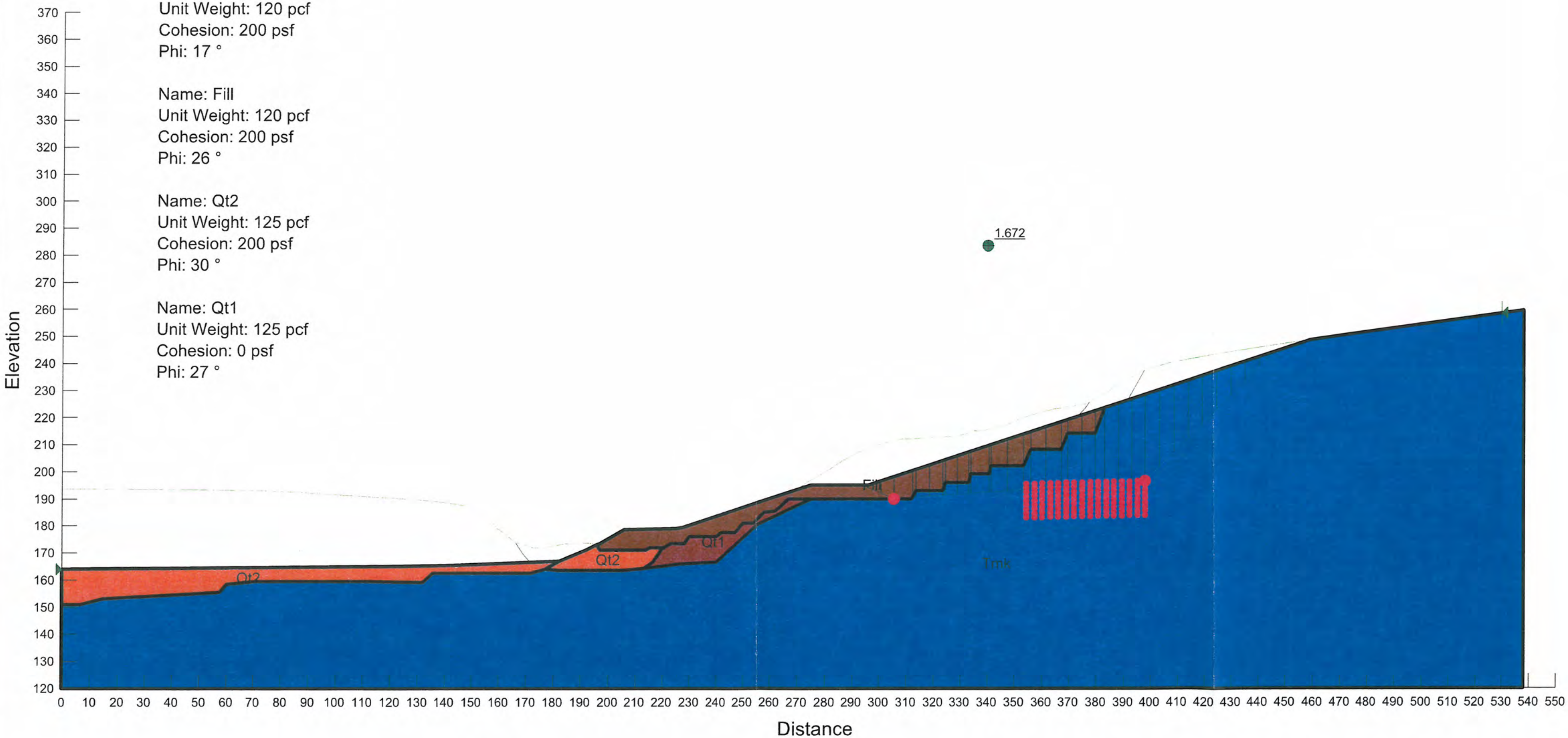
081070-Upper Sand Creek Basin
Section G

Name: Tmk
Unit Weight: 120 pcf
Cohesion: 200 psf
Phi: 17 °

Name: Fill
Unit Weight: 120 pcf
Cohesion: 200 psf
Phi: 26 °

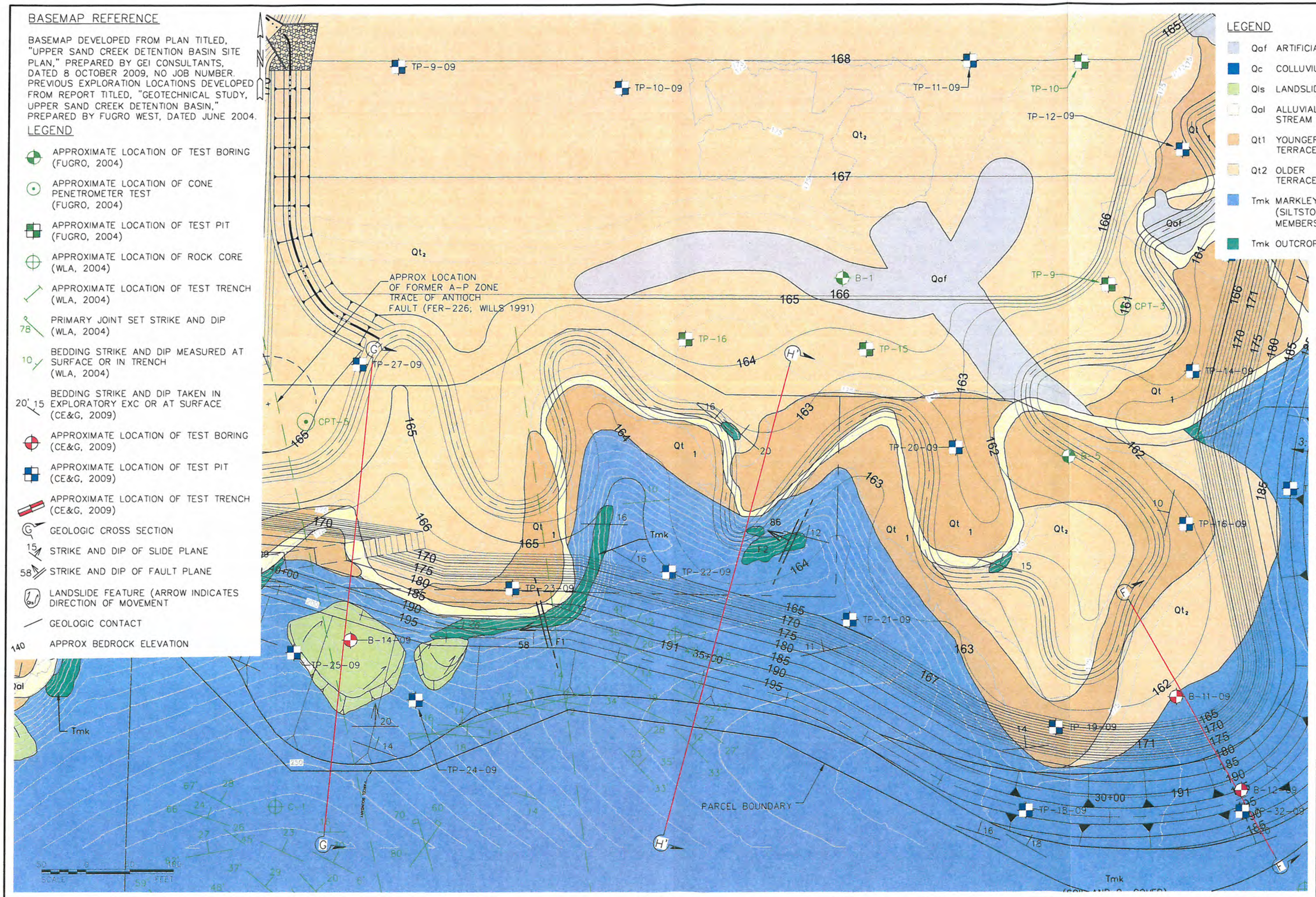
Name: Qt2
Unit Weight: 125 pcf
Cohesion: 200 psf
Phi: 30 °

Name: Qt1
Unit Weight: 125 pcf
Cohesion: 0 psf
Phi: 27 °



NO SCALE

CE&G CAL ENGINEERING & GEOLOGY	119 Filbert St. Oakland, CA 94607 Phone: (510) 451-2350	UPPER SAND CREEK BASIN CCC FLOOD CONTROL & WATER CONSERVATION DISTRICT MARTINEZ, CALIFORNIA **STABILITY ANALYSIS - UPPER REPAIR**
JOB NO. 081070	JUNE 2010	FIGURE 5



UPPER SAND CREEK BASIN
CCC FLOOD CONTROL & WATER CONSERVATION DISTRICT
MARTINEZ, CALIFORNIA

SUBSURFACE EXPLORATION & GEOLOGIC MAP

081070 | 26 APRIL 2010 | FIGURE 6

1870 Olympic Blvd.
Suite 100
Walnut Creek, CA 94596
Phone: (925) 935-9771

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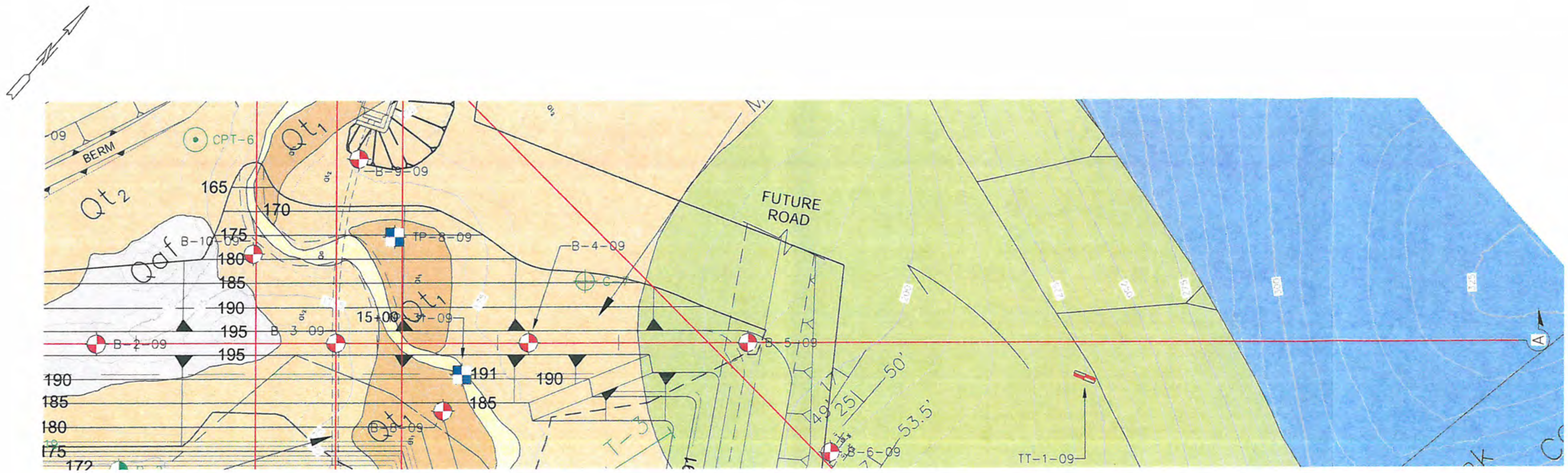
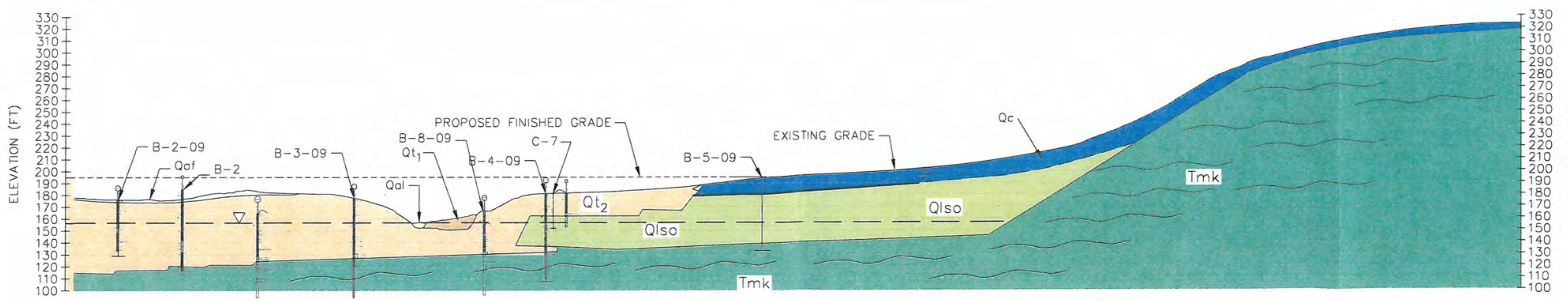


PLAN AT SECTION A (EAST HALF)

50 0 50 100
SCALE FEET



SECTION A (EAST HALF)

50 0 50 100
SCALE FEET

UPPER SAND CREEK BASIN
CCC FLOOD CONTROL & WATER CONSERVATION DISTRICT
MARTINEZ, CALIFORNIA

GEOLOGIC CROSS SECTION A

FIGURE 7

JUNE 2010

081070

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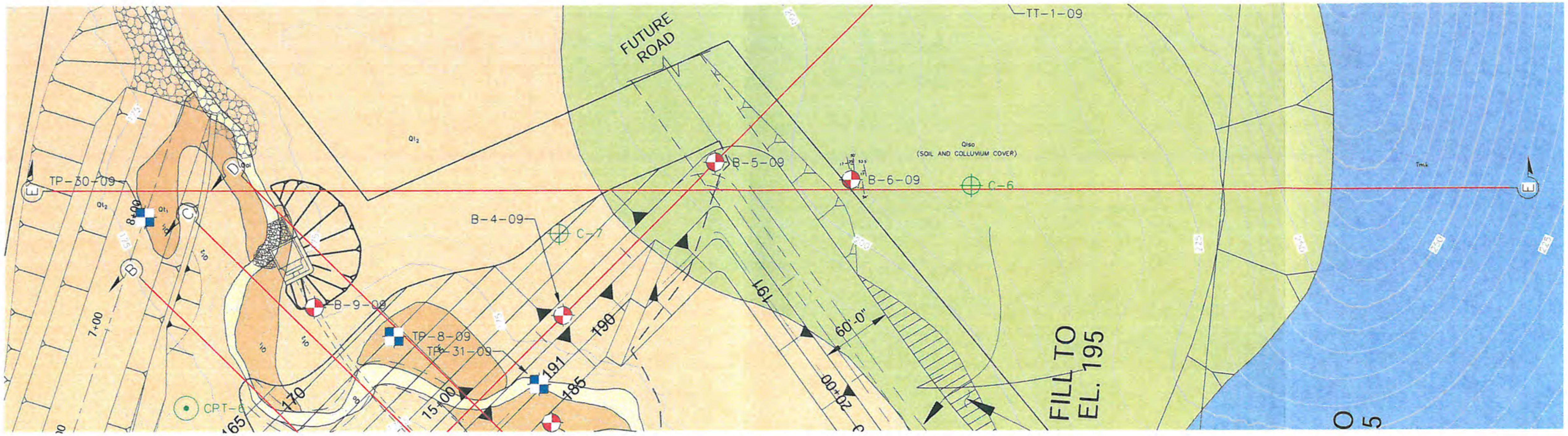

PLAN AT SECTION E

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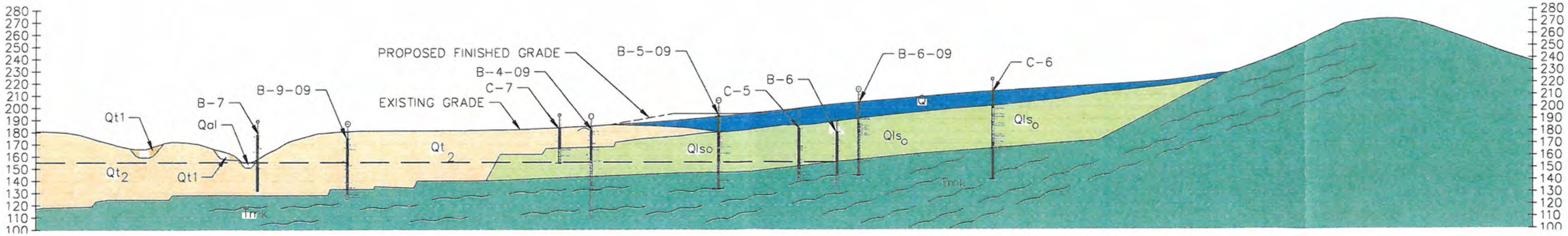
SCALE FEET



SECTION E

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SCALE FEET

UPPER SAND CREEK BASIN
CCC FLOOD CONTROL & WATER CONSERVATION DISTRICT
MARTINEZ, CALIFORNIA

GEOLOGIC CROSS SECTION E

081070 | JUNE 2010 | FIGURE 8

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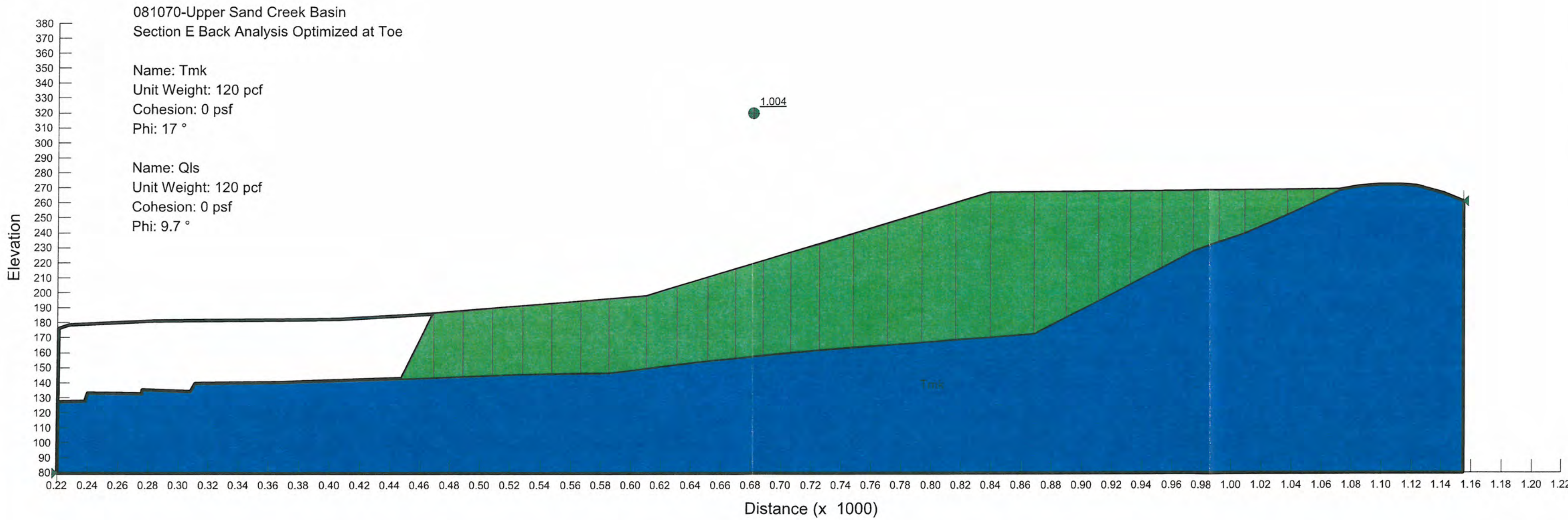
Phone: (925) 935-9771



NO SCALE

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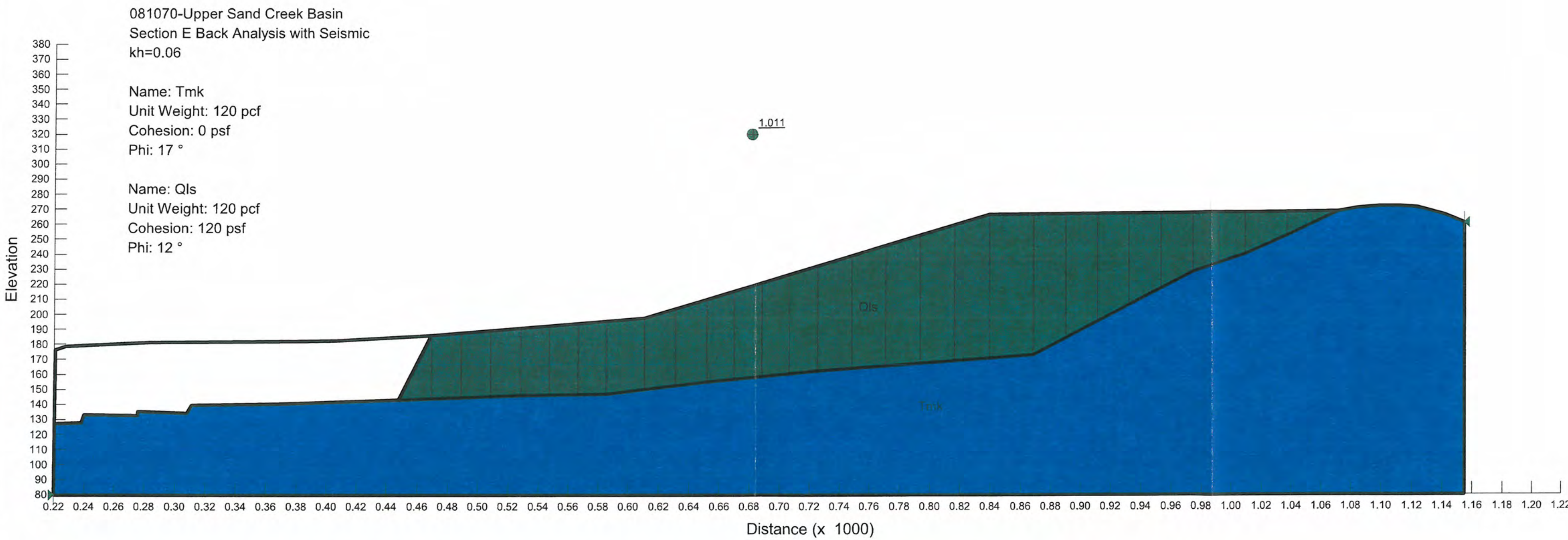
119 Filbert St.
Oakland, CA 94607
Phone: (510) 451-2350

UPPER SAND CREEK BASIN
CCC FLOOD CONTROL & WATER CONSERVATION DISTRICT
MARTINEZ, CALIFORNIA

ANCIENT LANDSLIDE BACK ANALYSIS

JOB NO. 081070	JUNE 2010	FIGURE 9



NO SCALE

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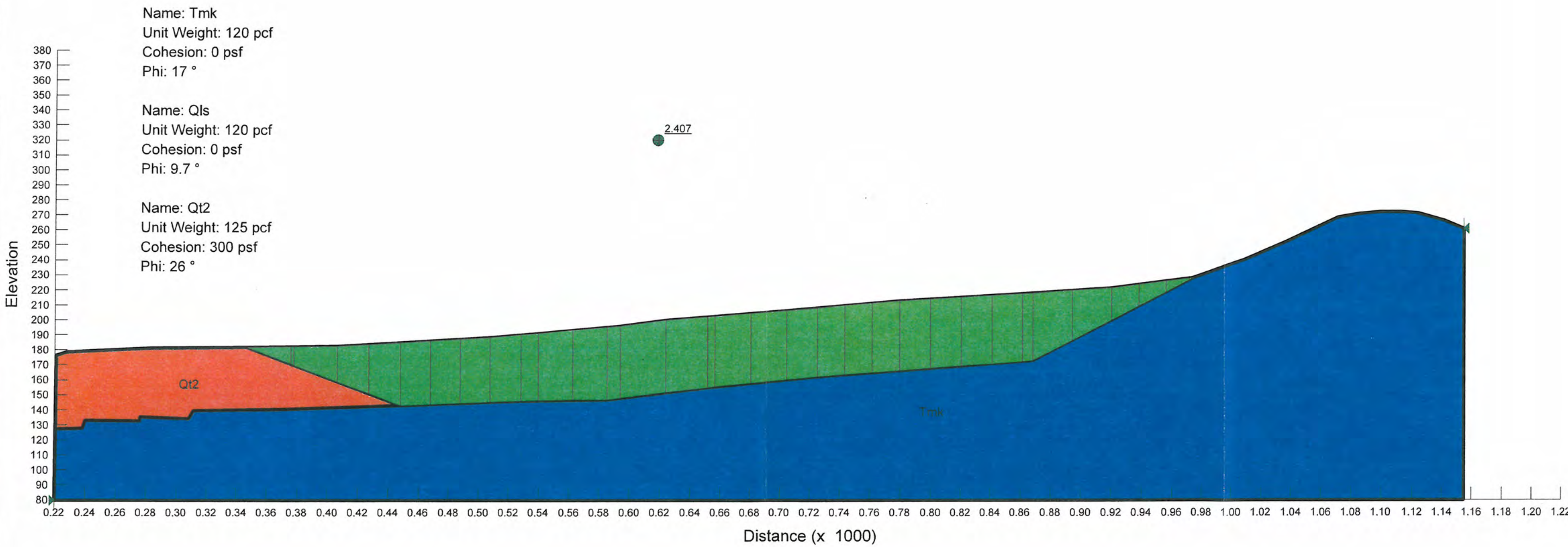
Oakland, CA 94607
Phone: (510) 451-2350

UPPER SAND CREEK BASIN
CCC FLOOD CONTROL & WATER CONSERVATION DISTRICT
MARTINEZ, CALIFORNIA

ANCIENT LANDSLIDE SEISMIC EVENT

JOB NO. 081070	JUNE 2010	FIGURE 10



NO SCALE

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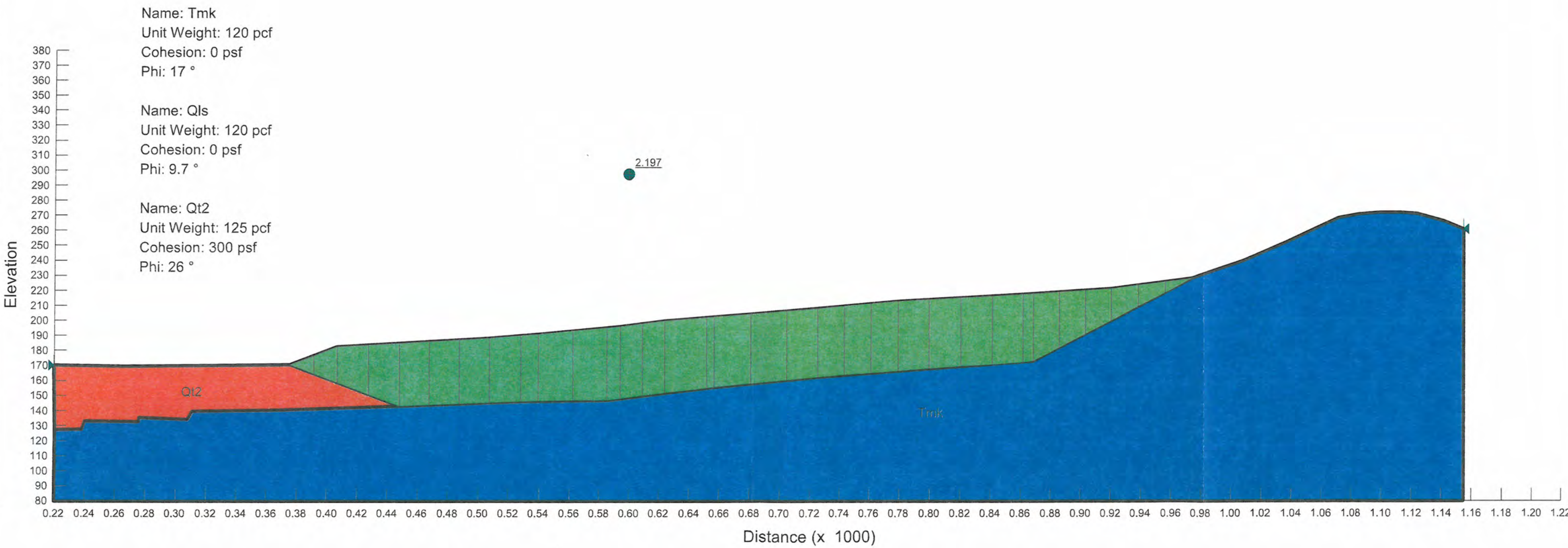
Phone: (510) 451-2350

UPPER SAND CREEK BASIN
CCC FLOOD CONTROL & WATER CONSERVATION DISTRICT
MARTINEZ, CALIFORNIA

ANCIENT LANDSLIDE EXISTING STABILITY

JOB NO. 081070	JUNE 2010	FIGURE 11



NO SCALE

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Oakland, CA 94607
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UPPER SAND CREEK BASIN
CCC FLOOD CONTROL & WATER CONSERVATION DISTRICT
MARTINEZ, CALIFORNIA

ANCIENT LANDSLIDE STABILITY DURING CONSTRUCTION

JOB NO. 081070	JUNE 2010	FIGURE 12

Appendix 2.2

Ground Motion Estimates

INPUT DATA PREPARATION - NGA Equations (2008) - Updated 2009

Lsansone 04/19/10 **DO NOT CHANGE LOCATIONS OF ANY CELLS** To be used w/Linda Atik S/S (NCEER, 2009)

$M (M_w)$	6.90	= Moment magnitude	FAULT: Concord- Green Valley	Project: Upper Sand Creek
			Type: SS	Owner: CCWD
			Rupture Width: 20.0 km	

R	21.00	= Distance to plane of rupture (km)	*[distance to plane of rupture]*
R_{JB}	21.00	= Distance to surf. projection of coseismic rupture (km)	*[hor. dist. to vert. projection of rupture]*
$Z_{2.5}$	2.5	= Depth of 2.5 km/s shear-wave velocity horizon (km)	*[estimated depth to basement rock]*
$Z_{2.5km}$	DEFAULT	= Depth to 2.5 km/sec Vs horizon (m) [CB Input!A1]. Enter "DEFAULT" for default values or enter known value	
R_{RUP}	21.0	= Closest distance to coseismic rupture (km) [Rx]	*[distance to zone of seismogenic rupture]*
F_{RV}	0	= 0 for strike slip, normal & normal-oblique; 1 for reverse, reverse-oblique & thrust	[Reverse-faulting factor]
F_{NM}	0	= 0 for strike slip, reverse, reverse-oblique & thrust; 1 for normal & normal-oblique	[Normal-faulting factor]
F_{HW}	0	= Hanging-Wall Factor = 1 for site on down-dip, 0 otherwise (foot wall or vertical fault), see AS & CY	HW
Z_{TOR}	0	= Depth to top of coseismic rupture (km)	*[observed/assumed surface rupture, $Z_{TOR} = 0$]*
$Z_{1.0km}$	DEFAULT	= Depth to 1.0 km/sec Vs horizon (m) [AS & CY]. Enter "DEFAULT" for default values or enter known value	
δ	90	= Average dip of rupture plane (degrees) c **12**	*[adjusted from 10 in Idriss' original version]*
F_{AS}	0	= Aftershock Factor. 0 = Main rupture. 1 = Aftershock	
V_{S30}	300	= Average shear-wave velocity in top 30m of site profile (m/s)	
$F_{Measured}$	0	V_s measured? (yes or no) **no** (V_s upper 30m = **984** ft/s)	$F_{measured}$ =1 if measured, $F_{measured}$ = 0 otherwise
U	0	= Unspecified Mechanism Factor (Boore-Atkinson): 1 is unspecified, 0 otherwise.	
Hw_{taper}	0	= Hanging Wall Taper. 0 = Abrahamson & Silva (2008) or 1 = Abrahamson alternate formula (revised)	

GRAPH TITLE Line 1:	**Concord -Green Valley Fault**	FAULT NAME
GRAPH TITLE Line 2:	**Upper Sand Creek Detention Basin**	FACILITY NAME

ORIGINAL I+S1NPUT BY NCEER (2009 - LINDA ATIK)

This Excel file calculates the weighted average of the natural logarithm of the spectral values from the NGA models

NGA Model:	AS08	BA08	CB08	CY08	I08
Weight:	0	0.333	0.333	0.333	0
					100%

N	1	Fraction (or number) of standard deviations to be considered in the calculations

AS08: Abrahamson & Silva 2008 NGA Model
BA08: Boore & Atkinson 2008 NGA Model
CB08: Campbell & Bozorgnia 2008 NGA Model
CY08: Chiou & Youngs 2008 NGA Model
I08: Idriss 2008 NGA Model

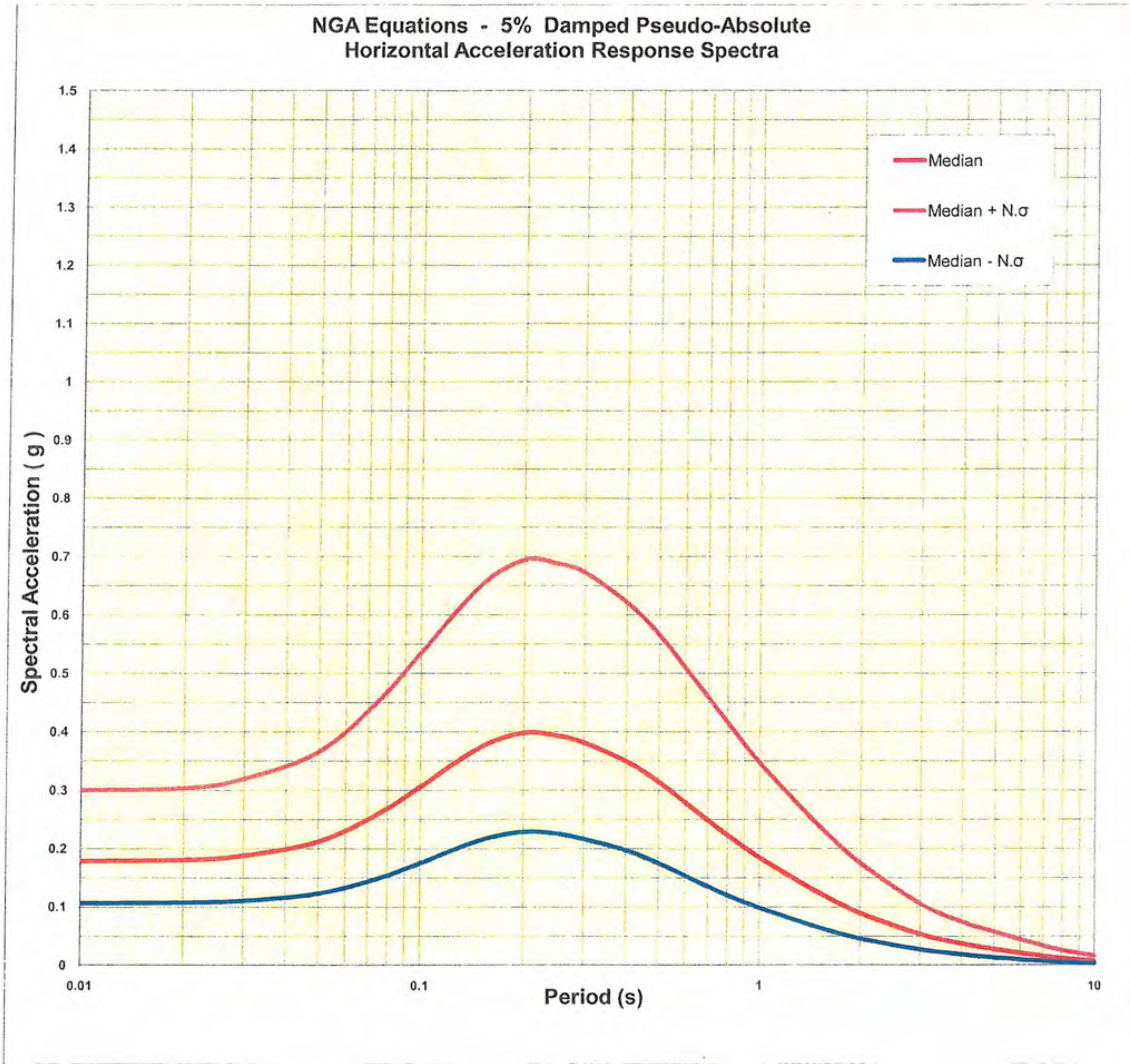
PSA	= Pseudo-absolute acceleration response spectrum (g; 5% damping)
PGA	= Peak ground acceleration (g)
PGV	= Peak ground velocity (cm/s)
SD	= Relative displacement response spectrum (cm; 5% damping)

Explanatory Variables
Original NCEER file

DEFINITION OF FAULT AND DISTANCE PARAMETERS:

Variable	Value	Symbol	Definition
M	6.90	M	= Moment magnitude
R_{rup} (km)	21.00	R_{RUP}	= Closest distance to coseismic rupture (km), used in AS08, CB08 and CY08. See Figures a, b and c for illustation
R_{JB} (km)	21.00	R_{JB}	= Closest distance to surface projection of coseismic rupture (km). See Figures a, b and c for illustation
R_X (km)	21.00	R_X	= Horizontal distance from top of rupture measured perpendicular to fault strike (km), used in AS08 and CY08. See Figures a, b and c for illustation
U	0	U	= Unspecified-mechanism factor: 1 for unspecified; 0 otherwise, used in BA08
F_{RV}	0	F_{RV}	= Reverse-faulting factor: 0 for strike slip, normal, normal-oblique; 1 for reverse, reverse-oblique and thrust
F_{NM}	0	F_N	= Normal-faulting factor: 0 for strike slip, reverse, reverse-oblique, thrust and normal-oblique; 1 for normal
F_{HW}	0	F_{HW}	= Hanging-wall factor: 1 for site on down-dip side of top of rupture; 0 otherwise, used in AS08 and CY08
Z_{TOR} (km)	0.00	Z_{TOR}	= Depth to top of coseismic rupture (km), used in AS08, CB08 and CY08
δ	90.00	δ	= Average dip of rupture plane (degrees), used in AS08, CB08 and CY08
V_{S30} (m/sec)	300	V_{S30}	= Average shear-wave velocity in top 30m of site profile
$F_{Measured}$	0	$F_{Measured}$	= Vs30 Factor: 1 if VS30 is measured, 0 if Vs30 is inferred, used in AS08 and CY08
$Z_{1.0}$ (m)	DEFAULT	$Z_{1.0}$	= Depth to 1.0 km/sec velocity horizon (m), used in AS08 and CY08. Enter "DEFAULT" in order to use the default values or enter your site-specific number
$Z_{2.5}$ (km)	DEFAULT	$Z_{2.5}$	= Depth of 2.5 km/s shear-wave velocity horizon (km), used in CB08. Enter "DEFAULT" in order to use the default value or enter your site-specific number
W (km)	20.00	W	= Fault rupture width (km), used in AS08
F_{AS}	0	F_{AS}	= Aftershock factor: 0 for mainshock; 1 for aftershock, used in AS08 and CY08
HW Taper	0	HW Taper	= To choose the hanging wall taper to be used in AS08. Enter 0 to use the hanging wall taper as published in Abrahamson and Silva (2008), or enter 1 to use the revised hanging wall taper suggested by Norm Abrahamson

Upper Sand Creek Detention Basin
Concord -Green Valley Fault



NGA Model	A-S	B-A	C-B	C-Y	I
Weights	0.00	0.33	0.33	0.33	0.00
			Sum of weights =		*100%*

N =	1.00

MCE MOMENT MAGNITUDE (M_w) 6.90

DISTANCE TO SURFACE PROJECTION OF COSEISMIC RUPTURE (R_{JB}) 21.00 km

PSA @ T = 0.01s (g)	Mean	Mean+σ	PSA @ T = 0.2s (g)	Mean	Mean+σ	PSA @ T = 1.0 (g)	Mean	Mean+σ
Abrahamson & Silva (2008)	0.156		A. & S. (2008)	0.343		A. & S. (2008)	0.091	
Boore & Atkinson (2008)	0.203		B. & A. (2008)	0.424		B. & A. (2008)	0.264	
Campbell & Bozorgnia (2008)	0.161		C. & B. (2008)	0.382		C. & B. (2008)	0.178	
Chiou & Youngs (2008)	0.173		Chiou & Youngs (2008)	0.389		Chiou & Youngs (2008)	0.066	
Idriss (2008)	Vs30 Limit		I. (2008)	Vs30 Limit		I. (2008)	Vs30 Limit	
Arithmetic Average	0.139	N/C	Arithmetic Average	0.385	N/C	Arithmetic Average	0.120	N/C
Geometric Average	0.178	0.303	Geometric Average	0.398	0.694	Geometric Average	0.185	0.348
PGA (g)	0.178	0.299						

INPUT DATA PREPARATION - NGA Equations (2008) - Updated 2009

Lsansone 04/19/10 **DO NOT CHANGE LOCATIONS OF ANY CELLS** To be used w/Linda Atik S/S (NCEER, 2009)

$M (M_w)$	6.90	= Moment magnitude

FAULT: Greenville
Type: SS
Rupture Width: 18.0 km

Project: Upper Sand Creek
Owner: CCWD

R	9.00	= Distance to plane of rupture (km)	*[distance to plane of rupture]*
R_{JB}	9.00	= Distance to surf. projection of coseismic rupture (km)	*[hor. dist. to vert. projection of rupture]*
$Z_{2.5}$	2.5	= Depth of 2.5 km/s shear-wave velocity horizon (km)	*[estimated depth to basement rock]*
$Z_{2.5km}$	DEFAULT	= Depth to 2.5 km/sec Vs horizon (m) [CB Input!A1]. Enter "DEFAULT" for default values or enter known value	
R_{RUP}	9.0	= Closest distance to coseismic rupture (km) [Rx]	*[distance to zone of seismogenic rupture]*
F_{RV}	0	= 0 for strike slip, normal & normal-oblique; 1 for reverse, reverse-oblique & thrust [Reverse-faulting factor]	
F_{NM}	0	= 0 for strike slip, reverse, reverse-oblique & thrust; 1 for normal & normal-oblique [Normal-faulting factor]	
F_{HW}	0	= Hanging-Wall Factor = 1 for site on down-dip, 0 otherwise (foot wall or vertical fault), see AS & CY	HW
Z_{TOR}	0	= Depth to top of coseismic rupture (km)	*[observed/assumed surface rupture, $Z_{TOR} = 0$]*
$Z_{1.0km}$	DEFAULT	= Depth to 1.0 km/sec Vs horizon (m) [AS & CY]. Enter "DEFAULT" for default values or enter known value	
δ	90	= Average dip of rupture plane (degrees)	*c* 12 *[adjusted from 10 in Idriss' original version]*
F_{AS}	0	= Aftershock Factor. 0 = Main rupture. 1 = Aftershock	
V_{S30}	300	= Average shear-wave velocity in top 30m of site profile (m/s)	
$F_{Measured}$	0	V_s measured? (yes or no) no (V_s upper 30m = 984 ft/s)	$F_{measured}$ =1 if measured, $F_{measured}$ = 0 otherwise
U	0	= Unspecified Mechanism Factor (Boore-Atkinson): 1 is unspecified, 0 otherwise.	
Hw_{taper}	0	= Hanging Wall Taper. 0 = Abrahamson & Silva (2008) or 1 = Abrahamson alternate formula (revised)	

GRAPH TITLE Line 1:	**Greenville Fault**	FAULT NAME
GRAPH TITLE Line 2:	**Upper Sand Creek Detention Basin**	FACILITY NAME

ORIGINAL I+S1NPUT BY NCEER (2009 - LINDA ATIK)

This Excel file calculates the weighted average of the natural logarithm of the spectral values from the NGA models

NGA Model:	AS08	BA08	CB08	CY08	I08
Weight:	0	0.333	0.333	0.333	0
					100%

N	1	Fraction (or number) of standard deviations to be considered in the calculations

AS08: Abrahamson & Silva 2008 NGA Model
BA08: Boore & Atkinson 2008 NGA Model
CB08: Campbell & Bozorgnia 2008 NGA Model
CY08: Chiou & Youngs 2008 NGA Model
I08: Idriss 2008 NGA Model

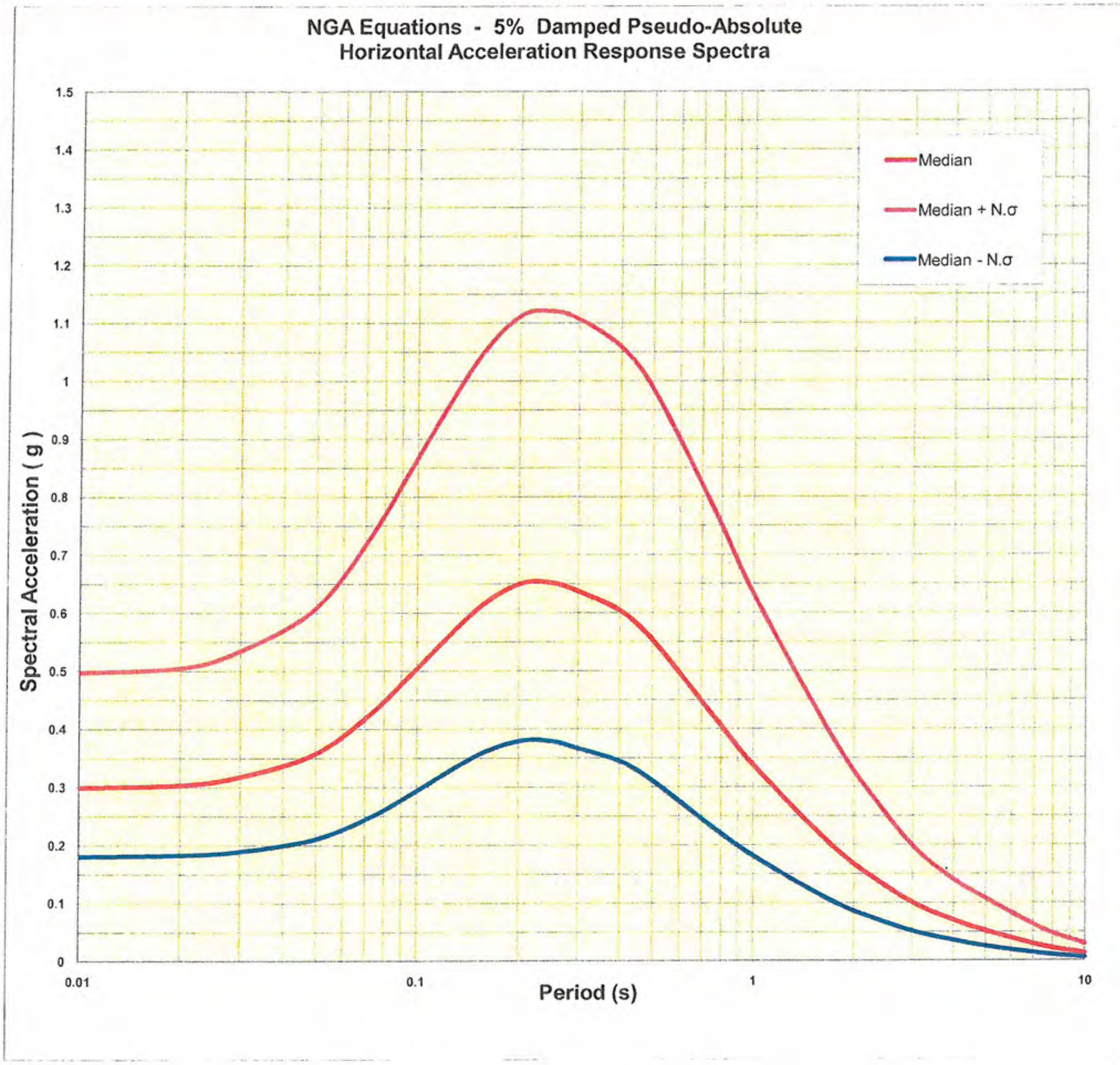
PSA	= Pseudo-absolute acceleration response spectrum (g; 5% damping)
PGA	= Peak ground acceleration (g)
PGV	= Peak ground velocity (cm/s)
SD	= Relative displacement response spectrum (cm; 5% damping)

Explanatory Variables
Original NCEER file

DEFINITION OF FAULT AND DISTANCE PARAMETERS:

Value	Parameter	Definition
6.90	M	= Moment magnitude
9.00	R_{RUP}	= Closest distance to coseismic rupture (km), used in AS08, CB08 and CY08. See Figures a, b and c for illustation
9.00	R_{JB}	= Closest distance to surface projection of coseismic rupture (km). See Figures a, b and c for illustation
9.00	R_X	= Horizontal distance from top of rupture measured perpendicular to fault strike (km), used in AS08 and CY08 See Figures a, b and c for illustation
0	U	= Unspecified-mechanism factor: 1 for unspecified; 0 otherwise, used in BA08
0	F_{RV}	= Reverse-faulting factor: 0 for strike slip, normal, normal-oblique; 1 for reverse, reverse-oblique and thrust
0	F_N	= Normal-faulting factor: 0 for strike slip, reverse, reverse-oblique, thrust and normal-oblique; 1 for normal
0	F_{HW}	= Hanging-wall factor: 1 for site on down-dip side of top of rupture; 0 otherwise, used in AS08 and CY08
0.00	Z_{TOR}	= Depth to top of coseismic rupture (km), used in AS08, CB08 and CY08
90.00	δ	= Average dip of rupture plane (degrees), used in AS08, CB08 and CY08
300	V_{S30}	= Average shear-wave velocity in top 30m of site profile
0	$F_{Measured}$	= Vs30 Factor: 1 if VS30 is measured, 0 if Vs30 is inferred, used in AS08 and CY08
DEFAULT	$Z_{1.0}$	= Depth to 1.0 km/sec velocity horizon (m), used in AS08 and CY08. Enter "DEFAULT" in order to use the default values or enter your site-specific number
DEFAULT	$Z_{2.5}$	= Depth of 2.5 km/s shear-wave velocity horizon (km), used in CB08. Enter "DEFAULT" in order to use the default value or enter your site-specific number
18.00	W	= Fault rupture width (km), used in AS08
0	F_{AS}	= Aftershock factor: 0 for mainshock; 1 for aftershock, used in AS08 and CY08
0	*HW Taper*	= To choose the hanging wall taper to be used in AS08. Enter 0 to use the hanging wall taper as published in Abrahamson and Silva (2008), or enter 1 to use the revised hanging wall taper suggested by Norm Abrahamson

Upper Sand Creek Detention Basin
Greenville Fault



NGA Model	A-S	B-A	C-B	C-Y	I
Weights	0.00	0.33	0.33	0.33	0.00
			Sum of weights =		100%

N =	1.00

MCE MOMENT MAGNITUDE (M_w) 6.90

DISTANCE TO SURFACE PROJECTION OF COSEISMIC RUPTURE (R_{JB}) 9.00 km

PSA @ T = 0.01s (g)	Mean	Mean+σ	PSA @ T = 0.2s (g)	Mean	Mean+σ	PSA @ T = 1.0 (g)	Mean	Mean+σ
Abrahamson & Silva (2008)	0.281		A. & S. (2008)	0.585		A. & S. (2008)	0.163	
Boore & Atkinson (2008)	0.298		B. & A. (2008)	0.649		B. & A. (2008)	0.439	
Campbell & Bozorgnia (2008)	0.279		C. & B. (2008)	0.617		C. & B. (2008)	0.324	
Chiou & Youngs (2008)	0.320		Chiou & Youngs (2008)	0.683		Chiou & Youngs (2008)	0.141	
Idriss (2008)	Vs30 Limit		I. (2008)	Vs30 Limit		I. (2008)	Vs30 Limit	
Arithmetic Average	0.236	N/C	Arithmetic Average	0.633	N/C	Arithmetic Average	0.213	N/C
Geometric Average	0.299	0.505	Geometric Average	0.649	1.112	Geometric Average	0.338	0.633
PGA (g)	0.299	0.496						

INPUT DATA PREPARATION - NGA Equations (2008) - Updated 2009

Lsansone 04/19/10 **DO NOT CHANGE LOCATIONS OF ANY CELLS** To be used w/Linda Atik S/S (NCEER, 2009)

M (M_w)	6.70	= Moment magnitude	FAULT: Pittsburg- Kirby Hills (GV 5)	**Project:**	Upper Sand Creek
			Type: SS	Owner:	CCWD
			Rupture Width: 25.0 km		

R	14.00	= Distance to plane of rupture (km) *[distance to plane of rupture]*
R_{JB}	14.00	= Distance to surf. projection of coseismic rupture (km) *[hor. dist. to vert. projection of rupture]*
$Z_{2.5}$	2.5	= Depth of 2.5 km/s shear-wave velocity horizon (km) *[estimated depth to basement rock]*
$Z_{2.5km}$	DEFAULT	= Depth to 2.5 km/sec Vs horizon (m) [CB Input!A1]. Enter "DEFAULT" for default values or enter known value
R_{RUP}	10.0	= Closest distance to coseismic rupture (km) [Rx] *[distance to zone of seismogenic rupture]*
F_{RV}	0	= 0 for strike slip, normal & normal-oblique; 1 for reverse, reverse-oblique & thrust [Reverse-faulting factor]
F_{NM}	0	= 0 for strike slip, reverse, reverse-oblique & thrust; 1 for normal & normal-oblique [Normal-faulting factor]
F_{HW}	0	= Hanging-Wall Factor = 1 for site on down-dip, 0 otherwise (foot wall or vertical fault), see AS & CY HW
Z_{TOR}	0	= Depth to top of coseismic rupture (km) *[observed/assumed surface rupture, $Z_{TOR} = 0$]*
$Z_{1.0km}$	DEFAULT	= Depth to 1.0 km/sec Vs horizon (m) [AS & CY]. Enter "DEFAULT" for default values or enter known value
δ	90	= Average dip of rupture plane (degrees) *c* **12** *[adjusted from 10 in Idriss' original version]*
F_{AS}	0	= Aftershock Factor. 0 = Main rupture. 1 = Aftershock
V_{S30}	300	= Average shear-wave velocity in top 30m of site profile (m/s)
$F_{Measured}$	0	V_s measured? (yes or no) no (V_s upper 30m = **984** ft/s) $F_{measured}$ =1 if measured, $F_{measured}$ = 0 otherwise
U	0	= Unspecified Mechanism Factor (Boore-Atkinson): 1 is unspecified, 0 otherwise.
Hw_{taper}	0	= Hanging Wall Taper. 0 = Abrahamson & Silva (2008) or 1 = Abrahamson alternate formula (revised)

GRAPH TITLE Line 1:	**Great Valley 5, Pittsburg-Kirby**	FAULT NAME
GRAPH TITLE Line 2:	**Upper Sand Creek Detention Basin**	FACILITY NAME

ORIGINAL I+S INPUT BY NCEER (2009 - LINDA ATIK)

This Excel file calculates the weighted average of the natural logarithm of the spectral values from the NGA models

NGA Model:	AS08	BA08	CB08	CY08	I08
Weight:	0	0.333	0.333	0.333	0
					100%

N	1	Fraction (or number) of standard deviations to be considered in the calculations

AS08: Abrahamson & Silva 2008 NGA Model
BA08: Boore & Atkinson 2008 NGA Model
CB08: Campbell & Bozorgnia 2008 NGA Model
CY08: Chiou & Youngs 2008 NGA Model
I08: Idriss 2008 NGA Model

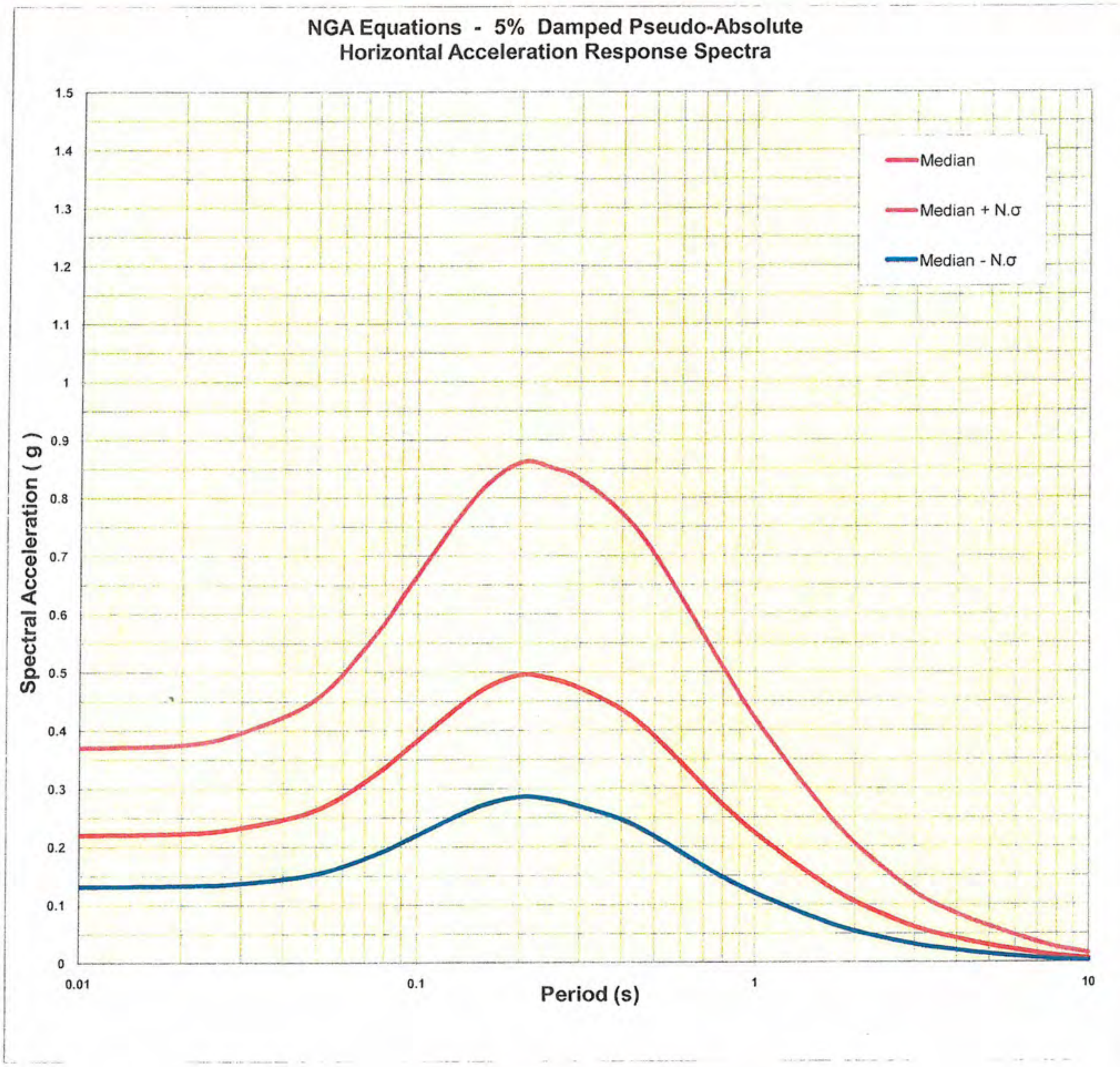
PSA	**= Pseudo-absolute acceleration response spectrum (g; 5% damping)**
PGA	**= Peak ground acceleration (g)**
PGV	**= Peak ground velocity (cm/s)**
SD	**= Relative displacement response spectrum (cm; 5% damping)**

Explanatory Variables
Original NCEER file

DEFINITION OF FAULT AND DISTANCE PARAMETERS:

Variable	Value	Parameter	Definition
$\mathbf{M}$	6.70	$\mathbf{M}$	= Moment magnitude
R_{RUP} (km)	14.00	R_{RUP}	= Closest distance to coseismic rupture (km), used in AS08, CB08 and CY08. See Figures a, b and c for illustation
R_{JB} (km)	14.00	R_{JB}	= Closest distance to surface projection of coseismic rupture (km). See Figures a, b and c for illustation
R_X (km)	10.00	R_X	= Horizontal distance from top of rupture measured perpendicular to fault strike (km), used in AS08 and CY08. See Figures a, b and c for illustation
U	0	U	= Unspecified-mechanism factor: 1 for unspecified; 0 otherwise, used in BA08
F_{RV}	0	F_{RV}	= Reverse-faulting factor: 0 for strike slip, normal, normal-oblique; 1 for reverse, reverse-oblique and thrust
F_N	0	F_N	= Normal-faulting factor: 0 for strike slip, reverse, reverse-oblique, thrust and normal-oblique, 1 for normal
F_{HW}	0	F_{HW}	= Hanging-wall factor: 1 for site on down-dip side of top of rupture; 0 otherwise, used in AS08 and CY08
Z_{TOR} (km)	0.00	Z_{TOR}	= Depth to top of coseismic rupture (km), used in AS08, CB08 and CY08
δ	90.00	δ	= Average dip of rupture plane (degrees), used in AS08, CB08 and CY08
V_{S30} (m/sec)	300	V_{S30}	= Average shear-wave velocity in top 30m of site profile
$F_{Measured}$	0	$F_{Measured}$	= Vs30 Factor: 1 if VS30 is measured, 0 if Vs30 is inferred, used in AS08 and CY08
$Z_{1.0}$ (m)	DEFAULT	$Z_{1.0}$	= Depth to 1.0 km/sec velocity horizon (m), used in AS08 and CY08. Enter "DEFAULT" in order to use the default values or enter your site-specific number
$Z_{2.5}$ (km)	DEFAULT	$Z_{2.5}$	= Depth of 2.5 km/s shear-wave velocity horizon (km), used in CB08. Enter "DEFAULT" in order to use the default value or enter your site-specific number
W (km)	25.00	W	= Fault rupture width (km), used in AS08
F_{AS}	0	F_{AS}	= Aftershock factor: 0 for mainshock; 1 for aftershock, used in AS08 and CY08
HW Taper	0	***HW Taper***	= To choose the hanging wall taper to be used in AS08. Enter 0 to use the hanging wall taper as published in Abrahamson and Silva (2008), or enter 1 to use the revised hanging wall taper suggested by Norm Abrahamson

Upper Sand Creek Detention Basin
Great Valley 5, Pittsburg-Kirby



NGA Model	A-S	B-A	C-B	C-Y	I
Weights	0.00	0.33	0.33	0.33	0.00
			Sum of weights =		100%

N =	1.00

MCE MOMENT MAGNITUDE (M_w) **6.70**

DISTANCE TO SURFACE PROJECTION OF COSEISMIC RUPTURE (R_{JB}) **14.00 km**

PSA @ T = 0.01s (g)	Mean	Mean+σ	PSA @ T = 0.2s (g)	Mean	Mean+σ	PSA @ T = 1.0 (g)	Mean	Mean+σ
Abrahamson & Silva (2008)	0.195		A. & S. (2008)	0.420		A. & S. (2008)	0.104	
Boore & Atkinson (2008)	0.231		B. & A. (2008)	0.508		B. & A. (2008)	0.309	
Campbell & Bozorgnia (2008)	0.209		C. & B. (2008)	0.492		C. & B. (2008)	0.214	
Chiou & Youngs (2008)	0.221		Chiou & Youngs (2008)	0.486		Chiou & Youngs (2008)	0.085	
Idriss (2008)	Vs30 Limit		I. (2008)	Vs30 Limit		I. (2008)	Vs30 Limit	
Arithmetic Average	0.171	N/C	Arithmetic Average	0.476	N/C	Arithmetic Average	0.143	N/C
Geometric Average	0.220	0.375	Geometric Average	0.495	0.860	Geometric Average	0.224	0.421
PGA (g)	0.220	0.369						

INPUT DATA PREPARATION - NGA Equations (2008) - Updated 2009

Lsansone 04/19/10 **DO NOT CHANGE LOCATIONS OF ANY CELLS** To be used w/Linda Atik S/S (NCEER, 2009)

$M (M_w)$ 6.70 = Moment magnitude

FAULT: Mount Diablo
Type: *Blind Thrust-R*
Rupture Width: 19.0 km

Project: Upper Sand Creek
Owner: CCWD

R	20.00	= Distance to plane of rupture (km) *[distance to plane of rupture]*
R_{JB}	7.00	= Distance to surf. projection of coseismic rupture (km) *[hor. dist. to vert. projection of rupture]*
$Z_{2.5}$	2.5	= Depth of 2.5 km/s shear-wave velocity horizon (km) *[estimated depth to basement rock]*
$Z_{2.5km}$	DEFAULT	= Depth to 2.5 km/sec Vs horizon (m) [CB Input!A1]. Enter "DEFAULT" for default values or enter known value
R_{RUP}	14.0	= Closest distance to coseismic rupture (km) [Rx] *[distance to zone of seismogenic rupture]*
F_{RV}	1	= 0 for strike slip, normal & normal-oblique; 1 for reverse, reverse-oblique & thrust [Reverse-faulting factor]
F_{NM}	0	= 0 for strike slip, reverse, reverse-oblique & thrust; 1 for normal & normal-oblique [Normal-faulting factor]
F_{HW}	1	= Hanging-Wall Factor = 1 for site on down-dip, 0 otherwise (foot wall or vertical fault), see AS & CY HW
Z_{TOR}	1	= Depth to top of coseismic rupture (km) *[observed/assumed surface rupture, $Z_{TOR} = 0$]*
$Z_{1.0km}$	DEFAULT	= Depth to 1.0 km/sec Vs horizon (m) [AS & CY]. Enter "DEFAULT" for default values or enter known value
δ	30	= Average dip of rupture plane (degrees) **c** 12 *[adjusted from 10 in Idriss' original version]*
F_{AS}	0	= Aftershock Factor. 0 = Main rupture. 1 = Aftershock
V_{S30}	300	= Average shear-wave velocity in top 30m of site profile (m/s)
$F_{Measured}$	0	V_s measured? (yes or no) no (V_s upper 30m = 984 ft/s) $F_{measured}$ =1 if measured, $F_{measured}$ = 0 otherwise
U	0	= Unspecified Mechanism Factor (Boore-Atkinson): 1 is unspecified, 0 otherwise.
Hw_{taper}	1	= Hanging Wall Taper. 0 = Abrahamson & Silva (2008) or 1 = Abrahamson alternate formula (revised)

GRAPH TITLE Line 1: **Mount Diablo Blind Thrust Fault** FAULT NAME
GRAPH TITLE Line 2: **Upper Sand Creek Detention Basin** FACILITY NAME

ORIGINAL I+S INPUT BY NCEER (2009 - LINDA ATIK)

This Excel file calculates the weighted average of the natural logarithm of the spectral values from the NGA models

NGA Model:	AS08	BA08	CB08	CY08	I08
Weight:	0	0.33	0.33	0.33	0
					99%

Weights should sum up to 1

N	1

Fraction (or number) of standard deviations to be considered in the calculations

AS08: Abrahamson & Silva 2008 NGA Model
BA08: Boore & Atkinson 2008 NGA Model
CB08: Campbell & Bozorgnia 2008 NGA Model
CY08: Chiou & Youngs 2008 NGA Model
I08: Idriss 2008 NGA Model

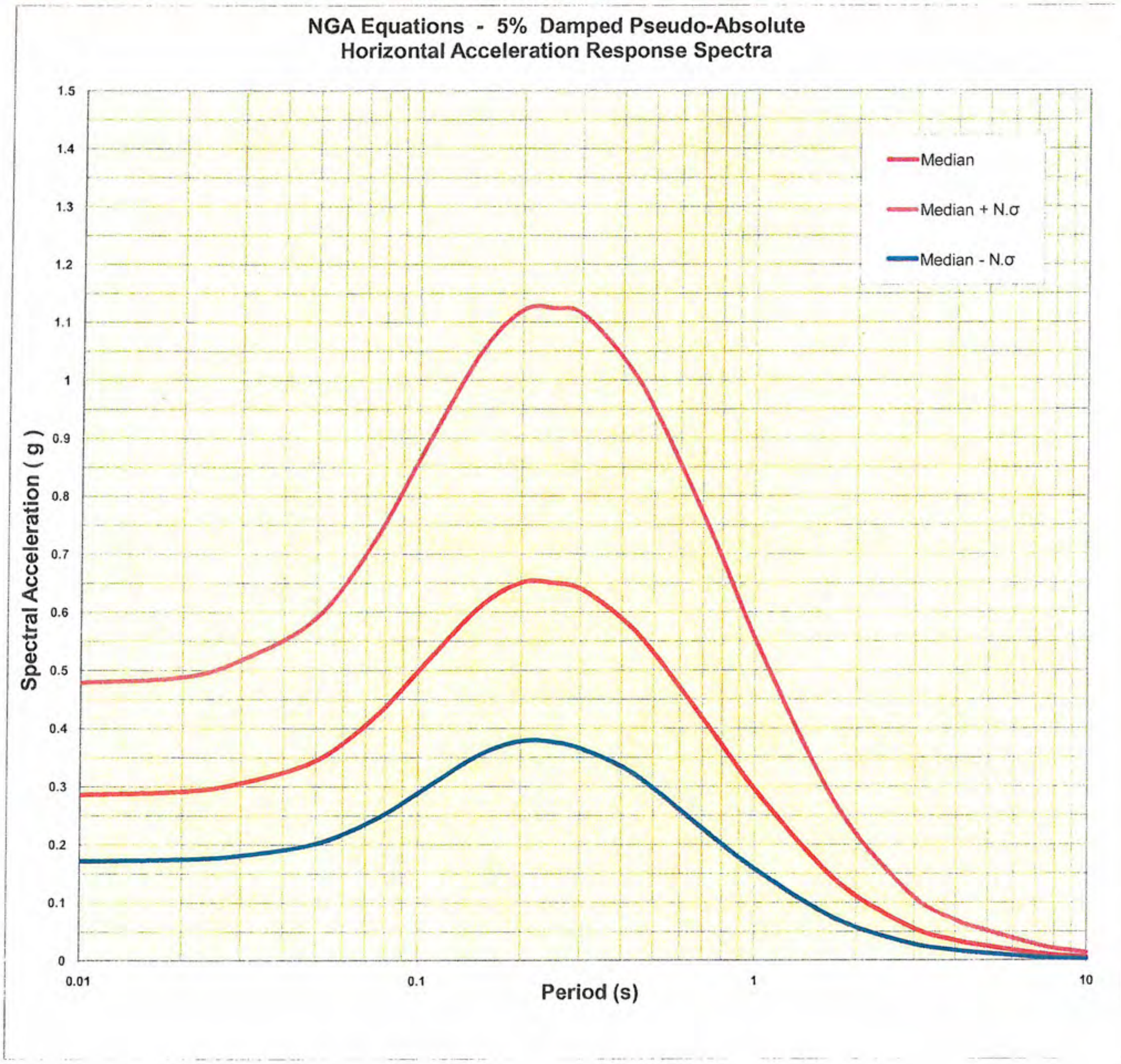
PSA	**= Pseudo-absolute acceleration response spectrum (g; 5% damping)**
PGA	**= Peak ground acceleration (g)**
PGV	**= Peak ground velocity (cm/s)**
SD	**= Relative displacement response spectrum (cm; 5% damping)**

Explanatory Variables
Original NCEER file

DEFINITION OF FAULT AND DISTANCE PARAMETERS:

Value	Parameter	Definition
6.70	**M**	= Moment magnitude
20.00	R_{RUP}	= Closest distance to coseismic rupture (km), used in AS08, CB08 and CY08. See Figures a, b and c for illustation
7.00	R_{JB}	= Closest distance to surface projection of coseismic rupture (km). See Figures a, b and c for illustation
14.00	R_X	= Horizontal distance from top of rupture measured perpendicular to fault strike (km), used in AS08 and CY08. See Figures a, b and c for illustation
0	U	= Unspecified-mechanism factor: 1 for unspecified; 0 otherwise, used in BA08
1	F_{RV}	= Reverse-faulting factor: 0 for strike slip, normal, normal-oblique; 1 for reverse, reverse-oblique and thrust
0	F_N	= Normal-faulting factor: 0 for strike slip, reverse, reverse-oblique, thrust and normal-oblique; 1 for normal
1	F_{HW}	= Hanging-wall factor: 1 for site on down-dip side of top of rupture; 0 otherwise, used in AS08 and CY08
1.00	Z_{TOR}	= Depth to top of coseismic rupture (km), used in AS08, CB08 and CY08
30.00	δ	= Average dip of rupture plane (degrees), used in AS08, CB08 and CY08
300	V_{S30}	= Average shear-wave velocity in top 30m of site profile
0	$F_{Measured}$	= Vs30 Factor: 1 if VS30 is measured, 0 if Vs30 is inferred, used in AS08 and CY08
DEFAULT	$Z_{1.0}$	= Depth to 1.0 km/sec velocity horizon (m), used in AS08 and CY08. Enter "DEFAULT" in order to use the default values or enter your site-specific number
DEFAULT	$Z_{2.5}$	= Depth of 2.5 km/s shear-wave velocity horizon (km), used in CB08. Enter "DEFAULT" in order to use the default value or enter your site-specific number
19.00	W	= Fault rupture width (km), used in AS08
0	F_{AS}	= Aftershock factor: 0 for mainshock; 1 for aftershock, used in AS08 and CY08
1	*HW Taper*	= To choose the hanging wall taper to be used in AS08. Enter 0 to use the hanging wall taper as published in Abrahamson and Silva (2008), or enter 1 to use the revised hanging wall taper suggested by Norm Abrahamson

Upper Sand Creek Detention Basin
Mount Diablo Blind Thrust Fault



NGA Model	A-S	B-A	C-B	C-Y	I
Weights	0.00	0.33	0.33	0.33	0.00
			Sum of weights =		99%

N = 1.00

MCE MOMENT MAGNITUDE (M_w) 6.70

DISTANCE TO SURFACE PROJECTION OF COSEISMIC RUPTURE (R_{JB}) 7.00 km

PSA @ T = 0.01s (g)	Mean	Mean+σ	PSA @ T = 0.2s (g)	Mean	Mean+σ	PSA @ T = 1.0 (g)	Mean	Mean+σ
Abrahamson & Silva (2008)	0.250		A. & S. (2008)	0.554		A. & S. (2008)	0.140	
Boore & Atkinson (2008)	0.310		B. & A. (2008)	0.719		B. & A. (2008)	0.505	
Campbell & Bozorgnia (2008)	0.259		C. & B. (2008)	0.590		C. & B. (2008)	0.275	
Chiou & Youngs (2008)	0.291		Chiou & Youngs (2008)	0.650		Chiou & Youngs (2008)	0.099	
Idriss (2008)	Vs30 Limit		I. (2008)	Vs30 Limit		I. (2008)	Vs30 Limit	
Arithmetic Average	0.222	N/C	Arithmetic Average	0.628	N/C	Arithmetic Average	0.204	N/C
Geometric Average	0.286	0.488	Geometric Average	0.651	1.122	Geometric Average	0.297	0.557
PGA (g)	0.286	0.478						

INPUT DATA PREPARATION - NGA Equations (2008) - Updated 2009

Lsansone 04/19/10 **DO NOT CHANGE LOCATIONS OF ANY CELLS** To be used w/Linda Atik S/S (NCEER, 2009)

M (M_w)	6.50	= Moment magnitude	FAULT: Montezuma Hills	Project: Upper Sand Creek
			Type: *Floating E-Quake- RO*	Owner: CCWD
			Rupture Width: 20.0 km	

R	10.00	= Distance to plane of rupture (km)	*[distance to plane of rupture]*
R_{JB}	10.00	= Distance to surf. projection of coseismic rupture (km)	*[hor. dist. to vert. projection of rupture]*
$Z_{2.5}$	2.5	= Depth of 2.5 km/s shear-wave velocity horizon (km)	*[estimated depth to basement rock]*
$Z_{2.5km}$	DEFAULT	= Depth to 2.5 km/sec Vs horizon (m) [CB Input!A1]. Enter "DEFAULT" for default values or enter known value	
R_{RUP}	10.0	= Closest distance to coseismic rupture (km) [Rx]	*[distance to zone of seismogenic rupture]*
F_{RV}	1	= 0 for strike slip, normal & normal-oblique; 1 for reverse, reverse-oblique & thrust [Reverse-faulting factor]	
F_{NM}	0	= 0 for strike slip, reverse, reverse-oblique & thrust; 1 for normal & normal-oblique [Normal-faulting factor]	
F_{HW}	1	= Hanging-Wall Factor = 1 for site on down-dip, 0 otherwise (foot wall or vertical fault), see AS & CY	HW
Z_{TOR}	0	= Depth to top of coseismic rupture (km)	*[observed/assumed surface rupture, $Z_{TOR} = 0$]*
$Z_{1.0km}$	DEFAULT	= Depth to 1.0 km/sec Vs horizon (m) [AS & CY]. Enter "DEFAULT" for default values or enter known value	
δ	70	= Average dip of rupture plane (degrees)	c 12 *[adjusted from 10 in Idriss' original version]*
F_{AS}	0	= Aftershock Factor. 0 = Main rupture. 1 = Aftershock	
V_{S30}	300	= Average shear-wave velocity in top 30m of site profile (m/s)	
$F_{Measured}$	0	V_s measured? (yes or no) no (V_s upper 30m = 984 ft/s)	$F_{measured}$ =1 if measured, $F_{measured}$ = 0 otherwise
U	0	= Unspecified Mechanism Factor (Boore-Atkinson): 1 is unspecified, 0 otherwise.	
Hw_{taper}	1	= Hanging Wall Taper. 0 = Abrahamson & Silva (2008) or 1 = Abrahamson alternate formula (revised)	

GRAPH TITLE Line 1:	**Montezuma Hills Zone (Sherman Island Fault)**	FAULT NAME
GRAPH TITLE Line 2:	**Upper Sand Creek Reservoir**	FACILITY NAME

ORIGINAL I+S INPUT BY NCEER (2009 - LINDA ATIK)

This Excel file calculates the weighted average of the natural logarithm of the spectral values from the NGA models

NGA Model:	AS08	BA08	CB08	CY08	I08	
Weight:	0	0.33	0.33	0.33	0	**Weights should sum up to 1**
					99%	

N	1	**Fraction (or number) of standard deviations to be considered in the calculations**

AS08: Abrahamson & Silva 2008 NGA Model
BA08: Boore & Atkinson 2008 NGA Model
CB08: Campbell & Bozorgnia 2008 NGA Model
CY08: Chiou & Youngs 2008 NGA Model
I08: Idriss 2008 NGA Model

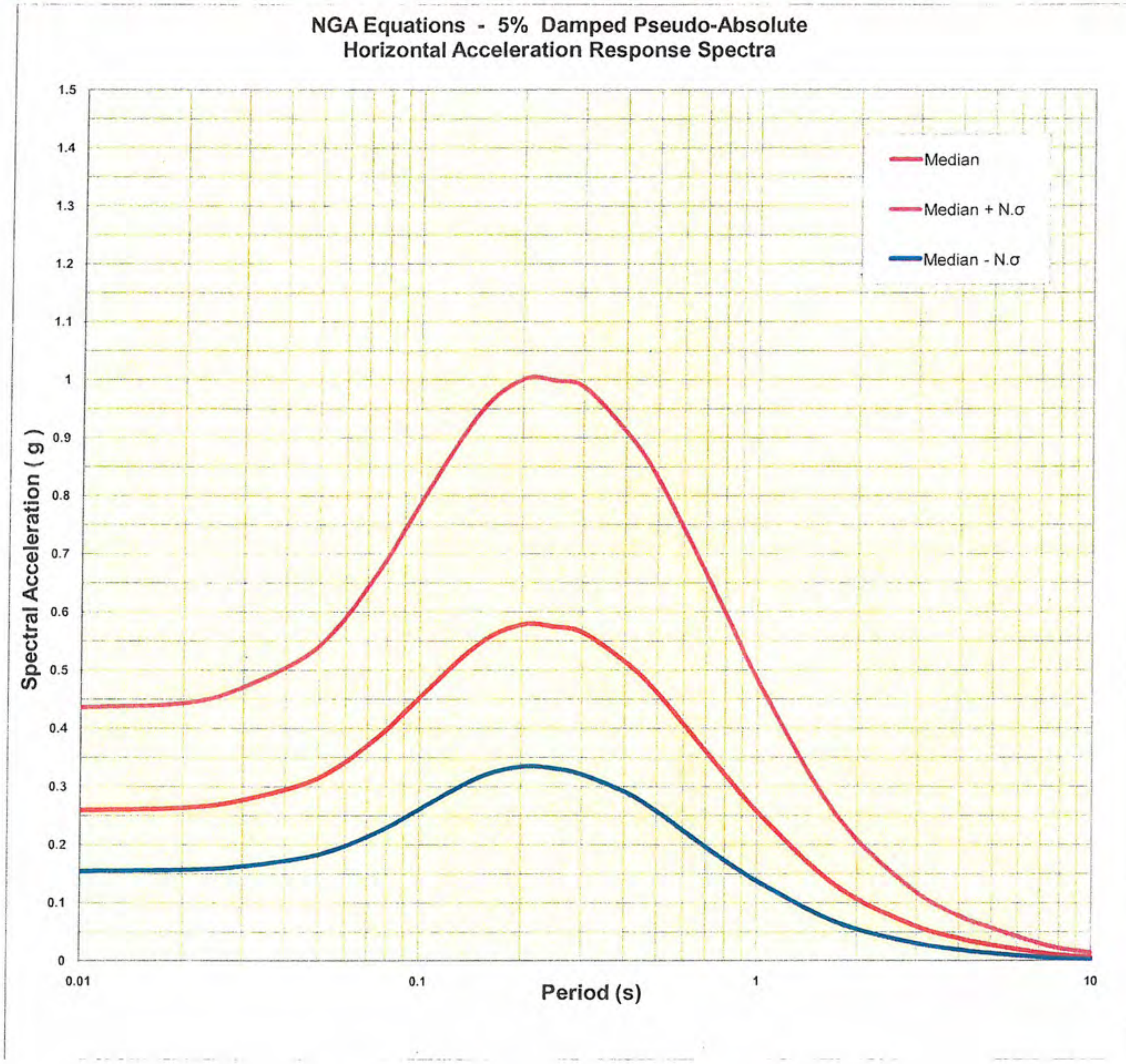
PSA	**= Pseudo-absolute acceleration response spectrum (g; 5% damping)**
PGA	**= Peak ground acceleration (g)**
PGV	**= Peak ground velocity (cm/s)**
SD	**= Relative displacement response spectrum (cm; 5% damping)**

Explanatory Variables
Original NCEER file

DEFINITION OF FAULT AND DISTANCE PARAMETERS:

Value	Parameter	Definition
6.50	M	= Moment magnitude
10.00	R_{RUP}	= Closest distance to coseismic rupture (km), used in AS08, CB08 and CY08. See Figures a, b and c for illustation
10.00	R_{JB}	= Closest distance to surface projection of coseismic rupture (km). See Figures a, b and c for illustation
10.00	R_X	= Horizontal distance from top of rupture measured perpendicular to fault strike (km), used in AS08 and CY08. See Figures a, b and c for illustation
0	U	= Unspecified-mechanism factor: 1 for unspecified; 0 otherwise, used in BA08
1	F_{RV}	= Reverse-faulting factor: 0 for strike slip, normal, normal-oblique; 1 for reverse, reverse-oblique and thrust
0	F_N	= Normal-faulting factor: 0 for strike slip, reverse, reverse-oblique, thrust and normal-oblique; 1 for normal
1	F_{HW}	= Hanging-wall factor: 1 for site on down-dip side of top of rupture; 0 otherwise, used in AS08 and CY08
0.00	Z_{TOR}	= Depth to top of coseismic rupture (km), used in AS08, CB08 and CY08
70.00	δ	= Average dip of rupture plane (degrees), used in AS08, CB08 and CY08
300	V_{S30}	= Average shear-wave velocity in top 30m of site profile
0	$F_{Measured}$	= Vs30 Factor: 1 if VS30 is measured, 0 if Vs30 is inferred, used in AS08 and CY08
DEFAULT	$Z_{1.0}$	= Depth to 1.0 km/sec velocity horizon (m), used in AS08 and CY08. Enter "DEFAULT" in order to use the default values or enter your site-specific number
DEFAULT	$Z_{2.5}$	= Depth of 2.5 km/s shear-wave velocity horizon (km), used in CB08. Enter "DEFAULT" in order to use the default value or enter your site-specific number
20.00	*W*	= Fault rupture width (km), used in AS08
0	F_{AS}	= Aftershock factor: 0 for mainshock; 1 for aftershock, used in AS08 and CY08
1	*HW Taper*	= To choose the hanging wall taper to be used in AS08. Enter 0 to use the hanging wall taper as published in Abrahamson and Silva (2008), or enter 1 to use the revised hanging wall taper suggested by Norm Abrahamson

Upper Sand Creek Reservoir
Montezuma Hills Zone (Sherman Island Fault)



NGA Model	A-S	B-A	C-B	C-Y	I
Weights	0.00	0.33	0.33	0.33	0.00
			Sum of weights =		99%

N = 1.00

MCE MOMENT MAGNITUDE (M_w) 6.50

DISTANCE TO SURFACE PROJECTION OF COSEISMIC RUPTURE (R_{JB}) 10.00 km

PSA @ T = 0.01s (g)	Mean	Mean+σ	PSA @ T = 0.2s (g)	Mean	Mean+σ	PSA @ T = 1.0 (g)	Mean	Mean+σ
Abrahamson & Silva (2008)	0.250		A. & S. (2008)	0.544		A. & S. (2008)	0.132	
Boore & Atkinson (2008)	0.242		B. & A. (2008)	0.548		B. & A. (2008)	0.340	
Campbell & Bozorgnia (2008)	0.254		C. & B. (2008)	0.582		C. & B. (2008)	0.248	
Chiou & Youngs (2008)	0.283		Chiou & Youngs (2008)	0.606		Chiou & Youngs (2008)	0.108	
Idriss (2008)	Vs30 Limit		I. (2008)	Vs30 Limit		I. (2008)	Vs30 Limit	
Arithmetic Average	0.206	N/C	Arithmetic Average	0.570	N/C	Arithmetic Average	0.166	N/C
Geometric Average	0.260	0.444	Geometric Average	0.578	1.001	Geometric Average	0.258	0.484
PGA (g)	0.259	0.435						

Appendix 2.3

Filter Compatibility

Filter Compatibility Calculations

Objective:

The purpose of this document is to summarize our filter design calculations for the Upper Sand Creek Detention Basin dam.

References:

[1] ASTM C33-03 – Standard Specifications for Concrete Aggregates

[2] Standard Specifications, State of California Department of Transportation, May 2006

[3] U.S. Bureau of Reclamation, 1987a, *Design Standards No. 13 - Embankment Dams, Chapter 5 - Protective Filters,* May 13, 1987.

[4] United States Department of Agriculture (1994), Natural Resources Conservation Service, Part 633 National Engineering Handbook, Chapter 26 – Gradation Design of Sand and Gravel Filters, October 1994.

[5] Cal Engineering & Geology, Inc. (2010). Geotechnical Data Report, Contra Costa County Flood Control & Water Conservation District, Upper Sand Creek Detention Basin, Deer Creek Road, Antioch, California, May 5.

[6] GEI Consultants, Inc. (2009). Upper Sand Creek Detention Basin, Basis of Design Memorandum, May.

[7] GEI Consultants, Inc. (2010). Upper Sand Creek Detention Basin, 35% Design Review Drawings, May.

Approach:

1. Follow step-by-step procedure outlined in NRCS Chapter 26 (Ref. 4) to develop maximum and minimum grain size distributions for the proposed sand and gravel filters.
2. Confirm that the proposed filter materials fit within the grain size curves developed in the previous step.

Results:

The proposed filter sand (Type 2) is an acceptable filter for the embankment fill (Type 1). The proposed drain gravel (Type 3) is an acceptable filter for the filter sand (Type 2).

Summary:

The Basis of Design Memorandum indicates that filter compatibility will be required between the embankment fill (Type 1 and 4) and the downstream filter zone (Type 2) and between the downstream filter zone (Type 2) and the base drain (Zone 3). Filter compatibility is also required between the dam foundation soils and the downstream filter zone (Type 2). However, the finest-grained borrow materials are typically obtained from soils of the same unit (Q_{t2}) that will serve as the dam foundation.

Project specifications indicate that the Type 1 material will be classified in accordance with ASTM D 2487 as SM, SC, CL, or ML from Contractor-furnished borrow areas, Type 2 material will be ASTM C-33 Fine Aggregate, Type 3 material will be CalTrans Class 1 Type A material.

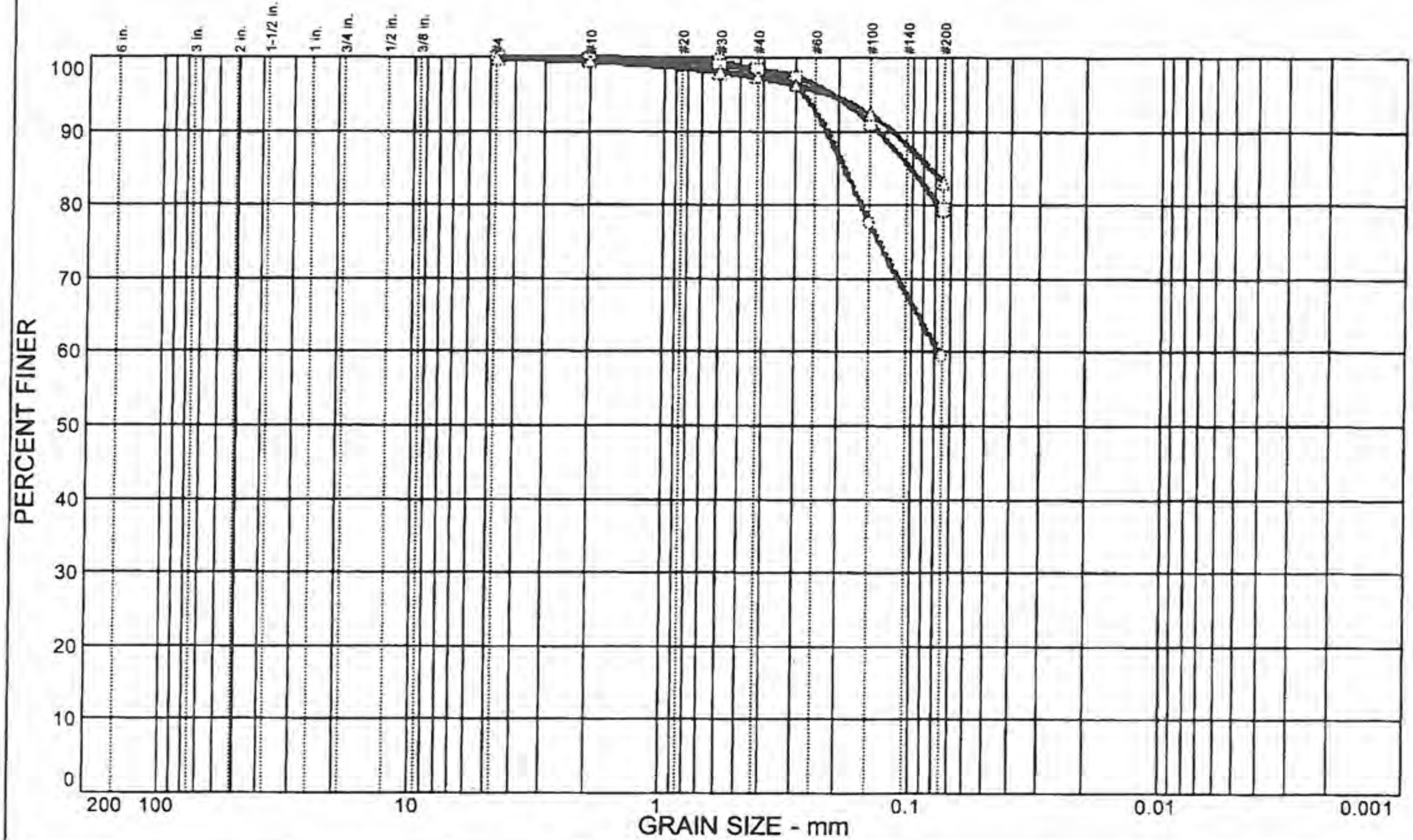
We reviewed test pit logs and grain size analysis test results from samples taken from the proposed borrow and dam foundation areas to develop a typical grain size curve for the Type 1 fill and foundation materials.

Detailed filter compatibility calculations are attached.

Attachments:

Grain Size Test Results for Borrow Area and Foundation Samples
Filter Compatibility Calculations – Type 1 vs. Type 2
Filter Compatibility Calculations – Type 2 vs. Type 3

Particle Size Distribution Report



	% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY	USCS	AASHTO	PL	LL
○			40.6	59.4		CL		17.2	39.1
□			20.6	79.4		CL		17.4	37.3
△			16.9	83.1		CL		17.8	38.5

SIEVE inches size	PERCENT FINER ○	□	△

	GRAIN SIZE		
D_{60}	0.0769		
D_{30}			
D_{10}			

	COEFFICIENTS		
C_c			
C_u			

SIEVE number size	PERCENT FINER ○	□	△
#4	100.0	100.0	100.0
#10	99.9	99.8	99.5
#30	99.6	99.0	98.1
#40	98.2	98.4	97.4
#50	96.2	97.4	96.3
#100	77.6	91.1	92.4
#200	59.4	79.4	83.1

SOIL DESCRIPTION

○ Brown Mottled Gray Sandy Lean CLAY

□ Brown Lean CLAY w/ Sand

△ Brown Lean CLAY w/ Sand

REMARKS:

○

□

△

○ Source: B10

Elev./Depth: 19.0(Tip)

□ Source: Test Pits 1,2,4

△ Source: Test Pits 9,10,11

COOPER TESTING LABORATORY

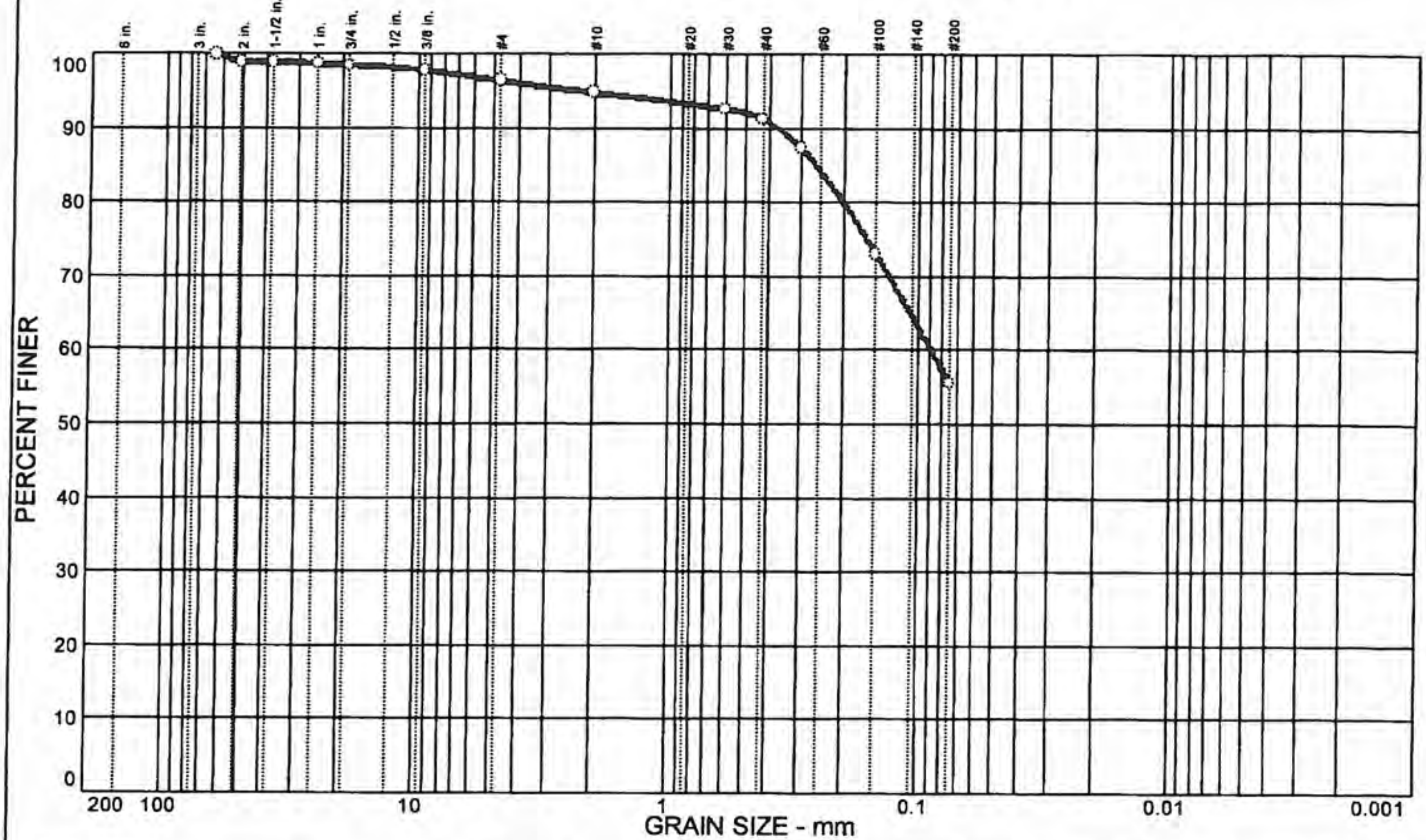
Client: Cal Engineering & Geology

Project: Upper Sand Creek Basin - 081070

Project No.: 471-051

Figure

Particle Size Distribution Report



	% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY	USCS	AASHTO	PL	LL
○		3.6	40.8	55.6		CL		18.7	36.3

SIEVE inches size	PERCENT FINER ○		
2.5	100.0		
2	98.9		
1.5"	98.9		
1"	98.7		
3/4"	98.5		
3/8"	97.8		
	GRAIN SIZE		
D_{60}	0.0890		
D_{30}			
D_{10}			
	COEFFICIENTS		
C_c			
C_u			

○ Source: Test Pits 20,12,26

SIEVE number size	PERCENT FINER ○		
#4	96.4		
#10	94.8		
#30	92.6		
#40	91.3		
#50	87.4		
#100	73.1		
#200	55.6		

SOIL DESCRIPTION

○ Brown Sandy Lean CLAY

REMARKS:

○

COOPER TESTING LABORATORY

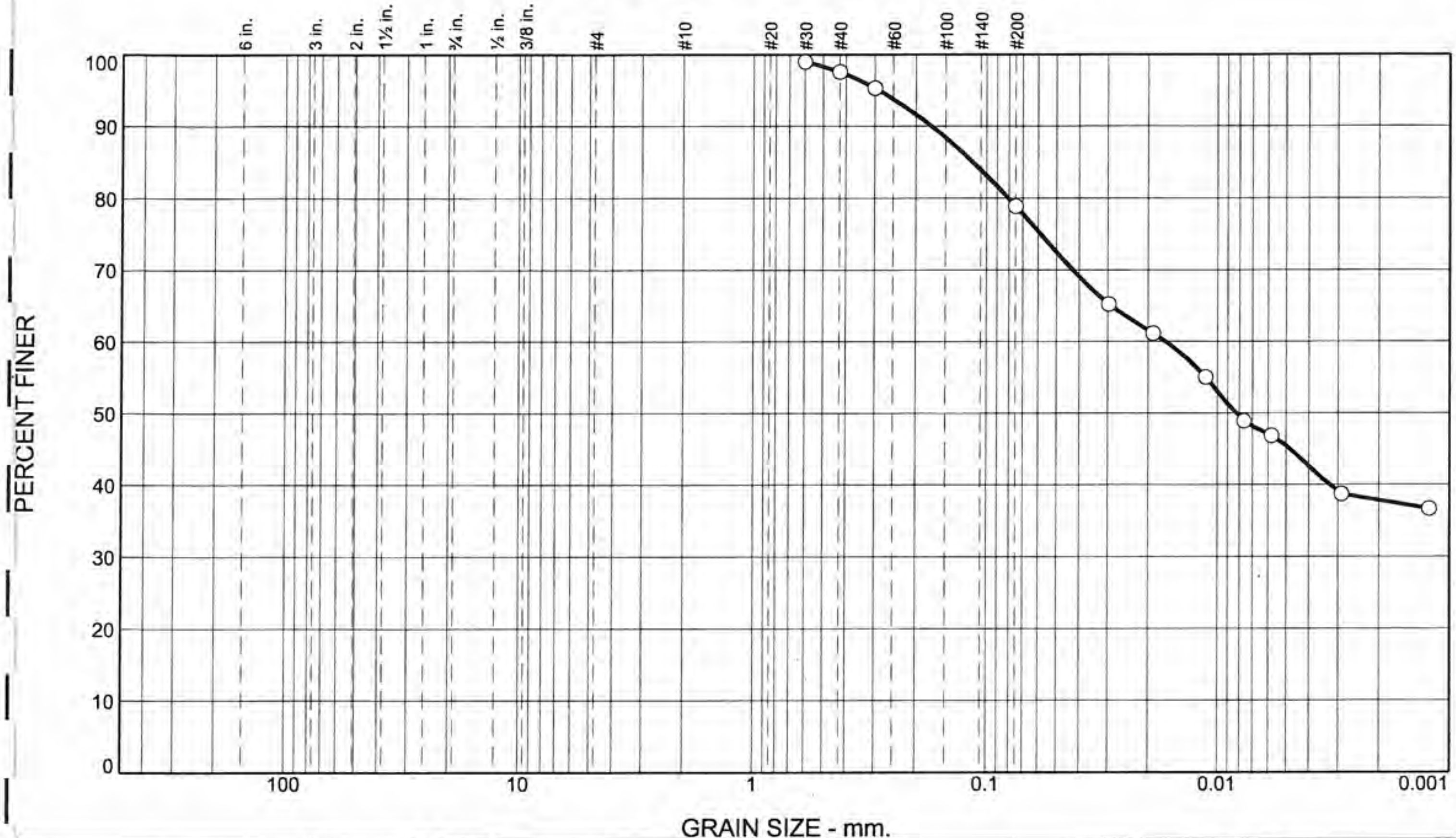
Client: Cal Engineering & Geology

Project: Upper Sand Creek Basin - 081070

Project No.: 471-051

Figure

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
					18.7	33.4	45.5

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#30	99.0		
#40	97.6		
#50	95.3		
#200	78.9		
0.0297 mm.	65.2		
0.0191 mm.	61.2		
0.0113 mm.	55.0		
0.0076 mm.	48.9		
0.0058 mm.	46.9		
0.0029 mm.	38.7		
0.0012 mm.	36.7		

* (no specification provided)

Soil Description

Yellowish Brown Lean Clay w/ Sand

Atterberg Limits

PL= 20 LL= 44 PI= 24

Coefficients

D_{85}= 0.1143 D_{60}= 0.0168 D_{50}= 0.0083
D_{30}= D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= CL AASHTO=

Remarks

Testing Completed on 4/1/10

Sample No.: 3-2 **Source of Sample:** B3 **Date:** 4/1/10
Location: **Elev./Depth:** 4.0'

Cal Engineering & Geology

Walnut Creek, CA

Client: GEI
Project: Upper Sand Creek Basin
Project No: 081070 **Figure**

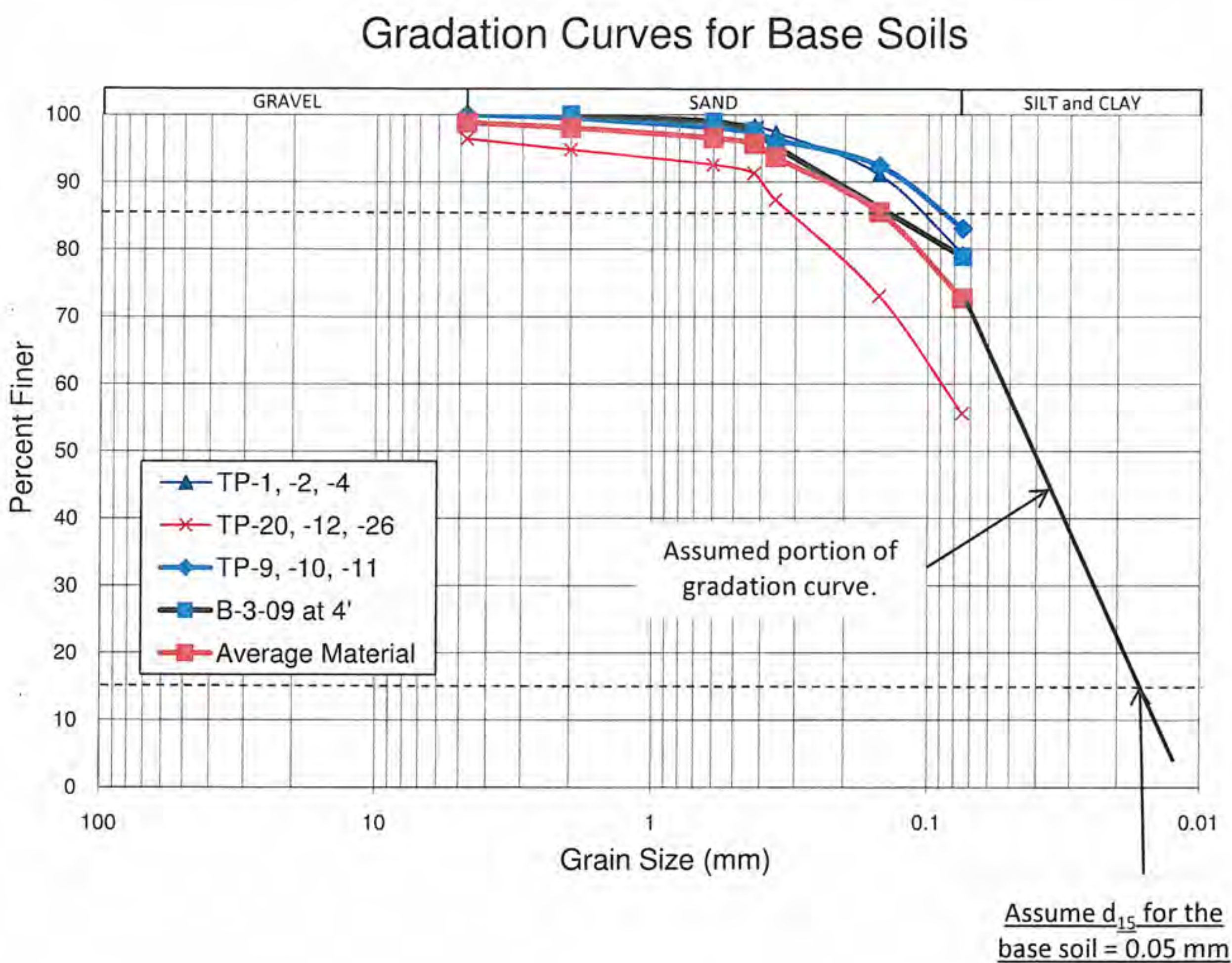
Tested By: TK

Filter Compatibility Calculations

Embankment Clay (Type 1) vs. ASTM C-33 Fine Aggregate (Type 2)

Step 1: Plot the gradation curve of the base soil material.

Sieve No.	Sieve Size	Lab Test Data: TP-1,-2, -4	TP-9,-10, -11	TP-20,-12, -26	B-3-09	Average Material
#4	4.75	100	100	96.4		99
#10	2	99.8	99.5	94.8	100	98
#30	0.6	99	98.1	92.6	99	97
#40	0.425	98.4	97.4	91.3	97.6	96
#50	0.355	97.4	96.3	87.4	95.3	94
#100	0.15	91.1	92.4	73.1		86
#200	0.075	79.4	83.1	55.6	78.9	73



Filter Compatibility Calculations

Step 2: Determine gravel content of the base soil.

Sample	% Gravel	% Sand	% Fines
TP-1, -2, -4	0	20.6	79.4
TP-9, -10, -11	0	16.9	83.1
TP-20, -12, -26	3.6	40.8	55.6
B-3-09	0	21.1	78.9
Average Material	1	25	74

Based on results of the grain-size tests, there is little to no gravel in the proposed Type 1 fill material. Project specifications require that Type 1 fill material contains no gravel. Therefore we will skip step 3 (gravel adjustment) and proceed to step 4.

Step 4: Place the base soil in a category according to Table 26-1.

Table 26–1 Regraded gradation curve data

Base soil category	% finer than No. 200 sieve (0.075 mm) (after regrading, where applicable)	Base soil description
1	> 85	Fine silt and clays
2	40 – 85	Sands, silts, clays, and silty & clayey sands
3	15 – 39	Silty & clayey sands and gravel
4	< 15	Sands and gravel

Based on Table 26-1, the base soil fits into Base Soil Category 2 - Sands, silts, clays, and silty & clayey sands.

Filter Compatibility Calculations

Step 5: Determine the maximum allowable D_{15} in accordance with Table 26-2.

Table 26-2 Filtering criteria — Maximum D_{15}

Base soil category	Filtering criteria
~~1~~	~~$\leq 9 \times d_{85}$ but not less than 0.2 mm~~
2	≤ 0.7 mm
3	$\leq \left(\dfrac{40-A}{40-15}\right)\left[(4 \times d_{85}) - 0.7\text{mm}\right] + 0.7\text{mm}$ A = % passing #200 sieve after regrading (If 4 x d_{85} is less than 0.7 mm, use 0.7 mm)
4	$\leq 4 \times d_{85}$ of base soil after regrading

Based on Table 26-2, the maximum D_{15} particle size for Base Soil Category 2 is <= 0.7mm.

Step 6: Determine the minimum allowable D_{15} in accordance with Table 26-3.

Table 26-3 Permeability criteria

Base soil category	Minimum D_{15}
All categories	$\geq 4 \times d_{15}$ of the base soil before regrading, but not less than 0.1 mm

Based on Table 26-3, and assuming d_{15} of 0.05 for the base soil, the minimum D_{15} = 0.2mm.

Step 7: The maximum and minimum D_{15} sizes for the filter determined in Steps 5 and 6 should be kept to 5 or less at any given percentage passing of 60 or less.

Max D_{15} = 0.70 mm
Min D_{15} = 0.20 mm
Ratio = 3.5 Checks

Step 8: The limits of the design filter should be adjusted so that they have a coefficient of uniformity of 6 or less. The width of the filter band should be kept to 5 or less at any given percentage passing of 60 or less.

We will apply these rules in step 11 below.

Step 9: Determine the minimum D_5 and maximum D_{100} sizes of the filter according to Table 26-5.

Table 26-5 Maximum and minimum particle size criteria[a]

Base soil category	Maximum D_{100}	Minimum D_5, mm
All categories	≤ 3 inches (75 mm)	0.075 mm (No. 200 sieve)

Based on Table 26-5, the Max D_{100} is 3 inches, and the Min D_5 is 0.075mm (No. 200).

Filter Compatibility Calculations

Step 10: To minimize segregation during construction, calculate a preliminary minimum D_{10} size by dividing the minimum D_{15} size by 1.2. Determine the maximum D_{90} using Table 26-6.

Min D_{15} = 0.20 mm

Min D_{10} = 0.17 mm

Table 26-6 Segregation criteria

Base soil category	If D_{10} is :	Then maximum D_{90} is:
	(mm)	(mm)
All categories	< 0.5	20
	0.5 – 1.0	25
	1.0 – 2.0	30
	2.0 – 5.0	40
	5.0 – 10	50
	> 10	60

Based on Table 26-6, the Max D_{90} is 20mm.

Step 11: Plot control points calculated above and confirm that ASTM C33 sand falls within the plotted range of acceptable filter materials.

Min Filter			Max Filter			
Control	% Finer	GS (mm)	Control	% Finer	GS (mm)	Width Ratio
D_{100}	100	4.0	**D_{100}**	**100**	**75**	--
D_{90}	90	1.2	**D_{90}**	**90**	**20**	--
D_{60}	60	0.48	D_{60}	60	2.4	5.0
D_{15}	**15**	**0.20**	**D_{15}**	**15**	**0.7**	3.5
D_{10}	**10**	**0.17**	D_{10}	10	0.58	3.5
D_5	**5**	**0.075**	D_5	5	0.375	5.0

• The numbers in bold in the table above are the control points that are calculated in Steps 1 through 10. The rest of the control points are calculated using the the rules in Steps 7[(2)] and 8[(3)]

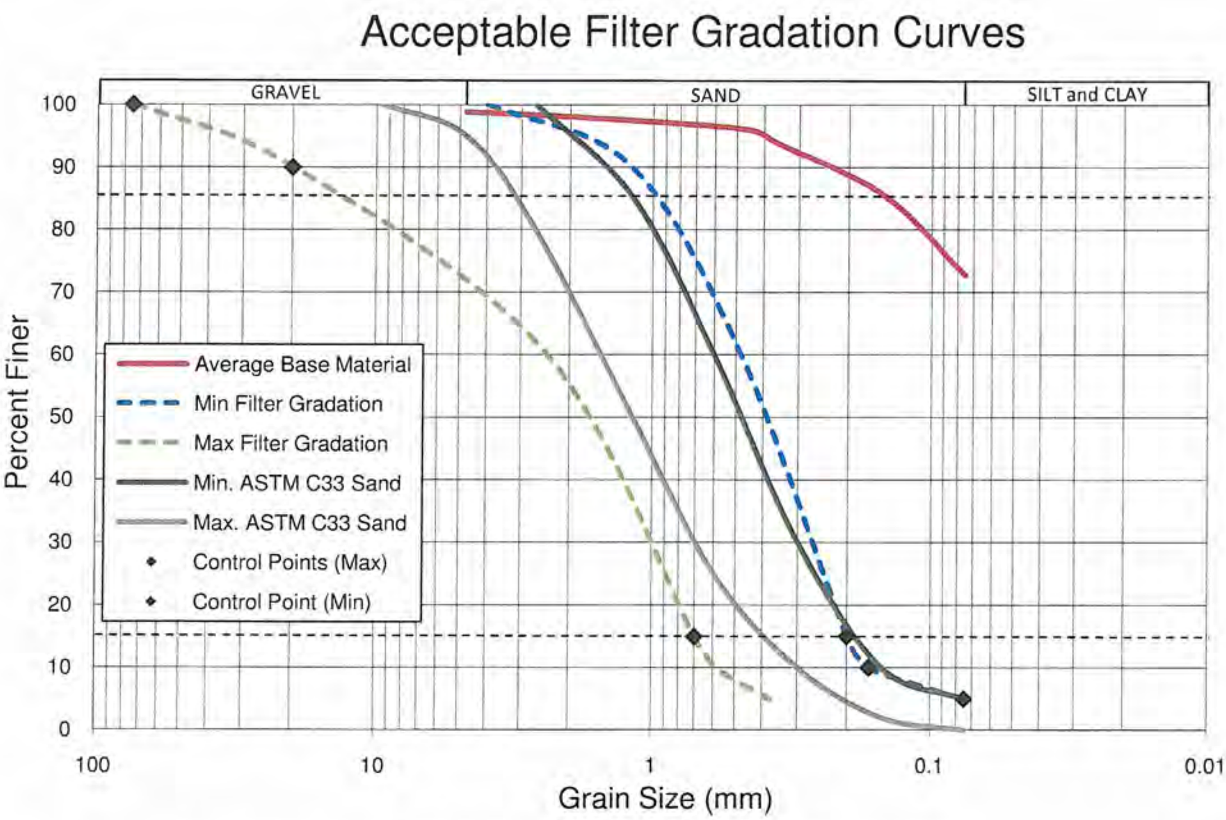
- Cu for Min Filter = 2.9
- Cu for Max Filter = 4.1

Notes:

(1) C_u = coefficient of uniformity = D_{60} / D_{10}

(2) Design criteria states that the width ratio should be less than or equal to 5 (Step 7).

(3) Design criteria states that the coefficient of uniformity should be less than or equal to 6 (Step 8).



Note: Gradation for ASTM C-33 Fine Aggregate taken from ASTM Standard C-33:

Table 26B-1 Selected standard aggregate gradations

Fine aggregate—ASTM C-33

ASTM size	Percent finer than sieve no. #200	#100	#50	#30	#16	#8	#4	3/8"
Fine	3-5*	2-10	10-30	25-60	50-85	80-100	95-100	100

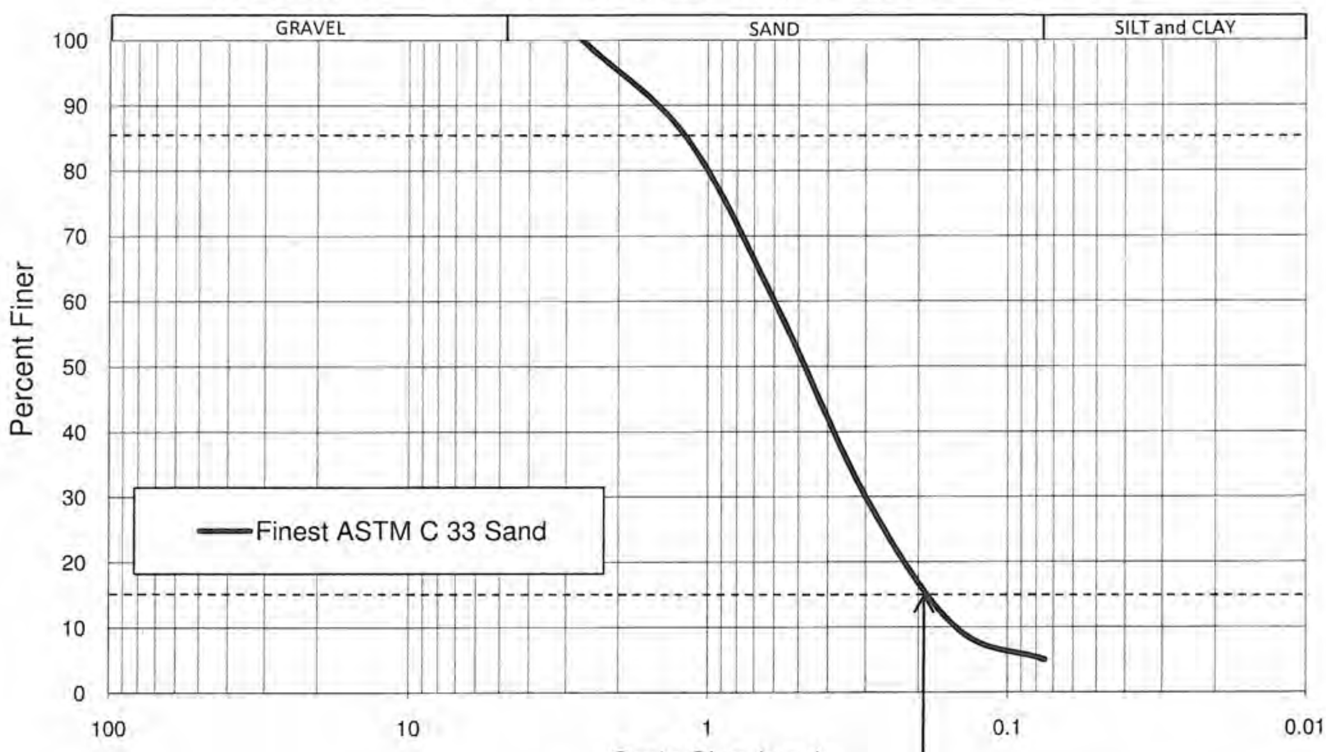
The proposed filter sand (ASTM C-33 Fine Aggregate) is an acceptable filter vs. the base soil (clay embankment fill).

ASTM C-33 Fine Aggregate (Type 2) vs. CalTrans Class 1 Type A (Type 3)

Step 1: Plot the gradation curve of the base soil material.

Sieve No.	Sieve Size	ASTM C-33 Sand
3/8"	9.525	--
#4	4.75	--
#8	2.66	100
#16	1.18	85
#30	0.6	60
#50	0.3	30
#100	0.15	10
#200	0.075	5

Gradation Curve for Base Soil (ASTM C-33 Sand)



Assume d_{15} for the base soil = 0.2 mm

Step 2: Determine gravel content of the base soil.

Assume the filter sand will contain no gravel. Therefore we will skip step 3 (gravel adjustment) and proceed to step 4.

Step 4: Place the base soil in a category according to Table 26-1.

Table 26-1 Regraded gradation curve data

Base soil category	% finer than No. 200 sieve (0.075 mm) (after regrading, where applicable)	Base soil description
1	> 85	Fine silt and clays
2	40 – 85	Sands, silts, clays, and silty & clayey sands
3	15 – 39	Silty & clayey sands and gravel
4	< 15	Sands and gravel

Based on Table 26-1, the base soil fits into Base Soil Category 4 - Sands and gravel.

Step 5: Determine the maximum allowable D_{15} in accordance with Table 26-2.

Table 26-2 Filtering criteria — Maximum D_{15}

Base soil category	Filtering criteria
1	$\leq 9 \times d_{85}$ but not less than 0.2 mm
2	≤ 0.7 mm
3	$\leq \left(\frac{40-A}{40-15}\right)\left[\left(4 \times d_{85}\right) - 0.7\text{mm}\right] + 0.7\text{mm}$
	A = % passing #200 sieve after regrading (If 4 x d_{85} is less than 0.7 mm, use 0.7 mm)
4	$\leq 4 \times d_{85}$ of base soil after regrading

Based on Table 26-2, and assuming d85 = 1.3mm, the maximum D_{15} particle size for Base Soil Category 4 is <= 5.2mm.

Step 6: Determine the minimum allowable D_{15} in accordance with Table 26-3.

Table 26-3 Permeability criteria

Base soil category	Minimum D_{15}
All categories	$\geq 4 \times d_{15}$ of the base soil before regrading, but not less than 0.1 mm

Based on Table 26-3, and assuming d_{15} of 0.2mm for the base soil, the minimum D_{15} = 0.8mm.

Step 7: The ratio of maximum and minimum D_{15} sizes for the filter determined in Steps 5 and 6 should be kept to 5 or less at any given percentage passing of 60 or less.

Max D_{15} =	5.2 mm	
Min D_{15} =	0.8 mm	
Ratio =	6.5	No Good

Adjust max and min D15 to keep the ratio between them <= 5:

Max D_{15} =	5.2 mm
Min D_{15} =	1.0 mm
Ratio =	5.0

Step 8: The limits of the design filter should be adjusted so that they have a coefficient of uniformity of 6 or less. The width of the filter band should be kept to 5 or less at any given percentage passing of 60 or less.

We will apply these rules in step 11 below.

Filter Compatibility Calculations

Step 9: Determine the minimum D_5 and maximum D_{100} sizes of the filter according to Table 26-5.

Table 26-5 Maximum and minimum particle size criteria*

Base soil category	Maximum D_{100}	Minimum D_5, mm
All categories	≤ 3 inches (75 mm)	0.075 mm (No. 200 sieve)

Based on Table 26-5, the Max D_{100} is 3 inches, and the Min D_5 is 0.075mm (No. 200).

Step 10: To minimize segregation during construction, calculate a preliminary minimum D_{10} size by dividing the minimum D_{15} size by 1.2. Determine the maximum D_{90} using Table 26-6.

Min D_{15} = 1.0 mm
Min D_{10} = 0.8 mm

Table 26-6 Segregation criteria

Base soil category	If D_{10} is :	Then maximum D_{90} is:
	(mm)	(mm)
All categories	< 0.5	20
	0.5 – 1.0	25
	1.0 – 2.0	30
	2.0 – 5.0	40
	5.0 – 10	50
	> 10	60

Based on Table 26-6, the Max D_{90} is 30mm.

Step 11: Plot control points calculated above and confirm that ASTM C33 sand falls within the plotted range of acceptable filter materials.

Min Filter			Max Filter			
Control	**% Finer**	**GS (mm)**	**Control**	**% Finer**	**GS (mm)**	**Width Ratio**
D_{100}	100	9.0	**D_{100}**	**100**	**75**	--
D_{90}	90	7.0	**D_{90}**	**90**	**25**	--
D_{60}	60	3.2	D_{60}	60	16.0	5.0
D_{15}	**15**	**1.0**	**D_{15}**	**15**	**5.2**	5.0
D_{10}	**10**	**0.8**	D_{10}	10	4.00	5.0
D_5	**5**	**0.5**	D_5	5	2.5	5.0

• The numbers in bold in the table above are the control points that are calculated in Steps 1 through 10. The rest of the control points are calculated using the the rules in Steps 7(2) and 8(3).

- Cu for Min Filter = 4.0
- Cu for Max Filter = 4.0

Notes:

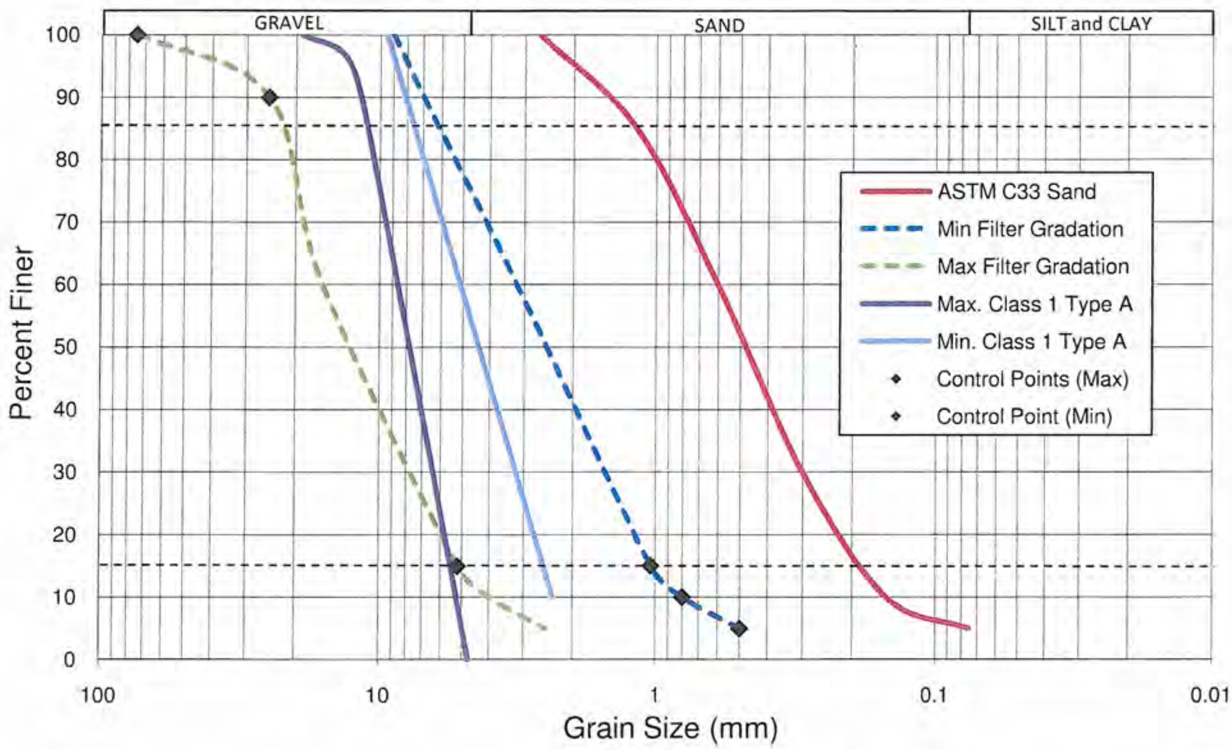
(1) C_u = coefficient of uniformity = D_{60} / D_{10}

(2) Design criteria states that the width ratio should be less than or equal to 5 (Step 7).

(3) Design criteria states that the coefficient of uniformity should be less than or equal to 6 (Step 8).

Filter Compatibility Calculations

Acceptable Filter Gradation Curves



Note: Grain Size Properties for CalTrans Class 1 Type A Aggregate (From Standard Specifications, State of California Department of Transportation, May 2006):

CLASS 1

Sieve Sizes	Percentage Passing Type A	Percentage Passing Type B
2"	—	100
1 1/2"	—	95-100
3/4"	100	50-100
1/2"	95-100	—
3/8"	70-100	15-55
No.-4	0-55	0-25
No.-8	0-10	0-5
No. -200	0-3	0-3

The proposed drain gravel (CalTrans Class 1 Type A Fill) is an acceptable filter vs. the filter sand (ASTM C-33 Sand).

Appendix 2.4

Dam Seepage and Stability Analyses

Seepage and Slope Stability Analysis

Table of Contents:

Figures

Attachments

Seepage and Slope Stability Analysis

Purpose:

The purpose of this document is to present our seepage and slope stability evaluation for the Upper Sand Creek Detention Basin Dam. Seepage analysis was completed in general accordance with the U.S. Army Corps of Engineers' (Corps) Engineer Manual EM 1110-2-1901, Seepage Analysis and Control for Dams. Slope stability analysis was completed for the design cases outlined in the Corps EM 1110-2-1902, Slope Stability and in U.S. Bureau of Reclamation Design Standards No. 13 – Embankment Dams, Chapter 4 – Static Stability Analyses.

Summary:

The design of the proposed Upper Sand Creek Detention Basin Dam incorporates upstream and downstream slopes with a 4 horizontal to 1 vertical (4H:1V) ratio and a 20-foot-wide crest. Seepage and slope stability analyses were completed on representative sections of the embankment dam to verify that this design will provide adequate factors of safety.

We completed seepage and stability calculations at three representative sections. Sections were selected at the highest embankment section and where there was significant variation in the downstream and upstream ground surfaces. The locations of the three representative sections (B, C, and D) are shown on Figure 1.

The seepage analysis was conducted for reservoir level at the overflow spillway crest El. 191.0, and assuming steady state seepage conditions. The results of our seepage analysis are summarized in the following table:

Station	Exit Gradient
14+05	0.18
14+70	0.35
15+55	0.07

Under steady-state seepage conditions, the estimated seepage flow at the toe-drain is approximately 5 gallons per minute (1,000 cubic feet per day).

Results of our stability analysis are provided in the following table:

Stability Case	*Min. Design Value*	Section B Station 14+05	Section C Station 14+70	Section D Station 15+55	See Figures
Case I: End-of-Construction (U) Upstream Slope (D) Downstream Slope	***1.3***	3.0 (U) 2.0 (D)	2.9 (U) 2.2 (D)	3.0 (U) 2.2 (D)	6-1 through 6-3
Case II: Steady State Seepage with Variable Water Level (Upstream Slope)	***1.5***	2.6	2.4	3.0	7-1 through 7-3
Case III: Steady State Seepage with Water Level at Spillway (Downstream Slope)	***1.5***	2.5	2.6	2.6	8-1 through 8-3
Case IV: Pseudo-Static (U) Upstream Slope (D) Downstream Slope	***1.1***	1.7 (U) 1.2 (D)	1.7 (U) 1.3 (D)	1.7 (U) 1.3 (D)	9-1 through 9-6
Case V: Rapid Drawdown	***1.3***	2.1	1.7	1.9	10-1 through 10-3

Our calculations indicate that the proposed dam cross-section with 4H:1V side slopes provides adequate factors of safety for each of the design sections.

Representative Cross Sections

We completed seepage and slope stability analysis at generalized sections at Stations 14+05, 14+70, and 15+55. These stations are shown in plan in Figure 1 as Sections B, C, and D. These stations were selected based on site geology, results of the subsurface explorations, and proposed embankment dam geometry.

Based on the borings drilled and logged in the vicinity of these three stations we created generalized cross sections for our evaluation. These sections are shown in Figures 2, 3, and 4.

While the subsurface conditions vary slightly in each section, they are generalized into the following layers:

Elevation	Generalized Layer
El. 170 to 153	CL
El. 153 to 147	SM-1
El. 147 to 142	ML/CL-1
El. 142 to 135	SM-2
El. 135 to 129	ML/CL-2
Below about El. 129	Bedrock

Seepage Analysis Approach

We evaluated seepage using guidance from EM 1110-2-1901, Seepage Analysis and Control for Dams (USACE, 1901), and ETL 1110-2-569, Design Guidance for Levee Underseepage (USACE, 2005). The analyses are based on the assumption that steady-state seepage conditions could occur during the design flood duration. This is a very conservative assumption because the reservoir is a flood control facility and has no provisions for long-term storage. A key result in a steady-state seepage evaluation is the exit gradient (the estimated gradient at the point of exit). These guidance documents indicate that the maximum exit gradient should be limited to 0.5 for steady-state seepage conditions.

We performed our seepage analyses using SEEP/W, a two-dimensional finite element computer model, developed by GEO-SLOPE International, Ltd. Using SEEP/W we computed steady-state phreatic surfaces and pore pressures within the embankment and foundation soils for the 100-year 12-hour storm. We then estimated exit gradients and uplift gradients near the landside toe of the embankment.

Seepage Analysis Model Development

Hydraulic Conductivities

We selected hydraulic conductivities for each soil type based on published ranges of values for different soil types and available hydraulic conductivity test data. Attachment 1 includes references to documents reviewed, estimated hydraulic conductivity values based on soil grain size, and our selected hydraulic conductivities for analysis.

SEEP/W considers flow of moisture through partially saturated soil above the phreatic surface, as well as flow through saturated soil below the phreatic surface. In SEEP/W, we defined hydraulic conductivity above the phreatic surface as a function of negative pore pressure, or soil suction. A summary of how we estimated these functions is provided in Attachment 1.

Reservoir Water Surface
We considered a reservoir level at El. 191 (spillway crest) resulting from the 100-year 12-hour storm when evaluating seepage through the embankment dam.

Boundary Conditions
We applied a no flow boundary condition to the nodes along the model's bottom.

We applied a constant total head boundary condition to boundary nodes along the upstream horizontal ground surface, upstream vertical edge of the model, upstream slope, and downstream vertical edge of the model. The constant head boundary condition on the upstream was equal to the 100-year 12-hour event level (El. 191 feet). The constant head condition on the downstream vertical edge of the model was set equal to the downstream ground surface elevation.

We applied a potential seepage face review boundary condition to the nodes along the downstream slope and downstream horizontal ground surface. The potential seepage boundary condition evaluates the head at the node after each iteration. If the evaluation shows that water will exit the system at this node, SEEP/W converts the boundary condition to a constant head boundary with the total head equal to the node's corresponding elevation and re-computes the solution.

In the center of the proposed drainage layers we applied a zero-pressure node.

Seepage Analysis Results

Gradients

The exit gradients at all three sections are below the recommended maximum value of 0.5. The results of our seepage analyses are shown on Figures 5-1 through 5-9. The results are summarized in the following table:

Station	Exit Gradient
14+05	0.18
14+70	0.35
15+55	0.07

Flow Quantity

Seepage through the dam is collected in a graded chimney filter. The flows from this chimney filter are collected in the gravel zone of the filter and conveyed to an intercepting base drain at approximately Sta. 15+25. The base drain then runs approximately 160 feet before it daylights. The base drain receives flows from the graded chimney filter, as well as through-seepage not captured by the graded filter.

We evaluated the adequacy of the base drain by estimating flow rates to the drain from both sources. We evaluated the flow at the end of the drain, where the base drain receives the most flow: from the entire length of the graded chimney filter and from the entire length of the base drain. We calculated a flow into the graded filter at each of the three cross sections. Because the base drain runs parallel to the dam for a portion of its length, the base drain is also present in two of the cross sections. Therefore we calculated the flow from throughseepage into the base drain at these two locations as well.

To estimate a flow rate into the base drain, we multiplied the flow rate into the graded filter per unit length along the alignment (calculated for each cross section) by their tributary lengths along the alignment. To calculate the flow rate into the base drain from throughseepage, we multiplied the flow to the base drain via throughseepage per unit length along the base drain by the length of the base drain. The estimated flow rates per unit width are shown on Figs. 5-1 through 5-9. This methodology is reasonably conservative for the following reasons:

1. We assumed steady-state conditions. The design of the basin as an uncontrolled flood control reservoir prevents the reservoir from sustaining prolonged high water levels. Thus the embankment is unlikely to become saturated.
2. The cross sections that we analyzed are at the tallest portions of the dam. In other locations, the height of the dam is lower, which would likely result in less flow to the graded chimney filter. We ignored the reduced flows and assumed that the entire length of the dam would contribute these larger flows to the chimney filter.
3. The contribution of throughseepage to the base drain will likely diminish with increasing distance from the dam. In our calculations we used a flow rate from a cross section where the base drain is relatively close to the dam.

Our estimate of the flow rate in the base drain at its outlet is summarized in the table below.

Section	Station	Tributary Length (feet)	Flow rate per unit length from SEEP/W (ft^3/day/ft)	Flow rate (ft^3/day)
Graded filter contribution				
B	14+05	1437.5 (0+00 to 14+37.5)	0.061	88
C	14+70	75 (14+37.5 to 15+12.5)	0.036	3
D	15+55	307.5 (15+12.5 to 18+20)	1.78	549
			Chimney drain total contribution:	640
Base drain contribution				
-	-	163	2.14	349
			Total flow rate at Outlet:	**989 ft^3/day** (approx. 5 gpm)

We used this total flow rate to estimate the gradient required to maintain this flow rate. We assumed a hydraulic conductivity for the permeable gravel core of the drain of 1 cm/sec (2,835 ft/day) and an area of 30 ft2 (the minimum cross sectional area as specified on the 35% Design drawings. Using Darcy's Law:

$$Q = kiA \rightarrow i = Q/kA = 989 \text{ ft3/day} / (2835 \text{ ft/day} \cdot 30 \text{ ft2}) = 0.01$$

We feel that this gradient is low enough that the base drain can adequately convey the flows.

Slope Stability Approach:

Slope stability was evaluated in general accordance with Corps' Engineer Manual EM 1110-2-1902, *Slope Stability* dated October 31, 2003 and U.S. Bureau of Reclamation Design Standards No. 13 – Embankment Dams, Chapter 4 – Static Stability Analyses.

The USBR Manual recommends slope stability analysis for the following design cases:

- Case I – End of Construction
- Case II – Steady State Seepage (Upstream)
- Case III – Steady State Seepage (Downstream)
- Case IV – Pseudo-Static Seepage
- Case V – Rapid Drawdown

Slope stability for each case was evaluated using the appropriate piezometric conditions and corresponding shear strengths. The approach for each case is summarized in the sections below.

Case I: End-of-Construction

This case represents the condition during and immediately after embankment construction. When differing upstream and downstream foundation strengths or ground surfaces were evident, both the upstream and downstream slopes were evaluated. The groundwater level was assumed to be at El. 160 based on groundwater readings in the area. Shear strengths of the soils were defined using estimated undrained strengths. As specified in USBR 1987b, the required minimum factor of safety for this condition is 1.3.

Case II: Steady State Seepage With Variable Water Level (Upstream Slope)

This case represents the embankment under steady-state seepage conditions resulting from a variable water level in the reservoir. The analysis assumes the water level ranges from the intake level (El. 151) to the spillway crest (El. 191) and the duration of the water level is long enough to establish steady-state seepage conditions through the embankment. A seepage analysis of the embankment and foundation subjected to the design flood was conducted using SEEP/W to estimate the phreatic surface and pore- water pressures. This phreatic surface and pore-water pressures were used in the stability evaluations. Shear strengths of the soils were defined using estimated drained strengths. Stability analysis is evaluated on the upstream slope. As specified in USBR 1987b, the required minimum factor of safety for this condition is 1.5.

Case III: Steady State Seepage With Water Level At Spillway Crest (Downstream Slope)

This case represents the embankment under the design reservoir pool (El. 191). The analysis assumes the duration of the design pool is long enough to establish steady-state seepage conditions through the embankment. A seepage analysis of the embankment and foundation subjected to the design reservoir pool was conducted using SEEP/W to estimate the phreatic surface and pore-water pressures. This phreatic surface and pore-water pressures were used in the stability evaluations. Shear strengths of the soils were defined using estimated drained strengths. Stability analysis is evaluated on the downstream slope. As specified in USBR 1987b, the required minimum factor of safety for this condition is 1.5.

Case IV: Pseudo Static

This case is similar to Cases I but a horizontal acceleration of 0.15 g is applied to the analysis models. Shear strengths of the soils were defined using estimated undrained strengths. As specified in USBR 1987b, the required minimum factor of safety for this condition is 1.1.

Case V: Rapid Drawdown

This case assumes the embankment has been saturated long enough under the design flood to develop steady state seepage conditions, and the flood recedes too quickly for the embankment and foundation pore pressures to dissipate. Shear strengths of the fine-grained foundation soils were defined with undrained strengths, and shear strengths for free-draining foundation soils were modeled using drained strengths. Shear strengths of the embankment soils were defined with a bi-linear strength envelope as outlined in Appendix G, Section G-3 of EM 1110-2-1902. As specified in USBR 1987b, the required minimum factor of safety for this condition is 1.3.

Approach to Estimating Shear Strengths:

The general approach used to evaluate shear strengths for the embankment and foundation soils is summarized below. Specific shear strength information used for analysis is presented in Attachment 2 for the embankment soils, Attachment 3 for foundation undrained strengths, and Attachment 4 for foundation drained strengths.

Embankment Soils

Soils for the proposed embankment dam consist of generally low to moderate plasticity, fine-grained material. Shear strengths for the proposed embankment borrow material were estimated from laboratory Consolidated Undrained (CU) triaxial tests on remolded specimens. Specimens were composited from representative test pit samples from potential borrow areas. Attachment 2 presents our evaluation of embankment shear strengths. Undrained strengths of the embankment soils were estimated using an undrained (Kc=1) failure envelope. This envelope represents the undrained shear strength resulting from the anticipated consolidation pressures. Drained strengths in the embankment soils were estimated based on a drained friction angle and no cohesion.

Undrained Shear Strengths – Foundation Soils

Our approach to evaluating undrained shear strengths in the foundation soils is presented in Attachment 3. In summary, data from laboratory triaxial shear (both CU and UU) tests, cone penetrometer soundings, and Standard Penetration Test blow counts were used to evaluate undrained shear strength profiles.

Coarse-grained foundation soils were assumed to drain reasonably; therefore, the shear strengths for coarse-grained foundation soils were assumed to be the same for undrained and drained conditions.

Drained Shear Strengths – Foundation Soils

Our approach to evaluating drained shear strengths in the foundation soils is summarized in Attachment 4. Drained strengths of the foundation soils were estimated using an empirical correlation to SPT N-values, an empirical correlation to plasticity index, and data from CU tests with pore pressure measurements.

Summary of Evaluation Shear Strengths

Material	Case I End-of-Construction		Case II, III Steady-State Seepage		Case V Rapid Drawdown	
	Cohesion (psf)	ϕ (degrees)	Cohesion (psf)	ϕ (degrees)	Cohesion (psf)	ϕ (degrees)
Embankment – Clay	300	14	0	31	--[1]	--[1]
Embankment – Random Fill	300	14	0	31	--[1]	--[1]
Embankment – Filter Sand	0	35	0	35	0	35
Embankment – Drain Gravel	0	35	0	35	0	35
ML	1000	0	0	28	1000	0
SC	0	32	0	32	0	30
CL	1,600	0	0	30	1,600	0
SM-1	0	32	0	32	0	32
ML/CL-1	1,000	0	0	31	1,000	0
SM-2	0	32	0	32	0	32
ML/CL-2	1,400	0	0	25	1,400	25
Bedrock	2,300	0	0	35	2,300	0

Note:

1. *As outlined in Appendix G, Section G-3 of EM 1110-2-1902, a bilinear strength envelope was used to model the shear strengths of the foundation clay core and random fill. The envelope is defined in Attachment 2 – Embankment Shear Strengths.*

Assessment of Pore Water Pressures

Pore-water pressures in the embankment and foundation soils differ for each slope stability case. Each case is discussed in the following paragraphs.

Case I – End-of-Construction Pore Water Pressures

In the Case 1 End-of-Construction stability evaluation, pore-water pressures in the foundation soils were estimated by assigning a static phreatic surface. Test borings along the alignment and regional groundwater data suggest that the static groundwater level typically ranges in depth from about 10 feet below ground surface during the wet winter months to about 20 feet during summer months. We assigned a phreatic surface in our models at Elevation 160, which is approximately 10 feet below the ground surface.

Case II: Steady State Seepage with Variable Water Level (Upstream Slope)

In the Case II Steady State Seepage with Variable Water Level (Upstream Slope) stability evaluation, pore-water pressures in the embankment and foundation soils are caused by the normal pool water level between the intake level (El. 158) and the spillway crest (El. 191). Based on the geometry of the cross-sections (Figures 2-4), we selected a level of El. 180 in our evaluation. We created a SEEP/W model to estimate pore water pressures within the embankment and foundation soils due to steady-state seepage from the normal pool water level of El. 180.

Case III: Steady State Seepage with Water Level at Spillway Weir (Downstream Slope)

In the Case III Steady State Seepage with Water Level at Spillway (Downstream Slope) stability evaluation, pore-water pressures in the embankment and foundation soils are caused by the design reservoir pool (El. 191). We created a SEEP/W model to estimate pore water pressures within the embankment and foundation soils due to steady-state seepage from the design reservoir pool (El. 191). The results of the SEEP/W evaluation are shown in Figures 5-1 through 5-9.

Case IV – Pseudo Static Pore Water Pressures

In the Pseudo Static Case stability evaluation, pore-water pressures in the foundation soils were estimated by assigning a static phreatic surface. Similar to Case I, we assigned a phreatic surface in our models at Elevation 160, which is approximately 10 feet below the ground surface.

Case V – Rapid Drawdown Pore Water Pressures

For the Case V Rapid Drawdown condition, we assumed that the dam has been saturated long enough under the design reservoir pool (El. 191) to develop steady-state seepage conditions and then the flood recedes quickly. We modeled the pore water pressures with a phreatic surface consisting of the post-drawdown water surface elevation upstream of the dam embankment, and the phreatic surface developed in the steady-state seepage analysis (inside and downstream of the dam embankment). We assumed that the post-drawdown water surface elevation on the waterside of the dam embankment follows the ground surface up to the flood water surface elevation.

Stability Analysis Results

Dam slope stability analyses were performed using SLOPE/W software by GEO-SLOPE International, Ltd. Stability was evaluated using the Spencer analysis method, which satisfies both moment and force equilibrium. Slip surfaces were defined using the entry and exit method

The results of the Case I – End-of-Construction stability evaluation are presented in Figures 6-1 through 6-6.

The results of the Case II – Steady State Seepage with Variable Water Level (Upstream Slope) Stability Evaluation are presented in Figures 7-1 through 7-3.

The results of the Case III – Steady State Seepage with Water Level at Spillway (Downstream Slope) Stability Evaluation are presented in Figures 8-1 through 8-3.

The results of the Case IV – Pseudo Static End-of-Construction stability evaluation are presented in Figures 9-1 through 9-6.

The results of the Case V – Rapid Drawdown Stability Evaluation are presented in Figures 10-1 through 10-3.

Stability Case	*Min. Design Value*	Section B Station 14+05	Section C Station 14+70	Section D Station 15+55	See Figures
Case I: End-of-Construction (U) Upstream Slope (D) Downstream Slope	***1.3***	3.0 (U) 2.0 (D)	2.9 (U) 2.2 (D)	3.0 (U) 2.2 (D)	6-1 through 6-3
Case II: Steady State Seepage with Variable Water Level (Upstream Slope)	***1.5***	2.6	2.4	3.0	7-1 through 7-3
Case III: Steady State Seepage with Water Level at Spillway (Downstream Slope)	***1.5***	2.5	2.6	2.6	8-1 through 8-3
Case IV: Pseudo-Static (U) Upstream Slope (D) Downstream Slope	***1.1***	1.7 (U) 1.2 (D)	1.7 (U) 1.3 (D)	1.7 (U) 1.3 (D)	9-1 through 9-6
Case V: Rapid Drawdown	***1.3***	2.1	1.7	1.9	10-1 through 10-3

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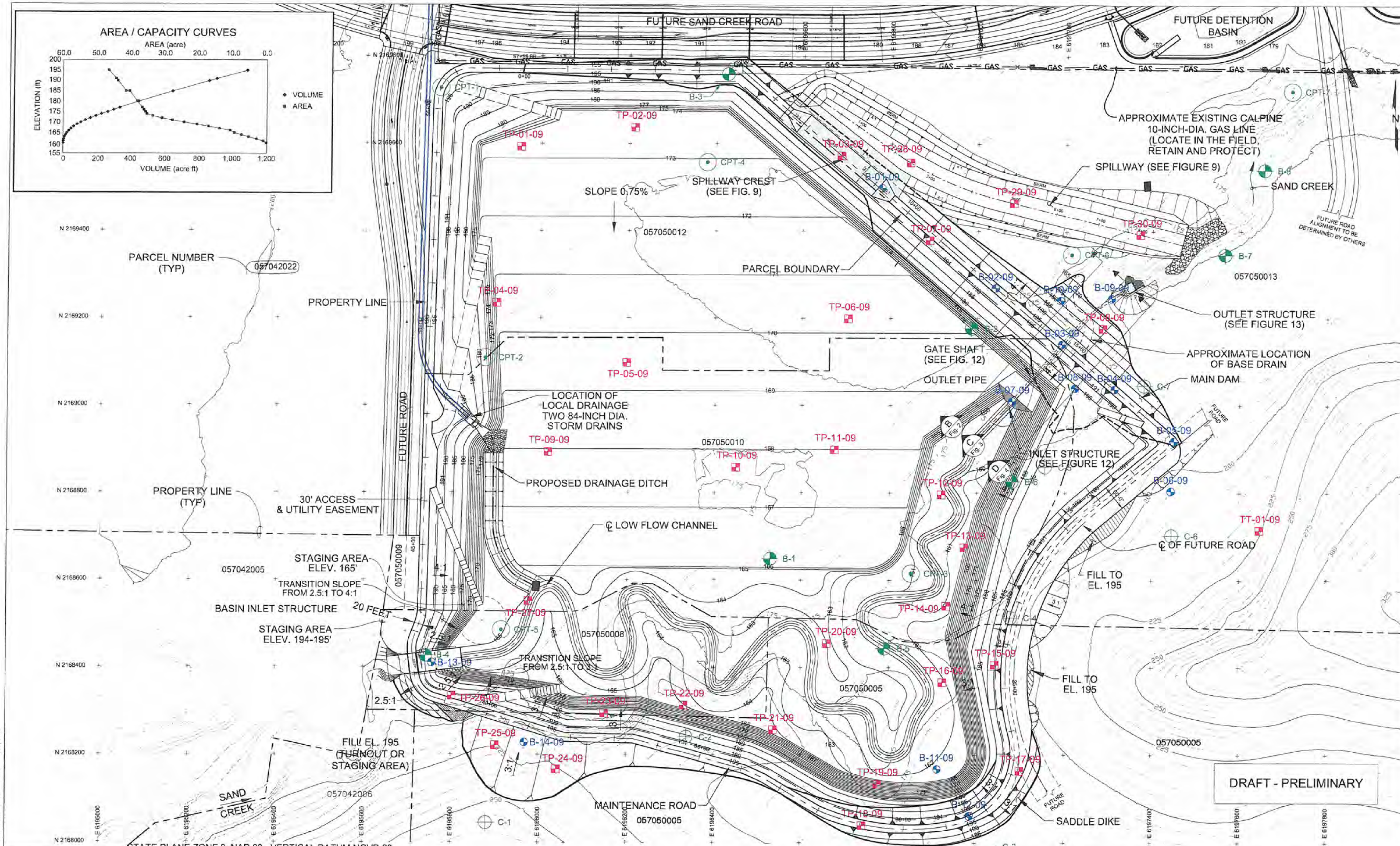


REVISIONS			
NO.	DESCRIPTION	BY	DATE

DES.: K.V.S.
DRAWN: P.Y.M.
CHKD.: A.T.
DATE: 02-18-10
SCALE: 1" = 100'
FLD. BK. XXXX

REGISTERED PROFESSIONAL ENGINEER ALBERTO PUJOL-RIUS No. 37051 EXP. 6/30/10 CIVIL STATE OF CALIFORNIA

PROJECT ENGINEER

PLANS APPROVAL DATE

GEI Consultants

CONTRA COSTA COUNTY PUBLIC WORKS DEPARTMENT 255 GLACIER DRIVE MARTINEZ, CALIFORNIA 94553

0 100 200
SCALE, FEET

FOR REDUCED PLANS ORIGINAL SCALE IS IN INCHES
0 1 2 3

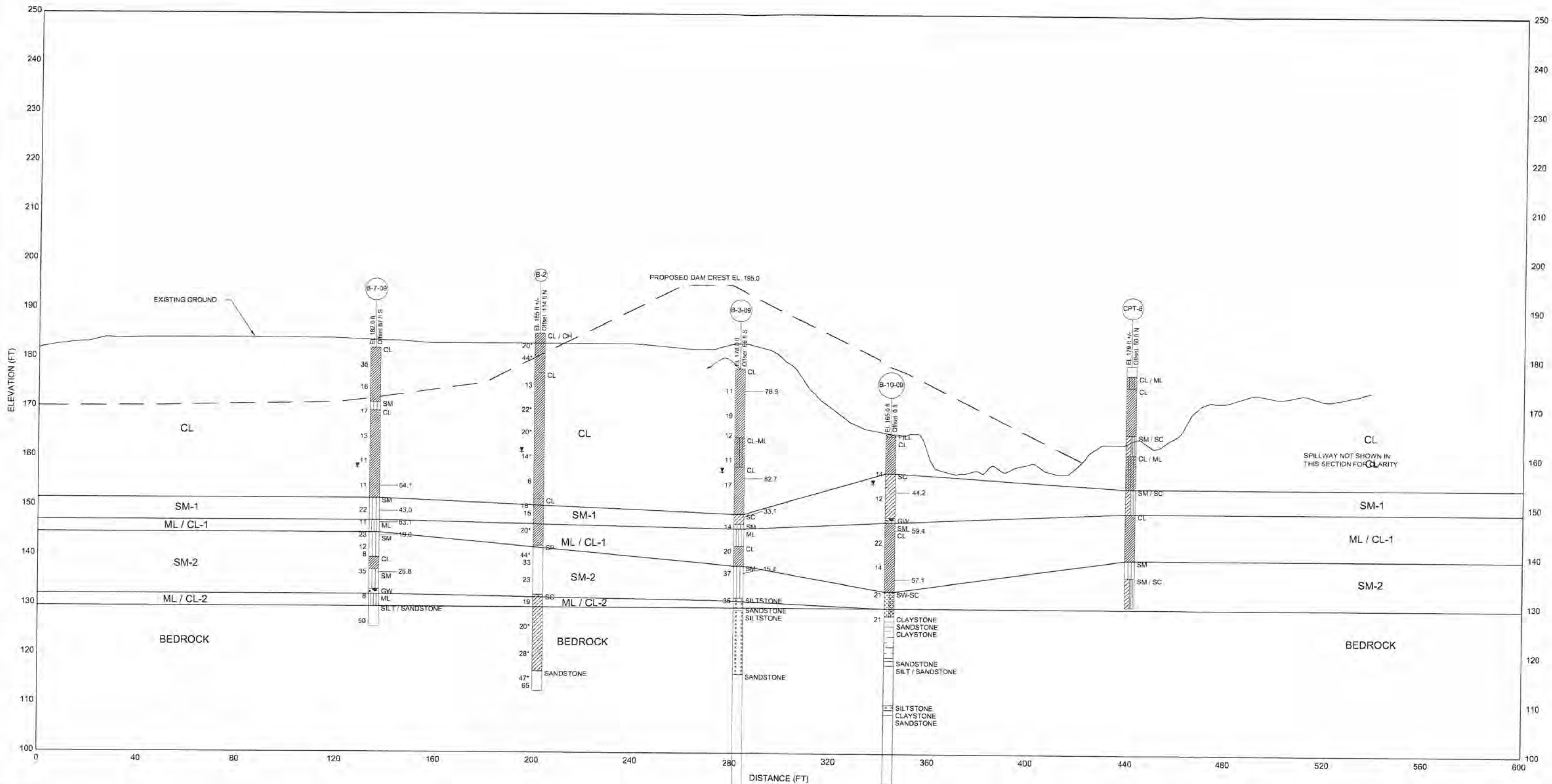
BASE MAP	EAST COORD.	NORTH COORD.
XXX	XXXX	XXX

UPPER SAND CREEK DETENTION BASIN SITE PLAN

FILE NO.

Figure 1

USCB Contours Figure 3 Site 021810 02-18-10 KVS/PYM



SECTION B
STATION 14+05
Fig. 1

NOTES:

1. BLOW COUNTS ON CE&G BORINGS (B-1-09 ETC.) ARE CORRECTED BASED ON TABER DRILLING ENERGY CALIBRATION IN ACCORDANCE WITH DWR AUGUST 2008 MANUAL.
2. BLOW COUNTS ON HISTORIC BORINGS (B-# AND C-#) ARE UNCORRECTED.
3. OFFSET DISTANCES SHOWN FOR EACH BORING REPRESENT THE PERPENDICULAR DISTANCE OF THE BORING TO THE SECTION LINE. OFFSET BORINGS ARE PROJECTED HORIZONTALLY (MAINTAINING ELEVATION).
4. LOGS SHOWN REPRESENT SUMMARIZED AND GENERALIZED SUBSURFACE CONDITIONS. REFER TO BORING LOGS FOR SPECIFIC SUBSURFACE CONDITIONS.
5. SUBSURFACE CONDITIONS SHOWN REPRESENT OBSERVATIONS AT THE SPECIFIC BORING LOCATIONS AT THE TIME THE EXPLORATIONS WERE COMPLETED. SUBSURFACE CONDITIONS BETWEEN BORING LOCATIONS AND SAMPLES ARE UNKNOWN. GROUNDWATER LEVELS ARE LIKELY TO VARY WITH TIME.

LEGEND:

SEE DRAWING S-2.

DRAFT - 35% REVIEW

REVISIONS

NO.	DESCRIPTION	BY	DATE

DES.: DD
DRAWN: DD
CHKD.: JN
DATE: 04-26-10
SCALE: VARIES
FLD. BK. - -

PROFESSIONAL ENGINEER, No. 37051, CIVIL, STATE OF CALIFORNIA

PROJECT ENGINEER

PLANS APPROVAL DATE

GEI Consultants

CONTRA COSTA COUNTY
PUBLIC WORKS DEPARTMENT
255 GLACIER DRIVE
MARTINEZ, CALIFORNIA 94553

0 10 20 VERTICAL SCALE, FEET

0 20 40 HORIZONTAL SCALE, FEET

FOR REDUCED PLANS ORIGINAL SCALE IS IN INCHES
0 1 2 3

BASE MAP	EAST COORD.	NORTH COORD.
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UPPER SAND CREEK DETENTION BASIN

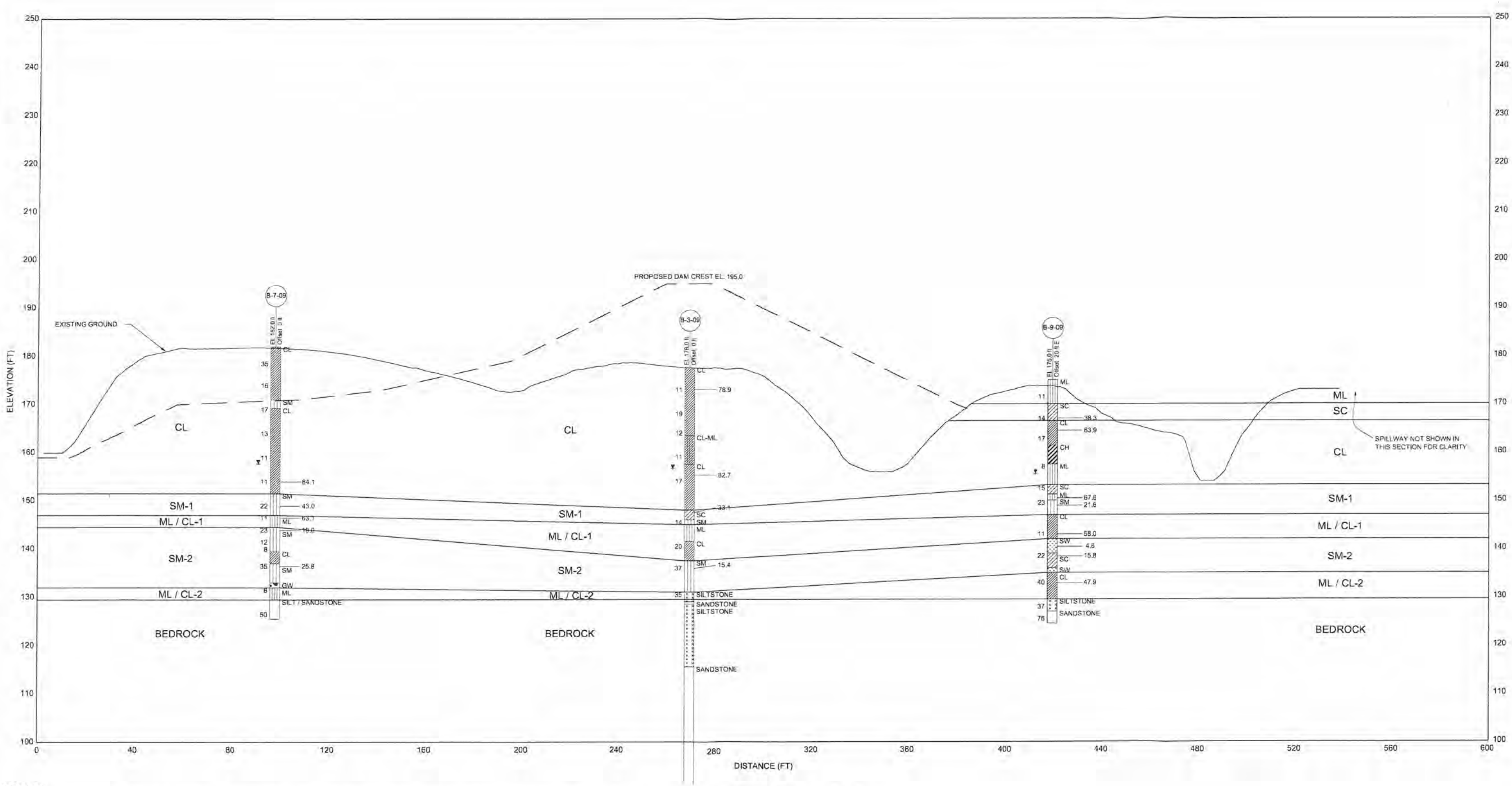
GEOLOGIC SECTION B
Sta. 14+05

FILE NO. —

Figure 2

CADD FILE: S-4.dwg   PEN TBL: GEI Boston Standard 2009.ctb

S-4 04-26-10 D DeCesaris



SECTION C
STATION 14+70
Fig. 1

NOTES:

1. BLOW COUNTS ON CE&G BORINGS (B-1-09 ETC.) ARE CORRECTED BASED ON TABER DRILLING ENERGY CALIBRATION IN ACCORDANCE WITH DWR AUGUST 2008 MANUAL.
2. BLOW COUNTS ON HISTORIC BORINGS (B-# AND C-#) ARE UNCORRECTED.
3. OFFSET DISTANCES SHOWN FOR EACH BORING REPRESENT THE PERPENDICULAR DISTANCE OF THE BORING TO THE SECTION LINE. OFFSET BORINGS ARE PROJECTED HORIZONTALLY (MAINTAINING ELEVATION).
4. LOGS SHOWN REPRESENT SUMMARIZED AND GENERALIZED SUBSURFACE CONDITIONS. REFER TO BORING LOGS FOR SPECIFIC SUBSURFACE CONDITIONS.
5. SUBSURFACE CONDITIONS SHOWN REPRESENT OBSERVATIONS AT THE SPECIFIC BORING LOCATIONS AT THE TIME THE EXPLORATIONS WERE COMPLETED. SUBSURFACE CONDITIONS BETWEEN BORING LOCATIONS AND SAMPLES ARE UNKNOWN. GROUNDWATER LEVELS ARE LIKELY TO VARY WITH TIME.

LEGEND:

SEE DRAWING S-2.

DRAFT - 35% REVIEW

REVISIONS			
NO.	DESCRIPTION	BY	DATE

DES.: DD
DRAWN: DD
CHKD.: JN
DATE: 04-26-10
SCALE: VARIES
FLD. BK.: --

REGISTERED PROFESSIONAL ENGINEER ALBERTO PUJOL-RIOS No. 37051 CIVIL STATE OF CALIFORNIA

PROJECT ENGINEER

PLANS APPROVAL DATE

GEI Consultants

CONTRA COSTA COUNTY
PUBLIC WORKS DEPARTMENT
255 GLACIER DRIVE
MARTINEZ, CALIFORNIA 94553

VERTICAL SCALE, FEET: 0 10 20
HORIZONTAL SCALE, FEET: 0 20 40

FOR REDUCED PLANS ORIGINAL SCALE IS IN INCHES: 0 1 2 3

BASE MAP	EAST COORD.	NORTH COORD.
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UPPER SAND CREEK DETENTION BASIN

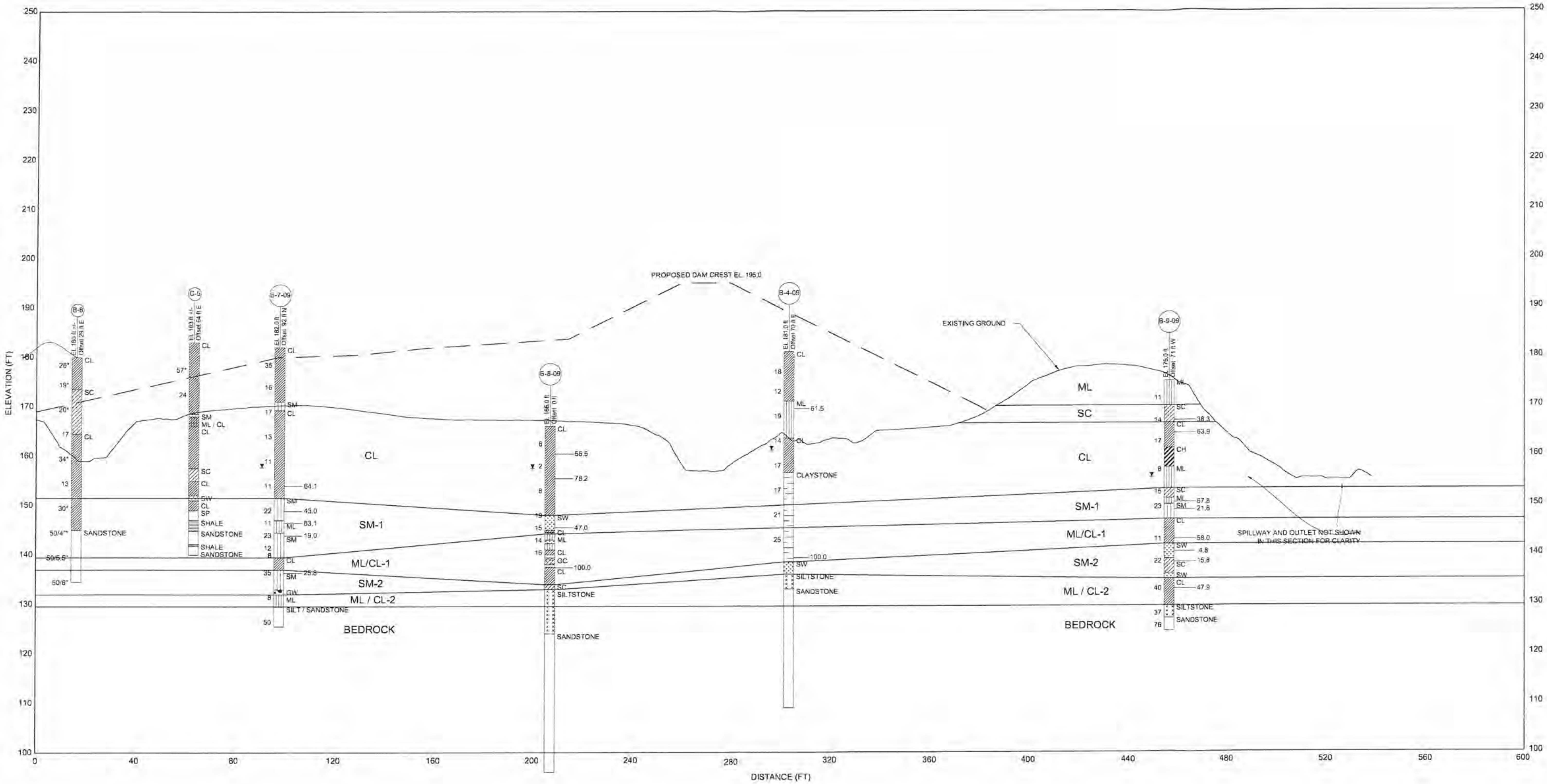
GEOLOGIC SECTION C
Sta. 14+70

FILE NO. --

Figure 3

CADD FILE# S-5.dwg PEN TBL: GEI Boston Standard 2009.ctb

S-5 04-26-10 D DeCesaris



NOTES:

1. BLOW COUNTS ON CE&G BORINGS (B-1-09 ETC.) ARE CORRECTED BASED ON TABER DRILLING ENERGY CALIBRATION IN ACCORDANCE WITH DWR AUGUST 2008 MANUAL.
2. BLOW COUNTS ON HISTORIC BORINGS (B-# AND C-#) ARE UNCORRECTED.
3. OFFSET DISTANCES SHOWN FOR EACH BORING REPRESENT THE PERPENDICULAR DISTANCE OF THE BORING TO THE SECTION LINE. OFFSET BORINGS ARE PROJECTED HORIZONTALLY (MAINTAINING ELEVATION).
4. LOGS SHOWN REPRESENT SUMMARIZED AND GENERALIZED SUBSURFACE CONDITIONS. REFER TO BORING LOGS FOR SPECIFIC SUBSURFACE CONDITIONS.
5. SUBSURFACE CONDITIONS SHOWN REPRESENT OBSERVATIONS AT THE SPECIFIC BORING LOCATIONS AT THE TIME THE EXPLORATIONS WERE COMPLETED. SUBSURFACE CONDITIONS BETWEEN BORING LOCATIONS AND SAMPLES ARE UNKNOWN. GROUNDWATER LEVELS ARE LIKELY TO VARY WITH TIME.

SECTION D
STATION 15+55
Fig. 1

LEGEND:

SEE DRAWING S-2.

DRAFT - 35% REVIEW

REVISIONS					
NO.	DESCRIPTION	BY	DATE	DES.: DD	
				DRAWN: DD	
				CHKD.: JN	
				DATE: 04-26-10	
				SCALE: VARIES	
				FLD. BK. - -	

REGISTERED PROFESSIONAL ENGINEER No. 37051 CIVIL STATE OF CALIFORNIA

PROJECT ENGINEER

PLANS APPROVAL DATE

GEI Consultants

CONTRA COSTA COUNTY
PUBLIC WORKS DEPARTMENT
255 GLACIER DRIVE
MARTINEZ, CALIFORNIA 94553

VERTICAL SCALE, FEET: 0 10 20

HORIZONTAL SCALE, FEET: 0 20 40

FOR REDUCED PLANS ORIGINAL SCALE IS IN INCHES: 0 1 2 3

BASE MAP	EAST COORD.	NORTH COORD.
—	—	—

UPPER SAND CREEK DETENTION BASIN

GEOLOGIC SECTION D
Sta. 15+55

FILE NO. --

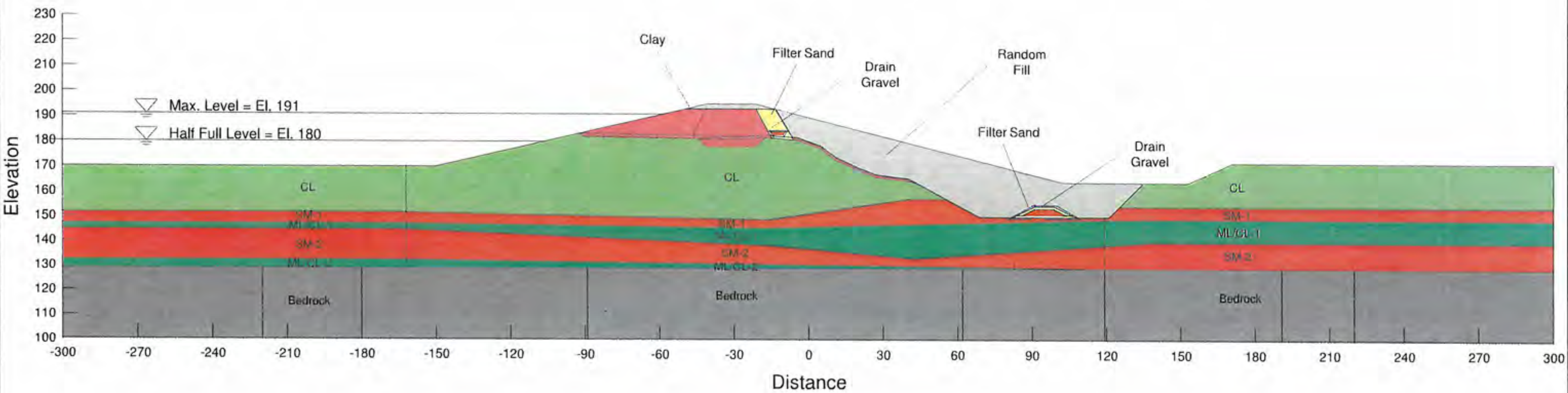
Figure 4

CADD FILE: S-6.dwg PEN TBL: GEI Boston Standard 2009.ctb

S-6 04-26-10 D DeCesare

Steady-State Seepage - Section B
Name: Steady-State Seepage (Downstream)
Method: Steady-State
Last Modified: 6/22/2010 12:29:06 PM

(1) Embankment - Clay, Kh =0.028 ft/day, K Ratio=1
(2) CL, Kh=0.014 ft/day, K Ratio= 0.25
(3) SM-1, Kh=0.28 ft/day, K Ratio= 0.25
(4) ML/CL-1, Kh=0.28 ft/day, K Ratio= 0.25
(5) SM-2, Kh=1.41 ft/day, K Ratio= 0.25
(6) ML/CL-2, Kh=0.28 ft/day, K Ratio= 0.25
(7) Embankment - Filter Sand, Kh=28.3 ft/day, K Ratio= 1
(8) Embankment - Drain Gravel, Kh=2835ft/day, K Ratio= 1
(9) Embankment - Random Fill, Kh=0.0283ft/day, K Ratio= 1
(10) Bedrock, Kh=0.0028ft/day, K Ratio= 1



Notes:
1.Elevations in NAVD88
2.The vertical landside boundary condition is set as a total head equal to the elevation at the landside toe of the embankment (EL 164 ft).
3.Hydraulic conductivities are presented in ft/day.

Upper Sand Creek Detention Basin Antioch, California	GEI Consultants	**Section B** Steady State Seepage Model	
Contra Costa County Public Works Department Martinez, California	Project 08318-0	June 2010	Fig. 5-1

Steady-State Seepage - Section B
Name: Steady-State Seepage (Downstream)
Method: Steady-State
Last Modified: 6/22/2010 11:27:01 AM

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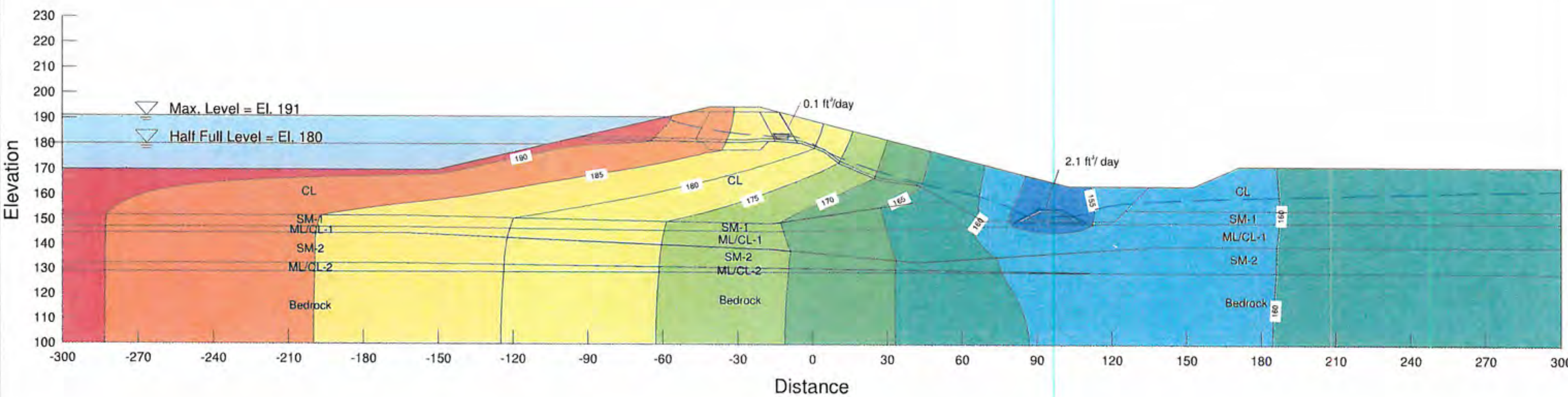
(3) SM-1, Kh=0.28 ft/day, K Ratio= 0.25
(4) ML/CL-1, Kh=0.28 ft/day, K Ratio= 0.25
(5) SM-2, Kh=1.41 ft/day, K Ratio= 0.25
(6) ML/CL-2, Kh=0.28 ft/day, K Ratio= 0.25
(7) Embankment - Filter Sand, Kh=28.3 ft/day, K Ratio= 1
(8) Embankment - Drain Gravel, Kh=2835ft/day, K Ratio= 1
(9) Embankment - Random Fill, Kh=0.0283ft/day, K Ratio= 1
(10) Bedrock, Kh=0.0028ft/day, K Ratio= 1



Notes:
1.Elevations in NAVD88
2.The vertical landside boundary condition is set as a total head equal to the elevation at the landside toe of the embankment (EL 164 ft).
3.Hydraulic conductivities are presented in ft/day.

Upper Sand Creek Detention Basin Antioch, California	GEI Consultants	**Section B** Steady State Seepage Total Head
Contra Costa County Public Works Department Martinez, California	Project 08318-0	June 2010 / Fig. 5-2

Steady-State Seepage - Section B
Name: Steady-State Seepage (Downstream)
Method: Steady-State
Last Modified: 6/22/2010 11:27:01 AM

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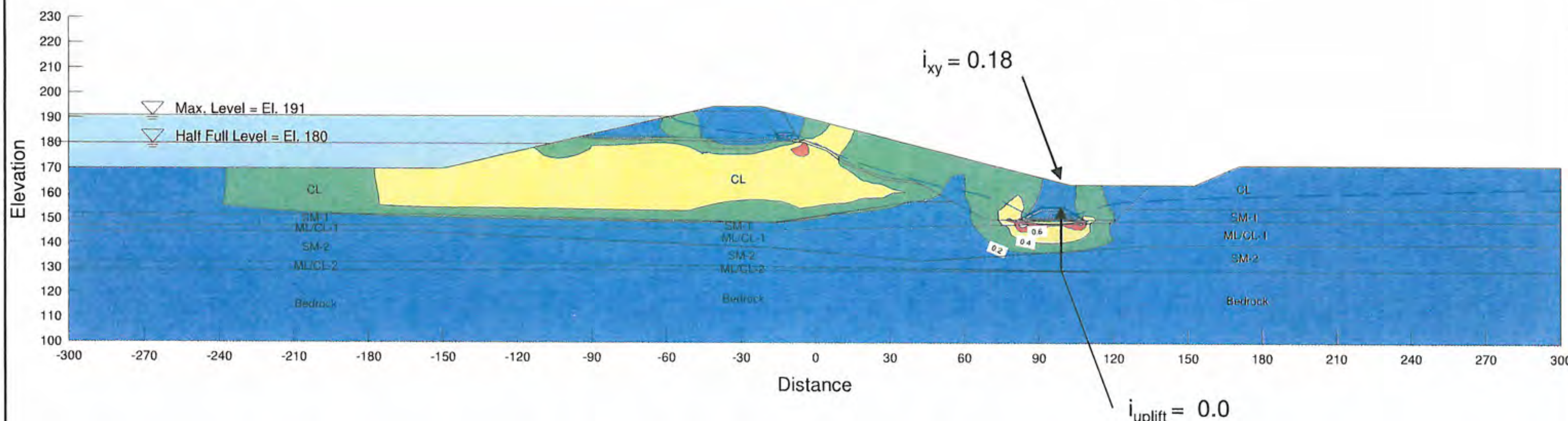
(2) CL, Kh=0.014 ft/day, K Ratio= 0.25
(3) SM-1, Kh=0.28 ft/day, K Ratio= 0.25
(4) ML/CL-1, Kh=0.28 ft/day, K Ratio= 0.25
(5) SM-2, Kh=1.41 ft/day, K Ratio= 0.25
(6) ML/CL-2, Kh=0.28 ft/day, K Ratio= 0.25
(7) Embankment - Filter Sand, Kh=28.3 ft/day, K Ratio= 1
(8) Embankment - Drain Gravel, Kh=2835ft/day, K Ratio= 1
(9) Embankment - Random Fill, Kh=0.0283ft/day, K Ratio= 1
(10) Bedrock, Kh=0.0028ft/day, K Ratio= 1



Notes:
1.Elevations in NAVD88
2.The vertical landside boundary condition is set as a total head equal to the elevation at the landside toe of the embankment (EL 164 ft).
3.Hydraulic conductivities are presented in ft/day.

Upper Sand Creek Detention Basin Antioch, California	GEI Consultants	**Section B** Steady State Seepage XY-Gradient	
Contra Costa County Public Works Department Martinez, California	Project 08318-0	June 2010	Fig. 5-3

Steady-State Seepage - Section C
Name: Steady-State Seepage (Downstream)
Method: Steady-State
Last Modified: 6/22/2010 12:30:36 PM

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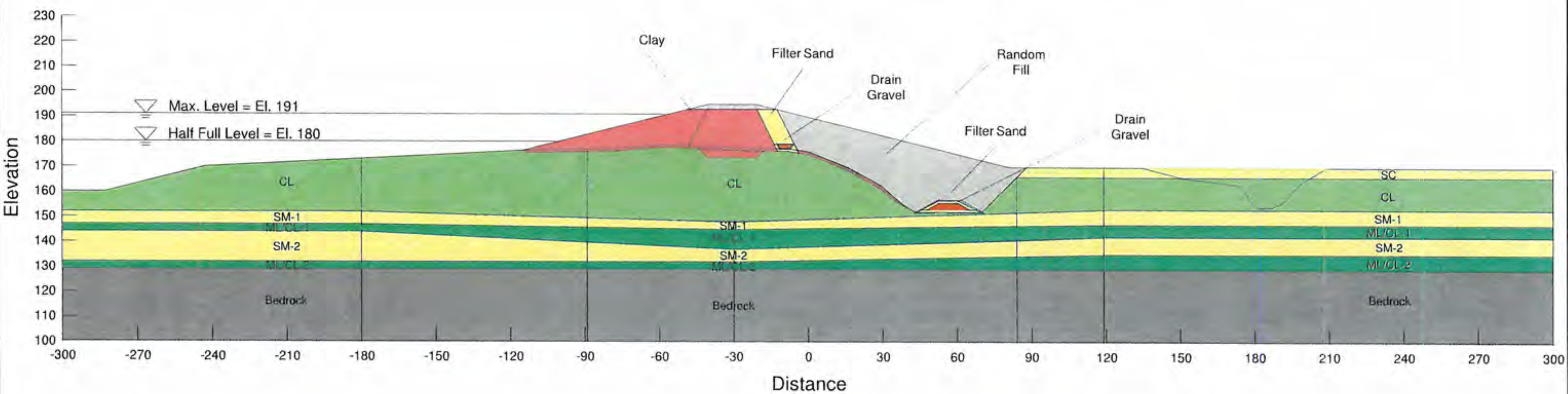
(4) CL, Kh=0.0113 ft/day, K Ratio= 0.25
(5) SM-1, Kh=0.28 ft/day, K Ratio= 0.25
(6) ML/CL-1, Kh=0.28 ft/day, K Ratio= 0.25
(7) SM-2, Kh=1.41 ft/day, K Ratio= 0.25
(8) ML/CL-2, Kh=0.28ft/day, K Ratio= 0.25
(9) Embankment - Filter Sand, Kh=28.3ft/day, K Ratio= 1
(10) Embankment - Drain Gravel, Kh=2835ft/day, K Ratio= 1
(11) Embankment - Random Fill, Kh=0.028ft/day, K Ratio= 1
(12) Bedrock, Kh=0.0028ft/day, K Ratio= 1



Notes:
1.Elevations in NAVD88
2.The vertical landside boundary condition is set as a total head equal to the elevation at the landside toe of the embankment (EL 164 ft).
3.Hydraulic conductivities are presented in ft/day.

Upper Sand Creek Detention Basin Antioch, California	GEI Consultants	**Section C** Steady State Seepage Model
Contra Costa County Public Works Department Martinez, California	Project 08318-0	June 2010 Fig. 5-4

Steady-State Seepage - Section C
Name: Steady-State Seepage (Downstream)
Method: Steady-State
Last Modified: 6/22/2010 12:30:36 PM

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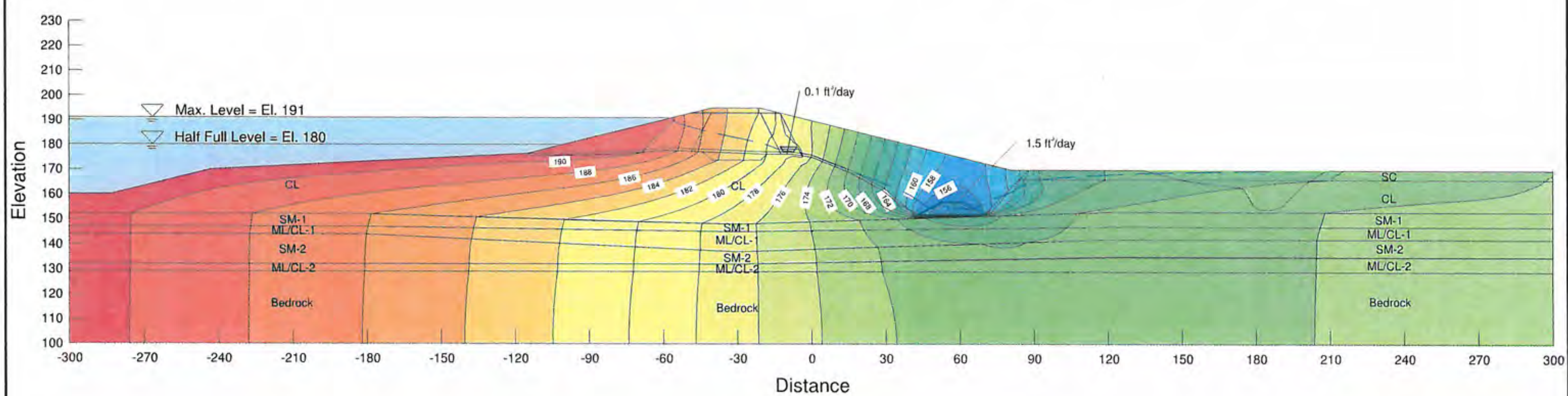
(2) ML, Kh=0.28 ft/day, K Ratio= 0.25
(3) SC, Kh=0.28 ft/day, K Ratio= 0.25
(4) CL, Kh=0.0113 ft/day, K Ratio= 0.25
(5) SM-1, Kh=0.28 ft/day, K Ratio= 0.25
(6) ML/CL-1, Kh=0.28 ft/day, K Ratio= 0.25
(7) SM-2, Kh=1.41 ft/day, K Ratio= 0.25
(8) ML/CL-2, Kh=0.28ft/day, K Ratio= 0.25
(9) Embankment - Filter Sand, Kh=28.3ft/day, K Ratio= 1
(10) Embankment - Drain Gravel, Kh=2835ft/day, K Ratio= 1
(11) Embankment - Random Fill, Kh=0.028ft/day, K Ratio= 1
(12) Bedrock, Kh=0.0028ft/day, K Ratio= 1

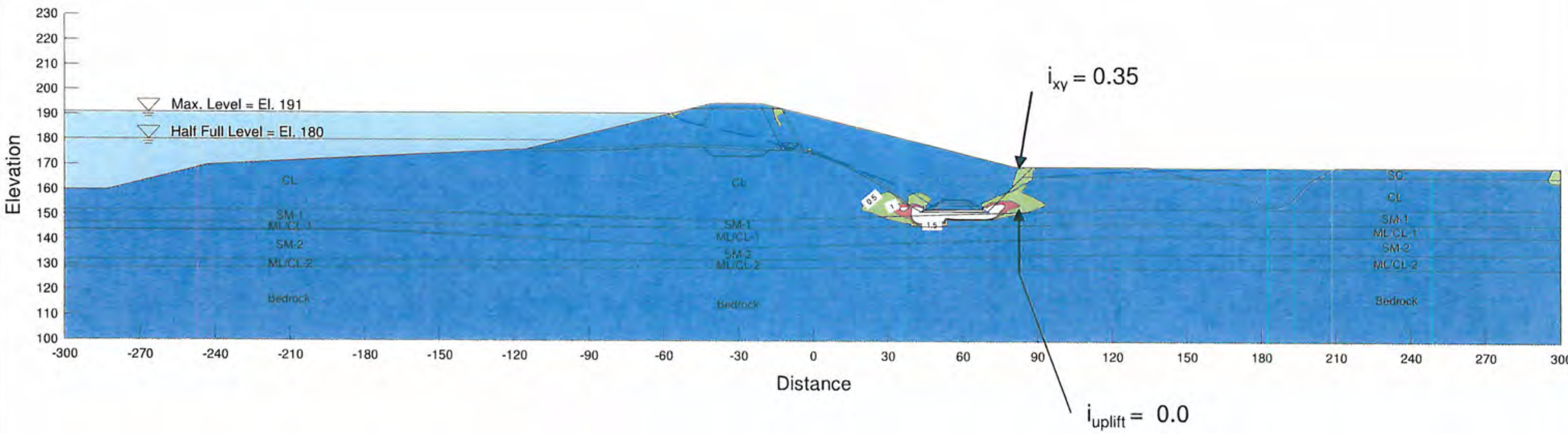


Notes:
1.Elevations in NAVD88
2.The vertical landside boundary condition is set as a total head equal to the elevation at the landside toe of the embankment (EL 164 ft).
3.Hydraulic conductivities are presented in ft/day.

Upper Sand Creek Detention Basin Antioch, California	GEI Consultants	**Section C** Steady State Seepage Total Head
Contra Costa County Public Works Department Martinez, California	Project 08318-0	June 2010 — Fig. 5-5

Steady-State Seepage - Section C
Name: Steady-State Seepage (Downstream)
Method: Steady-State
Last Modified: 6/22/2010 12:30:36 PM

(1) Embankment - Clay, Kh =0.028 ft/day, K Ratio=1
(2) ML, Kh=0.28 ft/day, K Ratio= 0.25
(3) SC, Kh=0.28 ft/day, K Ratio= 0.25
(4) CL, Kh=0.0113 ft/day, K Ratio= 0.25
(5) SM-1, Kh=0.28 ft/day, K Ratio= 0.25
(6) ML/CL-1, Kh=0.28 ft/day, K Ratio= 0.25
(7) SM-2, Kh=1.41 ft/day, K Ratio= 0.25
(8) ML/CL-2, Kh=0.28ft/day, K Ratio= 0.25
(9) Embankment - Filter Sand, Kh=28.3ft/day, K Ratio= 1
(10) Embankment - Drain Gravel, Kh=2835ft/day, K Ratio= 1
(11) Embankment - Random Fill, Kh=0.028ft/day, K Ratio= 1
(12) Bedrock, Kh=0.0028ft/day, K Ratio= 1



Notes:
1.Elevations in NAVD88
2.The vertical landside boundary condition is set as a total head equal to the elevation at the landside toe of the embankment (EL 164 ft).
3.Hydraulic conductivities are presented in ft/day.

Upper Sand Creek Detention Basin Antioch, California Contra Costa County Public Works Department Martinez, California	GEI Consultants Project 08318-0	**Section C** Steady State Seepage XY-Gradient June 2010 Fig. 5-6

Steady-State Seepage - Section D
Name: Steady-State Seepage (Downstream)
Method: Steady-State
Last Modified: 6/22/2010 12:52:38 PM

(1) Embankment - Clay, Kh =0.028 ft/day, K Ratio=1
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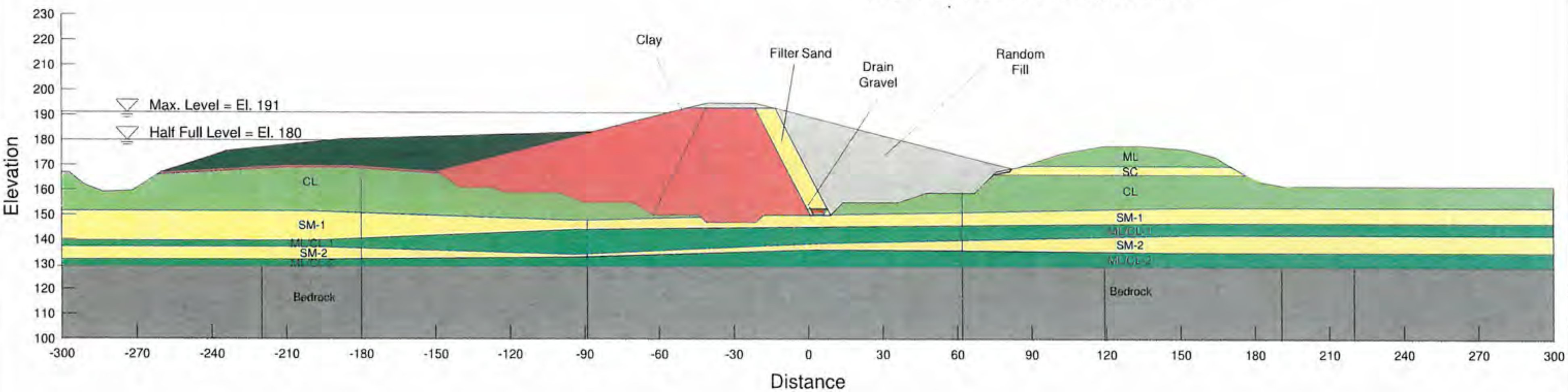
(3) SC, Kh=0.28 ft/day, K Ratio= 0.25
(4) CL, Kh=0.0113 ft/day, K Ratio= 0.25
(5) SM-1, Kh=0.28 ft/day, K Ratio= 0.25
(6) ML/CL-1, Kh=0.28 ft/day, K Ratio= 0.25
(7) SM-2, Kh=1.41 ft/day, K Ratio= 0.25
(8) ML/CL-2, Kh=0.28ft/day, K Ratio= 0.25
(9) Embankment - Filter Sand, Kh=28.3ft/day, K Ratio= 1
(10) Embankment - Drain Gravel, Kh=2835ft/day, K Ratio= 1
(11) Embankment - Random Fill, Kh=0.028ft/day, K Ratio= 1
(12) Basin Fill, Kh=0.028ft/day, K Ratio= 1
(13) Bedrock, Kh=0.0028ft/day, K Ratio= 1

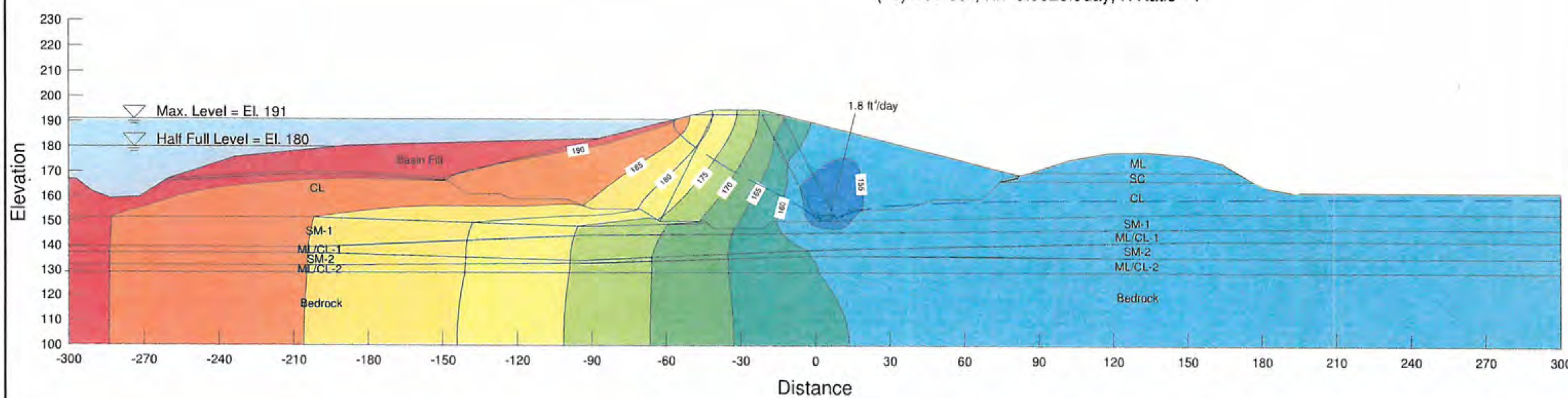


Notes:
1.Elevations in NAVD88
2.The vertical landside boundary condition is set as a total head equal to the elevation at the landside toe of the embankment (EL 164 ft).
3.Hydraulic conductivities are presented in ft/day.

Upper Sand Creek Detention Basin Antioch, California	GEI Consultants	**Section D** Steady State Seepage Model	
Contra Costa County Public Works Department Martinez, California	Project 08318-0	June 2010	Fig. 5-7

Steady-State Seepage - Section D
Name: Steady-State Seepage (Downstream)
Method: Steady-State
Last Modified: 6/23/2010 3:05:09 PM

(1) Embankment - Clay, Kh =0.028 ft/day, K Ratio=1
(2) ML, Kh=0.28 ft/day, K Ratio= 0.25
(3) SC, Kh=0.28 ft/day, K Ratio= 0.25
(4) CL, Kh=0.0113 ft/day, K Ratio= 0.25
(5) SM-1, Kh=0.28 ft/day, K Ratio= 0.25
(6) ML/CL-1, Kh=0.28 ft/day, K Ratio= 0.25
(7) SM-2, Kh=1.41 ft/day, K Ratio= 0.25
(8) ML/CL-2, Kh=0.28ft/day, K Ratio= 0.25
(9) Embankment - Filter Sand, Kh=28.3ft/day, K Ratio= 1
(10) Embankment - Drain Gravel, Kh=2835ft/day, K Ratio= 1
(11) Embankment - Random Fill, Kh=0.028ft/day, K Ratio= 1
(12) Basin Fill, Kh=0.028ft/day, K Ratio= 1
(13) Bedrock, Kh=0.0028ft/day, K Ratio= 1

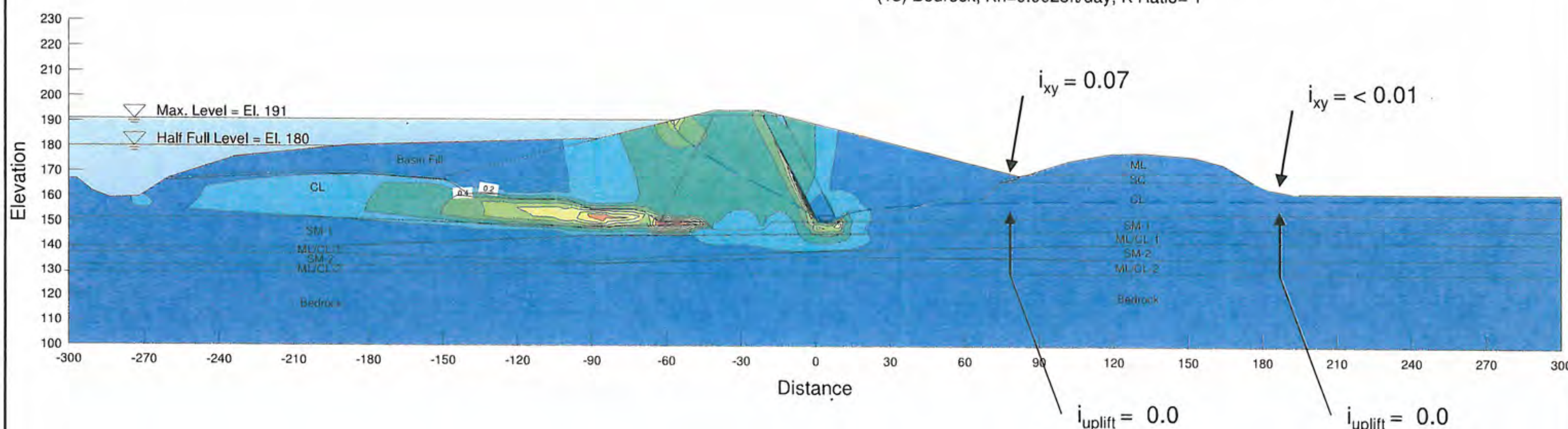


Notes:
1.Elevations in NAVD88
2.The vertical landside boundary condition is set as a total head equal to the elevation at the landside toe of the embankment (EL 164 ft).
3.Hydraulic conductivities are presented in ft/day.

Upper Sand Creek Detention Basin Antioch, California	GEI Consultants	**Section D** Steady State Seepage Total Head
Contra Costa County Public Works Department Martinez, California	Project 08318-0	June 2010 Fig. 5-8

Steady-State Seepage - Section D
Name: Steady-State Seepage (Downstream)
Method: Steady-State
Last Modified: 6/23/2010 3:05:09 PM

(1) Embankment - Clay, Kh =0.028 ft/day, K Ratio=1
(2) ML, Kh=0.28 ft/day, K Ratio= 0.25
(3) SC, Kh=0.28 ft/day, K Ratio= 0.25
(4) CL, Kh=0.0113 ft/day, K Ratio= 0.25
(5) SM-1, Kh=0.28 ft/day, K Ratio= 0.25
(6) ML/CL-1, Kh=0.28 ft/day, K Ratio= 0.25
(7) SM-2, Kh=1.41 ft/day, K Ratio= 0.25
(8) ML/CL-2, Kh=0.28ft/day, K Ratio= 0.25
(9) Embankment - Filter Sand, Kh=28.3ft/day, K Ratio= 1
(10) Embankment - Drain Gravel, Kh=2835ft/day, K Ratio= 1
(11) Embankment - Random Fill, Kh=0.028ft/day, K Ratio= 1
(12) Basin Fill, Kh=0.028ft/day, K Ratio= 1
(13) Bedrock, Kh=0.0028ft/day, K Ratio= 1

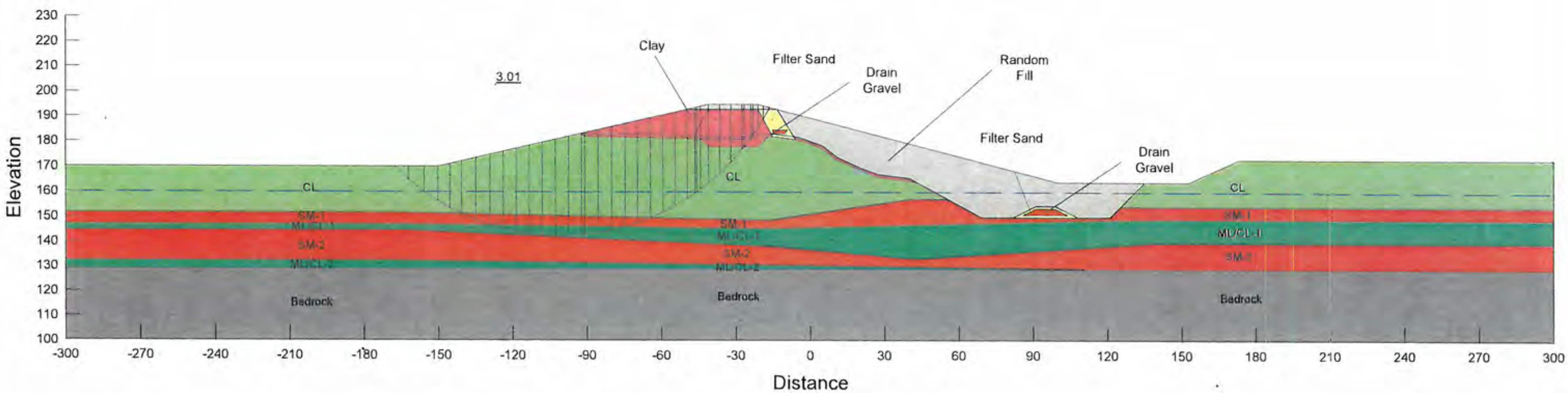


Notes:
1.Elevations in NAVD88
2.The vertical landside boundary condition is set as a total head equal to the elevation at the landside toe of the embankment (EL 164 ft).
3.Hydraulic conductivities are presented in ft/day.

Upper Sand Creek Detention Basin Antioch, California	GEI Consultants	**Section D** Steady State Seepage XY-Gradient	
Contra Costa County Public Works Department Martinez, California	Project 08318-0	June 2010	Fig. 5-9

Upper Sand Creek - Section B
Name: End of Construction - Upstream Slope
Method: Spencer
Last Modified: 6/17/2010 11:02:44 AM

(1) Embankment - Clay: Unit Weight = 125 pcf, phi = 14 °, c = 300 psf
(2) CL: Unit Weight = 120 pcf, phi = 0 °, c = 1600 psf
(3) SM-1: Unit Weight = 120 pcf, phi = 32 °, c = 0 psf
(4) ML/CL-1: Unit Weight = 120 pcf, phi = 0 °, c = 1000 psf
(5) SM-2: Unit Weight = 120 pcf, phi = 32 °, c = 0 psf
(6) ML/CL-2: Unit Weight = 120 pcf, phi = 0 °, c = 1400 psf
(7) Embankment - Filter Sand: Unit Weight = 125 pcf, phi = 35 °, c = 0 psf
(8) Embankment - Drain Gravel: Unit Weight = 125 pcf, phi = 35 °, c = 0 psf
(9) Embankment - Random Fill: Unit Weight = 125 pcf, phi = 14 °, c = 300 psf
(10) Bedrock: Unit Weight = 120 pcf, phi = 0 °, c = 2300 psf

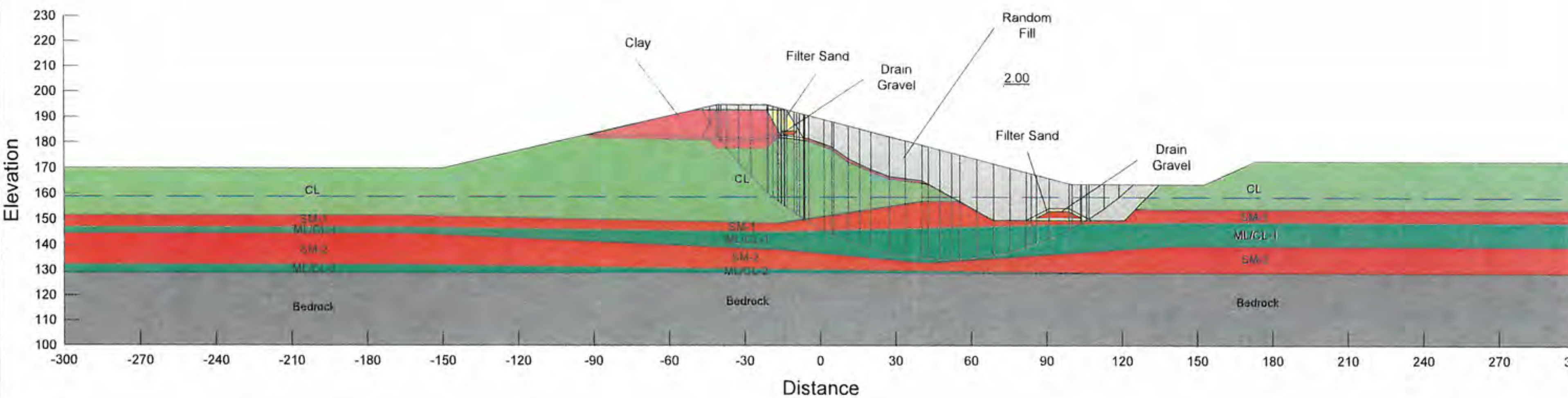


Notes:
1.Elevations in NAVD88
2.The Factor of Safety (FS) value above is for the critical failure surface

Upper Sand Creek Detention Basin Antioch, California	GEI Consultants	**Section B** Case I - End of Construction Upstream Stability
Contra Costa County Public Works Department Martinez, California	Project 08318-0	June 2010 — Fig. 6-1

Upper Sand Creek - Section B
Name: End of Construction - Downstream Slope
Method: Spencer
Last Modified: 6/17/2010 11:02:44 AM

(1) Embankment - Clay: Unit Weight = 125 pcf, phi = 14 °, c = 300 psf
(2) CL: Unit Weight = 120 pcf, phi = 0 °, c = 1600 psf
(3) SM-1: Unit Weight = 120 pcf, phi = 32 °, c = 0 psf
(4) ML/CL-1: Unit Weight = 120 pcf, phi = 0 °, c = 1000 psf
(5) SM-2: Unit Weight = 120 pcf, phi = 32 °, c = 0 psf
(6) ML/CL-2: Unit Weight = 120 pcf, phi = 0 °, c = 1400 psf
(7) Embankment - Filter Sand: Unit Weight = 125 pcf, phi = 35 °, c = 0 psf
(8) Embankment - Drain Gravel: Unit Weight = 125 pcf, phi = 35 °, c = 0 psf
(9) Embankment - Random Fill: Unit Weight = 125 pcf, phi = 14 °, c = 300 psf
(10) Bedrock: Unit Weight = 120 pcf, phi = 0 °, c = 2300 psf

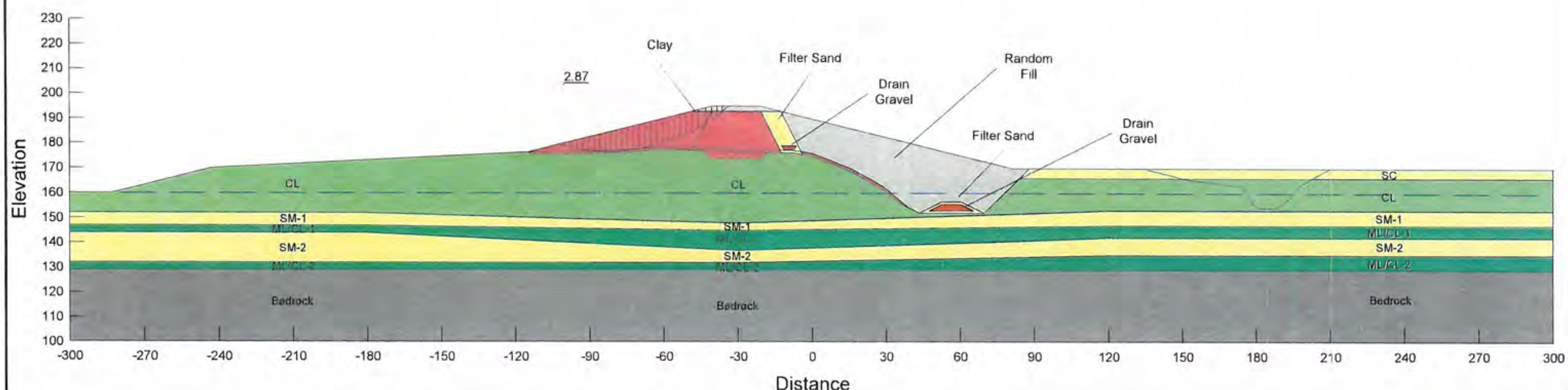


Notes:
1.Elevations in NAVD88
2.The Factor of Safety (FS) value above is for the critical failure surface

Upper Sand Creek Detention Basin Antioch, California	GEI Consultants	**Section B** Case I - End of Construction Downstream Stability	
Contra Costa County Public Works Department Martinez, California	Project 08318-0	June 2010	Fig. 6-2

Upper Sand Creek - Section C
Name: End of Construction - Upstream
Method: Spencer
Last Modified: 6/17/2010 11:00:32 AM

(1) Embankment - Clay: Unit Weight = 125 pcf, phi = 14 °, c = 300 psf
(2) ML: Unit Weight = 120 pcf, phi = 0 °, c = 1000 psf
(3) SC: Unit Weight = 120 pcf, phi = 32 °, c = 0 psf
(4) CL: Unit Weight = 120 pcf, phi = 0 °, c = 1600 psf
(5) SM-1: Unit Weight = 120 pcf, phi = 32 °, c = 0 psf
(6) ML/CL-1: Unit Weight = 120 pcf, phi = 0 °, c = 1000 psf
(7) SM-2: Unit Weight = 120 pcf, phi = 32 °, c = 0 psf
(8) ML/CL-2: Unit Weight = 120 pcf, phi = 0 °, c = 1400 psf
(9) Embankment - Filter Sand: Unit Weight = 125 pcf, phi = 35 °, c = 0 psf
(10) Embankment - Drain Gravel: Unit Weight = 125 pcf, phi = 35 °, c = 0 psf
(11) Embankment - Random Fill: Unit Weight = 125 pcf, phi = 14 °, c = 300 psf
(12) Bedrock: Unit Weight = 120 pcf, phi = 0 °, c = 2300 psf

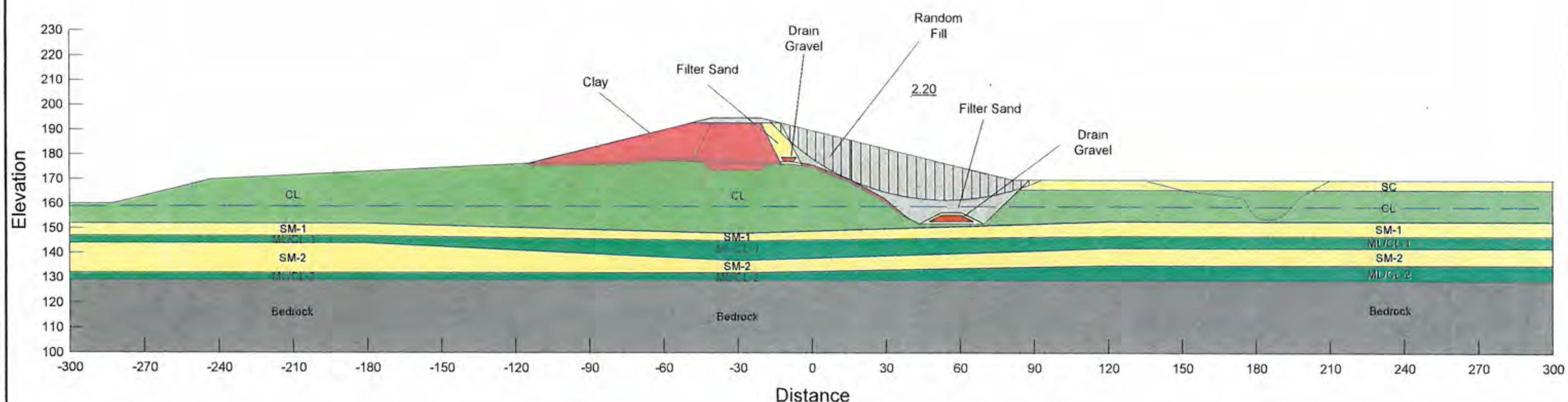


Notes:
1.Elevations in NAVD88
2.The Factor of Safety (FS) value above is for the critical failure surface

Upper Sand Creek Detention Basin Antioch, California	GEI Consultants	**Section C** Case I - End of Construction Upstream Stability
Contra Costa County Public Works Department Martinez, California	Project 08318-0	June 2010 — Fig. 6-3

Upper Sand Creek - Section C
Name: End of Construction - Downstream
Method: Spencer
Last Modified: 6/17/2010 11:00:32 AM

(1) Embankment - Clay: Unit Weight = 125 pcf, phi = 14 °, c = 300 psf
(2) ML: Unit Weight = 120 pcf, phi = 0 °, c = 1000 psf
(3) SC: Unit Weight = 120 pcf, phi = 32 °, c = 0 psf
(4) CL: Unit Weight = 120 pcf, phi = 0 °, c = 1600 psf
(5) SM-1: Unit Weight = 120 pcf, phi = 32 °, c = 0 psf
(6) ML/CL-1: Unit Weight = 120 pcf, phi = 0 °, c = 1000 psf
(7) SM-2: Unit Weight = 120 pcf, phi = 32 °, c = 0 psf
(8) ML/CL-2: Unit Weight = 120 pcf, phi = 0 °, c = 1400 psf
(9) Embankment - Filter Sand: Unit Weight = 125 pcf, phi = 35 °, c = 0 psf
(10) Embankment - Drain Gravel: Unit Weight = 125 pcf, phi = 35 °, c = 0 psf
(11) Embankment - Random Fill: Unit Weight = 125 pcf, phi = 14 °, c = 300 psf
(12) Bedrock: Unit Weight = 120 pcf, phi = 0 °, c = 2300 psf

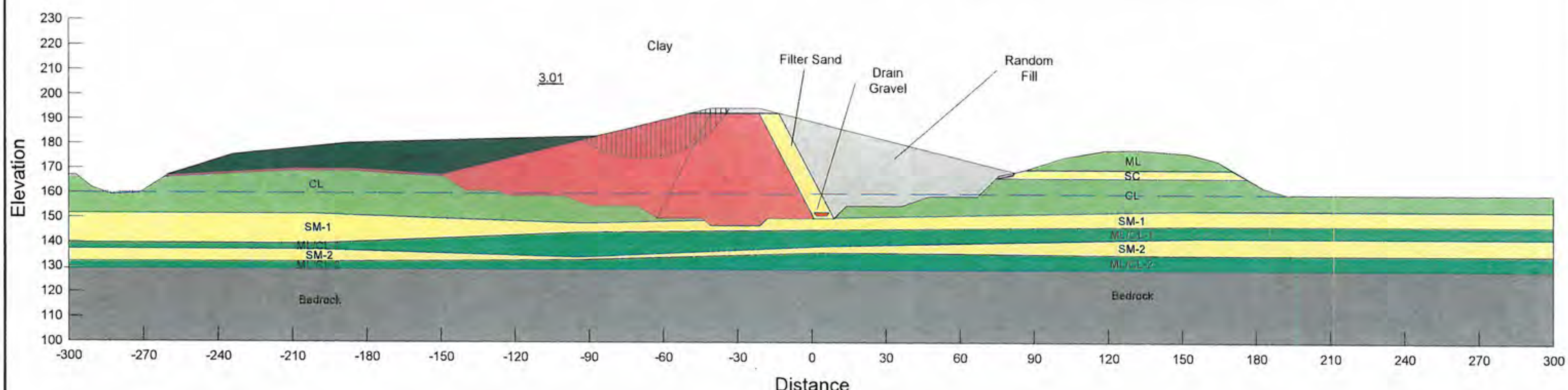


Notes:
1.Elevations in NAVD88
2.The Factor of Safety (FS) value above is for the critical failure surface

Upper Sand Creek Detention Basin Antioch, California	GEI Consultants	**Section C** Case I - End of Construction Downstream Stability
Contra Costa County Public Works Department Martinez, California	Project 08318-0	June 2010 Fig.6- 4

Upper Sand Creek - Section D
Name: End of Construction - Upstream
Method: Spencer
Last Modified: 6/17/2010 10:58:33 AM

(1) Embankment - Clay: Unit Weight = 125 pcf, phi = 14 °, c = 300 psf
(2) ML: Unit Weight = 120 pcf, phi = 0 °, c = 1000 psf
(3) SC: Unit Weight = 120 pcf, phi = 32 °, c = 0 psf
(4) CL: Unit Weight = 120 pcf, phi = 0 °, c = 1600 psf
(5) SM-1: Unit Weight = 120 pcf, phi = 32 °, c = 0 psf
(6) ML/CL-1: Unit Weight = 120 pcf, phi = 0 °, c = 1000 psf
(7) SM-2: Unit Weight = 120 pcf, phi = 32 °, c = 0 psf
(8) ML/CL-2: Unit Weight = 120 pcf, phi = 0 °, c = 1400 psf
(9) Embankment - Filter Sand: Unit Weight = 125 pcf, phi = 35 °, c = 0 psf
(10) Embankment - Drain Gravel: Unit Weight = 125 pcf, phi = 35 °, c = 0 psf
(11) Embankment - Random Fill: Unit Weight = 125 pcf, phi = 14 °, c = 300 psf
(12) Basin Fill: Unit Weight = 125 pcf, phi = 14 °, c = 300 psf
(13) Bedrock: Unit Weight = 120 pcf, phi = 0 °, c = 2300 psf

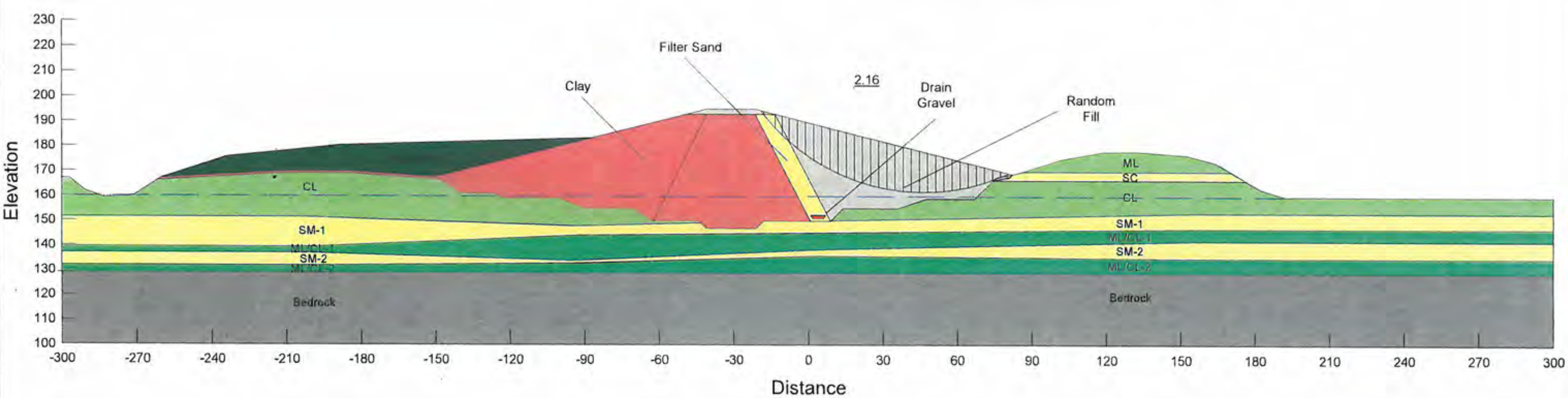


Notes:
1.Elevations in NAVD88
2.The Factor of Safety (FS) value above is for the critical failure surface

Upper Sand Creek Detention Basin Antioch, California	GEI Consultants	**Section D** Case I - End of Construction Upstream Stability
Contra Costa County Public Works Department Martinez, California	Project 08318-0	June 2010 Fig. 6-5

Upper Sand Creek - Section D
Name: End of Construction - Downstream
Method: Spencer
Last Modified: 6/17/2010 10:58:33 AM

(1) Embankment - Clay: Unit Weight = 125 pcf, phi = 14 °, c = 300 psf
(2) ML: Unit Weight = 120 pcf, phi = 0 °, c = 1000 psf
(3) SC: Unit Weight = 120 pcf, phi = 32 °, c = 0 psf
(4) CL: Unit Weight = 120 pcf, phi = 0 °, c = 1600 psf
(5) SM-1: Unit Weight = 120 pcf, phi = 32 °, c = 0 psf
(6) ML/CL-1: Unit Weight = 120 pcf, phi = 0 °, c = 1000 psf
(7) SM-2: Unit Weight = 120 pcf, phi = 32 °, c = 0 psf
(8) ML/CL-2: Unit Weight = 120 pcf, phi = 0 °, c = 1400 psf
(9) Embankment - Filter Sand: Unit Weight = 125 pcf, phi = 35 °, c = 0 psf
(10) Embankment - Drain Gravel: Unit Weight = 125 pcf, phi = 35 °, c = 0 psf
(11) Embankment - Random Fill: Unit Weight = 125 pcf, phi = 14 °, c = 300 psf
(12) Basin Fill: Unit Weight = 125 pcf, phi = 14 °, c = 300 psf
(13) Bedrock: Unit Weight = 120 pcf, phi = 0 °, c = 2300 psf

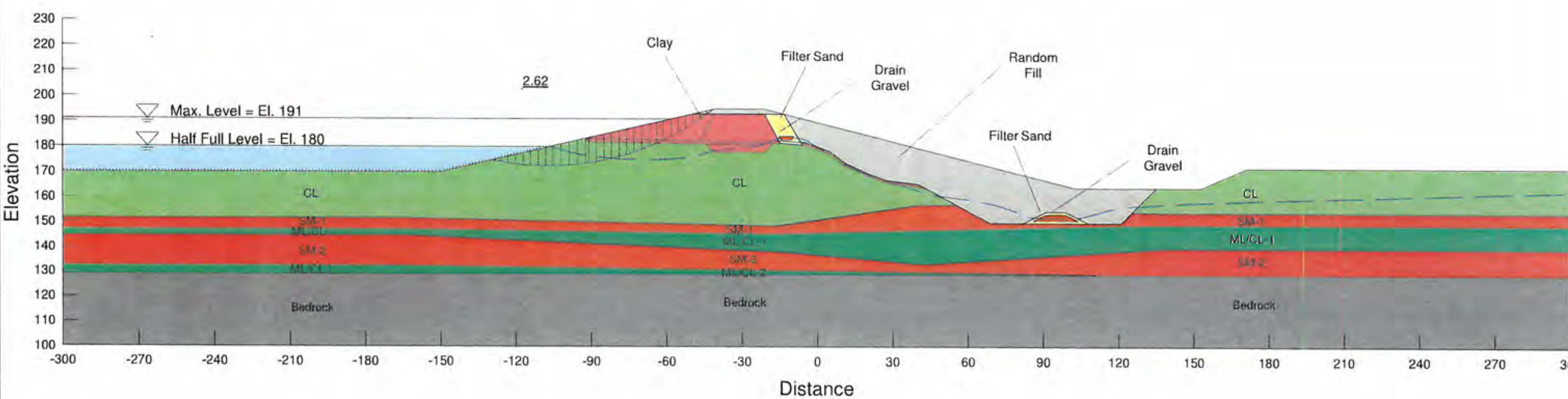


Notes:
1.Elevations in NAVD88
2.The Factor of Safety (FS) value above is for the critical failure surface

Upper Sand Creek Detention Basin Antioch, California	GEI Consultants	**Section D** Case I - End of Construction Downstream Stability
Contra Costa County Public Works Department Martinez, California	Project 08318-0	June 2010 Fig. 6-6

Steady-State Seepage Slope Stability - Section B
Name: Slope Stability (Upstream)
Method: Spencer
Last Modified: 6/23/2010 2:05:10 PM

(1) Embankment - Clay: Unit Weight = 125 pcf, phi = 31 °, c = 0 psf
(2) CL: Unit Weight = 120 pcf, phi = 30 °, c = 0 psf
(3) SM-1: Unit Weight = 120 pcf, phi = 32 °, c = 0 psf
(4) ML/CL-1: Unit Weight = 120 pcf, phi = 31 °, c = 0 psf
(5) SM-2: Unit Weight = 120 pcf, phi = 32 °, c = 0 psf
(6) ML/CL-2: Unit Weight = 120 pcf, phi = 25 °, c = 0 psf
(7) Embankment - Filter Sand: Unit Weight = 125 pcf, phi = 35 °, c = 0 psf
(8) Embankment - Drain Gravel: Unit Weight = 125 pcf, phi = 35 °, c = 0 psf
(9) Embankment - Random Fill: Unit Weight = 125 pcf, phi = 31 °, c = 0 psf
(10) Bedrock: Unit Weight = 120 pcf, phi = 35 °, c = 0 psf

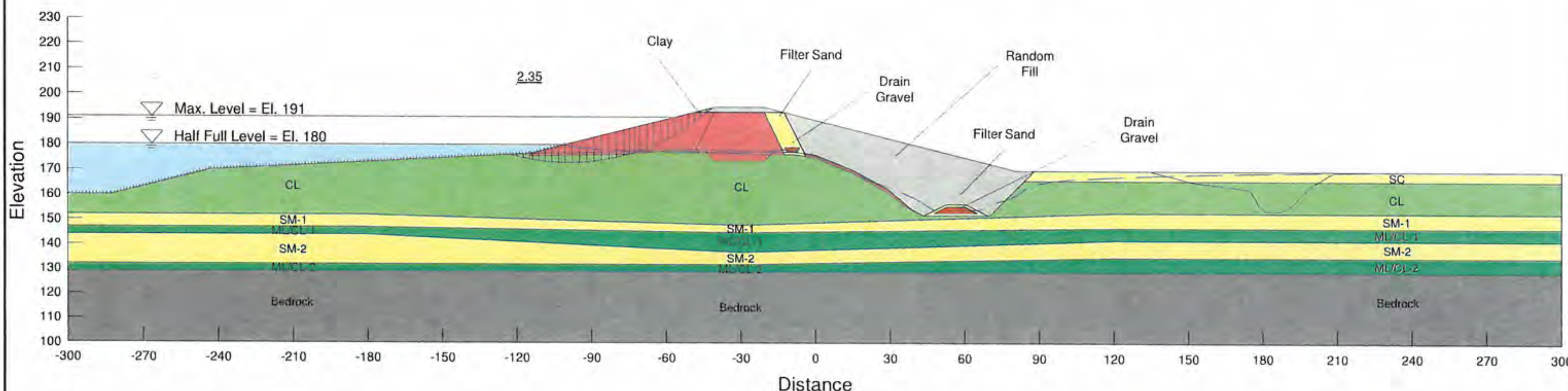


Notes:
1.Elevations in NAVD88
2.The Factor of Safety (FS) value above is for the critical failure surface.

Upper Sand Creek Detention Basin Antioch, California	GEI Consultants	**Section B** Case II - Steady State Seepage Upstream Slope Stability
Contra Costa County Public Works Department Martinez, California	Project 08318-0	June 2010 — Fig. 7-1

Steady State Seepage Slope Stability - Section C
Name: Slope Stability (Upstream)
Method: Spencer
Last Modified: 6/23/2010 2:08:31 PM

(1) Embankment - Clay: Unit Weight = 125 pcf, phi = 31 °, c = 0 psf
(2) ML: Unit Weight = 120 pcf, phi = 28 °, c = 0 psf
(3) SC: Unit Weight = 120 pcf, phi = 32 °, c = 0 psf
(4) CL: Unit Weight = 120 pcf, phi = 30 °, c = 0 psf
(5) SM-1: Unit Weight = 120 pcf, phi = 32 °, c = 0 psf
(6) ML/CL-1: Unit Weight = 120 pcf, phi = 31 °, c = 0 psf
(7) SM-2: Unit Weight = 120 pcf, phi = 32 °, c = 0 psf
(8) ML/CL-2: Unit Weight = 120 pcf, phi = 25 °, c = 0 psf
(9) Embankment - Filter Sand: Unit Weight = 125 pcf, phi = 35 °, c = 0 psf
(10) Embankment - Drain Gravel: Unit Weight = 125 pcf, phi = 35 °, c = 0 psf
(11) Embankment - Random Fill: Unit Weight = 125 pcf, phi = 31 °, c = 0 psf
(12) Bedrock: Unit Weight = 120 pcf, phi = 35 °, c = 0 psf

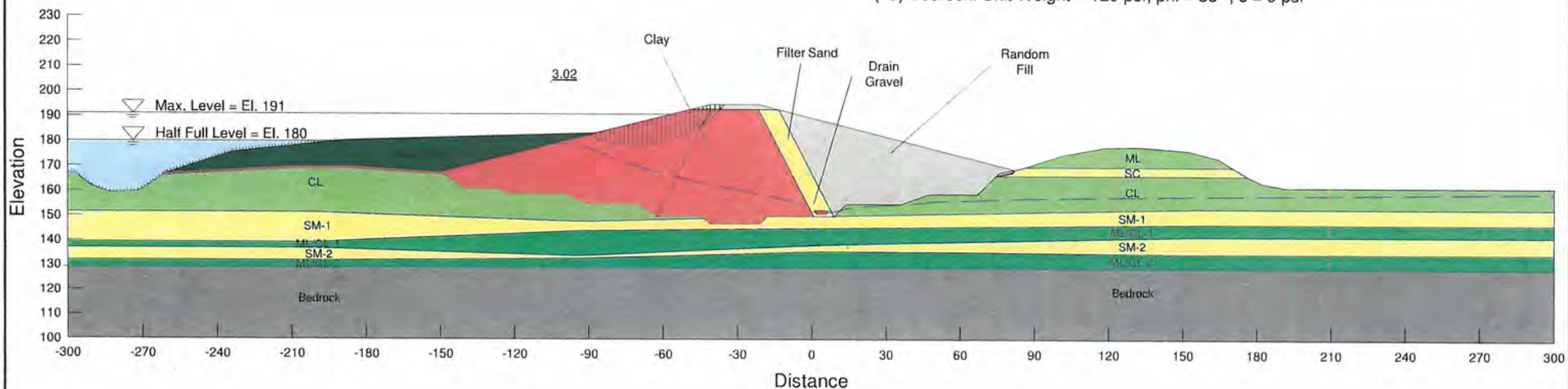


Notes:
1. Elevations in NAVD88
2. The Factor of Safety (FS) value above is for the critical failure surface.

Upper Sand Creek Detention Basin Antioch, California	GEI Consultants	**Section C** Case II - Steady State Seepage Upstream Slope Stability
Contra Costa County Public Works Department Martinez, California	Project 08318-0	June 2010 — Fig. 7-2

Steady State Seepage Slope Stability (Section D)
Name: Slope Stability (Upstream)
Method: Spencer
Last Modified: 6/23/2010 2:14:03 PM

(1) Embankment - Clay: Unit Weight = 125 pcf, phi = 31 °, c = 0 psf
(2) ML: Unit Weight = 120 pcf, phi = 28 °, c = 0 psf
(3) SC: Unit Weight = 120 pcf, phi = 32 °, c = 0 psf
(4) CL: Unit Weight = 120 pcf, phi = 30 °, c = 0 psf
(5) SM-1: Unit Weight = 120 pcf, phi = 32 °, c = 0 psf
(6) ML/CL-1: Unit Weight = 120 pcf, phi = 31 °, c = 0 psf
(7) SM-2: Unit Weight = 120 pcf, phi = 32 °, c = 0 psf
(8) ML/CL-2: Unit Weight = 120 pcf, phi = 25 °, c = 0 psf
(9) Embankment - Filter Sand: Unit Weight = 125 pcf, phi = 35 °, c = 0 psf
(10) Embankment - Drain Gravel: Unit Weight = 125 pcf, phi = 35 °, c = 0 psf
(11) Embankment - Random Fill: Unit Weight = 125 pcf, phi = 31 °, c = 0 psf
(12) Basin Fill: Unit Weight = 125 pcf, phi = 33 °, c = 0 psf
(13) Bedrock: Unit Weight = 120 pcf, phi = 35 °, c = 0 psf

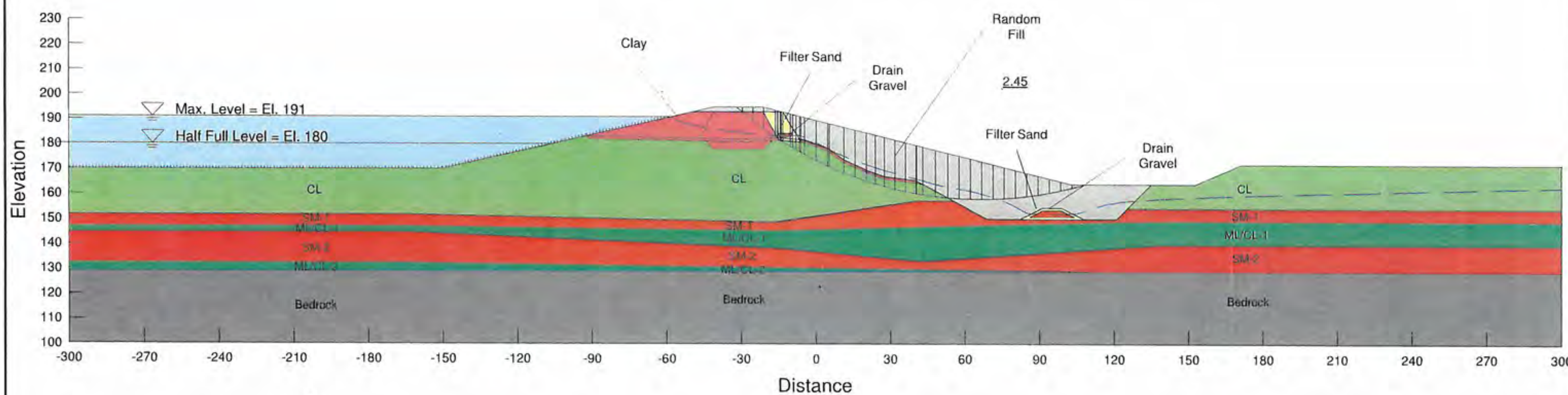


Notes:
1.Elevations in NAVD88
2.The Factor of Safety (FS) value above is for the critical failure surface.

Upper Sand Creek Detention Basin Antioch, California	GEI Consultants	**Section D** Case II - Steady State Seepage Upstream Slope Stability
Contra Costa County Public Works Department Martinez, California	Project 08318-0	June 2010 Fig. 7-3

Steady State Seepage Slope Stability - Section B
Name: Slope Stability (Downstream)
Method: Spencer
Last Modified: 6/22/2010 11:27:01 AM

(1) Embankment - Clay: Unit Weight = 125 pcf, phi = 31 °, c = 0 psf
(2) CL: Unit Weight = 120 pcf, phi = 30 °, c = 0 psf
(3) SM-1: Unit Weight = 120 pcf, phi = 32 °, c = 0 psf
(4) ML/CL-1: Unit Weight = 120 pcf, phi = 31 °, c = 0 psf
(5) SM-2: Unit Weight = 120 pcf, phi = 32 °, c = 0 psf
(6) ML/CL-2: Unit Weight = 120 pcf, phi = 25 °, c = 0 psf
(7) Embankment - Filter Sand: Unit Weight = 125 pcf, phi = 35 °, c = 0 psf
(8) Embankment - Drain Gravel: Unit Weight = 125 pcf, phi = 35 °, c = 0 psf
(9) Embankment - Random Fill: Unit Weight = 125 pcf, phi = 31 °, c = 0 psf
(10) Bedrock: Unit Weight = 120 pcf, phi = 35 °, c = 0 psf



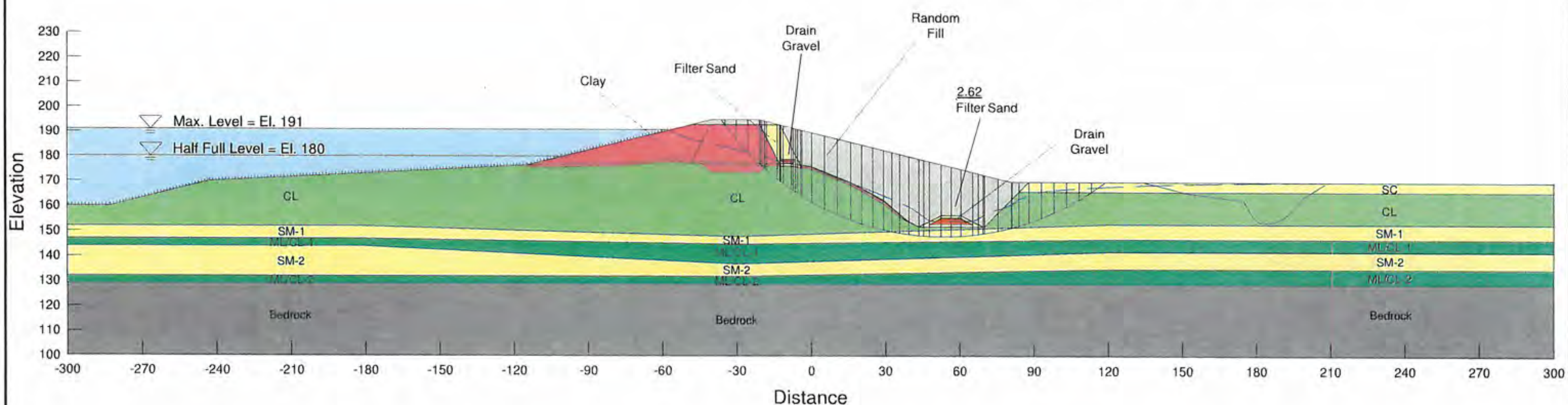
Notes:
1.Elevations in NAVD88
2.The Factor of Safety (FS) value above is for the critical failure surface.

Upper Sand Creek Detention Basin Antioch, California	GEI Consultants	**Section B** Case III - Steady State Seepage Downstream Slope Stability
Contra Costa County Public Works Department Martinez, California	Project 08318-0	June 2010 Fig. 8-1

M:\DATA_GEOTECH\08318 Upper Sand Creek\Analysis\Attach F - SSS Stability

Steady-State Seepage Slope Stability - Section C
Name: Slope Stability (Downstream)
Method: Spencer
Last Modified: 6/22/2010 12:30:36 PM

(1) Embankment - Clay: Unit Weight = 125 pcf, phi = 31 °, c = 0 psf
(2) ML: Unit Weight = 120 pcf, phi = 28 °, c = 0 psf
(3) SC: Unit Weight = 120 pcf, phi = 32 °, c = 0 psf
(4) CL: Unit Weight = 120 pcf, phi = 30 °, c = 0 psf
(5) SM-1: Unit Weight = 120 pcf, phi = 32 °, c = 0 psf
(6) ML/CL-1: Unit Weight = 120 pcf, phi = 31 °, c = 0 psf
(7) SM-2: Unit Weight = 120 pcf, phi = 32 °, c = 0 psf
(8) ML/CL-2: Unit Weight = 120 pcf, phi = 25 °, c = 0 psf
(9) Embankment - Filter Sand: Unit Weight = 125 pcf, phi = 35 °, c = 0 psf
(10) Embankment - Drain Gravel: Unit Weight = 125 pcf, phi = 35 °, c = 0 psf
(11) Embankment - Random Fill: Unit Weight = 125 pcf, phi = 31 °, c = 0 psf
(12) Bedrock: Unit Weight = 120 pcf, phi = 35 °, c = 0 psf

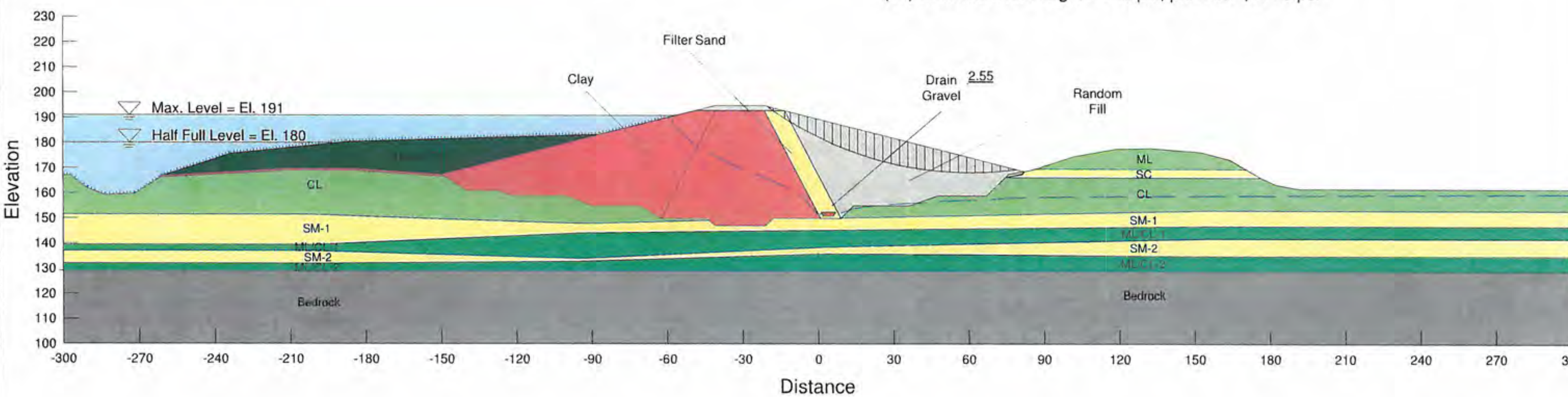


Notes:
1. Elevations in NAVD88
2. The Factor of Safety (FS) value above is for the critical failure surface.

Upper Sand Creek Detention Basin Antioch, California	GEI Consultants	**Section C** Case III - Steady State Seepage Downstream Slope Stability
Contra Costa County Public Works Department Martinez, California	Project 08318-0	June 2010 Fig. 8-2

Steady-State Seepage Slope Stability - Section D
Name: Slope Stability (Downstream)
Method: Spencer
Last Modified: 6/22/2010 12:44:03 PM

(1) Embankment - Clay: Unit Weight = 125 pcf, phi = 31 °, c = 0 psf
(2) ML: Unit Weight = 120 pcf, phi = 28 °, c = 0 psf
(3) SC: Unit Weight = 120 pcf, phi = 32 °, c = 0 psf
(4) CL: Unit Weight = 120 pcf, phi = 30 °, c = 0 psf
(5) SM-1: Unit Weight = 120 pcf, phi = 32 °, c = 0 psf
(6) ML/CL-1: Unit Weight = 120 pcf, phi = 31 °, c = 0 psf
(7) SM-2: Unit Weight = 120 pcf, phi = 32 °, c = 0 psf
(8) ML/CL-2: Unit Weight = 120 pcf, phi = 25 °, c = 0 psf
(9) Embankment - Filter Sand: Unit Weight = 125 pcf, phi = 35 °, c = 0 psf
(10) Embankment - Drain Gravel: Unit Weight = 125 pcf, phi = 35 °, c = 0 psf
(11) Embankment - Random Fill: Unit Weight = 125 pcf, phi = 31 °, c = 0 psf
(12) Basin Fill: Unit Weight = 125 pcf, phi = 33 °, c = 0 psf
(13) Bedrock: Unit Weight = 120 pcf, phi = 35 °, c = 0 psf



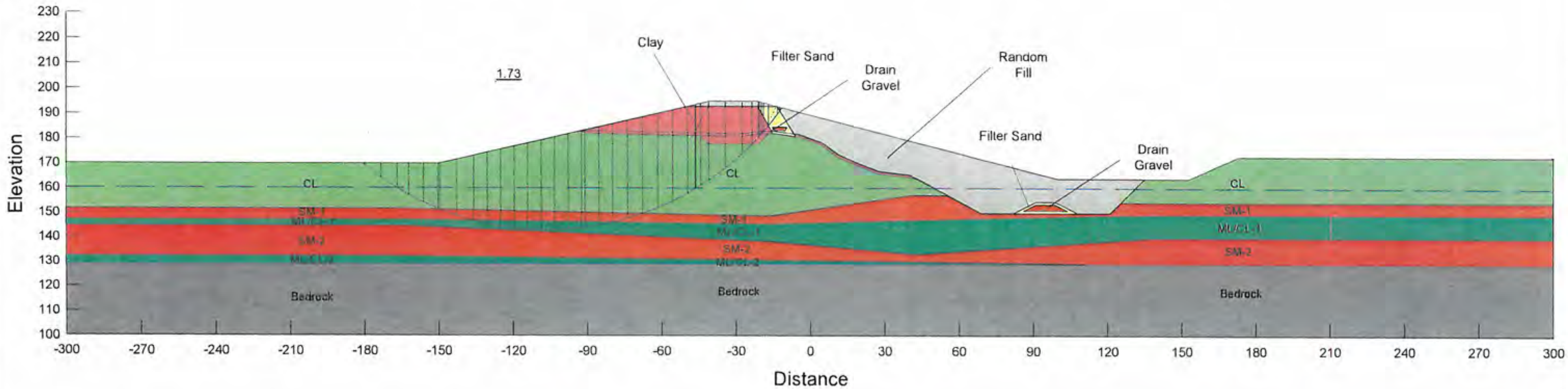
Notes:
1.Elevations in NAVD88
2.The Factor of Safety (FS) value above is for the critical failure surface.

Upper Sand Creek Detention Basin Antioch, California	GEI Consultants	**Section D** Case III - Steady State Seepage Downstream Slope Stability
Contra Costa County Public Works Department Martinez, California	Project 08318-0	June 2010 Fig. 8-3

Upper Sand Creek – Section B
Name: Pseudo-Static – Upstream Slope
Method: Spencer
Last Modified: 6/17/20/2010 12:00:19 PM

Horizontal Acceleration = 0.15 g

(1) Embankment - Clay: Unit Weight = 125 pcf, phi = 14 °, c = 300 psf
(2) CL: Unit Weight = 120 pcf, phi = 0 °, c = 1600 psf
(3) SM-1: Unit Weight = 120 pcf, phi = 32 °, c = 0 psf
(4) ML/CL-1: Unit Weight = 120 pcf, phi = 0 °, c = 1000 psf
(5) SM-2: Unit Weight = 120 pcf, phi = 32 °, c = 0 psf
(6) ML/CL-2: Unit Weight = 120 pcf, phi = 0 °, c = 1400 psf
(7) Embankment - Filter Sand: Unit Weight = 125 pcf, phi = 35 °, c = 0 psf
(8) Embankment - Drain Gravel: Unit Weight = 125 pcf, phi = 35 °, c = 0 psf
(9) Embankment - Random Fill: Unit Weight = 125 pcf, phi = 14 °, c = 300 psf
(10) Bedrock: Unit Weight = 120 pcf, phi = 0 °, c = 2300 psf



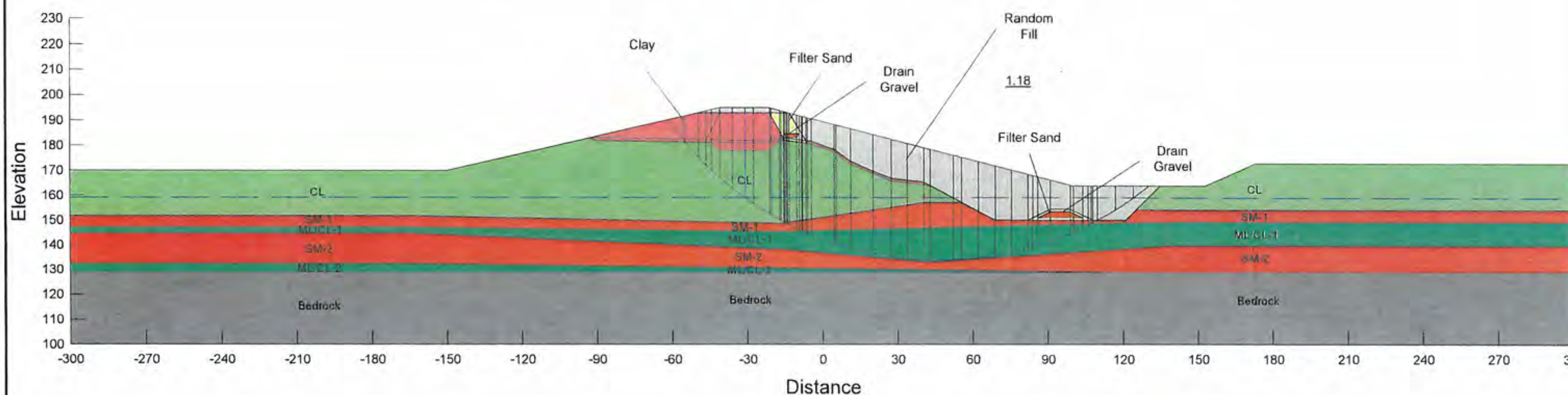
Notes:
1.Elevations in NAVD88
2.The Factor of Safety (FS) value above is for the critical failure surface

Upper Sand Creek Detention Basin Antioch, California	GEI Consultants	**Section B** Case IV – Pseudo Static Stability (Upstream)
Contra Costa County Public Works Department Martinez, California	Project 08318-0	June 2010 — Fig. 9-1

Upper Sand Creek – Section B
Name: Pseudo-Static – Downstream Slope
Method: Spencer
Last Modified: 6/17/20/2010 11:54:07 AM

Horizontal Acceleration = 0.15 g

(1) Embankment - Clay: Unit Weight = 125 pcf, phi = 14 °, c = 300 psf
(2) CL: Unit Weight = 120 pcf, phi = 0 °, c = 1600 psf
(3) SM-1: Unit Weight = 120 pcf, phi = 32 °, c = 0 psf
(4) ML/CL-1: Unit Weight = 120 pcf, phi = 0 °, c = 1000 psf
(5) SM-2: Unit Weight = 120 pcf, phi = 32 °, c = 0 psf
(6) ML/CL-2: Unit Weight = 120 pcf, phi = 0 °, c = 1400 psf
(7) Embankment - Filter Sand: Unit Weight = 125 pcf, phi = 35 °, c = 0 psf
(8) Embankment - Drain Gravel: Unit Weight = 125 pcf, phi = 35 °, c = 0 psf
(9) Embankment - Random Fill: Unit Weight = 125 pcf, phi = 14 °, c = 300 psf
(10) Bedrock: Unit Weight = 120 pcf, phi = 0 °, c = 2300 psf



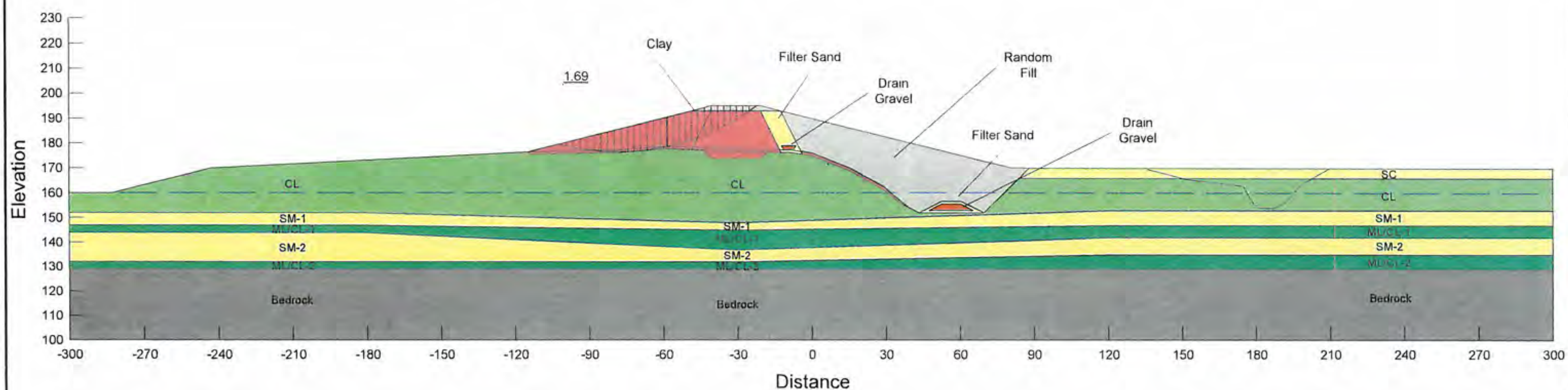
Notes:
1.Elevations in NAVD88
2.The Factor of Safety (FS) value above is for the critical failure surface

Upper Sand Creek Detention Basin Antioch, California	GEI Consultants	**Section B** Case IV – Pseudo Static Stability (Downstream)
Contra Costa County Public Works Department Martinez, California	Project 08318-0	June 2010 Fig. 9-2

Upper Sand Creek – Section C
Name: Pseudo-Static – Upstream Slope
Method: Spencer
Last Modified: 6/17/20/2010 11:58:19 AM

Horizontal Acceleration = 0.15 g

(1) Embankment - Clay: Unit Weight = 125 pcf, phi = 14 °, c = 300 psf
(2) ML: Unit Weight = 120 pcf, phi = 0 °, c = 1000 psf
(3) SC: Unit Weight = 120 pcf, phi = 32 °, c = 0 psf
(4) CL: Unit Weight = 120 pcf, phi = 0 °, c = 1600 psf
(5) SM-1: Unit Weight = 120 pcf, phi = 32 °, c = 0 psf
(6) ML/CL-1: Unit Weight = 120 pcf, phi = 0 °, c = 1000 psf
(7) SM-2: Unit Weight = 120 pcf, phi = 32 °, c = 0 psf
(8) ML/CL-2: Unit Weight = 120 pcf, phi = 0 °, c = 1400 psf
(9) Embankment - Filter Sand: Unit Weight = 125 pcf, phi = 35 °, c = 0 psf
(10) Embankment - Drain Gravel: Unit Weight = 125 pcf, phi = 35 °, c = 0 psf
(11) Embankment - Random Fill: Unit Weight = 125 pcf, phi = 14 °, c = 300 psf
(12) Bedrock: Unit Weight = 120 pcf, phi = 0 °, c = 2300 psf



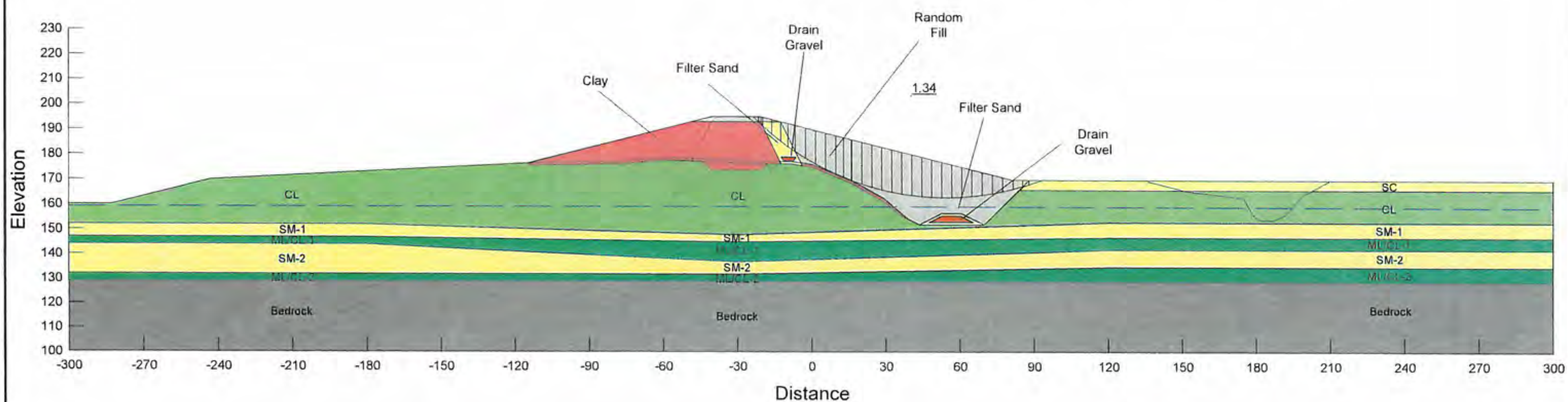
Notes:
1.Elevations in NAVD88
2.The Factor of Safety (FS) value above is for the critical failure surface

Upper Sand Creek Detention Basin Antioch, California	GEI Consultants	**Section C** Case IV – Pseudo Static Stability (Upstream)
Contra Costa County Public Works Department Martinez, California	Project 08318-0	June 2010 Fig. 9-3

Upper Sand Creek – Section C
Name: Pseudo-Static – Downstream Slope
Method: Spencer
Last Modified: 6/17/20/2010 11:58:07 AM

Horizontal Acceleration = 0.15 g

(1) Embankment - Clay: Unit Weight = 125 pcf, phi = 14 °, c = 300 psf
(2) ML: Unit Weight = 120 pcf, phi = 0 °, c = 1000 psf
(3) SC: Unit Weight = 120 pcf, phi = 32 °, c = 0 psf
(4) CL: Unit Weight = 120 pcf, phi = 0 °, c = 1600 psf
(5) SM-1: Unit Weight = 120 pcf, phi = 32 °, c = 0 psf
(6) ML/CL-1: Unit Weight = 120 pcf, phi = 0 °, c = 1000 psf
(7) SM-2: Unit Weight = 120 pcf, phi = 32 °, c = 0 psf
(8) ML/CL-2: Unit Weight = 120 pcf, phi = 0 °, c = 1400 psf
(9) Embankment - Filter Sand: Unit Weight = 125 pcf, phi = 35 °, c = 0 psf
(10) Embankment - Drain Gravel: Unit Weight = 125 pcf, phi = 35 °, c = 0 psf
(11) Embankment - Random Fill: Unit Weight = 125 pcf, phi = 14 °, c = 300 psf
(12) Bedrock: Unit Weight = 120 pcf, phi = 0 °, c = 2300 psf



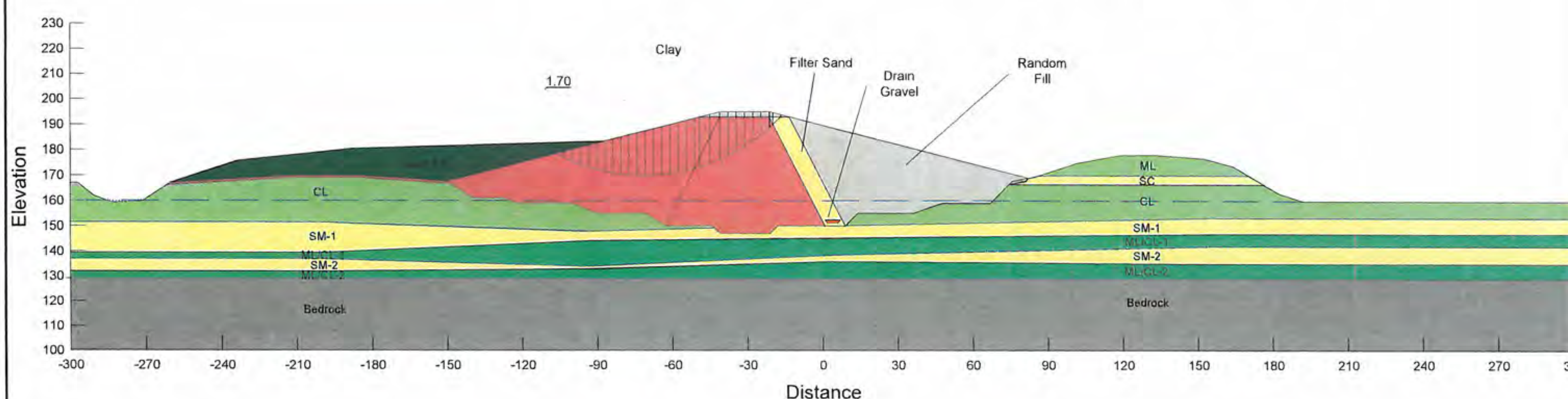
Notes:
1.Elevations in NAVD88
2.The Factor of Safety (FS) value above is for the critical failure surface

Upper Sand Creek Detention Basin Antioch, California	GEI Consultants	**Section C** Case IV – Pseudo Static Stability (Downstream)
Contra Costa County Public Works Department Martinez, California	Project 08318-0	June 2010 — Fig.9- 4

Upper Sand Creek – Section D
Name: Pseudo-Static – Upstream Slope
Method: Spencer
Last Modified: 6/17/20/2010 11:54:07 AM

Horizontal Acceleration = 0.15 g

(1) Embankment - Clay: Unit Weight = 125 pcf, phi = 14 °, c = 300 psf
(2) ML: Unit Weight = 120 pcf, phi = 0 °, c = 1000 psf
(3) SC: Unit Weight = 120 pcf, phi = 32 °, c = 0 psf
(4) CL: Unit Weight = 120 pcf, phi = 0 °, c = 1600 psf
(5) SM-1: Unit Weight = 120 pcf, phi = 32 °, c = 0 psf
(6) ML/CL-1: Unit Weight = 120 pcf, phi = 0 °, c = 1000 psf
(7) SM-2: Unit Weight = 120 pcf, phi = 32 °, c = 0 psf
(8) ML/CL-2: Unit Weight = 120 pcf, phi = 0 °, c = 1400 psf
(9) Embankment - Filter Sand: Unit Weight = 125 pcf, phi = 35 °, c = 0 psf
(10) Embankment - Drain Gravel: Unit Weight = 125 pcf, phi = 35 °, c = 0 psf
(11) Embankment - Random Fill: Unit Weight = 125 pcf, phi = 14 °, c = 300 psf
(12) Basin Fill: Unit Weight = 125 pcf, phi = 14 °, c = 300 psf
(13) Bedrock: Unit Weight = 120 pcf, phi = 0 °, c = 2300 psf



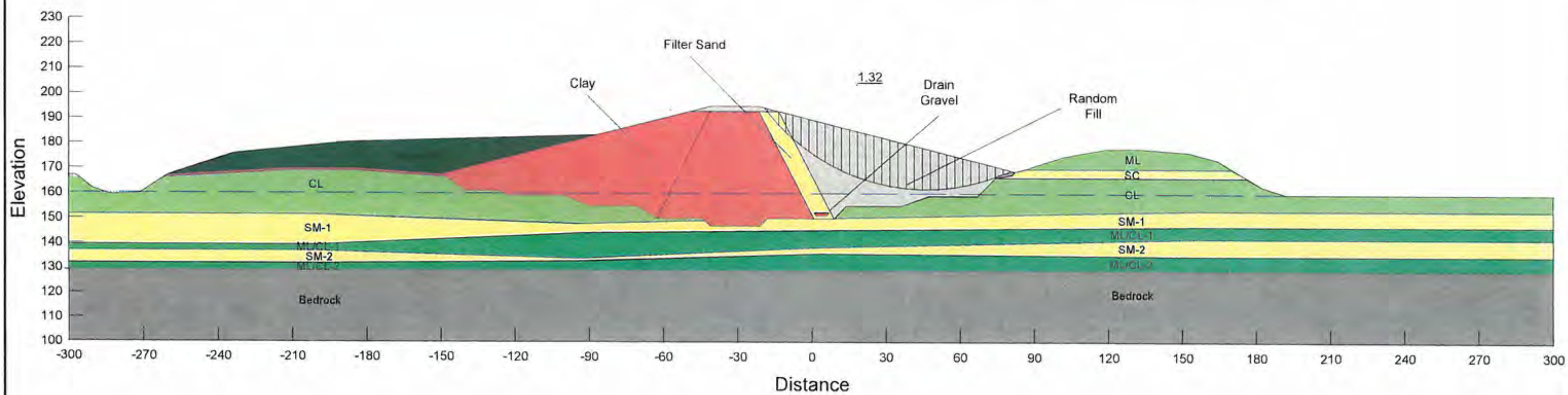
Notes:
1.Elevations in NAVD88
2.The Factor of Safety (FS) value above is for the critical failure surface

Upper Sand Creek Detention Basin Antioch, California	GEI Consultants	**Section D** Case IV – Pseudo Static Stability (Upstream)
Contra Costa County Public Works Department Martinez, California	Project 08318-0	June 2010 — Fig. 9-5

Upper Sand Creek – Section D
Name: Pseudo-Static – Downstream Slope
Method: Spencer
Last Modified: 6/17/20/2010 11:54:07 AM

Horizontal Acceleration = 0.15 g

(1) Embankment - Clay: Unit Weight = 125 pcf, phi = 14 °, c = 300 psf
(2) ML: Unit Weight = 120 pcf, phi = 0 °, c = 1000 psf
(3) SC: Unit Weight = 120 pcf, phi = 32 °, c = 0 psf
(4) CL: Unit Weight = 120 pcf, phi = 0 °, c = 1600 psf
(5) SM-1: Unit Weight = 120 pcf, phi = 32 °, c = 0 psf
(6) ML/CL-1: Unit Weight = 120 pcf, phi = 0 °, c = 1000 psf
(7) SM-2: Unit Weight = 120 pcf, phi = 32 °, c = 0 psf
(8) ML/CL-2: Unit Weight = 120 pcf, phi = 0 °, c = 1400 psf
(9) Embankment - Filter Sand: Unit Weight = 125 pcf, phi = 35 °, c = 0 psf
(10) Embankment - Drain Gravel: Unit Weight = 125 pcf, phi = 35 °, c = 0 psf
(11) Embankment - Random Fill: Unit Weight = 125 pcf, phi = 14 °, c = 300 psf
(12) Basin Fill: Unit Weight = 125 pcf, phi = 14 °, c = 300 psf
(13) Bedrock: Unit Weight = 120 pcf, phi = 0 °, c = 2300 psf

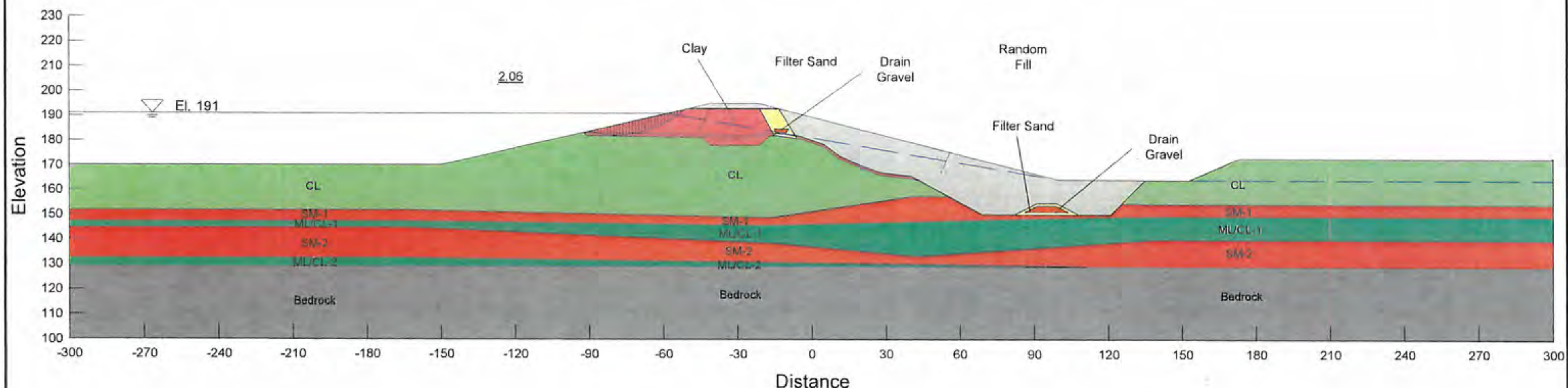


Notes:
1.Elevations in NAVD88
2.The Factor of Safety (FS) value above is for the critical failure surface

Upper Sand Creek Detention Basin Antioch, California	GEI Consultants	**Section D** Case IV – Pseudo Static Stability (Downstream)
Contra Costa County Public Works Department Martinez, California	Project 08318-0	June 2010 Fig. 9-6

Upper Sand Creek - Section B
Name: Rapid Drawdown
Method: Spencer
Last Modified: 6/17/2010 11:10:23 AM

(1) Embankment - Clay: Unit Weight = 125 pcf, phi = 16 °, Normal = 1114 psf
(2) CL: Unit Weight = 120 pcf, phi = 0 °, c = 1600 psf
(3) SM-1: Unit Weight = 120 pcf, phi = 32 °, c = 0 psf
(4) ML/CL-1: Unit Weight = 120 pcf, phi = 0 °, c = 1000 psf
(5) SM-2: Unit Weight = 120 pcf, phi = 32 °, c = 0 psf
(6) ML/CL-2: Unit Weight = 120 pcf, phi = 0 °, c = 1400 psf
(7) Embankment - Filter Sand: Unit Weight = 125 pcf, phi = 35 °, c = 0 psf
(8) Embankment - Drain Gravel: Unit Weight = 125 pcf, phi = 35 °, c = 0 psf
(9) Embankment - Random Fill: Unit Weight = 125 pcf, phi = 16 °, Normal = 1114 psf
(10) Bedrock: Unit Weight = 120 pcf, phi = 0 °, c = 2300 psf

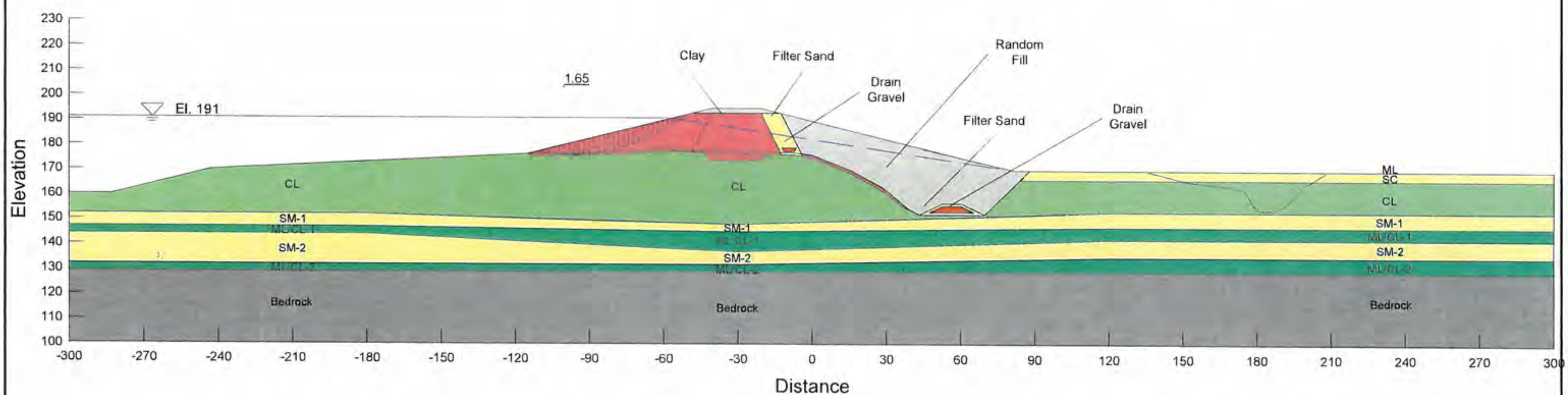


Notes:
1. Elevations in NAVD88
2. The Factor of Safety (FS) value above is for the critical failure surface.

Upper Sand Creek Detention Basin Antioch, California	GEI Consultants	**Section B** Case V - Rapid Drawdown Stability
Contra Costa County Public Works Department Martinez, California	Project 08318-0	June 2010 Fig. 10-1

Upper Sand Creek - Section C
Name: Rapid Drawdown
Method: Spencer
Last Modified: 6/17/2010 11:08:50 AM

(1) Embankment - Clay: Unit Weight = 125 pcf, phi = 16 °, Normal = 1114 psf
(2) ML: Unit Weight = 120 pcf, phi = 0 °, c = 1000 psf
(3) SC: Unit Weight = 120 pcf, phi = 32 °, c = 0 psf
(4) CL: Unit Weight = 120 pcf, phi = 0 °, c = 1600 psf
(5) SM-1: Unit Weight = 120 pcf, phi = 32 °, c = 0 psf
(6) ML/CL-1: Unit Weight = 120 pcf, phi = 0 °, c = 1000 psf
(7) SM-2: Unit Weight = 120 pcf, phi = 32 °, c = 0 psf
(8) ML/CL-2: Unit Weight = 120 pcf, phi = 0 °, c = 1400 psf
(9) Embankment - Filter Sand: Unit Weight = 125 pcf, phi = 35 °, c = 0 psf
(10) Embankment - Drain Gravel: Unit Weight = 125 pcf, phi = 35 °, c = 0 psf
(11) Embankment - Random Fill: Unit Weight = 125 pcf, phi = 16 °, Normal = 1114 psf
(12) Bedrock: Unit Weight = 120 pcf, phi = 0 °, c = 2300 psf

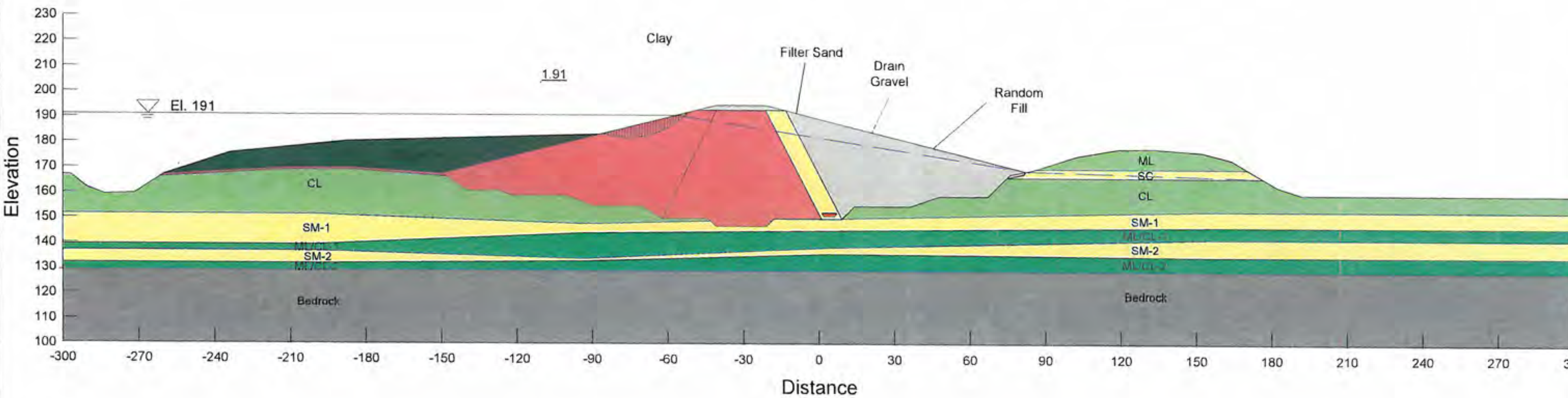


Notes:
1.Elevations in NAVD88
2.The Factor of Safety (FS) value above is for the critical failure surface.

Upper Sand Creek Detention Basin Antioch, California	GEI Consultants	**Section C** Case V - Rapid Drawdown Stability
Contra Costa County Public Works Department Martinez, California	Project 08318-0	June 2010 — Fig. 10-2

Upper Sand Creek - Section D
Name: Rapid Drawdown
Method: Spencer
Last Modified: 6/17/2010 11:07:13 AM

(1) Embankment - Clay: Unit Weight = 125 pcf, phi = 16 °, Normal = 1114 psf
(2) ML: Unit Weight = 120 pcf, phi = 0 °, c = 1000 psf
(3) SC: Unit Weight = 120 pcf, phi = 32 °, c = 0 psf
(4) CL: Unit Weight = 120 pcf, phi = 0 °, c = 1600 psf
(5) SM-1: Unit Weight = 120 pcf, phi = 32 °, c = 0 psf
(6) ML/CL-1: Unit Weight = 120 pcf, phi = 0 °, c = 1000 psf
(7) SM-2: Unit Weight = 120 pcf, phi = 32 °, c = 0 psf
(8) ML/CL-2: Unit Weight = 120 pcf, phi = 0 °, c = 1400 psf
(9) Embankment - Filter Sand: Unit Weight = 125 pcf, phi = 35 °, c = 0 psf
(10) Embankment - Drain Gravel: Unit Weight = 125 pcf, phi = 35 °, c = 0 psf
(11) Embankment - Random Fill: Unit Weight = 125 pcf, phi = 16 °, Normal = 1114 psf
(12) Basin Fill: Unit Weight = 125 pcf, phi = 14 °, c = 300 psf
(13) Bedrock: Unit Weight = 120 pcf, phi = 0 °, c = 2300 psf



Notes:
1.Elevations in NAVD88
2.The Factor of Safety (FS) value above is for the critical failure surface.

Upper Sand Creek Detention Basin Antioch, California	GEI Consultants	**Section D** Case V – Rapid Drawdown Stability	
Contra Costa County Public Works Department Martinez, California	Project 08318-0	June 2010	Fig. 10-3

Attachment 1 – Seepage Parameters

Seepage Parameters

Purpose:

Estimate hydraulic conductivity values in the proposed embankment and foundation materials for the proposed detention basin dam.

References:

U.S. Army Corps of Engineers (Corps), 1986, Seepage Analysis and Control for Dams, EM 1110-2-1901, September 30.

Federal Highway Administration (FHWA), 2002, Geotechnical Engineering Circular No. 5 Evaluation of Soil and Rock Properties, FHWA-IF-02-034, April.

Terzaghi K. and Peck, R. 1948, Soil Mechanics in Engineering Practice, John Wiley & Sons, New York.
Cedergren, H.,1989, Seepage, Drainage, and Flow Nets, John Wiley & Sons, New York.

Peck, Hanson & Thornburn, 1974, Foundation Engineering, John Wiley & Sons, Inc., New York.
Naval Facilities Engineering Command (NAVFAC), 1982, Foundations and Earth Structures, Design Manual 7.2, May.

Mitchell, J.K., 1993, Fundamentals of Soil Behavior, John Wiley and Sons, New York.

Lambe, T.W., and Whitman, R.V., 1969, Soil Mechanics, John Wiley and Sons, New York.

CESPK Levee Task Force, 2003, Recommendations for Seepage Design Criteria, Evaluation and Design Practices.

Cal Engineering & Geology, Inc., *Geotechnical Data Report, Contra Costa County Flood Control & Water Conservation District, Upper Sand Creek Detention Basin, Deer Creek Road, Antioch, California,* May 5, 2010.

Turnbull, W.J. and Mansur, C.I., 1959, Investigation of Underseepage – Mississippi River Levees, Journal of the Soil Mechanics and Foundation Division, ASCE, SM 4 August.

Approach:

We reviewed published hydraulic conductivity values for different soil types, test results, and selected design hydraulic conductivity values based on soil type, published values, and experience from previous investigations.

Analysis:

Hydraulic Conductivities

We estimated hydraulic conductivities for the different soil types along the dam observed in test borings based on published ranges for material types and on experience from previous investigations.

Published Ranges for Material Types

From EM 1110-2-1901, pages 2-10, the following figure is given:

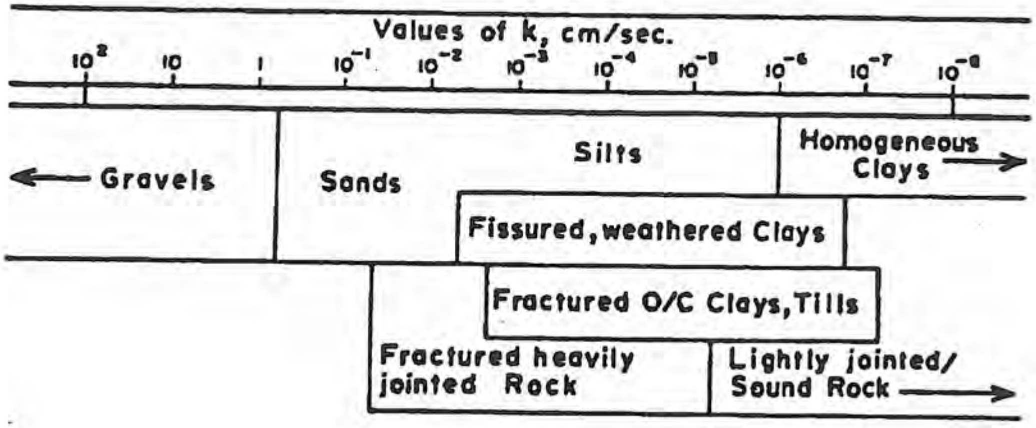


Figure 2-5. Approximate range in coefficient of permeability of soils and rocks (from Milligan[224])

GEI Consultants

Client: Contra Costa FC&WCD
Project: Upper Sand Creek Detention Basin
Project No.: 08318-0

Prepared By: D. DeCesaris
Date: May 14, 2010
Checked By: J. Nickerson
Date: June 18, 2010

Seepage Parameters

From FHWA-IF-02-034, the following figure is given:

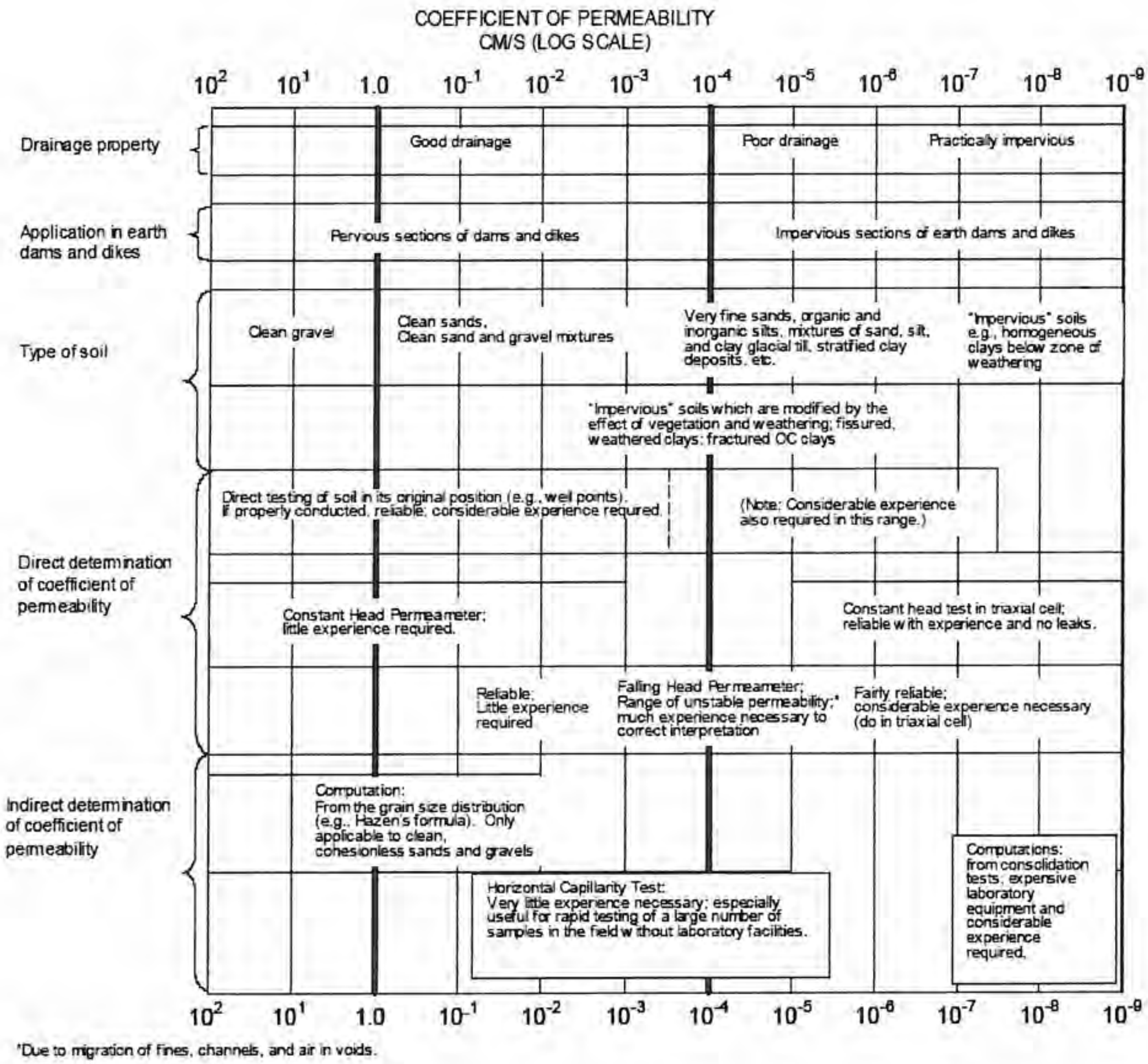


Figure 90. Range of hydraulic conductivity values based on soil type.

Terzaghi and Peck (1948), Peck, Hanson, and Thornburn (1974), and Cedergren (1989) all reference the following table:

TABLE 6

PERMEABILITY AND DRAINAGE CHARACTERISTICS OF SOILS

Coefficient of Permeability k in cm per sec (log scale)

10^2 10^1 1.0 10^{-1} 10^{-2} 10^{-3} 10^{-4} 10^{-5} 10^{-6} 10^{-7} 10^{-8} 10^{-9}

<table>
<tr><td>Drainage</td><td colspan="6">Good</td><td colspan="2">Poor</td><td colspan="3">Practically Impervious</td></tr>
<tr><td rowspan="2">Soil types</td><td colspan="2" rowspan="2">Clean gravel</td><td colspan="3">Clean sands, clean sand and gravel mixtures</td><td colspan="4">Very fine sands, organic and inorganic silts, mixtures of sand silt and clay, glacial till, stratified clay deposits, etc.</td><td colspan="2" rowspan="2">"Impervious" soils, e.g., homogeneous clays below zone of weathering</td></tr>
<tr><td colspan="2"></td><td colspan="5">"Impervious" soils modified by effects of vegetation and weathering</td></tr>
<tr><td rowspan="2">Direct determination of k</td><td colspan="6">Direct testing of soil in its original position—pumping tests. Reliable if properly conducted. Considerable experience required</td><td colspan="5"></td></tr>
<tr><td colspan="5">Constant-head permeameter. Little experience required</td><td colspan="6"></td></tr>
<tr><td rowspan="2">Indirect determination of k</td><td colspan="2"></td><td colspan="3">Falling-head permeameter. Reliable. Little experience required</td><td colspan="3">Falling-head permeameter. Unreliable. Much experience required</td><td colspan="3">Falling-head permeameter. Fairly reliable. Considerable experience necessary</td></tr>
<tr><td colspan="5">Computation from grain-size distribution. Applicable only to clean cohesionless sands and gravels</td><td colspan="4"></td><td colspan="2">Computation based on results of consolidation tests. Reliable. Considerable experience required</td></tr>
</table>

After A. Casagrande and R. E. Fadum

The reference for this table is:

Casagrande, A., and R.E. Fadum (1940), "Notes on Soil Testing for Engineering Purposes", Harvard University Graduate School of Engineering Publication No. 8, 74pp.

GEI Consultants

Client: Contra Costa FC&WCD
Project: Upper Sand Creek Detention Basin
Project No.: 08318-0

Prepared By: D. DeCesaris
Date: May 14, 2010
Checked By: J. Nickerson
Date: June 18, 2010

Seepage Parameters

NAVFAC DM 7.2 provides the following table to estimate hydraulic conductivity of compacted soils:

TABLE 1

Typical Properties of Compacted Soils

Group Symbol	Soil Type	Range of Maximum Dry Unit Weight, pcf	Range of Optimum Moisture, Percent	Typical Value of Compression — At 1.4 tsf (20 psi)	Typical Value of Compression — At 3.6 tsf (50 psi)	Typical Strength Characteristics — Cohesion (as compacted) psf	Cohesion (saturated) psf	ϕ (Effective Stress Envelope Degrees)	Tan ϕ	Typical Coefficient of Permeability ft./min.	Range of CBR Values	Range of Subgrade Modulus k lbs/cu in.
				Percent of Original Height								
GW	Well graded clean gravels, gravel-sand mixtures.	125 - 135	11 - 8	0.3	0.6	0	0	>38	>0.79	5×10^{-2}	40 - 80	300 - 500
GP	Poorly graded clean gravels, gravel-sand mix	115 - 125	14 - 11	0.4	0.9	0	0	>37	>0.74	10^{-1}	30 - 60	250 - 400
GM	Silty gravels, poorly graded gravel-sand-silt.	120 - 135	12 - 8	0.5	1.1	.....	.....	>34	>0.67	$>10^{-6}$	20 - 60	100 - 400
GC	Clayey gravels, poorly graded gravel-sand-clay.	115 - 130	14 - 9	0.7	1.6	.....	.....	>31	>0.60	$>10^{-7}$	20 - 40	100 - 300
SW	Well graded clean sands, gravelly sands.	110 - 130	16 - 9	0.6	1.2	0	0	38	0.79	$>10^{-3}$	20 - 40	200 - 300
SP	Poorly graded clean sands, sand-gravel mix.	100 - 120	21 - 12	0.8	1.4	0	0	37	0.74	$>10^{-3}$	10 - 40	200 - 300
SM	Silty sands, poorly graded sand-silt mix.	110 - 125	16 - 11	0.8	1.6	1050	420	34	0.67	$5 \times >10^{-5}$	10 - 40	100 - 300
SM-SC	Sand-silt clay mix with slightly plastic fines.	110 - 130	15 - 11	0.8	1.4	1050	300	33	0.66	$2 \times >10^{-6}$	5 - 30	100 - 300
SC	Clayey sands, poorly graded sand-clay-mix.	105 - 125	19 - 11	1.1	2.2	1550	230	31	0.60	$5 \times >10^{-7}$	5 - 20	100 - 300
ML	Inorganic silts and clayey silts.	95 - 120	24 - 12	0.9	1.7	1400	190	32	0.62	$>10^{-5}$	15 or less	100 - 200
ML-CL	Mixture of inorganic silt and clay.	100 - 120	22 - 12	1.0	2.2	1350	460	32	0.62	$5 \times >10^{-7}$	.....	
CL	Inorganic clays of low to medium plasticity.	95 - 120	24 - 12	1.3	2.5	1800	270	28	0.54	$>10^{-7}$	15 or less	50 - 200
OL	Organic silts and silt-clays, low plasticity.	80 - 100	33 - 21	.....	.....	.....	.....	.....	.....	.....	5 or less	50 - 100
MH	Inorganic clayey silts, elastic silts.	70 - 95	40 - 24	2.0	3.8	1500	420	25	0.47	$5 \times >10^{-7}$	10 or less	50 - 100
CH	Inorganic clays of high plasticity	75 - 105	36 - 19	2.6	3.9	2150	230	19	0.35	$>10^{-7}$	15 or less	50 - 150
OH	Organic clays and silty clays	65 - 100	45 - 21	.....	.....	.....	.....	.....	.....	.....	5 or less	25 - 100

Notes:

1. All properties are for condition of "Standard Proctor" maximum density, except values of k and CBR which are for "modified Proctor" maximum density.
2. Typical strength characteristics are for effective strength envelopes and are obtained from USBR data.
3. Compression values are for vertical loading with complete lateral confinement.
4. (>) Indicates that typical property is greater than the value shown.
   (..) Indicates insufficient data available for an estimate.

Hydraulic Conductivity Tests

CE&G engaged Cooper Testing Laboratories to perform hydraulic conductivity testing on three remolded specimens. Specimens were selected from representative test pit samples obtained from potential borrow areas. Test data from representative borrow materials are summarized in the table below:

Test Pit No.	Ave. Perm (cm/sec)	Ave. Perm (ft/day)
TP-1, -2, -4 Blend	1.0E-06	0.0028
TP-9, -10, -11 Blend	2.0E-07	0.0006
TP-20, -12, -26 Blend	1.0E-06	0.0028

CE&G performed four packer tests within two borings. Interpretation of the test data is provided at the end of this section. Test 1 from Boring B-4-09 and Test 2 from Boring B-8-09 indicate the rock (siltstone and sandstone) zone may have a permeability in the order of 10^{-6} cm/sec. Test 2 from B-4-09 and Test 1 from B-8-09 appear to have been conducted with excessive pressure and were stopped because the packer seals were leaking and broke; water measured may have just been flowing into the borehole above the test zone, instead of into the test zone.

Ratio of Horizontal Permeability (k_h) to Vertical Permeability (k_v)

We reviewed the following references to evaluate a kh / kv ratio:

Mitchell (1993) indicates ratios varying from less than 1 to more than 7 have been observed in clays, and ratios varying from a range of less than 5 to 15 were observed for varved clays.

Lambe and Whitman (1969), indicates ratios of 2 to 10 have been observed for normally consolidated sedimentary clay.

CESPK (2003) indicates ratios of both 4 and 9 have been used for levee foundations on recent projects in the Sacramento, California area. Additionally, for man-made levee fill, a horizontal permeability ranging from 4 to 9 times the vertical permeability has been used.

Turnbull, and Mansur (1959) indicates the horizontal permeability of a sand stratum from field pumping tests is 1.5 to 4 times the vertical permeability from lab tests.

These references provide a wide range of ratios that vary based on the nature of the soil deposit. In our opinion, a kh/kv ratio of 4 is appropriate for the foundation soils under the embankment dam. However, for the landside and waterside surficial soils, we selected a kh/kv ratio of 1 to model the potential surficial defects as described by Turnbull and Mansur. These defects would increase the vertical permeability of the surficial soils.

For aggregate drains, a kh/kv ratio of 1 is appropriate, based on inherent properties of each material and assuming that the material will be placed without significant segregation.

Design Permeability Values

A summary of our design permeability and k-ratio values for the seepage model layers are assigned as follows:

Soil Layer	k_h (ft/day)	k_h (cm/sec)	k_v/k_h	Resulting k_v (ft/day)	Resulting k_v (cm/sec)
Embankment - Clay	0.028	10^{-5}	1	0.028	10^{-5}
Embankment – Random Fill	0.028	10^{-5}	1	0.028	10^{-5}
Embankment – Filter Sand	28.3	10^{-2}	1	28.3	10^{-2}
Embankment – Drain Gravel	2835	1	1	2835	1
ML	0.28	10^{-4}	0.25	0.07	2.5×10^{-5}
SC	0.28	10^{-4}	0.25	0.07	2.5×10^{-5}
CL	0.014	5×10^{-6}	0.25	0.003	10^{-7}
SM-1	0.28	10^{-4}	0.25	0.07	2.5×10^{-5}
ML/CL-1	0.28	10^{-4}	0.25	0.07	2.5×10^{-5}
SM-2	1.41	5×10^{-4}	0.25	0.35	1.3×10^{-4}
ML/CL-2	0.28	10^{-4}	0.25	0.07	2.5×10^{-5}
Bedrock	0.0028	10^{-6}	1	0.0028	10^{-6}

Unsaturated Soil Properties

SEEP/W considers flow of water through both partially saturated and saturated soil. In SEEP/W we defined hydraulic conductivity as a function of soil suction (negative pore pressures) for soils that may become partially saturated in the model. We modeled unsaturated soil properties using SEEP/W's built-in volumetric water content (VWC) functions, which are based on soil gradation. We used these curves to estimate hydraulic conductivity (K) functions (hydraulic conductivity versus suction) for each soil type using the following procedure.

1. We used SEEP/W to generate a VWC function based on a saturated volumetric water content (i.e. porosity), which were based on typical ranges for the soil types, and on the selection of one of the seven built-in functions. SEEP/W then generated the VWC.
2. We used SEEP/W to generate a K-function based on the VWC function. The generation of the K-function is based on the method proposed by VanGenuchten (1980), which estimates points of the K-function based on the shape of the VWC function. The required input parameters are saturated hydraulic conductivity and residual volumetric water content. The saturated hydraulic conductivities are listed in the previous table. We estimated that the residual volumetric water content for each soil was 10% of the saturated volumetric water content.

The input parameters that we used for each soil type are listed in the table below.

Soil Layer	Built-in function used	Saturated volumetric water content	Residual volumetric water content
Embankment - Clay	Silt	0.44	0.044
Embankment – Random Fill	Silt	0.44	0.044
Embankment – Filter Sand	Sand	0.44	0.044
Embankment – Drain Gravel	Gravel	0.34	0.034
ML	Silt	0.49	0.049
SC	Silty Sand	0.44	0.044
CL	Clay	0.57	0.057
SM-1	Silty Sand	0.44	0.044
ML/CL-1	Silt	0.49	0.049
SM-2	Silty Sand	0.44	0.044
ML/CL-2	Silt	0.49	0.049

WATER PRESSURE TEST IN BEDROCK				B-4-09	
PROJECT:	Upper Sand Creek Detention Basin				
LOCATION:	Antioch, California			PROJ NO.:	081070
CLIENT:	Contra Costa FC&WCD			TEST NO.:	1
PERF'D BY:	CE&G	DATE:		DATUM:	NGVD29
CALC'D BY:	H. Shields	DATE:	6/9/2010	GROUND	
CHECKED:	J. Nickerson	DATE:	6/15/2010	SURF. EL.:	181

PACKER INSTALLATION DETAILS

Type of packer:	Pneumatic
Packer pressure, psi	59
Gage Height (above ground surface, feet):	1.0
Estimated Depth to Water Table (below ground surface, feet):	20

TEST INTERVAL DETAILS

Depth to Top of test zone (below ground surface, feet):	50.0
Depth to Bottom of Test Zone (below ground surface, feet):	60.0
Length, L, feet:	10.0
Borehole Diameter, D, inches:	4.875

Rock Type at test interval: Sandstone

m = SQRT (Kh/Kv) = 3

APPLIED WATER PRESSURE, p	TIME INTERVAL AT THIS PRESSURE	TOTAL VOLUME OF WATER	AVERAGE FLOW RATE, q, DURING INTERVAL		COEFFICIENT OF PERMEABILITY, K	
(psi)	(min)	(gal)	(gal/min)	(cu ft/min)	(cm/sec)	(feet/min)
30	5.0	0.20	0.04	0.01	2.4E-06	4.7E-06
45	5.0	0.10	0.02	0.00	8.6E-07	1.7E-06
59	5.0	0.00	0.00	0.00	0.0E+00	0.0E+00
44	5.0	0.00	0.00	0.00	0.0E+00	0.0E+00
28	5.0	0.00	0.00	0.00	0.0E+00	0.0E+00
				Average	**6.5E-07**	**1.3E-06**

NOTES:

1.- Water pressure, p, was measured with a pressure gage attached to the water line above ground.

2. - Friction losses in the water pipes were considered negligible.

3.- Hc represents the total head of water in feet at the midpoint of the test section length,

$$H_C = \frac{p}{.433} + h$$

4.- Hydraulic Conductivity,

$$K_h = \frac{q * Ln[\frac{m*L}{D} + \sqrt{1+(\frac{m*L}{D})^2}]}{2\pi * L * H_C}$$

per Lambe & Whitman, Soil Mechanics, 1969, p. 285, case G, constant head test.

GEI Consultants

WATER PRESSURE TEST IN BEDROCK				B-4-09	
PROJECT:	Upper Sand Creek				
LOCATION:	Antioch, California			PROJ NO.:	081070
CLIENT:	GEI			TEST NO.:	2
PERF'D BY:	CE&G	DATE:		DATUM:	NGVD29
CALC'D BY:	H. Shields	DATE:	6/9/2010	GROUND	
CHECKED:	J. Nickerson	DATE:	6/15/2010	SURF. EL.:	181

PACKER INSTALLATION DETAILS

Type of packer:	Pneumatic
Packer pressure, psi	28
Gage Height (above ground surface, feet):	15.0
Estimated Depth to Water Table (below ground surface, feet):	20

TEST INTERVAL DETAILS

Depth to Top of test zone (below ground surface, feet):	36.0
Depth to Bottom of Test Zone (below ground surface, feet):	46.0
Length, L, feet:	10.0
Borehole Diameter, D, inches:	4.875

Rock Type at test interval: Claystone and Widely Graded Sand

m = SQRT (Kh/Kv) = 3

APPLIED WATER PRESSURE, p	TIME INTERVAL AT THIS PRESSURE	TOTAL VOLUME OF WATER	AVERAGE FLOW RATE, q, DURING INTERVAL		COEFFICIENT OF PERMEABILITY, K	
(psi)	(min)	(gal)	(gal/min)	(cu ft/min)	(cm/sec)	(feet/min)
28	5.0	21.10	4.22	0.56	2.3E-04	4.5E-04
Test stopped because upper packer seal would not seal and broke						

NOTES:

1.- Water pressure, p, was measured with a pressure gage attached to the water line above ground.

2. - Friction losses in the water pipes were considered negligible.

3.- Hc represents the total head of water in feet at the midpoint of the test section length,

$$H_C = \frac{p}{.433} + h$$

4.- Hydraulic Conductivity,

$$K_h = \frac{q * Ln[\frac{m*L}{D} + \sqrt{1+(\frac{m*L}{D})^2}]}{2\pi * L * H_C}$$

per Lambe & Whitman, Soil Mechanics, 1969, p. 285, case G, constant head test.

GEI Consultants

WATER PRESSURE TEST IN BEDROCK				B-8-09	
PROJECT:	Upper Sand Creek Detention Basin				
LOCATION:	Antioch, California			PROJ NO.:	081070
CLIENT:	Contra Costa FC&WCD			TEST NO.:	1
PERF'D BY:	CE&G	DATE:		DATUM:	NGVD29
CALC'D BY:	H. Shields	DATE:	6/9/2010	GROUND	
CHECKED:	J. Nickerson	DATE:	6/15/2010	SURF. EL.:	166

PACKER INSTALLATION DETAILS

Type of packer: Pneumatic

Packer pressure, psi: 33

Gage Height (above ground surface, feet): 4.5

Estimated Depth to Water Table (below ground surface, feet): 9

TEST INTERVAL DETAILS

Depth to Top of test zone (below ground surface, feet): 24.0

Depth to Bottom of Test Zone (below ground surface, feet): 40.0

Length, L, feet: 16.0

Borehole Diameter, D, inches: 4.875

Rock Type at test interval: Gravelly Lean Clay, Siltstone and Sandstone

m = SQRT (Kh/Kv) = 3

APPLIED WATER PRESSURE, p	TIME INTERVAL AT THIS PRESSURE	TOTAL VOLUME OF WATER	AVERAGE FLOW RATE, q, DURING INTERVAL		COEFFICIENT OF PERMEABILITY, K	
(psi)	(min)	(gal)	(gal/min)	(cu ft/min)	(cm/sec)	(feet/min)
18	5.0	8.50	1.70	0.23	1.2E-04	2.3E-04
22	5.0	98.50	19.70	2.63	1.1E-03	2.2E-03
31	5.0	133.00	26.60	3.56	1.2E-03	2.3E-03
24	5.0	94.50	18.90	2.53	1.0E-03	2.0E-03
Stopped test because upper packer seal broke						

NOTES:

1.- Water pressure, p, was measured with a pressure gage attached to the water line above ground.

2. - Friction losses in the water pipes were considered negligible.

3.- Hc represents the total head of water in feet at the midpoint of the test section length,

$$H_C = \frac{p}{.433} + h$$

4.- Hydraulic Conductivity,

$$K_H = \frac{q * Ln[\frac{m*L}{D} + \sqrt{1+(\frac{m*L}{D})^2}]}{2\pi * L * H_C}$$

per Lambe & Whitman, Soil Mechanics, 1969, p. 285, case G, constant head test.

GEI Consultants

WATER PRESSURE TEST IN BEDROCK				B-8-09	
PROJECT:	Upper Sand Creek Detention Basin				
LOCATION:	Antioch, California			PROJ NO.:	081070
CLIENT:	Contra Costa FC&WCD			TEST NO.:	2
PERF'D BY:	CE&G	DATE:		DATUM:	NGVD29
CALC'D BY:	H. Shields	DATE:	6/9/2010	GROUND	
CHECKED:	J. Nickerson	DATE:	6/15/2010	SURF. EL.:	166

PACKER INSTALLATION DETAILS

Type of packer:	Pneumatic
Packer pressure, psi	50
Gage Height (above ground surface, feet):	3.5
Estimated Depth to Water Table (below ground surface, feet):	9

TEST INTERVAL DETAILS

Depth to Top of test zone (below ground surface, feet):	40.0
Depth to Bottom of Test Zone (below ground surface, feet):	50.0
Length, L, feet:	10.0
Borehole Diameter, D, inches:	4.875

Rock Type at test interval: Siltstone and Sandstone

m = SQRT (Kh/Kv) = 3

APPLIED WATER PRESSURE, p	TIME INTERVAL AT THIS PRESSURE	TOTAL VOLUME OF WATER	AVERAGE FLOW RATE, q, DURING INTERVAL		COEFFICIENT OF PERMEABILITY, K	
(psi)	(min)	(gal)	(gal/min)	(cu ft/min)	(cm/sec)	(feet/min)
25	5.0	0.10	0.02	0.00	1.5E-06	3.0E-06
38	5.0	0.10	0.02	0.00	1.1E-06	2.1E-06
51	5.0	0.10	0.02	0.00	8.3E-07	1.6E-06
38	5.0	-0.10	-0.02	0.00	-1.1E-06	-2.1E-06
24	5.0	-0.10	-0.02	0.00	-1.6E-06	-3.2E-06
				Average	**1.6E-07**	**3.1E-07**

NOTES:

1.- Water pressure, p, was measured with a pressure gage attached to the water line above ground.

2. - Friction losses in the water pipes were considered negligible.

3.- Hc represents the total head of water in feet at the midpoint of the test section length,

$$H_C = \frac{p}{.433} + h$$

4.- Hydraulic Conductivity,

$$K_R = \frac{q * Ln[\frac{m*L}{D} + \sqrt{1+(\frac{m*L}{D})^2}]}{2\pi * L * H_C}$$

per Lambe & Whitman, Soil Mechanics, 1969, p. 285, case G, constant head test.

GEI Consultants

Attachment 2 – Embankment Shear Strengths

Evaluation of Embankment Shear Strengths

Problem:

Select representative strength parameter values for the proposed embankment material for slope stability analysis.

References:

(1) Cal Engineering & Geology, Inc., *Geotechnical Data Report, Contra Costa County Flood Control & Water Conservation District, Upper Sand Creek Detention Basin, Deer Creek Road, Antioch, California,* May 5, 2010.

(2) Naval Facilities Engineering Command (NAVFAC), 1986, Soil Mechanics, Design Manual 7.1 and 7.2, September.

Approach:

Materials in the proposed embankment include a clay core, random fill, and a graded filter zone. The shear strength properties for the filter materials were estimated based on published values for compacted coarse-grained soils. The process for evaluating shear strength for the clay core is described below. The random fill also will consist of clayey soil, and we have assumed that the random fill has the same properties as the clay core.

Clay Core and Random Fill:

The clay core and random fill sections of the embankment will be constructed with borrow from cut areas of the site. Shear strengths for the proposed embankment borrow material were estimated from laboratory Consolidated Undrained (CU) triaxial tests on remolded specimens. Specimens were selected from representative test pit samples at potential borrow areas.

Index and Moisture-Density Test Results

Index test and moisture-density test (ASTM D698) data from representative borrow materials are summarized in the table below:

Test Pit No.	USCS	LL	PL	PI	Max. Dry Density (pcf)	Optimum Moisture Content (%)	Max. Total Density (pcf)
TP-1, -2, -4 Blend	CL	37	17	20	106.7	16.5	124.3
TP-9, -10, -11 Blend	CL	39	18	21	106.9	17.7	125.8
TP-20, -12, -26 Blend	CL	36	19	18	106.6	18.1	125.9

Assume Typical Value = 125 pcf

CU Test Results

CE&G engaged Cooper Testing Laboratories to perform three sets of 3-point laboratory CU tests on the proposed embankment fill. The samples were compacted to 97% of maximum laboratory dry densifty (ASTM D698) and optimum moisture content. Stresses at failure (defined at 5% strain) were evaluated for each set of data. For each set, we estimated a drained strength envelope (S-Envelope) and an undrained strength envelope (R-Envelope).

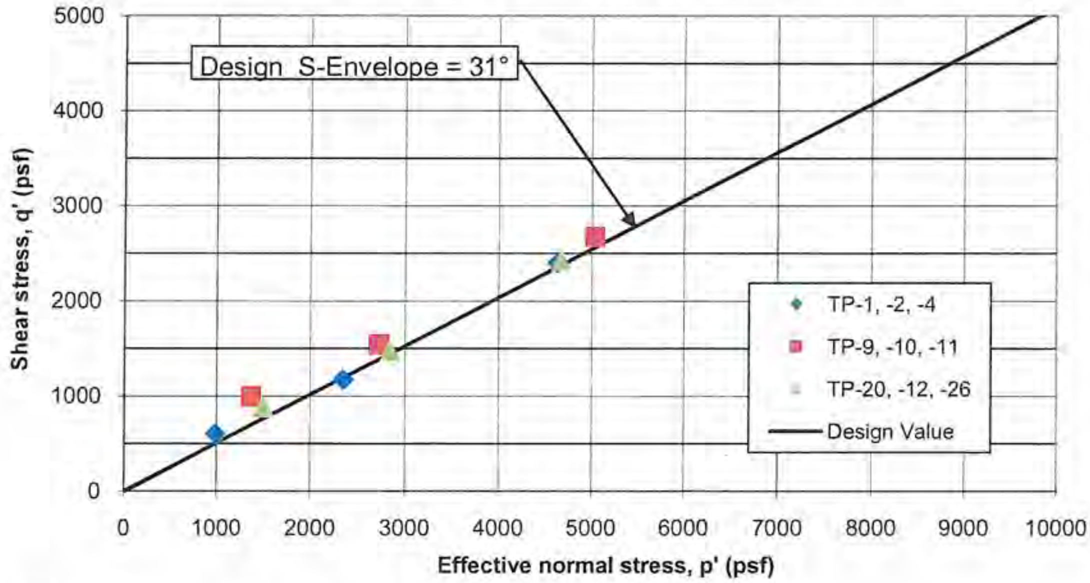
Summaries of each set of CU tests and our interpretation of each test are attached.

Embankment Shear Strengths

Drained Strength Interpretation –S-Envelope

Drained strengths in the proposed embankment fill were estimated by plotting the failure points (at 5% strain) for each test in p-q stress space. A failure envelope was then best-fit to the data to provide an effective friction angle. Cohesion was assumed to be zero. The data and the design failure envelope are summarized in the following figure:

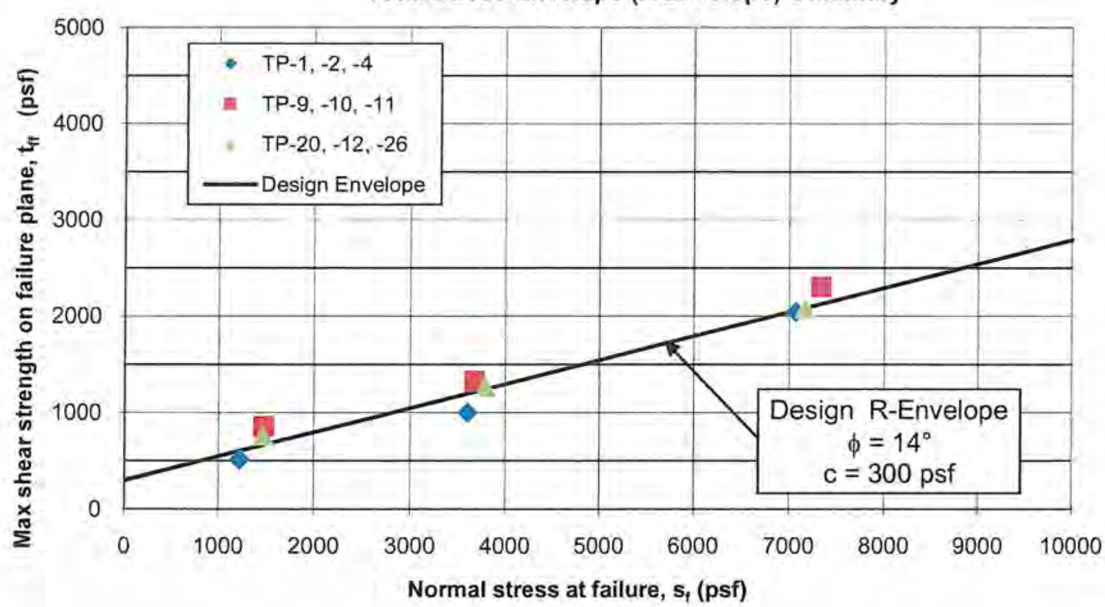
Effective Stress Envelope (S Envelope) Summary



Undrained Strength Interpretation – R-Envelope

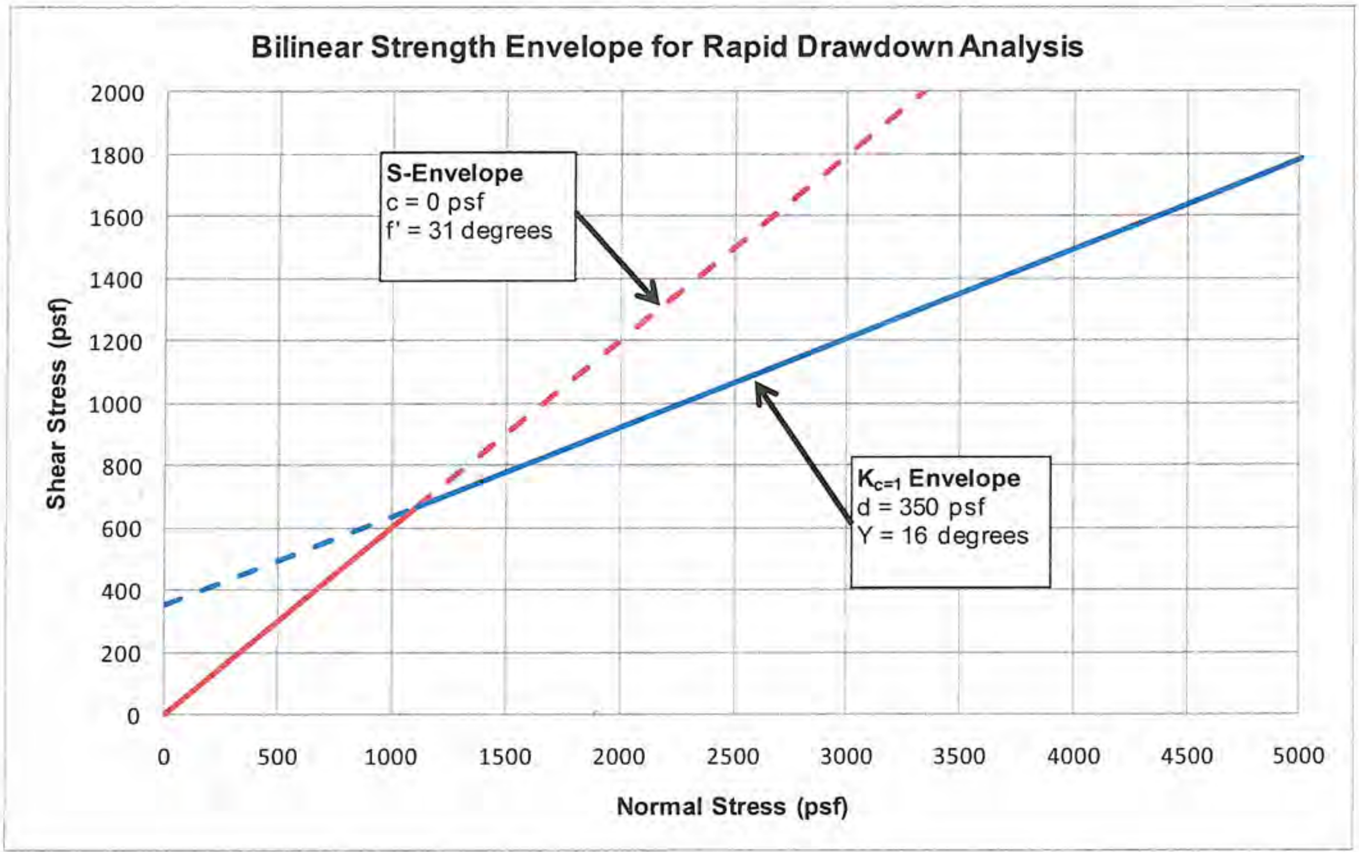
The R-envelope is obtained by plotting the undrained shear strength (τ_{ff}) versus the effective stress on the failure plane at consolidation (σ'_{fc}). This represents an undrained shear strength for the given consolidation pressure. Failure points were plotted from each test point and the data was best-fit with a design failure envelope. The data and the design failure envelope are summarized in the following figure:

Total Stress Envelope (R Envelope) Summary



Rapid Drawdown Strength Envelope

The rapid drawdown analyses performed using the three-stage procedure outlined in Appendix G, Section G-3 of EM 1110-2-1902 requires a bi-linear strength envelope developed from CU tests. The bi-linear envelope is defined by the lower bound portions of the K_c=1 envelope and the S-envelope. The bi-linear envelope for the embankment soils is shown on the figure below.



Drainage Material:

We estimated material properties for the drainage materials based on the properties given in Table 1 of NAVFAC DM 7.2 – Typical Properties of Compacted Soils (attached). The estimated properties are given in the table below.

GEI Consultants	**Client:** Contra Costa FC&WCD	**Prepared By:**	D. DeCesaris
	Project: Upper Sand Creek Detention Basin	**Date:**	April 30, 2010
	Project No.: 08318-0	**Checked By:**	M. Quinn
	Embankment Shear Strengths	**Date:**	May 10, 2010

Summary:

Shear strengths for the materials in the embankment are summarized below:

Material	Unit Weight (pcf)	Total Stress Envelope		Effective Stress Envelope		Kc=1 Envelope	
		c' (psf)	ϕ' (deg)	c (psf)	ϕ (deg)	$d_{Kc=1}$ (psf)	$\Psi_{Kc=1}$ (deg)
Clay Core	125	300	14	0	31	350	16
Random Fill	125	300	14	0	31	350	16
Filter Sand	125	0	35	0	35	--	--
Drain Gravel	125	0	35	0	35	--	--

Attachments:

NAVFAC DM 7.2 – Table 1 – Typical Properties of Compacted Soils
CU Test Evaluation sheets for TP-1, -2, -4 Blend, TP-9, -10, -11 Blend and TP-20, -12, -26 Blend
CU Test Data sheets for TP-1, -2, -4 Blend, TP-9, -10, -11 Blend and TP-20, -12, -26 Blend

Assumed Gravel Filter

Assumed Sand Filter

TABLE 1
Typical Properties of Compacted Soils

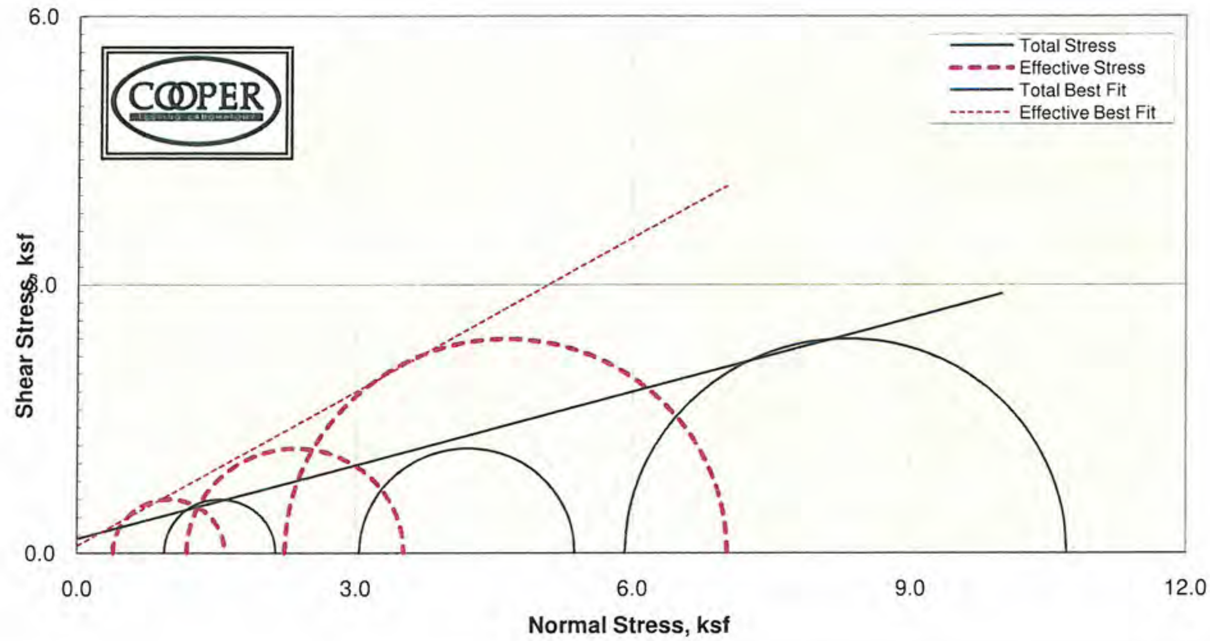
Group Symbol	Soil Type	Range of Maximum Dry Unit Weight, pcf	Range of Optimum Moisture, Percent	Typical Value of Compression: At 1.4 tsf (20 psi)	Typical Value of Compression: At 3.6 tsf (50 psf)	Typical Strength Characteristics: Cohesion (as compacted) psf	Typical Strength Characteristics: Cohesion (saturated) psf	Typical Strength Characteristics: ∅ (Effective Stress Envelope Degrees)	Typical Strength Characteristics: Tan ∅	Typical Coefficient of Permeability ft./min.	Range of CBR Values	Range of Subgrade Modulus k lbs/cu in.
				Percent of Original Weight								
GW	Well graded clean gravels, gravel-sand mixtures.	125 - 135	11 - 8	0.3	0.6	0	0	>38	>0.79	5×10^{-2}	40 - 80	300 - 500
GP	Poorly graded clean gravels, gravel-sand mix	115 - 125	14 - 11	0.4	0.9	0	0	>37	>0.74	10^{-1}	30 - 60	250 - 400
GM	Silty gravels, poorly graded gravel-sand-silt.	120 - 135	12 - 8	0.5	1.1	.....	.....	>34	>0.67	$>10^{-6}$	20 - 60	100 - 400
GC	Clayey gravels, poorly graded gravel-sand-clay.	115 - 130	14 - 9	0.7	1.6	.....	.....	>31	>0.60	$>10^{-7}$	20 - 40	100 - 300
SW	Well graded clean sands, gravelly sands.	110 - 130	16 - 9	0.6	1.2	0	0	38	0.79	$>10^{-3}$	20 - 40	200 - 300
SP	Poorly graded clean sands, sand-gravel mix.	100 - 120	21 - 12	0.8	1.4	0	0	37	0.74	$>10^{-3}$	10 - 40	200 - 300
SM	Silty sands, poorly graded sand-silt mix.	110 - 125	16 - 11	0.8	1.6	1050	420	34	0.67	$5 \times >10^{-5}$	10 - 40	100 - 300
SM-SC	Sand-silt clay mix with slightly plastic fines.	110 - 130	15 - 11	0.8	1.4	1050	300	33	0.66	$2 \times >10^{-6}$	5 - 30	100 - 300
SC	Clayey sands, poorly graded sand-clay-mix.	105 - 125	19 - 11	1.1	2.2	1550	230	31	0.60	$5 \times >10^{-7}$	5 - 20	100 - 300
ML	Inorganic silts and clayey silts.	95 - 120	24 - 12	0.9	1.7	1400	190	32	0.62	$>10^{-5}$	15 or less	100 - 200
ML-CL	Mixture of inorganic silt and clay.	100 - 120	22 - 12	1.0	2.2	1350	460	32	0.62	$5 \times >10^{-7}$	.....	
CL	Inorganic clays of low to medium plasticity.	95 - 120	24 - 12	1.3	2.5	1800	270	28	0.54	$>10^{-7}$	15 or less	50 - 200
OL	Organic silts and silt-clays, low plasticity.	80 - 100	33 - 21	.....	.....	.....	.....	.....	.....	.....	5 or less	50 - 100
MH	Inorganic clayey silts, elastic silts.	70 - 95	40 - 24	2.0	3.8	1500	420	25	0.47	$5 \times >10^{-7}$	10 or less	50 - 100
CH	Inorganic clays of high plasticity	75 - 105	36 - 19	2.6	3.9	2150	230	19	0.35	$>10^{-7}$	15 or less	50 - 150
OH	Organic clays and silty clays	65 - 100	45 - 21	.....	.....	.....	.....	.....	.....	.....	5 or less	25 - 100

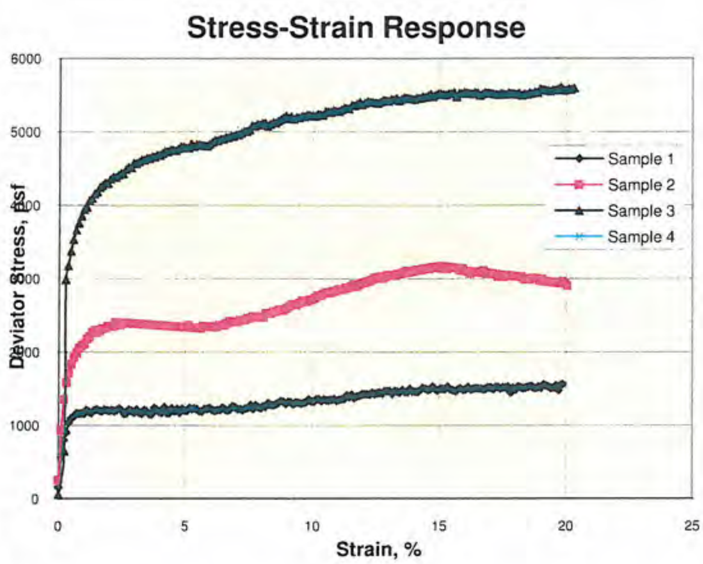
Notes:

1. All properties are for condition of "Standard Proctor" maximum density, except values of k and CBR which are for "modified Proctor" maximum density.
2. Typical strength characteristics are for effective strength envelopes and are obtained from USBR data.
3. Compression values are for vertical loading with complete lateral confinement.
4. (>) indicates that typical property is greater than the value shown. (..) indicates insufficient data available for an estimate.

Taken from:
Naval Facilities Engineering Command (NAVFAC), Foundations and Earth Structures, Design Manual 7.2, September 1986.

Triaxial Consolidated-Undrained
(ASTM D4767)





Sample:	1	2	3	4
MC, %	16.2	16.3	16.1	
Dry Dens, pcf	103.4	103.3	103.5	
Sat. %	69.5	70.0	69.4	
Void Ratio	0.629	0.631	0.628	
Diameter in	2.38	2.38	2.38	
Height, in	5.00	5.00	5.00	
	Final			
MC, %	24.7	22.8	21.3	
Dry Dens, pcf	101.1	104.3	107.0	
Sat. %	100.0	100.0	100.0	
Void Ratio	0.667	0.616	0.575	
Diameter, in	2.40	2.38	2.36	
Height, in	5.03	4.95	4.91	
Cell, psi	45.4	59.3	80.2	
BP, psi	38.9	38.2	39.1	
	Effective Stresses At:			
Strain, %	5.0	5.0	5.0	
Deviator ksf	1.201	2.339	4.790	
Excess PP	0.547	1.858	3.686	
Sigma 1	1.589	3.520	7.022	
Sigma 3	0.389	1.181	2.232	
P, ksf	0.989	2.350	4.627	
Q, ksf	0.600	1.169	2.395	
Stress Ratio	4.088	2.981	3.146	
Rate in/min	0.0005	0.0005	0.0005	
Total C	0.2	ksf		
Total Phi	15.3	Degrees		
Eff. C	0.1	ksf		
Eff. Phi	29.9	Degrees		

Job No.: 471-051 **Date:** 3/30/2010

Client: Cal Engineering & Geology BY:MD/DC

Project: 081070

Sample 1) Test Pits 1,2,4; Composite Brown Lean CLAY w/ Sand

Sample 2) Test Pits 1,2,4; Composite Brown Lean CLAY w/ Sand

Sample 3) Test Pits 1,2,4; Composite Brown Lean CLAY w/ Sand

Sample 4)

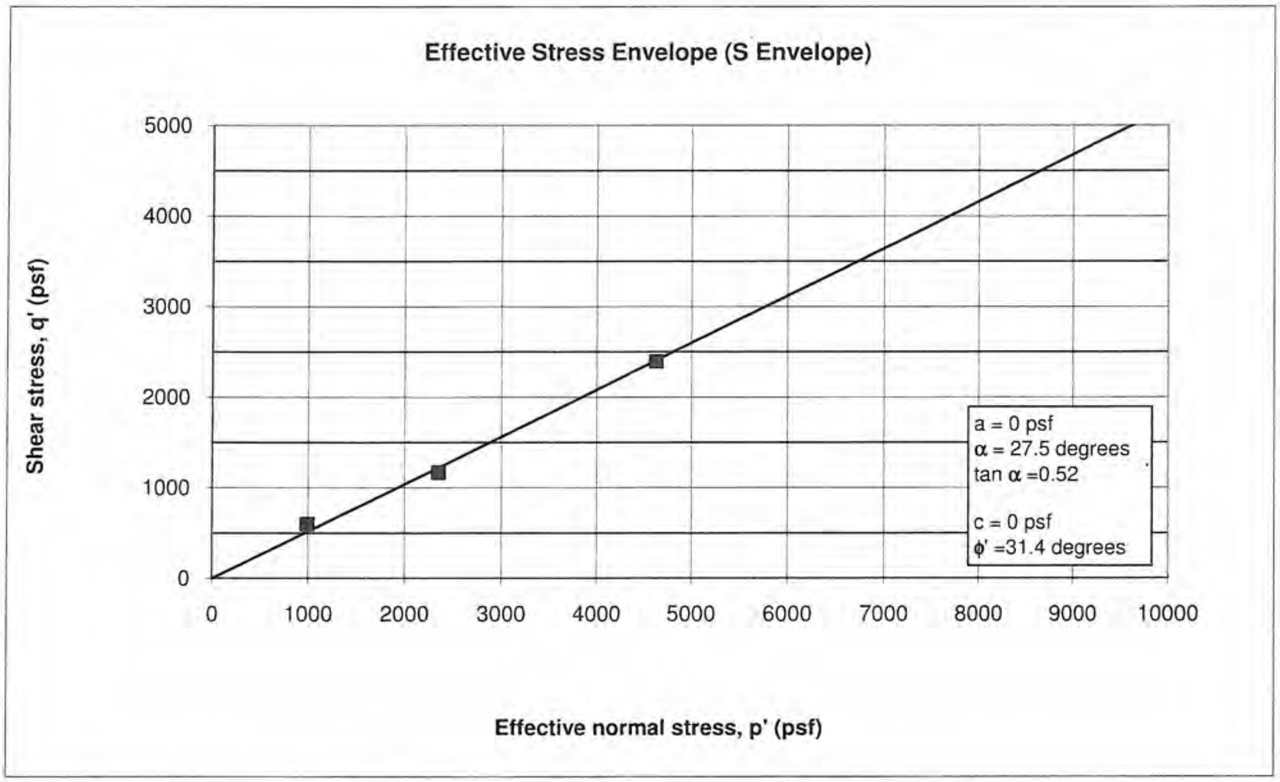
REMARKS: Strengths picked at 5% strain. Remolding Target= 97% of 106.7 @ 16.5 (OPT).

GEI Consultants	Client:	Contra Costa FC&WCD	Project: 08318-0	
	Subject:	CU Evaluation	Date: 4/9/10	By: D. DeCesaris
	Boring:	TP-1, -2, -4 Blend	Checked: 4/29/10	By: M. Quinn

Boring	Sample	Sample Depth (feet)	σ'_{3c} (psi)	$(\sigma_1\text{-}\sigma_3)_f$ (psi)	σ'_{1f} (psi)	σ'_{3f} (psi)	uf (psi)	p'_f (psf)	q_f (psf)
TP-1, -2, -4	1	--	6.5	8.3	11.0	2.7	3.8	990	601
	2	--	21.1	16.2	24.4	8.2	12.9	2350	1170
	3	--	41.1	33.3	48.8	15.5	25.6	4627	2395

$$p' = \frac{\sigma_1' + \sigma_3'}{2} \qquad q' = \frac{\sigma_1' - \sigma_3'}{2}$$

Effective Stress Strength Envelope (S-Envelope)



The slope of the "best fit line", α, is:	27.5	degrees
Sin ϕ' =	tan α	
Accordingly, the drained friction angle, ϕ', is:	31.4	degrees

	Client:	Contra Costa FC&WCD	Project:	08318-0		
GEI Consultants	Subject:	CU Evaluation	Date:	4/9/10	By:	D. DeCesaris
	Boring:	TP-1, -2, -4 Blend	Checked:	4/29/10	By:	M. Quinn

R-Envelope or Total Stress Envelope

Plot the maximum shear strength on the failure plane vs. the corresponding normal stress at failure.

Boring	Sample Depth (feet)	Sample	σ_{3c}' (psf)	$\sigma_1 - \sigma_3$ (psf)	τ_{max} (psf)	τ_{ff} (psf)	σ_f (psf)
TP-1, -2, -4	--	1	936	1201	601	513	1224
		2	3038	2339	1170	999	3599
		3	5918	4790	2395	2045	7067

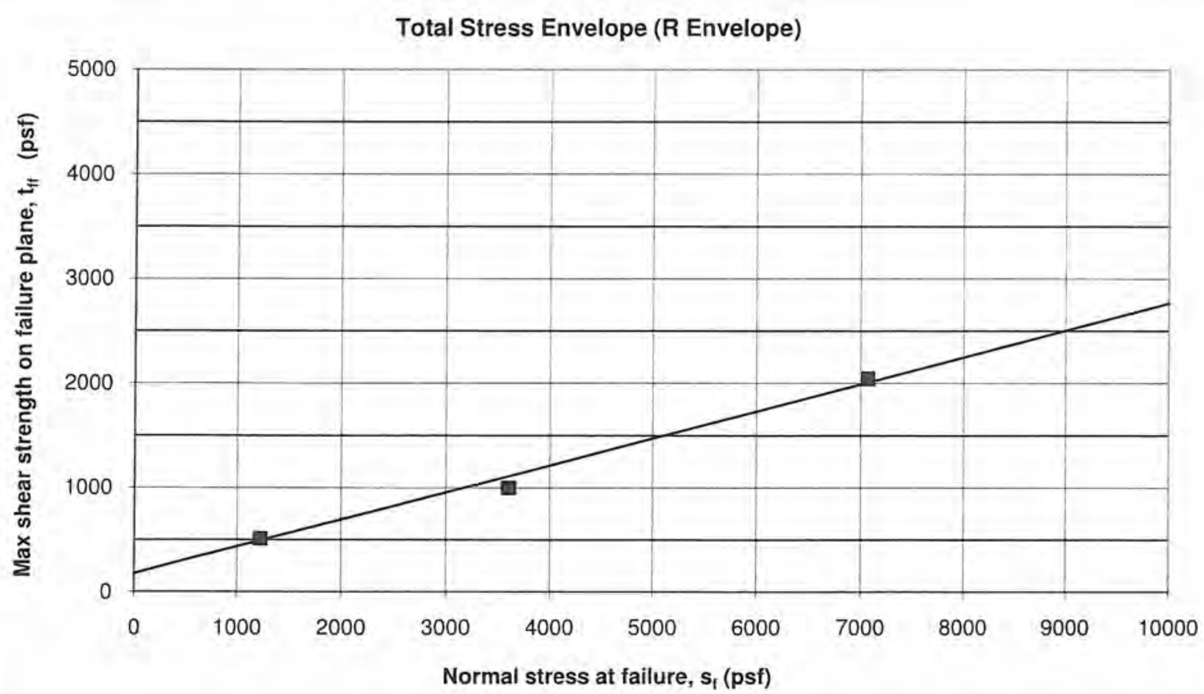
Maximum shear strength on the failure plane is determined from: where,

$$\tau_{ff} = \tau_{max} \cos\phi'$$

ϕ' = 31.4 degrees

Corresponding normal stress at failure is determined from:

$$\sigma_f = \sigma_{3c}' + \frac{\sigma_1 - \sigma_3}{2} - \tan(\phi') * \tau_{ff}$$



The R-Envelope is defined by: ϕ_R = 14.5 degrees

C_R = 180 psf

Develop the $K_c = 1$ Envelope for Rapid Drawdown Analysis.

The $K_c = 1$ envelope is estimated by plotting the same τ_{ff} value as the R-envelope but plotted against the consolidation pressure σ_{3c}' instead of σ_f.

Point	σ_{3c}' (psf)	τ_{ff} (psf)
1	936	513
2	3038	999
3	5918	2045

Similarly, the relationship between the R-envelope and K_c=1 envelope can be expressed from the following equations from page 155 of Duncan and Wright (2005), Soil Strength and Slope Stability,

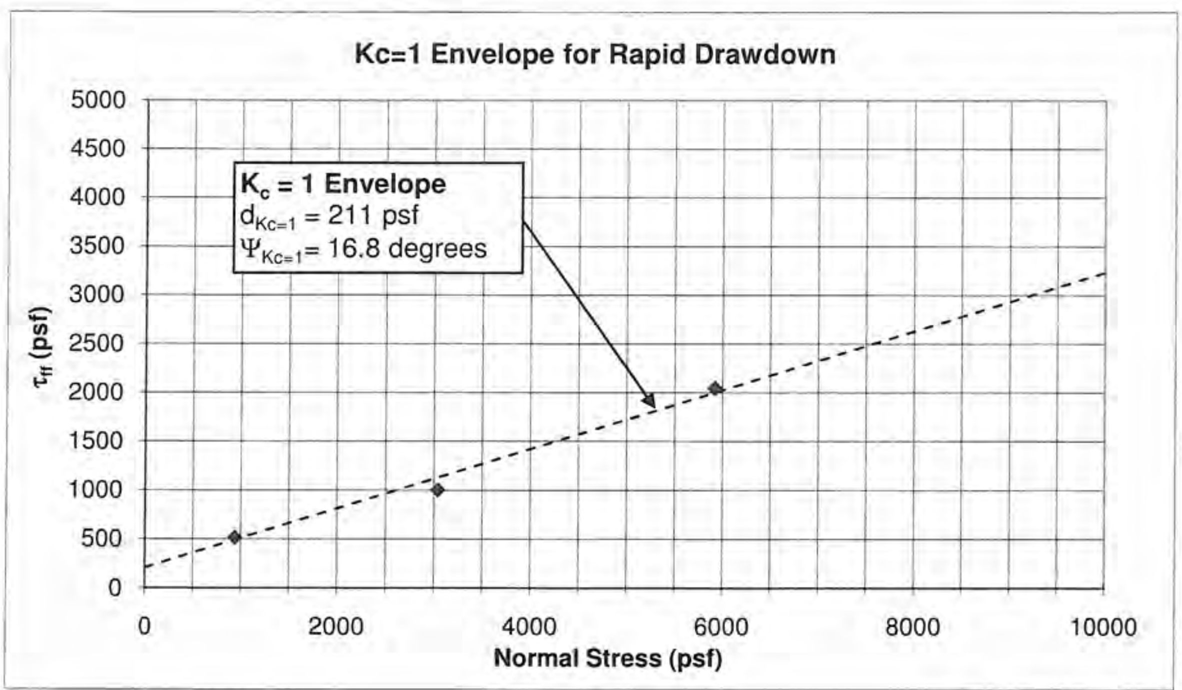
The K_c=1 envelope is defined by the slope $\Psi_{Kc=1}$ and the intercept $d_{Kc=1}$ which can be calculated from the following equations:

Eq. 9.8
$$d_{K_c=1} = \frac{c_R}{1+(\sin\phi'-1)\tan\phi_R/\cos\phi'}$$

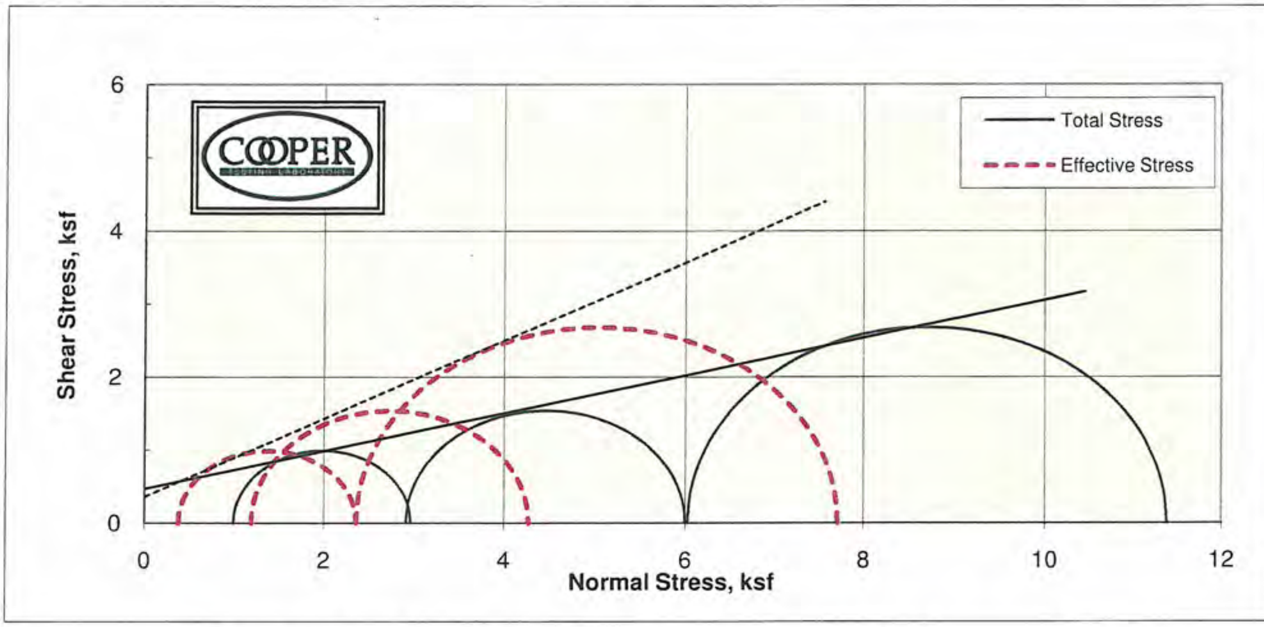
Eq. 9.9
$$\Psi_{K_c=1} = \arctan\frac{\tan(\phi_R)}{1+(\sin\phi'-1)\tan\phi_R/\cos\phi'}$$

From steps (1) and (2),

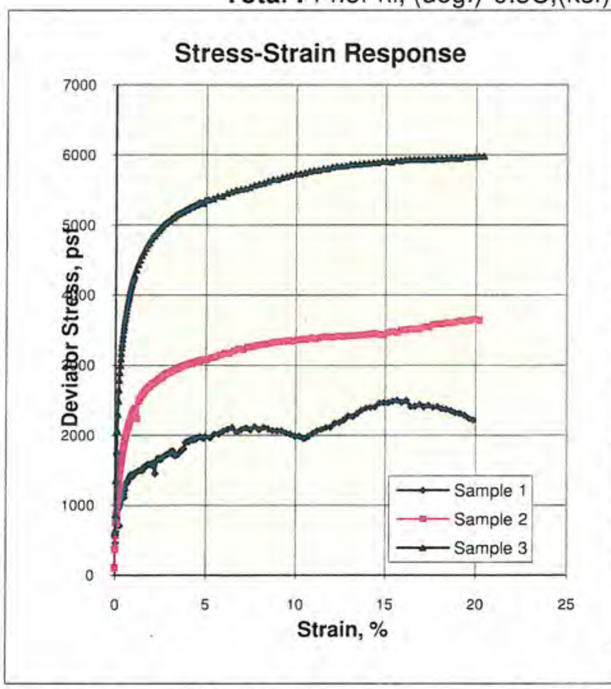
ϕ' =	31.4	degrees		therefore,		
c_R =	180	psf	⇨	$d_{Kc=1}$ =	211	psf
ϕ_R =	14.5	degrees		$\Psi_{Kc=1}$ =	16.8	degrees



Triaxial Consolidated Undrained with Pore Pressure
ASTM D4767



Total : 14.5Phi, (deg.) 0.5C,(ksf) **Effective:** 28.1Phi, (deg.) 0.4C,(ksf)



Sample:	1	2	3	4
MC, %	17.7	17.6	17.9	
DD, pcf	103.2	103.3	102.9	
Sat. %	73.9	73.7	74.3	
Void Ratio	0.657	0.656	0.661	
Diameter in	2.38	2.38	2.38	
Height, in	5.00	5.00	5.00	
		Final		
MC, %	25.0	23.0	21.6	
DD, pcf	101.5	104.9	107.4	
Sat. %	100.0	100.0	100.0	
Void Ratio	0.685	0.630	0.592	
Diameter, in	2.39	2.37	2.35	
Height, in	5.04	4.96	4.90	
Cell, psi	55.9	69.8	90.7	
BP, psi	49.0	49.5	48.9	
		Effective Stresses At:		
Strain, %	5.0	5.0	5.0	
Deviator ksf	1.982	3.078	5.357	
Excess PP	0.610	1.729	3.661	
Sigma 1	2.363	4.271	7.720	
Sigma 3	0.381	1.193	2.363	
P, ksf	1.372	2.732	5.041	
Q, ksf	0.991	1.539	2.678	
Stress Ratio	6.200	3.581	3.267	
Rate in/min	0.000	0.001	0.001	

Job No.: 471-051 **Date:** 4/5/2010

Client: Cal Engineering & Geology BY:DC

Project: 081070

Sample 1)	Test Pits 9,10,11;Composite	Brown Lean CLAY w/ Sand
Sample 2)	Test Pits 9,10,11;Composite	Brown Lean CLAY w/ Sand
Sample 3)	Test Pits 9,10,11;Composite	Brown Lean CLAY w/ Sand

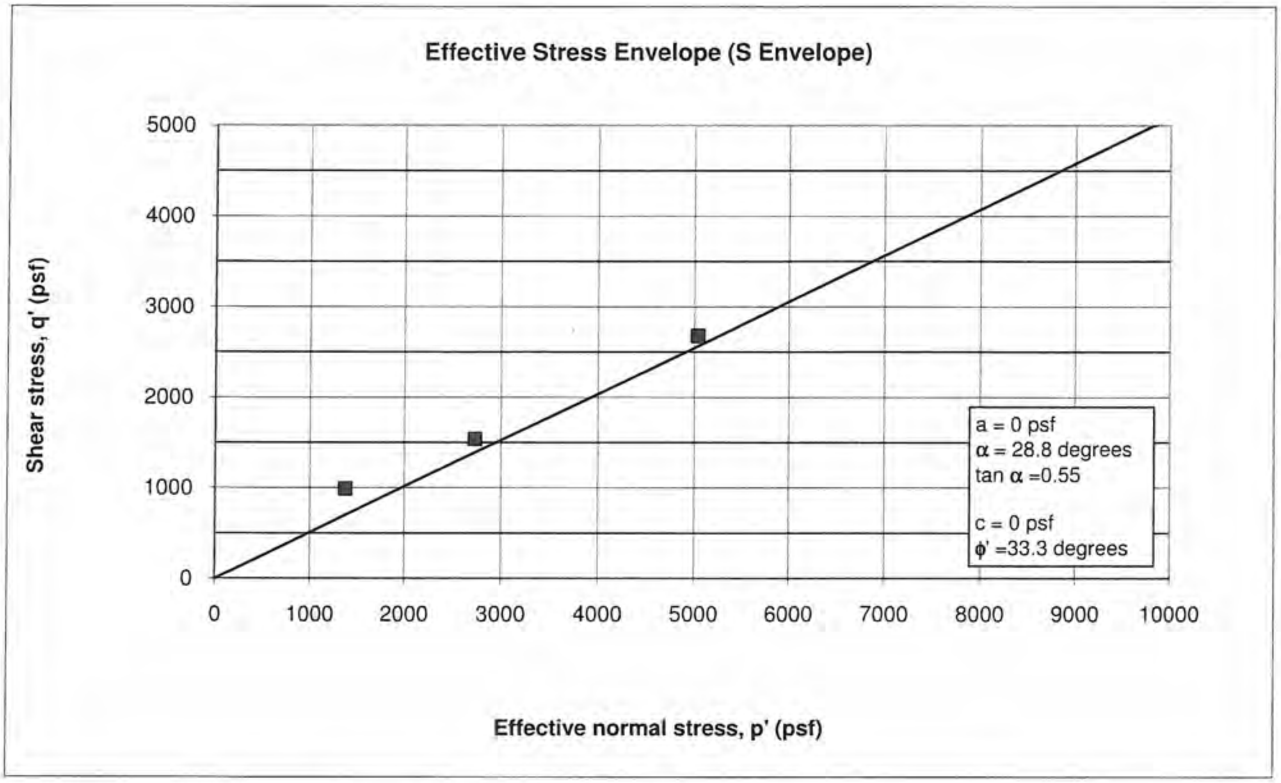
Remolding Target= 97% of 106.9pcf @ 17.7 (OPT).

GEI Consultants	Client: Contra Costa FC&WCD	Project: 08318-0	
	Subject: CU Evaluation	Date: 4/9/10	By: D. DeCesaris
	Boring: TP-9, -10, -11 Blend	Checked: 4/29/10	By: M. Quinn

Boring	Sample	Sample Depth (feet)	σ'_{3c} (psi)	$(\sigma_1-\sigma_3)_f$ (psi)	σ'_{1f} (psi)	σ'_{3f} (psi)	uf (psi)	p'_f (psf)	q_f (psf)
TP-9, -10, -11	1	--	6.9	13.8	16.4	2.7	4.2	1375	991
	2	--	20.3	21.4	29.7	8.3	12.0	2733	1539
	3	--	41.8	37.2	53.6	16.4	25.4	5037	2679

$$p' = \frac{\sigma_1' + \sigma_3'}{2} \qquad q' = \frac{\sigma_1' - \sigma_3'}{2}$$

Effective Stress Strength Envelope (S-Envelope)



The slope of the "best fit line", α, is:	27.0	degrees
Sin ϕ' =	tan α	
Accordingly, the drained friction angle, ϕ', is:	30.6	degrees

GEI Consultants	Client:	Contra Costa FC&WCD	Project:	08318-0		
	Subject:	CU Evaluation	Date:	4/9/10	By:	D. DeCesaris
		Boring: TP-9, -10, -11 Blend	Checked:	4/29/10	By:	M. Quinn

R-Envelope or Total Stress Envelope

Plot the maximum shear strength on the failure plane vs. the corresponding normal stress at failure.

Boring	Sample Depth (feet)	Sample	σ_{3c}' (psf)	$\sigma_1 - \sigma_3$ (psf)	τ_{max} (psf)	τ_{ff} (psf)	σ_f (psf)
TP-9, -10, -11	--	1	994	1982	991	853	1480
		2	2923	3078	1539	1324	3678
		3	6019	5357	2679	2305	7333

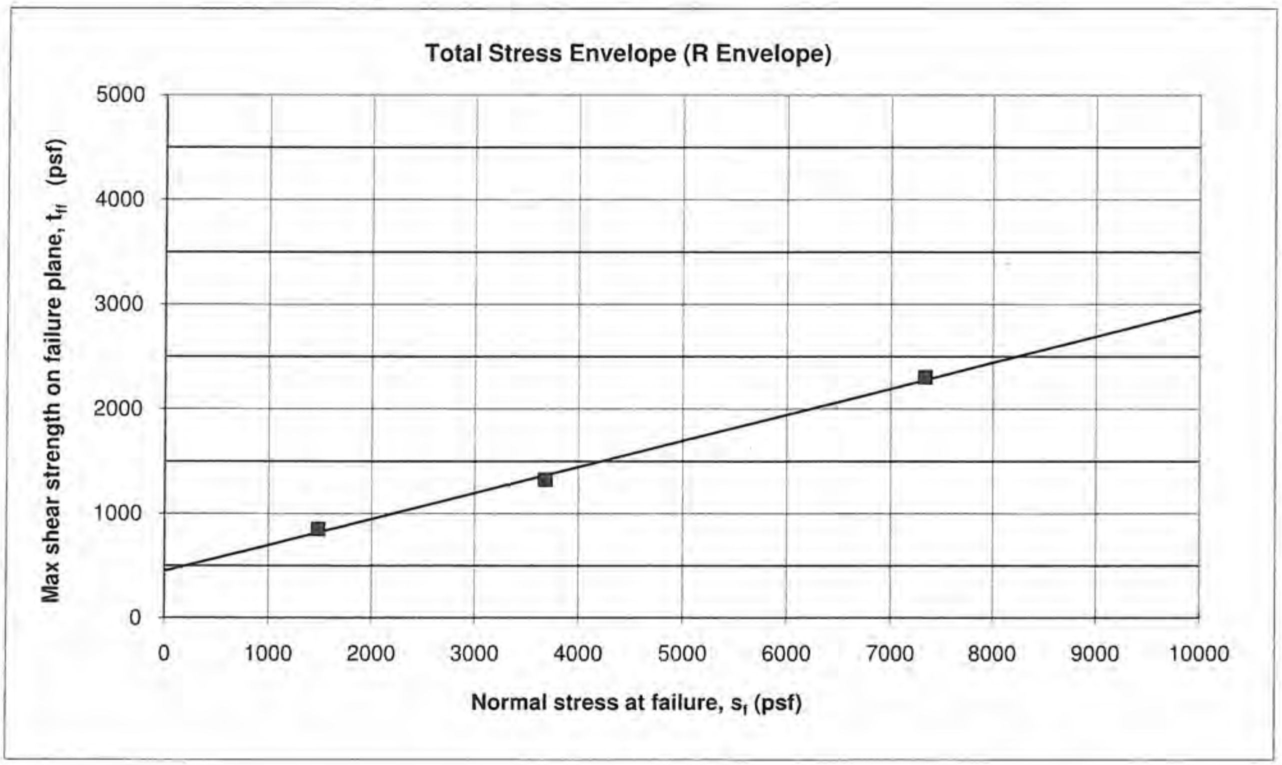
Maximum shear strength on the failure plane is determined from: where,

$$\tau_{ff} = \tau_{max} \cos\phi'$$

ϕ' = 30.6 degrees

Corresponding normal stress at failure is determined from:

$$\sigma_f = \sigma_{3c}' + \frac{\sigma_1 - \sigma_3}{2} - \tan(\phi') * \tau_{ff}$$



The R-Envelope is defined by: ϕ_R = 14.0 degrees

c_R = 450 psf

Develop the $K_c = 1$ Envelope for Rapid Drawdown Analysis.

The $K_c = 1$ envelope is estimated by plotting the same τ_{ff} value as the R-envelope but plotted against the consolidation pressure σ_{3c}' instead of σ_f.

Point	σ_{3c}' (psf)	τ_{ff} (psf)
1	994	853
2	2923	1324
3	6019	2305

Similarly, the relationship between the R-envelope and K_c=1 envelope can be expressed from the following equations from page 155 of Duncan and Wright (2005), Soil Strength and Slope Stability,

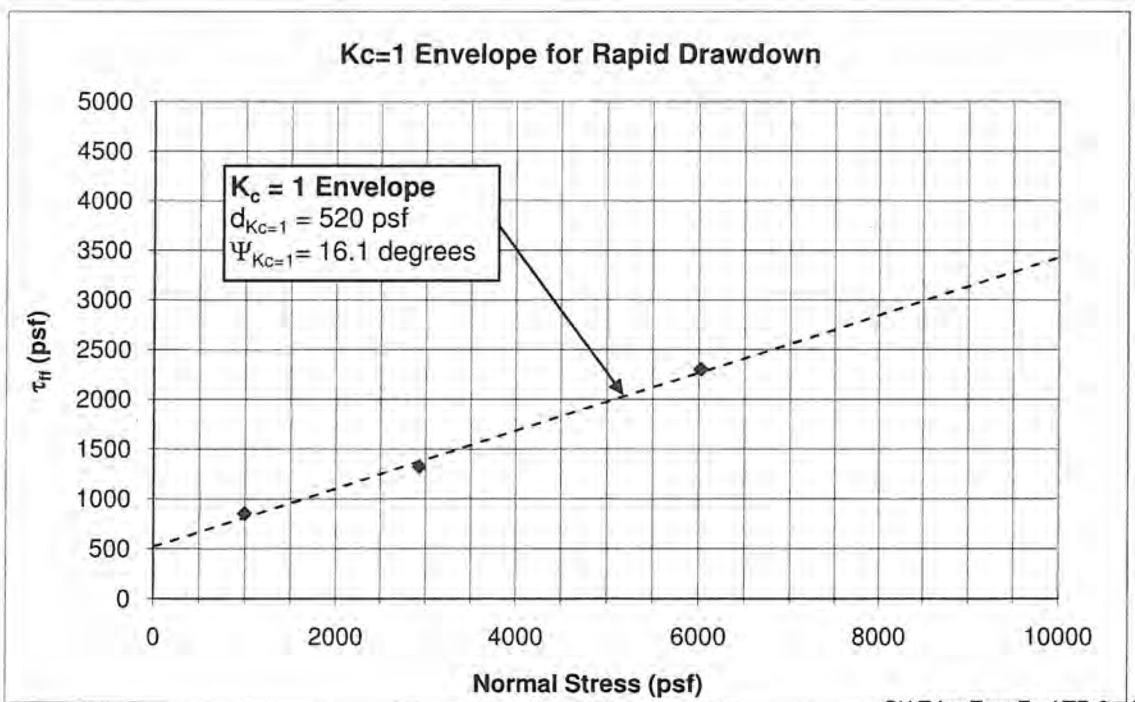
The K_c=1 envelope is defined by the slope $\Psi_{Kc=1}$ and the intercept $d_{Kc=1}$ which can be calculated from the following equations:

Eq. 9.8 $$d_{K_c=1} = \frac{c_R}{1+(\sin\phi'-1)\tan\phi_R/\cos\phi'}$$

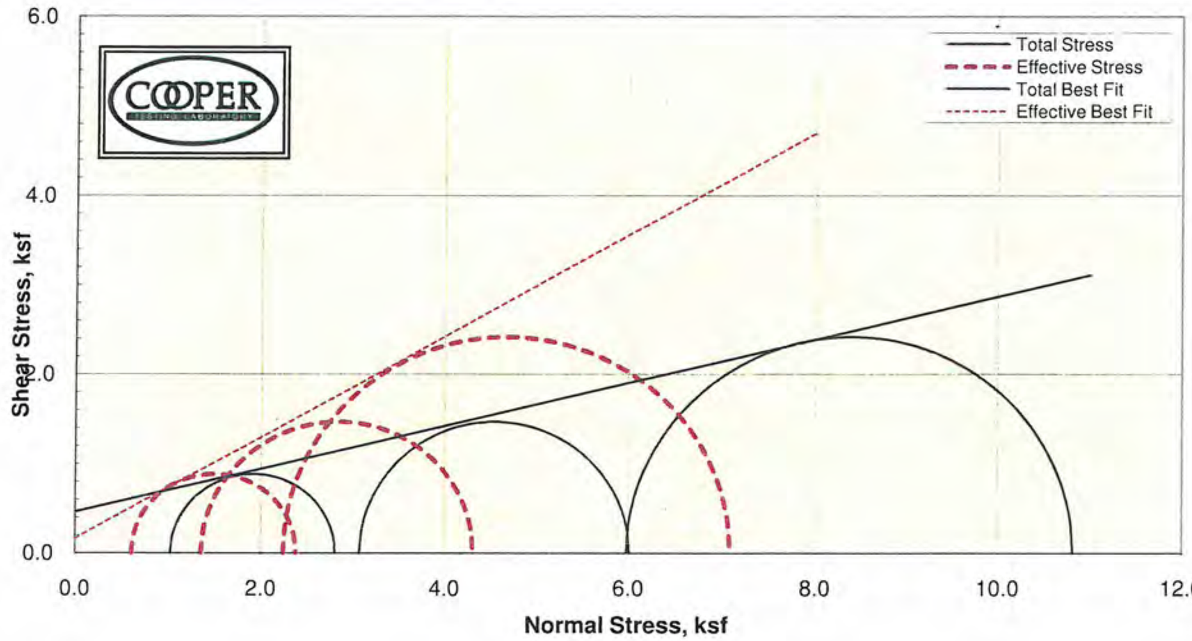
Eq. 9.9 $$\Psi_{K_c=1} = \arctan\frac{\tan(\phi_R)}{1+(\sin\phi'-1)\tan\phi_R/\cos\phi'}$$

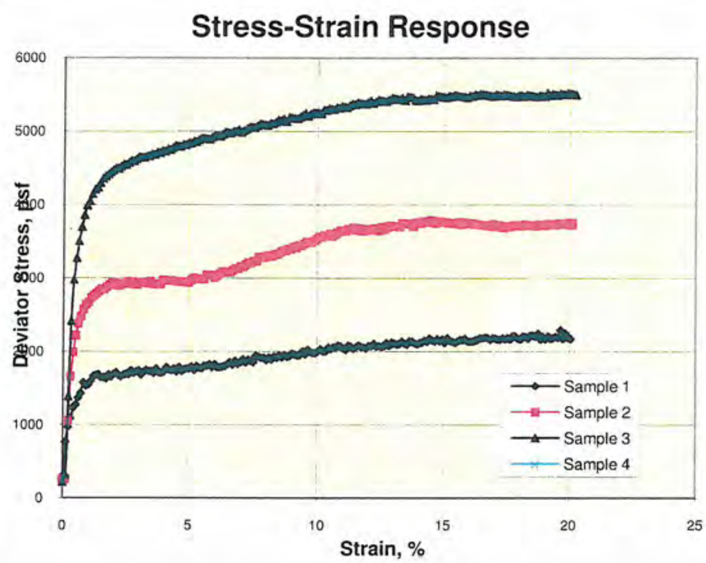
From steps (1) and (2),

ϕ' =	30.6	degrees		therefore,		
c_R =	450	psf	⇨	$d_{Kc=1}$ =	525	psf
ϕ_R =	14.0	degrees		$\Psi_{Kc=1}$ =	16.2	degrees



Triaxial Consolidated-Undrained
(ASTM D4767)





Sample:	1	2	3	4
MC, %	18.5	18.6	18.7	
Dry Dens, pcf	102.7	102.5	102.5	
Sat. %	76.9	76.9	77.1	
Void Ratio	0.658	0.661	0.663	
Diameter in	2.38	2.38	2.38	
Height, in	5.00	5.00	5.00	
		Final		
MC, %	23.8	22.0	21.6	
Dry Dens, pcf	103.3	106.4	107.1	
Sat. %	100.0	100.0	100.0	
Void Ratio	0.650	0.600	0.590	
Diameter, in	2.37	2.34	2.34	
Height, in	4.99	4.95	4.91	
Cell, psi	65.9	79.8	90.7	
BP, psi	58.7	58.5	49.2	
		Effective Stresses At:		
Strain, %	5.0	5.0	5.0	
Deviator ksf	1.766	2.940	4.842	
Excess PP	0.418	1.699	3.730	
Sigma 1	2.385	4.308	7.088	
Sigma 3	0.619	1.368	2.246	
P, ksf	1.502	2.838	4.667	
Q, ksf	0.883	1.470	2.421	
Stress Ratio	3.852	3.149	3.155	
Rate in/min	0.001	0.001	0.001	
Total C	0.5	ksf		
Total Phi	13.5	Degrees		
Eff. C	0.2	ksf		
Eff. Phi	29.6	Degrees		

Job No.:	471-051	**Date:**	4/6/2010
Client:	Cal Engineering & Geology		BY:MD/DC
Project:	081070		
Sample 1)	Test Pits 20,12,26;C Brown Sandy Lean CLAY		
Sample 2)	Test Pits 20,12,26;C Brown Sandy Lean CLAY		
Sample 3)	Test Pits 20,12,26;C Brown Sandy Lean CLAY		
Sample 4)			

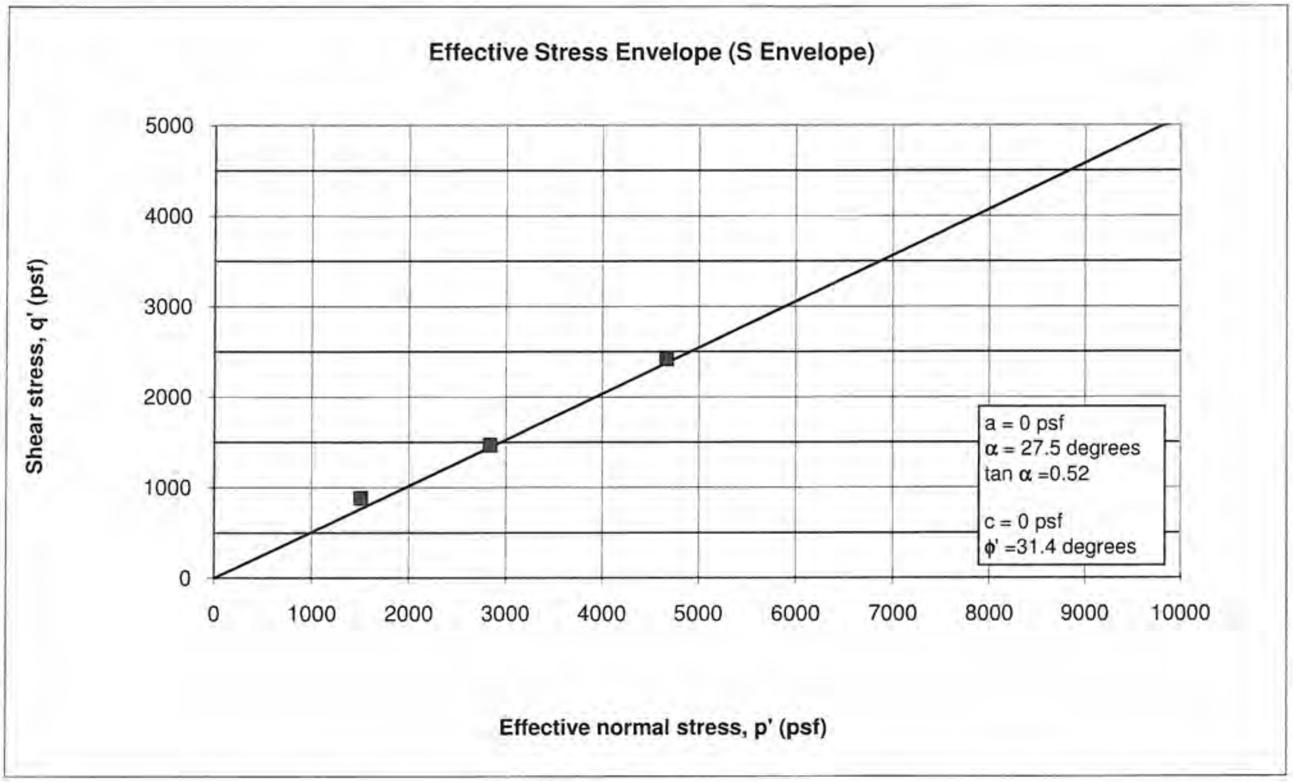
REMARKS: Strengths picked at 5% strain. Remolding Target= 97% of 106.6 pcf @ 18.1 (OPT).

	Client:	Contra Costa FC&WCD	Project:	08318-0	
GEI Consultants	Subject:	CU Evaluation	Date:	4/9/10	By: D. DeCesaris
		Boring: TP-20, -12, -26 Blend	Checked:	4/29/10	By: M. Quinn

Boring	Sample	Sample Depth (feet)	σ'_{3c} (psi)	$(\sigma_1-\sigma_3)_f$ (psi)	σ'_{1f} (psi)	σ'_{3f} (psi)	uf (psi)	p'_f (psf)	q_f (psf)
TP-20, -12, -26	1	--	7.2	12.3	16.6	4.3	2.9	1502	883
	2	--	21.3	20.4	29.9	9.5	11.8	2838	1470
	3	--	41.5	33.6	49.2	15.6	25.9	4667	2421

$$p' = \frac{\sigma_1' + \sigma_3'}{2} \qquad q' = \frac{\sigma_1' - \sigma_3'}{2}$$

Effective Stress Strength Envelope (S-Envelope)



The slope of the "best fit line", α, is:	27.0	degrees
Sin ϕ' =	tan α	
Accordingly, the drained friction angle, ϕ', is:	30.6	degrees

R-Envelope or Total Stress Envelope

Plot the maximum shear strength on the failure plane vs. the corresponding normal stress at failure.

Boring	Sample Depth (feet)	Sample	σ_{3c}' (psf)	$\sigma_1 - \sigma_3$ (psf)	τ_{max} (psf)	τ_{ff} (psf)	σ_f (psf)
TP-20, -12, -26	--	1	1037	1766	883	760	1470
		2	3067	2940	1470	1265	3788
		3	5976	4842	2421	2083	7163

Maximum shear strength on the failure plane is determined from:

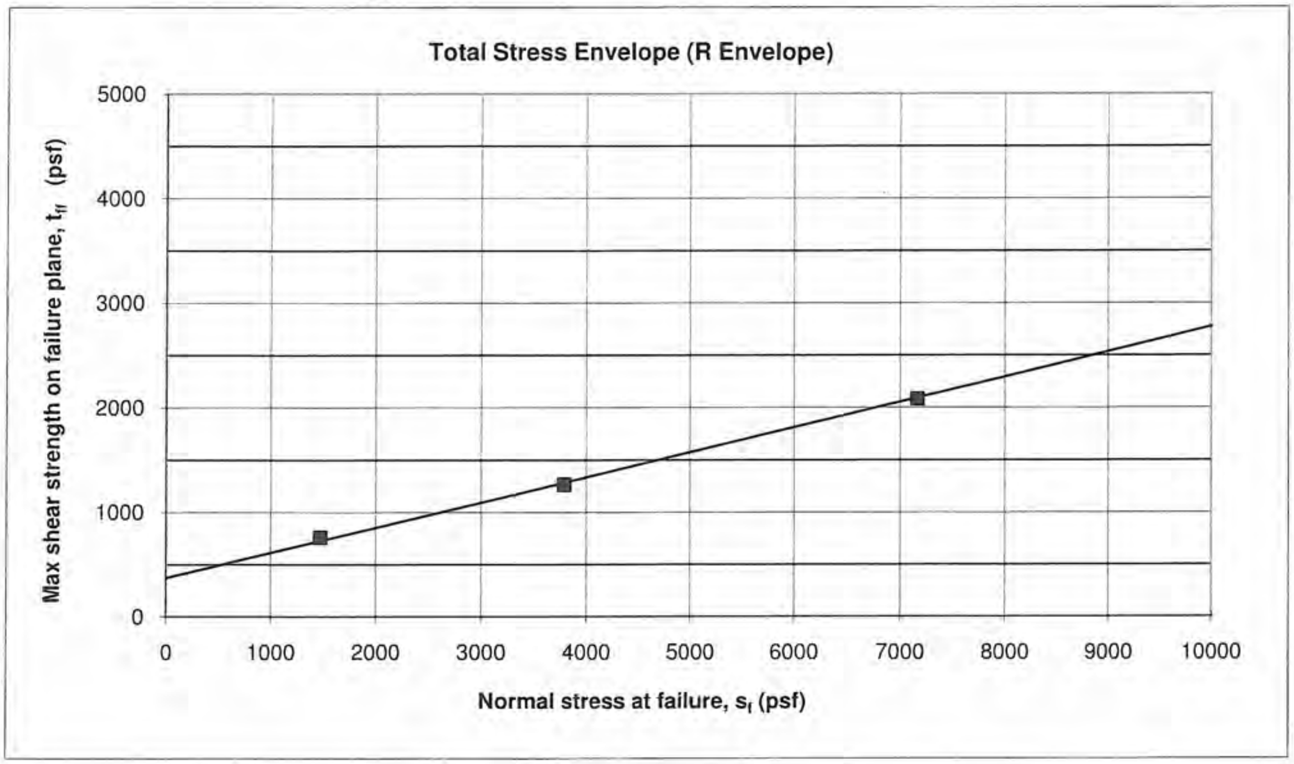
$$\tau_{ff} = \tau_{max} \cos \phi'$$

where,

ϕ' = 30.6 degrees

Corresponding normal stress at failure is determined from:

$$\sigma_f = \sigma_{3c}' + \frac{\sigma_1 - \sigma_3}{2} - \tan(\phi') * \tau_{ff}$$



The R-Envelope is defined by:

ϕ_R = 13.5 degrees

c_R = 375 psf

Develop the $K_c = 1$ Envelope for Rapid Drawdown Analysis.

The $K_c = 1$ envelope is estimated by plotting the same τ_{ff} value as the R-envelope but plotted against the consolidation pressure σ_{3c}' instead of σ_f.

Point	σ_{3c}' (psf)	τ_{ff} (psf)
1	1037	760
2	3067	1265
3	5976	2083

Similarly, the relationship between the R-envelope and K_c=1 envelope can be expressed from the following equations from page 155 of Duncan and Wright (2005), Soil Strength and Slope Stability,

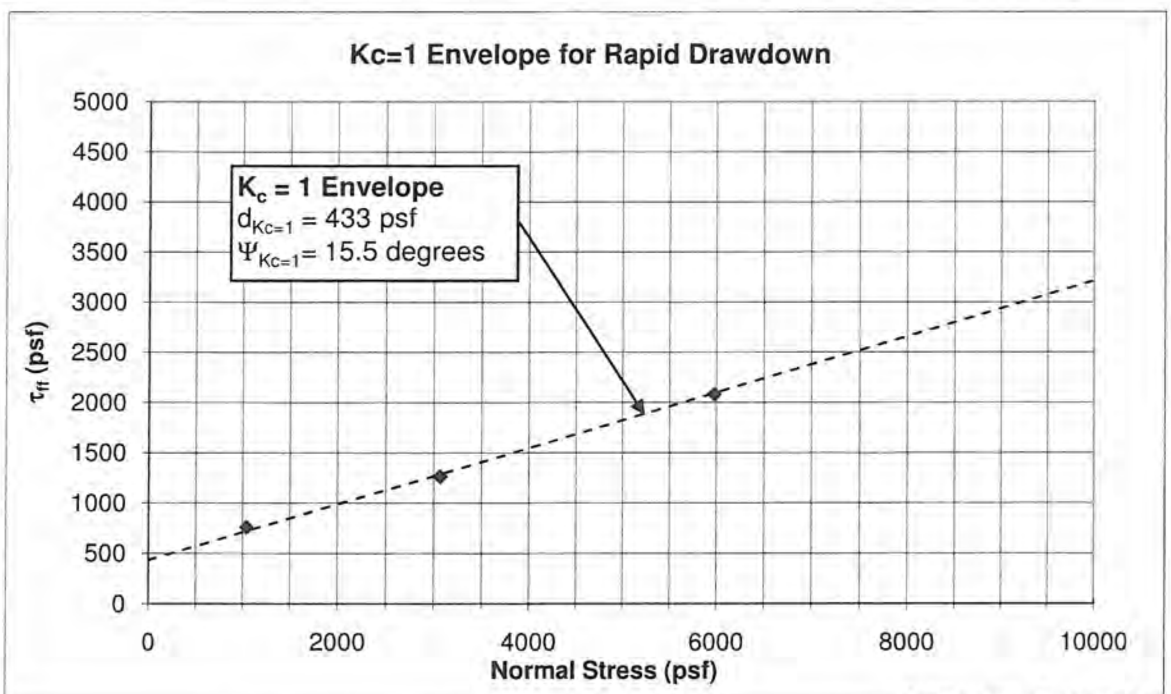
The K_c=1 envelope is defined by the slope $\Psi_{Kc=1}$ and the intercept $d_{Kc=1}$ which can be calculated from the following equations:

Eq. 9.8 $$d_{K_c=1} = \frac{c_R}{1 + (\sin\phi' - 1)\tan\phi_R / \cos\phi'}$$

Eq. 9.9 $$\Psi_{K_c=1} = \arctan\frac{\tan(\phi_R)}{1 + (\sin\phi' - 1)\tan\phi_R / \cos\phi'}$$

From steps (1) and (2),

ϕ' =	30.6	degrees		therefore,		
c_R =	375	psf	⇨	$d_{Kc=1}$ =	434	psf
ϕ_R =	13.5	degrees		$\Psi_{Kc=1}$ =	15.5	degrees



Attachment 3 – Foundation Undrained Shear Strengths

Undrained Shear Strength Estimates of Foundation Soils

Problem:

Select representative undrained shear strength values for the foundation soils at cross sections being evaluated for slope stability. Use historic and recent subsurface exploration and geotechnical lab testing.

References:

(1) Naval Facilities Engineering Command (NAVFAC), 1986, Soil Mechanics, Design Manual 7.1, September.

(2) Cal Engineering & Geology, Inc., *Geotechnical Data Report, Contra Costa County Flood Control & Water Conservation District, Upper Sand Creek Detention Basin, Deer Creek Road, Antioch, California,* May 5, 2010.

Summary:

The selected undrained shear strengths for use in our stability analysis are summarized below:

Soil Layer	Undrained Shear Strength
ML (Downstream Above El 170	1,000 psf
CL (Ground Surface to El. 152)	1,600 psf
ML/CL-1 Layer (~ El 152 to 144)	1,000 psf
ML/CL-2 Layer (~ El 133 to 129)	1,400 psf
Bedrock	2,300 psf

Foundation Undrained Shear Strengths

Approach:

Undrained shear strengths in the fine-grained soils (silts and clays) along the dam alignment were estimated from several sources of data including:

- Recent Laboratory CU tests
- Historic Laboratory CU, UU, and UC tests
- Empirical Correlation to CPT Soundings
- Empirical Correlation to SPT N-Value

Recent Laboratory CU Tests

CE&G engaged Cooper Testing Laboratories to complete three sets of three-point (where each point is on a different specimen) laboratory CU tests. For each series we estimated effective stress, total stress, and K_c=1 failure envelopes.

For each CU test series, we estimated the existing overburden effective stress and the maximum past pressure. Where consolidation test data were available, maximum past pressure from the consolidation test evaluation was used. We used the greater of these two stresses and the K_c=1 envelope to estimate an undrained shear strength at the appropriate consolidation pressure.

Summaries of each set of CU tests and our estimated undrained shear strengths are shown in the table below. Our interpretation of each test is attached.

Boring Number	Depth Interval (feet)	USCS	Sample Information				Shear Strengths							
			Specimen	Water Content (%)	Dry Unit Weight (pcf)	Moist Unit Weight (pcf)	Effective Stress c (psf)	Effective Stress φ' (psf)	Total Stress c (psf)	Total Stress φ (psf)	Kc=1[1] c (psf)	Kc=1[1] φ (psf)	Estimated S_u (psf)	Model Soil Layer
B-7	21.5'	CL	1	29.2	94.2	121.7	0	32.1	400	16.0	475	18.8	2009	CL
	21.0'	CL	2	25.1	100.4	125.6								
	20.5'	CL	3	22.0	105.7	129.0								
B-9	16.3'	CL	1	32.5	90.6	120.0	0	30.3	350	17.0	425	20.3	2093	CL
	15.8'	CL	2	31.8	91.6	120.7								
	15.3'	CL	3	28.2	96.7	124.0								
B-10	20.4'	CL	1	23.6	102.9	127.2	0	32.9	875	11.5	984	12.9	2036	CL
	19.8'	CL	2	22.9	104.1	127.9								
	19.3'	CL	3	22.0	105.7	129.0								

Foundation Undrained Shear Strengths

Historic Laboratory Strength Tests

Additional laboratory CU, UU, and UC testing data is available from previous studies on the project site completed by William Lettis & Associates (2004) and Fugo West (2004). The Fugro CU test envelope was developed from a staged test on one specimen, and (in accordance with DSOD review comments) will not be used. A summary of the UU and UC tests, estimated undrained shear strengths, and their associated layers in our subsurface stratigraphy model are shown on the tables below.

Historic UU Tests

Boring Number	Depth Interval (feet)	USCS	Sample Information			Peak Deviator Stress (psf)	Strain at Failure (%)	Estimated S_u (psf)	Soil Model Layer
			Water Content (%)	Dry Unit Weight (pcf)	Moist Unit Weight (pcf)				
B-1	20.0'	CL	22.2	99.8	122.0	3,520	13.3	1,760	ML/CL-1
B-2	20.5'	CL/ML	17.8	104.3	122.9	2,278	3.6	1,139	CL
B-2	40.5'	CL	33.1	89.7	119.4	1,983	12.5	992	ML/CL-1
B-3	15.5'	CL	19.0	104.7	124.6	7,045	4.0	3,523	CL
B-4	21.0'	CL	23.6	103.1	127.4	3,288	13.9	1,644	CL
B-4	26.0'	SM	21.6	108.8	132.3	6,104	3	3,052	CL

Historic UC Tests

Boring Number	Depth Interval (feet)	USCS	Sample Information			Unconfined Strength (psf)	Strain at Failure (%)	Estimated S_u (psf)	Soil Model Layer
			Water Content (%)	Dry Unit Weight (pcf)	Moist Unit Weight (pcf)				
B-2	60.5'	CL	27.8	96.1	122.8	2,140	15	1,070	ML/CL-1
C-7-3	16.1'	SM	23.6	98.7	122.0	1,442	3.5	721	CL

Historic UC Tests

Boring Number	Depth Interval (feet)	USCS	Sample Information			Unconfined Strength (psf)	Strain at Failure (%)	Estimated S_u (psf)	Soil Model Layer
			Water Content (%)	Dry Unit Weight (pcf)	Moist Unit Weight (pcf)				
C-1-3	24.8'	SM(1)	16.0	111.2	129.0	40,760	5.5	20,380	Bedrock
C-1-4	42.1'	SM(1)	15.3	110.9	127.9	17,431	4.3	8,716	Bedrock
C-1-6	56.1'	SM(2)	17.7	111.9	131.7	33,137	5.2	16,569	Bedrock
C-4-3	21.0'	ML(1)	18.7	111.5	132.4	35,852	8.8	17,926	Bedrock
C-6-3	36.0'	ML(3)	16.8	113.1	132.1	17,069	5.7	8,535	Bedrock
C-6-5	44.7'	ML(3)	19	109.8	130.7	4,618	3.3	2,309	Bedrock

Notes:

1. Sample taken in w eathered bedrock (Weathered T_{mk}).
2. Sample taken in bedrock (T_{mk}).
3. Sample taken in ancient landside debris (Q_{lso}).

Cone Penetrometer Soundings

We estimated undrained shear strengths using the Fugro (2004) cone penetrometer data based on the following formula as defined in Lunne et. al. (1997):

$$S_u = \frac{q_t - \sigma_{vo}}{N_{kt}}$$

where N_{kt} is a constant ranging from 10 to 20. Since no site specific N_{kt} has been previously established, we estimated an undrained shear strength using an N_{kt} value of 15.

We evaluated data from CPT-3, CPT-4, and CPT-6, which were located near the analysis sections. A summary of our estimated undrained shear strengths are presented in the tables below. Plots of our interpretation of the cone data vs. depth are attached.

CPT-3

Depth	Soil Layer	Average q_t (tsf)	Average σ_v (tsf)	Undrained Shear Strength (tsf)
0 – 20 ft	CL (upper layer)	55 tsf	0.63 tsf	Su = (55-0.63)/15 = 3.6 tsf
20 – 35 ft	CL (lower layer)	15 tsf	1.7 tsf	Su = (15-1.7)/15 = 0.89 tsf

CPT-4

Depth	Soil Layer	Average q_t (tsf)	Average σ_v (tsf)	Undrained Shear Strength (tsf)
0 – 6 ft	CL (upper layer)	35 tsf	0.19 tsf	Su = (35-0.19)/15 = 2.3 tsf
6 – 30 ft	CL (lower layer)	20 tsf	1.1 tsf	Su = (20-1.1)/15 = 1.26 tsf

CPT-6

Depth	Soil Layer	Average q_t (tsf)	Average σ_v (tsf)	Undrained Shear Strength (tsf)
0 – 8 ft	ML	70 tsf	0.25 tsf	Su = (70-0.25)/15 = 4.7 tsf
8 – 15 ft	SC	30 tsf	0.72 tsf	Su = (30-0.72)/15 = 1.9 tsf
15 – 32 ft	CL	20 tsf	1.5 tsf	Su = (20-1.5)/15 = 1.2 tsf
32 – 37 ft	ML/CL-1	40 tsf	2.2 tsf	Su = (40-2.2)/15 = 2.5 tsf
37 –50 ft	ML/CL-2	150 tsf	2.7 tsf	Su = (150-2.7)/15 = 9.8 tsf

Foundation Undrained Shear Strengths

Correlation to SPT N-Value

We also estimated undrained shear strength using a correlation from NAVFAC (1986) for fine-grained soils with PI > 8. A friction angle of zero was assumed.

Location	Boring	USCS	N
CL - Ground Surface to ~El. 152			
Crest	B-3-09	CL	11
Crest	B-3-09	CL	19
Crest	B-3-09	CL	12
Crest	B-3-09	CL/ML	11
Crest	B-3-09	CL	17
LS Slope	B-4-09	CL	18
LS Slope	B-4-09	CL	12
LS Slope	B-4-09	ML	19
LS Slope	B-4-09	CL	14
LS Slope	B-4-09	CL	17
WS Toe	B-7-09	CL	35
WS Toe	B-7-09	CL	16
WS Toe	B-7-09	CL	17
WS Toe	B-7-09	CL	13
WS Toe	B-7-09	CL	11
WS Toe	B-7-09	CL	11
WS Toe	B-8-09	CL	6
WS Toe	B-8-09	CL	2
WS Toe	B-8-09	CL	8
LS Toe	B-9-09	CL	17
LS Toe	B-9-09	ML	8
LS Sope	B-10-09	CL	14
		Average =	14 bpf
		Assumed Typical Value =	11 bpf
		Estimated Su =	1600 psf

Location	Boring	USCS	N
ML/CL-1 - ~El. 144 to 150			
Crest	B-3-09	CL	20
WS Toe	B-7-09	CL	8
WS Toe	B-8-09	ML	14
WS Toe	B-8-09	CL	16
LS Toe	B-9-09	CL	11
LS Slope	B-10-09	CL	22
LS Slope	B-10-09	CL	14
		Average =	15 bpf
		Assumed Typical Value =	12 bpf
		Estimated Su =	850 psf

Location	Boring	USCS	N
ML - Above El. 170 (Downstream)			
LS Toe	B-9-09	ML	11
		Estimated Su =	800 psf

Location	Boring	USCS	N
ML/CL-2 - ~El. 133 to 129			
Crest	B-3-09	Siltstone	35
WS Toe	B-7-09	ML	8
LS Toe	B-9-09	CL	40
		Estimated Su =	28 bpf
		Assumed Typical Value =	18 bpf
		Estimated Su =	1400 psf

Foundation Undrained Shear Strengths

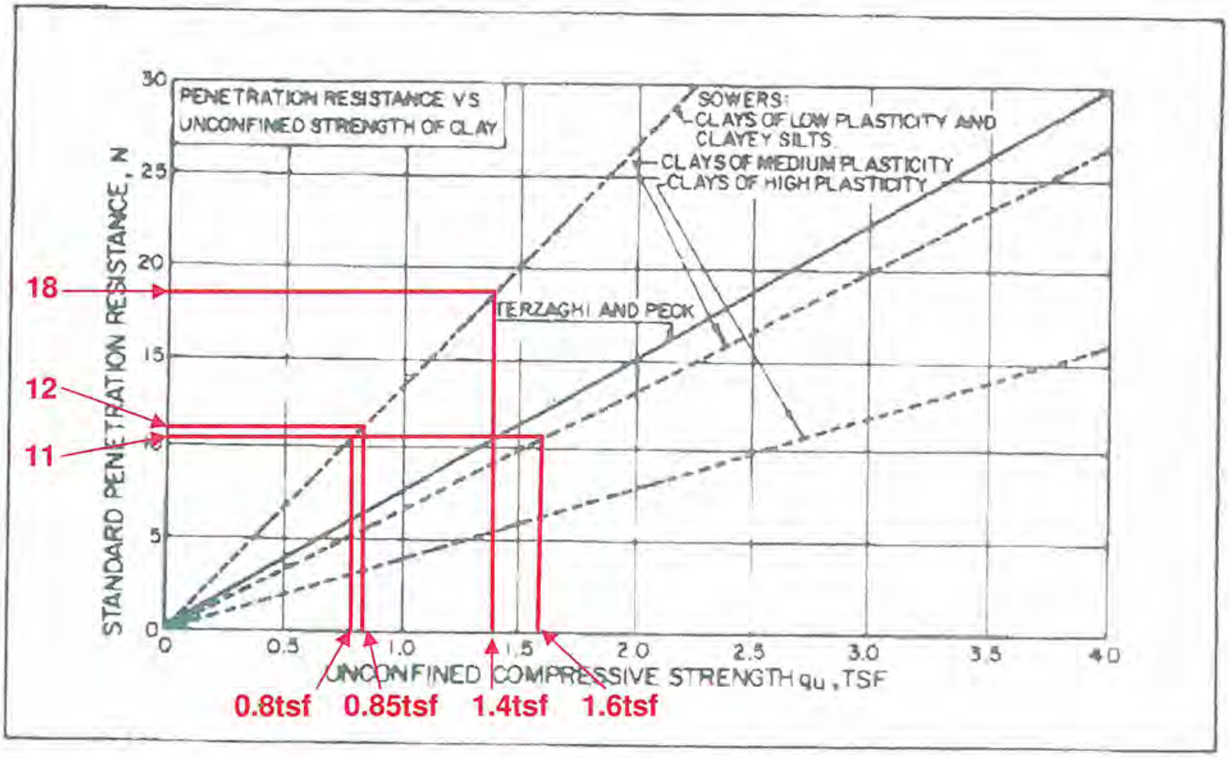


FIGURE 4
Correlations of Standard Penetration Resistance

Discussion:

The correlated and measured undrained strength data were used to develop strength parameters for each fine-grained soil stratum.

CL Layer (Ground Surface to ~El 152)

The laboratory data on samples from the CL stratum indicate undrained shear strengths could range from 720 to 3,500 psf with typical values in the 1,600 to 2,100 psf range. We gave more weight to the CU test results than the UU and UC tests because of the potentially greater effect of sample disturbance on the results of UU and UC test results for fine grained soils. Additionally, for the fine-grained soils, we weighted the shear strength estimates from CPT data greater than the correlated SPT data because SPT blow counts tend to be a relatively poor indicator of undrained shear strength for fine-grained soils. Estimates of undrained shear strength from the CPT data indicate the range could be from 1,720 to about 2,500 psf with a stiffer zone of 4,600 to 7,200 psf near the surface. The assumed typical SPT N-value in this layer results in an estimated undrained shear strength of 1600 psf.

We will use a value of 1,600 psf for the design calculations which is based on the lower range of the typical values measured in the laboratory.

ML/CL-1 Layer (~ El 152 to 144)

The laboratory UU and UC test data on samples from the ML/CL-1 stratum indicate undrained shear strengths could range from 1,000 to 1,800 psf. Estimates of undrained shear strength from the CPT data indicate the range could be about 5,000 psf. The assumed typical SPT N-value in this layer results in an estimated undrained shear strength of 850 psf.

We will use a value of 1,000 psf for the design calculations which is based on the lower end of the typical values measured in the laboratory.

ML/CL-2 Layer (~ El 133 to 129)
No laboratory testing was completed on samples from the ML/CL-2 stratum. Estimates of undrained shear strength from the CPT data indicate the strength could be about 19,600 psf. The assumed typical SPT N-value in this layer results in an estimated undrained shear strength of 1,400 psf.

We will use a value of 1,400 psf for the design calculations which is closer to that obtained from the empirical correlation to the SPT N-value.

ML (Downstream Toe from Ground Surface to ~ El 170)
No laboratory testing was completed on samples from the ML stratum. Estimates of undrained shear strength from the CPT data indicate the strength could be about 9,400 psf. The assumed typical SPT N-value in this layer results in an estimated undrained shear strength of 800 psf.

We will use a value of 1,000 psf for the design calculations which is close to that obtained from the empirical correlation to the SPT N-value.

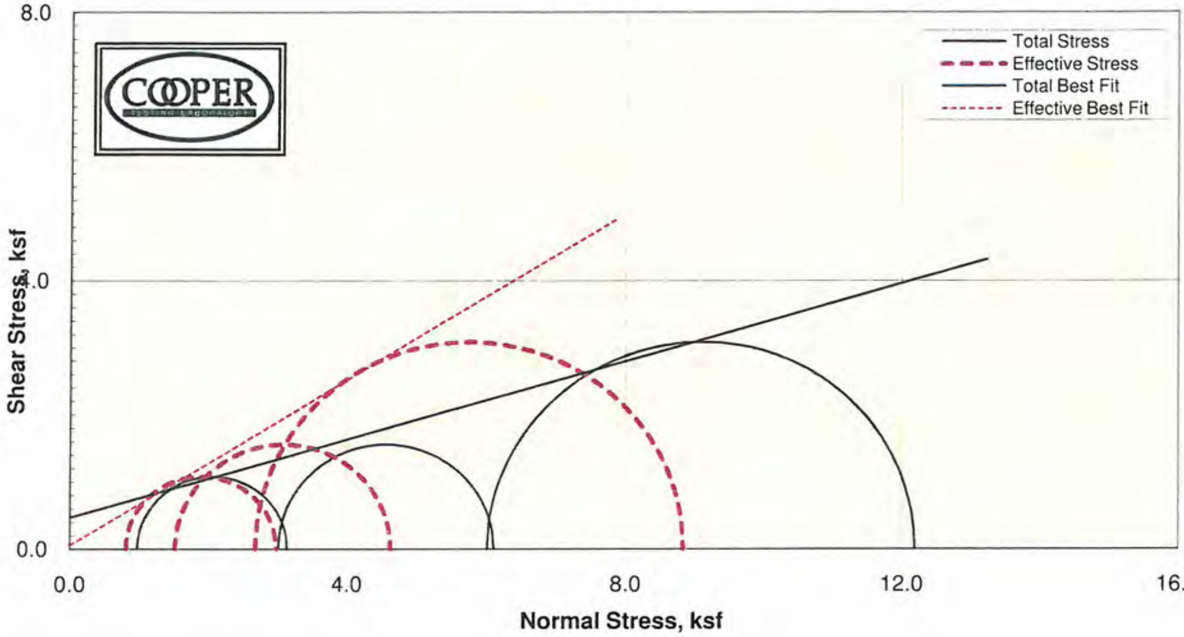
Bedrock
The laboratory data on samples of the bedrock stratum indicate undrained shear strengths could range from 2,300 to 23,400 psf.

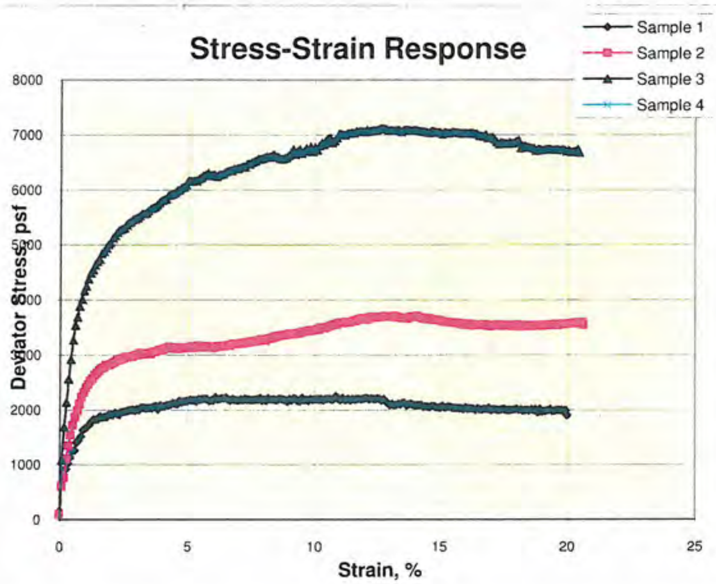
We will model the bedrock with an undrained shear strength of 2,300 psf to represent the landslide debris material.

Attachments:

GEI Interpretation of CE&G CU Tests
GEI Interpretation of Fugro (2004) CPT Data

Triaxial Consolidated-Undrained
(ASTM D4767)





Sample:	1	2	3	4
MC, %	24.9	23.4	22.1	
Dry Dens, pcf	97.5	100.4	102.6	
Sat. %	92.4	93.0	93.0	
Void Ratio	0.727	0.679	0.642	
Diameter in	2.87	2.87	2.87	
Height, in	6.00	5.91	6.00	
	Final			
MC, %	29.2	25.1	22.0	
Dry Dens, pcf	94.2	100.4	105.7	
Sat. %	100.0	100.0	100.0	
Void Ratio	0.788	0.678	0.594	
Diameter, in	2.92	2.89	2.86	
Height, in	6.01	5.81	5.87	
Cell, psi	45.4	59.3	80.2	
BP, psi	38.7	38.5	38.4	
	Effective Stresses At:			
Strain, %	5.0	5.0	5.0	
Deviator ksf	2.159	3.119	6.169	
Excess PP	0.158	1.483	3.355	
Sigma 1	2.965	4.631	8.833	
Sigma 3	0.806	1.512	2.664	
P, ksf	1.886	3.071	5.748	
Q, ksf	1.079	1.559	3.084	
Stress Ratio	3.677	3.063	3.316	
Rate in/min	0.0005	0.0005	0.0005	
Total C	0.5	ksf		
Total Phi	16.2	Degrees		
Eff. C	0.0	ksf		
Eff. Phi	31.7	Degrees		

Job No.:	471-051	Date:	3/30/2010
Client:	Cal Engineering & Geology		BY:MD/DC
Project:	081070		
Sample 1)	B7 @ 20.5(Tip-13")	Strong Brown Lean CLAY w/ CaCO3	
Sample 2)	B7 @ 20.5(Tip-6.5")	Strong Brown Lean CLAY w/ CaCO3	
Sample 3)	B7 @ 20.5(Tip)	Strong Brown Lean CLAY w/ CaCO3	
Sample 4)			

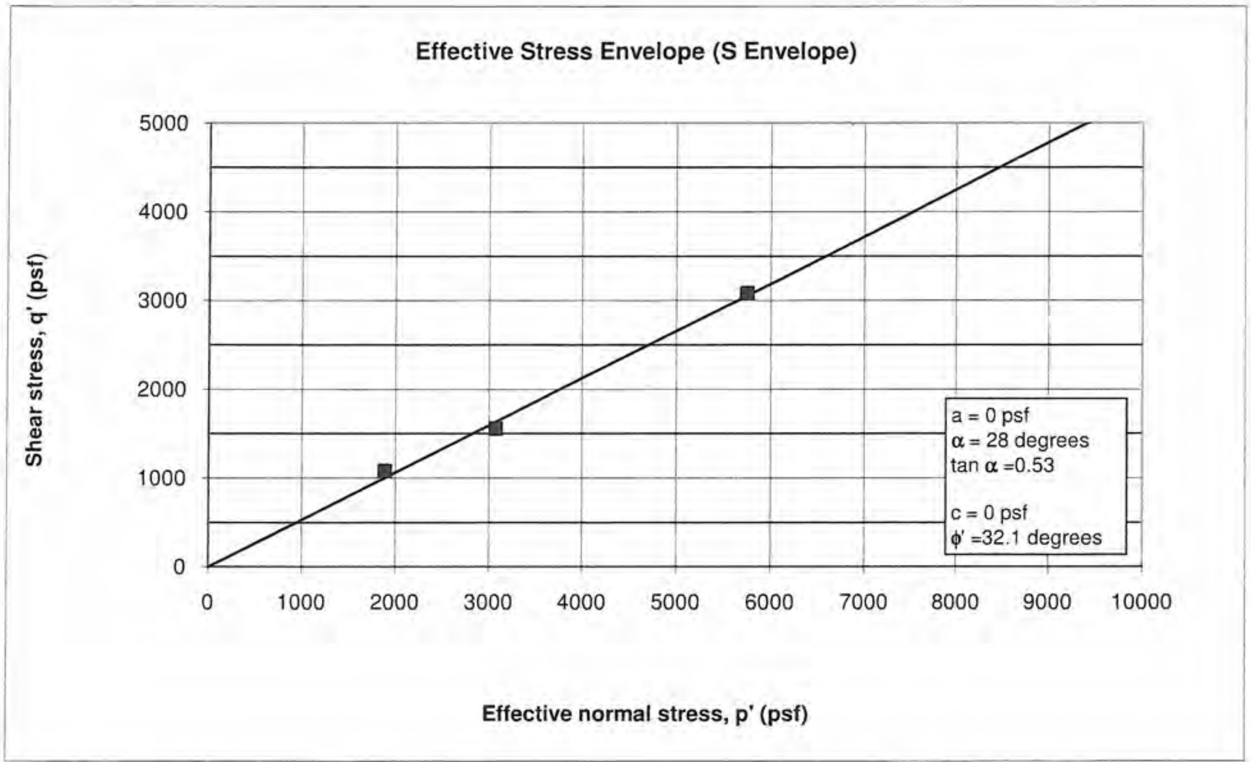
REMARKS: Strengths picked at 5% strain.

GEI Consultants	Client: Contra Costa FC&WCD Subject: CU Evaluation Boring: B-7 (20.5')	Project: 08318-0 Date: 4/9/10 By: D. DeCesaris Checked: 4/29/10 By: M. Quinn

Boring	Sample	Sample Depth (feet)	σ'_{3c} (psi)	$(\sigma_1-\sigma_3)_f$ (psi)	σ'_{1f} (psi)	σ'_{3f} (psi)	uf (psi)	p'_f (psf)	q_f (psf)
B-7	1	21.5	6.7	15.0	20.6	5.6	1.1	1886	1080
	2	21.0	20.8	21.7	32.2	10.5	10.3	3072	1560
	3	20.5	41.8	42.8	61.3	18.5	23.3	5749	3085

$$p' = \frac{\sigma_1' + \sigma_3'}{2} \qquad q' = \frac{\sigma_1' - \sigma_3'}{2}$$

Effective Stress Strength Envelope (S-Envelope)



The slope of the "best fit line", α, is:	28.0	degrees
Sin ϕ' =	tan α	
Accordingly, the drained friction angle, ϕ', is:	32.1	degrees

R-Envelope or Total Stress Envelope

Plot the maximum shear strength on the failure plane vs. the corresponding normal stress at failure.

Boring	Sample Depth (feet)	Sample	σ_{3c}' (psf)	$\sigma_1 - \sigma_3$ (psf)	τ_{max} (psf)	τ_{ff} (psf)	σ_f (psf)
B-7	21.0'	1	965	2159	1080	914	1470
		2	2995	3119	1560	1321	3725
		3	6019	6169	3085	2612	7464

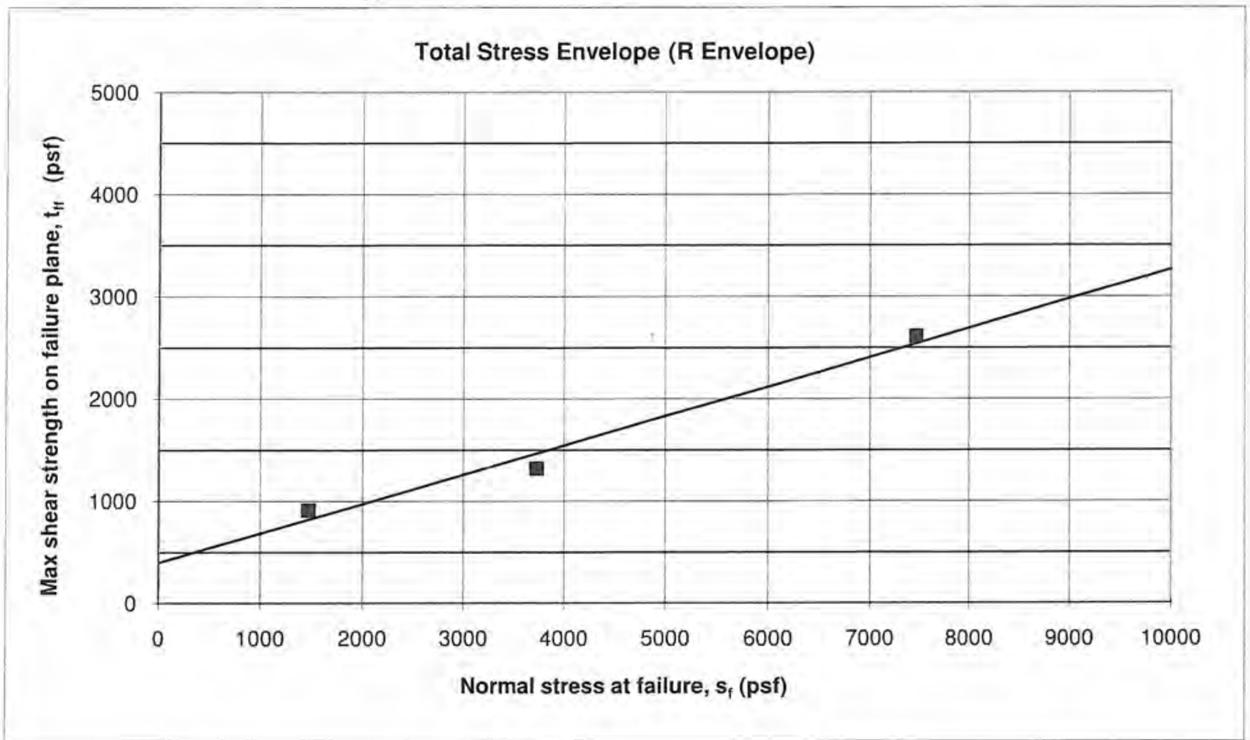
Maximum shear strength on the failure plane is determined from: where,

$$\tau_{ff} = \tau_{max} \cos \phi'$$

$\phi' =$ 32.1 degrees

Corresponding normal stress at failure is determined from:

$$\sigma_f = \sigma_{3c}' + \frac{\sigma_1 - \sigma_3}{2} - \tan(\phi') * \tau_{ff}$$



The R-Envelope is defined by: $\phi_R =$ 16.0 degrees

$C_R =$ 400 psf

Develop the $K_c = 1$ Envelope

The $K_c = 1$ envelope is estimated by plotting the same τ_{ff} value as the R-envelope but plotted against the consolidation pressure σ_{3c}' instead of σ_f.

Point	σ_{3c}' (psf)	τ_{ff} (psf)
1	965	914
2	2995	1321
3	6019	2612

Similarly, the relationship between the R-envelope and K_c=1 envelope can be expressed from the following equations from page 155 of Duncan and Wright (2005), Soil Strength and Slope Stability.

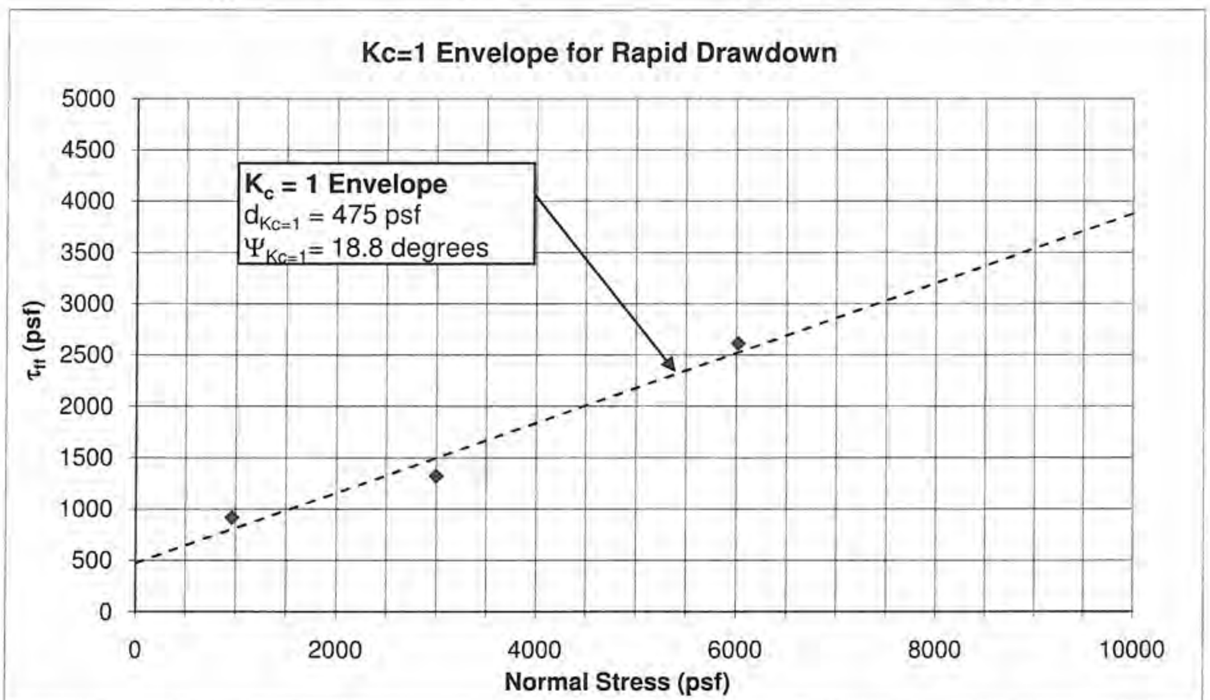
The K_c=1 envelope is defined by the slope $\Psi_{Kc=1}$ and the intercept $d_{Kc=1}$ which can be calculated from the following equations:

Eq. 9.8 $$d_{K_c=1} = \frac{C_R}{1 + (\sin\phi')\tan\phi_R / \cos\phi'}$$

Eq. 9.9 $$\Psi_{K_c=1} = \arctan\frac{\tan(\phi_R)}{1 + (\sin\phi' - 1)\tan\phi_R / \cos\phi'}$$

From above:

ϕ' =	32.1	degrees		therefore,		
c_R =	400	psf	⇨	$d_{Kc=1}$ =	475	psf
ϕ_R =	16.0	degrees		$\Psi_{Kc=1}$ =	18.8	degrees



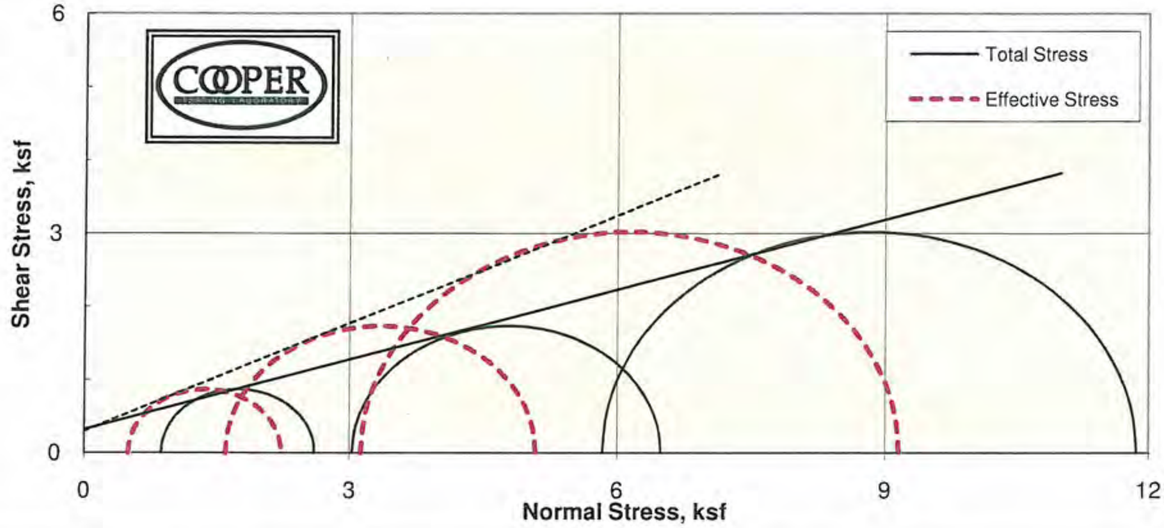
Estimate the undrained shear strength from the $K_c = 1$ Envelope.

$$S_u = \tan(\psi_{Kc=1}) * \sigma' + d_{Kc=1}$$

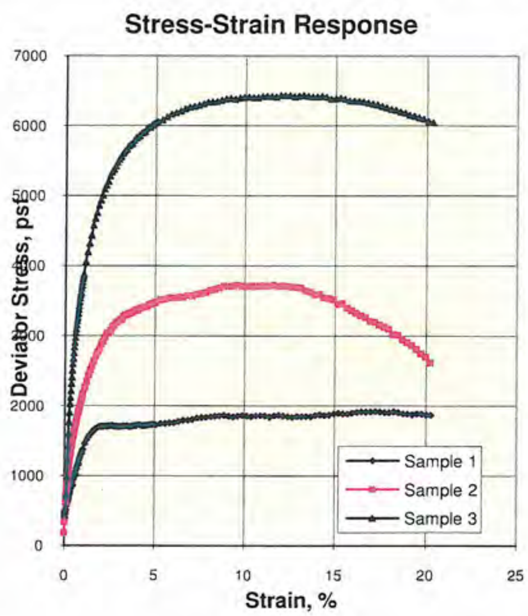
σ' in the equation is the effective overburden stress if the soil is normally consolidated, or the maximum past pressure if the soil is overconsolidated.

At this location, the existing effective stress is:	2460	psf
The maximum past pressure is:	4500	psf
OCR:	1.8	
The estimated undrained shear strength is, S_u:	2009	psf

Triaxial Consolidated Undrained with Pore Pressure
ASTM D4767



Total : 17.7Phi, (deg.) 0.3C,(ksf) **Effective:** 26.2Phi, (deg.) 0.3C,(ksf)



Sample:	1	2	3	4
MC, %	29.4	31.1	30.0	
DD, pcf	89.4	91.2	93.3	
Sat. %	88.0	97.1	98.6	
Void Ratio	0.919	0.881	0.838	
Diameter in	2.87	2.87	2.87	
Height, in	5.97	6.08	6.00	
	Final			
MC, %	32.5	31.8	28.2	
DD, pcf	90.6	91.6	96.7	
Sat. %	100.0	100.0	100.0	
Void Ratio	0.894	0.874	0.775	
Diameter, in	2.86	2.88	2.84	
Height, in	5.93	6.02	5.91	
Cell, psi	45.4	59.3	80.2	
BP, psi	39.3	38.3	39.7	
	Effective Stresses At:			
Strain, %	5.0	5.0	5.0	
Deviator ksf	1.735	3.464	6.036	
Excess PP	0.372	1.423	2.701	
Sigma 1	2.235	5.069	9.161	
Sigma 3	0.500	1.605	3.125	
P, ksf	1.367	3.337	6.143	
Q, ksf	0.868	1.732	3.018	
Stress Ratio	4.473	3.157	2.932	
Rate in/min	0.0005	0.0005	0.0005	

Job No.: 471-051 **Date:** 3/30/2010

Client: Cal Engineering & Geology BY:DC

Project: 081070

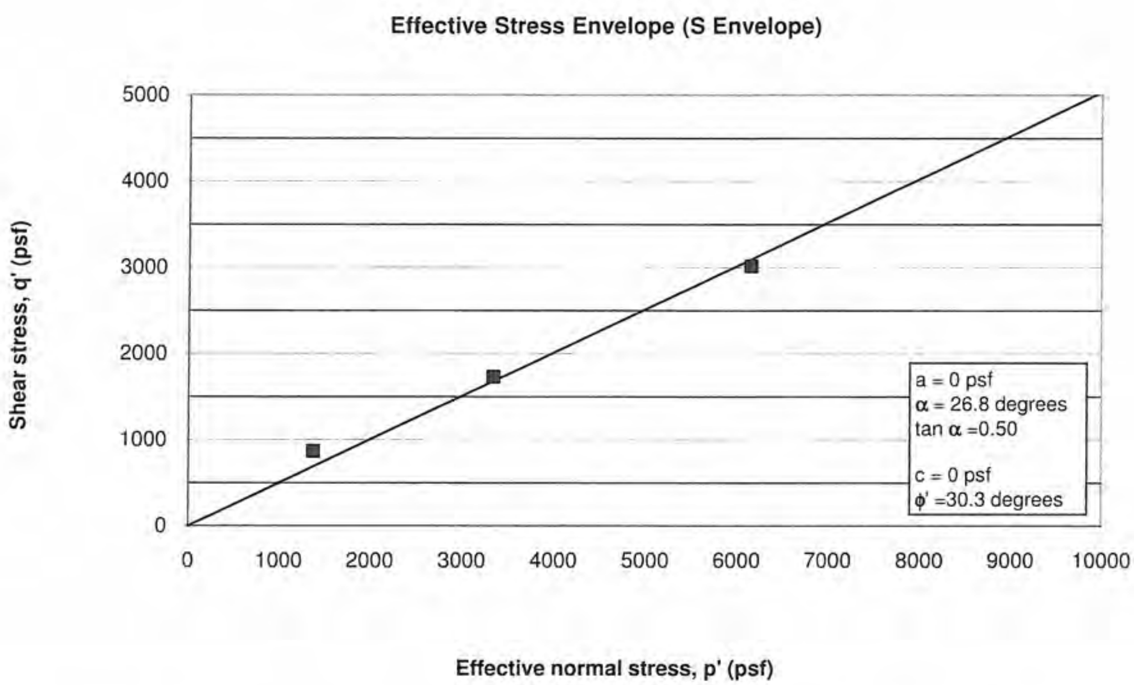
Sample 1)	B9 @ 15(Tip-16.5")	Brown Fat CLAY
Sample 2)	B9 @ 15(Tip-10")	Brown Fat CLAY
Sample 3)	B9 @ 15(Tip-3.5")	Brown Fat CLAY

	Client:	Contra Costa FC&WCD	Project:	08318-0		
GEI Consultants	Subject:	CU Evaluation	Date:	4/9/10	By:	D. DeCesaris
	Boring:	B-9 (15')	Checked:	4/29/10	By:	M. Quinn

Boring	Sample	Sample Depth (feet)	σ'_{3c} (psi)	$(\sigma_1-\sigma_3)_f$ (psi)	σ'_{1f} (psi)	σ'_{3f} (psi)	uf (psi)	p'_f (psf)	q_f (psf)
B-9	1	16.3	6.1	12.0	15.6	3.5	2.6	1374	868
	2	15.8	21.0	24.1	35.2	11.1	9.9	3333	1732
	3	15.3	40.5	41.9	63.7	21.7	18.8	6149	3018

$$p' = \frac{\sigma'_1 + \sigma'_3}{2} \qquad q' = \frac{\sigma'_1 - \sigma'_3}{2}$$

Effective Stress Strength Envelope (S-Envelope)



The slope of the "best fit line", α, is:	26.8	degrees
$\sin \phi'$ =	$\tan \alpha$	
Accordingly, the drained friction angle, ϕ', is:	30.3	degrees

R-Envelope or Total Stress Envelope

Plot the maximum shear strength on the failure plane vs. the corresponding normal stress at failure.

Boring	Sample Depth (feet)	Sample	σ_{3c}' (psf)	$\sigma_1 - \sigma_3$ (psf)	τ_{max} (psf)	τ_{ff} (psf)	σ_f (psf)
B-9	15.8'	1	878	1735	868	749	1309
		2	3024	3464	1732	1496	3883
		3	5832	6036	3018	2607	7329

Maximum shear strength on the failure plane is determined from:

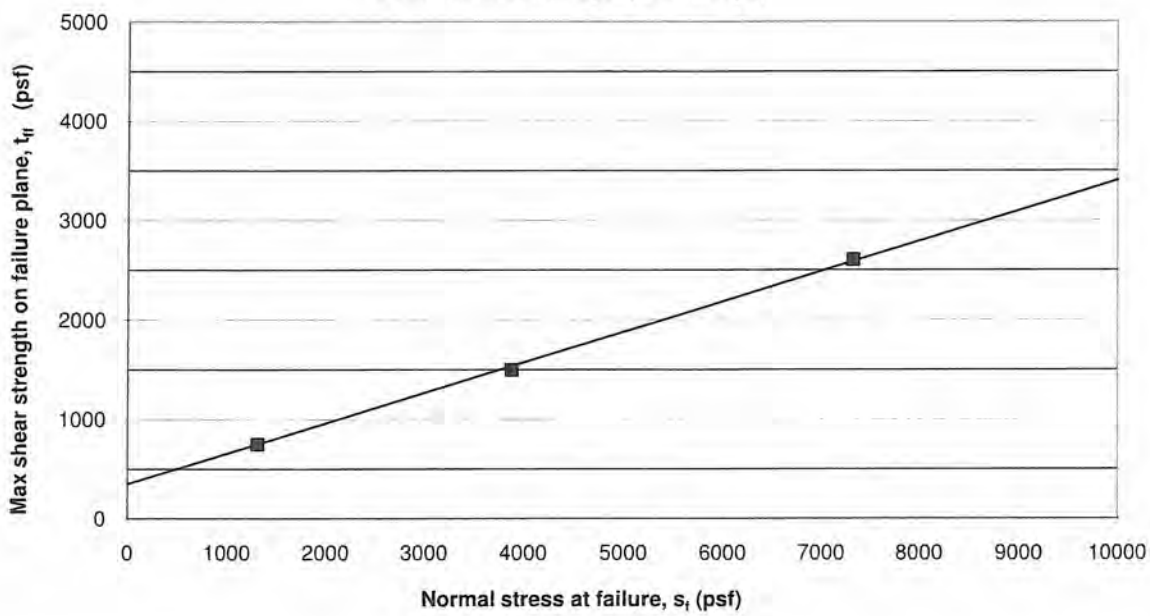
$$\tau_{ff} = \tau_{max} \cos \phi'$$

where, ϕ' = 30.3 degrees

Corresponding normal stress at failure is determined from:

$$\sigma_f = \sigma_{3c}' + \frac{\sigma_1 - \sigma_3}{2} - \tan(\phi') * \tau_{ff}$$

Total Stress Envelope (R Envelope)



The R-Envelope is defined by: ϕ_R = 17.0 degrees

c_R = 350 psf

Develop the $K_c = 1$ Envelope for Rapid Drawdown Analysis.

The $K_c = 1$ envelope is estimated by plotting the same τ_{ff} value as the R-envelope but plotted against the consolidation pressure σ_{3c}' instead of σ_f.

Point	σ_{3c}' (psf)	τ_{ff} (psf)
1	878	749
2	3024	1496
3	5832	2607

Similarly, the relationship between the R-envelope and K_c=1 envelope can be expressed from the following equations from page 155 of Duncan and Wright (2005), Soil Strength and Slope Stability,

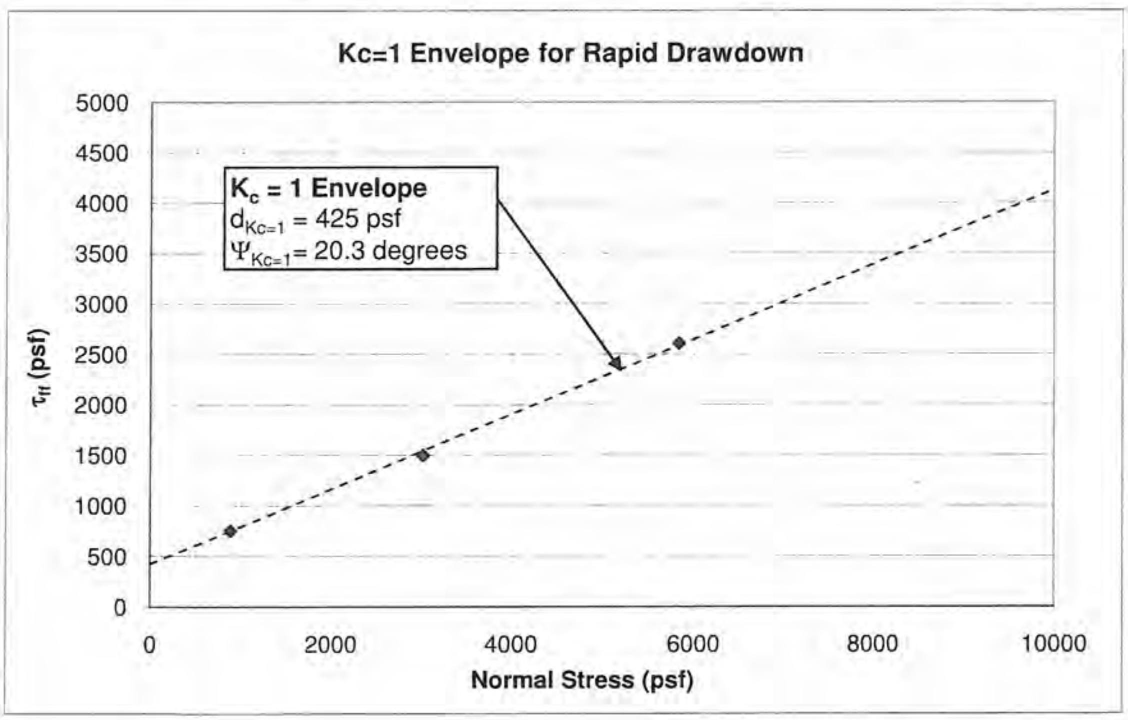
The K_c=1 envelope is defined by the slope $\Psi_{Kc=1}$ and the intercept $d_{Kc=1}$ which can be calculated from the following equations:

Eq. 9.8 $$d_{K_c=1} = \frac{c_R}{1+(\sin\phi'-1)\tan\phi_R/\cos\phi'}$$

Eq. 9.9 $$\Psi_{K_c=1} = \arctan\frac{\tan(\phi_R)}{1+(\sin\phi'-1)\tan\phi_R/\cos\phi'}$$

From steps (1) and (2),

ϕ' =	30.3	degrees		therefore,		
c_R =	350	psf	⇨	$d_{Kc=1}$ =	425	psf
ϕ_R =	17.0	degrees		$\Psi_{Kc=1}$ =	20.3	degrees



Estimate the undrained shear strength from the $K_c = 1$ Envelope.

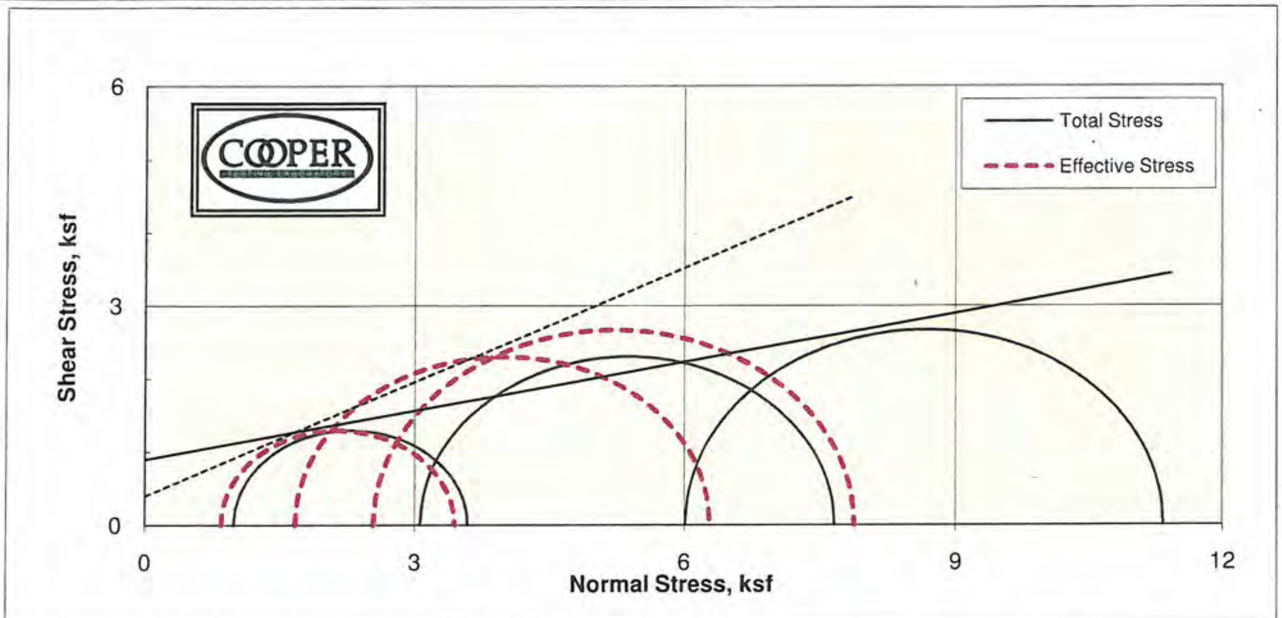
$$S_u = \tan(\Psi_{Kc=1}) * \sigma' + d_{Kc=1}$$

σ' in the equation is the effective overburden stress if the soil is normally consolidated, or the maximum past pressure if the soil is overconsolidated.

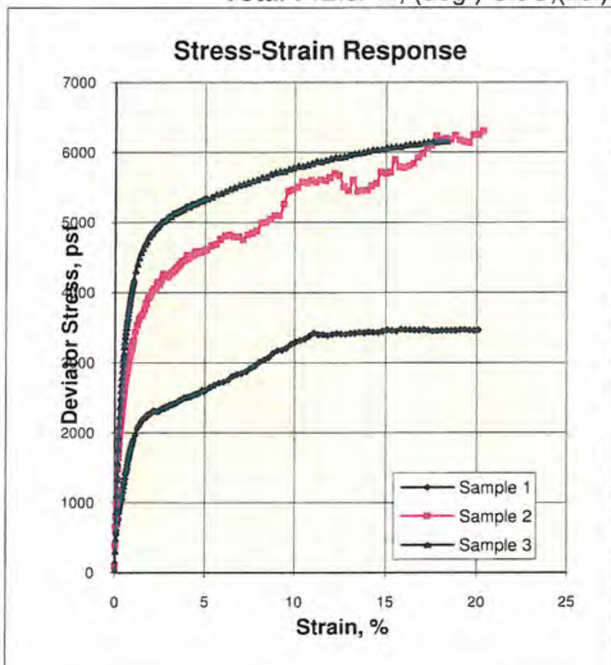
At this location, the existing effective stress is:	1800	psf
The maximum past pressure is:	4500	psf
OCR:	2.5	

The estimated undrained shear strength is, S_u:	2093	psf

Triaxial Consolidated Undrained with Pore Pressure
ASTM D4767



Total : 12.5Phi, (deg.) 0.9C,(ksf) **Effective:** 27.5Phi, (deg.) 0.4C,(ksf)



Sample:	1	2	3	4
MC, %	23.3	22.9	23.0	
DD, pcf	101.6	102.2	103.7	
Sat. %	95.4	95.4	99.3	
Void Ratio	0.659	0.649	0.625	
Diameter in	2.87	2.87	2.87	
Height, in	6.00	6.00	6.00	
	Final			
MC, %	23.6	22.9	22.0	
DD, pcf	102.9	104.1	105.7	
Sat. %	100.0	100.0	100.0	
Void Ratio	0.637	0.618	0.594	
Diameter, in	2.86	2.87	2.87	
Height, in	5.97	5.90	5.89	
Cell, psi	45.9	59.8	80.7	
BP, psi	39.0	38.5	39.0	
	Effective Stresses At:			
Strain, %	5.0	5.0	5.0	
Deviator ksf	2.589	4.603	5.335	
Excess PP	0.141	1.375	3.458	
Sigma 1	3.445	6.290	7.886	
Sigma 3	0.856	1.688	2.550	
P, ksf	2.150	3.989	5.218	
Q, ksf	1.294	2.301	2.668	
Stress Ratio	4.024	3.727	3.092	
Rate in/min	0.0005	0.0005	0.0005	

Job No.: 471-051 **Date:** 3/30/2010

Client: Cal Engineering & Geology BY:DC

Project: 081070

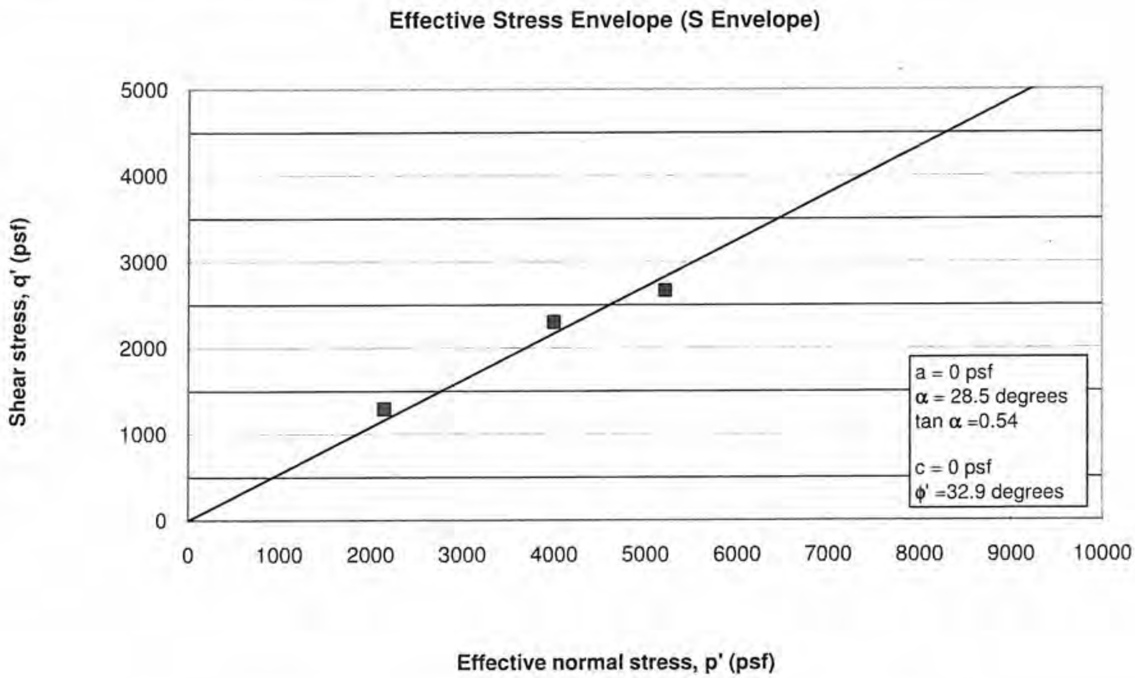
Sample 1)	B10;10-9 @ 19(Tip-16.5")	Brown Mottled Gray Sandy Lean CLAY
Sample 2)	B10;10-9 @ 19(Tip-10")	Brown Mottled Gray Sandy Lean CLAY
Sample 3)	B10;10-9 @ 19(Tip-3.5")	Brown Mottled Gray Sandy Lean CLAY

	Client:	Contra Costa FC&WCD	Project:	08318-0	
GEI Consultants	Subject:	CU Evaluation	Date:	4/9/10	By: D. DeCesaris
		Boring: B-10 (19')	Checked:	4/29/10	By: M. Quinn

Boring	Sample	Sample Depth (feet)	σ'_{3c} (psi)	$(\sigma_1-\sigma_3)_f$ (psi)	σ'_{1f} (psi)	σ'_{3f} (psi)	uf (psi)	p'_f (psf)	q_f (psf)
B-10	1	20.4	6.9	18.0	23.9	5.9	1.0	2147	1295
	2	19.8	21.3	32.0	43.7	11.8	9.5	3994	2302
	3	19.3	41.7	37.0	54.7	17.7	24.0	5214	2668

$$p' = \frac{\sigma_1' + \sigma_3'}{2} \qquad q' = \frac{\sigma_1' - \sigma_3'}{2}$$

Effective Stress Strength Envelope (S-Envelope)



The slope of the "best fit line", α, is:	28.5	degrees
Sin ϕ' =	tan α	
Accordingly, the drained friction angle, ϕ', is:	32.9	degrees

GEI Consultants	Client: Contra Costa FC&WCD	Project: 08318-0		
	Subject: CU Evaluation	Date: 4/9/10	By: D. DeCesaris	
	Boring: B-10 (19')	Checked: 4/29/10	By: M. Quinn	

R-Envelope or Total Stress Envelope

Plot the maximum shear strength on the failure plane vs. the corresponding normal stress at failure.

Boring	Sample Depth (feet)	Sample	σ_{3c}' (psf)	$\sigma_1 - \sigma_3$ (psf)	τ_{max} (psf)	τ_{ff} (psf)	σ_f (psf)
B-10	19.8'	1	994	2589	1295	1087	1585
		2	3067	4603	2302	1933	4119
		3	6005	5335	2668	2240	7224

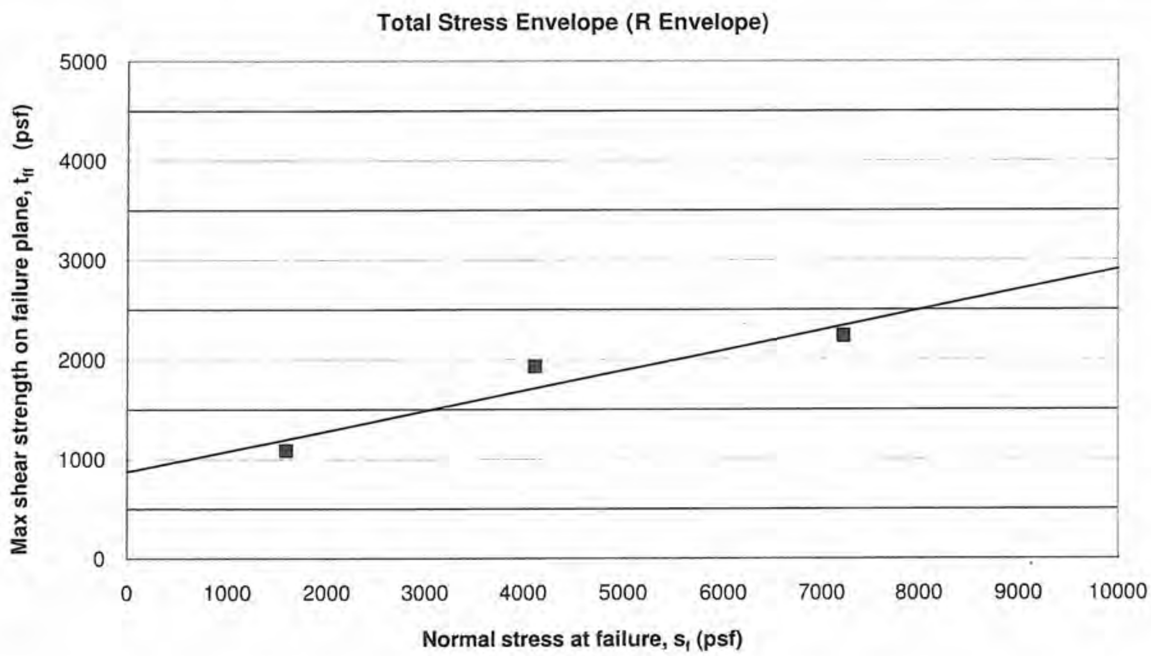
Maximum shear strength on the failure plane is determined from:

$$\tau_{ff} = \tau_{max} \cos\phi'$$

where, ϕ' = 32.9 degrees

Corresponding normal stress at failure is determined from:

$$\sigma_f = \sigma_{3c}' + \frac{\sigma_1 - \sigma_3}{2} - \tan(\phi') * \tau_{ff}$$



The R-Envelope is defined by: ϕ_R = 11.5 degrees

c_R = 875 psf

3) Develop the $K_c = 1$ Envelope for Rapid Drawdown Analysis.

The $K_c = 1$ envelope is estimated by plotting the same τ_{ff} value as the R-envelope but plotted against the consolidation pressure σ_{3c}' instead of σ_f.

Point	σ_{3c}' (psf)	τ_{ff} (psf)
1	994	1087
2	3067	1933
3	6005	2240

Similarly, the relationship between the R-envelope and K_c=1 envelope can be expressed from the following equations from page 155 of Duncan and Wright (2005), Soil Strength and Slope Stability,

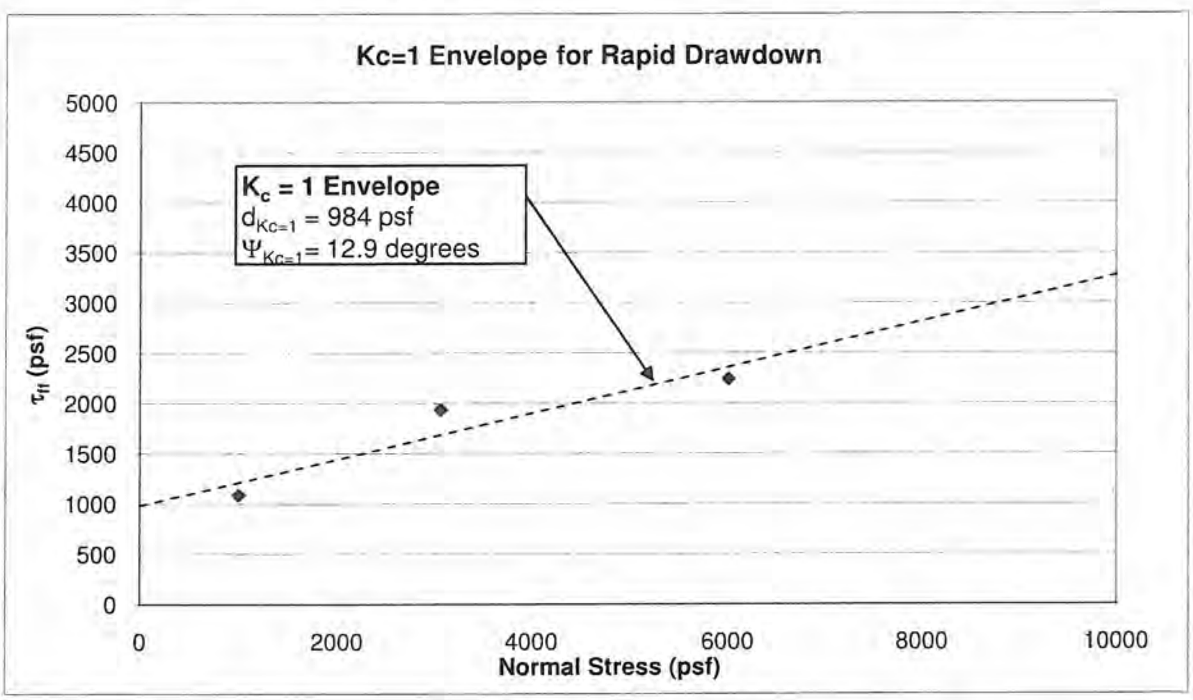
The K_c=1 envelope is defined by the slope $\Psi_{Kc=1}$ and the intercept $d_{Kc=1}$ which can be calculated from the following equations:

Eq. 9.8 $$d_{K_c=1} = \frac{c_R}{1+(\sin\phi'-1)\tan\phi_R/\cos\phi'}$$

Eq. 9.9 $$\Psi_{K_c=1} = \arctan\frac{\tan(\phi_R)}{1+(\sin\phi'-1)\tan\phi_R/\cos\phi'}$$

From steps (1) and (2),

ϕ' =	32.9	degrees	therefore,			
c_R =	875	psf	⇨	$d_{Kc=1}$ =	984	psf
ϕ_R =	11.5	degrees		$\Psi_{Kc=1}$ =	12.9	degrees



Estimate the undrained shear strength from the $K_c = 1$ Envelope.

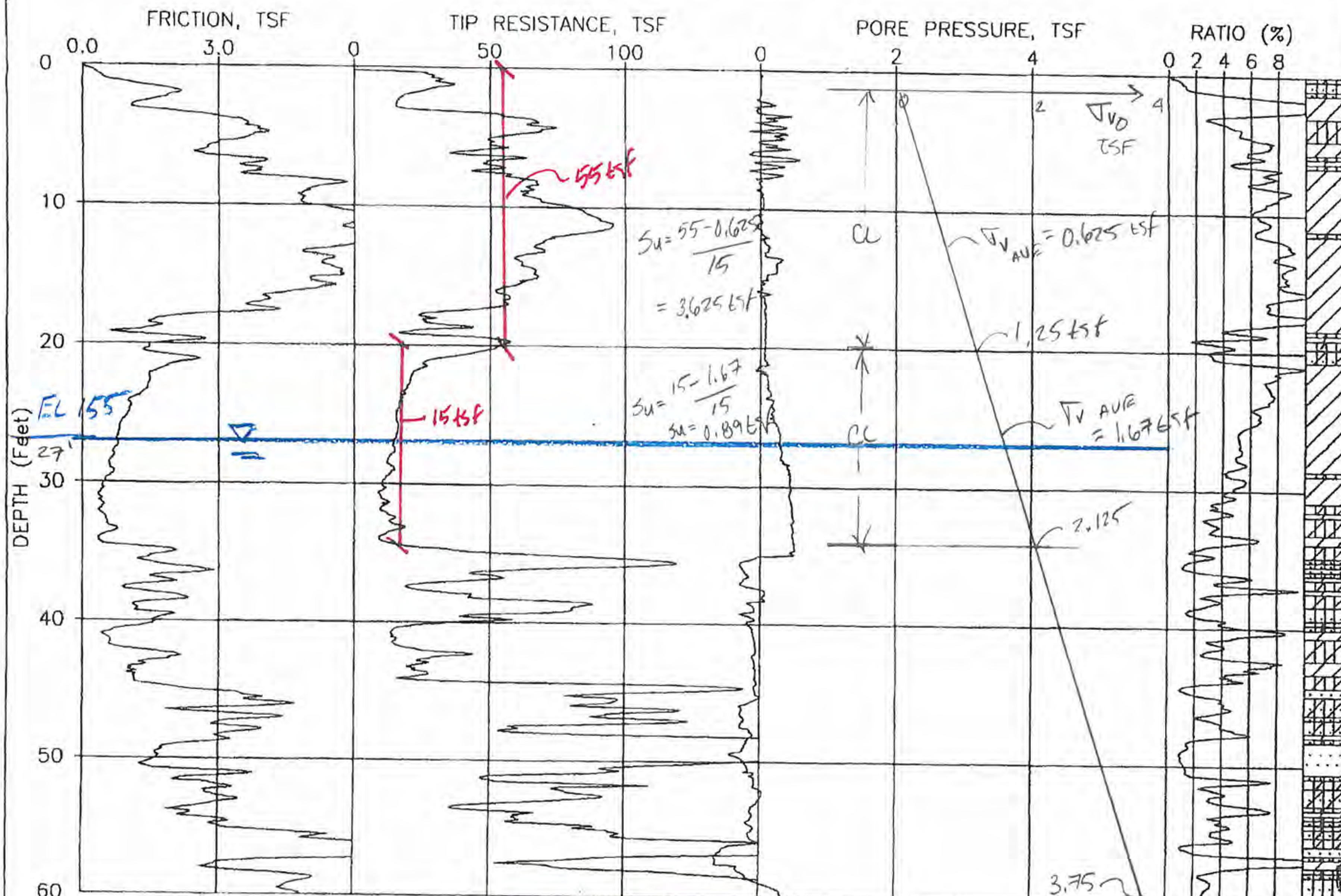
$$S_u = \tan(\Psi_{Kc=1}) * \sigma' + d_{Kc=1}$$

σ' in the equation is the effective overburden stress if the soil is normally consolidated, or the maximum past pressure if the soil is overconsolidated.

At this location, the existing effective stress is:	1720	psf
The maximum past pressure is:	4600	psf
OCR:	2.7	

The estimated undrained shear strength is, S_u:	2036	psf

G.S. EL = 182



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ELEVATION: 0.00

FUGRO GEOSCIENCES,INC

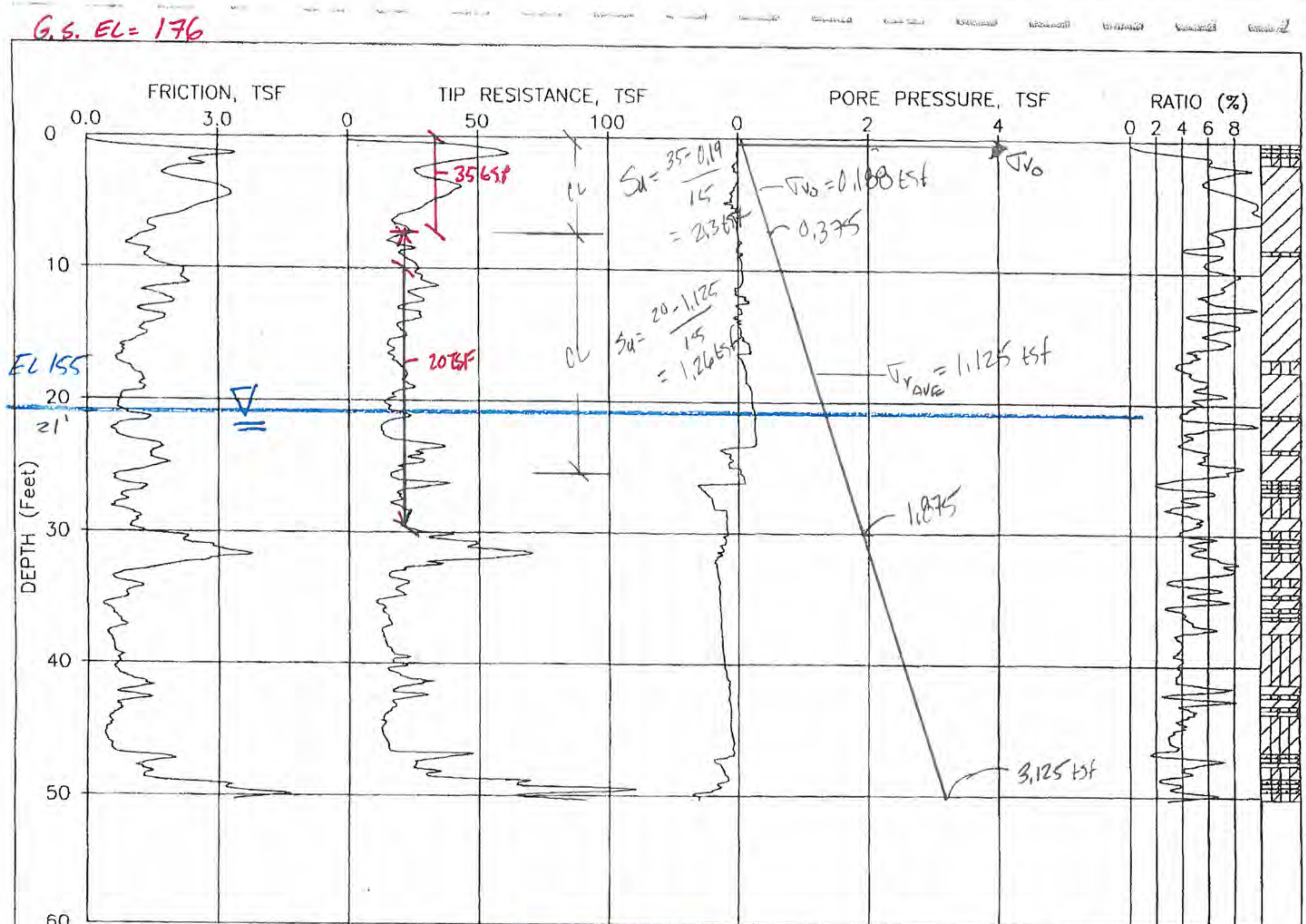
CPT NUMBER: 03

CONE NUMBER: F7.5CKEW1201

DATE: 04-11-2002

PLATE: 1 OF 1

DRAFT



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ELEVATION: 0.00

FUGRO GEOSCIENCES,INC

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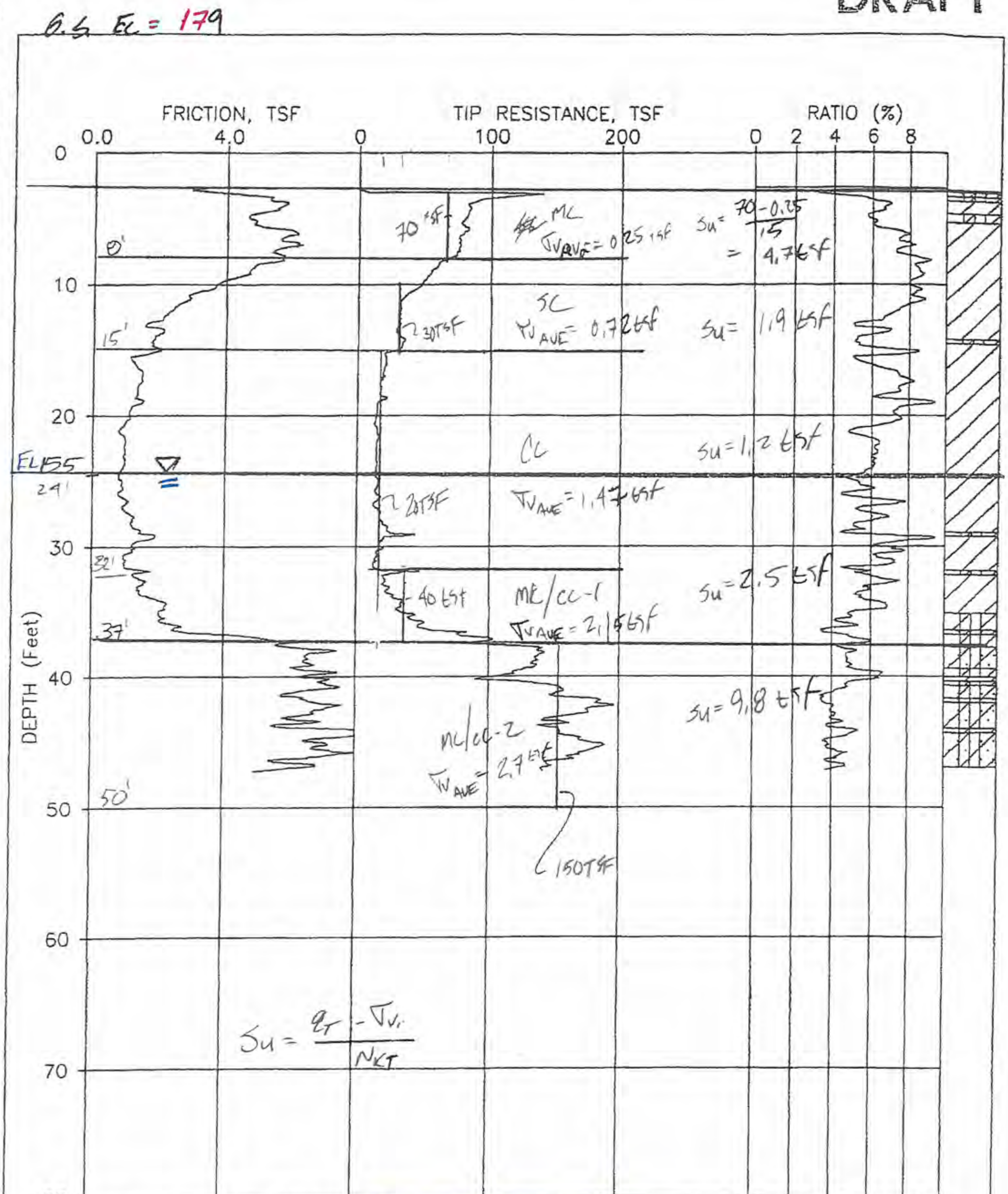
CONE NUMBER: F7.5CKEW1201

DATE: 04-11-2002

PLATE: 1 OF 1

DRAFT

DRAFT



JOB NUMBER: 02-0703

ELEVATION: 0.00

CPT NUMBER: ~~C07~~ 6

CONE NUMBER: F7.5CKEV611

DATE: 08-20-2002

PLATE: 1 OF 1

FUGRO GEOSCIENCES,INC

Attachment 4 – Foundation Drained Shear Strengths

Drained Shear Strength Estimates of Foundation Soils

Problem:

Select representative drained strength parameters for the foundation soils at the three cross sections being evaluated.

References:

(1) Carter, M. and Bentley, S.P., *Correlations of Soil Properties,* Pentech Press Publishers, London, 1991.
(2) Lambe, T.W., and Whitman, R.V., 1969, *Soil Mechanics,* John Wiley and Sons, New York.
(3) Cal Engineering & Geology, Inc., *Geotechnical Data Report, Contra Costa County Flood Control & Water Conservation District, Upper Sand Creek Detention Basin, Deer Creek Road, Antioch, California,* May 5, 2010.

Summary:

We will use the following drained shear strengths in our stability analysis:

Soil Layer	Drained Shear Strength	
	Cohesion (psf)	ϕ' (degrees)
ML (Downstream Above El 170)	0	28
SC	0	32
CL (Ground Surface to El. 152)	0	30
SM-1	0	32
ML/CL-1 Layer (~ El 152 to 144)	0	31
SM-2	0	32
ML/CL-2 Layer (~ El 133 to 129)	0	25
Bedrock	0	35

Approach:

Drained shear strengths in the foundation soils along the dam alignment were estimated from several sources of data including:

- Laboratory Consolidated Undrained Triaxial Tests (CU)
- Empirical Correlation to Plasticity Index (PI)
- Empirical Correlation to SPT N-Value

Coarse-grained Soils (Cohesionless)

We assumed that the coarse-grained foundation soils will drain relatively rapidly. We generally estimated drained strengths of coarse-grained foundation soils using an empirical correlation to SPT N-values (Ref. 1).

For each coarse-grained foundation layer, SPT N-values were averaged and an approximate friction angle was estimated. Our interpretations for each layer are attached.

Location	Boring	USCS	N_{60}
SM-1 - ~El. 153 to 147			
Crest	B-3-09	SC	14
WS Toe	B-7-09	SM	22
WS Toe	B-8-09	SW	19
WS Toe	B-8-09	SW	15
LS Toe	B-9-09	SM	15
LS Toe	B-9-09	SM	23
LS Toe	B-10-09	SC	12
		Average =	17 bpf
		Assumed Typical Values =	16 bpf
		ϕ' =	32 degrees

Location	Boring	USCS	N_{60}
SM-2 - ~El. 145 to 133			
Crest	B-3-09	SM	37
WS Toe	B-7-09	SM	23
WS Toe	B-7-09	SM	12
WS Toe	B-7-09	SM	8
WS Toe	B-7-09	SM	35
LS Toe	B-9-09	SC	22
LS Toe	B-10-09	SW-SC	21
		Average =	23 bpf
		Assumed Typical Values =	14 bpf
		ϕ' =	32 degrees

Location	Boring	USCS	N_{60}
SC - ~El. 170 - 167 (Downstream)			
LS Toe	B-9-09	SC	14
		Assumed Typical Values =	14 bpf
		ϕ' =	32 degrees

Foundation Drained Shear Strengths

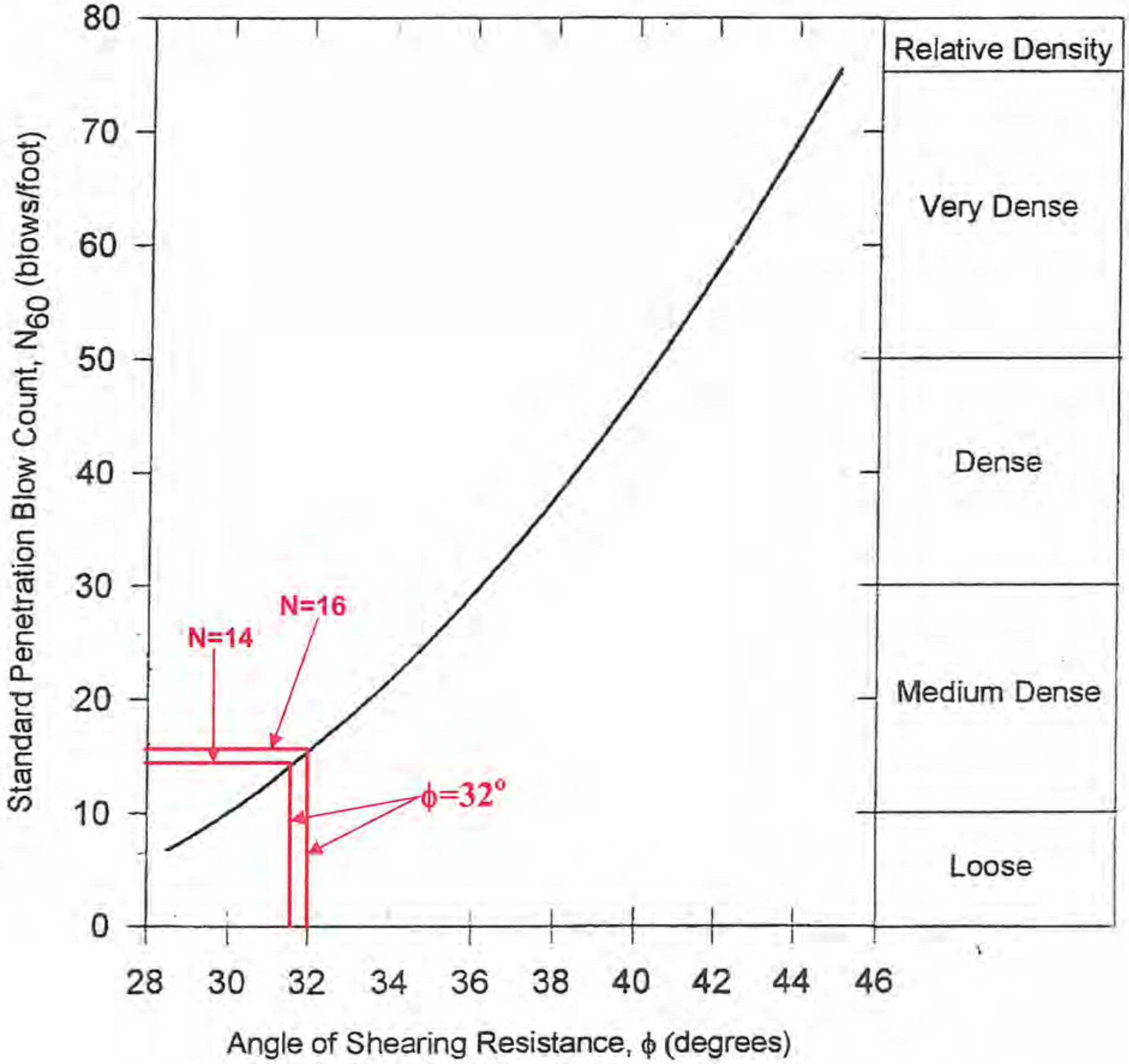


Figure 35. Estimation of the angle of shearing resistance of granular soils from standard penetration test results (Originally from Peck et al., 1974, modified by Carter and Bentley, 1991).

Foundation Drained Shear Strengths

Fine-grained Soils (Cohesive)

Drained shear strengths in the fine-grained foundation soils along the dam alignment were estimated from CU triaxial tests and from an empirical correlation to plasticity index (PI).

Where CU strength data were available, drained strengths were estimated by plotting the failure points at 5% strain for each test in p-q stress space. A failure envelope was then best-fit to the data to provide an effective friction angle for each test. Cohesion was assumed to be zero. A summary of the CU Test data is presented below:

CU Tests

Boring Number	Depth Interval (feet)	USCS	Sample Information: Specimen	Water Content (%)	Dry Unit Weight (pcf)	Moist Unit Weight (pcf)	Shear Strengths: Effective Stress c (psf)	ϕ' (psf)	Model Soil Layer
B-7	21.5'	CL	1	29.2	94.2	121.7	0	32.1	CL
	21.0'	CL	2	25.1	100.4	125.6			
	20.5'	CL	3	22.0	105.7	129.0			
B-9	16.3'	CL	1	32.5	90.6	120.0	0	30.3	CL
	15.8'	CL	2	31.8	91.6	120.7			
	15.3'	CL	3	28.2	96.7	124.0			
B-10	20.4'	CL	1	23.6	102.9	127.2	0	32.9	CL
	19.8'	CL	2	22.9	104.1	127.9			
	19.3'	CL	3	22.0	105.7	129.0			

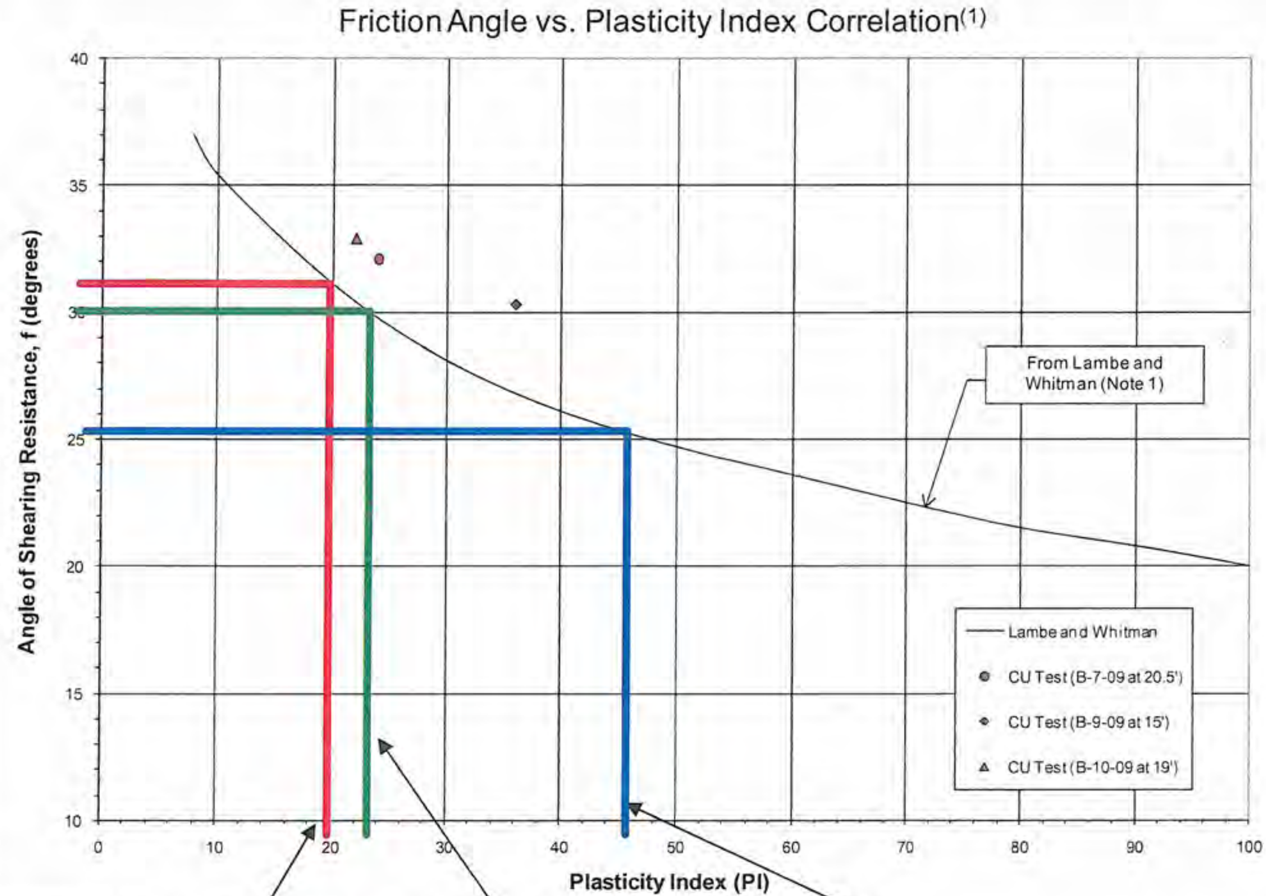
Where Atterberg limit test data was available, drained strengths were estimated using an empirical correlation to plasticity index from Lambe & Whitman (1969).

Location	Boring	USCS	PI
CL - Ground Surface to ~El. 152			
Crest	B-2-09	CL	28
Crest	B-2-09	CL	18
Crest	B-2-09	CL	25
Crest	B-3-09	CL	24
Crest	B-3-09	CL	24
Crest	B-3-09	CL	19
Crest	B-3-09	CL	14
LS Slope	B-4-09	CL	30
WS Toe	B-7-09	CL	24
WS Toe	B-7-09	CL	21
WS Toe	B-8-09	CL	12
WS Toe	B-8-09	CL	12
LS Toe	B-9-09	CL	36
LS Toe	B-10-09	CL	18
	Assume Typical Value =		**24**
	Estimated ϕ' =		**30**

Location	Boring	USCS	PI
ML/CL-1 - ~El. 144 to 150			
Crest	B-2-09	ML/CL-1	12
WS Toe	B-8-09	ML/CL-1	20
LS Toe	B-10-09	ML/CL-1	22
LS Toe	B-10-09	ML/CL-1	12
	Assume Typical Value =		**20**
	Estimated ϕ' =		**31**

Location	Boring	USCS	PI
ML/CL-2 - ~El. 133 to 129			
LS Slope	B-4-09	ML/CL-2	46
	Assume Typical Value =		**46**
	Estimated ϕ' =		**25**

Foundation Drained Shear Strengths



(1) Lambe, T.W., and Whitman, R.V. (1969), Figure 21.4, page 307, Soil Mechanics, John Wiley & Sons Inc., New York.

ML/CL-1 Layer	CL Layer	ML/CL-2 Layer
ML/CL-1 Layer (~El. 144 to 150)	*CL Layer (GS to ~El. 152)*	*ML/CL-2 Layer (~El. 133 to 129)*
Typical PI = 20	**Typical PI = 24**	**Typical PI = 46**
ϕ = 31	**ϕ = 30**	**ϕ = 25**

Appendix 2.5

Dam Seismic Stability

Seismic Stability Assessment

Purpose:

This document summarizes the seismic stability assessment performed for the proposed Upper Sand Creek Detention Basin Dam.

Analysis Approach:

Our approach to this analysis includes the following steps:

1. Estimate the peak ground acceleration from available documentation.
2. Evaluate the liquefaction susceptibility and cyclic strength reduction of the foundation soils.
3. Evaluate the static slope stability using post-seismic shear strengths.
4. Estimate post-seismic deformations using the Makdisi and Seed modified Newmark approach (1977).

Summary:

For this site, the peak ground acceleration is estimated to be 0.30g for the 50th percentile event and 0.50 g for the 84th percentile event. As indicated in the Design Report, analysis of the dam response to the 50th percentile event is considered applicable to this project because the reservoir is a flood control project and will be normally dry. The likelihood of the simultaneous occurrence of a rare flood (resulting in significant water storage in the reservoir) and an extraordinary earthquake is very remote. However, the response to the 84th percentile event has also been estimated and is presented herein.

50th Percentile Event

Several of the foundation soils layers below El. 170 are susceptible to liquefaction or cyclic strength reduction for the 50th percentile event. The estimated post-seismic shear strengths for analysis are summarized below:

Soil Layer	Soil Type	Post-Seismic Strength Reduction	Sr (psf)	Su (psf)
ML	Fine-Grained	No	--	1,000
SC	Coarse-Grained	Yes	1,200	--
CL(Above El. 170)	Fine-Grained	No	--	1,600
CL (Below El. 170)	Fine-Grained	No	--	1,600
SM-1	Coarse-Grained	Yes	1,040	--
ML/CL-1	Fine-Grained	Yes	800	--
SM-2	Coarse-Grained	Yes	750	--
ML/CL-2	Fine-Grained	No	--	1,400
Bedrock	--	No	--	2,300

The post-seismic static stability analyses resulted in a factor of safety on the downstream slope of 1.6 and on the upstream slope of 2.2 for the critical case (Section B). The stability analysis results are summarized in the following table, and presented in Figures 1 through 6 (attached):

Station	Upstream Factor of Safety	Downstream Factor of Safety
14+05	2.2	1.6
14+70	2.5	2.4
15+55	3.0	2.0

Using the downstream post-seismic static stability for Section B as the critical case, multiple pseudo-static stability analyses were run to determine k_y. Assuming a PGA = 0.30g (50th percentile event), k_{max} = 0.18 g, the yield acceleration k_y = 0.14g, and k_y/k_{max} = 0.75.

Using the Makdisi and Seed approach and k_y/k_{max} = 0.75, we estimate the 50th percentile event could result in about 0.2 to 1 inches of horizontal displacement and an estimated vertical displacement of approximately 0 to 0.7 inches.

84th Percentile Event

Several of the foundation soils layers below El. 170 are susceptible to liquefaction or cyclic strength reduction for the 84th percentile event. The estimated post-seismic shear strengths for analysis are summarized below:

Soil Layer	Soil Type	Post-Seismic Strength Reduction	Sr (psf)	Su (psf)
ML	Fine-Grained	No	--	1,000
SC	Coarse-Grained	Yes	1,200	--
CL(Above El. 170)	Fine-Grained	No	--	1,600
CL (Below El. 170)	Fine-Grained	No	--	1,600
SM-1	Coarse-Grained	Yes	1,040	--
ML/CL-1	Fine-Grained	Yes	800	--
SM-2	Coarse-Grained	Yes	750	--
ML/CL-2	Fine-Grained	Yes	1,120	--
Bedrock	--	No	--	2,300

The only difference in the foundation post-seismic shear strength profile with respect to the 50th percentile event is the strength characterization for the ML/CL-2 layer. This is a very thin layer located deep in the foundation, and it does not affect the results of the stability analyses. Thus, the post-seismic static stability analyses for the 84th percentile event also resulted in a factor of safety on the downstream slope of 1.6 and on the upstream slope of 2.2 for the critical case (Section B). The stability analysis results are summarized in the following table, and presented in Figures 7 through 12 (attached):

Station	Upstream Factor of Safety	Downstream Factor of Safety
14+05	2.2	1.6
14+70	2.5	2.4
15+55	3.0	2.0

Using the downstream post-seismic static stability for Section B as the critical case, multiple pseudo-static stability analyses were run to determine k_y. Assuming a PGA = 0.50g (84th percentile event), k_{max} = 0.3g, the yield acceleration k_y = 0.13g, and k_y/k_{max} = 0.43.

Using the Makdisi and Seed approach and k_y/k_{max} = 0.43, we estimate the 84th percentile event could result in about 2 to 8 inches of horizontal displacement and an estimated vertical displacement of approximately 1.5 to 6 inches.

Analysis:

Estimate the Peak Horizontal Ground Accelerations

As described in Section 4.3 of the design report, the 50th percentile peak ground accelerations (PGA) is 0.30g and the 84th percentile PGA is 0.50g for a magnitude M_W 6.9 event.

Evaluate Liquefaction Susceptibility and Cyclic Strength Reduction of Foundation Soils

We evaluated the liquefaction susceptibility and cyclic strength softening of the foundation soils at the tallest cross section of the embankment dam for the 50th and 84th percentile seismic events.

Coarse-Grained Soils

For coarse-grained soils we corrected SPT blow counts from the field exploration program performed by Cal Engineering and Geology (CE&G). For each coarse grained soil layer in each boring, we corrected standard penetration N-values for sampler type, overburden, equipment energy, rod size effects and fines content. The N-values were further corrected to an equivalent clean sand value, $(N_1)_{60\text{-}cs}$, based on the procedures outlined in Seed (2003).

For each blow count we estimated a Cyclic Stress Ratio (CSR) and Cyclic Resistance Ratio (CRR). A factor of safety against liquefaction was determined by dividing CRR by CSR.

If the layer contained a blow count resulting in a factor of safety against liquefaction that was less than 1.0, the layer was considered liquefiable. If all the factors of safety in the layer were greater than 1.2, the layer was considered not liquefiable. If the factors of safety were between 1.0 and 1.2 we used some judgment to determine if the layer was liquefiable. In general, for this case, if most of the factors of safety were greater than 1.2 with the exception of a few in the 1.0 to 1.2 range, the soil was considered not liquefiable. If the factors of safety were mostly around 1.05 or less, then we considered the soil liquefiable.

If a soil layer was determined to be liquefiable, we estimated approximate residual undrained shear strengths (S_r) based on Fig. 4-14 from Seed (2010) and from Fig. 88 from Idriss and Boulanger (2004). To obtain values from these figures we used an $(N_1)_{60\text{-}cs}$ value in the lower 1/3rd of the range of $(N_1)_{60\text{-}cs}$ for the layer. The values obtained from Seed (2010) were less conservative than the values from Idriss and Boulanger (2004), therefore we conservatively used the values obtained from Idriss and Boulanger (2004) in our analyses.

Results of this analysis are presented in Table 1 for the 50th percentile PGA and Table 2 for the 84th percentile PGA. Based on the results of our evaluation the coarse-grained soils in layers SC, SM-1 and SM-2 were considered liquefiable under both PGA. The resulting undrained shear strengths were estimated at approximately 1200 psf in the SC layer, 1040 psf in the SM-1 layer, and 750 psf in the SM-2 layer (see the figure on the following page).

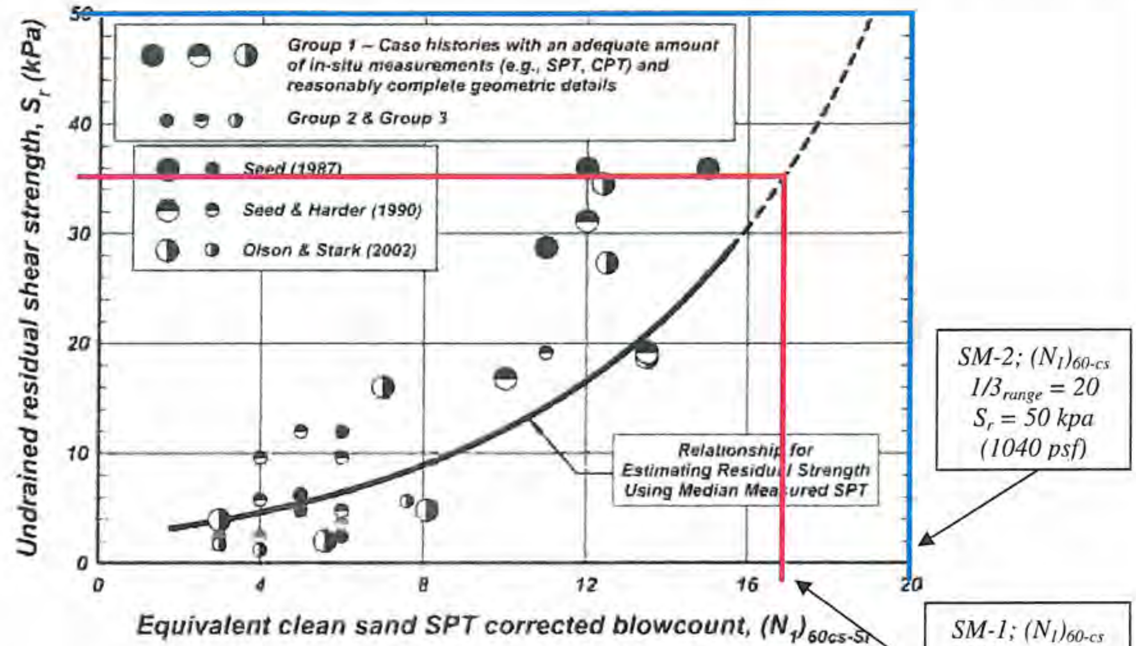


Figure 88. Residual shear strength of liquefied sand versus equivalent clean-sand SPT-corrected blow count, using the case histories published by Seed (1987), Seed and Harder (1990), and Olson and Stark (2002).

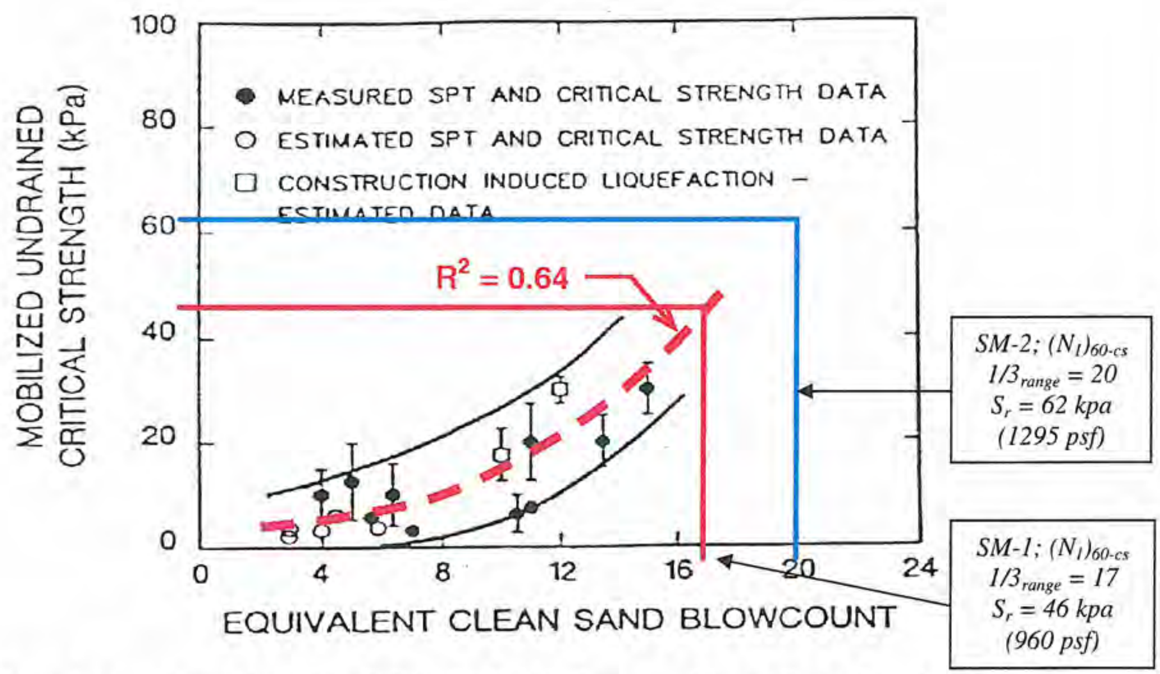


Figure 4-14: Regression of the Full-Scale Field Failure Case History Data of Seed and Harder (1990) Plotted in the Critical State Context as $S_{u,r}$ vs. $N_{1,60,CS}$.

Fine-Grained Soils

We evaluated the fine-grained foundation materials for susceptibility to strength reduction due to cyclic loading. We used the procedure detailed in Boulanger and Idriss (2004).

We assumed the earthquake event will occur after the foundation soils consolidate due to the loading of the proposed embankment. We evaluated the susceptibility to strength reduction under the embankment centerline. The results of the evaluation are presented in Table 3 for the 50th percentile PGA and Table 4 for the 84th percentile PGA. Based on our analysis, the ML/CL-1 layer is susceptible to strength reduction for both PGAs. The ML/CL-2 layer is not susceptible for the 50th percentile event, but is susceptible to strength reduction for the 84th percentile event.

We applied a strength reduction of 20% to all of the fine-grained foundation layers susceptible to strength reduction, based on recommendations by Karavanjian et al. (1997) for saturated and sensitive clays. The resulting undrained shear strengths were estimated to be 800 psf in the ML/CL-1 layer (for both 50th and 84th percentile seismic events) and 1,120 psf in the ML/CL-2 layer (for only the 84th percentile event).

Based on our assessment, we will use the following strengths for our post-seismic stability analyses:

Soil Layer	Soil Type	Post-Seismic Strength Reduction	Sr (psf)	Su (psf)
ML	Fine-Grained	No	--	1000
SC	Coarse-Grained	Yes	1,200	--
CL(Above El. 170)	Fine-Grained	No	--	1,600
CL (Below El. 170)	Fine-Grained	No	--	1,600
SM-1	Coarse-Grained	Yes	1,040	--
ML/CL-1	Fine-Grained	Yes	800	--
SM-2	Coarse-Grained	Yes	750	--
ML/CL-2	Fine-Grained	Yes	1,120 (84th)	1,400 (50th)
Bedrock	--	No	--	2,300

Evaluate Post-Seismic Slope Stability

We evaluated post-seismic slope stability analyses for the three cross sections (Section B, C and D) using SLOPE/W. This analysis used the post-liquefaction shear strengths and no seismic coefficient. The factors of safety for both the downstream and upstream slopes were greater than 1.0 indicating the dam embankment will be stable based on the estimated post-seismic shear strengths. Results of the static stability analyses are shown of Figures 1-12.

Evaluate Post-Seismic Deformation

Since the post-seismic stability analysis indicates the dam will be stable after the seismic event, we estimated the post-seismic deformation using the Makdisi and Seed (1977) modified Newmark method.

To complete this analysis, we estimated the maximum seismic acceleration at the dam crest (U_{max}) and we estimated the average acceleration for a potential sliding mass (k_{max}) to determine the yield acceleration (k_y). The yield acceleration (k_y) was determined by running a series of slope stability analyses with various horizontal accelerations to determine the acceleration that results in a factor of safety equal to 1.0. We then used k_y and k_{max} to estimate horizontal and vertical deformations.

A. Estimate the Maximum Seismic Acceleration at the Dam Crest, U_{max}:

Estimated the period of the earthquake (T_m) from Figure 2 of Bray et al. (1998):

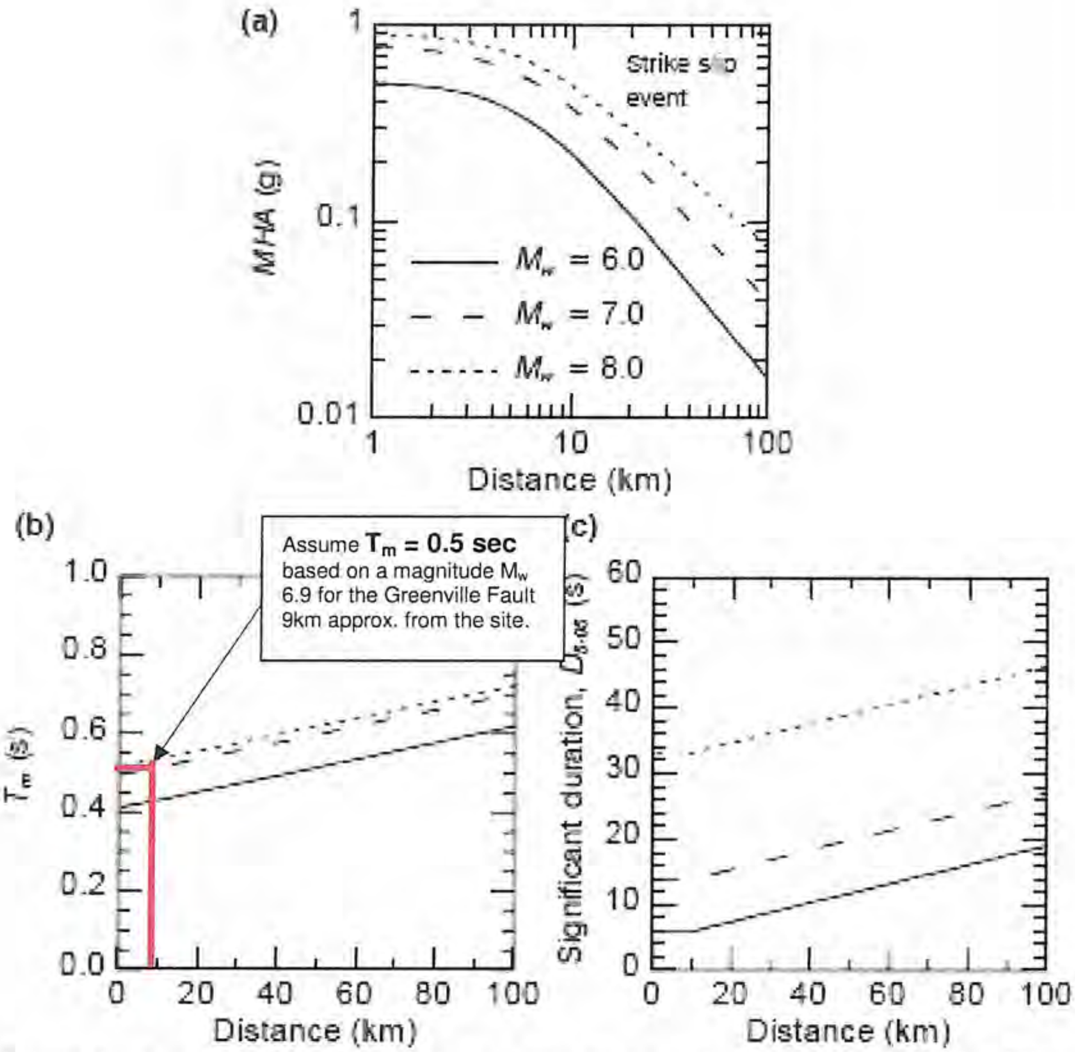


Figure 2. Simplified characterization of earthquake rock motions: (a) intensity, *MHA* for strike-slip faults (for reverse faults, use 1.3 × *MHA* for $M_w \geq 6.4$ and 1.64 × *MHA* for $M_w = 6.0$, with linear interpolation for $6.0 < M_w < 6.4$) (Abrahamson and Silva 1997); (b) frequency content, T_m (Rathje et al. 1998); (c) duration, D_{5-95} (Abrahamson and Silva 1996).

Estimate the period of the dam embankment (T_s) from:

$$T_s = 4H / Vs$$

Assuming:

H = Height of the dam embankment, estimated to be 40 feet

Vs = weighed average shear wave velocity of the sliding mass; estimate to be 985 feet/sec (300 meters/sec) based on geophysical investigations completed by NORCAL in October 2009.

Therefore,

$$T_s = 4 * 40 \text{ ft} / 985 \text{ ft/sec}$$

$$\underline{T_s = 0.16 \text{ sec}}$$

$$T_s / T_m = 0.16 \text{ sec} / 0.5 \text{ sec}$$

$$\underline{T_s / T_m = 0.32}$$

From Figure 8 of Bray et al. (1998):

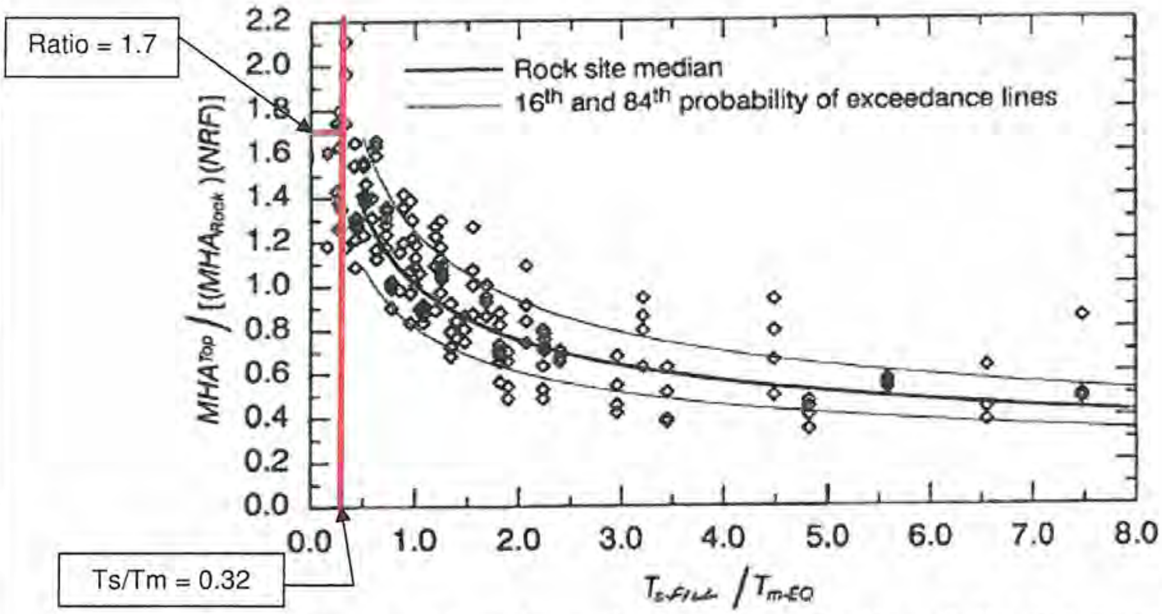


Figure 8. Normalized maximum horizontal acceleration at the top versus the normalized fundamental period of the earth fill (from Bray and Rathje 1998).

Estimate U_{max} by:

$$U_{max} = MHA_{top}$$

$$PGA = MHA_{rock}$$

$$U_{max} = 1.7 * MHA_{rock}$$

For the 50th Percentile Event:

$$U_{max} = 1.7 * 0.30 \text{ g}$$

$$\mathbf{U_{max} = 0.50\ g}$$

For the 84th Percentile Event:

$$U_{max} = 1.7 * 0.50 \text{ g}$$

$$\mathbf{U_{max} = 0.85 \text{ g}}$$

B. Estimate the Average Acceleration for a Potential Sliding Mass, k_{max}:

Estimate k_{max}/U_{max} ratio using Figure 10.18 of Sharma and Lewis (1994),

Where,

y is the height of the sliding mass, assumed to be = 40 feet
h, is the height of the embankment = 40 feet

Therefore, $y/h = 1.0$

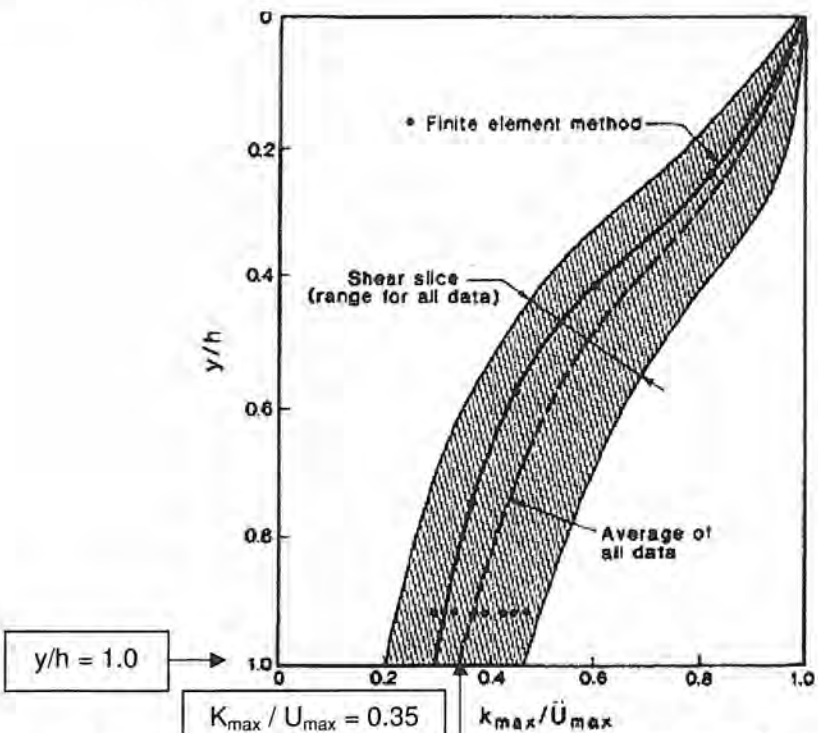


Figure 10.18 Variation of effective peak acceleration with depth of base of potential slide mass. (From Seed, 1979. Reproduced by permission of the Institution of Civil Engineers.)

From Figure 10.18, $k_{max}/U_{max} = 0.35$

For the 50th Percentile Event:

$$\text{Since } U_{max} = 0.50g$$

$$k_{max} = 0.35 * 0.50$$

$$\mathbf{k_{max} = 0.18\ g}$$

For the 84th Percentile Event:

$$\text{Since } U_{max} = 0.85g$$

$$k_{max} = 0.35 * 0.85\ g$$

$$\mathbf{k_{max} = 0.30\ g}$$

C. Estimate Yield Acceleration (k_y)

Based on our analysis, the yield acceleration (k_y) is estimated to be 0.14 g for the 50th percentile PGA.

50th Percentile Seismic Event k_y:

Estimate Yield Acceleration for 50th Percentile Event

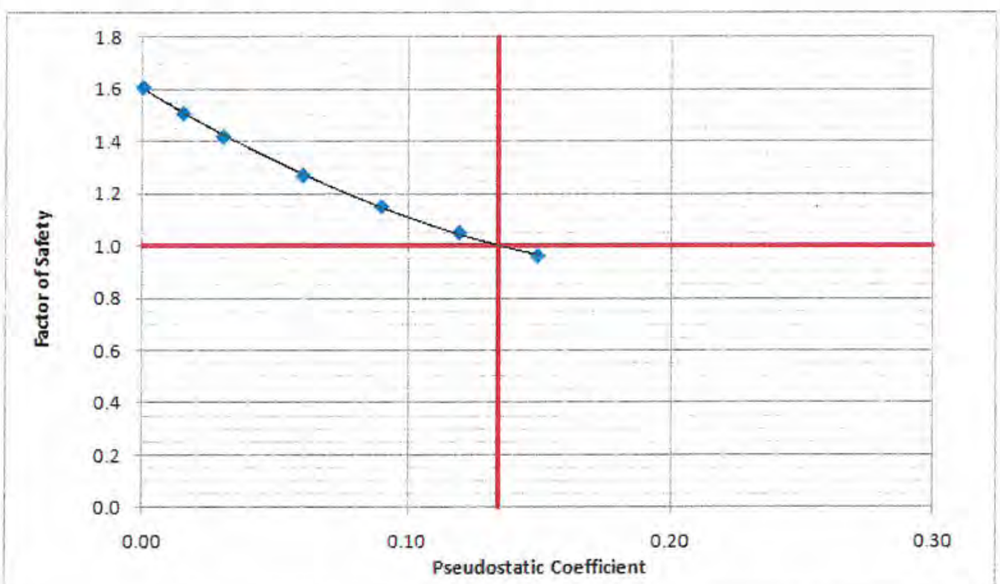
Critical case = Section B Landside Slope (Downstream Slope FOS = 1.61)

Summary of Runs:

Run	Psuedostatic Coefficient	Computed Factor of Safety
1	0.0000	1.61
2	0.0150	1.51
3	0.0300	1.42
4	0.0600	1.27
5	0.0900	1.15
6	0.1200	1.05
7	0.1500	0.96

Ky = 0.14

Ky/Kmax = 0.75



Based on our analysis, the yield acceleration (k_y) is estimated to be 0.13 g for the 84th percentile PGA.

84th Percentile Seismic Event k_y:

Estimate Yield Acceleration for 84th Percentile Event

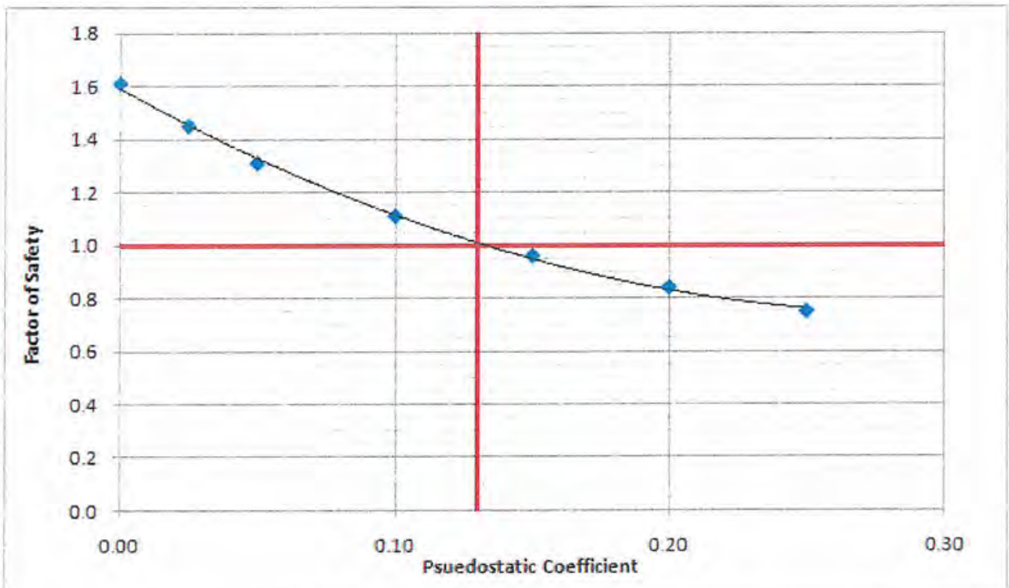
Critical case = Section B Landside Slope (Downstream Slope FOS = 1.61)

Summary of Runs:

Run	Psuedostatic Coefficient	Computed Factor of Safety
1	0.0000	1.61
2	0.0250	1.45
3	0.0500	1.31
4	0.1000	1.11
5	0.1500	0.96
6	0.2000	0.84
7	0.2500	0.75

Ky = 0.13

Ky/Kmax = 0.43



D. Estimate Horizontal and Vertical Displacement

For the 50th Percentile Event:

Estimate the k_y/k_{max} ratio:

Since $k_y = 0.14$ g
And $k_{max} = 0.18$ g
$k_y/k_{max} = 0.75$

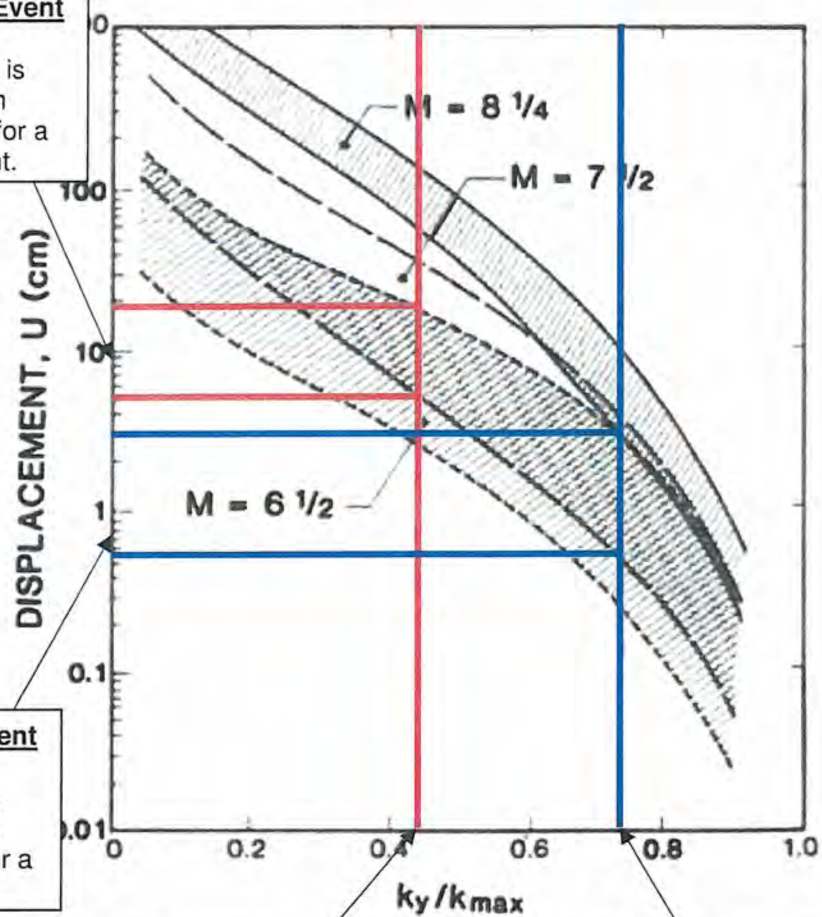
For the 84th Percentile Event:

Estimate the k_y/k_{max} ratio:

Since $k_y = 0.13$ g
And $k_{max} = 0.30$ g
$k_y/k_{max} = 0.43$

Use the Makdisi and Seed Chart (shown in Figure 10.23 of Sharma and Lewis (1994)) to estimate the horizontal displacement of the dam crest:

<u>84th Percentile Event</u>
Horizontal displacement is ~ 5 to 20 cm (2 to 8 inches) for a M = 6.9 event.



<u>50th Percentile Event</u>
Horizontal displacement is ~ 0.5 to 2.5 cm (0.2 to 1 inches) for a M = 6.9 event.

Figure 10.23 Variation of permanent displacement with yield acceleration. (From Makdisi and Seed, 1977.)

<u>84th Percentile Event</u>
$k_y/k_{max} \approx 0.43$

<u>50th Percentile Event</u>
$k_y/k_{max} \approx 0.75$

As recommended by URS (2007), the horizontal displacement was then multiplied by 0.7 to estimate a vertical displacement. This was considered the vertical deformation for the event.

Based on our analysis the vertical deformation is estimated to be 1.5 to 6 inches for the 84th percentile event and 0 to 0.7 inches for the 50th percentile event.

References:

(1) URS Corporation (2007), "Guidance Document for Geotechnical Analyses, DWR Urban Levee Geotechnical Evaluations Program", Report to DWR, Division of Flood Management, dated July 2, 2007.

(2) Seed, R.B., et. al. (2003), "Recent Advances in Soil Liquefaction Engineering: A Unified and Consistent Framework", 26th Annual ASCE Los Angeles Technical Spring Seminar, Keynote Presentation, Long Beach, CA, pp. 1-71.

(3) Seed, R.B., Harder, L.F.H., Jr. (1990), "SPT-based Analysis of Cyclic Pore Pressure Generation and Undrained Residual Shear Strength", in Proceedings of "*H.B. Seed Memorial Symposium*", Vol. II, May, pp. 351-376.

(4) Boulanger, R.W. and Idriss, I.M. (2004), "Evaluating the Potential for Liquefaction or Cyclic Failure of Silts and Clays", Report No. UCD/CGM-04/01, University of California at Davis.

(5) Sharma, H.D. and Lewis, S.P. (1994), "Waste Containment Systems, Waste Stabilization and Landfills Design and Evaluation."

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(7) Karavanjian, E. Jr., Matasovic, N., Hadj-Hamou, T., Sabatini, P.J. (1997) *Design Guidance: Geotechnical Earthquake Engineering for Highways, Vol. 1: Design Principles*, Geotechnical Engineering Circular 3, FHWA-SA-97-076, Federal Highways Administration, U.S. Department of Transportation, Washington, D.C.

(8) Seed, R.B., (2010), "Technical Review and Comments: 2008 EERI Monograph, *Soil Liquefaction during Earthquakes* by I.M. Idriss and R.W. Boulanger, Geotechnical Report No. UCB/GT – 2010/01, University of California at Berkeley, April 2010.

Table 1 - Coarse-Grained Liquefaction Triggering Analysis - <u>50th Percentile Event</u>
Upper Sand Creek Detention Basin
Contra Costa County Flood Control & Water Conservation District
Martinez, California

GEI

Response Period: 500 year
Estimated PGA for Seed's simplified method (updated by Idriss, 1999): 0.30 g
Earthquake Magnitude, M_w: 6.9
Magnitude Scaling Factor, MSF = 6.9 exp (-M_w/4) - 0.058, with MSF <= 1.8 (from equation 8a of Idriss and Boulanger (2004)
MSF = 1.17

By: D. DeCesris
Checked: M. Quinn

Date: May 12, 2010
Date: June 18, 2010

Boring	Layer	Estimated USCS	Depth (ft)	Elevation (ft)	N_{raw} Field	N_{gravel} Field [1]	Sigma (psf)	U (psf)	Sigma' (psf)	C_N [2]	N_1 [3]	C_R	C [4]	N_1[60] [5]	FC [6]	C_{fines} [7]	Equivalent $N_1[60]_{cs}$ [8]	r_d (M) [9]	CSR at M [10]	CSR_{eq} at 7.5 [11]	CRR_{eq} at 7.5 [12]	Factor of Safety [13]	Suceptable to Liquefaction?
B-3-09	SM-1	SM	32	145	9	9	3840	1061	2779	0.850	7.6	0.95	1.835	14	33	1.2	15	0.861	0.232	0.198	0.159	0.80	Y
B-3-09	SM-2	SM	42	135	24	24	5040	1685	3355	0.765	18.3	1.00	1.932	35	15	1.1	37	0.803	0.235	0.201	1.560	7.77	N
B-3-09	SM-2	SM	47	130	23	23	5640	1997	3643	0.728	16.7	1.00	1.932	32	15	1.1	33	0.774	0.234	0.200	0.819	4.11	N
B-7-09	SM-1	SM	33	149	14	14	3960	811	3149	0.793	11.1	0.95	1.835	20	43	6.0	26	0.855	0.210	0.179	0.327	1.83	N
B-7-09	SM-2	SM	38	144	15	15	4560	1123	3437	0.754	11.3	1	1.932	22	19	1.1	23	0.826	0.214	0.182	0.249	1.36	N
B-7-09	SM-2	SM	41	141	8	8	4920	1310	3610	0.732	5.9	1	1.932	11	19	1.2	12	0.809	0.215	0.183	0.136	0.74	Y
B-7-09	SM-2	SM	46	136	23	23	5520	1622	3898	0.699	16.1	1	1.932	31	26	1.1	32	0.780	0.215	0.184	0.664	3.61	N
B-8-09	SM-1	SC	18	148	12	12	2160	874	1286	1.194	14.3	0.85	1.642	24	50	6.0	30	0.936	0.306	0.262	0.457	1.75	N
B-8-09	SM-1	SC	21	145	10	10	2520	1061	1459	1.140	11.4	0.95	1.835	21	47	6.0	27	0.921	0.310	0.265	0.344	1.30	N
B-8-09	SM-2	SW	23	143	9	9	2760	1186	1574	1.107	10.0	0.95	1.835	18	5	1.0	19	0.910	0.311	0.266	0.197	0.74	Y
B-8-09	SM-2	SC	26	140	10	10	3120	1373	1747	1.061	10.6	0.95	1.835	19	25	1.2	21	0.894	0.311	0.266	0.214	0.80	Y
B-9-09	SC	SC	9	167	9	9	1020	--	1020	1.287	11.6	0.75	1.449	17	64	6.0	23	0.978	0.191	0.163	0.246	1.51	N
B-9-09	SM-1	SC	23	152	10	10	2760	624	2136	0.970	9.7	0.95	1.835	18	25	1.2	19	0.910	0.229	0.196	0.194	0.99	Y
B-9-09	SM-1	SM	25	150	15	15	3000	749	2251	0.946	14.2	0.95	1.835	26	22	1.1	27	0.900	0.234	0.200	0.352	1.77	N
B-9-09	SM-2	SC	37	138	14	14	4440	1498	2942	0.824	11.5	1.00	1.932	22	16	1.1	23	0.832	0.245	0.209	0.256	1.23	N
B-10-09	SM-1	SC	8	157	9	9	990	328	662	1.437	12.9	0.75	1.449	19	25	1.2	20	0.979	0.285	0.244	0.205	0.84	Y
B-10-09	SM-1	SC	13	152	8	8	1560	624	936	1.319	10.6	0.85	1.642	17	45	6.0	23	0.959	0.312	0.266	0.255	0.96	Y
B-10-09	SM-2	SW-SM	33	132	14	14	3960	1872	2088	0.980	13.7	0.95	1.835	25	10	1.1	26	0.855	0.316	0.270	0.323	1.20	Y
B-10-09	SM-2	SW-SM	38	128	14	14	4500	2153	2347	0.927	13.0	1.00	1.932	25	10	1.1	26	0.829	0.310	0.265	0.319	1.21	N

Notes

[1] No gravel correction is used. N_{gravel} is equal to N_{raw}
[2] C_N - Overburden correction factor from Youd et. al. (2001), where $C_N = 2.2/(1.2 + \sigma'_{vo}/Pa)$ with a maximum value of $C_N = 1.7$
[3] N_1 is $N_{raw} \times C_N$
[4] Correction factor for sampling, $C = C_H+C_L+C_D+C_R$
[5] $N_1[60] = N_1 \times C$
[6] Percent fines from laboratory testing — Percent fines estimated from boring logs
[7] C_{fines} based on Equation 7 from Seed (2003), $C_{fines} = (1+0.004 * FC)+0.05*(FC/N_1[60])$ with a maximum value of $C_{fines} = 6$
[8] $N_1[60]_{cs} = N_1[60] + C_{fines}$
[9] Stress Reduction Value from Idriss and Boulanger (2004), LN (r_d) = α (z) +β (z) M
[10] Cyclic stress ratio based on Equation 1 of Idriss and Boulanger (2004), CSR = $0.65*(\sigma_{vo} * PGA/\sigma'_{vo}) * r_d$
[11] Cyclic stress ratio for an earthquake magnitude, M = 7.5, based on Equation 2 of Idriss and Boulanger (2004), CSR_{eq} = CSR / MSF
[12] Cyclic resistance ratio for a Magnitude 7.5 earthquake, $CRR_{eq} = EXP\{(N_1[60]_{cs}/14.1) + (N_1[60]_{cs}/126)^2 - (N_1[60]_{cs}/23.6)^3 + (N_1[60]_{cs}/25.4)^4 - 2.8\}$
[13] Factor of safety is CRR_{eq} / CSR_{eq}
[14] From Taber Nov. 2009 SPT Energy Measurements, assume 100% efficiency: C_H = 100/60 = 1.6

C_H =	hammer	1.60	(Note 14)
C_L =	no liner	1.15	(No liner)
C_D =	hole size	1.05	6-inch OD hole
C_R =	rod length	0.75	(depth < 13')
		0.85	(13'< depth < 20')
		0.95	(20'< depth < 33')
		1.00	(33'< depth)

	$N_1[60]_{cs}$			
	Layer Average	Range Min	Max	Lower 1/3
SC	23	23	23	23
SM-1	23	15	30	20
SM-2	22	12	26	17

Excludes Equivalent N1[60]cs where FOS is > 1.20.

Table 2 - Coarse-Grained Liquefaction Triggering Analysis - 84th Percentile PGA
Upper Sand Creek Detention Basin
Contra Costa County Flood Control & Water Conservation District
Martinez, California

GEI

Response Period: 500 year
Estimated PGA for Seed's simplified method (updated by Idriss, 1999): 0.50 g
Earthquake Magnitude, M_w: 6.9
Magnitude Scaling Factor, MSF = 6.9 exp (-M_w/4) - 0.058, with MSF <= 1.8 (from equation 8a of Idriss and Boulanger (2004)
MSF = 1.17

By: D. DeCesris Date: May 12, 2010
Checked: M. Quinn Date: June 1, 2010

Boring	Layer	Estimated USCS	Depth (ft)	Elevation (ft)	N_{raw} Field	N_{gravel} Field [1]	Sigma (psf)	U (psf)	Sigma' (psf)	C_N [2]	N_1 [3]	C_R	C [4]	$N_1[60]$ [5]	FC [6]	C_{fines} [7]	Equivalent $N_1[60]_{cs}$ [8]	r_d (M) [9]	CSR at M [10]	CSR_{eq} at 7.5 [11]	CRR_{eq} at 7.5 [12]	Factor of Safety [13]	Suceptable to Liquefaction?
B-3-09	SM-1	SM	32	145	9	9	3840	1061	2779	0.850	7.6	0.95	1.835	14	33	1.2	15	0.861	0.386	0.330	0.159	0.48	Y
B-3-09	SM-2	SM	42	135	24	24	5040	1685	3355	0.765	18.3	1.00	1.932	35	15	1.1	37	0.803	0.392	0.335	1.560	4.66	N
B-3-09	SM-2	SM	47	130	23	23	5640	1997	3643	0.728	16.7	1.00	1.932	32	15	1.1	33	0.774	0.389	0.333	0.819	2.46	N
B-7-09	SM-1	SM	33	149	14	14	3960	811	3149	0.793	11.1	0.95	1.835	20	43	6.0	26	0.855	0.349	0.298	0.327	1.10	Y
B-7-09	SM-2	SM	38	144	15	15	4560	1123	3437	0.754	11.3	1	1.932	22	19	1.1	23	0.826	0.356	0.304	0.249	0.82	Y
B-7-09	SM-2	SM	41	141	8	8	4920	1310	3610	0.732	5.9	1	1.932	11	19	1.2	12	0.809	0.358	0.306	0.136	0.44	Y
B-7-09	SM-2	SM	46	136	23	23	5520	1622	3898	0.699	16.1	1	1.932	31	26	1.1	32	0.780	0.359	0.306	0.664	2.17	N
B-8-09	SM-1	SC	18	148	12	12	2160	874	1286	1.194	14.3	0.85	1.642	24	50	6.0	30	0.936	0.511	0.436	0.457	1.05	Y
B-8-09	SM-1	SC	21	145	10	10	2520	1061	1459	1.140	11.4	0.95	1.835	21	47	6.0	27	0.921	0.517	0.441	0.344	0.78	Y
B-8-09	SM-2	SW	23	143	9	9	2760	1186	1574	1.107	10.0	0.95	1.835	18	5	1.0	19	0.910	0.519	0.443	0.197	0.45	Y
B-8-09	SM-2	SC	26	140	10	10	3120	1373	1747	1.061	10.6	0.95	1.835	19	25	1.2	21	0.894	0.519	0.443	0.214	0.48	Y
B-9-09	SC	SC	9	167	9	9	1020	—	1020	1.287	11.6	0.75	1.449	17	64	6.0	23	0.978	0.318	0.271	0.246	0.90	Y
B-9-09	SM-1	SC	23	152	10	10	2760	624	2136	0.970	9.7	0.95	1.835	18	25	1.2	19	0.910	0.382	0.326	0.194	0.59	Y
B-9-09	SM-1	SM	25	150	15	15	3000	749	2251	0.946	14.2	0.95	1.835	26	22	1.1	27	0.900	0.390	0.333	0.352	1.06	Y
B-9-09	SM-2	SC	37	138	14	14	4440	1498	2942	0.824	11.5	1.00	1.932	22	16	1.1	23	0.832	0.408	0.348	0.256	0.74	Y
B-10-09	SM-1	SC	8	157	9	9	990	328	662	1.437	12.9	0.75	1.449	19	25	1.2	20	0.979	0.476	0.406	0.205	0.50	Y
B-10-09	SM-1	SC	13	152	8	8	1560	624	936	1.319	10.6	0.85	1.642	17	45	6.0	23	0.959	0.520	0.444	0.255	0.58	Y
B-10-09	SM-2	SW-SM	33	132	14	14	3960	1872	2088	0.980	13.7	0.95	1.835	25	10	1.1	26	0.855	0.527	0.450	0.323	0.72	Y
B-10-09	SM-2	SW-SM	38	128	14	14	4500	2153	2347	0.927	13.0	1.00	1.932	25	10	1.1	26	0.829	0.517	0.441	0.319	0.72	Y

Notes

[1] No gravel correction is used. N_{gravel} is equal to N_{raw}
[2] C_N - Overburden correction factor from Youd et. al. (2001), where $C_N = 2.2/(1.2 + \sigma'_{v0}/Pa)$ with a maximum value of $C_N = 1.7$
[3] N_1 is $N_{raw} \times C_N$
[4] Correction factor for sampling, $C = C_H + C_L + C_D + C_R$
[5] $N_1[60] = N_1 \times C$
[6] Percent fines from laboratory testing   Percent fines estimated from boring logs
[7] C_{fines} based on Equation 7 from Seed (2003), $C_{fines} = (1+0.004 * FC)+0.05*(FC/N_1[60])$ with a maximum value of $C_{fines} = 6$
[8] $N_1[60]_{cs} = N_1[60] + C_{fines}$
[9] Stress Reduction Value from Idriss and Boulanger (2004), $LN(r_d) = \alpha(z) + \beta(z) M$
[10] Cyclic stress ratio based on Equation 1 of Idriss and Boulanger (2004), $CSR = 0.65*(\sigma_{vo} * PGA/\sigma'_{vo}) * r_d$
[11] Cyclic stress ratio for an earthquake magnitude, M = 7.5, based on Equation 2 of Idriss and Boulanger (2004), $CSR_{eq} = CSR / MSF$
[12] Cyclic resistance ratio for a Magnitude 7.5 earthquake, $CRR_{eq} = EXP((N_1[60]_{cs}/14.1) + (N_1[60]_{cs}/126)^2 - (N_1[60]_{cs}/23.6)^3 + (N_1[60]_{cs}/25.4)^4 - 2.8)$
[13] Factor of safety is CRR_{eq} / CSR_{eq}
[14] From Taber Nov. 2009 SPT Energy Measurements, assume 100% efficiency: $C_H = 100/60 = 1.6$.

C_H =	hammer	1.60	(Note 14)
C_L =	no liner	1.15	(No liner)
C_D =	hole size	1.05	6-inch OD hole
C_R =	rod length	0.75	(depth < 13')
		0.85	(13'< depth < 20')
		0.95	(20'< depth < 33')
		1.00	(33'< depth)

	$N_1[60]_{cs}$			
	Layer Average	Range Min	Max	Lower 1/3
SC	23	23	23	23
SM-1	23	15	30	20
SM-2	22	12	26	17
Excludes Equivalent N1[60]cs where FOS is > 1.20.				

Table 3 - Evaluation of Potential for Cyclic Strength Reduction in Fine-Grained Soils -50th Percentile Event
Upper Sand Creek Detention Basin
Contra Costa County Flood Control & Water Conservation District
Martinez, California

GEI Consultants

By: D. DeCesris Date: May 12, 2010
Checked: M. Quinn Date: June 18, 2010

Response Period: 500 year
Estimated PGA for Seed's simplified method (updated by Idriss, 1999): 0.30 g
Earthquake Magnitude, M_w: 6.9
Magnitude Scaling Factor, MSF = 1.12 exp ($-M_w/4$) + 0.828, with MSF <= 1.13 (from equation 8a of Idriss and Boulanger (2004)
MSF = 1.03
Seed's (1983) initial static shear stress correction factor, K_α (assume K_o conditions, so $K_\alpha = 1$)
K_α = 1

Layer	Elevation (ft)	Depth (ft)	Depth (m)	σ_{vc} (psf)	σ'_{vc} (psf)	α	β	r_d [1]	CSR [2]	S_u (psf) [3]	S_u/σ'_{vc}	CRR at M=7.5 [4]	CRR at M_w [5]	Factor of Safety [6]	Susceptible to cyclic strength reduction?
								Under Embankment							
	183	12	3.6576	1440	1440	-0.175	0.020	0.964	0.188	1600	1.111	0.889	0.913	4.86	No
	182	13	3.9624	1560	1560	-0.195	0.022	0.959	0.187	1600	1.026	0.821	0.843	4.51	No
	181	14	4.2672	1680	1680	-0.215	0.024	0.955	0.186	1600	0.952	0.762	0.783	4.20	No
	180	15	4.572	1800	1800	-0.236	0.027	0.950	0.185	1600	0.889	0.711	0.731	3.94	No
	179	16	4.8768	1920	1920	-0.257	0.029	0.946	0.184	1600	0.833	0.667	0.685	3.72	No
	178	17	5.1816	2040	2040	-0.279	0.032	0.941	0.183	1600	0.784	0.627	0.645	3.51	No
	177	18	5.4864	2160	2160	-0.302	0.034	0.936	0.182	1600	0.741	0.593	0.609	3.34	No
	176	19	5.7912	2280	2280	-0.325	0.037	0.931	0.182	1600	0.702	0.561	0.577	3.18	No
	175	20	6.096	2400	2400	-0.348	0.039	0.926	0.181	1600	0.667	0.533	0.548	3.04	No
	174	21	6.4008	2520	2520	-0.372	0.042	0.921	0.180	1600	0.635	0.508	0.522	2.91	No
	173	22	6.7056	2640	2640	-0.396	0.045	0.916	0.179	1600	0.606	0.485	0.498	2.79	No
	172	23	7.0104	2760	2760	-0.421	0.047	0.910	0.178	1600	0.580	0.464	0.477	2.68	No
	171	24	7.3152	2880	2880	-0.446	0.050	0.905	0.176	1600	0.556	0.444	0.457	2.59	No
	170	25	7.62	3000	3000	-0.472	0.053	0.900	0.175	1600	0.533	0.427	0.438	2.50	No
CL	169	26	7.9248	3120	3120	-0.497	0.056	0.894	0.174	1600	0.513	0.410	0.422	2.42	No
	168	27	8.2296	3240	3240	-0.524	0.059	0.889	0.173	1600	0.494	0.395	0.406	2.34	No
	167	28	8.5344	3360	3360	-0.550	0.062	0.883	0.172	1600	0.476	0.381	0.391	2.27	No
	166	29	8.8392	3480	3480	-0.577	0.065	0.878	0.171	1600	0.460	0.368	0.378	2.21	No
	165	30	9.144	3600	3974	-0.604	0.068	0.872	0.154	1600	0.403	0.322	0.331	2.15	No
	164	31	9.4488	3720	4032	-0.632	0.071	0.866	0.156	1600	0.397	0.317	0.326	2.09	No
	163	32	9.7536	3840	4090	-0.659	0.074	0.861	0.158	1600	0.391	0.313	0.322	2.04	No
	162	33	10.0584	3960	4147	-0.687	0.077	0.855	0.159	1600	0.386	0.309	0.317	1.99	No
	161	34	10.3632	4080	4205	-0.715	0.080	0.849	0.161	1600	0.381	0.304	0.313	1.95	No
	160	35	10.668	4200	4262	-0.744	0.083	0.843	0.162	1600	0.375	0.300	0.309	1.90	No
	159	36	10.9728	4320	4320	-0.772	0.086	0.838	0.163	1600	0.370	0.296	0.304	1.86	No
	158	37	11.2776	4440	4378	-0.801	0.089	0.832	0.165	1600	0.365	0.292	0.300	1.83	No
	157	38	11.5824	4560	4435	-0.830	0.093	0.826	0.166	1600	0.361	0.289	0.297	1.79	No
	156	39	11.8872	4680	4493	-0.858	0.096	0.820	0.167	1600	0.356	0.285	0.293	1.76	No
	155	40	12.192	4800	4550	-0.887	0.099	0.814	0.168	1600	0.352	0.281	0.289	1.73	No
	154	41	12.4968	4920	4608	-0.917	0.102	0.809	0.168	1600	0.347	0.278	0.285	1.70	No
	145	50	15.24	6000	5126	-1.179	0.131	0.757	0.173	1000	0.195	0.156	0.160	0.93	Yes
	144	51	15.5448	6120	5184	-1.208	0.134	0.752	0.173	1000	0.193	0.154	0.159	0.92	Yes
ML/CL-1	143	52	15.8496	6240	5242	-1.237	0.137	0.746	0.173	1000	0.191	0.153	0.157	0.91	Yes
	142	53	16.1544	6360	5299	-1.265	0.140	0.740	0.173	1000	0.189	0.151	0.155	0.90	Yes
	141	54	16.4592	6480	5357	-1.294	0.143	0.735	0.173	1000	0.187	0.149	0.153	0.89	Yes
	140	55	16.764	6600	5414	-1.322	0.146	0.730	0.173	1000	0.185	0.148	0.152	0.88	Yes
	131	64	19.5072	7680	5933	-1.564	0.171	0.683	0.172	1400	0.236	0.189	0.194	1.13	No
ML/CL-2	130	65	19.812	7800	5990	-1.590	0.174	0.678	0.172	1400	0.234	0.187	0.192	1.12	No
	129	66	20.1168	7920	6048	-1.615	0.177	0.673	0.172	1400	0.231	0.185	0.190	1.11	No

[1] Stress Reduction Value from Boulanger and Idriss (2004), LN (r_d) = α (z) +β (z) M
[2] Cyclic stress ratio based on Equation 3-2 of Idriss and Boulanger (2004), CSR = 0.65*(σ_{vo} * PGA/σ'_{vo}) * r_d
[3] Undrained shear strengths based on values provided in Geotechnical Parameter Memo.
[4] Cyclic resistance ratio for an earthquake magnitude, M = 7.5, based on Equation 3-29 of Boulanger and Idriss (2004), CRR = 0.8 * (S_u / σ'_{vc}) * K_α
[5] Cyclic resistance ratio for the design earthquake magnitude, Mw, based on Equation 3-4 of Boulanger and Idriss, $CRR_M = CRR_{M=7.5}$ * MSF
[6] Factor of safety is CRR_M / CSR_M

Table 4 - Evaluation of Potential for Cyclic Strength Reduction in Fine-Grained Soils -84th Percentile Event
Upper Sand Creek Detention Basin
Contra Costa County Flood Control & Water Conservation District
Martinez, California

GEI Consultants

By: D. DeCesris Date: May 12, 2010
Checked: M. Quinn Date: June 1, 2010

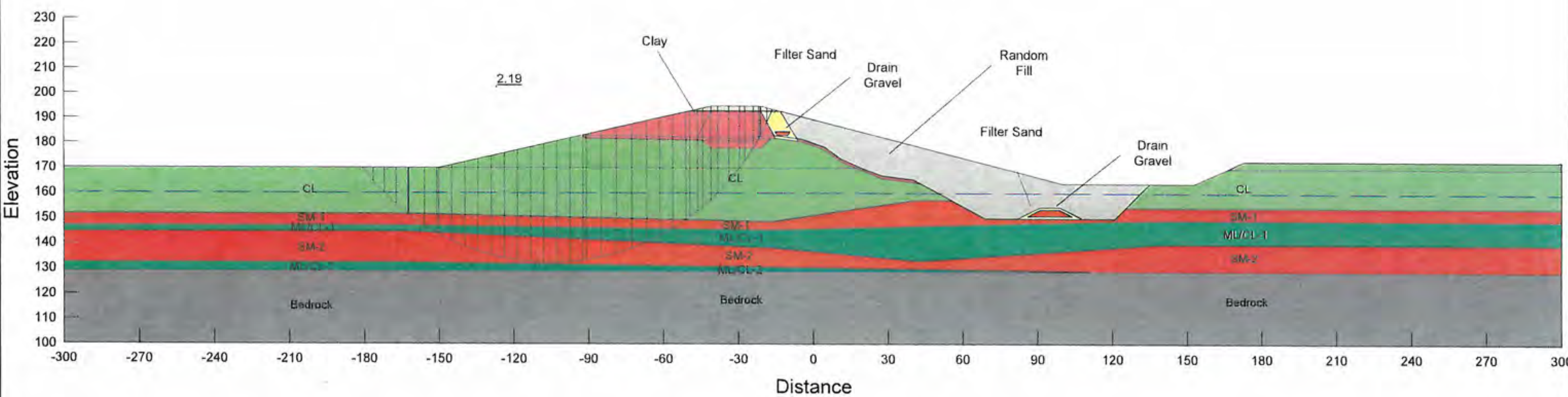
Response Period: 500 year
Estimated PGA for Seed's simplified method (updated by Idriss, 1999): 0.50 g
Earthquake Magnitude, M_w: 6.9
Magnitude Scaling Factor, MSF = 1.12 exp ($-M_w$/4) + 0.828, with MSF <= 1.13 (from equation 8a of Idriss and Boulanger (2004)
MSF = 1.03
Seed's (1983) initial static shear stress correction factor, K_α (assume K_0 conditions, so K_α = 1)
K_α = 1

Layer	Elevation (ft)	Depth (ft)	Depth (m)	σ_{vc} (psf)	σ'_{vc} (psf)	α	β	r_d [1]	CSR [2]	S_u (psf) [3]	S_u/σ'_{vc}	CRR at M=7.5 [4]	CRR at M_w [5]	Factor of Safety [6]	Susceptible to cyclic strength reduction?
Under Embankment															
	183	12	3.6576	1440	1440	-0.175	0.020	0.964	0.313	1600	1.111	0.889	0.913	2.92	No
	182	13	3.9624	1560	1560	-0.195	0.022	0.959	0.312	1600	1.026	0.821	0.843	2.70	No
	181	14	4.2672	1680	1680	-0.215	0.024	0.955	0.310	1600	0.952	0.762	0.783	2.52	No
	180	15	4.572	1800	1800	-0.236	0.027	0.950	0.309	1600	0.889	0.711	0.731	2.37	No
	179	16	4.8768	1920	1920	-0.257	0.029	0.946	0.307	1600	0.833	0.667	0.685	2.23	No
	178	17	5.1816	2040	2040	-0.279	0.032	0.941	0.306	1600	0.784	0.627	0.645	2.11	No
	177	18	5.4864	2160	2160	-0.302	0.034	0.936	0.304	1600	0.741	0.593	0.609	2.00	No
	176	19	5.7912	2280	2280	-0.325	0.037	0.931	0.303	1600	0.702	0.561	0.577	1.91	No
	175	20	6.096	2400	2400	-0.348	0.039	0.926	0.301	1600	0.667	0.533	0.548	1.82	No
	174	21	6.4008	2520	2520	-0.372	0.042	0.921	0.299	1600	0.635	0.508	0.522	1.74	No
	173	22	6.7056	2640	2640	-0.396	0.045	0.916	0.298	1600	0.606	0.485	0.498	1.67	No
	172	23	7.0104	2760	2760	-0.421	0.047	0.910	0.296	1600	0.580	0.464	0.477	1.61	No
	171	24	7.3152	2880	2880	-0.446	0.050	0.905	0.294	1600	0.556	0.444	0.457	1.55	No
	170	25	7.62	3000	3000	-0.472	0.053	0.900	0.292	1600	0.533	0.427	0.438	1.50	No
CL	169	26	7.9248	3120	3120	-0.497	0.056	0.894	0.291	1600	0.513	0.410	0.422	1.45	No
	168	27	8.2296	3240	3240	-0.524	0.059	0.889	0.289	1600	0.494	0.395	0.406	1.41	No
	167	28	8.5344	3360	3360	-0.550	0.062	0.883	0.287	1600	0.476	0.381	0.391	1.36	No
	166	29	8.8392	3480	3480	-0.577	0.065	0.878	0.285	1600	0.460	0.368	0.378	1.33	No
	165	30	9.144	3600	3974	-0.604	0.068	0.872	0.257	1600	0.403	0.322	0.331	1.29	No
	164	31	9.4488	3720	4032	-0.632	0.071	0.866	0.260	1600	0.397	0.317	0.326	1.26	No
	163	32	9.7536	3840	4090	-0.659	0.074	0.861	0.263	1600	0.391	0.313	0.322	1.22	No
	162	33	10.0584	3960	4147	-0.687	0.077	0.855	0.265	1600	0.386	0.309	0.317	1.20	No
	161	34	10.3632	4080	4205	-0.715	0.080	0.849	0.268	1600	0.381	0.304	0.313	1.17	No
	160	35	10.668	4200	4262	-0.744	0.083	0.843	0.270	1600	0.375	0.300	0.309	1.14	No
	159	36	10.9728	4320	4320	-0.772	0.086	0.838	0.272	1600	0.370	0.296	0.304	1.12	No
	158	37	11.2776	4440	4378	-0.801	0.089	0.832	0.274	1600	0.365	0.292	0.300	1.10	No
	157	38	11.5824	4560	4435	-0.830	0.093	0.826	0.276	1600	0.361	0.289	0.297	1.07	No
	156	39	11.8872	4680	4493	-0.858	0.096	0.820	0.278	1600	0.356	0.285	0.293	1.05	No
	155	40	12.192	4800	4550	-0.887	0.099	0.814	0.279	1600	0.352	0.281	0.289	1.04	No
	154	41	12.4968	4920	4608	-0.917	0.102	0.809	0.281	1600	0.347	0.278	0.285	1.02	No
	145	50	15.24	6000	5126	-1.179	0.131	0.757	0.288	1000	0.195	0.156	0.160	0.56	Yes
	144	51	15.5448	6120	5184	-1.208	0.134	0.752	0.288	1000	0.193	0.154	0.159	0.55	Yes
ML/CL-1	143	52	15.8496	6240	5242	-1.237	0.137	0.746	0.289	1000	0.191	0.153	0.157	0.54	Yes
	142	53	16.1544	6360	5299	-1.265	0.140	0.740	0.289	1000	0.189	0.151	0.155	0.54	Yes
	141	54	16.4592	6480	5357	-1.294	0.143	0.735	0.289	1000	0.187	0.149	0.153	0.53	Yes
	140	55	16.764	6600	5414	-1.322	0.146	0.730	0.289	1000	0.185	0.148	0.152	0.53	Yes
	131	64	19.5072	7680	5933	-1.564	0.171	0.683	0.287	1400	0.236	0.189	0.194	0.68	Yes
ML/CL-2	130	65	19.812	7800	5990	-1.590	0.174	0.678	0.287	1400	0.234	0.187	0.192	0.67	Yes
	129	66	20.1168	7920	6048	-1.615	0.177	0.673	0.287	1400	0.231	0.185	0.190	0.66	Yes

[1] Stress Reduction Value from Boulanger and Idriss (2004), LN (r_d) = α (z) +β (z) M
[2] Cyclic stress ratio based on Equation 3-2 of Idriss and Boulanger (2004), CSR = 0.65*(σ_{vo} * PGA/σ'_{vo}) * r_d
[3] Undrained shear strengths based on values provided in Geotechnical Parameter Memo.
[4] Cyclic resistance ratio for an earthquake magnitude, M = 7.5, based on Equation 3-29 of Boulanger and Idriss (2004), CRR = 0.8 * (S_u / σ'_{vc}) * K_α
[5] Cyclic resistance ratio for the design earthquake magnitude, Mw, based on Equation 3-4 of Boulanger and Idriss, CRR_M = $CRR_{M=7.5}$ * MSF
[6] Factor of safety is CRR_M / CSR_M

Upper Sand Creek - Section B (50th Percentile)
Name: Post-Liquefaction - Upstream Slope
Method: Spencer
Last Modified: 6/18/2010 8:03:00 AM

(1) Embankment - Clay: Unit Weight = 125 pcf, phi = 14 °, c = 300 psf
(2) CL (Above El. 170): Unit Weight = 120 pcf, phi = 0 °, c = 1600 psf
(3) CL (Below El. 170): Unit Weight = 120 pcf, phi = 0 °, c = 1600 psf
(4) SM-1: Unit Weight = 120 pcf, phi = 0 °, c = 1040 psf
(5) ML/CL-1: Unit Weight = 120 pcf, phi = 0 °, c = 800 psf
(6) SM-2: Unit Weight = 120 pcf, phi = 0 °, c = 750 psf
(7) ML/CL-2: Unit Weight = 120 pcf, phi = 0 °, c = 1400 psf
(8) Embankment - Filer Sand: Unit Weight = 125 pcf, phi = 35 °, c = 0 psf
(9) Embankment - Drain Gravel: Unit Weight = 125 pcf, phi = 35 °, c = 0 psf
(10) Embankment - Random Fill: Unit Weight = 125 pcf, phi = 14 °, c = 300 psf
(11) Bedrock: Unit Weight = 125 pcf, phi = 0 °, c = 2300 psf

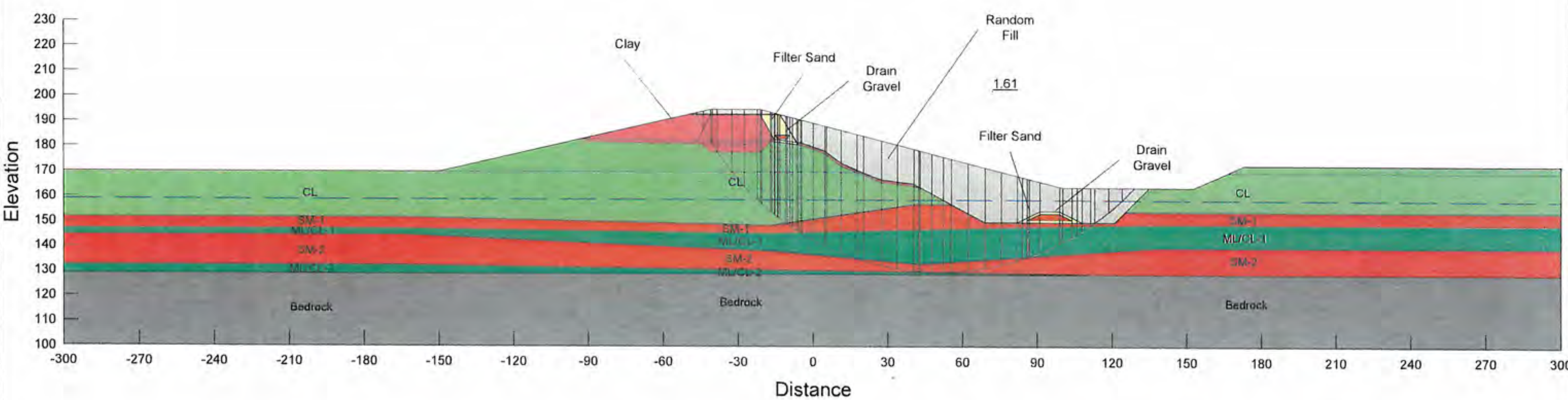


Notes:
1.Elevations in NAVD88
2.The Factor of Safety (FS) value above is for the critical failure surface

Upper Sand Creek Detention Basin Antioch, California	GEI Consultants	**Section B** Post-Seismic Slope Stability Upstream Slope (50^{th} Percentile Seismic Event)
Contra Costa County Public Works Department Martinez, California	Project 08318-0	June 2010 Fig. 1

Upper Sand Creek - Section B (50th Percentile)
Name: Post-Liquefaction - Downstream Slope
Method: Spencer
Last Modified: 6/18/2010 8:03:00 AM

(1) Embankment - Clay: Unit Weight = 125 pcf, phi = 14 °, c = 300 psf
(2) CL (Above El. 170): Unit Weight = 120 pcf, phi = 0 °, c = 1600 psf
(3) CL (Below El. 170): Unit Weight = 120 pcf, phi = 0 °, c = 1600 psf
(4) SM-1: Unit Weight = 120 pcf, phi = 0 °, c = 1040 psf
(5) ML/CL-1: Unit Weight = 120 pcf, phi = 0 °, c = 800 psf
(6) SM-2: Unit Weight = 120 pcf, phi = 0 °, c = 750 psf
(7) ML/CL-2: Unit Weight = 120 pcf, phi = 0 °, c = 1400 psf
(8) Embankment - Filer Sand: Unit Weight = 125 pcf, phi = 35 °, c = 0 psf
(9) Embankment - Drain Gravel: Unit Weight = 125 pcf, phi = 35 °, c = 0 psf
(10) Embankment - Random Fill: Unit Weight = 125 pcf, phi = 14 °, c = 300 psf
(11) Bedrock: Unit Weight = 125 pcf, phi = 0 °, c = 2300 psf

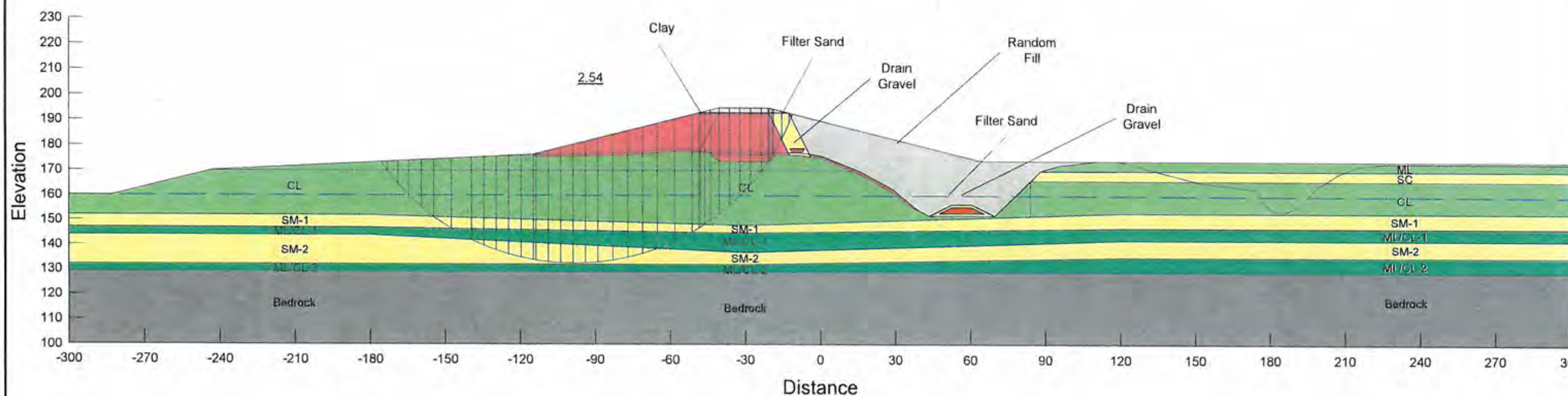


Notes:
1.Elevations in NAVD88
2.The Factor of Safety (FS) value above is for the critical failure surface

Upper Sand Creek Detention Basin Antioch, California	GEI Consultants	**Section B** Post-Seismic Slope Stability Downstream Slope (50th Percentile Seismic Event)
Contra Costa County Public Works Department Martinez, California	Project 08318-0	June 2010 Fig. 2

Upper Sand Creek - Section C (50th Percentile)
Name: Post-Liquefaction - Upstream
Method: Spencer
Last Modified: 6/18/2010 8:07:10 AM

(1) Embankment - Clay: Unit Weight = 125 pcf, phi = 14 °, c = 300 psf
(2) ML: Unit Weight = 120 pcf, phi = 0 °, c = 1000 psf
(3) SC: Unit Weight = 120 pcf, phi = 0 °, c = 1200 psf
(4) CL (Above El. 170): Unit Weight = 120 pcf, phi = 0 °, c = 1600 psf
(5) CL (Below El. 170): Unit Weight = 120 pcf, phi = 0 °, c = 1600 psf
(6) SM-1: Unit Weight = 120 pcf, phi = 0 °, c = 1040 psf
(7) ML/CL-1: Unit Weight = 120 pcf, phi = 0 °, c = 800 psf
(8) SM-2: Unit Weight = 120 pcf, phi = 0 °, c = 750 psf
(9) ML/CL-2: Unit Weight = 120 pcf, phi = 0 °, c = 1400 psf
(10) Embankment - Filter Sand: Unit Weight = 125 pcf, phi = 35 °, c = 0 psf
(11) Embankment - Drain Gravel: Unit Weight = 125 pcf, phi = 35 °, c = 0 psf
(12) Embankment - Random Fill: Unit Weight = 125 pcf, phi = 14 °, c = 300 psf
(13) Bedrock: Unit Weight = 125 pcf, phi = 0 °, c = 2300 psf

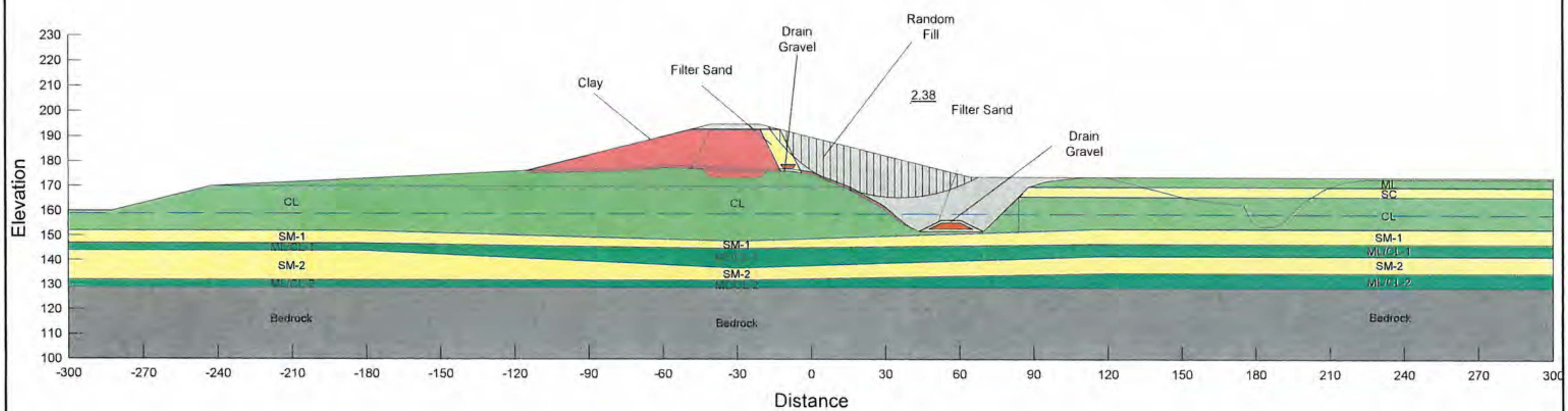


Notes:
1.Elevations in NAVD88
2.The Factor of Safety (FS) value above is for the critical failure surface

Upper Sand Creek Detention Basin Antioch, California	GEI Consultants	**Section C** Post-Seismic Slope Stability Upstream Slope (50th Percentile Seismic Event)
Contra Costa County Public Works Department Martinez, California	Project 08318-0	June 2010 Fig. 3

Upper Sand Creek - Section C (50th Percentile)
Name: Post-Liquefaction - Downstream
Method: Spencer
Last Modified: 6/18/2010 8:07:10 AM

(1) Embankment - Clay: Unit Weight = 125 pcf, phi = 14 °, c = 300 psf
(2) ML: Unit Weight = 120 pcf, phi = 0 °, c = 1000 psf
(3) SC: Unit Weight = 120 pcf, phi = 0 °, c = 1200 psf
(4) CL (Above El. 170): Unit Weight = 120 pcf, phi = 0 °, c = 1600 psf
(5) CL (Below El. 170): Unit Weight = 120 pcf, phi = 0 °, c = 1600 psf
(6) SM-1: Unit Weight = 120 pcf, phi = 0 °, c = 1040 psf
(7) ML/CL-1: Unit Weight = 120 pcf, phi = 0 °, c = 800 psf
(8) SM-2: Unit Weight = 120 pcf, phi = 0 °, c = 750 psf
(9) ML/CL-2: Unit Weight = 120 pcf, phi = 0 °, c = 1400 psf
(10) Embankment - Filter Sand: Unit Weight = 125 pcf, phi = 35 °, c = 0 psf
(11) Embankment - Drain Gravel: Unit Weight = 125 pcf, phi = 35 °, c = 0 psf
(12) Embankment - Random Fill: Unit Weight = 125 pcf, phi = 14 °, c = 300 psf
(13) Bedrock: Unit Weight = 125 pcf, phi = 0 °, c = 2300 psf

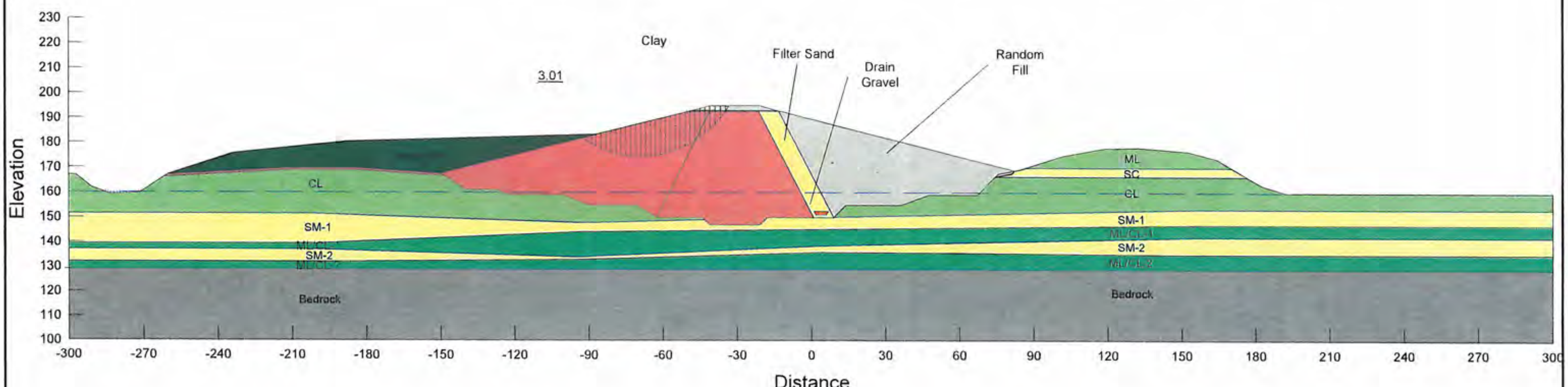


Notes:
1.Elevations in NAVD88
2.The Factor of Safety (FS) value above is for the critical failure surface

Upper Sand Creek Detention Basin Antioch, California	GEI Consultants	**Section C** Post-Seismic Slope Stability Downstream Slope (50th Percentile Seismic Event)	
Contra Costa County Public Works Department Martinez, California	Project 08318-0	June 2010	Fig. 4

Upper Sand Creek - Section D (50th Percentile)
Name: Post-Liquefaction - Upstream
Method: Spencer
Last Modified: 6/18/2010 8:12:33 AM

(1) Embankment - Clay: Unit Weight = 125 pcf, phi = 14 °, c = 300 psf
(2) ML: Unit Weight = 120 pcf, phi = 0 °, c = 1000 psf
(3) SC: Unit Weight = 120 pcf, phi = 0 °, c = 1200 psf
(4) CL: Unit Weight = 120 pcf, phi = 0 °, c = 1600 psf
(5) SM-1: Unit Weight = 120 pcf, phi = 0 °, c = 1040 psf
(6) ML/CL-1: Unit Weight = 120 pcf, phi = 0 °, c = 800 psf
(7) SM-2: Unit Weight = 120 pcf, phi = 0 °, c = 750 psf
(8) ML/CL-2: Unit Weight = 120 pcf, phi = 0 °, c = 1400 psf
(9) Embankment - Filter Sand: Unit Weight = 125 pcf, phi = 35 °, c = 0 psf
(10) Embankment - Drain Gravel: Unit Weight = 125 pcf, phi = 35 °, c = 0 psf
(11) Embankment - Random Fill: Unit Weight = 125 pcf, phi = 14 °, c = 300 psf
(12) Basin Fill: Unit Weight = 125 pcf, phi = 14 °, c = 300 psf
(13) Bedrock: Unit Weight = 125 pcf, phi = 0 °, c = 2300 psf

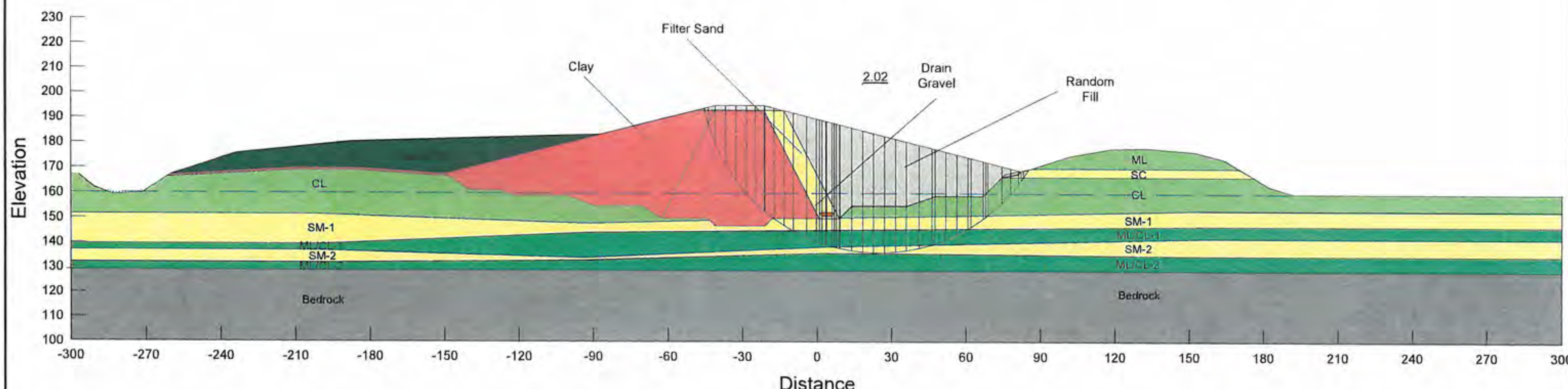


Notes:
1.Elevations in NAVD88
2.The Factor of Safety (FS) value above is for the critical failure surface

Upper Sand Creek Detention Basin Antioch, California	GEI Consultants	**Section D** Post-Seismic Slope Stability Upstream Slope (50th Percentile Seismic Event)
Contra Costa County Public Works Department Martinez, California	Project 08318-0	June 2010 Fig. 5

Upper Sand Creek - Section D (50th Percentile)
Name: Post-Liquefaction - Downstream
Method: Spencer
Last Modified: 6/18/2010 8:12:33 AM

(1) Embankment - Clay: Unit Weight = 125 pcf, phi = 14 °, c = 300 psf
(2) ML: Unit Weight = 120 pcf, phi = 0 °, c = 1000 psf
(3) SC: Unit Weight = 120 pcf, phi = 0 °, c = 1200 psf
(4) CL: Unit Weight = 120 pcf, phi = 0 °, c = 1600 psf
(5) SM-1: Unit Weight = 120 pcf, phi = 0 °, c = 1040 psf
(6) ML/CL-1: Unit Weight = 120 pcf, phi = 0 °, c = 800 psf
(7) SM-2: Unit Weight = 120 pcf, phi = 0 °, c = 750 psf
(8) ML/CL-2: Unit Weight = 120 pcf, phi = 0 °, c = 1400 psf
(9) Embankment - Filter Sand: Unit Weight = 125 pcf, phi = 35 °, c = 0 psf
(10) Embankment - Drain Gravel: Unit Weight = 125 pcf, phi = 35 °, c = 0 psf
(11) Embankment - Random Fill: Unit Weight = 125 pcf, phi = 14 °, c = 300 psf
(12) Basin Fill: Unit Weight = 125 pcf, phi = 14 °, c = 300 psf
(13) Bedrock: Unit Weight = 125 pcf, phi = 0 °, c = 2300 psf

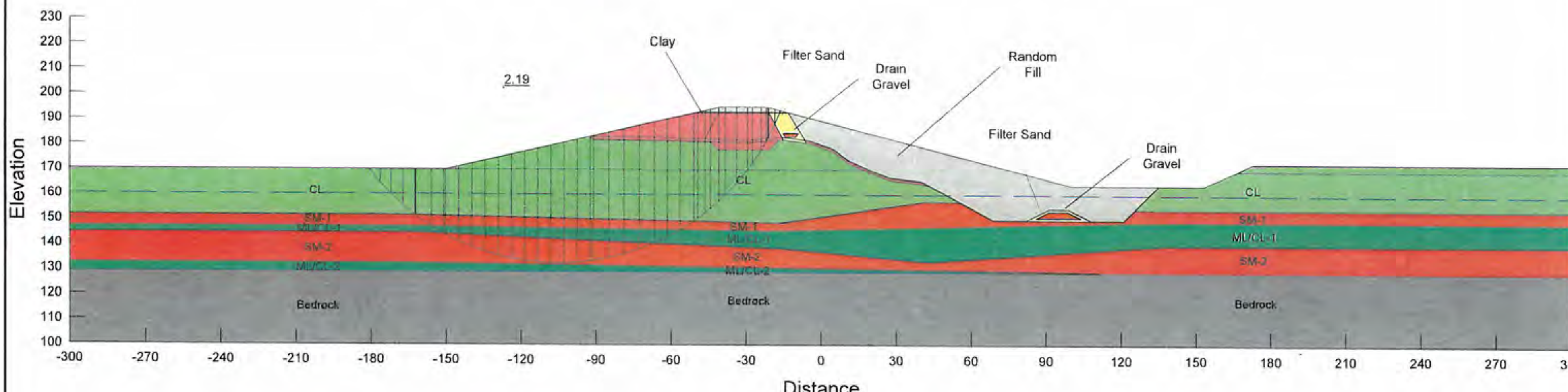


Notes:
1.Elevations in NAVD88
2.The Factor of Safety (FS) value above is for the critical failure surface

Upper Sand Creek Detention Basin Antioch, California	GEI Consultants	**Section D** Post-Seismic Slope Stability Downstream Slope (50th Percentile Seismic Event)	
Contra Costa County Public Works Department Martinez, California	Project 08318-0	June 2010	Fig. 6

Upper Sand Creek - Section B (84th Percentile)
Name: Post-Liquefaction - Upstream Slope
Method: Spencer
Last Modified: 6/17/2010 4:41:52 PM

(1) Embankment - Clay: Unit Weight = 125 pcf, phi = 14 °, c = 300 psf
(2) CL (Above El. 170): Unit Weight = 120 pcf, phi = 0 °, c = 1600 psf
(3) CL (Below El. 170): Unit Weight = 120 pcf, phi = 0 °, c = 1600 psf
(4) SM-1: Unit Weight = 120 pcf, phi = 0 °, c = 1040 psf
(5) ML/CL-1: Unit Weight = 120 pcf, phi = 0 °, c = 800 psf
(6) SM-2: Unit Weight = 120 pcf, phi = 0 °, c = 750 psf
(7) ML/CL-2: Unit Weight = 120 pcf, phi = 0 °, c = 1120 psf
(8) Embankment - Filer Sand: Unit Weight = 125 pcf, phi = 35 °, c = 0 psf
(9) Embankment - Drain Gravel: Unit Weight = 125 pcf, phi = 35 °, c = 0 psf
(10) Embankment - Random Fill: Unit Weight = 125 pcf, phi = 14 °, c = 300 psf
(11) Bedrock: Unit Weight = 125 pcf, phi = 0 °, c = 2300 psf

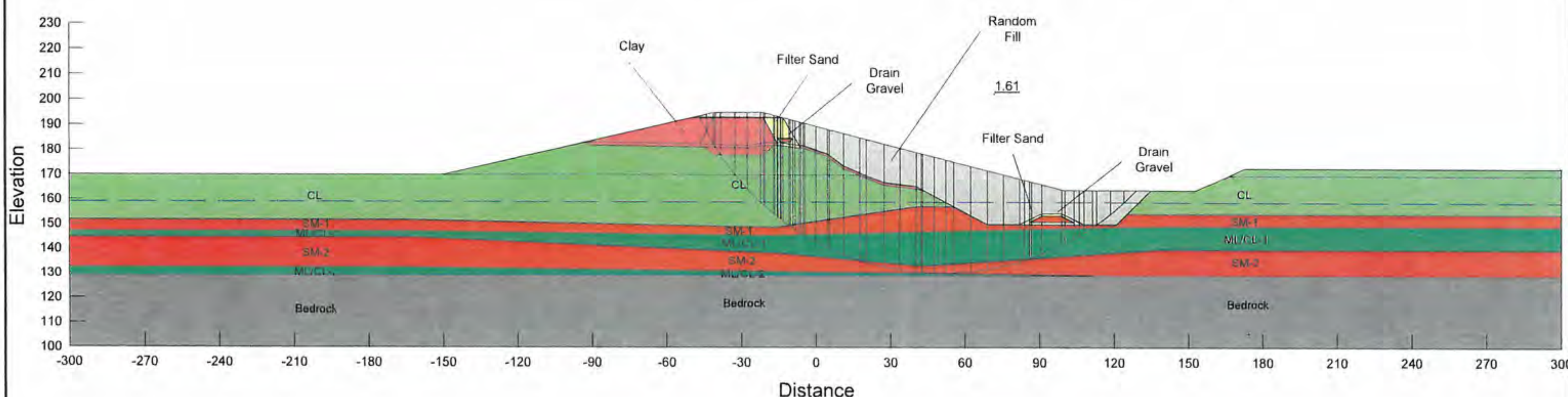


Notes:
1.Elevations in NAVD88
2.The Factor of Safety (FS) value above is for the critical failure surface

Upper Sand Creek Detention Basin Antioch, California	GEI Consultants	**Section B** Post-Seismic Slope Stability Upstream Slope (84th Percentile Seismic Event)
Contra Costa County Public Works Department Martinez, California	Project 08318-0	June 2010 Fig. 7

Upper Sand Creek - Section B (84th Percentile)
Name: Post-Liquefaction - Downstream Slope
Method: Spencer
Last Modified: 6/17/2010 4:41:52 PM

(1) Embankment - Clay: Unit Weight = 125 pcf, phi = 14 °, c = 300 psf
(2) CL (Above El. 170): Unit Weight = 120 pcf, phi = 0 °, c = 1600 psf
(3) CL (Below El. 170): Unit Weight = 120 pcf, phi = 0 °, c = 1600 psf
(4) SM-1: Unit Weight = 120 pcf, phi = 0 °, c = 1040 psf
(5) ML/CL-1: Unit Weight = 120 pcf, phi = 0 °, c = 800 psf
(6) SM-2: Unit Weight = 120 pcf, phi = 0 °, c = 750 psf
(7) ML/CL-2: Unit Weight = 120 pcf, phi = 0 °, c = 1120 psf
(8) Embankment - Filer Sand: Unit Weight = 125 pcf, phi = 35 °, c = 0 psf
(9) Embankment - Drain Gravel: Unit Weight = 125 pcf, phi = 35 °, c = 0 psf
(10) Embankment - Random Fill: Unit Weight = 125 pcf, phi = 14 °, c = 300 psf
(11) Bedrock: Unit Weight = 125 pcf, phi = 0 °, c = 2300 psf

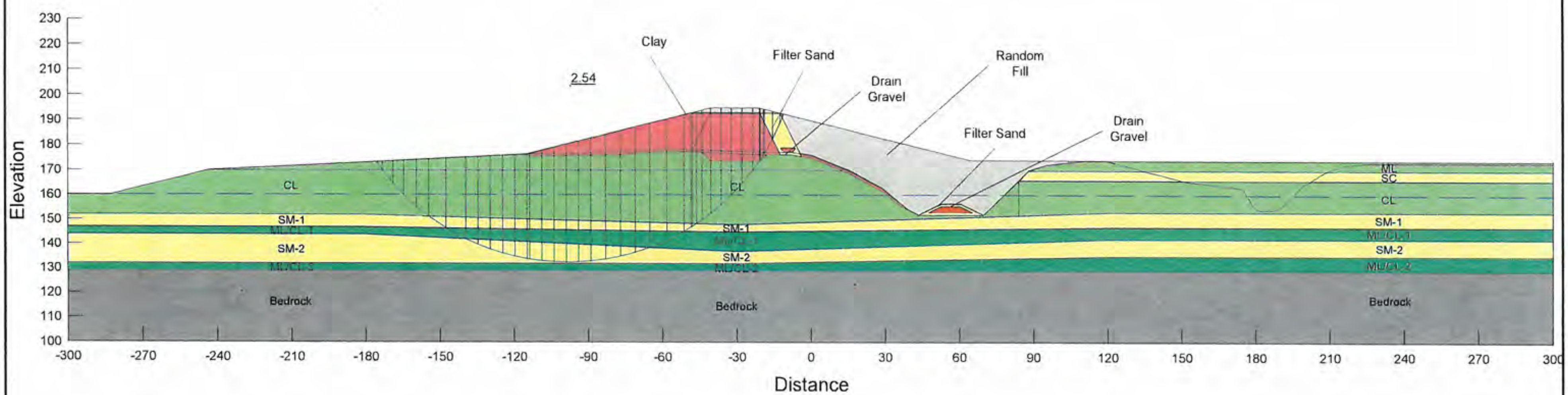


Notes:
1.Elevations in NAVD88
2.The Factor of Safety (FS) value above is for the critical failure surface

Upper Sand Creek Detention Basin Antioch, California	GEI Consultants	**Section B** Post-Seismic Slope Stability Downstream Slope (84th Percentile Seismic Event)
Contra Costa County Public Works Department Martinez, California	Project 08318-0	June 2010 Fig. 8

Upper Sand Creek - Section C (84th Percentile)
Name: Post-Liquefaction - Upstream
Method: Spencer
Last Modified: 6/17/2010 4:44:50 PM

(1) Embankment - Clay: Unit Weight = 125 pcf, phi = 14 °, c = 300 psf
(2) ML: Unit Weight = 120 pcf, phi = 0 °, c = 1000 psf
(3) SC: Unit Weight = 120 pcf, phi = 0 °, c = 1200 psf
(4) CL (Above El. 170): Unit Weight = 120 pcf, phi = 0 °, c = 1600 psf
(5) CL (Below El. 170): Unit Weight = 120 pcf, phi = 0 °, c = 1600 psf
(6) SM-1: Unit Weight = 120 pcf, phi = 0 °, c = 1040 psf
(7) ML/CL-1: Unit Weight = 120 pcf, phi = 0 °, c = 800 psf
(8) SM-2: Unit Weight = 120 pcf, phi = 0 °, c = 750 psf
(9) ML/CL-2: Unit Weight = 120 pcf, phi = 0 °, c = 1120 psf
(10) Embankment - Filter Sand: Unit Weight = 125 pcf, phi = 35 °, c = 0 psf
(11) Embankment - Drain Gravel: Unit Weight = 125 pcf, phi = 35 °, c = 0 psf
(12) Embankment - Random Fill: Unit Weight = 125 pcf, phi = 14 °, c = 300 psf
(13) Bedrock: Unit Weight = 125 pcf, phi = 0 °, c = 2300 psf

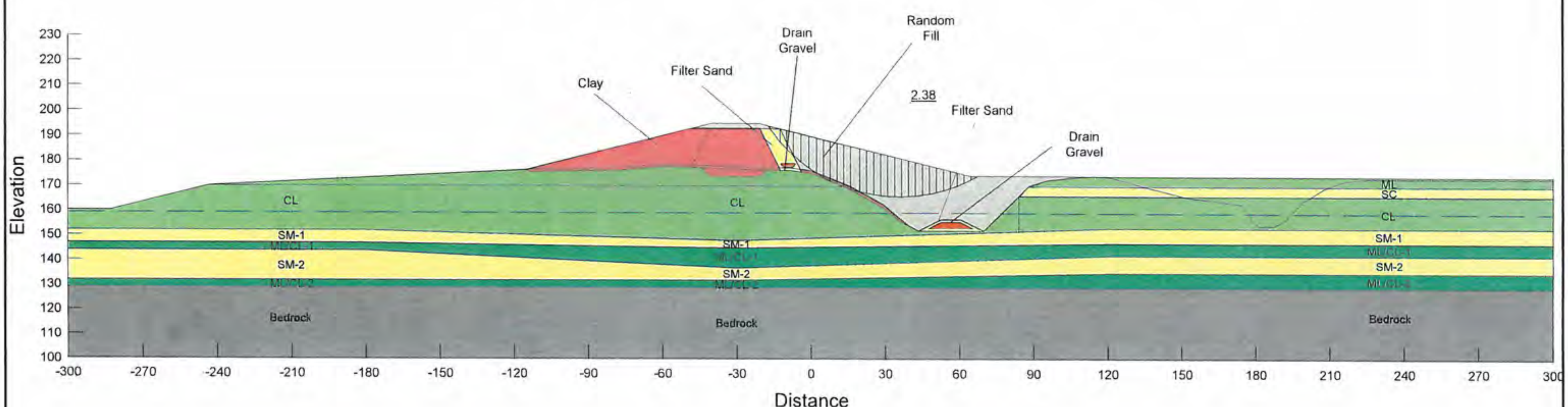


Notes:
1.Elevations in NAVD88
2.The Factor of Safety (FS) value above is for the critical failure surface

Upper Sand Creek Detention Basin Antioch, California	GEI Consultants	**Section C** Post-Seismic Slope Stability Upstream Slope (84^{th} Percentile Seismic Event)
Contra Costa County Public Works Department Martinez, California	Project 08318-0	June 2010 Fig. 9

Upper Sand Creek - Section C (84th Percentile)
Name: Post-Liquefaction - Downstream
Method: Spencer
Last Modified: 6/17/2010 4:44:50 PM

(1) Embankment - Clay: Unit Weight = 125 pcf, phi = 14 °, c = 300 psf
(2) ML: Unit Weight = 120 pcf, phi = 0 °, c = 1000 psf
(3) SC: Unit Weight = 120 pcf, phi = 0 °, c = 1200 psf
(4) CL (Above El. 170): Unit Weight = 120 pcf, phi = 0 °, c = 1600 psf
(5) CL (Below El. 170): Unit Weight = 120 pcf, phi = 0 °, c = 1600 psf
(6) SM-1: Unit Weight = 120 pcf, phi = 0 °, c = 1040 psf
(7) ML/CL-1: Unit Weight = 120 pcf, phi = 0 °, c = 800 psf
(8) SM-2: Unit Weight = 120 pcf, phi = 0 °, c = 750 psf
(9) ML/CL-2: Unit Weight = 120 pcf, phi = 0 °, c = 1120 psf
(10) Embankment - Filter Sand: Unit Weight = 125 pcf, phi = 35 °, c = 0 psf
(11) Embankment - Drain Gravel: Unit Weight = 125 pcf, phi = 35 °, c = 0 psf
(12) Embankment - Random Fill: Unit Weight = 125 pcf, phi = 14 °, c = 300 psf
(13) Bedrock: Unit Weight = 125 pcf, phi = 0 °, c = 2300 psf

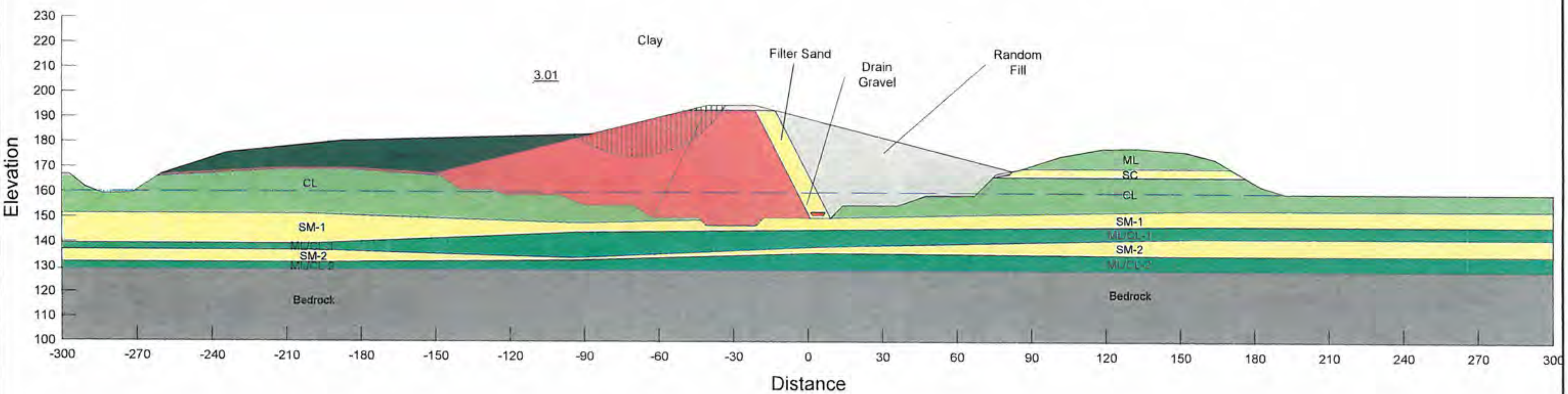


Notes:
1.Elevations in NAVD88
2.The Factor of Safety (FS) value above is for the critical failure surface

Upper Sand Creek Detention Basin Antioch, California	GEI Consultants	**Section C** Post-Seismic Slope Stability Downstream Slope (84th Percentile Seismic Event)
Contra Costa County Public Works Department Martinez, California	Project 08318-0	June 2010 Fig. 10

Upper Sand Creek - Section D (84th Percentile)
Name: Post-Liquefaction - Upstream
Method: Spencer
Last Modified: 6/17/2010 4:49:01 PM

(1) Embankment - Clay: Unit Weight = 125 pcf, phi = 14 °, c = 300 psf
(2) ML: Unit Weight = 120 pcf, phi = 0 °, c = 1000 psf
(3) SC: Unit Weight = 120 pcf, phi = 0 °, c = 1200 psf
(4) CL: Unit Weight = 120 pcf, phi = 0 °, c = 1600 psf
(5) SM-1: Unit Weight = 120 pcf, phi = 0 °, c = 1040 psf
(6) ML/CL-1: Unit Weight = 120 pcf, phi = 0 °, c = 800 psf
(7) SM-2: Unit Weight = 120 pcf, phi = 0 °, c = 750 psf
(8) ML/CL-2: Unit Weight = 120 pcf, phi = 0 °, c = 1120 psf
(9) Embankment - Filter Sand: Unit Weight = 125 pcf, phi = 35 °, c = 0 psf
(10) Embankment - Drain Gravel: Unit Weight = 125 pcf, phi = 35 °, c = 0 psf
(11) Embankment - Random Fill: Unit Weight = 125 pcf, phi = 14 °, c = 300 psf
(12) Basin Fill: Unit Weight = 125 pcf, phi = 14 °, c = 300 psf
(13) Bedrock: Unit Weight = 125 pcf, phi = 0 °, c = 2300 psf

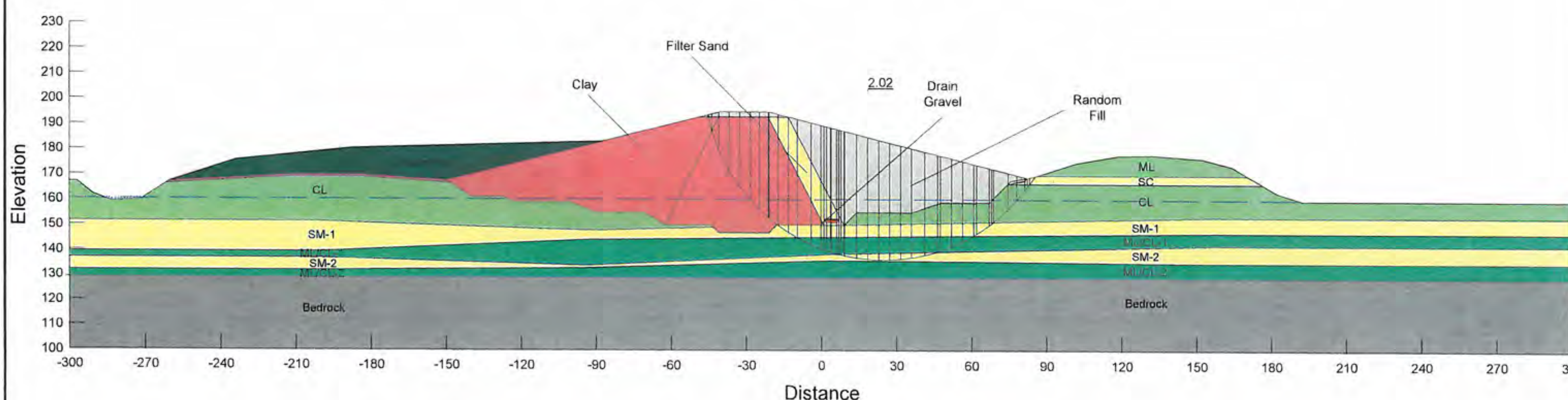


Notes:
1.Elevations in NAVD88
2.The Factor of Safety (FS) value above is for the critical failure surface

Upper Sand Creek Detention Basin Antioch, California	GEI Consultants	**Section D** Post-Seismic Slope Stability Upstream Slope (84th Percentile Seismic Event)
Contra Costa County Public Works Department Martinez, California	Project 08318-0	June 2010 — Fig. 11

Upper Sand Creek - Section D (84th Percentile)
Name: Post-Liquefaction - Downstream
Method: Spencer
Last Modified: 6/17/2010 4:49:01 PM

(1) Embankment - Clay: Unit Weight = 125 pcf, phi = 14 °, c = 300 psf
(2) ML: Unit Weight = 120 pcf, phi = 0 °, c = 1000 psf
(3) SC: Unit Weight = 120 pcf, phi = 0 °, c = 1200 psf
(4) CL: Unit Weight = 120 pcf, phi = 0 °, c = 1600 psf
(5) SM-1: Unit Weight = 120 pcf, phi = 0 °, c = 1040 psf
(6) ML/CL-1: Unit Weight = 120 pcf, phi = 0 °, c = 800 psf
(7) SM-2: Unit Weight = 120 pcf, phi = 0 °, c = 750 psf
(8) ML/CL-2: Unit Weight = 120 pcf, phi = 0 °, c = 1120 psf
(9) Embankment - Filter Sand: Unit Weight = 125 pcf, phi = 35 °, c = 0 psf
(10) Embankment - Drain Gravel: Unit Weight = 125 pcf, phi = 35 °, c = 0 psf
(11) Embankment - Random Fill: Unit Weight = 125 pcf, phi = 14 °, c = 300 psf
(12) Basin Fill: Unit Weight = 125 pcf, phi = 14 °, c = 300 psf
(13) Bedrock: Unit Weight = 125 pcf, phi = 0 °, c = 2300 psf



Notes:
1. Elevations in NAVD88
2. The Factor of Safety (FS) value above is for the critical failure surface

Upper Sand Creek Detention Basin Antioch, California	GEI Consultants	**Section D** Post-Seismic Slope Stability Downstream Slope (84th Percentile Seismic Event)
Contra Costa County Public Works Department Martinez, California	Project 08318-0	June 2010 Fig. 12

Appendix 2.6

Dam Settlement

Upper Sand Creek Detention Basin - Settlement Evaluation

Objective:

Estimate the potential settlement of the Upper Sand Creek Detention Basin Dam. Perform settlement calculations in general accordance with USACE Engineer Manual EM 1110-1-1904, Settlement Analysis.

References:

[1] U.S. Army Corps of Engineers, (1990). Settlement Analysis, Engineer Manual EM 1110-2-1904, September 30.
[2] Prototype Engineering, Inc. (2002). WinSAF-I Version 1.0.
[3] Mesri (1977), Time- and Stress-Compressibility Interrelationship, from Journal of the Geotechnical Engineering Division, ASCE, Vol. 103, No. GT5, pp. 417-430.
[4] Mesri (2001), Primary compression and Secondary Compression, from Proc. Soil Behavior and Soft Ground Construction, Geotech. Spec. Publ. No. 119, ASCE, pp. 122-166.
[5] Cal Engineering & Geology, Inc. (2010). Geotechnical Data Report, Contra Costa County Flood Control & Water Conservation District, Upper Sand Creek Detention Basin, Deer Creek Road, Antioch, California, May 13.
[6] Lambe, T.W., and Whitman, R.V. (1969). Soil Mechanics. John Wiley and Sons, New York.
[7] GEI Consultants, Inc. (2009). Upper Sand Creek Detention Basin, Basis of Design Memorandum, May.
[8] GEI Consultants, Inc. (2010). Upper Sand Creek Detention Basin, 35% Design Review Drawings, May.

Approach:

1. Develop generalized subsurface profile at representative cross-sections of the embankment and outlet works.

2. Assign soil parameters and maximum past pressures.

3. Use WinSAF-I (Prototype Engineering, 2002) to estimate the primary consolidation settlement of the foundation soils due to the loading from the proposed embankment.

4. We did not estimate secondary compression settlements because published literature (Mesri, 1977; Mesri, 2001) indicates that the magnitude of secondary compression is directly related to the magnitude and rate of primary settlement. The compressible soils at the site are overconsolidated. After the loading, most of the soil will remain overconsolidated. Based on Mesri's findings, the rate of secondary compression settlement in the overconsolidated region will likely be a fraction of those in the normally consolidated region. It is our opinion that the secondary compression at this site would be small due to the loadings proposed.

Conclusion:

The estimated settlements along the centerline of the dam are as follows:

	Dam Station		
	6+00	12+11	15+29
Estimated Settlement (inch)	2.4	4.5	5.9

A fraction of this estimated settlement will occur during construction, while the rest will develop after construction of the dam has ended. From the point of view of maintaining freeboard, a conservative approach entails assuming that the dam will be constructed very quickly and thus the majority of the settlement will occur after construction. Thus, the dam design should consider providing camber equal to the total estimated settlement to ensure that the required freeboard is maintained over the long term.

The estimated settlements along the outlet works are as follows:

	Outlet Works Station				
	0+90	1+85	2+20	2+65	2+98
Estimated Settlement (inch)	0.19	0.77	0.98	1.92	0.20

Model Development

Selection of Representative Cross Sections

We performed settlement analyses at three generalized cross sections of the dam embankment (Sta. 6+00, 12+11, and 15+29) and at one section along the outlet works. The locations of these sections are shown on Fig. 1.

We developed model geometry based on existing topography and the 35% Design Drawings. The model geometries are shown on Figs. 2, 3, 4 and 5. We developed subsurface profiles based on recent and historic borings.

Selection of Soil Properties

Unit Weight

We estimated unit weights for our settlement calculations based on the laboratory data testing data presented in CE&G (2010).

Two moisture density relationship tests (ASTM D698) were conducted on potential borrow materials. The results of the tests are summarized below:

Boring Number	USCS	Moisture Density Relationship		
		Optimum MC (%)	Max. Dry Density (pcf)	Total Density (pcf)
TP-9, -10, -11 Blend	CL	17.6	103.1	121.2
TP-20, -12, -26 Blend	CL	17.8	103.0	121.3

For our calculations, we used a total unit weight of 125 pcf for the embankment fill, which is slightly higher than the average of these two tests, but reasonably conservative.

GEI Consultants

Client: CCCFCWCD
Project: Upper Sand Creek Detention Basin
Project No.: 09318-0

Prepared By: JAD / DRD
Date: 7/12/10
Checked By: J. Nickerson
Date: 7/12/2010

Settlement Evaluation

Unit weights of foundation materials were measured in the lab using selected samples taken in the recent borings. The results of the lab measurements are summarized below:

Boring Number	Sample Number	Depth Interval (feet)	USCS	Moisture Density Relationship MC (%)	Moisture Density Relationship Dry Density (pcf)	Moisture Density Relationship Total Density (pcf)
B-2-09	2-4	8.2 - 10.7	CL	12.9	94.2	106.4
B-2-09	2-7	20.5 - 23.0	CL	16.9	113.3	132.4
B-3-09	3-3	6.5 - 9.0	CL	14.1	116.3	132.7
B-3-09	3-9	20.5 - 23.0	CL	23.3	99.9	123.2
B-4-09	4-5	10.0 - 12.5	ML	15.6	105.6	122.1
B-7-09	7-7	15.5	CL	22.8	94.1	115.6
B-8-09	8-5	10.0 - 12.5	CL	25.5	101.7	117.3
B-10-09	10-9	19.0 - 21.5	CL	23.8	100.9	124.9
B-1	--	20.0	CL	22.2	99.8	122.0
B-2	--	20.5	CL/ML	17.8	104.3	122.9
B-2	--	40.5	CL	33.1	89.7	119.4
B-2	--	60.5	CL	27.8	96.1	122.8
B-3	--	15.5	CL	19	104.7	124.6
B-4	--	21.0	CL	23.6	103.1	127.4
B-4	--	26.0	SM	21.6	108.8	132.3
B-6	--	21.0	CL	22.4	104.3	127.7
C-1	3	24.8	SM	16	111.2	129.0
C-1	4	42.1	SM	15.3	110.9	127.9
C-1	6	56.1	SM	17.7	111.9	131.7
C-4	3	21.0	ML	18.7	111.5	132.4
C-6	3	36.0	ML	16.8	113.1	132.1
C-6	5	44.7	ML	19	109.8	130.7
C-7	3	16.1	SM	23.6	98.7	122.0

The lab data show total unit weights ranging from about 106 pcf to 132 pcf and averaging 125 pcf. For our calculations, we used a total unit weight of 125 pcf for the foundation materials.

Settlement Evaluation

Compression Ratio and Recompression Ratio

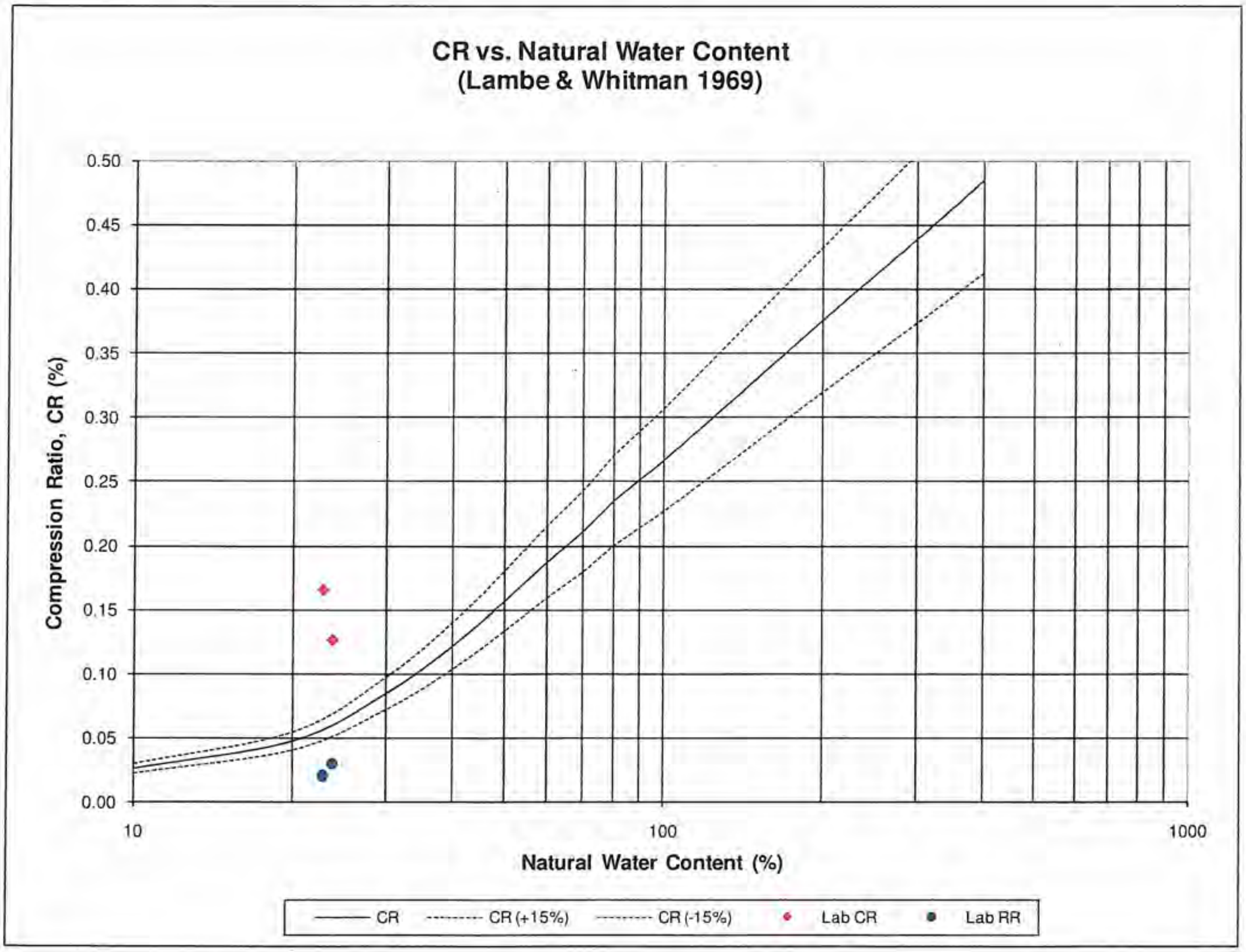
We estimated compression ratio (CR) and recompression ratio (RR) values based on

- two laboratory consolidation tests on the clay stratum
- an empirical correlation to moisture content published in Lambe & Whitman (1969)

A summary of the two consolidation tests is shown below:

Boring	Depth (ft)	USCS	M.C. %	CR	RR	CR/RR
B-7-09	15.5	CL	22.8	0.17	0.02	7.9
B-10-09	19.0	CL	23.8	0.13	0.03	4.2

These data points plotted on the Lambe and Whitman (1969) correlation, below, show that the values are slightly higher than estimated with the correlation.



Settlement Evaluation

We applied the Lambe and Whitman correlation to samples from the recent lab testing program where we performed moisture content tests:

Boring	Surface Elevation	Depth (ft)	Sample Elevation (ft)	USCS	Water Content (%)	Correlation CR (%)	Correlation RR (%)
B-1-09	187	22.3	164.8	CL	24.6	0.073	0.010
B-2-09	176	9.5	166.6	CL	12.9	0.027	0.004
B-2-09	176	21.8	154.3	CL	16.9	0.043	0.006
B-3-09	178	7.8	170.3	CL	14.1	0.032	0.004
B-3-09	178	21.8	156.3	CL	23.3	0.068	0.009
B-4-09	181	11.3	169.8	ML	15.6	0.038	0.005
B-7-09	182	15.5	166.5	CL	22.8	0.066	0.009
B-8-09	166	2.0	164.0	CL	7.8	0.006	0.001
B-8-09	166	11.3	154.8	CL	25.5	0.076	0.010
B-8-09	166	29.0	137.0	CL	3.8	-0.012	-0.002
B-10-09	165	20.3	144.8	CL	23.8	0.070	0.009
B-1	193	20.0	173.0	CL	22.2	0.064	0.009
B-2	174	5.0	169.0	CH/CL	15	0.036	0.005
B-2	174	15.5	158.5	CL	21	0.059	0.008
B-2	174	20.5	153.5	CL/ML	17.8	0.047	0.006
B-2	174	40.5	133.5	CL	33.1	0.103	0.014
B-2	174	60.5	113.5	CL	27.8	0.085	0.011
B-3	191	15.5	175.5	CL	19	0.052	0.007
B-4	193	3.0	190.0	CL	10	0.015	0.002
B-4	193	15.5	177.5	CL	19	0.052	0.007
B-4	193	21.0	172.0	CL	23.6	0.069	0.009
B-4	193	25.0	168.0	CL	20	0.056	0.007
B-4	193	26.0	167.0	SM	21.6	0.062	0.008
B-6	183	21.0	162.0	CL	22.4	0.065	0.009
B-8	166	21.0	145.0	ML	22	0.063	0.008
B-8	166	36.0	130.0	CL	20	0.056	0.007
C-4	195	21.0	174.0	ML	18.7	0.051	0.007
C-6	213	36.0	177.0	ML	16.8	0.043	0.006
C-6	213	44.7	168.3	ML	19	0.052	0.007
C-7	183	16.1	166.9	SM	23.6	0.069	0.009

We used CR = 0.16 and RR = 0.03 for the clay layers.

In our analysis along the centerline of the dam, we excluded the silt and silty sand layers since these layers will settle relatively quickly during construction. However we did include these materials in our evaluation of settlement under the outlet works.

We used CR = 0.06 and RR = 0.01 for the silty layers.

GEI Consultants	**Client:** CCCFCWCD **Project:** Upper Sand Creek Detention Basin **Project No.:** 09318-0	**Prepared By:** JAD / DRD **Date:** 7/12/10 **Checked By:** J. Nickerson **Date:** 7/12/2010	

Settlement Evaluation

Stress History

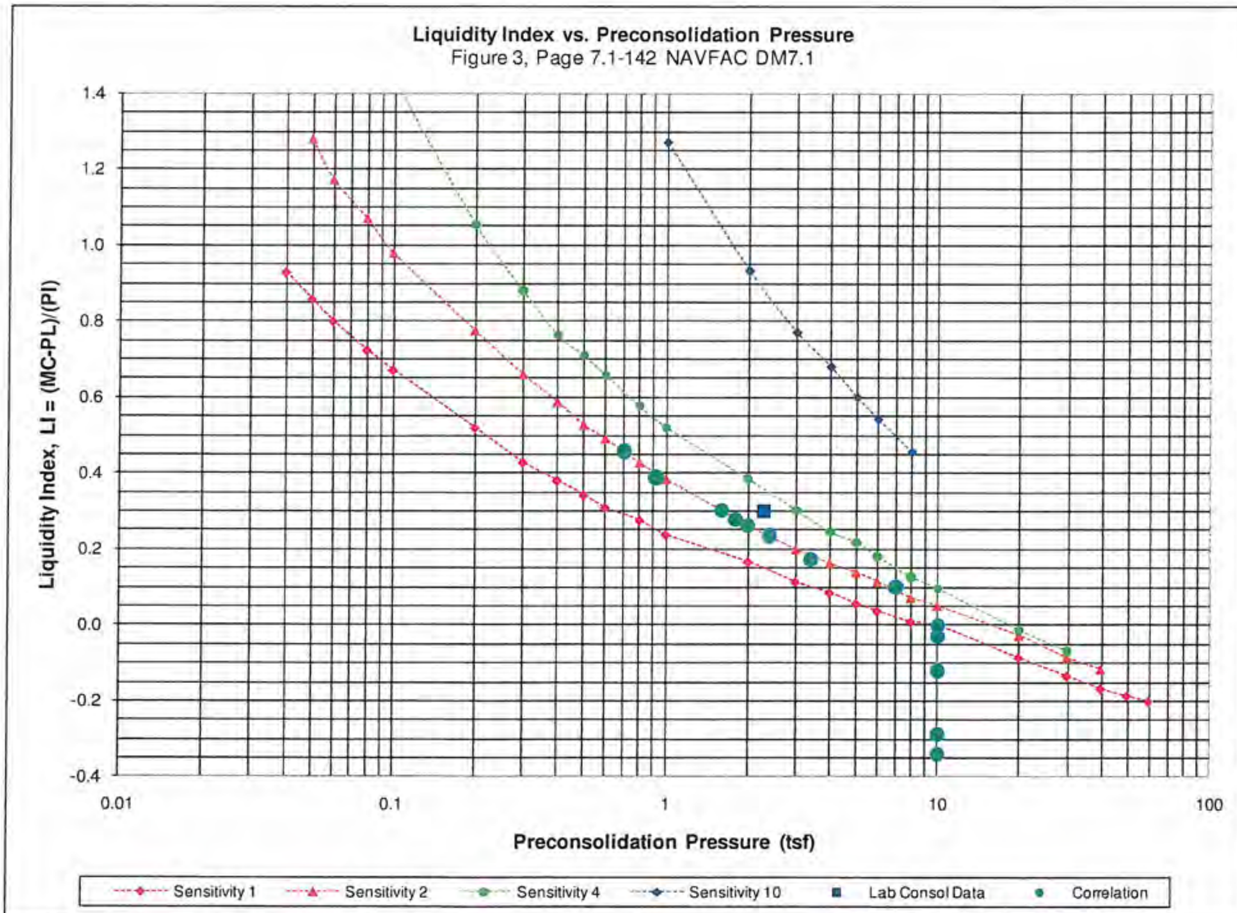
Results from the consolidation tests indicate that the clay stratum is overconsolidated. At sample depths of 15.5 and 19 feet, we estimated maximum past pressures of approximately 4,500 to 4,600 psf, resulting in overconsolidation ratios (OCRs) of 2.3 to 2.5.

We also estimated maximum past pressures based on an empirical correlation to liquidity index published in NAVFAC.

From Recent Laboratory Consolidation Tests:

Boring	Depth (ft)	USCS	M.C. %	LL	PL	PI	LI	Po' (tsf)	Pp' (tsf)	OCR
B-7-09	15.5'	CL	22.8	--	--	--	--	0.94	2.25	2.3
B-10-09	19.0'	CL	23.8	39	17	22	0.30	0.91	2.30	2.5

NAVFAC Correlation:



Settlement Evaluation

Summary of liquidity index vs. Pp' from NAVFAC (assuming a Sensitivity of 2):

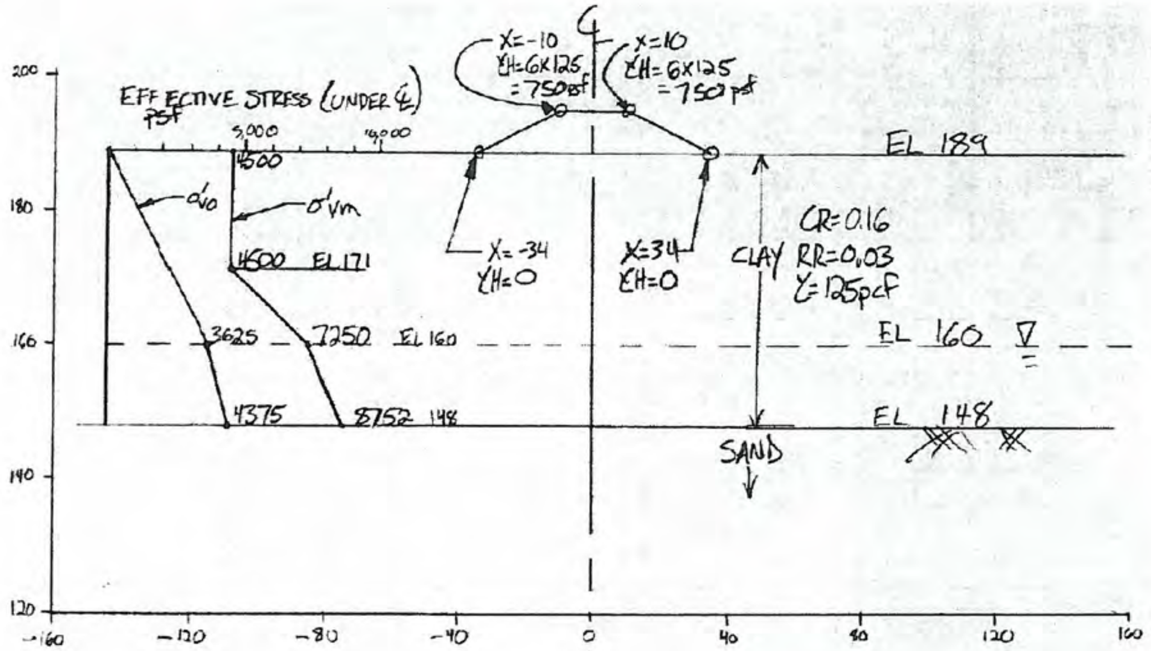
Boring	Depth (ft)	USCS	M.C. %	LL	PL	PI	LI	Po' (tsf)	Pp' (tsf)	OCR
B-1-09	22.25	CL	24.6	37	22	15	0.17	1.39	3.4	2.4
B-2-09	9.45	CL	12.9	37	19	18	-0.34	0.59	10.0	16.9
B-3-09	21.25	CL	14.1	45	21	24	-0.29	1.33	10.0	7.5
B-4-09	5.75	CL	23.3	34	20	14	0.24	0.36	2.4	6.7
B-8-09	2.00	CL	7.8	32	20	12	-1.02	0.13	10.0	80.0
B-8-09	11.25	CL	25.5	32	20	12	0.46	0.70	0.7	1.0
B-8-09	29.00	CL	3.8	45	25	20	-1.06	1.81	10.00	5.5
B-10-09	20.25	CL	23.8	39.1	17	21.9	0.30	1.27	1.60	1.3
B-2	5	CH/CL	15	50	16	34	-0.03	0.31	10.00	32.0
B-2	16	CL	21	38	15	23	0.26	0.97	2.00	2.1
B-4	3	CL	10	38	13	25	-0.12	0.19	10.00	53.3
B-4	16	CL	19	46	16	30	0.10	0.97	7.00	7.2
B-4	25	CL	20	33	15	18	0.28	1.56	1.80	1.2
B-8	21	ML	22	27	22	5	0.00	1.31	10.00	7.6
B-8	36	CL	20	31	13	18	0.39	2.25	0.92	0.4

For analysis, we assumed that 4,500 psf would be the lower bound of maximum past pressure, resulting in higher OCRs near the ground surface. Such a stress history is reasonable, as the groundwater table is 10 to 20 feet below the ground surface in each of the cross sections. We assumed that at depth, the clay would still be at least slightly overconsolidated. We assumed that the minimum OCR the clay would have is 2.0.

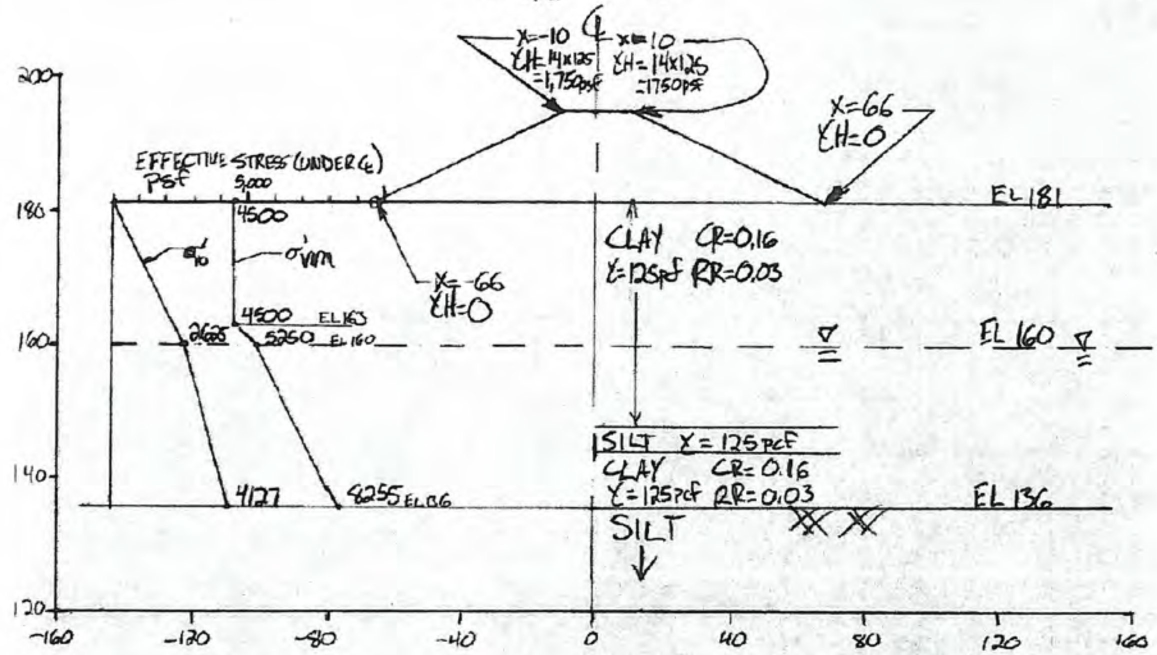
Therefore, at depths where the effective overburden stress is lower than 2,250 psf, the maximum past pressure was taken as 4500 psf, and was assumed to increase corresponding to an OCR of 2.0 at greater depths. Stress histories for each cross section are shown below.

Settlement Evaluation

Typical Sections & Stress History of Sections along Dam Centerline

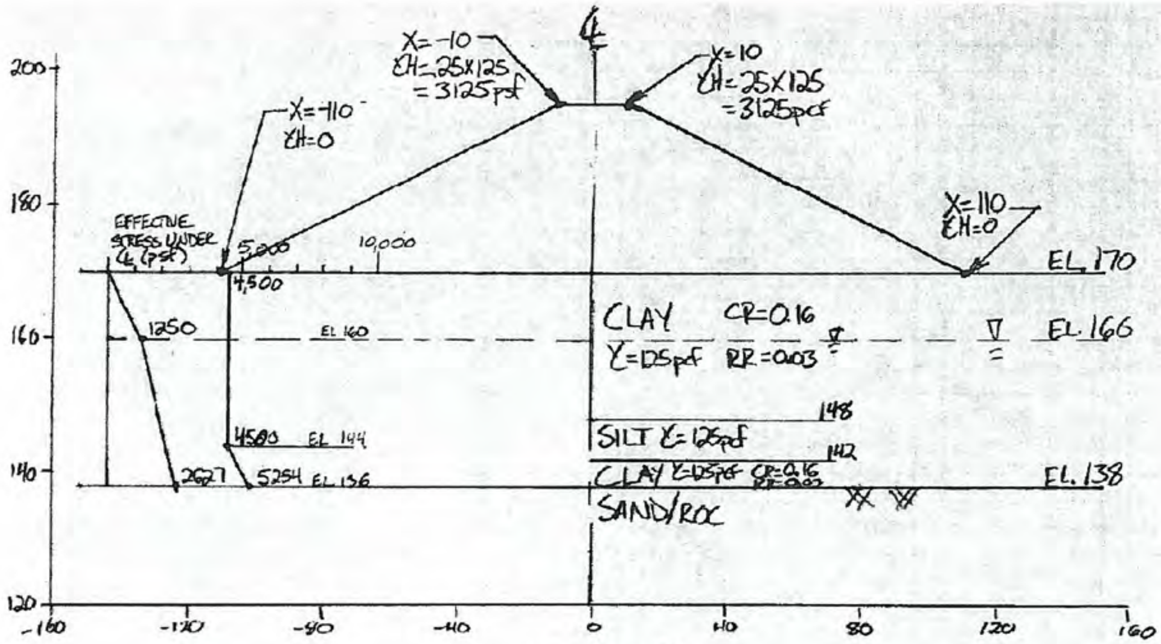


SECTION 1
STA. 6+00



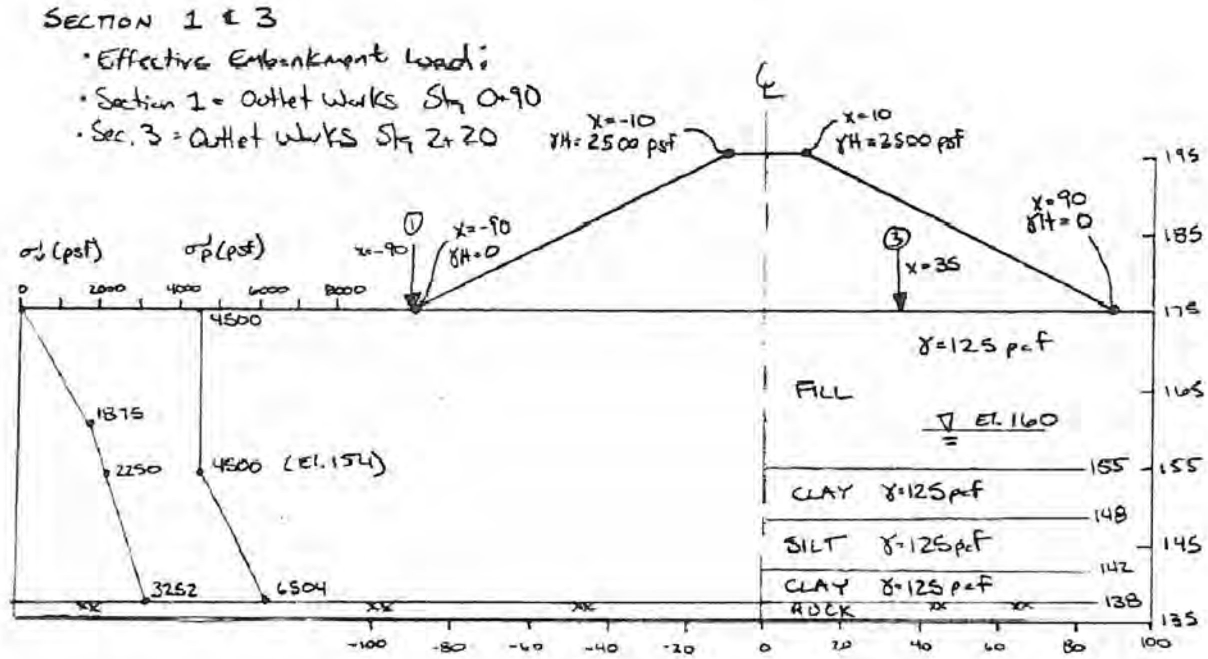
SECTION 2
STA. 12+11

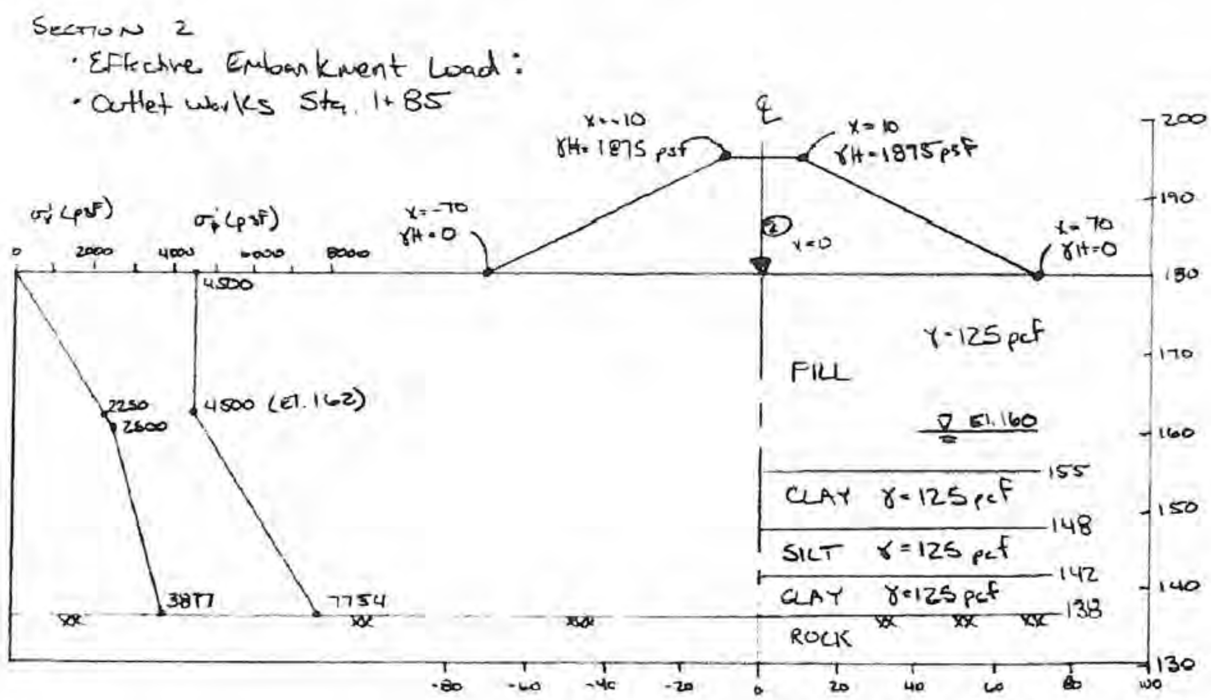
Client:	CCCFCWCD	**Prepared By:**	JAD / DRD
Project:	Upper Sand Creek Detention Basin	**Date:**	7/12/10
Project No.:	09318-0	**Checked By:**	J. Nickerson
	Settlement Evaluation	**Date:**	7/12/2010



SECTION 3
STA. 15+29

Typical Sections & Stress History of Sections along Outlet Works

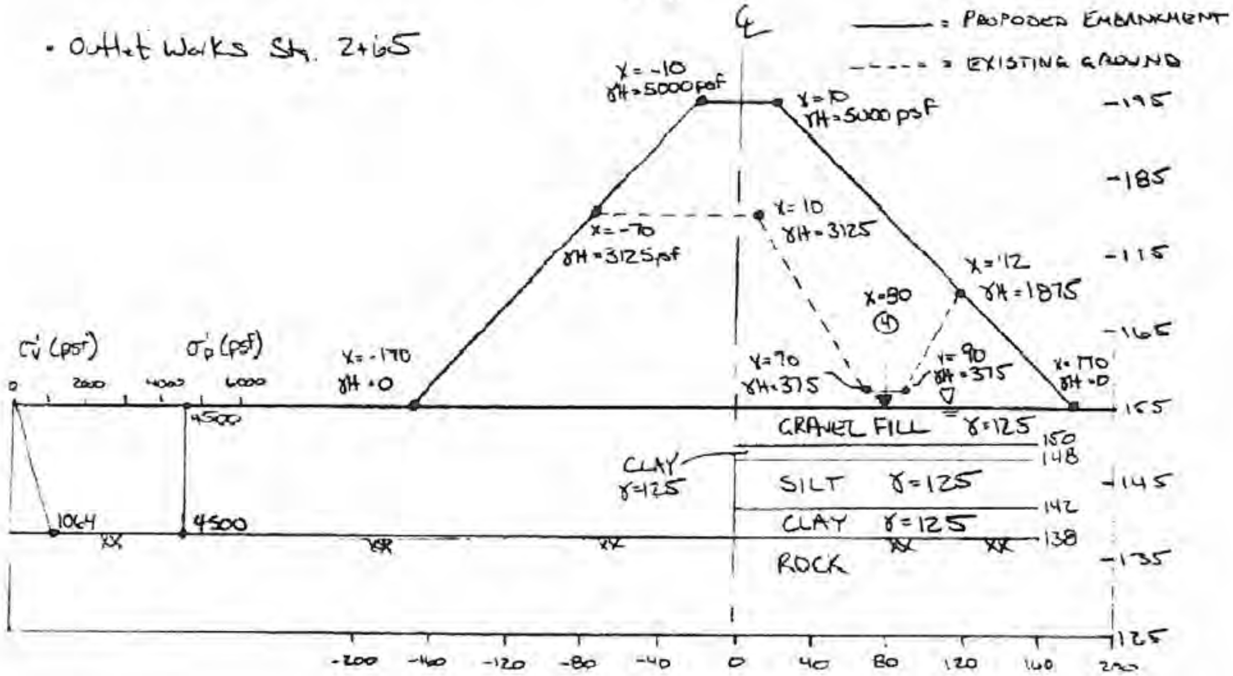




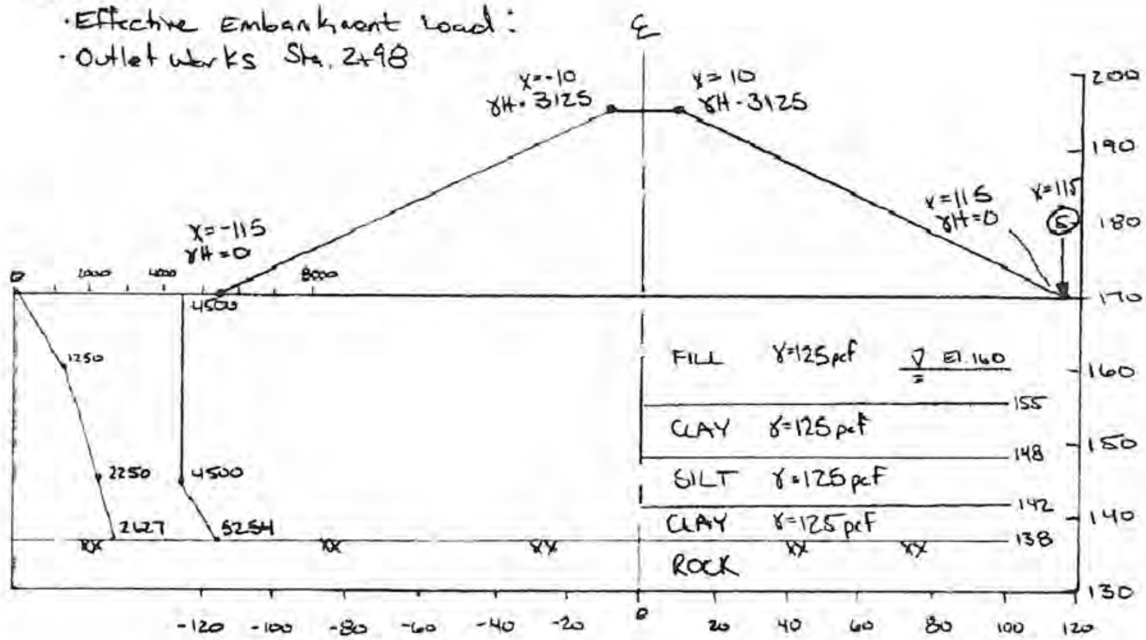
Settlement Evaluation

SECTION 4
·Effective Embankment Load:

· Outlet Works Sta. 2+65



SECTION 5
·Effective Embankment Load:
·Outlet Works Sta. 2+98

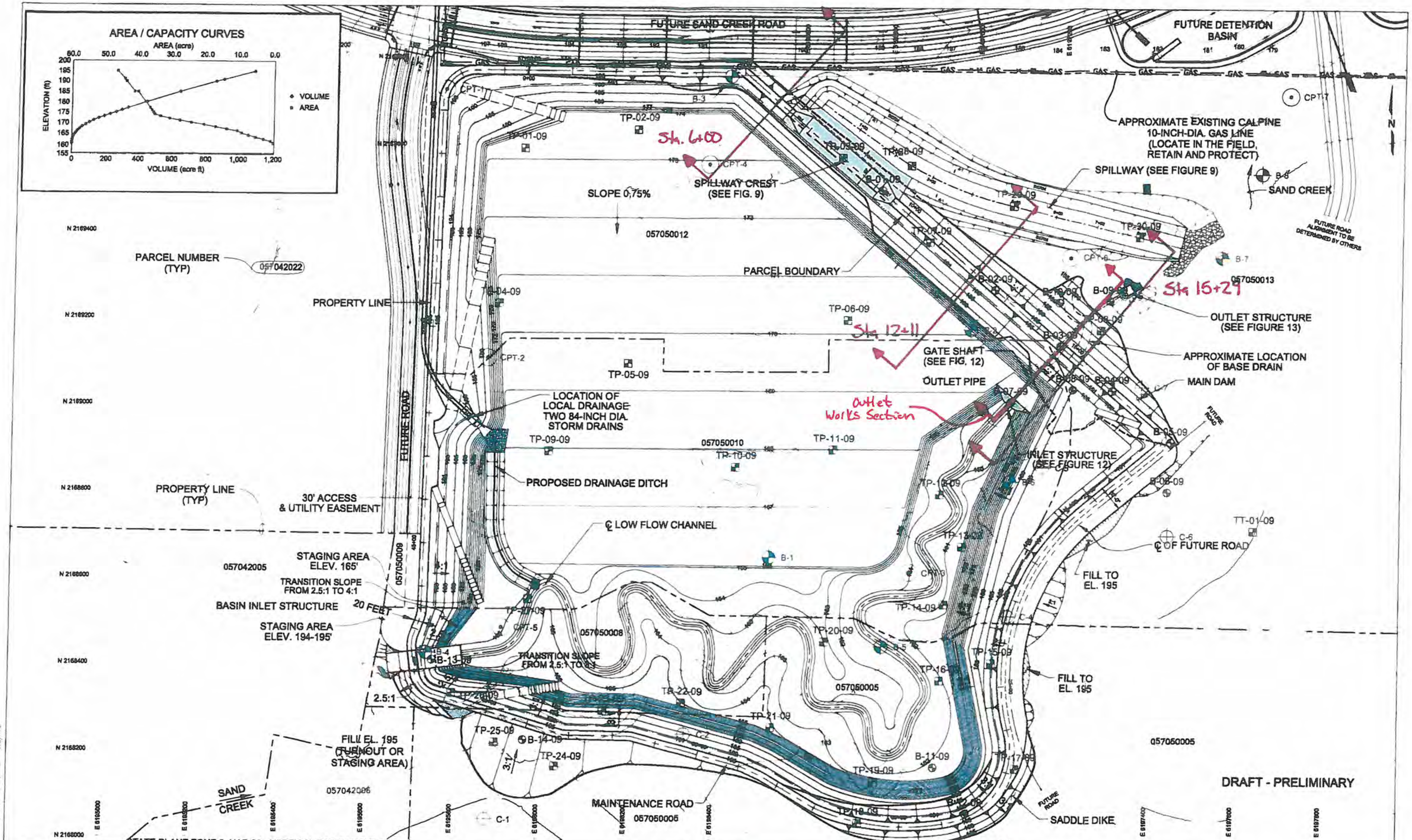


Modeling in WinSAF-I:

We used WinSAF-I Version 1.0 (Prototype Engineering, Inc., 2002) to evaluate stress distributions and primary consolidation settlements due to fill loading.

We modeled embankment fill loads by using the "variable shape strip loading" option to model the loading due to the embankment.

Due to the complex geometry at outlet works Sta. 2+65, we estimated settlements using superposition. We first estimated increase in stress versus depth for the proposed embankment loading assuming an estimated horizontal ground surface at El. 155 (Run 1). Then we estimated the increase in stress versus depth caused by the existing contours assuming a horizontal ground surface at El. 155 (Run 2). We then subtracted the stress increase with depth of Run 2 from Run 1 to estimate the net increase in stress versus depth for the proposed embankment.



REVISIONS			
NO.	DESCRIPTION	BY	DATE

DES.: K.V.S.
DRAWN: P.Y.M.
CHKD.: A.T.
DATE: 02-16-10
SCALE: 1" = 100'
FLD. BK. XXXX

PROJECT ENGINEER
PLANS APPROVAL DATE

GEI Consultants

CONTRA COSTA COUNTY
PUBLIC WORKS DEPARTMENT
255 GLACIER DRIVE
MARTINEZ, CALIFORNIA 94553

0 100 200
SCALE, FEET

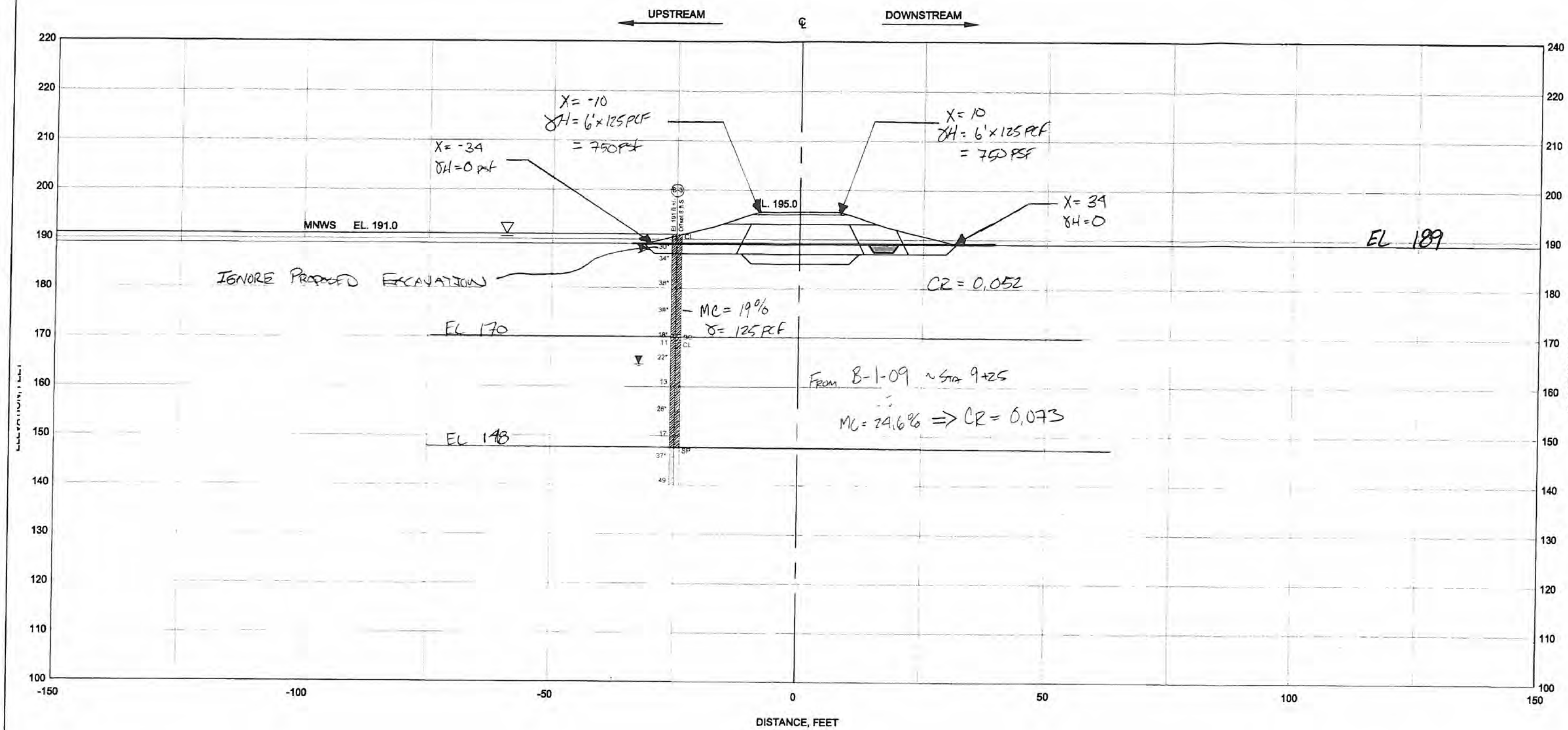
FOR REDUCED PLANS ORIGINAL SCALE IS IN INCHES
0 1 2 3

BASE MAP	EAST COORD.	NORTH COORD.
XXX	XXXX	XXX

UPPER SAND CREEK DETENTION BASIN
SITE PLAN

FILE NO.

Figure 1



MATERIAL TYPE

1. CLAY
2. FILTER SAND
3. DRAIN GRAVEL
4. RANDOM FILL
5. AGGREGATE BASE

LEGEND

- PROPOSED FINAL GRADE (EMBANKMENT AND EXCAVATION)
- EXISTING GROUND THAT REMAINS
- ORIGINAL GROUND TO BE EXCAVATED
- BOUNDARY BETWEEN DAM FILL AND BASIN FILL

NOTES

1. SECTION A IS A GENERALIZED CROSS-SECTION TO ILLUSTRATE THE EMBANKMENT CONFIGURATION AND INTERNAL ZONING.

REVISIONS			
NO.	DESCRIPTION	BY	DATE

DES.: DD
DRAWN: DD
CHKD.: JN
DATE: 05-19-10
SCALE: 1" = 20'
FLD. BK. XXXX

REGISTERED PROFESSIONAL ENGINEER ALBERTO PUJOL-RUIZ No. 37051 CIVIL STATE OF CALIFORNIA

PROJECT ENGINEER

PLANS APPROVAL DATE

GEI Consultants

CONTRA COSTA COUNTY
PUBLIC WORKS DEPARTMENT
255 GLACIER DRIVE
MARTINEZ, CALIFORNIA 94553

0 20 40
SCALE, FEET

FOR REDUCED PLANS ORIGINAL SCALE IS IN INCHES 0 1 2 3

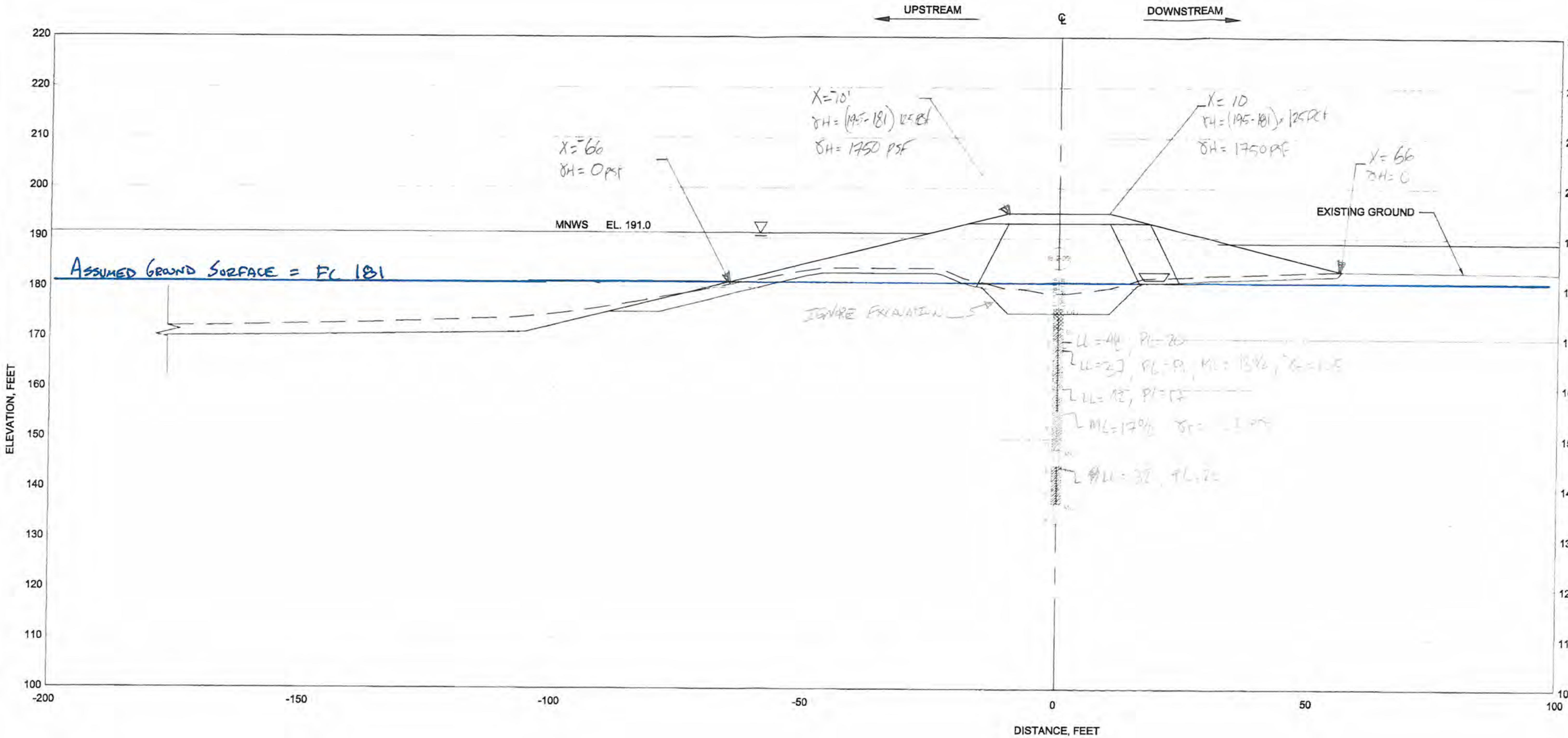
BASE MAP	EAST COORD.	NORTH COORD.
XXX	XXXX	XXX

UPPER SAND CREEK DETENTION BASIN

DAM EMBANKMENT - CROSS SECTIONS
Sta. 6+00

FILE NO.

Figure 7



MATERIAL TYPE

1. CLAY
2. FILTER SAND
3. DRAIN GRAVEL
4. RANDOM FILL
5. AGGREGATE BASE

LEGEND

- PROPOSED FINAL GRADE (EMBANKMENT AND EXCAVATION)
- EXISTING GROUND THAT REMAINS
- ORIGINAL GROUND TO BE EXCAVATED
- BOUNDARY BETWEEN DAM FILL AND BASIN FILL

NOTES

1. SECTION A IS A GENERALIZED CROSS-SECTION TO ILLUSTRATE THE EMBANKMENT CONFIGURATION AND INTERNAL ZONING.

REVISIONS			
NO.	DESCRIPTION	BY	DATE

DES.: DD
DRAWN: DD
CHKD.: JN
DATE: 05-19-10
SCALE: 1" = 20'
FLD. BK. XXXX

REGISTERED PROFESSIONAL ENGINEER ALBERTO PUJOL-RIOS No. 37051 CIVIL STATE OF CALIFORNIA

PROJECT ENGINEER

PLANS APPROVAL DATE

GEI Consultants

CONTRA COSTA COUNTY
PUBLIC WORKS DEPARTMENT
255 GLACIER DRIVE
MARTINEZ, CALIFORNIA 94553

0 20 40
SCALE, FEET

FOR REDUCED PLANS ORIGINAL SCALE IS IN INCHES
0 1 2 3

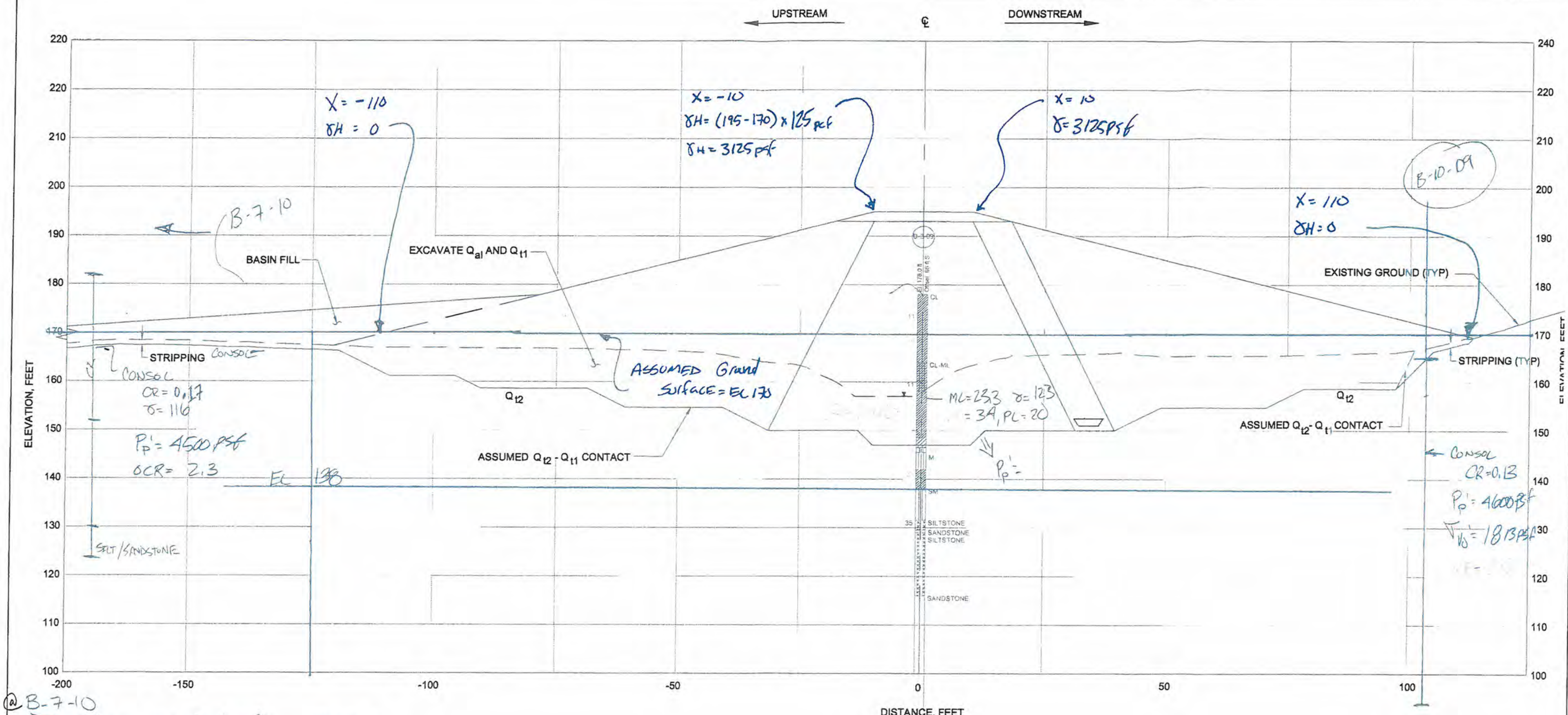
BASE MAP	EAST COORD.	NORTH COORD.
XXX	XXXX	XXX

UPPER SAND CREEK DETENTION BASIN

DAM EMBANKMENT - CROSS SECTIONS
Sta. 12+11

FILE NO.

Figure 2



@ B-7-10

$\sigma_{v0} = (182 - 166.5)(125) = 1937.5$

@ B-10-09

$\sigma_{v0}' = (165 - 155)\ 125\ psf + (155 - 146)(125 - 62.4)$

1813 psf

MATERIAL TYPE

1. CLAY
2. FILTER SAND
3. DRAIN GRAVEL
4. RANDOM FILL
5. AGGREGATE BASE

LEGEND

- PROPOSED FINAL GRADE (EMBANKMENT AND EXCAVATION)
- EXISTING GROUND THAT REMAINS
- ORIGINAL GROUND TO BE EXCAVATED
- BOUNDARY BETWEEN DAM FILL AND BASIN FILL

NOTES

1. SECTION A IS A GENERALIZED CROSS-SECTION TO ILLUSTRATE THE EMBANKMENT CONFIGURATION AND INTERNAL ZONING.

REVISIONS			
NO.	DESCRIPTION	BY	DATE

DES: DD
DRAWN: DD
CHKD.: JN
DATE: 05-19-10
SCALE: 1" = 20'
FLD. BK XXXX

REGISTERED PROFESSIONAL ENGINEER ALBERTO PUJOL-RIUS No 37051 CIVIL STATE OF CALIFORNIA

PROJECT ENGINEER

PLANS APPROVAL DATE

GEI Consultants

CONTRA COSTA COUNTY
PUBLIC WORKS DEPARTMENT
255 GLACIER DRIVE
MARTINEZ, CALIFORNIA 94553

SCALE, FEET (0, 20, 40)

FOR REDUCED PLANS ORIGINAL SCALE IS IN INCHES

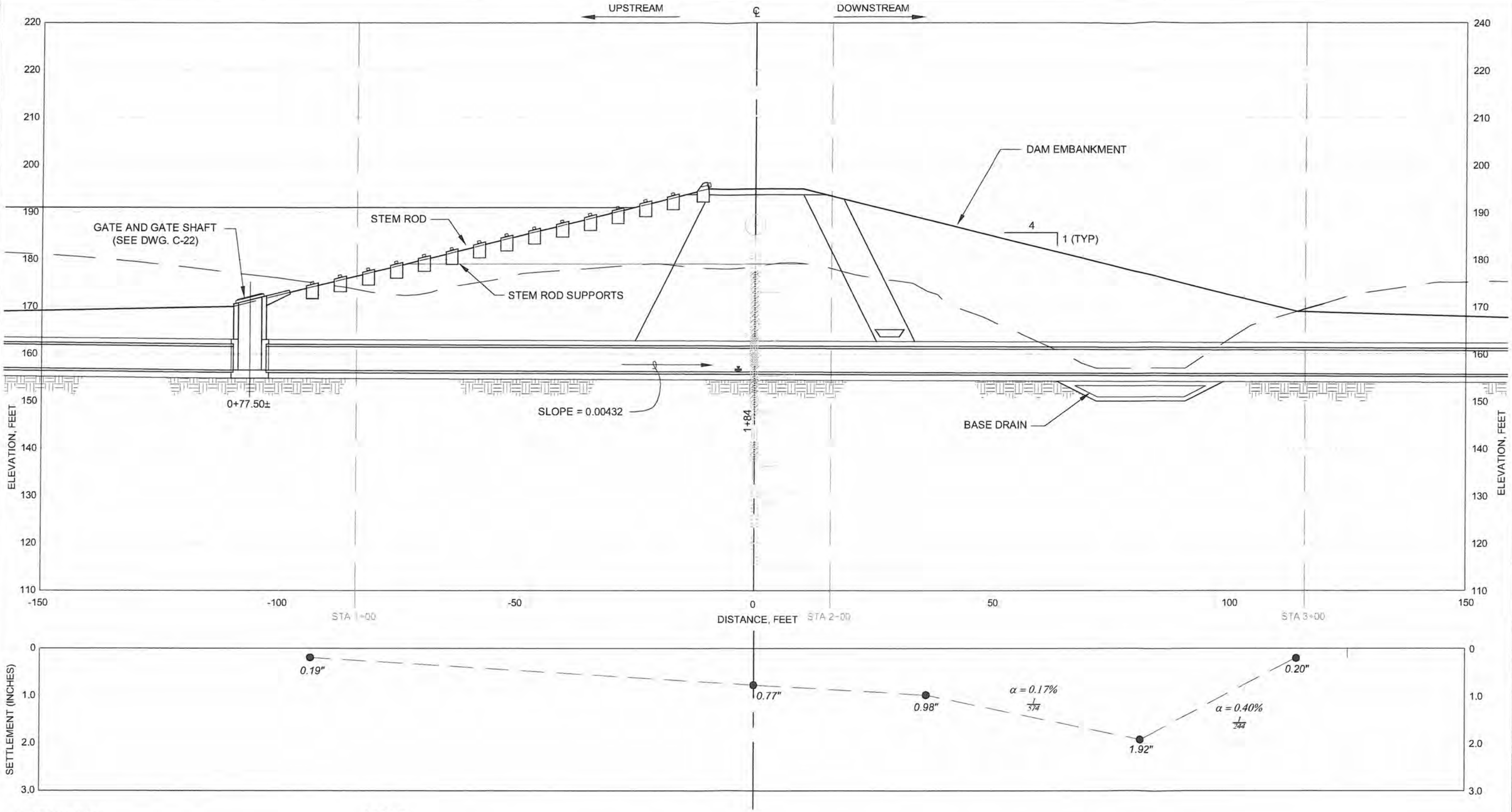
BASE MAP	EAST COORD.	NORTH COORD.
XXX	XXXX	XXX

UPPER SAND CREEK DETENTION BASIN

DAM EMBANKMENT - CROSS SECTIONS
Sta. 15+29

FILE NO

Figure 8 4



MATERIAL TYPE

1. CLAY
2. FILTER SAND
3. DRAIN GRAVEL
4. RANDOM FILL
5. AGGREGATE BASE

LEGEND

PROPOSED FINAL GRADE (EMBANKMENT AND EXCAVATION)
EXISTING GROUND THAT REMAINS
ORIGINAL GROUND TO BE EXCAVATED
BOUNDARY BETWEEN DAM FILL AND BASIN FILL

NOTES

1. SECTION A IS A GENERALIZED CROSS-SECTION TO ILLUSTRATE THE EMBANKMENT CONFIGURATION AND INTERNAL ZONING.

REVISIONS			
NO.	DESCRIPTION	BY	DATE

DES.: DD
DRAWN: DD
CHKD.: JN
DATE: 05-19-10
SCALE: 1" = 20'
FLD. BK. XXXX

REGISTERED PROFESSIONAL ENGINEER No. 37051 CIVIL STATE OF CALIFORNIA

PROJECT ENGINEER
PLANS APPROVAL DATE

GEI Consultants

CONTRA COSTA COUNTY
PUBLIC WORKS DEPARTMENT
255 GLACIER DRIVE
MARTINEZ, CALIFORNIA 94553

0 20 40
SCALE, FEET

FOR REDUCED PLANS ORIGINAL SCALE IS IN INCHES

BASE MAP	EAST COORD.	NORTH COORD.
XXX	XXXX	XXX

UPPER SAND CREEK DETENTION BASIN

OUTLET WORKS CROSS SECTION
DAM STATION 14+65

FILE NO. _

Figure 5

C-12 05-09-10 PYM

WINSAF-I Output for Settlement Along Centerline of Dam

STRIP LOADING VARIABLE SHAPE

Project Name: Upper Sand Cr. Det. Basin Project Number : 08318 SECTION 1
Client : CCCFCWCD Project Manager: JFN
Date : 5/14/2010 Computed by : JAD STA 6+00

Increment of stresses obtained using : Boussinesq

Settlement for X = 0.00 (ft)

Point #	X(ft)	Load (psf)
1	-34.00	0.00
2	-10.00	750.00
3	10.00	750.00
4	34.00	0.00

Foundation Elev. = 189.00 (ft) Ground Surface Elev.= 189.00 (ft)
Water table Elev. = 160.00 (ft) Unit weight of Wat. = 62.40 (pcf)

N°.	Layer Type	Thick. (ft)	Comp.	Recomp. Ratio	Swell.	Unit Weight (pcf)	Primary Settlement (in.)	Secondary Settlement (in.)
	COMP.	41.0	0.160	0.030	0.030	125.00	2.38	0.00
						Total Settlement =	2.38	0.00

N°.	Sublayer Thick. (ft)	Elev. (ft)	Initial (psf)	Soil Stresses Increment (psf)	Max.Past Press. (psf)	Settlement (in.)
1	4.10	186.95	256.25	749.49	4500.00	0.88
2	4.10	182.85	768.75	738.65	4500.00	0.43
3	4.10	178.75	1281.25	710.65	4500.00	0.28
4	4.10	174.65	1793.75	671.28	4500.00	0.20
5	4.10	170.55	2306.25	627.39	4612.50	0.15
6	4.10	166.45	2818.75	583.34	5637.50	0.12
7	4.10	162.35	3331.25	541.41	6662.50	0.10
8	4.10	158.25	3734.55	502.62	7469.04	0.08
9	4.10	154.15	3991.21	467.31	7982.22	0.07
10	4.10	150.05	4247.87	435.43	8495.41	0.06

Total Settlement = 2.38 (in.)

STRIP LOADING VARIABLE SHAPE

Project Name: Upper Sand Cr. Det. Basin Project Number : 08318
Client : CCCFCWCD Project Manager: JFN
Date : 5/14/2010 Computed by : JAD

SECTION 2
STA 12+11

Increment of stresses obtained using : Boussinesq

Settlement for X = 0.00 (ft)

Point #	X(ft)	Load (psf)
1	-66.00	0.00
2	-10.00	1750.00
3	10.00	1750.00
4	66.00	0.00

Foundation Elev. = 181.00 (ft) Ground Surface Elev.= 181.00 (ft)
Water table Elev. = 160.00 (ft) Unit weight of Wat. = 62.40 (pcf)

.	Layer Type	Thick. (ft)	Comp.	Recomp. Ratio	Swell.	Unit Weight (pcf)	Primary Settlement (in.)	Secondary Settlement (in.)
1	COMP.	33.0	0.160	0.030	0.030	125.00	4.17	0.00
2	INCOMP.	4.0				125.00	0.00	0.00
3	COMP.	8.0	0.160	0.030	0.030	125.00	0.36	0.00
				Total Settlement =			4.53	0.00

Nº.	Sublayer Thick. (ft)	Elev. (ft)	Soil Stresses Initial (psf)	Increment (psf)	Max.Past Press. (psf)	Settlement (in.)
1	4.13	178.94	257.81	1749.45	4500.00	1.32
2	4.13	174.81	773.44	1737.50	4500.00	0.76
3	4.13	170.69	1289.06	1705.80	4500.00	0.54
4	4.13	166.56	1804.69	1659.22	4500.00	0.42
5	4.13	162.44	2320.31	1604.20	4640.63	0.34
6	4.13	158.31	2730.64	1545.17	5461.29	0.29
7	4.13	154.19	2988.86	1484.81	5977.77	0.26
8	4.13	150.06	3247.09	1424.76	6494.26	0.23
9	INCOMP.					
10	4.00	142.00	3751.80	1311.79	7503.75	0.19
11	4.00	138.00	4002.20	1258.85	8004.58	0.17

Total Settlement = 4.53 (in.)

STRIP LOADING VARIABLE SHAPE

?roject Name: Upper Sand Cr. Det. Basin Project Number : 08318
'lient : CCCFCWCD Project Manager: JFN
ate : 5/14/2010 Computed by : JAD

SECTION 3
15+29

Increment of stresses obtained using : Boussinesq

Settlement for X = 0.00 (ft)

Point #	X(ft)	Load (psf)
1	-110.00	0.00
2	-10.00	3125.00
3	10.00	3125.00
4	110.00	0.00

'oundation Elev. = 170.00 (ft) Ground Surface Elev.= 170.00 (ft)
ater table Elev. = 160.00 (ft) Unit weight of Wat. = 62.40 (pcf)

'.	Layer Type	Thick. (ft)	Comp.	Recomp. Ratio	Swell.	Unit Weight (pcf)	Primary Settlement (in.)	Secondary Settlement (in.)
-	COMP.	22.0	0.160	0.030	0.030	125.00	5.29	0.00
2	INCOMP.	6.0				125.00	0.00	0.00
3	COMP.	4.0	0.160	0.030	0.030	125.00	0.62	0.00
						Total Settlement =	5.92	0.00

N°.	Sublayer Thick. (ft)	Sublayer Elev. (ft)	Soil Stresses Initial (psf)	Soil Stresses Increment (psf)	Soil Stresses Max.Past Press. (psf)	Settlement (in.)
1	4.40	167.80	275.00	3124.32	4500.00	1.73
2	4.40	163.40	825.00	3109.91	4500.00	1.07
3	4.40	159.00	1312.60	3072.60	4500.00	0.83
4	4.40	154.60	1588.04	3018.53	4500.00	0.80
5	4.40	150.20	1863.48	2954.72	4500.00	0.86
6	INCOMP.					
7	1.33	141.33	2418.53	2812.83	4835.11	0.23
8	1.33	140.00	2502.00	2790.83	5002.67	0.21
9	1.33	138.67	2585.47	2768.77	5170.22	0.18

Total Settlement = 5.92 (in.)

WINSAF-I Output for Settlement Along Outlet Works

STRIP LOADING VARIABLE SHAPE

Project Name: Upper Sand Cr. Det. Basin Project Number : 08318
Client : CCCFCWCD
Date : 7/8/2010
Project Manager: JFN
Computed by : DRD

Increment of stresses obtained using : Boussinesq

Settlement for X = -90.00 (ft)

SECTION 1
OUTLET WORKS
STA. 0+90

Point #	X(ft)	Load (psf)
1	-90.00	0.00
2	-10.00	2500.00
3	10.00	2500.00
4	90.00	0.00

Foundation Elev. = 175.00 (ft) Ground Surface Elev.= 175.00 (ft)
Water table Elev. = 160.00 (ft) Unit weight of Wat. = 62.40 (pcf)

N°.	Layer Type	Thick. (ft)	Comp.	Recomp. Ratio	Swell.	Unit Weight (pcf)	Primary Settlement (in.)	Secondary Settlement (in.)
1	INCOMP.	20.0				125.00	0.00	0.00
2	COMP.	7.0	0.160	0.030	0.030	125.00	0.10	0.00
3	COMP.	6.0	0.060	0.010	0.010	125.00	0.03	0.00
4	COMP.	4.0	0.160	0.030	0.030	125.00	0.06	0.00
				Total Settlement =			0.19	0.00

N°.	Sublayer Thick. (ft)	Elev. (ft)	Soil Stresses Initial (psf)	Increment (psf)	Max.Past Press. (psf)	Settlement (in.)
1	INCOMP.					
2	1.75	154.13	2242.78	201.10	4500.00	0.02
3	1.75	152.38	2352.33	216.78	4703.53	0.02
4	1.75	150.63	2461.88	232.18	4922.72	0.02
5	1.75	148.88	2571.43	247.30	5141.91	0.03
6	2.00	147.00	2688.80	263.18	5376.75	0.01
7	2.00	145.00	2814.00	279.72	5627.25	0.01
8	2.00	143.00	2939.20	295.85	5877.75	0.01
9	1.33	141.33	3043.53	308.95	6086.50	0.02
10	1.33	140.00	3127.00	319.22	6253.50	0.02
11	1.33	138.67	3210.47	329.29	6420.50	0.02

Total Settlement = 0.19 (in.)

STRIP LOADING VARIABLE SHAPE

Project Name: Upper Sand Cr. Det. Basin Project Number : 08318
Client : CCCFCWCD Project Manager: JFN
Date : 7/8/2010 Computed by : DRD

Increment of stresses obtained using : Boussinesq

Settlement for X = 0.00 (ft)

SECTION 2
OUTLET WORKS
STA. 1+85

Point #	X(ft)	Load (psf)
1	-70.00	0.00
2	-10.00	1875.00
3	10.00	1875.00
4	70.00	0.00

Foundation Elev. = 180.00 (ft) Ground Surface Elev.= 180.00 (ft)
Water table Elev. = 160.00 (ft) Unit weight of Wat. = 62.40 (pcf)

N°.	Layer Type	Thick. (ft)	Comp.	Recomp. Ratio	Swell.	Unit Weight (pcf)	Primary Settlement (in.)	Secondary Settlement (in.)
1	INCOMP.	25.0				125.00	0.00	0.00
2	COMP.	7.0	0.160	0.030	0.030	125.00	0.46	0.00
3	COMP.	6.0	0.060	0.010	0.010	125.00	0.11	0.00
4	COMP.	4.0	0.160	0.030	0.030	125.00	0.20	0.00
				Total Settlement =			0.77	0.00

N°.	Sublayer Thick. (ft)	Elev. (ft)	Soil Stresses Initial (psf)	Increment (psf)	Max.Past Press. (psf)	Settlement (in.)
1	INCOMP.					
2	2.33	153.83	2886.03	1616.71	5607.26	0.16
3	2.33	151.50	3032.10	1581.92	5923.62	0.15
4	2.33	149.17	3178.17	1547.31	6239.99	0.14
5	2.00	147.00	3313.80	1515.48	6533.75	0.04
6	2.00	145.00	3439.00	1486.46	6804.92	0.04
7	2.00	143.00	3564.20	1457.84	7076.08	0.04
8	1.33	141.33	3668.53	1434.35	7302.06	0.07
9	1.33	140.00	3752.00	1415.80	7482.83	0.07
10	1.33	138.67	3835.47	1397.48	7663.61	0.06

Total Settlement = 0.77 (in.)

STRIP LOADING VARIABLE SHAPE

Project Name: Upper Sand Cr. Det. Basin Project Number : 08318
Client : CCCFCWCD Project Manager: JFN
Date : 7/8/2010 Computed by : DRD

Increment of stresses obtained using : Boussinesq

Settlement for X = 35.00 (ft)

SECTION 3

OUTLET WORKS
STA. 2+20

Point #	X(ft)	Load (psf)
1	-90.00	0.00
2	-10.00	2500.00
3	10.00	2500.00
4	90.00	0.00

Foundation Elev. = 175.00 (ft) Ground Surface Elev.= 175.00 (ft)
Water table Elev. = 160.00 (ft) Unit weight of Wat. = 62.40 (pcf)

N°.	Layer Type	Thick. (ft)	Comp.	Recomp. Ratio	Swell.	Unit Weight (pcf)	Primary Settlement (in.)	Secondary Settlement (in.)
1	INCOMP.	20.0				125.00	0.00	0.00
2	COMP.	7.0	0.160	0.030	0.030	125.00	0.58	0.00
3	COMP.	6.0	0.060	0.010	0.010	125.00	0.14	0.00
4	COMP.	4.0	0.160	0.030	0.030	125.00	0.26	0.00
				Total Settlement =			0.98	0.00

N°.	Sublayer Thick. (ft)	Elev. (ft)	Soil Stresses Initial (psf)	Increment (psf)	Max.Past Press. (psf)	Settlement (in.)
1	INCOMP.					
2	2.33	153.83	2261.03	1680.70	4520.87	0.20
3	2.33	151.50	2407.10	1669.82	4813.12	0.19
4	2.33	149.17	2553.17	1657.77	5105.37	0.18
5	2.00	147.00	2688.80	1645.65	5376.75	0.05
6	2.00	145.00	2814.00	1633.75	5627.25	0.05
7	2.00	143.00	2939.20	1621.24	5877.75	0.05
8	1.33	141.33	3043.53	1610.41	6086.50	0.09
9	1.33	140.00	3127.00	1601.52	6253.50	0.09
10	1.33	138.67	3210.47	1592.43	6420.50	0.08

Total Settlement = 0.98 (in.)

STRIP LOADING VARIABLE SHAPE

Project Name: Upper Sand Cr. Det. Basin Project Number : 08318
Client : CCCFCWCD Project Manager: JFN
Date : 7/8/2010 Computed by : DRD

Increment of stresses obtained using : Boussinesq

Settlement for X = 80.00 (ft)

SECTION 4 - RUN 1
(ENTIRE EMBANKMENT)

Point #	X(ft)	Load (psf)
1	-170.00	0.00
2	-10.00	5000.00
3	10.00	5000.00
4	175.00	0.00

Foundation Elev. = 155.00 (ft) Ground Surface Elev.= 155.00 (ft)
Water table Elev. = 155.00 (ft) Unit weight of Wat. = 62.40 (pcf)

N°.	Layer Type	Thick. (ft)	Comp.	Recomp. Ratio	Swell.	Unit Weight (pcf)	Primary Settlement (in.)	Secondary Settlement (in.)
1	INCOMP.	5.0				125.00	0.00	0.00
2	COMP.	2.0	0.160	0.030	0.030	125.00	0.68	0.00
3	COMP.	6.0	0.060	0.010	0.010	125.00	0.54	0.00
4	COMP.	4.0	0.160	0.030	0.030	125.00	0.88	0.00
				Total Settlement =			2.10	0.00

N°.	Sublayer Thick. (ft)	Elev. (ft)	Soil Stresses Initial (psf)	Increment (psf)	Max.Past Press. (psf)	Settlement (in.)
1	INCOMP.					
2	2.00	149.00	375.60	2878.65	4500.00	0.68
3	2.00	147.00	500.80	2878.45	4500.00	0.20
4	2.00	145.00	626.00	2878.14	4500.00	0.18
5	2.00	143.00	751.20	2877.68	4500.00	0.16
6	1.33	141.33	855.53	2877.15	4500.00	0.31
7	1.33	140.00	939.00	2876.64	4500.00	0.29
8	1.33	138.67	1022.47	2876.03	4500.00	0.28

Total Settlement = 2.10 (in.)

STRIP LOADING VARIABLE SHAPE

Project Name: Upper Sand Cr. Det. Basin Project Number : 08318
Client : CCCFCWCD Project Manager: JFN
Date : 7/8/2010 Computed by : DRD

Increment of stresses obtained using : Boussinesq

Settlement for X = 80.00 (ft)

SECTION 4 - RUN 2
(EXISTING CONTOURS)

Point #	X(ft)	Load (psf)
1	-170.00	0.00
2	-70.00	3125.00
3	10.00	3125.00
4	70.00	375.00
5	90.00	375.00
6	112.00	1875.00
7	170.00	0.00

Foundation Elev. = 155.00 (ft) Ground Surface Elev.= 155.00 (ft)
Water table Elev. = 155.00 (ft) Unit weight of Wat. = 62.40 (pcf)

N°.	Layer Type	Thick. (ft)	Comp.	Recomp. Ratio	Swell.	Unit Weight (pcf)	Primary Settlement (in.)	Secondary Settlement (in.)
1	INCOMP.	5.0				125.00	0.00	0.00
2	COMP.	2.0	0.160	0.030	0.030	125.00	0.22	0.00
3	COMP.	6.0	0.060	0.010	0.010	125.00	0.17	0.00
4	COMP.	4.0	0.160	0.030	0.030	125.00	0.28	0.00
				Total Settlement =			0.67	0.00

N°.	Sublayer Thick. (ft)	Elev. (ft)	Initial (psf)	Soil Stresses Increment (psf)	Max.Past Press. (psf)	Settlement (in.)
1	INCOMP.					
2	2.00	149.00	375.60	394.28	4500.00	0.22
3	2.00	147.00	500.80	414.98	4500.00	0.06
4	2.00	145.00	626.00	442.39	4500.00	0.06
5	2.00	143.00	751.20	474.83	4500.00	0.05
6	1.33	141.33	855.53	504.51	4500.00	0.10
7	1.33	140.00	939.00	529.40	4500.00	0.09
8	1.33	138.67	1022.47	554.90	4500.00	0.09

Total Settlement = 0.67 (in.)

Section 4 - Outlet Works Sta. 2+65
Net Stress Increase Calculation

Run 1 Stress Dist.		Run 2 Stress Dist.		
Elevation (ft)	Stress (psf)	Elevation (ft)	Stress (psf)	Net Stress (psf)
149	2878.65	149	394.28	2484.37
147	2878.45	147	414.98	2463.47
145	2878.14	145	442.39	2435.75
143	2877.68	143	474.83	2402.85
141.3	2877.15	141.3	504.51	2372.64
140	2876.64	140	529.4	2347.24
138.7	2876.03	138.7	554.9	2321.13

Notes:

1. Run 1 estimates the increase in stress versus depth for the proposed embankment loading assuming an estimated horizontal ground surface at El. 155.

2. Run 2 estimates the increase in stress versus depth caused by the existing contours assuming a horizontal ground surface at El. 155.

USER DEFINED INCREMENT OF VERTICAL STRESS WITH DEPTH

Project Name: Upper Sand Cr. Det. Basin Project Number : 08318
Client : CCCFCWCD Project Manager: JFN
Date : 7/8/2010 Computed by : DRD

SECTION 4
OUTLET WORKS
STA. 2+65

Foundation Elev. = 155.00 (ft) Ground Surface Elev.= 155.00 (ft)
Water table Elev. = 155.00 (ft) Unit weight of Wat. = 62.40 (pcf)

N°.	Layer Type	Thick. (ft)	Comp.	Recomp. Ratio	Swell.	Unit Weight (pcf)	Primary Settlement (in.)	Secondary Settlement (in.)
1	INCOMP.	5.0				125.00	0.00	0.00
2	COMP.	2.0	0.160	0.030	0.030	125.00	0.63	0.00
3	COMP.	6.0	0.060	0.010	0.010	125.00	0.50	0.00
4	COMP.	4.0	0.160	0.030	0.030	125.00	0.78	0.00
						Total Settlement =	1.92	0.00

N°.	Sublayer Thick. (ft)	Sublayer Elev. (ft)	Soil Stresses Initial (psf)	Increment (psf)	Max.Past Press. (psf)	Settlement (in.)
1	INCOMP.					
2	2.00	149.00	375.60	2484.37	4500.00	0.63
3	2.00	147.00	500.80	2463.47	4500.00	0.19
4	2.00	145.00	626.00	2435.75	4500.00	0.17
5	2.00	143.00	751.20	2402.85	4500.00	0.15
6	1.33	141.33	855.53	2373.23	4500.00	0.28
7	1.33	140.00	939.00	2347.24	4500.00	0.26
8	1.33	138.67	1022.47	2320.46	4500.00	0.25

Total Settlement = 1.92 (in.)

STRIP LOADING VARIABLE SHAPE

Project Name: Upper Sand Cr. Det. Basin Project Number : 08318
Client : CCCFCWCD
Project Manager: JFN
Date : 7/8/2010
Computed by : DRD

Increment of stresses obtained using : Boussinesq

Settlement for X = 115.00 (ft)

SECTION 5
OUTLET WORKS
STA. 2+98

Point #	X(ft)	Load (psf)
1	-115.00	0.00
2	-10.00	3125.00
3	10.00	3125.00
4	115.00	0.00

Foundation Elev. = 170.00 (ft) Ground Surface Elev.= 170.00 (ft)
Water table Elev. = 160.00 (ft) Unit weight of Wat. = 62.40 (pcf)

N°.	Layer Type	Thick. (ft)	Comp.	Recomp. Ratio	Swell.	Unit Weight (pcf)	Primary Settlement (in.)	Secondary Settlement (in.)
1	INCOMP.	15.0				125.00	0.00	0.00
2	COMP.	7.0	0.160	0.030	0.030	125.00	0.10	0.00
3	COMP.	6.0	0.060	0.010	0.010	125.00	0.03	0.00
4	COMP.	4.0	0.160	0.030	0.030	125.00	0.06	0.00
				Total Settlement =			0.20	0.00

N°.	Sublayer Thick. (ft)	Elev. (ft)	Initial (psf)	Soil Stresses Increment (psf)	Max.Past Press. (psf)	Settlement (in.)
1	INCOMP.					
2	2.33	153.83	1636.03	151.37	4500.00	0.03
3	2.33	151.50	1782.10	172.59	4500.00	0.03
4	2.33	149.17	1928.17	193.57	4500.00	0.03
5	6.00	145.00	2189.00	230.35	4500.00	0.03
6	1.33	141.33	2418.53	261.89	4835.11	0.02
7	1.33	140.00	2502.00	273.15	5002.67	0.02
8	1.33	138.67	2585.47	284.29	5170.22	0.02

Total Settlement = 0.20 (in.)

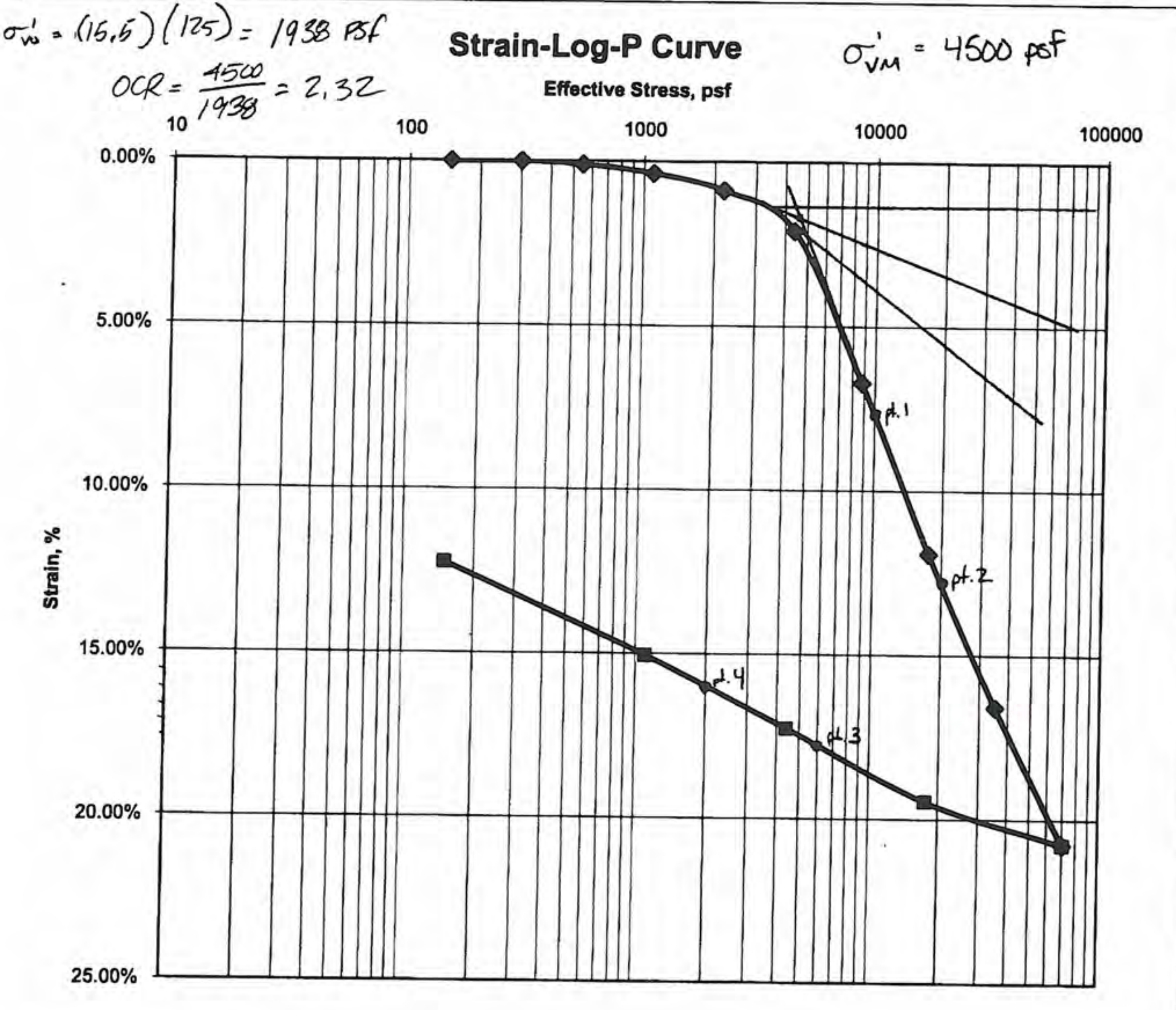
Consolidation Test
ASTM D2435

Job No.:	471-051	**Boring:**	B7	**Run By:**	MD
Client:	Cal Engineering & Geology	**Sample:**	7-7	**Reduced:**	PJ
Project:	Upper Sand Creek Basin - 081070	**Depth, ft.:**	15.5(Tip)	**Checked:**	PJ/DC
Soil Type:	Brown Lean CLAY, trace Sand & small root holes			**Date:**	4/6/2010

$\sigma'_{vo} = (15.5)(125) = 1938$ psf

$OCR = \frac{4500}{1938} = 2.32$

$\sigma'_{VM} = 4500$ psf



Ass. Gs = 2.75	**Initial**	**Final**
Moisture %:	22.8	22.8
Dry Density, pcf:	94.1	105.6
Void Ratio:	0.824	0.626
% Saturation:	76.1	100

Remarks:

$CR = \frac{0.125 - 0.075}{\log\left(\frac{20{,}000}{10{,}000}\right)} = 0.166$

pt. 1 = (10,000, .075)
pt. 2 = (20,000, 0.125)

$RR = \frac{0.175 - 0.165}{\log\left(\frac{6000}{2000}\right)} = 0.021$

pt 3 = (6000, 0.175)
pt. 4 = (2000, 0.165)

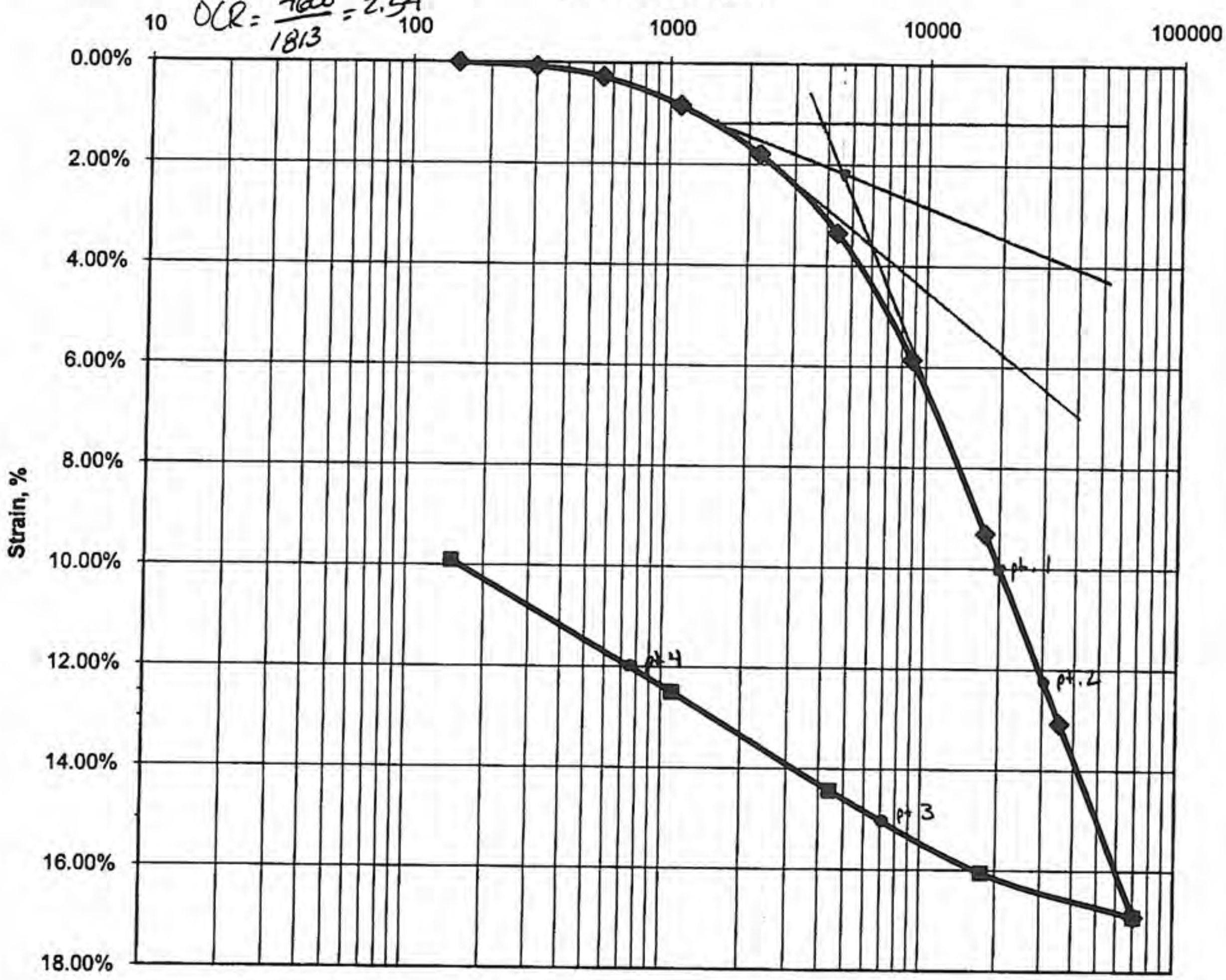
Consolidation Test
ASTM D2435

Job No.:	471-051	Boring:	B10	Run By:	MD
Client:	Cal Engineering & Geology	Sample:	10-9	Reduced:	PJ
Project:	Upper Sand Creek Basin - 081070	Depth, ft.:	19.0(Tip)	Checked:	PJ/DC
Soil Type:	Brown Mottled Gray Sandy Lean CLAY			Date:	4/2/2010

$\sigma_{v_o}' = (165-155)\,125\text{psf} + (155-146)(125-62.4) = 1813\text{psf}$

$OCR = \frac{4600}{1813} = 2.54$

$\sigma_{VM}' = 4600\text{psf}$



Ass. Gs = 2.7	Initial	Final
Moisture %:	23.8	19.7
Dry Density, pcf:	100.9	110.1
Void Ratio:	0.671	0.531
% Saturation:	95.6	100

Remarks:

$CR = \frac{0.1225 - 0.10}{\log\left(\frac{30,000}{20,000}\right)} = 0.127$

pt.1 = (20,000, 0.10)
pt.2 = (30,000, 0.1225)

$RR = \frac{0.148 - 0.12}{\log\left(\frac{7000}{750}\right)} = 0.03$

pt.3 = (7000, 0.148)
pt.4 = (750, 0.12)

Appendix 3 – Hydraulic Analyses

Appendix 3.1

Spillway Hydraulics

Emergency Spillway Design

1. Introduction

The Upper Sand Creek Dam Emergency Spillway will be located at the Northeast corner of the embankment dam. The dam crest will be at El. 195.5 and the spillway weir invert will be at El. 191.0. The spillway will be an un-gated weir, designed to pass the 5,700-cfs design flood flow with 0.1-feet of freeboard.

The spillway will include a broad crested weir, an inclined chute, a side channel, a 950-ft long discharge channel, and a riprap blanket at the downstream end. The spillway configuration will allow at-grade maintenance vehicle access across the spillway.

2. Site Specific Features

Previous studies and several meetings with District engineers established the following criteria for the spillway design:

- The spillway control structure will be an un-gated weir to preclude manual spillway operation in emergency flooding situations.
- The weir side slopes may not exceed a 10 percent grade to allow at-grade maintenance vehicle crossing over the spillway.
- An articulated concrete block lining in the spillway discharge channel is preferred for the following two purposes: to blend with the surrounding ground and vegetation due to the particular usage of the area as park grounds, and to provide adequate flexibility necessary for the clay foundation materials at the site.

3. Hydraulic Design of the Emergency Spillway

3.1. Spillway Weir and Chute

The optimum spillway weir length was determined in the preliminary design stage to be 225-ft. The spillway side slopes were set at a 10 percent grade to provide an at-grade spillway crossing for maintenance vehicle during normal reservoir operation.

The spillway design capacity was based on established design conditions over a broad-crested weir. Experimental observations have shown that flow passing over a broad-crested weir will be at critical depth at about $4d_c$ (four times critical depth) from the downstream edge of the weir (from the brink). At the brink, the flow depth will be approximately $0.7d_c$. The energy gradient at the critical flow depth is equal to $1.5d_c$ (one-and-half times the critical depth). Accordingly, the reservoir water surface level corresponding to the design discharge of 5700-cfs was estimated as weir crest level El. 191.0 plus one-and-half times the critical depth (d_c).

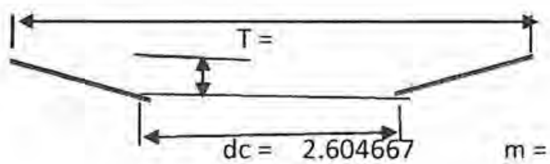
The capacity over the trapezoidal shaped weir section was calculated utilizing the general formula:

$Q^2/g = A^3/T$

Where:

Q = design discharge (cfs)
g = acceleration of gravity (32.3 ft/s^2)
A = Cross sectional area (ft^2)
T = Width of water surface (ft)

Q = 5700
m =
10/1 10
B = 225
T = B+2*dc*m
A = (B+dc*m)*dc



Q^2/g = A^3/T

(1) Q	(2) dc	(3) (Q)^2/g	(4) A	(5) A^3	(6) T	(7) A^3/T	(8) (3)-(7)=0	(9) V	(10) F
5700	2.60	1009006	653.89	3.E+08	277.09	1009006	0	8.72	1.00

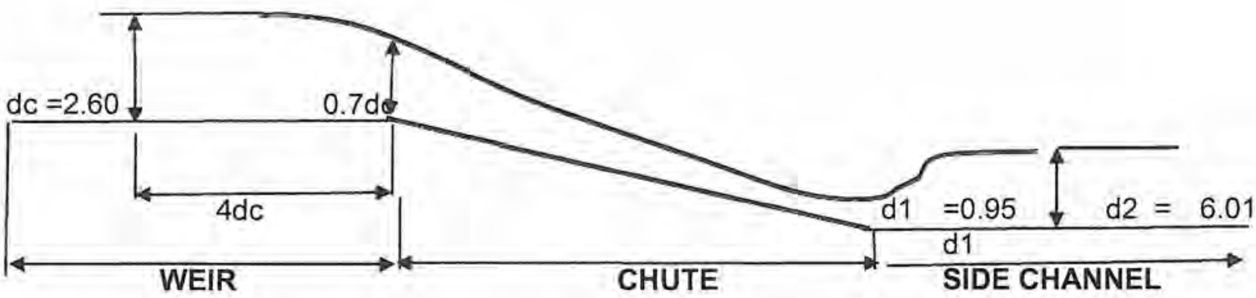
The water level profile over the spillway weir was computed for the 5,700-cfs design flow using the calculated critical depth value. Using the above equations, the critical depth of flow over the weir was calculated to be 2.60 feet, corresponding to a reservoir elevation of 194.9 during the design flood.

The inclined chute at the downstream end of the spillway weir will drop at a 4H:1V slope from El. 191.0 on the weir crest to El.184.50 at the north end and El. 182.0 at the south end. The flow profile from the end of the weir down to the end of the inclined chute was computed using Bernoulli's equation.

The corresponding flow velocity and Froude Number were computed based on the depth of flow (d_1) at the lower end of the inclined chute.

The flow conjugate depth (d_2) in the side channel is computed using the relation between the flow depths (d_1) and (d_2), and the Froude number.

Computations carried out for the design discharge of 5700-cfs resulted the following values:



Copies of the hydraulic computations for the weir and the chute are provided at the end of this report.

A roller-flow regime, characteristic of side channel flow, cannot be obtained due to the excessive side channel width; however, a hydraulic jump flow condition will prevail. Hydraulic calculations also indicate that the kinetic energy of the incoming flow down the inclined chute will be sufficiently dissipated, however the computed conjugate depth of the water (6.01 ft) will remain lower than the bank surrounding the side channel (7.5 ft).

As the incoming flow increases, the water level in the side channel will rise and, due to cross channel slope, the water will change direction to flow through the spillway discharge channel.

3.2. Spillway Discharge Channel

The discharge channel will extend downstream 950-ft, including the 225-ft long side channel segment, and drops from elevation 184.50 down to elevation 155.

The channel width will vary from 25-ft to 50-ft in the first 200-ft segment, then will remain constant at 50-ft until the end. The discharge channel and the side slopes will be lined with articulated concrete blocks that will extend above the maximum water surface on each side.

The discharge channel slope will vary, complying with hydraulic and topographic requirements. The channel will begin with a -1.1 percent grade, will increase to a -9.0 percent grade, and will terminate with a -0.4 percent grade as it reaches the creek bed.

The computed design flows in the channel will have velocities approaching 27 ft/sec, and will enter the creek at a velocity of nearly 22 ft/sec. Riprap protection will be provided in the creek to avoid scouring. However, at the design discharge, the high tailwater level in the creek will absorb most of the kinetic energy of the flow, assuming the outlet works is operational during the flood.

Channel hydraulic computations for estimating water depths and flow velocities for the design discharge of 5,700-cfs were carried on using the step method. The channel design was also analyzed with a discharge of 3,000-cfs to evaluate the flow characteristics under an intermediate discharge. Excel spreadsheets of hydraulic calculations carried on for 5.700-cfs and for 3,000-cfs are included with this report.

4. Tailwater Rating Curve

The flow regime of the creek downstream of the spillway discharge channel outlet is fixed by the natural conditions along the creek. The tailwater rating curve provides the stage-discharge relationship of the natural creek. It is necessary for providing appropriate measures to dissipate the kinetic energy of the incoming flow.

Two locations, one at 160-ft downstream and the other at 220-ft downstream of the spillway discharge channel outfall were selected as the representative creek cross sections for developing the tailwater rating curve.

The longitudinal slope of the Upper Sand Creek downstream of the spillway discharge channel outlet is estimated from the 1-in =40-ft scale topo map as shown on the table.

Stretch		Distance	Slope
Point	**El.**	**ft**	**%**
*Start	155	0	
1	153	340	0.588%
2	151	720	0.526%
3	150	860	0.714%
4	148	1300	0.455%
*spillway outlet		Average	0.571%

The Manning's roughness coefficient values for various reaches of Sand Creek developed by the District were provided to GEI. Based on the table of values provided, the "n" value of 0.05, corresponding to the creek segment starting from the outlet and extending approximately 800-ft downstream, was adopted for the tailwater rating curve computations.

Manning's equation was used to determine the stage-discharge relationship at the selected creek sections. Cross sectional areas and wetted perimeters were calculated at each 5-ft elevation interval to compute the discharge at the creek stage using the above determined "n" and slope values. The tailwater curves and related calculations are provided with this report.

Both of the curves show similar values, Elevation 172.0 and 171.5, for the design discharge.

5. Energy Dissipation and Riprap Design

5.1. Submergence Requirement

Spillway discharge channel hydraulic computations carried on for the design flow of 5700-cfs and for 3000-cfs provide the following results:

Spillway Discharge Channel				Upper Sand Creek		
Discharge	Depth	Velocity	Froude Nb.	Conj.Depth	Water El.	T.W.El.
cfs	ft	ft.sec		ft		ft
5700	3.95	21.95	2.17	10.3	165	172
3000	2.99	16.21	1.8	6.3	161.3	168

A comparison of the flow depth in the spillway discharge channel and the conjunctive depth in Upper Sand Creek suggest that the incoming flow will be submerged and the kinetic energy due to flows in the spillway will dissipated completely. The necessary submergence will be available in Upper Sand Creek to dissipate the kinetic energy across the full range of spillway discharges, assuming the outlet works is operational during the flood.

5.2. Scour protection

Based on tests conducted by the Bureau of Reclamation, a series of small rollers will develop on the water surface as the Froude Number of the incoming flow approaches 1.7. These become more intense with the increasingly higher values of the Froude Number. Other than surface roller phenomena, relatively smooth flow prevails throughout the Froude Number range up to about 2.5. This is the flow condition expected to prevail at the confluence of the discharge channel and the creek since the computed Froude numbers will be varying from 1.8 to 2.2.

When the incoming supercritical flow drowns in the creek water, which will be much higher than the spillway channel flow level, velocity will drop to subcritical. At 5,700-cfs design flow discharge, the creek water level will rise to about El. 172, causing the tailwater to back up into the discharge channel approximately 260-ft, corresponding to discharge channel station 6+90. Therefore, the flow regime downstream of station 6+90 will be subcritical with some small surface rollers as explained above.

Scour protection at the end of spillway channel was designed for the estimated creek flow velocity, and not the incoming spillway flow velocity, since the kinetic energy in the spillway channel flow will be dissipated before it reaches the creek bed.

The scour protection design was designed based on the guidelines prescribed in the State of California Department of Transportation publication "California Bank and Shore Rock Slope Protection Design", dated October 2000.

Equation 1 of section 5-1-C in the above Caltrans publication was used to determine the minimum stone weight. The flow velocity in the creek was estimated at 9-ft/sec based on the 5,700-cfs design flow through the existing Upper Sand Creek channel. Multiplying 9-ft/sec by a factor of 1.33 for impinging flow conditions increased the flow velocity to 12-ft/sec. Values used in the computation were as follows:

- W = Minimum mass rock mass Size or weight
- V = Velocity of flow (12-ft/sec)
- SG = Specific gravity of rock (2.65)
- SGW = Specific gravity of water (1)
- r = 70 degrees (for random placed ruble)
- a = Outside slope face angle with horizontal (degrees) arc tan(1/4)=14^0

Calculations with the above values resulted in a minimum stone weight W = 42.5-lb rock. Therefore, from Tables 5-1, 5-2, and 5-3 of the above referred publication, 75-lb rock, Backing No.1(B), RSP-Fabric Type A, and Backing layer thickness 1.8-ft with method of placement B are required. As a precautionary measure, the RSP class in the creek bed was increased to "Light" class.

Client	Contra Costa County	Project	070181	Page	1/
Subject	Public works Department	Date	Dec. 23, 2009	By	AT
		Checked	8/2/10	By	RDS
		Approved		By	

SPILLWAY CHANNEL HYDRAULIC CALCULATIONS

REF:

1. Open Channel Hydraulics, Ven Te Chaw
2. USBR Design of Small Dams

PPER D CH BASII M

SPILLWAY DISCHARGE CHANNEL HYDRAULICS

July 02/10

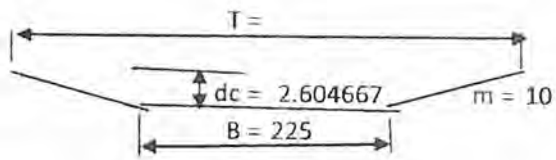
CHECK. RDS, 8/2/10

Q =	5700 cfs
Width	50 ft
m	4
n	0.03

Station	Distance	Invert	Slope	y	Base	m	Area	Perimeter	r	T	A/T	Q	v	Sf	Sf-ave	EGL	EGL-hv-A/T	Froude#	Water El.	Bank El.
	0																			192
0		184.50		6.39	25.00		482.37	102.66	4.70	101.08	4.77	0	0.00	0		189.27	0.00	0.00	190.89	192
	25		-0.01111			4									3.52E-05					
25		184.22		6.80	28.13		524.62	106.05	4.95	104.37	5.03	633.3	1.21	7.05E-05		189.27	0.00	0.09	191.02	192
	25		-0.01111			4									0.000151					
50		183.94		7.15	31.25		562.35	108.99	5.16	107.23	5.24	1266.7	2.25	0.000232		189.27	0.00	0.17	191.10	192
	25		-0.01111			4									0.000337					
75		183.67		7.47	34.38		596.53	111.59	5.35	109.75	5.44	1900.0	3.19	0.000442		189.26	0.00	0.24	191.14	192
	25		-0.01111			4									0.000563					
100		183.39		7.75	37.50		627.43	113.88	5.51	111.98	5.60	2533.3	4.04	0.000683		189.25	0.00	0.30	191.14	192
	25		0.01111			4									0.000814					
125		183.11		7.99	40.63		655.40	115.93	5.65	113.96	5.75	3166.7	4.83	0.000945		189.22	0.00	0.36	191.11	192
	25		-0.01111			4									0.001086					
150		182.83		8.21	43.75		680.20	117.71	5.78	115.69	5.88	3800.0	5.59	0.001227		189.20	0.00	0.41	191.04	192
	25		-0.01111			4									0.00138					
175		182.56		8.39	46.88		701.22	119.20	5.88	117.13	5.99	4433.3	6.32	0.001534		189.16	0.00	0.46	190.95	192
	25		-0.01111			4									0.001707					
200		182.28		8.53	50.00		717.56	120.34	5.96	118.24	6.07	5066.7	7.06	0.001879		189.12	0.00	0.51	190.81	192
	25		-0.01111			4									0.004948					
225		182.00		6.21	50.00		465.09	101.24	4.59	99.71	4.66	5700.0	12.26	0.008016		189.00	0.00	1.00	188.21	192
	25		-0.01323			4									0.008694					
250		181.67		5.96	50.00		440.08	99.15	4.44	97.68	4.51	5700.0	12.95	0.009373		188.78	0.00	1.08	187.63	191.1
	50		-0.01323			4									0.010184					
300		181.01		5.71	50.00		416.02	97.09	4.28	95.69	4.35	5700.0	13.70	0.010995		188.27	0.00	1.16	186.72	189.3
	50		-0.01323			4									0.011441					
350		180.35		5.59	50.00		404.74	96.12	4.21	94.74	4.27	5700.0	14.08	0.011888		187.70	0.00	1.20	185.94	187.5
	50		-0.01323			4									0.012152					
400		179.68		5.53	50.00		398.62	95.58	4.17	94.22	4.23	5700.0	14.30	0.012415		187.09	0.00	1.23	185.21	185.7
	50		-0.01323			4									0.012574					
450		179.02		5.49	50.00		395.09	95.27	4.15	93.92	4.21	5700.0	14.43	0.012733		186.46	0.00	1.24	184.51	185.02
	50		-0.01323			4									0.01283					
500		178.36		5.47	50.00		393.00	95.09	4.13	93.74	4.19	5700.0	14.50	0.012927		185.82	0.00	1.25	183.83	184.36
	50		-0.01323			4									0.012986					
550		177.70		5.45	50.00		391.75	94.98	4.12	93.64	4.18	5700.0	14.55	0.013045		185.17	0.00	1.25	183.15	183.70
	50		-0.02			4									0.014363					
600		176.70		5.19	50.00		367.28	92.80	3.96	91.52	4.01	5700.0	15.57	0.015681		184.45	0.00	1.37	181.89	182.70
	50		-0.045			4									0.021086					
650		174.45		4.50	50.00		305.97	87.10	3.51	86.00	3.56	5700.0	18.63	0.02649		183.40	0.00	1.74	178.95	180.45
	50		-0.065			4									0.033787					
700		171.20		3.99	50.00		262.94	82.88	3.17	81.90	3.21	5700.0	21.68	0.041084		181.71	0.00	2.13	175.19	177.20
	50		-0.09			4									0.0509					
750		166.70		3.58	50.00		230.00	79.49	2.89	78.61	2.93	5700.0	24.78	0.060716		179.16	0.00	2.55	170.28	172.70
	50		0.09			4									0.06689					
800		162.20		3.40	50.00		215.93	78.00	2.77	77.17	2.80	5700.0	26.40	0.073065		175.82	0.00	2.78	165.60	168.20
	50		0.08			4									0.074559					
850		158.20		3.36	50.00		213.01	77.69	2.74	76.86	2.77	5700.0	26.76	0.076053		172.09	0.00	2.83	161.56	164.20
	50		-0.06			4									0.072613					
900		155.20		3.45	50.00		220.00	78.44	2.80	77.59	2.84	5700.0	25.91	0.069173		168.46	0.00	2.71	158.65	161.20
	50		-0.004			4									0.055888					
950		155.00		3.95	50.00		259.68	82.55	3.15	81.58	3.18	5700.0	21.95	0.042603		165.67	0.00	2.17	158.95	161.00

FLOW OVER THE SPILLWAY WEIR AT ZERO FREEBOARD RESERVOIR LEVEL

Q = 5700
m = 10/1 10
B = 225
T = B+2*dc*m
A = (B+dc*m)*dc

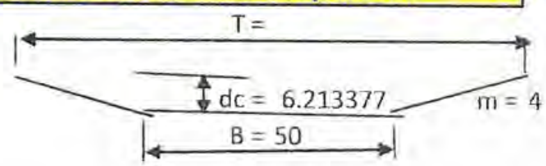


Q^2/g = A^3/T

(1) Q	(2) dc	(3) (Q)^2/g	(4) A	(5) A^3	(6) T	(7) A^3/T	(8) (3)-(7)=0	(9) V	(10) F
5700	2.60	1009006	653.89	3.E+08	277.09	1009006	0	8.72	1.00

CRITICAL DEPTH AT THE END OF SPILLWAY SIDE CHANNEL (Sta.2+25

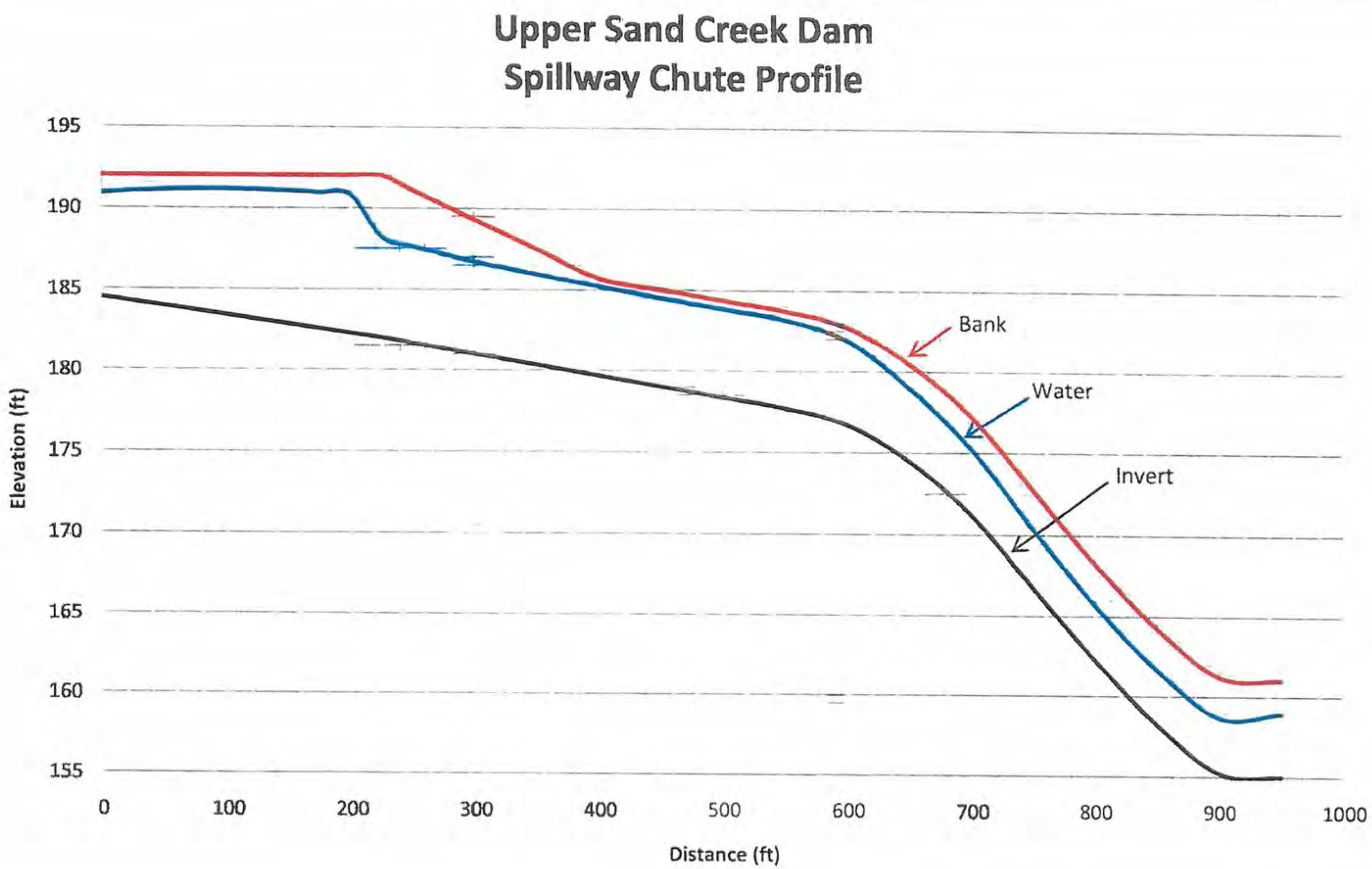
Q = 5700
m = 4/1 4
B = 50
T = B+2*dc*m
A = (B+dc*m)*dc



Q^2/g = A^3/T

(1) Q	(2) dc	(3) (Q)^2/g	(4) A	(5) A^3	(6) T	(7) A^3/T	(8) (3)-(7)=0	(9) V	(10) F
5700	6.21	1009006	465.09	1.E+08	99.71	1009006	0	12.26	1.00

	Station	Invert	Water	Bank
11	0	184.50	190.89	192.00
13	25	184.22	191.02	192.00
15	50	183.94	191.10	192.00
17	75	183.67	191.14	192.00
19	100	183.39	191.14	192.00
21	125	183.11	191.11	192.00
23	150	182.83	191.04	192.00
25	175	182.56	190.95	192.00
27	200	182.28	190.81	192.00
29	225	182.00	188.21	192.00
31	250	181.67	187.63	191.10
33	300	181.01	186.72	189.30
35	350	180.35	185.94	187.50
37	400	179.68	185.21	185.70
39	450	179.02	184.51	185.02
41	500	178.36	183.83	184.36
43	550	177.70	183.15	183.70
45	600	176.70	181.89	182.70
47	650	174.45	178.95	180.45
49	700	171.20	175.19	177.20
51	750	166.70	170.28	172.70
53	800	162.20	165.60	168.20
55	850	158.20	161.56	164.20
57	900	155.20	158.65	161.20
59	950	155.00	158.95	161.00



CHECK: RDS 8/2/10

UPPER AND CREEL BASIN DAM
SPILLWAY DISCHARGE CHANNEL HYDRAULICS

July 06/10

Q = 3000 cfs
Widtth 50 ft
m 4
n 0.03

Station	Distance	Invert	Slope	y	Base	m	Area	Perimeter	r	T	A/T	Q	v	Sf	Sf-ave	EGL	EGL-hv-A/T	Froude#	Water El.	Bank El.
0	0	184.50		3.64	25.00		235.30	80.05	2.94	79.15	2.97	0	0.00	0		187.47	0.00	0.00	188.14	192 192
	25		-0.01111			4									6.9E-05					
25		184.22		4.01	28.13		264.59	83.04	3.19	82.06	3.22	333.3	1.26	0.000138		187.47	0.00	0.12	188.23	192
	25		-0.01111			4									0.000279					
50		183.94		4.32	31.25		290.68	85.63	3.39	84.56	3.44	666.7	2.29	0.00042		187.46	0.00	0.22	188.26	192
	25		-0.01111			4									0.000586					
75		183.67		4.60	34.38		314.83	87.95	3.58	86.82	3.63	1000.0	3.18	0.000751		187.45	0.00	0.29	188.27	192
	25		-0.01111			4									0.000923					
100		183.39		4.86	37.50		337.24	90.05	3.74	88.86	3.80	1333.3	3.95	0.001095		187.43	0.00	0.36	188.25	192
	25		-0.01111			4									0.001268					
125		183.11		5.09	40.63		358.10	91.97	3.89	90.72	3.95	1666.7	4.65	0.001441		187.39	0.00	0.41	188.20	192
	25		-0.01111			4									0.001612					
150		182.83		5.30	43.75		377.60	93.73	4.03	92.42	4.09	2000.0	5.30	0.001784		187.35	0.00	0.46	188.14	192
	25		-0.01111			4									0.001955					
175		182.56		5.50	46.88		395.63	95.32	4.15	93.97	4.21	2333.3	5.90	0.002126		187.31	0.00	0.51	188.05	192
	25		-0.01111			4									0.0023					
200		182.28		5.67	50.00		411.93	96.74	4.26	95.35	4.32	2666.7	6.47	0.002475		187.25	0.00	0.55	187.95	192
	25		-0.01111			4									0.005672					
225		182.00		4.27	50.00		286.54	85.22	3.36	84.17	3.40	3000.0	10.47	0.00887		187.11	0.00	1.00	186.27	192
	25		-0.01323			4									0.004927					
250		181.67		7.69	50.00		621.32	113.43	5.48	111.54	5.57	3000.0	4.83	0.000984		186.98	0.62	0.36	189.36	191.1
	50		-0.01323			4									0.000764					
300		181.01		8.96	50.00		768.79	123.86	6.21	121.66	6.32	3000.0	3.90	0.000544		186.95	0.62	0.27	189.96	189.3
	50		-0.01323			4									0.000435					
350		180.35		10.19	50.00		924.86	134.03	6.90	131.52	7.03	3000.0	3.24	0.000326		186.92	0.62	0.22	190.54	187.5
	50		-0.01323			4									0.000267					
400		179.68		11.41	50.00		1091.87	144.13	7.58	141.31	7.73	3000.0	2.75	0.000207		186.91	0.62	0.17	191.10	185.7
	50		-0.01323			4									0.000172					
450		179.02		12.64	50.00		1270.83	154.22	8.24	151.11	8.41	3000.0	2.36	0.000136		186.90	0.62	0.14	191.66	185.02
	50		-0.01323			4									0.000115					
500		178.36		13.87	50.00		1462.56	164.35	8.90	160.94	9.09	3000.0	2.05	9.3E-05		186.90	0.62	0.12	192.23	184.36
	50		-0.01323			4									7.91E-05					
550		177.70		15.10	50.00		1666.91	174.51	9.55	170.79	9.76	3000.0	1.80	6.51E-05		186.89	0.62	0.10	192.80	183.70
	50		-0.02			4									5.24E-05					
600		176.70		16.98	50.00		2001.75	190.00	10.54	185.82	10.77	3000.0	1.50	3.96E-05		186.89	0.62	0.08	193.68	182.70
	50		-0.045			4									0.049574					
650		174.45		2.16	50.00		126.78	67.83	1.87	67.29	1.88	3000.0	23.66	0.099107		184.41	0.62	3.04	176.61	180.45
	50		-0.065			4									0.088423					
700		171.20		2.32	50.00		137.40	69.12	1.99	68.54	2.00	3000.0	21.83	0.077738		179.99	0.62	2.72	173.52	177.20
	50		-0.09			4									0.081526					
750		166.70		2.26	50.00		133.23	68.61	1.94	68.06	1.96	3000.0	22.52	0.085314		175.91	0.62		168.96	172.70
	50		-0.09			4									0.086778					
800		162.20		2.24	50.00		131.75	68.43	1.93	67.88	1.94	3000.0	22.77	0.088242		171.57	0.62	2.88	164.44	168.20
	50		-0.08			4									0.085672					
850		158.20		2.27	50.00		134.39	68.75	1.95	68.19	1.97	3000.0	22.32	0.083101		167.29	0.62	2.80	160.47	164.20
	50		-0.06			4									0.076057					
900		155.20		2.40	50.00		142.94	69.78	2.05	69.19	2.07	3000.0	20.99	0.069012		163.49	0.62	2.57	157.60	161.20
	50		-0.004			4									0.050452					
950		155.00		2.99	50.00		185.10	74.64	2.48	73.90	2.50	3000.0	16.21	0.031892		160.97	0.62	1.80	157.99	161.00

CHECK: RDS 8/2/1

GEI Consultants

Client	Contra Costa County	Project	070181	Page	1/
Subject	Public works Department	Date	Dec. 23, 2009	By	AT
		Checked	8/2/10	By	RDS
		Approved		By	

TAILWATER RATING CURVE CALCULATIONS

REF:

1. 1/40 Scale topo map
2. Open Channel Hydraulics, Ven Te Chaw
3. USBR Design of Small Damsw

UPPER SAND CREEK BASIN

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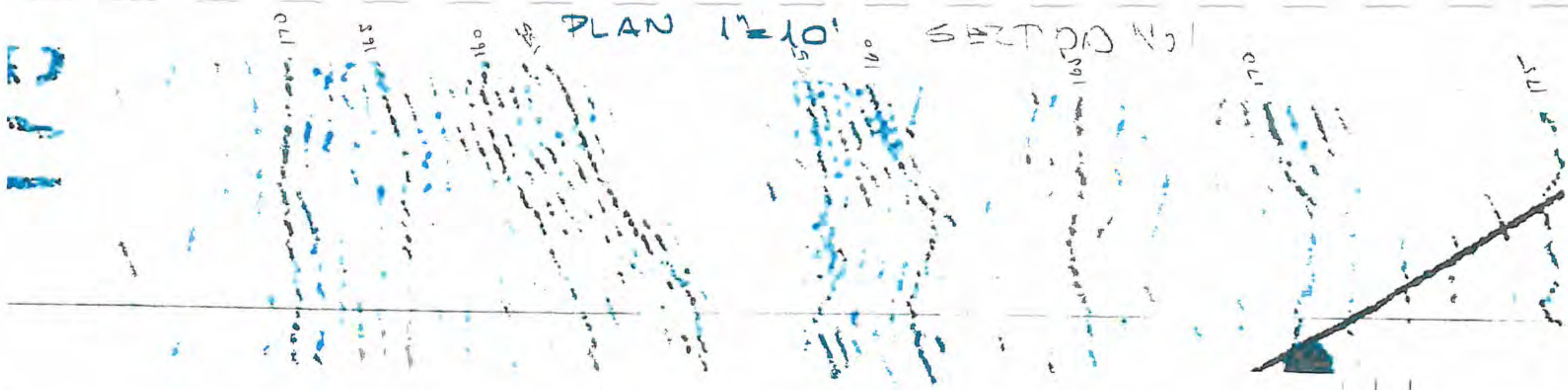
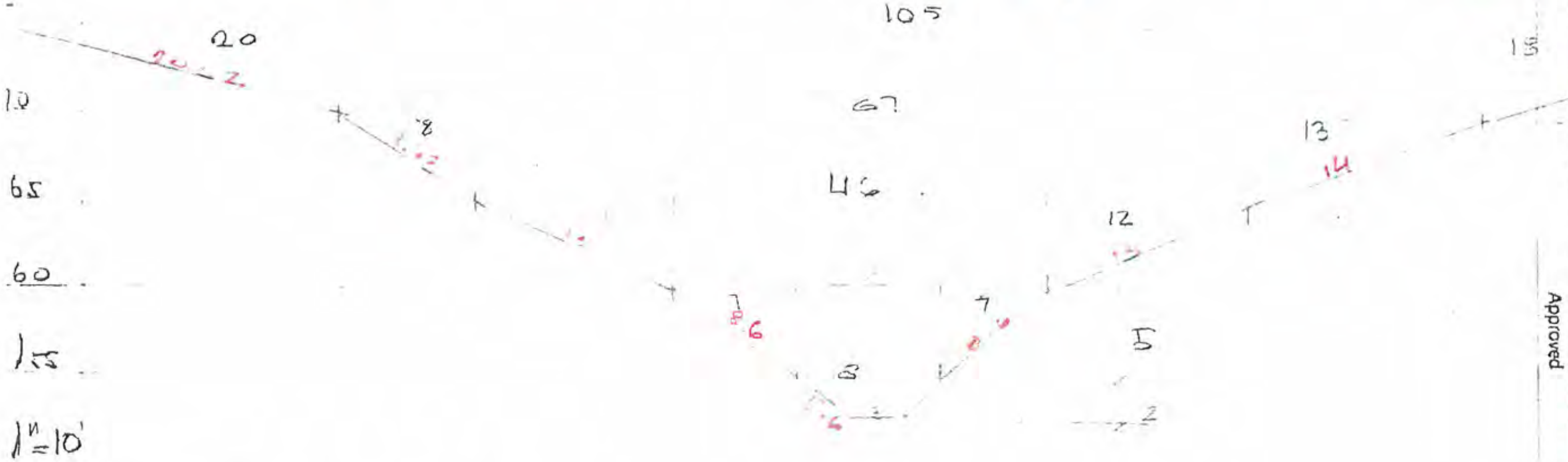
PLAN 1" = 10'

SECTION No 1





$\frac{1}{2}(11+8) \times 2 = 19$

$(22+8) \times 2.5 = 75$

$(22+46)$ 170

$(46+67)\ 2.5 = 282.5$

$(67+105)$ 430

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By Date
Checked RDS Date 8/2/10
Approved Date

UPPER SAND CREEK SPILLWAY

TAILWATER CURVE No 1

n = 0.05 s = 0.571%

Elevation	Width	Av. Width	Height	Area	Cu.Area.	L. slope	R. slope	p	r	Q
153	3				0	0	0			0
		5.5	2	11						
155	8				11	5.6	5.6	14.2	0.774648	21
		15	5	75						
160	22				86	8.6	8.6	31.4	2.738854	378
		34	5	170						
165	46				256	13	13	57.4	4.45993	1,558
		56.5	5	282.5						
170	67				538.5	9.4	14	80.8	6.664604	4,282
		86	5	430						
175	105				968.5	20.6	18.6	120	8.070833	8,750

Elev	Q
153	0
155	21
160	378
165	1,558
170	4,282
175	8,750

JPPER SAND CREEK SLOPE
From 1" = 40' scale topo map

Stretch		Distance	Slope
Point	El.	ft	%
*Start	155	0	
1	153	340	0.588%
2	151	720	0.526%
3	150	860	0.714%
4	148	1300	0.455%
*pillway outlet		Average	0.571%

CHECK · RDS, 8/7/

UPPER SAND CREEK SPILLWAY

TAILWATER CURVE No 2

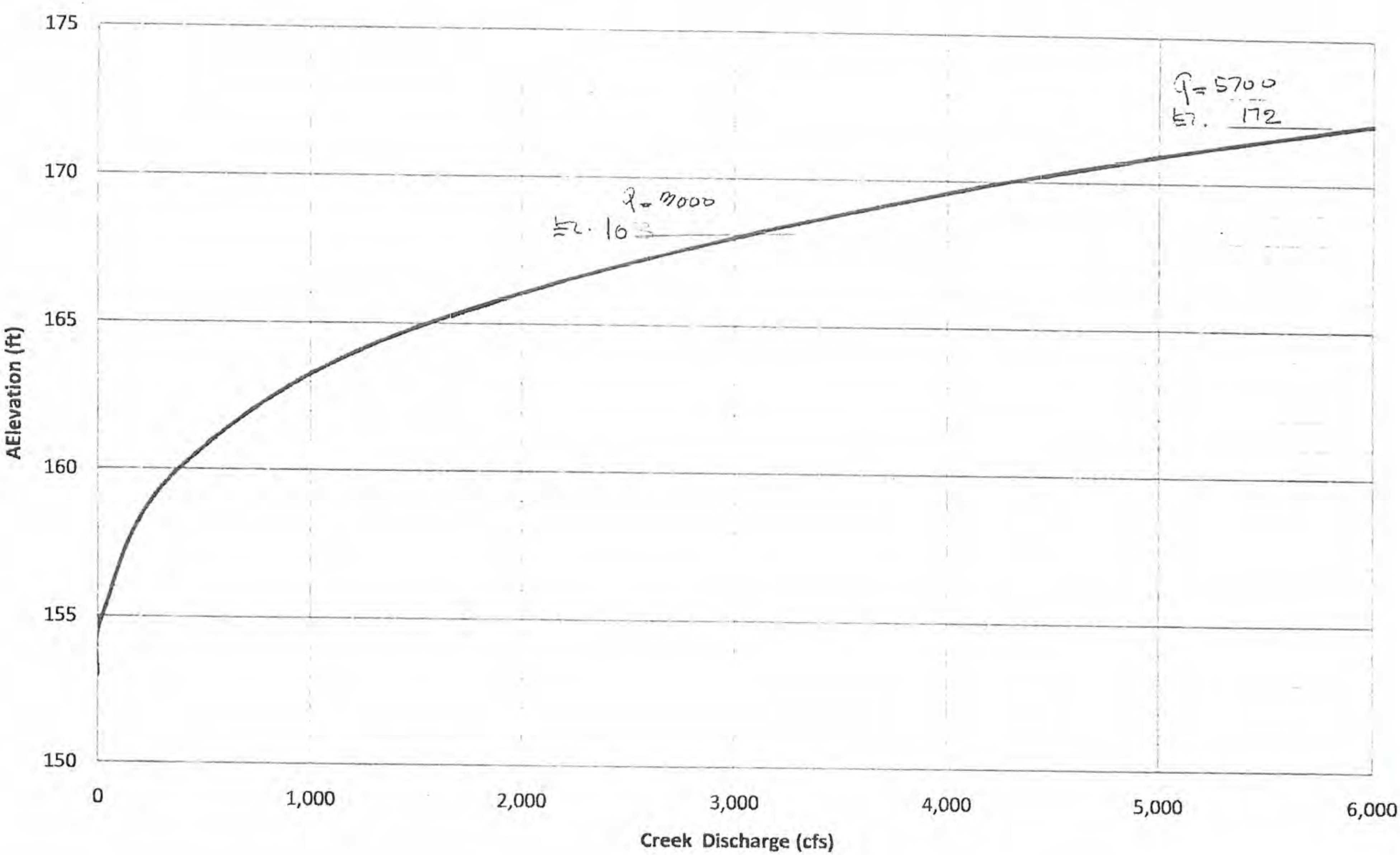
n = 0.05 s = 0.571%

Elevation	Width	Av. Width	Height	Area	Cu.Area.	L. slope	R. slope	p	r	Q
153	4				0	0	0	0		0
		7	2	14						
155	10				14	4	3	11	1.272727	37
		18	5	90						
160	26				104	9	9	29	3.586207	547
		36	5	180						
165	46				284	11	13	53	5.358491	1,953
		56	5	280						
170	66				564	8	15	76	7.421053	4,818
		84	5	420						
175	102				984	16	20	112	8.785714	9,408

Elev	Q	Area
153	0	0
155	37	14
160	547	104
165	1,953	284
170	4,818	564
175	9,408	984

CHECK: RDS, 8/2/10

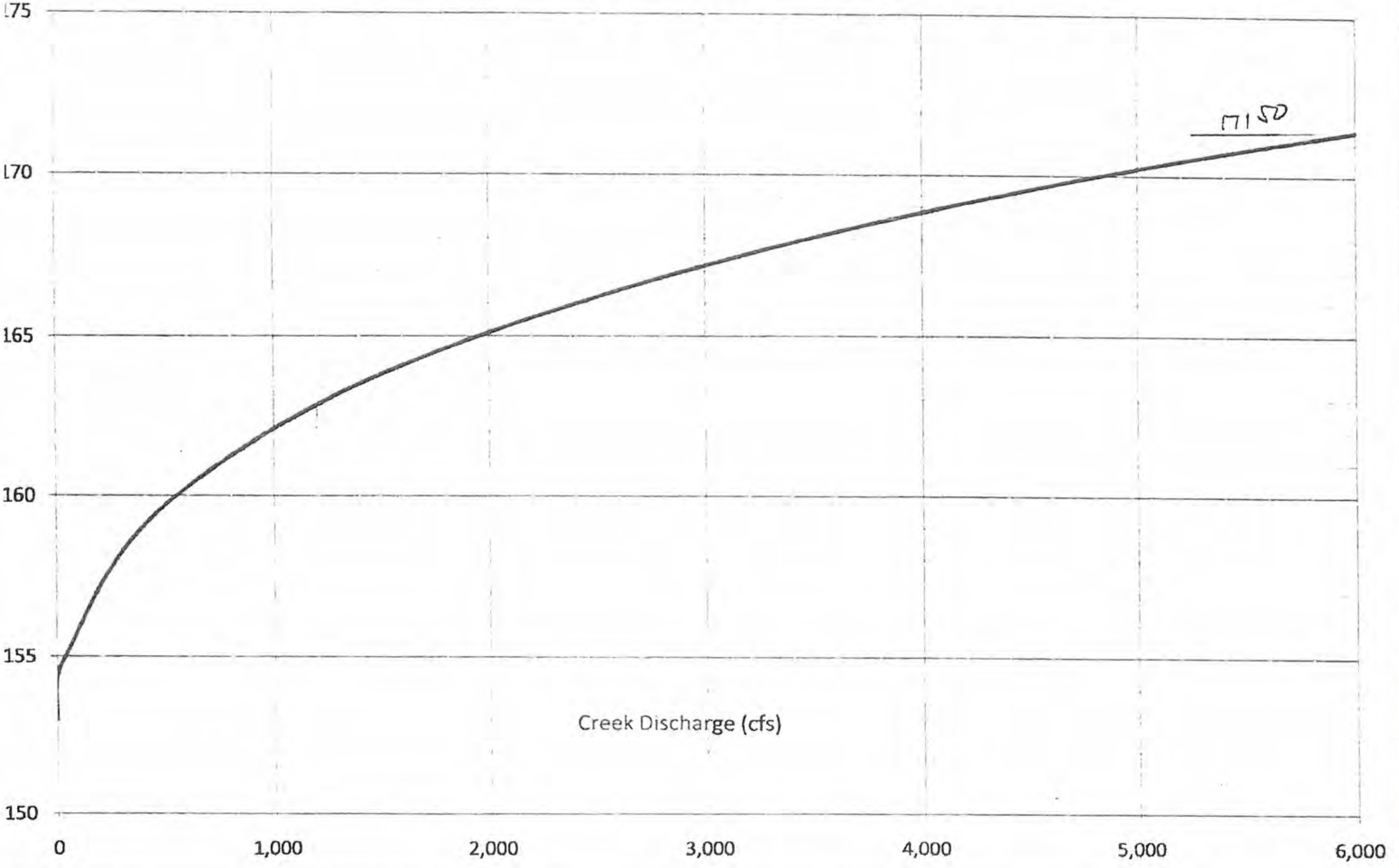
Upper Sand Creek Tailwater Rating Curve No.1



CHECK! RDS 8/2/10

CHECK: RDS 8/2/10

Upper Sand Creek Tailwater Rating Curve No.2



MANNING'S "N" VALUES

THE PURPOSE OF THIS ANALYSIS IS TO ESTIMATE THE MANNING'S "N" VALUES FOR EXISTING CONDITIONS IN SAND CREEK UPSTREAM OF THE RT 4 BYPASS. THESE VALUES WERE ESTIMATED BASED UPON THE STANDARD TABULATED VALUES, AS PRESENTED IN THE HEC-RAS MANUAL OR V.T. CHOW, AND REFLECT FIELD CONDITIONS OBSERVED DURING A RECONNAISSANCE CONDUCTED ON 6/19/01.

MANNING'S "N" VALUES WERE PREVIOUSLY ESTIMATED ON 12/1/00 FOR SAND CREEK DOWNSTREAM OF THE RT 4 BYPASS. AT THAT TIME, MANNING'S "N" WAS ESTIMATED TO BE 0.04 IN THE VICINITY OF THE RT 4 BRIDGE, BETWEEN SECTIONS 11557 AND 11710 (SEE COMPUTATIONS DATED 12/5/00). THIS LOW VALUE OF MANNING'S "N" WAS PROBABLY A RESULT OF CLEARING ACTIVITIES PERFORMED IN CONNECTION WITH THE BRIDGE CONSTRUCTION. AS OF 6/19/01, A SIGNIFICANT AMOUNT OF VEGETATION HAS ESTABLISHED ITSELF IN THE VICINITY OF THE RT 4 BRIDGE. THEREFORE, I RECOMMEND USING A MANNING "N" VALUE OF 0.06 BETWEEN SECTIONS 11460 AND 11710

([illegible] SECTIONS 11460 AND 11557)

FOR SAND CREEK UPSTREAM OF THE RT. 4 BYPASS, THE LEFT AND RIGHT OVERBANK CONDITIONS ARE CULTIVATED FIELDS W/ NO CROPS. N = 0.03.
THE MANNINGS "N" VALUES FOR THE MAIN CHANNEL ARE AS FOLLOWS:

	FROM SECTION	THRU SECTION	MANNING "N"
1.	11810	11910	0.08
2.	12010	12410	0.06
3.	12510	12692	0.08
4.	12742	14140	0.07
5	14240	14493	0.08
6.	14533	16280	0.07
7.	16380	16550	0.08
8	16650	17650	0.07
9.	17750	17840	0.06
10	17930	18320	0.07
11.	18420	19120	0.05
12	19220	19320	0.07
13	19420	19790	0.06
14.	19880	19940	0.07

19120
18320
800

(19)

(20)

(21)

Fig. 5-5 (19-21)

19. $n = 0.050$. Dredge channel with very irregular side slopes and bottom, in dark-colored waxy clay, with growth of weeds and grass. Slight variation in shape of cross section for variation in size.

20. $n = 0.060$. Ditch in heavy silty clay; irregular side slopes and bottom; practically entire section filled with large-size growth of trees, principally willows and cottonwoods. Quite uniform cross section.

21. $n = 0.080$. Dredge channel in black slippery clay and gray silty clay loam, irregular wide slopes and bottom, covered with dense growth of bushy willows, some in bottom; remainder of both slopes covered with weeds and a scattering growth of willows and poplars, no foliage; some silting on bottom.

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Subject	Public works Department	Date	Dec. 23, 2009	By	AT
		Checked	8/2/10	By	RDS
		Approved		By	

SPILLWAY CHANNEL RSP DESIGN

REF:

1. Open Channel Hydraulics, Ven Te Chaw
2. USBR Design of Small Damsw
3. California Bank ans Shore Rock Slope Protection Design

GEI Consultants

		Project	083180	Page	
Client	UPPER SAND CREEK BASIN	Date		By	[signature]
Subject	SPILLWAY CHANNEL END	Checked	8/2/10	By	RDS
	SCOUR PROTECTION	Approved		By	

$$W = \frac{0.00002\ V^6\ SG}{(SG-1)^3\ \sin(r-a)}$$

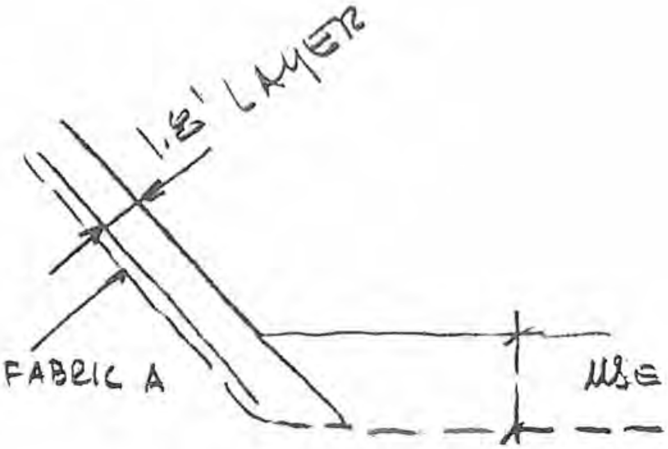
$a = \arctan 1/4 = 14.04 \rightarrow r-a = 56°$

$$= \frac{0.00002 \times (12)^6 \times 0.65}{(1.65)^3 \times \sin 56} = 42.49 \text{ lb.}$$

FROM TABLE 5-1 75 lb 50-100 BACKING 1B

FROM TABLE 5-2 RSP-FABRIC A

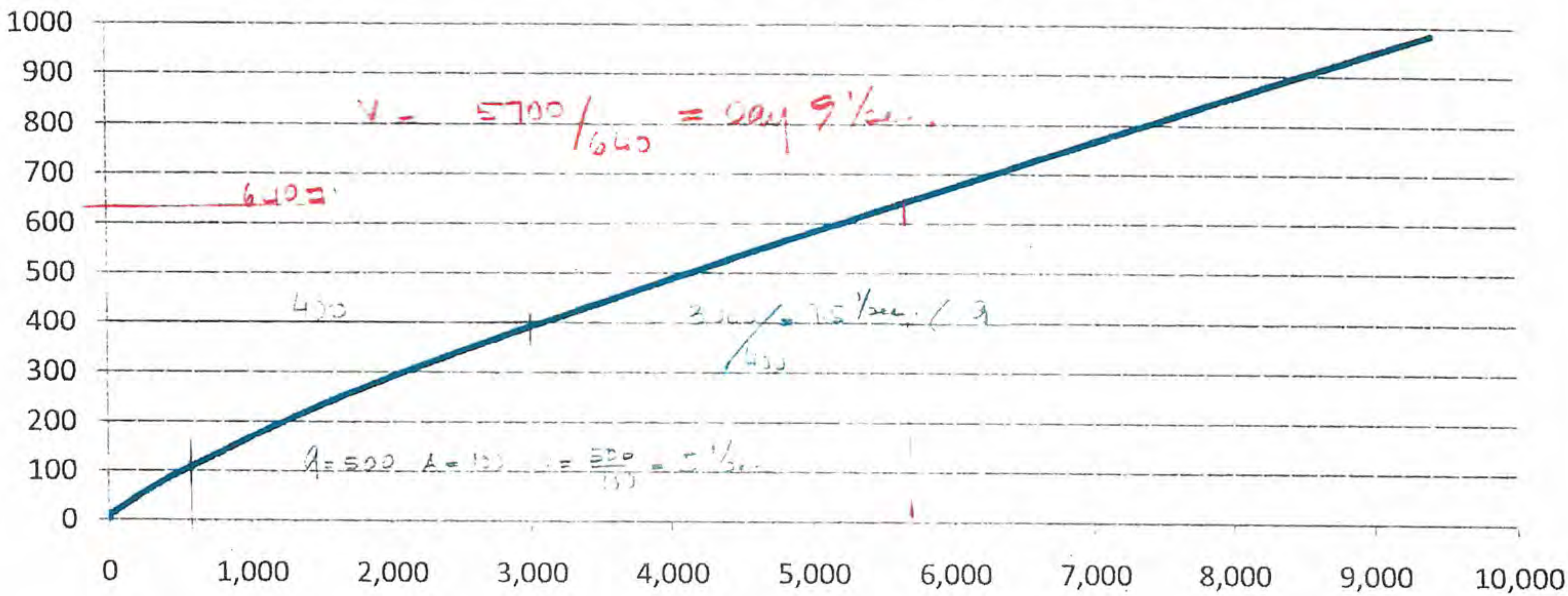
FROM TABLE 5-3 MIN. LAYER 1.8'



RSP-FABRIC A

use 3' Thick Rock.

Upper Sand Creek Flow/Area Curve+Sheet1!



CHECK: 8/2/10, KD.

STANDARD Rock SIZE or Rock MASS or Rock WEIGHT		GRADING OF ROCK SLOPE PROTECTION **PERCENTAGE LARGER THAN**											
		RSP-Classes [A]											
		Method A Placement					Method B Placement						
		RSP-Classes other than Backing									Backing No.		
US unit		8 ton	4 ton	2 ton	1 ton	1/2 ton	1 ton	1/2 ton	1/4 ton	Light	1 [B]	2	3
	SI unit	8 T	4 T	2 T	1 T	1/2 T	1 T	1/2 T	1/4 T	Light	1 [B]	2	3
16 ton	14.5 tonne	0-5											
8 ton	7.25 tonne	50-100	0-5										
4 ton	3.6 tonne	95-100	50-100	0-5									
2 ton	1.8 tonne		95-100	50-100	0-5		0-5						
1 ton	900 kg			95-100	50-100	0-5	50-100	0-5					
1/2 ton	450 kg				95-100	50-100	-----	50-100	0-5				
1/4 ton	220 kg					95-100	95-100	-----	50-100	0-5			
200 lb	90 kg							95-100	-----	50-100	0-5		
75 lb	34 kg								95-100	-----	50-100	0-5	
25 lb	11 kg									95-100	90-100	25-75	0-5
5 lb	2.2 kg											90-100	25-75
1 lb	0.4 kg												90-100

[A] US customary names (units) of RSP-Classes listed above SI names, example US is "2 ton" metric is "2 T".
[B] "Facing" has same gradation as "Backing No. 1". To conserve space "Facing" is not shown .

Example for determining RSP-Class of outside layer. By using Equation 1, if the calculated W=135 kg (minimum stable rock size):
1. Enter table at left and select closest value of STANDARD Rock SIZE which is greater than calculated W, in this case 220 kg
2. Trace to right and locate "50-100" entry 3. Trace upward and read column heading "1/4 T", then 1/4 T is first trial RSP-Class.

Table 5-1. Guide for Determining RSP-Class of Outside Layer

Table 5-2. California Layered RSP SI metric (US customary values shown for OUTSIDE LAYER only)			
OUTSIDE LAYER RSP-CLASS *	INNER LAYERS RSP-CLASS *	BACKING CLASS No. *	RSP-FABRIC TYPE **
8 T (8 ton)	2 T over 1/2 T	1	B
8 T (8 ton)	1 T over 1/4 T	1 or 2	B
4 T (4 ton)	1/2 T	1	B
4 T (4 ton)	1 T over 1/4 T	1 or 2	B
2 T (2 ton)	1/2 T	1	B
2 T (2 ton)	1/4 T	1 or 2	B
1 T (1 ton)	LIGHT	NONE	B
1 T (1 ton)	1/4 T	1 or 2	B
1/2 T (1/2 ton)	NONE	1	B
1/4 T (1/4 ton)	NONE	1 or 2	A
LIGHT (LIGHT)	NONE	NONE	A
Backing No. 1*** (Backing No. 1)	NONE	NONE	A

* Rock grading and quality requirements per Section 72-2.02 Materials of the Caltrans *Standard Specifications*. (See Appendix B).

** RSP-fabric Type of geotextile and quality requirements per Section 88-1.04 Rock Slope Protection Fabric of the Caltrans *Standard Specifications*. (See Appendix B). Type A RSP-fabric has lighter mass per unit area and it also has lower toughness (tensile x elongation, both at break) than Type B RSP-fabric. Both types require minimum permittivity of 0.5 per second.

*** "Facing" RSP-Class has same gradation as Backing No. 1.

Material property values were selected for the RSP-fabric in Section 88-1.04 of the Caltrans *Standard Specifications*, by assuming that construction inspectors will limit the maximum height of rockfall during placement to about 1 meter. End dumping of rock down embankments is not recommended, because rocks will damage and dislodge the RSP-fabric and the rock sizes will segregate.

Table 5-3. Minimum Layer Thickness SI metric (US customary)		
RSP-Class Layer	Method of Placement	Minimum Thickness
8 T (8 ton)	A	2.60 meters (8.5 feet)
4 T (4 ton)	A	2.07 meters (6.8 feet)
2 T (2 ton)	A	1.65 meters (5.4 feet)
1 T (1 ton)	A	1.31 meters (4.3 feet)
1/2 T (1/2 ton)	A	1.04 meters (3.4 feet)
1 T (1 ton)	B	1.65 meters (5.4 feet)
1/2 T (1/2 ton)	B	1.31 meters (4.3 feet)
1/4 T (1/4 ton)	B	1.00 meters (3.3 feet)
Light	B	760 millimeters (2.5 feet)
Facing	B	550 millimeters (1.8 feet)
Backing No. 1	B	550 millimeters (1.8 feet)
Backing No. 2	B	380 millimeters (1.25 feet)
Backing No. 3	B	230 millimeters (0.75 feet)

For total thickness, add each layer thickness. Use zero thickness for the RSP-fabric. Before adopting values in Table 5-3, consult with a materials engineer about rock sources, quality, shapes, and specific gravity. Calculate new thickness values if the shape factor is not spherical and specific gravity is not reasonably close to 2.65. "Minimum Thickness" values were calculated by starting with US customary units, hard-converting to a value in feet, then soft-converting to SI metric values.

5-1-G. Review Hydraulic Calculations at Site With RSP and Possibility of Vegetation. This step of the layered design process is required to help assure future success of the revetment under changed channel dimensions, roughness coefficients, and other permit/agreement requirements. Examples are: filling voids among RSP with soil and/or covering RSP with soil then planting local species, and/or enhancing fish habitat by placing large-sized rock along the toe. Discuss site hydraulics with people of permit agencies and feasible revegetation efforts. Historically, sites with no prior vegetation are usually not revegetated, especially when subjected to scouring velocities or high wave attack.

Appendix 3.2

Outlet Works Hydraulics

Outlet Works Design

1. Introduction

Sand Creek Dam Outlet Works will include an unregulated, orifice controlled, low level inlet; a gated shaft inlet; a 60-inch reinforced-concrete outlet works conduit; and a reinforced-concrete impact type stilling basin.

Incoming creek flows will normally pass through the low level inlet, set at the creek invert elevation of 157.0. The low level inlet will be capable of passing flows up to approximately 134 cubic feet per second (cfs) at the maximum pool level.

The gated shaft inlet was designed as an emergency provision to increase the discharge capacity up to about 500-cfs during flood events; however it was intended to remain closed under normal circumstances. The inlet rim will be at elevation 171.0, about 20-ft lower than the maximum pool level.

2. Outlet Works Structures

2.1. Low Level Inlet

The low level inlet will be a simple reinforced concrete structure, consisting of an invert slab, headwall, two vertical side walls, and a trashrack. It will serve to direct incoming creek flows from the low flow channel into the outlet works conduit. A steel trashrack will be mounted on the front face of the inlet structure to prevent large floating debris from entering the outlet conduit. A stainless steel orifice plate will be installed on the upstream face of the headwall to limit uncontrolled flows to a maximum of approximately 134-cfs. The orifice opening will be a 26-inch by 26-inch square, approximately 4.7 square feet, with an invert elevation of 157.0.

2.2. Secondary Inlet

A secondary inlet was designed to provide additional reservoir discharge capacity for emergency service, however it was intended to be closed during normal reservoir operations. The secondary inlet will consist of a 5-ft x 5-ft square drop inlet connected to the 60-inch diameter reinforced concrete outlet conduit at approximate outlet works station 0+78. Under maximum water surface conditions, the secondary inlet will increase the outlet works discharge capacity to approximately 500-cfs.

The inlet will be regulated with a sluice gate, mounted onto the face of the inlet and operated from the crest of the dam by a manual crank actuator. A rack will also be installed over the secondary inlet for safety purposes.

2.3. Outlet Conduit

The outlet works conduit will be a concrete encased reinforced concrete pipe with a 5-foot internal diameter. The conduit will be approximately 350-feet long, with invert level dropping 1.5-feet from El. 157.00 to El. 155.50. Access into the conduit will be provided at the secondary inlet for maintenance purposes.

2.4. Impact Type Stilling Basin

A USBR-type impact energy dissipation structure was selected since it does not require tailwater to dissipate energy. The structure was designed for the maximum design discharge of approximately 500-cfs. While 500-cfs is about 25 percent greater than the typical upper limit for this type of structure, exposure to the peak design discharge is expected to be very infrequent and lasting a short duration. The structure will normally be operated under flows not exceeding the low level inlet capacity of 134-cfs.

3. Erosion Protection

A 2.5-foot thick layer of 200 pound nominal size riprap will be provided immediately beyond the stilling basin terminal for erosion protection in the creek. The required riprap rock size, layer thickness, and geotextile were designed based on the estimated critical depth and flow velocities under the maximum reservoir level at the end of the structure. The required rock size was selected to match a standard product in the market, chosen from State of California DOT publication "California Bank and Shore Rock Protection Design" dated Oct.2000.

References:

1. USBR Canal and Related Structes
2. USBR Design of Small Dams
3. USBR EM No. 25, Hydraulic Design of Stilling Basins and Energy Dissipators

GEI Consultants

Client	UPPER SAND CREEK BASIN	Project 083180	Page 4
		Date May 18/09	By [illegible]
Subject	BOTTOM OUTLET	Checked 8/2/10	By RDS
		Approved	By

A) DATA

CONDUIT (Fig. 11, 12, & 13)

Sta 0+00 → Sta 4+01.16

L = 401.16 − 24' − 30 + 1.67 = 348.83'

Diameter 5'

Inlet EL 157.

outlet EL 155.50

$\Delta h = 1.5'$

Conduit slope = 1.5 / 348.83 = 4.30×10^{-3}

Full Pipe Flow without restriction

Pipe Flow

he =	0.62		0.009627 *V^2
hf=	29.16*(n)^2*L/r^(4/3)*1/64.4		0.0230 *V^2
hexit=	1		0.015528 *V^2
			0.0481 *V^2
n=	0.014		
D=	5 ft		
L=	348.83 ft	Σh =	0.000125 Q^2
A=	19.63 sq-ft	H - Σh =0	
r=	1.25	Q = √H/Σh	
r^(4/3)=	1.35		

	H (ft)	Q(cfs)
	5	200.0937
	10	282.9752
	15	346.5725
	20	400.1874
El.191 - El. 155,5-5 =	30.5	494.195

CHECK PARTIAL FLOW CONDITION
(No entrance restriction)

$Q = 131,\ D = 5,\ S = 0.0043,\ n = 0.014$

$\frac{Qn}{D^{8/3} \times S^{1/2}} = \frac{131 \times 0.014}{5^{(8/3)} \times (0.0043)^{1/2}}$ (See page 3

$0.38 \longrightarrow d/D = 0.69$

Depth of flow in the pipe $= 0.69 \times 5 = 3.45'$

Area $= 0.5780 \times 5^2 = 14.45^{\square'}$

$V = 131/14.45 = 9.07$ ft/sec.

When there is 1 foot of water in pipe $d/D = 1/5 = 0.20$

$0.0406 = \frac{Qn}{4.79}$ $Q = 0.0406 \times 4.79/0.014 = 13.9$ cfs

$A = 0.1118 \times 5^2 = 2.795$

$V = 13.9/2.795 = 4.97$ ft/sec.

RESULTS

1) When $Q = 131$ cfs conduit will flow paritally full $d/D = 0.69$ $V = 9$ ft/sec.

2) At 1-foot water level velocity of flow is 5.35 ft/sec.

GEI Consultants

		Project	Page 6
Client	UPPER SAND CREEK BASIN	Date 1/22/09	By
Subject	BOTTOM OUTLET	Checked 8/2/10	By RDS
		Approved	By

HYDRAULIC COMPUTATIONS

Table B-3.—Uniform flow in circular sections flowing partly full.

d = Depth of flow.
D = Diameter of pipe.
A = Area of flow.
r = Hydraulic radius.

Q = Discharge in second-feet by Manning's formula.
n = Manning's coefficient.
s = Slope of the channel bottom and of the water surface.

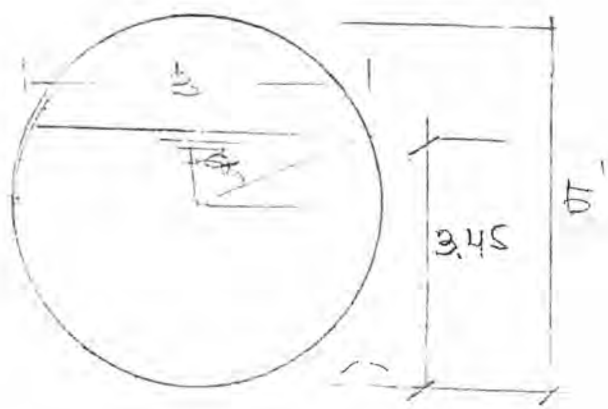
$\frac{d}{D}$	$\frac{A}{D^2}$	$\frac{r}{D}$	$\frac{Qn}{D^{8/3}s^{1/2}}$	$\frac{Qn}{d^{8/3}s^{1/2}}$	$\frac{d}{D}$	$\frac{A}{D^2}$	$\frac{r}{D}$	$\frac{Qn}{D^{8/3}s^{1/2}}$	$\frac{Qn}{d^{8/3}s^{1/2}}$
1	2	3	4	5	1	2	3	4	5
0.01	0.0013	0.0066	0.00007	15.04	0.51	0.4027	0.2531	0.239	1.442
.02	.0037	.0132	.00031	10.57	.52	.4127	.2562	.247	1.415
.03	.0069	.0197	.00074	8.56	.53	.4227	.2592	.255	1.388
.04	.0105	.0262	.00138	7.38	.54	.4327	.2621	.263	1.362
.05	.0147	.0325	.00222	6.55	.55	.4426	.2649	.271	1.336
.06	.0192	.0389	.00328	5.95	.56	.4526	.2676	.279	1.311
.07	.0242	.0451	.00455	5.47	.57	.4625	.2703	.287	1.286
.08	.0294	.0513	.00604	5.09	.58	.4724	.2728	.295	1.262
.09	.0350	.0575	.00775	4.76	.59	.4822	.2753	.303	1.238
.10	.0409	.0635	.00967	4.49	.60	.4920	.2776	.311	1.215
.11	.0470	.0695	.01181	4.25	.61	.5018	.2799	.319	1.192
.12	.0534	.0755	.01417	4.04	.62	.5115	.2821	.327	1.170
.13	.0600	.0813	.01674	3.86	.63	.5212	.2842	.335	1.148
.14	.0668	.0871	.01952	3.69	.64	.5308	.2862	.343	1.126
.15	.0739	.0929	.0225	3.54	.65	.5404	.2882	.350	1.105
.16	.0811	.0985	.0257	3.41	.66	.5499	.2900	.358	1.084
.17	.0885	.1042	.0291	3.28	.67	.5594	.2917	.366	1.064
.18	.0961	.1097	.0327	3.17	.68	.5687	.2933	.373	1.044
.19	.1039	.1152	.0365	3.06	.69	.5780	.2948	.380	1.024
.20	.1118	.1206	.0406	2.96	.70	.5872	.2962	.388	1.004
.21	.1199	.1259	.0448	2.87	.71	.5964	.2975	.395	0.985
.22	.1281	.1312	.0492	2.79	.72	.6054	.2987	.402	.965
.23	.1365	.1364	.0537	2.71	.73	.6143	.2998	.409	.947
.24	.1449	.1416	.0585	2.63	.74	.6231	.3008	.416	.928
.25	.1535	.1466	.0634	2.56	.75	.6319	.3017	.422	.910
.26	.1623	.1516	.0686	2.49	.76	.6405	.3024	.429	.891
.27	.1711	.1566	.0739	2.42	.77	.6489	.3031	.435	.873
.28	.1800	.1614	.0793	2.36	.78	.6573	.3036	.441	.856
.29	.1890	.1662	.0849	2.30	.79	.6655	.3039	.447	.838
.30	.1982	.1709	.0907	2.25	.80	.6736	.3042	.453	.821
.31	.2074	.1756	.0966	2.20	.81	.6815	.3043	.458	.804
.32	.2167	.1802	.1027	2.14	.82	.6893	.3043	.463	.787
.33	.2260	.1847	.1089	2.09	.83	.6969	.3041	.468	.770
.34	.2355	.1891	.1153	2.05	.84	.7043	.3038	.473	.753
.35	.2450	.1935	.1218	2.00	.85	.7115	.3033	.477	.736
.36	.2546	.1978	.1284	1.958	.86	.7186	.3026	.481	.720
.37	.2642	.2020	.1351	1.916	.87	.7254	.3018	.485	.703
.38	.2739	.2062	.1420	1.875	.88	.7320	.3007	.488	.687
.39	.2836	.2102	.1490	1.835	.89	.7384	.2995	.491	.670
.40	.2934	.2142	.1561	1.797	.90	.7445	.2980	.494	.654
.41	.3032	.2182	.1633	1.760	.91	.7504	.2963	.496	.637
.42	.3130	.2220	.1705	1.724	.92	.7560	.2944	.497	.621
.43	.3229	.2258	.1779	1.689	.93	.7612	.2921	.498	.604
.44	.3328	.2295	.1854	1.655	.94	.7662	.2895	.498	.588
.45	.3428	.2331	.1929	1.622	.95	.7707	.2865	.498	.571
.46	.3527	.2366	.201	1.590	.96	.7749	.2829	.496	.553
.47	.3627	.2401	.208	1.559	.97	.7785	.2787	.494	.535
.48	.3727	.2435	.216	1.530	.98	.7817	.2735	.489	.517
.49	.3827	.2468	.224	1.500	.99	.7841	.2666	.483	.496
.50	.3927	.2500	.232	1.471	1.00	.7854	.2500	.463	.463

SAND CREEK
OUTLET WORKS PIPE
Normal Operation

Length =	348.83	
Drop =	1.5	ft
Q =	131	cfs
D=	5	(ft)
n =	0.013	

slope	d/D	Coeff.	Dia.	Y	Θ (radian)	B (ft)	A (sq-ft)	P (ft)	r (ft)	Q (cfs)	v (ft/sec)	Froude
0.0043	0.10	0.00967	5	0.500	0.644	3.000	1.022	3.218	0.318	3.57	3.489	1.05
0.0043	0.20	0.0406	5	1.000	0.927	4.000	2.796	4.636	0.603	14.97	5.355	1.13
0.0043	0.30	0.0907	5	1.500	1.159	4.583	4.954	5.796	0.855	33.44	6.751	1.14
0.0043	0.40	0.1561	5	2.000	1.369	4.899	7.334	6.847	1.071	57.56	7.848	1.13
0.0043	0.50	0.252	5	2.500	1.571	5.000	9.817	7.854	1.250	92.92	9.465	1.19
0.0043	0.60	0.311	5	3.000	1.772	4.899	12.301	8.861	1.388	114.68	9.323	1.04
0.0043	0.68	0.373	5	3.38	1.931	4.680	14.125	9.653	1.463	137.54	9.737	0.99
0.0043	0.70	0.388	5	3.500	1.982	4.583	14.681	9.912	1.481	143.07	9.745	0.96
0.0043	0.80	0.453	5	4.000	2.214	4.000	16.839	11.071	1.521	167.04	9.919	0.85
0.0043	0.90	0.494	5	4.500	2.498	3.000	18.613	12.490	1.490	182.16	9.786	0.69
0.0043	1.00	0.463	5	5.000	3.142	0.000	19.635	15.708	1.250	170.72	8.695	0.77

Flow velocity at 1-foot depth = 5.36-ft/sec (supercritical stage)



CHECK RDS 8/2/10

LOW FLOWS
PRIOR TO ORIFICE CONTROL

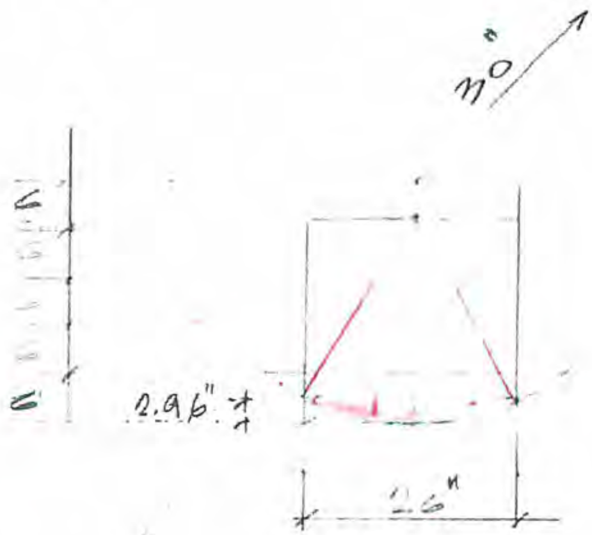
FIND THE RED MARKED AREA

$\alpha = 2 \arcsin 13/30 = 51.36°$

$A = \pi R^2 \times \frac{51.36}{360} = 2.8 \,\square'$

$- \frac{1}{2} \frac{26"}{12} \times \cos 25.68 \times 2.5 = -2.44$

Red marked Area = 0.36 □'



AREA #1

$(6"-2.96") \times 26/144 = 0.55$

AREA AT 6" — → 0.91

AREA $0.5 \times \frac{26}{12} = 1.08$

area at 1' = 1.99

Δ Area $= \frac{2.96 \times 26}{144} + 0.36\,\square' = 0.17\,\square'$

" @ 1.5' = 3.07

NET AREA OF OPENING = (4.69) − 0.17 = 4.52 □'

" 4 = 4.16

AREAS = $4.16 + \frac{2}{12} \times \frac{26}{12}$ = 4.52 □'

Incoming flow 10' wide will face an abrupt restriction and is to accelerate to pass through a restricted opening, if there is no tailwater effect flow depth will be critical depth to flow with minimum specific energy

GEI Consultants

Client	UPPER SAND CREEK	By	ASP/abar	Date	7/20/10
Subject	DISCHARGE CURVE	Checked	RDS	Date	8/2/10
		Approved		Date	

Project 083180 Page 9

Flow regimes investigated in the bottom outlet conduit indicates that at partial full conduit discharges in the conduit flow is supercritical. Therefore, there will be no backwater effect

using relation $\frac{Q^2}{g} = \frac{a^3}{T}$ find Q values

a known T known

$$Q = \sqrt{\frac{a^3 g}{T}}$$

Section at h = 1.96"	Area	T	Q
0.245	0.91	2.17	3.35
0.745	1.99	2.17	10.82
1.245	3.07	2.17	20.74
1.745	4.17	2.17	32.83
2.245	4.524	2.17	37.10

afp/abar
RDS 8/2/10

Full Pipe Flow without restriction

Pipe Flow

he =	0.62			0.009627	*V^2
hf=	29.16*(n)^2*L/r^(4/3)*1/64.4			0.0230	*V^2
hexit=	1			0.015528	*V^2
n=	0.014			0.0481	*V^2
D=	5 ft				
L=	348.83 ft		Σh =	0.000125	Q^2
A=	19.63 sq-ft		H - Σh =0		
r=	1.25		Q = √H/Σh		
r^(4/3)=	1.35				

	H (ft)	Q(cfs)
	5	200.0937
	10	282.9752
	15	346.5725
	20	400.1874
El.191 - El. 155,5-5 =	30.5	494.195

Head = Resv.-155.5-5

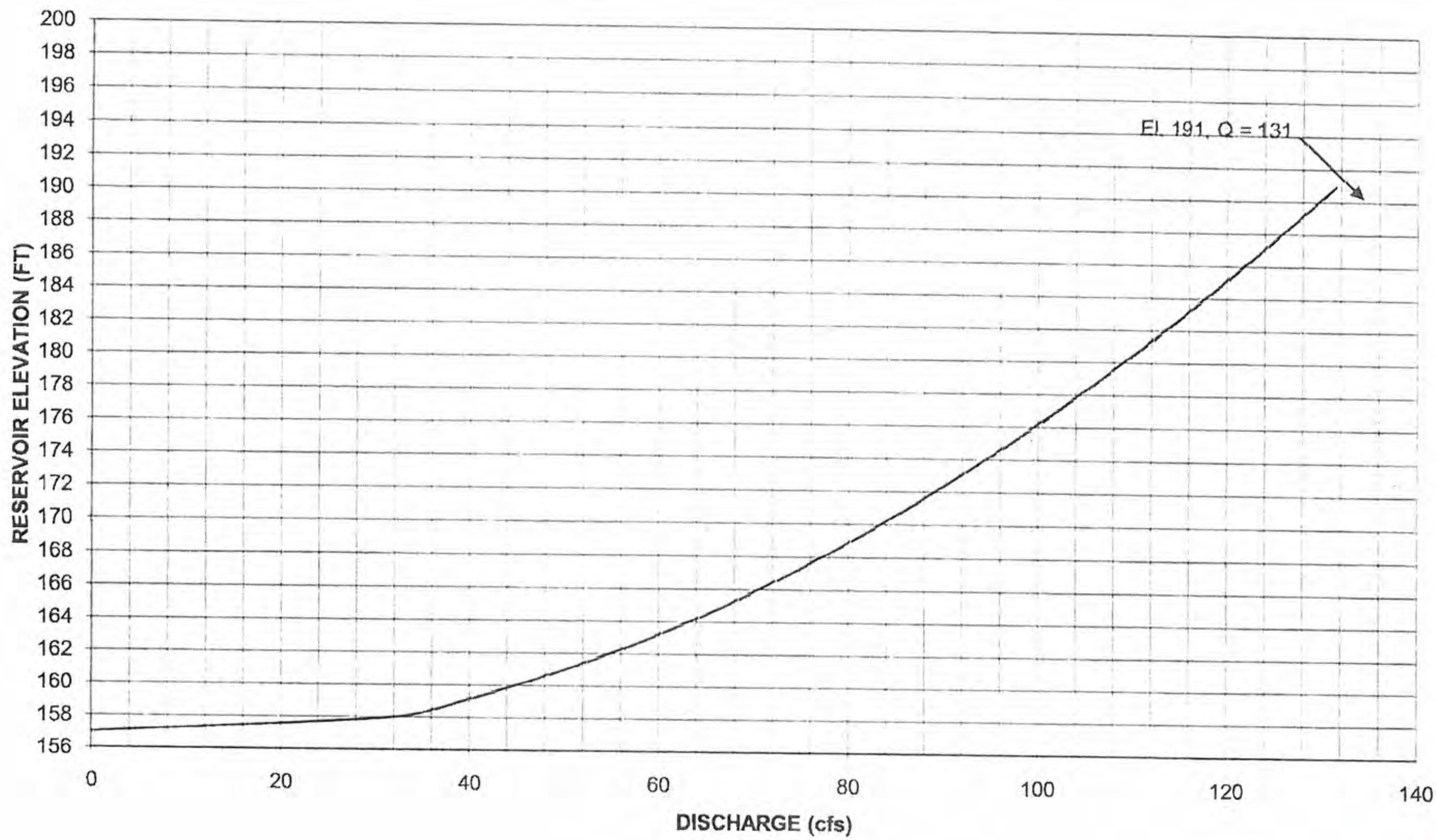
Emergency Operation Continued

LOW LEVEL OUTLET

For oricice flow	Head	C	Area	Q		Orifice flow	Head	Mid-inlt Flow	Total flow
		0.605	4.57	(cfs)		(cfs)	(ft)	(cfs)	
157	0.00			0	157				0
158	2.08			32	158				33.71
159	3.08			39	159				41.01
160	4.08			45	160	44.84	0		44.84
162	6.08			55	162	54.72	1.5		54.72
164	8.08			63	164	63.08	3.5		63.08
166	10.08			70	166	70.46	5.5		70.46
168	12.08			77	168	77.13	7.5		77.13
170	14.08			83	170	83.27	9.5		83.27
171	15.08			86	171	86.17	10.5		86.17
173	17.08			92	173	75.00	12.5	102	177.00
175	19.08			97	176		15.5	352	352.30
177	21.08			102	178		17.5	374	374.34
179	23.08			107	180		19.5	395	395.15
181	25.08			111	182		21.5	415	414.92
183	27.08			115	184		23.5	434	433.79
185	29.08			120	186		25.5	452	451.88
187	31.08			124	188		27.5	469	469.26
189	33.08			128	190		29.5	486	486.03
191	35.08			131	191		30.5	494	494.20

ONLY LOWER OUTLET
See Chart 3

OUTLET WORKS UNCONTROLLED DISCHARGE CURVE



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EMERGENCY GATE OPERATION

Depending on the water levels in the reservoir, three different hydraulic conditions are expected to occur in the emergency outlet system.

1. Up to water El. 171, no water will enter from the mid-inlet. Accordingly, orifice controlled discharge will take place in the conduit as for the normal operation case. The excel sheet on page 7 indicate that the conduit can discharge over 131-cfs without sealing the conduit.
2. As the water level in the reservoir rises about mid-inlet level; water will start entering the shaft. If the conduit downstream of the shaft is not sealed, partial pipe flow will continue up to about 182-cfs.
3. When the conduit is seals, water level in the shaft will start rising to balance the water head from the upstream end of the conduit. The flow coming from the shaft and the flow coming the low level inlet will be distributed to provide this condition which will reduce contribution of the upstream end as the reservoir level increases.

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Full Pipe Flow without restriction

Pipe Flow

he =	0.62		0.009627	*V^2
hf=	29.16*(n)^2*L/r^(4/3)*1/64.4		0.0230	*V^2
hexit=	1		0.015528	*V^2
n=	0.014		0.0481	*V^2
D=	5 ft			
L=	348.83 ft	Σh =	0.000125	Q^2
A=	19.63 sq-ft	H - Σh =0		
r=	1.25	Q = √H/Σh		
r^(4/3)=	1.35			

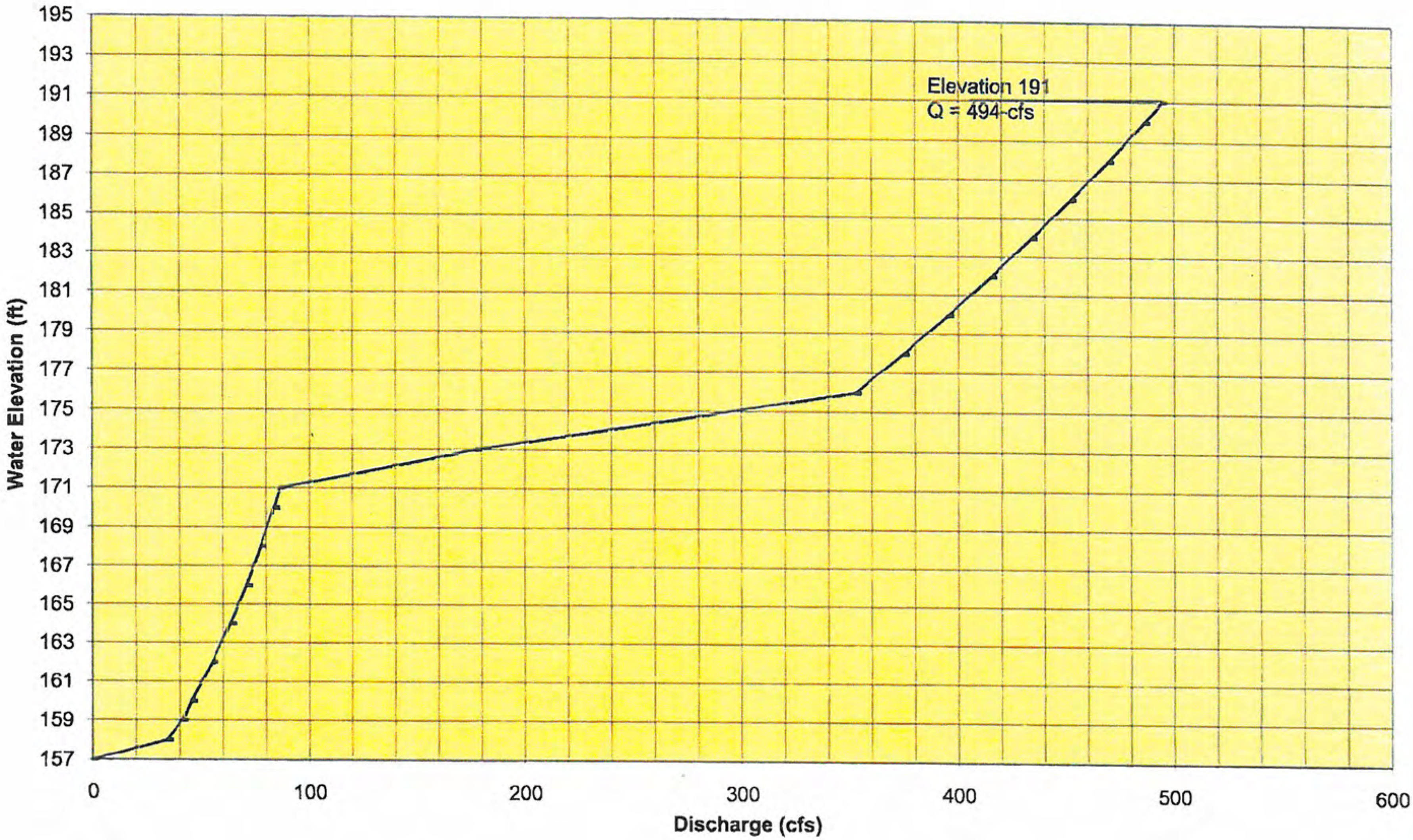
	H (ft)	Q(cfs)
	5	200.0937
	10	282.9752
	15	346.5725
	20	400.1874
El.191 - El. 155,5-5 =	30.5	494.195

Emergency Operation Continued

LOW LEVEL OUTLET

For oricice flow	Head	C 0.605	Area 4.57	Q (cfs)	Head = Resv.-155.5-5	Orifice flow (cfs)	Head (ft)	Mid-inlt Flow (cfs)	Total flow
157	0.00			0	157				0
158	2.08			32	158				33.71
159	3.08			39	159				41.01
160	4.08			45	160	44.84	0		44.84
162	6.08			55	162	54.72	1.5		54.72
164	8.08			63	164	63.08	3.5		63.08
166	10.08			70	166	70.46	5.5		70.46
168	12.08			77	168	77.13	7.5		77.13
170	14.08			83	170	83.27	9.5		83.27
171	15.08			86	171	86.17	10.5		86.17
173	17.08			92	173	75.00	12.5	102	177.00
175	19.08			97	176		15.5	352	352.30
177	21.08			102	178		17.5	374	374.34
179	23.08			107	180		19.5	395	395.15
181	25.08			111	182		21.5	415	414.92
183	27.08			115	184		23.5	434	433.79
185	29.08			120	186		25.5	452	451.88
187	31.08			124	188		27.5	469	469.26
189	33.08			128	190		29.5	486	486.03
191	35.08			131	191		30.5	494	494.20

Emergency Operation Rating Curve



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BOTTOM OUTLET ENERGY DISSIPATOR

DATA

A) NORMAL OPERATION

Max Q = 131 cfs, V = ~9 cfs d = 3.30

B) EMERGENCY OPERATION

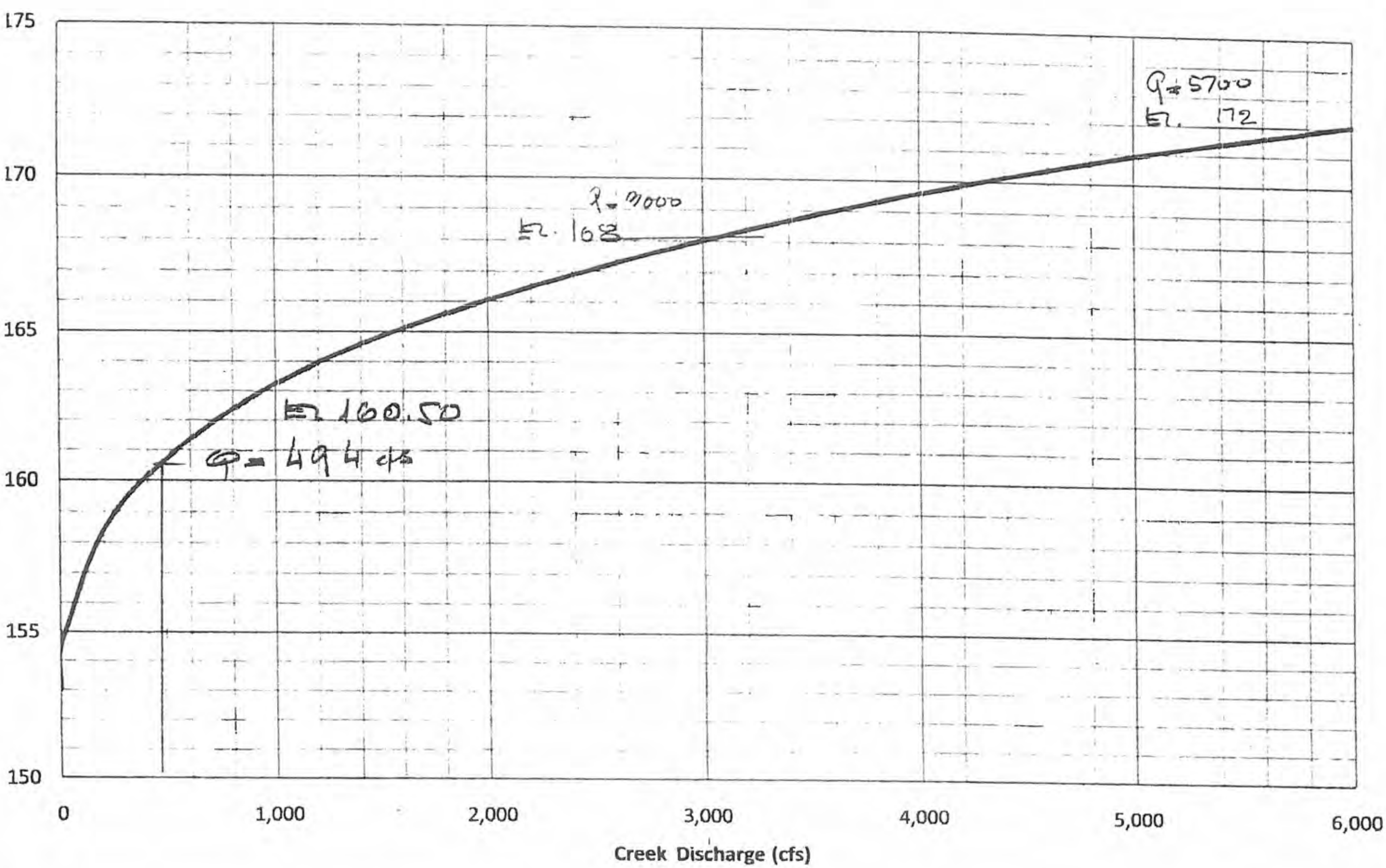
Max Q = 494 cfs. $V = \frac{494}{19.63} = 25.16$ ft/sec

Design impact basin for emergency condition. provide RSP also for emergency condition.

Tailwater level at Q = 494 cfs from page 16 is El. 160.50

Design Energy dissipator using and RSP USBR Publication EM No.25 Section 11

Upper Sand Creek Tailwater Rating Curve No.1



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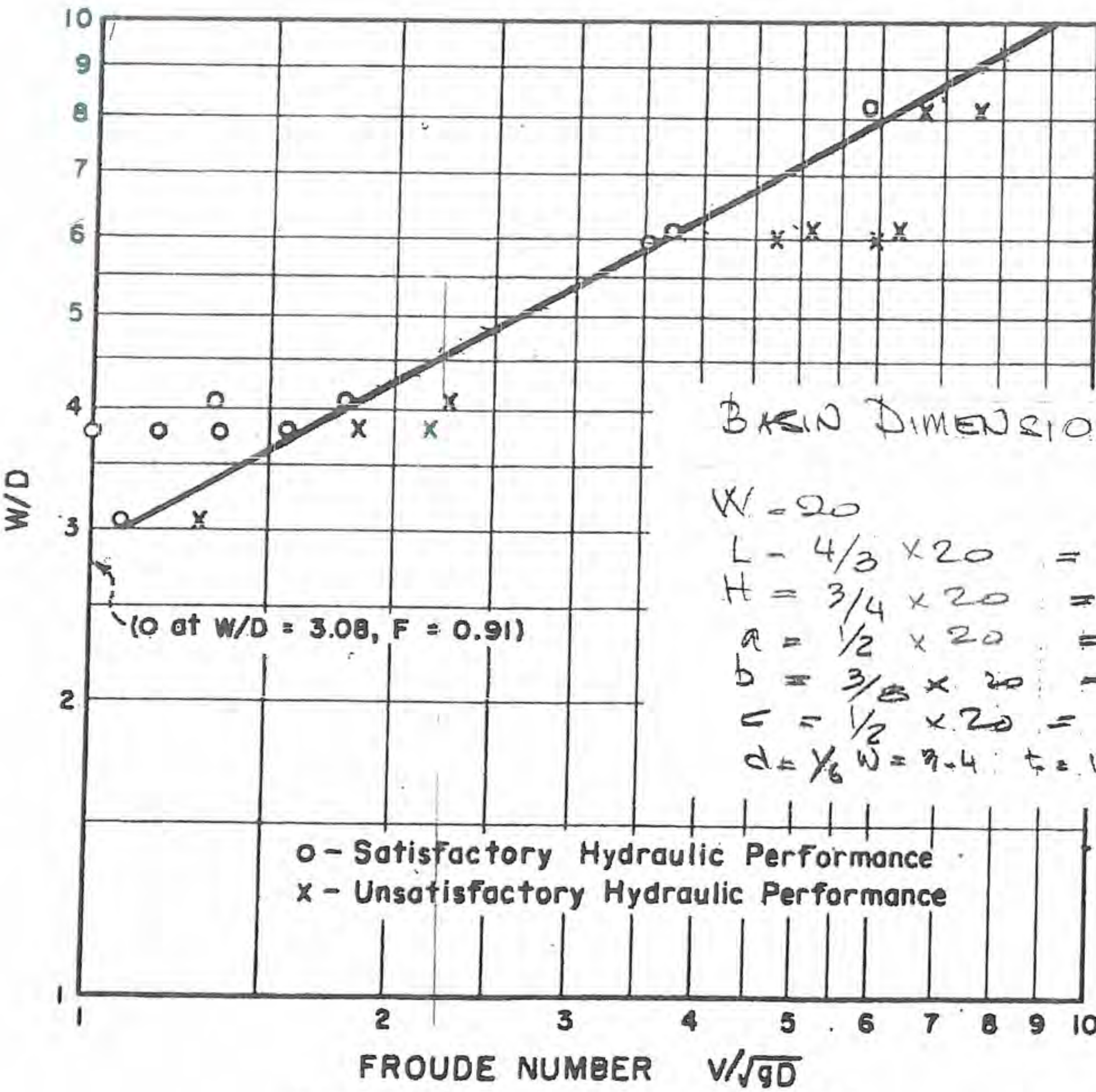
7) ENERGY DISSIPATOR (REF DESIGN OF SMALL DAMS)

Try impact type basin $D = \sqrt{(2.5)^2 \times \pi} = 4.43$

Max $F = 25.16 / \sqrt{32.2 \times 4.43} = 2.11$; $\rightarrow W/D = 4.5$

$= 4.43 \times 4.5 = 20'$



BASIN DIMENSIONS

$W = 20$

$L = 4/3 \times 20 = 26' - 8$

$H = 3/4 \times 20 = 15'$

$a = 1/2 \times 20 = 10$

$b = 3/8 \times 20 = 7.5$

$c = 1/2 \times 20 = 10$

$d = 1/6\ W = 3.4$ $t = 1'-8"$

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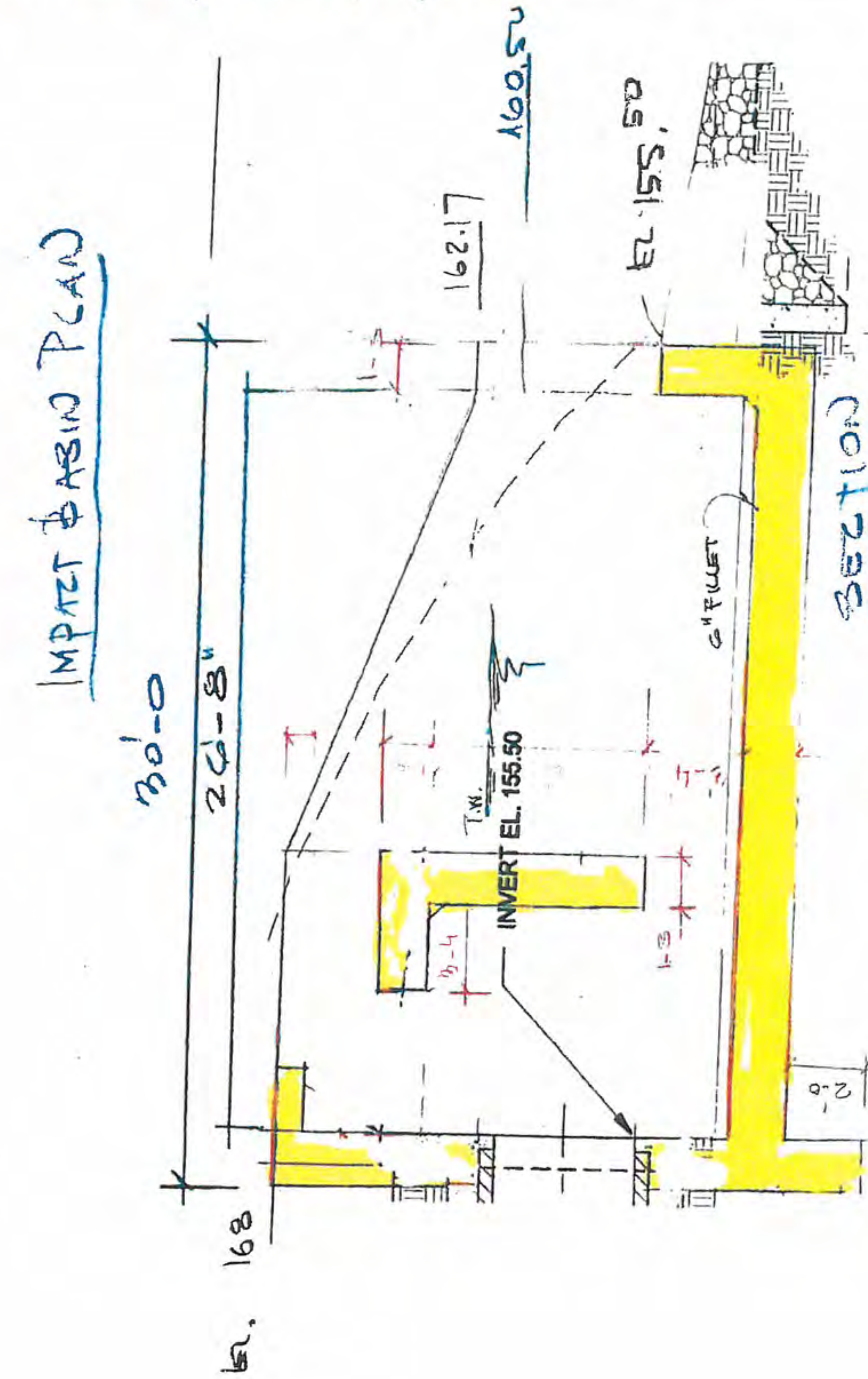

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Riprap Design

At the end of structure max. flow velocity:

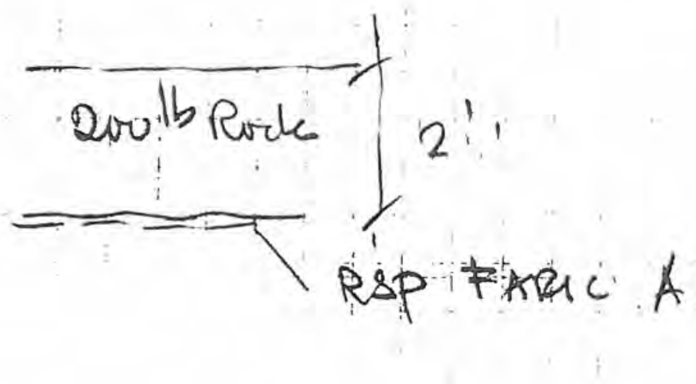
$$q = 494/20 = 24.70 \text{ cfs/foot}$$

$$d_c = \sqrt[3]{\frac{q^2}{g}} = \left(\frac{(24.7)^2}{32.2}\right)^{1/3} = 2.67'$$

$$V = \frac{q}{d_c} = 24.7/2.67 = 9.27 \text{ ft/sec}$$

Ref Figure 165 EM No 25 – 12" Dia. ~90 lb

Caltrans std. 200 lb size Banking No1: RSP FABRIC A



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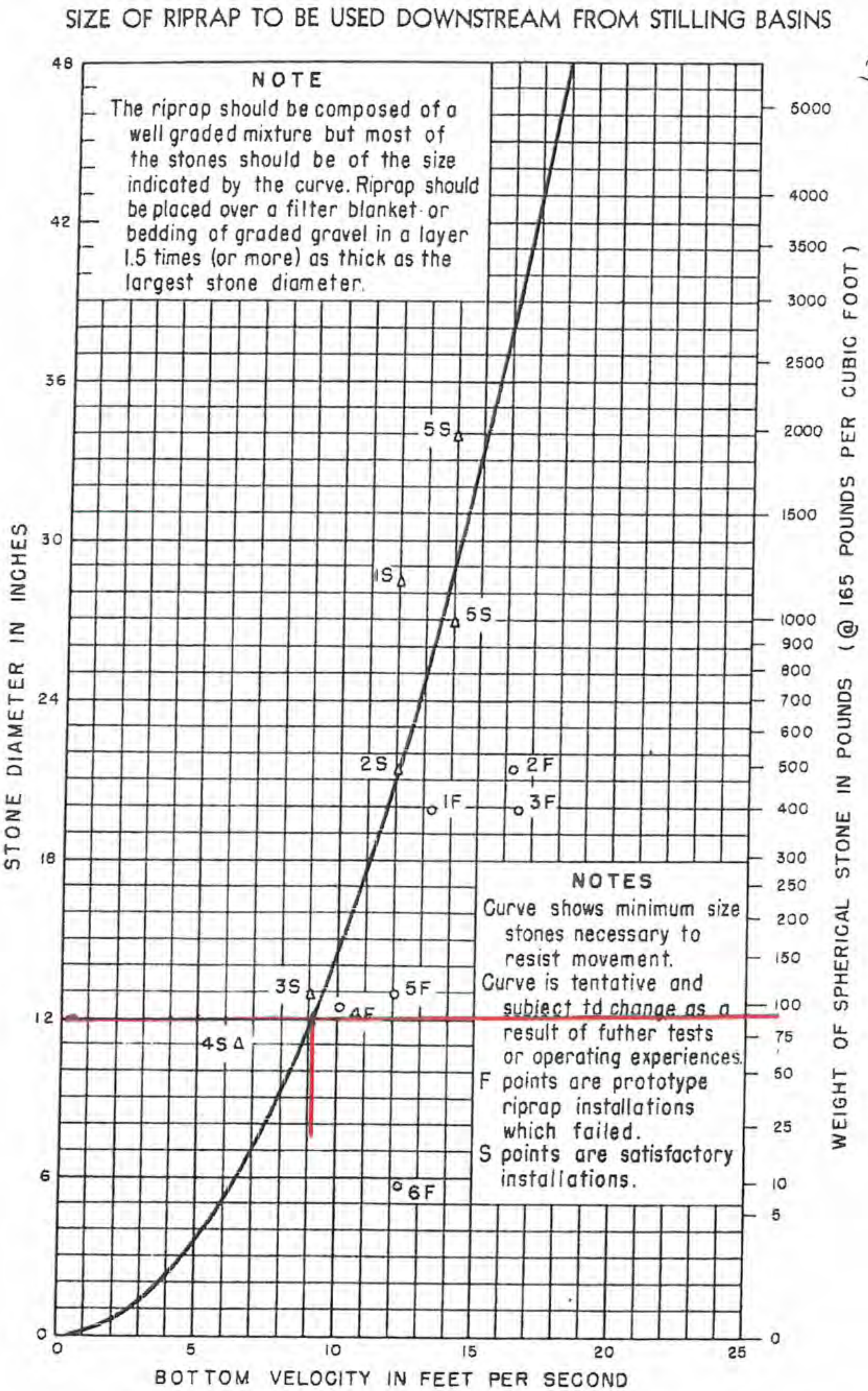

FIGURE 165.—*Curve to determine maximum stone size in riprap mixture.*