APPENDIX A - PROJECT DESIGN AND ENGINEERING TECHNICAL REPORTS

- A-1: Draft (90%) Geotechnical Investigation Report, Bellota Weir Modifications Project, Mormon Slough, Calaveras River, CA (HDR & KSN Inc. February 7, 2022)
- A-2: Draft 90% Hydraulic Modeling Summary Report, Bellota Weir Modifications Project Mormon Slough, Calaveras River (HDR Inc. & KSN Inc. February 4, 2022),
- A-3: Draft (90%) Design Documentation Report, Bellota Weir Modifications Project, Mormon Slough, Calaveras River, CA. (HDR Inc. February 7, 2022).



DRAFT (90%) Geotechnical Investigation Report

Bellota Weir Modifications Project

Mormon Slough, Calaveras River, CA

February 7, 2022

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Acronyms

AOS	apparent opening size
ASCE	American Society of Civil Engineers
ASTM	American Society for Testing and Materials
CBC	California Building Code
El.	elevation
FEMA	Federal Emergency Management Agency
H:V	Horizontal:Vertical
HDR	HDR Engineering, Inc.
Horizontal Datum	CCS83, Zone 3 (horizontal datum)
IBC	International Building Code
NAVD88	North American Vertical Datum of 1988
OSHA	Occupational Safety and Health Administration
pcf	pounds per cubic foot
Project	Bellota Weir Modifications Project
psf	pounds per square foot
SEWD	Stockton East Water District
SPT	Standard Penetration Test
USACE	U.S. Army Corps of Engineers

1 Introduction

1.1 Project Description

The Stockton East Water District (SEWD or District) retained the professional services of KSN, Inc. and HDR Engineering, Inc. (HDR) to advance the Bellota Weir Modifications Project (Project) in San Joaquin County California (Figure 1; all figures provided in Appendix A). The Bellota Weir is located immediately downstream of the Calaveras River bifurcation between the Mormon Slough and Old Calaveras River, about 17 miles downstream of the New Hogan Dam and immediately northeast of the intersection of State Route 26 and Escalon - Bellota Road.

The Project includes weir and intake improvements that will provide more reliable water delivery systems for SEWD customers, improve migration of anadromous fish, and reduce fish entrainment at the facilities, thereby enhancing populations of anadromous salmonids present in the Calaveras River. Initial efforts for this Project included alternative refinement and conceptual design activities in collaboration with SEWD, resource agencies, regulatory authorities, and other Project partners. The Concept Formulation and Design prepared by KSN and HDR (2021) summarize the key background information and process used to identify potential viable options and select a conceptual design that balances the opportunities, constraints, and objectives of this Project.

The current design has progressed to the 90% design level (Figure 2). The Project consists of:

- A new pile supported gated weir (automatic Obermeyer gate system) replacing the existing weir and flashboard system. The weir will be constructed with sheet pile cutoff along the upstream edge of the weir to cutoff seepage flow through the underlying granular channel deposits. Downstream of the weir a gently sloping roughened fishway will be constructed.
- Constructing a combined fish ladder and fish exclusion facility along the north bank that will allow fish to pass the new weir structure and divert flow to the Old Calaveras River and Bellota Intake. All structures will be pile supported. A wall will be constructed along the northside of the roughened channel to direct sluice water past the roughened channel.
- The fish exclusion structure will transfer water north into the Old Calaveras River through two parallel 54-inch-diameter piles discharging through a new headwall structure. It will also discharge water south to the Bellota Intake through three steel pipes that will cross beneath the new weir structure, beneath Mormon Slough, and connect to a new transition structure south of the existing intake structure.
- A new fish barrier, consisting of an earthen berm, to preclude fish entering the Old Calaveras River Channel from the Mormon Slough.

- Constructing appurtenant infrastructure for the fish screen project such as the Control Building/shop, antenna pad, crane pad, and roadway improvements

1.2 Purpose and Scope

The geotechnical investigation was conducted to:

- Perform four supplemental exploratory borings and associated laboratory testing to characterize subsurface surface conditions along the north bank of Mormon Slough and one exploratory boring at the southern end of the existing weir to confirm conditions on the south bank of Mormon Slough.
- Characterize subsurface conditions and develop geotechnical recommendations for the design of the Project structures
- Prepare this characterization report.

1.3 Coordinates Units

The vertical datum for this work is the North American Vertical Datum of 1988 (NAVD88) and the horizontal datum is the CCS83, Zone 3 (Horizontal Datum).

1.4 Codes and Standards and Units

Because the Project site was determined not to be located within the U.S. Army Corps of Engineers (USACE) federal interest, it will be designed in accordance with SEWD criteria which is applicable sections of the 2018 International Building Code (IBC) and 2019 California Building Code.

2 Geotechnical Explorations

2.1 Past Investigations

Prior to conducting the field exploration and laboratory testing, the following geotechnical reports prepared for the site were reviewed:

- Preliminary Geotechnical Conditions and Recommendations – Bellota Weir Fish Passage Facilities for the Calaveras River Anadromous Fish Protection Project prepared by CH2MHILL in 2005. The study included advancing five borings along the Mormon Slough south bank to depths between 30 and 76 feet.

The borings were drilled in October 2004 and January 2005 for the then proposed fish screen location on the Mormon Slough south bank. Figure 3 shows the approximate locations of the previous explorations and Appendix B provides the exploration logs.

2.2 HDR Investigations

HDR conducted supplemental exploration in December 2020 focused on the Mormon Slough north bank in particular for Fish Weir option 7B. The explorations included

advancing five borings to depths of 70 feet, Boring B-1 on the south bank to confirm the conditions described by the CH2MHILL borings and four borings (B-2 through B-5) on the Mormon Slough north bank. A summary of the exploration program and supplemental exploration logs are included in Appendix C. The location of the borings is summarized in Table 1.

Table 1. Summary of Geotechnical Explorations

Explorations	Depth (feet)	Latitude	Longitude
HDR-B1	70	38.052124	-121.011166
HDR-B2	70	38.052257	-121.012052
HDR-B3	70	38.052581	-121.011907
HDR-B4	70	38.052946	-121.011322
HDR-B5	70	38.053055	-121.011818

2.3 Laboratory Testing

Geotechnical laboratory testing was conducted for selected soil samples obtained during each subsurface investigation program. The CH2MHILL study submitted samples to Taber Consultants Laboratory in West Sacramento, California for moisture content, dry density, gradation, and Atterberg limits testing.

Selected samples from the current investigation were submitted to Gulf Shores Laboratory in Rancho Cordova, California for testing, which included:

- Six Fine Sieve Analyses (American Society for Testing and Materials [ASTM] D6913)
- Eight - #200 Sieve Wash (ASTM D1140)
- Eleven Atterberg Limits (ASTM D4318)
- Six Moisture Density (ASTM D2937)
- Two Unconsolidated Undrained Triaxial Shear (ASTM 2850)

Laboratory test results are presented on the boring logs at the appropriate sample depths in 0 through Appendix D.

3 Geologic Hazards

3.1 Geologic Setting

3.1.1 Regional Geology

San Joaquin County lies in the region of the confluence of the San Joaquin and Sacramento rivers. The San Joaquin Valley is bordered on the west by the coast ranges and to the east by the foothills of the Sierra Nevada. The San Joaquin Valley basin has

been filled over time with up to a 6-mile-thick sequence of interbedded clay, silt, sand, and gravel deposits. The sediments range in age from more than 144 million years old (Jurassic Period) to less than 10,000 years (Holocene Period). The most recent sediments consist of coarse-grained (sand and gravel) deposits along river courses and fine-grained (clay and silt) deposits located in low lying areas or flood basins and are referred to as alluvial deposits. These deposits are loose and not well consolidated soils. Older alluvial deposits underlie the edges of the valley. The older alluvial deposits are exposed in the foothill regions in the eastern portion of the county.

3.1.2 Site Geology

The site is mapped as being underlain by the lower member of the Modesto Formation, a quaternary age alluvial deposit that has infilled a narrow valley between ridges (Figure 4; Wagener et al. 1981). The lower member of the Modesto Formation is composed primarily of sand, a substantial proportion of the deposits is well-stratified silt and fine sand, especially near the base of the unit and toward alluvial fan toes. Gravel also makes up a significant part of the lower member, particularly close to the foothills near or above the lower Modesto fan apexes (Marchand and Allwardt 1981). The ridges north and south of the site are mapped as Pleistocene and/or Pliocene sandstone, shale and gravel deposits (https://maps.conservation.ca.gov/cgs/gmc/). Wagner et al. mapped the ridges as comprising Laguna and Mehrten formations.

3.1.3 Regional Seismicity

The site is situated within a region traditionally characterized by numerous active faults and high seismic activity. Many faults exist in the region and are capable of producing earthquakes, which may cause strong ground shaking at the Project site. The nearest active faults are shown in Figure 5 and summarized in Table 2.

An active fault is a fault that has experienced seismic activity during historic time (since roughly 1800) or exhibits evidence of surface displacement during Holocene time (Bryant and Hart 2007). The definition of potentially active varies, however is generally a fault showing evidence of displacement that is older than 11,700 years (Holocene age) and younger than 2.6 million years (Pleistocene age). However, potentially active is no longer used as criteria for zoning by the California Geologic Society.

Table 2. Near Surface Faults within 100 km (62 miles) of the Site

Fault	Distance to Site km (miles)	M_{Max}
Foothill Fault Zone	22 (13)	6.8
Vernalis Fault	48 (30)	6.0
Great Valley Thrust Fault	63 (39)	6.7
Greenville Fault	74 (46)	6.8
Calaveras Fault	97 (60)	6.8
Green Valley Fault	96 (60)	6.7
Midland Fault	60 (37)	6.6

km = kilometers

3.2 Geologic Hazards

3.2.1 Fault Rupture

Based on the information provided in Bryant and Hart (2007), by the California Geologic Survey (2000), and by the Caltrans ARS on-line report (V2.3.06), no known active faults or Alquist-Priolo earthquake zones are present in San Joaquin County. Although the site is within the Central Valley part of California, which is considered to be seismically stable, earthquake activity in neighboring regions, namely the Sierra Nevada and San Francisco Bay area, could affect the proposed Project site with ground shaking. However, the closest active fault to the site is the Vernalis Fault which is located 30 miles west of the Project site. The Foothill Fault Zone is listed in Table 2, however, the segments of the foothills fault system located to the east are generally not considered active nor used as independent seismic sources by the United States Geological Survey (2013).

3.2.2 Liquefaction

Earthquake-induced soil liquefaction can be described as a significant loss of soil strength and stiffness caused by an increase in pore water pressure resulting from cyclic loading during shaking. Liquefaction is associated primarily with loose (low density), saturated, fine- to medium-grained, cohesionless soils below the groundwater table, but can also occur in non-plastic to low-plasticity finer grained soils. The potential consequences of liquefaction to engineered structures include loss of bearing capacity, buoyancy forces on underground structures, ground oscillations or cyclic mobility, increased lateral earth pressures on retaining walls, liquefaction settlement, and lateral spreading or flow failures in slopes.

According to the San Joaquin County Community Development Department online natural hazards disclosure page, no areas within the county have been identified as Seismic Hazard Zones that include the occurrence of liquefaction. The CH2MHILL report identified the south bank of Mormon Slough had the potential to experience liquefaction. The potential for liquefaction at the site is moderate. The liquefaction analysis is discussed in Section 5.2.

3.3 Floods

The Federal Emergency Management Agency (FEMA) Flood Insurance Rate Maps for the site (FEMA 2010) indicate the Project site is mapped as Zone X, which is an area where the flood hazard exceeds 0.2 percent annual chance of flood hazard (Figure 7). Mormon Slough is also identified as a regulated flood way. The slough, located near the Weir, is not leveed and is downstream of the USACE project levee system.

4 Site Conditions

4.1 Surface Conditions

The site is located at the confluence of the Old Calaveras River and Mormon Slough between California State Route 26 on the west, north of Shelton Road on the southeast, and Old Calaveras River on the north. Outside of the channels, the site is relatively flat with elevation range from El. 130 feet near the channel to El. 132 feet near the roadways. The site is covered with native grasses and trees. The elevations within the channels range from approximately El. 130 feet near the top of bank to El. 108 feet within the channel (Figure 2) with local scour holes upstream of the weir extending down to El. 104 feet and 95 feet downstream of the weir. The banks of the slough and river channel are generally over steepened at slope of 1.5H:1V (Horizontal:Vertical) or flatter.

Site development has generally been limited to the Bellota Intake on the south bank, Bellota Weir, and Old Calaveras River Headworks Gate across the old river channel (approximately 570 feet west of Mormon Slough). Review of available historic topographic maps and aerial photographs indicate the site has remained relatively unchanged since the reconstruction of the Bellota Weir in the late 1960s and early 1970s. Review of a 1967 aerial photograph indicates there were once medium sized buildings located on the south bank, setback from the river which were subsequently removed. A structure was also present adjacent to and south of the Old Calaveras Headworks Gate Structure. The structures are shown on the 2002 U.S Geological Survey Topographic 7.5-minute quadrangle but were removed from the maps in the 2012 update. A 1993 aerial photograph shows the buildings had been removed prior to 1993. An existing house is located approximately 400 feet west of the weir that is shown on the 2002 topography.

4.2 Subsurface Conditions

Subsurface conditions at the site have been interpreted based on the 2005 and 2021 subsurface investigations discussed in Section 2.0 and presented in Appendix B and Appendix C. The locations of the borings are shown on Figure 7 and three cross sections presented in Figure 8 through Figure 10. In general, site conditions consist of interbedded silts, silty sands, sands with variable percentages of gravel, silty sand, sandy gravels and clay typical for the riverine environment that can be divided into three layers: upper overbank deposits, a middle alluvial layer, and older, stiffer overbank deposits that extend to 70 feet, the exploration depth.

Overbank deposits generally consist of loose brown silty sand and sandy silt to a depth of 20 to 25 feet. The underlaying alluvial layer varies between 20 and 25 feet thick and is comprised of a 15-foot-thick upper fine grained layer consisting of interlayered loose silts, silty sands, silty clayey sands, and soft clays with discontinuous loose gravel deposits within the meandering river channel. The lower layer has a coarser grained layer consisting of loose silty to clayey sands with varying gravel content and more pronounced gravel zones. Older medium dense to hard alluvium is located at a depth of

40 feet and consists of alternating layers of silt, and silty sand with some poorly graded sand layers encountered at depth.

More recent alluvium is located within the channel with the south bank consisting of a gravel bar. The existing weir includes an upstream sheet pile cutoff wall to cutoff seepage flow beneath the existing weir. Review of as-built drawings show that the cutoff wall extends 8 feet below the weir abutments and increased in depth to 14 feet beneath the central portion of the weir. The cutoff wall was likely deepened to extend through more permeable channel deposits to maintain an 8-foot depth of wall penetration into the upper alluvial layer (Figure 9). This assumption indicates that up to 6 feet of recent alluvium is located below the weir and roughly correlates to the depth of the gravel bar encountered in the south bank borings.

4.3 Groundwater

Groundwater correlated with the water level in the slough and impounded by the weir. The Mormon Slough flow is controlled by releases from New Hogan Dam located 11.7 miles northeast of the weir. Groundwater was encountered between El. 108 and 110 feet during the recent explorations. The lowest groundwater level was measured in boring HDR-B2 located on the right bank downstream of the weir.

5 Discussion and Conclusion

5.1 General

Geotechnically, the Project is feasible with implementing the recommendations provided in this report. The key geotechnical considerations are the presence of loose granular deposits (Overbank and Upper Alluvium) above El. 100 feet and its effect on foundation selection and seismic performance; seismic ground shaking and the potential for limited liquefaction resulting in seismically induced settlement; presence of over steep river banks with respect to static and seismic stability; and considerations for construction phases when working in the river.

The proposed weir and fish passage structures will require support from deep foundations extending below El. 90 feet and loaded in tension (uplift) and compression. A sheetpile cutoff will be installed just upstream of the weir to provide a seepage cutoff through the permeable channel deposits and prevent through seepage erosion beneath the weir foundation. The existing north bank of the slough and south bank of the Old Calaveras River channel are over steepened and subject to instability under both static and dynamic loading, which needs to be accounted for in the design. The Project will be phased as follows:

Phase 1 – Construct dam sill and roughened channel (including sheetpile cutoff)

Phase 2 – Construct dam super structure and mechanical gates

Phase 3 – Existing intake, Old Calaveras River Improvements

Phase 4 – Fish ladder and sluiceway construction

Phase 5 – Remaining portions of Bellota Conveyance Structure and tying into the existing intake/conveyance pipeline

Relevant geotechnical discussion on each of these items follows and specific geotechnical recommendations are provided in Section 6 of this report

5.2 Liquefaction

Liquefaction evaluations were based on the data presented in the 2005 boring logs and mud rotary wash borings performed for this investigation. For the soil layers encountered in the mud rotary borings, the soil composition criteria in Seed et al. (2003) was used to evaluate the liquefaction susceptibility. For soil layers determined to be potentially liquefiable, a liquefaction triggering analysis was performed using the methodologies in Idriss and Boulanger (2008). The liquefaction induced settlements were estimated using volumetric strain criteria in Tokimatsu and Seed (1987). The depth to groundwater measured at the time of drilling was used in the analysis cyclic strength resistance (Table 3). The groundwater at the time of the 2005 explorations was not documented in available reports, but was assumed approximately equal at El. 112 feet, the approximate bottom elevation of the weir, which is slightly above the highest groundwater elevation recorded in borings HDR-B3 and HDR-B4.

Earthquake magnitude and peak ground acceleration were determined using the 2008 Probabilistic Seismic Hazard Analysis Interactive Deaggregation website (https://geohazards.usgs.gov/deaggint/2008/). The analysis used an earthquake magnitude of 6.5 and a peak ground acceleration of 0.287g for the liquefaction analysis. A site-specific ground motion hazard analysis was not performed. The analysis was performed with groundwater at both the maximum and minimum pool elevation assuming that the groundwater would equal the pool water surface elevation at each boring. Review of the groundwater elevations at the time of drilling summarized in Table 3 suggest that Boring HDR-B2, located downstream of the weir, would have a lower groundwater elevation than assumed in the analysis.

Minimum factor of safety for a liquefaction hazard assessment are provide in the 2009 National Earthquake Hazards Reduction Program. Recommended seismic provisions that reference work published by Martin and Lew (1999). In general, the corrected equivalent clean sand Standard Penetration Test (SPT) blow counts in the potential liquefiable layers at the site are less than 15. Therefore, the following minimum factors of safety are recommended:

Settlement	1.1
Surfacing Manifestation	1.2
Lateral Spread	1.3

The site is comprised predominately of loose to medium dense sands silts and gravels above El. 90 feet. Below El. 90 feet the soils consist of alternating layers of medium dense to hard silts and silty sands. The soil composition of the upper soils meets the criterial of potentially liquefiable by the criteria presented in Seed et al. (2003). The liquefaction triggering analyses confirmed the 2005 investigation finding that the south bank contains soils with high potential for liquefaction and, if not mitigated, would result

in seismically induced surface settlements from soil layers that have factors of safety against liquefaction between 1.0 and 1.3. These layers range in thickness from 5 to 10 feet and are located between El. 100 and 115 feet, 10 to 20 feet below the ground surface. Analysis of the current investigation of the north bank found that the site has less potential for liquefaction, however, would also experience seismically induced settlements due to layers with factors of safety against liquefaction between 1.0 and 1.3.

No layers had a factor of safety against liquefaction less than 1.0 and the risk of strength loss is judged to be low, the risk of surface manifestation (sand boils) on the ground surface is judged to be extremely low. However, some surface manifestation could occur within the channel or south bank gravel bar deposits due to low confining pressures.

Excavation and replacement as a means to mitigate the low-density and saturated layers may not be a preferred alternative due to their depth below the top of bank. Alternatively, the layers could possibly be densified by performing compaction grouting; however, the high fines content layers could result in limited effective densification. Stone columns could also be installed on a grid pattern to improve and strengthen the entire soil mass. A typical stone column grid pattern could be 5 to 8 feet on center, in either a triangular or square grid, and focused along the riverbanks and beneath the fish ladder footprint, box culvert alignments, Bellota Intake Structure, and Bellota Weir. Stone columns would not replace the deep foundation needed to support the fish passage and weir.

Settlements were calculated using the volumetric strain procedure (Ishihara and Yoshimine 1992). The probability of liquefaction was compared to the relationship developed by Cetin et al. (2018). Post seismic settlement was estimated for soil layers calculated to have a factor of safety less than 1.2 for the design peak ground acceleration. Calculated post-liquefaction settlements for each boring are provided in Table 3 and range from no settlement in boring HDR-B4 to 5.8 inches in Boring BI-1 located adjacent to the Bellota Intake Structure. The settlements are greatest for the maximum pool elevation (El. 121.4 feet) and the design input acceleration (0.287g) with the probability of settlement occurring from 50 to 80 percent. The probability of seismically induced settlement reduces dramatical for accelerations less than 0.2g.

Table 3. Summary of Calculated Liquefaction Settlement

Boring ID	Location	Ground El. (NAVD88)	Depth to Groundwater at Time of Drilling (feet)	Approx Groundwater El. at Time of Drilling (feet)	Min Pool Liquefaction Settlement[1] (inches)	Max Pool Liquefaction Settlement[1] (inches)	Depth to Liquefiable Layers (feet)
HDR-B1	Left Abutment	124	18	106	0.2	0.2	2.5 ft thick layer at El. 108.2 ft
HDR-B2	North Bank Downstream	132	30	102	1.1	1.1	5 ft thick layer at El. 101.2 ft
HDR-B3	North Bank Right Abutment	135	25	110	2.5	2.5	10 ft thick layer at El. 104.2 ft
HDR-B4	Upstream North Bank	131	21	110	0	0	N/A
HDR-B5	South Bank of Old	131	25	106	0.1	0.1	5 ft thick layer at El. 105.2 ft

Boring ID	Location	Ground El. (NAVD88)	Depth to Groundwater at Time of Drilling (feet)	Approx Groundwater El. at Time of Drilling (feet)	Min Pool Liquefaction Settlement[1] (inches)	Max Pool Liquefaction Settlement[1] (inches)	Depth to Liquefiable Layers (feet)
	Calaveras River						
BD-2	Weir Right Abutment	113.9	1.9	112 [2]	2.4	2.4	4 ft thick layer at El. 104.1 ft and 5 ft thick layer at El. 74.1 ft
BF-1	Weir Left Abutment	122.4	10.4	112 [2]	2.2	2.2	5 ft thick layer at El. 96.6 ft, 5 ft thick layer at El. 76.6 ft, and 5.7 ft thick layer at El. 45.9 ft
BI-1	SEWD Intake	123.0	11	112 [2]	5.8	5.8	10 ft thick layer at El. 102.2 ft (3.5 inches), 5 ft thick layer at El. 92.2 ft (1.1 inches), and 10 ft thick layer at El. 72.2 ft (1.2 inches)
BI-2	South Bank Gravel Bar	116.4	4.4	112 [2]	1.1	1.1	5 ft thick layer at El. 100.6 ft and 3.5 ft thick layer at El. 87.1 ft
BI-3	South Bank Gravel Bar	115.9	3.9	112 [2]	0.5	0.5	5 ft thick layer at El. 107.6 ft and 5 ft thick layer at El. 92.6 ft

1. Minimum Pool El. 115.44 feet and Maximum Pool El. 121.44 feet

2. Historic Borings performed by others. Groundwater elevation at time of drilling not reported and is assumed to be equal to the base of the existing weir for these analyses.

3. Peak ground acceleration (PGAm) of 0.287g for Site Class D and earthquake magnitude (Mw) of 6.5

NOTE: Depth to Liquefiable Layer is the depth to layers with factor of safety < 1.2 for liquefaction (Elevation of bottom of the liquefiable layers shown).

The calculated post seismic settlement of specific structures are as follows:

Weir Abutments	2.2 to 2.5 inches
Right Bank downstream of Weir	Approximately 1 inch
Bellota Intake	5.8 inches
Proposed Fish Screen Intake	less than ½ inch to 2.5 inches
Old Calaveras River Bypass Outfall	less than ½ inch

Due to wide variation in calculated settlements and the proximity of the proposed structures to the river, differential settlements are anticipated to be significant. The structures should be supported on deep foundations to gain support in the stiff to hard soils in the older alluvium, below El. 90 feet.

5.3 Slope Stability

The Old Calaveras River Bank adjacent to boring HDR-B5 was selected for static and seismic stability analysis as this slope is overly steep and will have limited modifications by the Project. Soil strength properties were developed based on correlations with SPT equivalent clean sand blow counts ($N1_{60}cs$). The results for the stability analyses are shown in Figure 11 through Figure 14. The failure surface with a factor of safety equal to 1.0 is shown on Figure 12. The soil wedge with a factor of safety of 1.5 is a linear wedge projecting up from the toe of slope and setback a distance of 18 feet from the top of bank (44.4 feet from the toe of slope). This figure shows that overtime the upper portion of the slope will continue to erode to meet the factor of safety 1.5 projection line (Figure 13). Therefore, any structure supported with shallow foundations should be setback a distance of 18 feet from the top of slope.

Seismic pseudo – static stability was evaluated with a horizontal acceleration of 0.15g (approximately 50 percent of the peak ground acceleration) assuming the river bank would act as a rigid block (Figure 14). The setback of 18 feet results in a slope with a minimum factor of safety of 1.1 which is acceptable for the seismic case.

5.4 Roadways, Control Building, Antenna Pad, and Crane Pad Subgrade

The upper soils at the site are comprised of loose sand and non-plastic silts that vary in density across the Project site. Therefore, it is recommended that the upper 6 feet of the site be excavated, the material stockpiled and then placed back into the excavation as properly moisture conditioned and compacted engineered fill to provide a suitable foundation support for spread footings, slab-on-grade, and roadways. The excavations should extend a minimum of 5 feet beyond the edge of buildings or slabs. The intake structure and fish passage should be supported on deep foundations.

Care should be taken for the crane pad where a portion of the slab will be located over the deep excavation to install the Bellota Conveyance pipelines. The shape of the excavation to construct the intake structure distribution box and fish ladder is not specified and, depending on how the excavations are backfilled, could result in large differential settlements beneath the pad leading to cracking and potential large vertical and horizontal offsets. Therefore, the subgrade for the entire crane pad should be removed to EL 112 feet at the edge of the conveyance structure and fish ladder and uniformly sloped up to EL 120 feet, 5 feet beyond the north and west edges of the crane pad. The excavation should be backfilled with properly moisture conditioned and compacted native soils as recommended in Section 5.8 to control abrupt differential settlement of the slab. The slab could settle over time likely sloping away from the fish passage structure. Gapping between the slab and the structure could occur overtime and would require routine maintenance to ensure a watertight seal between the slab and the adjacent structures.

5.5 Weir Foundation, Sheetpile Cutoff, Roughen Channel Fill

5.5.1 Sheetpile Cutoff

Review of available as-built drawings show that the weir had been damaged in the late 1960s and the current weir constructed as a replacement. The site was regraded using channel deposits from upstream of the weir. The new weir included installing a sheet pile wall along the upstream edge of the foundation varying in depth from 8 feet at the abutments to 14 feet in the center of the channel (Figure 9). At its deepest point, the sheetpile cutoff appears to have tipped into the lower coarse-grained alluvial layer. The wall appears to cutoff seepage and reduce potential erosion of material placed to support the weir, which may have been the failure mode of the original weir. The basis for the wall depth is not stated on the as-built drawings, however it may be that the wall depth was selected to maintain 8 feet of penetration into the underlying alluvium located below the channel deposits. The method of sheet pile installation also is unknown as well as the current condition of the existing sheetpile wall.

To construct the new weir, the existing weir should be removed, and the foundation excavated to expose at least the top 7 feet of the existing sheetpile wall. Material excavated from underneath the weir likely consists of channel deposits and should be placed as fill within the scour holes upstream and downstream of the weir alignment. The existing sheetpile wall could then be exposed and either extracted and/or cut to the excavated subgrade. New free draining engineered fill could be placed and properly compacted to establish the new weir foundation subgrade for the new sheetpile wall and weir foundation piling to be installed. The new sheetpile wall should extend to a depth of 20 feet below the gated section of the weir and the tip elevation held constant as the wall progresses up the abutments terminating above the full pool water surface. A test pile program should be performed to confirm the appropriate method of pile installation to reach the design depth.

5.5.2 Roughened Channel

The fish weir includes a roughened approach channel downstream of the weir. This area will be subject to underseepage and end around flows, therefore the gradation of the fill should be specified to reduce the potential for transport of finer grained foundation or abutment material into new fill. Migration of finer grained material could, over time, lead to the formation of sinkholes or depressions. A drainage layer should also be included in the fill to provide a drainage pathway to the downstream toe. Section 6.9 provides the analysis and recommendations for the gradation of the filter layer and transition layer section below the roughened channel. It also provides the riprap to be placed on the channel slopes above the roughened channel and within the Old Calaveras River channel.

A seepage model was developed for the weir and roughened channel section (Appendix E). Values of hydraulic conductivity and drained strength for the granular soils were estimated based on material classification and particle size analyses. The inclination of

the roughened channel is less than 2 degrees; thus the channel has a high factor of safety against slope failure during both low and high pool conditions. The cutoff sheetpile wall primary function is to prevent undermining of erodible material beneath the weir slab during high flow conditions. Key to the performance of the weir and the roughened slab is extending the sheetpile cutoff into the underlying older alluvial layer as previously discussed. This modeling shows that the cutoff is effective in reducing downstream underseepage flow into the roughened channel fill. The banks of the slough are comprised of permeable materials and lateral/end around flow will occur, as had occurred for the current weir structure. Seepage flow also will be greater than indicated by the seepage models. Estimating the three-dimensional seepage effects around the weir was beyond the scope of these studies.

5.6 Old Calaveras Conveyance Pipeline

The fish ladder will include fish screens that connect to an afterbay to control flow velocity through the fish screens. Water will then flow into a water distribution structure connecting to the Old Calaveras River and/or the Bellota Intake structure through two systems of pipes. The Old Calaveras River diversion consists of two 54-inch-diameter pipes that extend below grade and discharge into the channel through an outfall structure. The pipe installation would provide a net unloading to the foundation, however post seismic settlement could vary from zero to 2.5 inches between the weir and the outfall structure (Table 3). Options to mitigate the settlement could include ground improvements to reduce settlements to tolerable levels supporting the box culverts on a rigid foundation or deep foundations. The box culvert will likely be constructed by the cut and cover methods or a combination of cut and cover with shoring. Dewatering of the excavation will also be likely. To reduce the potential for ground settling away from the base of the box culvert and provide a stable base to construct on is to install pattern short stone columns to support the culverts (Section 5.2). The width of treatment should be two times the width of the box culverts and extend from the entire length of the pipes. If ground settlement away from the base structure is not a concern for the performance of the diversion, the culvert can be supported on deep foundations.

The Bellota Intake diversion consists of three 54-inch pipes that extend beneath the weir and fish ladder structure and connect to a junction structure with the existing Bellota Distribution Pipeline . The pipelines will be constructed in three phases consisting of a 90 degree bend extending from the fish passage distribution structure on the north bank, extending beneath the weir structure along the downstream side of the sheetpile cutoff, and then extending up the south bank connecting to the Bellota junction structure. Table 3 indicates that the weir could experience 2.5 inches of settlement where it crosses the channel and less than 1 inch at the abutments. Also, the Bellota Intake Structure is founded on a thick sequence of loose granular soils and could experience up to 6 inches of settlement. Therefore, foundation options include supporting the Bellota Intake box culvert on pile foundations similar to those supporting the fish screen structures or treating the foundation with stone column ground improvements when adding deep foundations or soil anchors (such as helical anchors) to provide uplift support. The structural design team has selected installing steel cross members that will span between the weir pile foundations to support the pipelines where they cross under

the weir and encapsulating the pipes in controlled density backfill. The pile will provide support against settlement and uplift displacements.

5.7 Bellota Intake

Analysis of the one 2005 boring located adjacent to the intake structure indicates locally as much as 5.8 inches of post seismic settlement could occur. However, boring BI-1 calculated settlement is more than twice the amount of estimated for the next adjacent boring BF-1. It is recommended that additional cone penetrometer testing be performed around the intake to confirm that the subsurface conditions shown in boring BI-1 are accurate, specifically the 2.3 inches calculated at depth as no other boring drilled at this site calculated post seismic settlements at this depth. The current design has placed the Bellota junction structure further back from the south bank away from the thick loose granular deposits that were encountered along the south bank in an area that has not been investigated.

One approach for designing connections at this junction structure is to not mitigate all settlement but provide flexible couplings that could accommodate up to 1 inch of deferential settlement.

5.8 Construction Sequencing

The Project construction is proposed to occur in five sequential phases to minimize environmental impacts and optimize constructability, cost effectiveness, and operational continuity. The current sequencing strategy and level of effort is based on a minimum 2-year construction period; however, project elements may be constructed in a different order or over a longer period of time to accommodate costs or permitting restrictions.

A construction sequencing summary is provided in this section.

Sequence 1A (Non-Irrigation season, out of water work)

- Complete construction of access roads
- Remove trees

Sequence 1B-1D (Irrigation season, in-water work window)

Construction of sequences 1B-1D will take place during the irrigation season (mid-April to mid-October), during the in-water work window of mid-June to early October. Stream diversion will be provided using the proposed 54-inch sluiceway on the north bank and the existing 36-inch bypass on the south bank.

Sequence 1B

- Complete demolition of existing weir
- Complete removal and salvage of existing riprap
- Installation of sheet pile walls

- Installation of 54-inch sluiceway on north bank and extension of 36-inch bypass pipeline on south bank

Sequence 1C

- Complete in-water grading in Mormon Slough
- Partial construction of Bellota conveyance pipelines (under concrete weir structure)
- Complete construction of weir slab
- Complete pile installation for the weir and debris boom
- Complete construction of the roughened channel

Sequence 1D

- Complete Installation of debris boom components
- Remove temporary access road to roughened channel
- Installation of sheet pile wall S-3A and removal of sheet pile wall S2
- Complete installation of riverbank armoring
- Removal of in-river dewatering systems

Sequence 2 (Non-irrigation season, out-of-water work)

- Complete construction of the concrete fish ladder structure
- Completion of the downstream section of the intake used for sediment sluicing and installation of sluice gates
- Connection of 54-inch sluiceway pipeline to structure
- Complete construction of the diversion structure
- Complete construction of the control building

Sequence 3 (Irrigation season, in-water work window)

- Complete construction of the intake structure
- Complete construction of weir superstructure and installation of Obermeyer weirs

Sequence 4 (Non-irrigation season, out-of-water work)

- Complete construction of the Old Calaveras River conveyance pipelines
- Complete construction of the Bellota conveyance pipelines
- Complete remaining site work

Sequence 5 (Irrigation season, in-water work window)

- Completion of in-water grading in the Old Calaveras River

The Project is constrained to a relatively confined area and the effects of shoring installation, dewatering systems, and excavations immediately adjacent to other structures needs to be carefully evaluated to assess changes in loading conditions.

The phased construction could result in temporary unbalanced loads that may govern the design of a specific element. Portions of the permanent work may serve as support for temporary excavations and impose additional loading conditions that should be evaluated by the design team. Including the effects of differential dewatering, excavation induced lateral soil loads, surcharges, and structural connection between phases of work.

5.8.2 Seasonal Construction and Dewatering

Calaveras Dam and seasonal releases may affect construction. Specifically, the top elevation of temporary sheetpiles, cofferdams, or berms should be specified to provide adequate protection to the work areas and prevent inundation during infrequent releases that could overtop systems and flood the site. Also, seasonal shutdowns may affect the next season's work specifically the removal and re-installation of dewatering systems. The channel deposits and underlying alluvial deposit have a relatively high permeability and dewatering operations may account for high water conditions that continue during winter shutdowns

6 Recommendations

6.1 Seismic Design Criteria

The proposed structures should be designed to resist the lateral forces generated by earthquake shaking in accordance with local design practices. The seismic design parameters based on the American Society of Civil Engineers (ASCE) 7-16 as modified by Supplement #1 and the 2019 California Building Code for a Latitude of 38.052325 and a Longitude of -121.011499 are presented in Table 4.

Table 4. Seismic Design Parameters

Parameter/Coefficient Description	Parameter/Coefficient	Value
MCE_R for 0.2 sec Spectral Response Acceleration, Site Class D	S_s	0.490g
MCE_R for 1.0 sec Spectral Response Acceleration, Site Class D	S_1	0.287g
Soil Profile Type	Site Class	D
Site Coefficient	F_a	1.4
Site Coefficient	F_v	2.15
Adjusted MCE Spectral Design Parameters	S_{MS}	0.690g
	S_{M1}	0.480g
Design Spectral Acceleration Parameters	S_{DS}	0.460g
	S_{D1}	0.320g

Table 4. Seismic Design Parameters

Parameter/Coefficient Description	Parameter/Coefficient	Value
Seismic Design Category	Occupancy Category I to IV	III

MCR_R = Risk targeted maximum considered earthquake

6.2 Earthwork

6.2.1 Stripping and Demolition

The proposed site improvement areas currently consist of existing structures and undeveloped areas. Initial preparation of the site will include demolition of existing facilities (including demolition of existing structures, foundation systems, trees, and vegetation. Existing underground utilities that interfere with the proposed construction should be identified and properly abandoned or relocated.

Areas to receive fill and support structures should be stripped of debris and any other deleterious materials prior to over-excavation or placement of engineered fill. Stripping and removal of debris should extend laterally a minimum of 5 feet beyond the perimeters of roadways, concrete flatwork, and any other facilities supported on grade. Any stripped debris or organic matter should not be reused as engineered fill, but can be reused as non-engineered fill in designated areas or removed offsite. Excavations from removal of foundations, underground utilities, or other below-ground obstructions should be cleaned of loose soil and any potentially deleterious material and backfilled with compacted engineered fill to the requirements provided herein. Excavation backfilling operations should be carried out under the observation of an HDR geotechnical engineer. In addition, active or inactive utilities within the construction area should be protected, relocated, or abandoned.

During construction, soft or loose areas may be encountered especially near or below the existing groundwater level (roughly El. 110 feet). Loose or soft subgrade soils encountered during construction may require over-excavation. Assessment of these areas to determine the extent of over-excavation should be performed by the geotechnical engineer at the time of construction when conditions are exposed.

6.2.2 Densification of Potentially Liquefiable Soils

Granular soils within the riverbanks were found to have factors of safety greater than 1.0 for the peak ground acceleration and although not subject to the occurrence of full liquefaction and strength loss, they will develop pore pressures and experience post-seismic settlements. Total and differential settlements will vary across the site, and ground improvements to densify these soils are recommended. Two institute densification methods that could be performed at this site are compaction grouting and vibro-replacement (atone columns). Compaction grouting is performed by injecting grout at targeted depths creating a pressure bulb that compacts the soil. Vibro-replacement consists of advancing a vibratory probe to the specified depth and placing the constructing a compacted stone column by lifting the probe in increments, introducing

gravel through the bottom feed method and then repenetrating the gravel to compact it and surrounding soils. Both methods require an experienced specialty Contractor to perform the insitu densification.

Due to the variability in the fine-grained soil content compaction grouting may not be a feasible means to perform targeted densification. Instead short stone columns should be installed in a 5- to 8-foot pattern to densify the soils and strengthen the site. Stone columns should be constructed using the bottom up construction method with casing or a vibrating probe to support the soil column as the column is built Gravel is placed in the column in lifts and vibrated to densify the stone end loose granular soil layers. Densification should be performed prior to preparing the site subgrade as discussed in the following section so that disturbed surface soils can be properly prepared.

6.2.3 Subgrade Preparation

The site is blanketed with a loose silt and silty sand layer. Therefore, after stripping and performing any necessary removals, the upper 6 feet of the site should be excavated, moisture conditioned, and compacted to make the subgrade below any footings and slab-on-grades. The excavation should extend 5 feet beyond the limits of the proposed improvements. The material should be placed in accordance with the recommendations provided in Section 6.2.5.

Concrete for foundations and exterior flatwork (e.g., sidewalks or walkways) should be placed neat against undisturbed soil or properly-prepared engineered fill as described in Section 6.2.4. Foundation subgrades should not be allowed to dry out before placing concrete. If shrinkage cracks appear in the excavations, the excavations should be thoroughly moistened to close all cracks prior to concrete placement. The foundation excavations should be observed by the geotechnical engineer for compliance with appropriate moisture control and to confirm the adequacy of the supporting soils. Soft, loose, or otherwise deleterious materials encountered at the bottoms of footing excavations should be removed and replaced with compacted engineered fill per the requirements below.

6.2.4 Engineered Fill Materials

Material used as engineered fill to support structures should have an organic content of less than 3 percent by volume, be free of debris and other deleterious materials, be onsite lean clay with a plasticity index less than 15 and liquid limit less than 40 or granular non-expansive and have a maximum particle size less than 3 inches in dimension. Excavated onsite materials not meeting these requirements should be properly disposed of offsite or, if approved by San Joaquin County, placed as a wide, level, engineered fill. The fill should be setback a minimum 50 feet from the slough banks to not appear as a levee and should not concentrate flow onto adjacent properties. High moisture content soils that meet these requirements are not considered as unsuitable material but will require proper moisture conditioning prior to placement and compaction.

Imported fill should be predominantly granular, meet the size requirements previously discussed, have an organic content of less than 3 percent by volume, have a liquid limit less than 40 percent, have a plasticity index not exceeding 15, should be non-corrosive,

and should contain no environmental contaminants or debris. Imported materials for use as engineered fill should be sampled and tested by the geotechnical engineer prior to being transported to the site. Submitting representative samples of imported materials proposed for use as engineered fill should be submitted to the geotechnical engineer for testing and approval at least one week prior to the start of grading and import of the material.

6.2.5 Fill Placement and Compaction

Prior to fill placement, the subgrade should be prepared as discussed above. Engineered fill placed beneath or around structures should be placed in loose horizontal lifts less than 8 inches, moisture conditioned to 0 to +3 percent above optimum moisture content and compacted to at least 90 percent of relative compaction per ASTM D1557. The upper 2 feet of fill placed as structure, slab-on-grade, or pavement subgrade should be compacted to 95 percent per ASTM D1557.

Additional fill lifts should not be placed if the previous lift did not meet the required relative compaction or if soil conditions are not stable. Disking or blending may be required to uniformly moisture condition soils used for engineered fill. Ponding or jetting compaction methods should not be allowed. Jetting for placement of the roughened channel rock interfill material may be used in a controlled manner to sluice materials between the roughened channel rock fill while not plugging the underlying transition layers. All site preparation and fill placement should be observed by the geotechnical engineer. It is important that during the stripping and scarification processes, a geotechnical engineer representative should be present to observe whether any undesirable material is encountered in the construction area and exposed soils are similar to those encountered during the geotechnical site exploration.

6.3 Temporary Excavations

All excavations should comply with applicable local, state, and federal safety regulations including the current Occupational Safety and Health Administration (OSHA) Excavation and Trench Safety Standards. Construction site safety is the responsibility of the Contractor, who is also solely responsible for the means, methods, and sequencing of construction operations. The Contractor should prepare a temporary support plan for review by the design team to check that it does not impose additional loads on the installed work that was not anticipated during design.

6.3.1 Excavation

Slope height, slope inclination, or excavation depths (including utility trench excavations) should not exceed those specified in local, state, and/or federal safety regulations (e.g., OSHA Health and Safety Standards for Excavations, 29 Code of Federal Regulations Part 1926, or successor regulations). Such regulations are strictly enforced, and, if not followed, the owner, Contractor, or earthwork and utility subcontractors could be liable for substantial penalties. Based on interpretation of the OSHA regulations and the findings from this investigation, the existing near surface soils will likely be predominantly Type C soils. For planning purposes, excavations less than about 20 feet deep should have

maximum allowable slopes of 1.5H:1V. The actual OSHA soil type classification and sloping requirements should be determined by the Contractor's responsible person based on the materials exposed during construction. OSHA requires that excavated slope heights in excess of 20 feet be designed by a professional engineer.

Underground utilities should be located above a 1H:1V plane projected downward from the bottoms of new foundations to avoid undermining the foundations during the excavation of the utility trench. Given the permeable nature of the foundation soils, the groundwater elevations are anticipated to vary with the water surface elevation in the slough. Excavations are anticipated to extend below the low water slough elevation and will require dewatering to localize lower groundwater below the excavation subgrade elevation. Groundwater should be lowered to at least 2 feet below the bottom of any excavation.

6.3.2 Dewatering

Groundwater control may be required for the site excavations, depending on groundwater conditions encountered during construction, or to address seepage zones and/or locally perched groundwater. The extent of dewatering required and the dewatering system used will depend on the excavation support system selected and the Contractor's construction methods. The design, installation, and maintenance of all necessary systems for groundwater control and dewatering during construction are the responsibility of the Contractor, including testing of groundwater and permitting for proper disposal. Large scale dewatering outside excavation areas should not be permitted. Existing structures that may be sensitive to settlement induced by dewatering should be monitored during construction.

6.3.3 Pipe Culvert Backfill

Pipe backfill material should be placed under, on the sides, and a minimum of 1 foot above any pipes. The requirements of the pipe backfill material should be provided by the pipe supplier to include uniform and adequate support for the applied loads. The pipe backfill should consist of fill that is well-graded, free-draining, angular, and durable material. Pipe backfill material should be placed in loose horizontal lifts less than 8 inches and compacted to at least 90 percent relative compaction per ASTM D1557. Alternatively, the pipes can be backfilled with flowable controlled low strength material that is placed around the pipe. Alternatively, the pipes can be backfilled with flowable controlled low strength material that is placed around the pipe. Care must be taken to place the controlled low strength material in controlled sections and not allow it to flow down a significant length of the pipeline to provide full haunch support.

6.3.4 Trench Backfill

Trench backfill material will be placed above the pipe backfill material. Trench backfill material should be placed in 8-inch loose lifts, moisture conditioned to 0 to +3 percent above optimum moisture content and compacted to at least 90 percent of relative compaction per ASTM D1557. The upper 3 feet of trench backfill should be compacted to at least 95 percent of relative compaction per ASTM D1557.

6.4 Shallow Foundations

6.4.1 Allowable Bearing Capacity

The proposed Electrical/Control/Equipment Building can be supported by conventional spread and continuous wall foundations founded on engineered fill. Continuous perimeter foundation can be designed with an allowable bearing pressure of 2,000 pounds per square foot (psf) provided site preparation is prepared as recommended in Section 6.2.

A one-third increase may be applied to this value when considering the effects of transient loads such as wind or seismic. The recommended net allowable bearing pressure includes a safety factor of at least 3 for shear failure of the foundation soils.

Where footings are located near and parallel to existing or new underground utilities, the footings should extend below a plane projected at a slope of 1H:1V upward from the utility to avoid surcharging the underground utility. Where footings are located adjacent to below-grade structures or near major underground utilities, the footings should extend below a 2H:1V plane projected upward from the structure footing or bottom of the underground utility to avoid surcharging the below grade structure and underground utility.

6.4.2 Lateral Resistance

Resistance to lateral loads are developed from the frictional resistance of the footing bottom with the subgrade and from passive resistance against the buried portions of walls and footings. The buildings slab-on-grade should not be considered for providing lateral resistance since it will be structurally separated from the building footing foundations. A coefficient of friction of 0.4 may be used for the friction between the footing bottoms and the underlying soil. This coefficient of friction can be increased to 0.55 when evaluating the earthquake loading condition. These friction coefficients can be mobilized with lateral deformations of less than ¼-inch.

An allowable passive pressure equal to an equivalent fluid pressure of 350 pounds per cubic foot (pcf) can be used for lateral resistance against the buried portions of walls and footing foundations. This passive resistance value has a factor of safety of over 2. However, relatively large deformations would be required to mobilize the ultimate passive resistance. In order to limit deformations to generally accepted magnitudes of about ½-inch, we recommend that this passive resistance be considered an ultimate value. The upper 1 foot of soil should be ignored for passive resistance unless it is confined by a pavement or slab.

A HDR geotechnical representative should observe the foundation excavations prior to placing reinforcing steel and concrete, to check that they are founded on appropriate material. All foundation excavations should be cleaned of loose material and should be free of water prior to steel and concrete placement.

6.5 Mat Foundations

Mat foundations may be designed using a preliminary modulus of subgrade reaction (K) of 30kips per square foot per inch of deflection (for a 1 square-foot bearing plate). The modulus should be adjusted for the actual slab size using appropriate formulas or software.

6.6 Slab-on-Grade

The backfill against the slab should be compacted and graded to shed away from the slab perimeter using non-expansive soil. The slab should be found on minimum 18 inches compacted fill consisting of Aggregate Subbase (AS). If moisture migration through the slabs is undesirable, a moisture barrier and capillary break may be provided between the slab and the compacted layer of engineered fill. We recommend that the system consist of 4 inches of free-draining gravel, such as 3/4-inch, clean, crushed, uniformly graded gravel with less than 3 percent passing No. 200 sieve, or equivalent, overlain by a relatively impermeable vapor retarder that is placed between the subgrade soil and the slab. The vapor retarder should be at least 10-mil thick and should conform to the requirements for ASTM E 1745 Class A, B, or C Underslab Vapor Retarders.

Minor cracking and maintenance problems may be anticipated for slab-on-grade construction; however, separation from structure with concentrated loads and keeping the subgrade moist during construction can greatly reduce the amount of eventual cracking. Prior to concrete placement, any dry cracked subgrade should be sealed with appropriate watering or should be excavated and re-compacted. These conditions and construction procedures should be observed by the Project Geotechnical Engineer.

6.7 Deep Foundations

6.7.1 Axial Capacity

As discussed above, recommended foundation types include driven piles and Cast-in-Drilled Hole (CIDH) piles. In addition, the structure coup be supported on auger cast displacement piles (ADCP) if coupled with ground improvement to provide additional lateral support as the pile reinforcement is inserted after the pile concrete has been placed. Supplemental geotechnical recommendations for ADCP can be provided if this pile type is being considered. Figures 15 and 16 are plots of ultimate axial capacities for 24-inch diameter open ended pipe piles and 30-inch CIDH piles, respectively. Plots of axial capacity pipe piles where and internal plug does or does not form within the pile. For the condition where there an internal plug does form, it is assumed that the plug only forms below Elevation 90 feet, where the pile is driven into the underlying older alluvium. Due to the potential for liquefaction induced settlement downdrag forces should be included assuming the soil above Elevation 100 ft induces downdrag forces. Recommendation for soil properties to analyses pile lateral capacity is provided in Table 6-1. A specialty Contractor should be retained to construct the piles.

6.7.2 Group Effects

The recommended axial capacity applies to single, isolated piles with center-to-center spacings of at least three effective pile diameters. To avoid group effects, all piles should be spaced at a minimum center-to-center spacing of 2.5 times the pile diameter. Pile interaction factors for pile groups are provided in Table 5. Input parameters for Lpile Analyses for the Fish Passage Structure and the Weir are provided in Table 6 and Table 7.

Table 5. P-Multipliers[1] for Fish Passage Structure Lateral Pile Design using LPILE

Center-to-Center Pile Spacing (Number of Pile Widths, B)	Pile Row 1	Pile Row 2	Pile Row 3 or higher
3	0.75	0.55	0.4
4	0.875	0.7	0.55
5	1	0.85	0.7
6	1	0.93	0.8
7	1	1	0.9
8	1	1	1

P-multipliers are adapted from Section 10.7.2.4 of the AASHTO LRFD Bridge Design Specifications, Eighth Edition (AASHTO 2017) with California Amendments (Caltrans 2019). The pile cap example at right is taken from this reference.

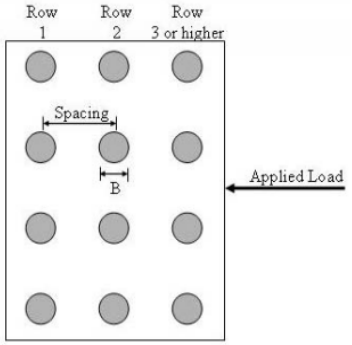


Table 6. L-pile Input Parameters Fish Screen Intake Structure

Layer and USCS Soil Type	Elevation (Feet, NAVD88)	Unit Wight (pcf)		Cohesionless Parameters	Cohesive Parameters	P-Y Curve Model for LPILE/Group
		Total	Effective	Friction Angel (degrees)	Undrain Strength (psf)	
1. SM	131 to 125	120	57.6	45	--	Sand (Reese)
2. CL/SM/ML	125 to 115	115	52.6	30		Soft Clay (Matlock)
3. SM/SC	115 to 95	120	57.6	33		Sand (Reese)
4. Silt	95 to 61	115	52.6	40	--	Stiff Clay (Reese)

Table 7. L-pile Input Parameters Weir and Fish Passage Structures

Layer and USCS Soil Type	Elevation (Feet, NAVD88)	Unit Wight (pcf)		Cohesionless Parameters	Cohesive Parameters	P-Y Curve Model for LPILE/Group
		Total	Effective	Friction Angel (degrees)	Undrain Strength (psf)	
1. ML	135 to 131	120	57.6	32	--	Sand (Reese)
2. SM/ML	125 to 115	115	52.6	30	--	Soft Clay (Matlock)
3. SM/SC/GC	115 to 92	120	57.6	28	--	Sand (Reese)
4. Silt	95 to 61	115	52.6	36	--	Stiff Clay (Reese)

6.7.3 Construction Considerations

All piles should be driven vertically to refusal in order to develop adequate vertical pile capacities. Preliminary refusal criteria should be developed. To limit the potential for damaging the piles, it is the responsibility of the pile-driving Contractor to select a pile-driving hammer appropriate to the soil conditions at the site. HDR recommends that a review of the Contractor-proposed hammer type be conducted. In addition, the method of handling and picking up of the piles should be established by the pile-driving Contractor and should be reviewed by HDR prior to construction.

HDR recommends that the final foundation plans and specifications be reviewed to confirm the recommendations presented in this report have been properly incorporated into the contract documents. To confirm that piles are installed to adequate depths and they have encountered sufficient resistance to develop the required supporting capacities, HDR also recommends that qualified staff observe the driving of all piles at the site on a full-time basis.

6.8 Retaining Structures

The fish passage structure and retaining walls are being designed as restrained walls that are not free to move or rotate. Therefore, at-rest pressures are being assumed for static loading conditions. To limit wall pressure, walls should be backfilled with compacted native granular soils. Based on the available geotechnical information at the overall Project area, onsite soil available for borrow and structure backfill is predominantly sandy sit and silty sand. High plasticity soil should be avoided for use as wall backfill. Refer to Section 6.7.4 for specific requirements for backfill. On this basis the walls may be designed assuming the following soil parameters and earth pressure coefficients appropriate for compacted lean clay backfill:

- Backfill total unit weight of 125 pcf above the phreatic surface for each design case.
- Backfill buoyant unit weight of 62.6 pcf below the phreatic surface.
- At-rest earth pressure coefficient (K_O) of 0.60 (for static loading).
- Active earth pressure coefficient (K_A) of 0.43 (for seismic loading).

Under static loading conditions, the walls are being designed to resist hydrostatic and lateral earth pressure, as well as additional loads caused by surcharging. Under seismic conditions, the active pressure is combined with uniform seismic pressure increment of 6H psf, where H is the height of the backfill. This seismic earth pressure was developed using the design peak ground acceleration of 0.28g. Earth pressure diagrams for design of the walls are presented Figure 17 and Figure 18 for static and seismic loading conditions, respectively.

Because the walls will be backfilled with granular soils, back drainage may not be required.

6.9 Filter Design

The Project includes constructing a roughened rock channel fill immediately downstream of the weir with strategically placed boulders to allow fish to swim up to the fish ladder (Figure 19). The roughened channel will be comprised of open graded rock infilled with finer coarse-grained material (rock infill material). The rock infill material will be washed into the rock fill as it is placed. Filter and transition layers will be required beneath the rock fill to prevent foundation material from being transported/eroded into the rock fill that could undermine the weir. A sheetpile cutoff will be located directly upstream of the weir, extending into the underlying older alluvium layer that will prevent lateral migration of material into the rock fill. The filter layer and transition material will need to extend across the entire bottom of the channel and up both banks to prevent material loss due to end around seepage flow through the permeable weir abutments/creek bank. These calculations determine gradations of filter material that are compatible with the natural granular stream bed. Riprap will be placed on the slopes as erosion protection and require a filter layer to prevent loss of soil into more riprap material. However, a geotextile with a sufficient apparent opening size (AOS) will be required to retain the filter layer and have sufficient permeability to allow water to flow into the channel.

6.9.1 Base Soil and Rock Infill Gradation

Gradation for the base soil (native riverbed soil) was determined using the available grain size distribution test data for the material near EL 100 feet from the existing and supplemental explorations. The base soil gradation bounds were selected based on the five grain size distribution test results shown in Table 8. The gradation for the rock infill material is provided in Table 9.

Table 8. Base Soil Gradation Limits:

Particle size (mm)	Sieve #	Percent Passing					Base Soil Gradation Limits	
		BI-1 6-SS	BI-2 2-SS	BI-3 4-SS	BD-2 3-SS	HDR-B2 S-10	Upper Bound	Lower Bound
37.5	--	--	--	--	--	100	100	100
25	--	100	--	--	100	95	90	100
19.05	--	66	--	100	78	86	--	--
12.5	--	58	--	88	69	71	--	--
9.5	--	50	100	83	66	62	--	--

Particle size (mm)	Sieve #	Percent Passing					Base Soil Gradation Limits	
		BI-1 6-SS	BI-2 2-SS	BI-3 4-SS	BD-2 3-SS	HDR-B2 S-10	Upper Bound	Lower Bound
4.75	4	38	97	67	55	46	40	97
2.36	8	--	--	--	--	34	--	--
2	10	28	95	47	47	--	30	95
1.18	16	--	--	--	--	27	--	--
0.6	30	--	--	--	--	20	--	--
0.425	40	16	83	18	37	--	16	85
0.3	50	--	--	--	--	15	--	--
0.15	100	--	--	--	--	11	--	--
0.075	200	8	42	7	30	9	0	45

Table 9. Rock Infill Gradation and Transition Layer Gradation

Particle size (mm)	Sieve #	Rock infill Material (original), % passing	Proposed Transition Layer Gradation (%)
		Upper Bound	Lower Bound
63	--	100.0	100.0
50	--	80.0	80.0
25.0	--	65.0	65.0
19.0	--	50.0	50.0
4.75	4	35.0	16
2.36	8	--	10
1.00	18	--	5
0.425	40	16.0	--
0.075	200	7.0	3
0.053	270	--	--
0.0001	--	0.0	0.0

6.9.2 Methodology and Criteria

Filter and transition layer gradation specifications were determined based on the range of particle sizes of the native riverbed soil (Table 1) using criteria by the USACE (1994) and the U.S. Bureau of Reclamation (Reclamation 2011). A spreadsheet tool developed by the Virginia Polytechnical Institute and State University (VPI 2004) was used to the evaluate the compatibility of candidate filter and drain material gradations relative to the respective base soils/materials (i.e., filter material with respect to the backfill soils and drain materials with respect to the filter material).

The need for a transition zone between the filter layer and the Caltrans Class III and Class V riprap selected for the Project. The Cistern-Ziems Chart from NCHRP Report 568 (NCHRP 2006) was used to calculate the transition layer gradation based on the ration of the D_{50} ratio of the riprap to transition layer material. The report includes a flow chart for selection of geotechnical filter fabric that is based on the 0_{95} value which is the

aperture size where 95 percent of the openings are smaller and the size is equivalent to the AOS of the geotextile filter. The AOS is based on the filter layer gradation which is a blend of coarse sand and fine gravel. There are available geotextile fabrics with adequate AOS. Use of filter fabrics along both the Mormon Slough and Old Calaveras River are comprised of overbank materials consisting of fine sands and silts. The use of a geotextile alone would not meet the requirements for a filter.

6.9.3 Recommendations

The calculation results indicate the standard ASTM C 33 sand gradation does not provide an adequate gradation range to act as a filter for the channel bed. A 40 to 60 percent blend of ASTM C33 -02a Size 9 sand and AASHTO M 43-99 Size #57 sand (by weight) provides a gradation that meets filter criteria with the gradation presented in Table 10. The Rock infill material gradation was found to satisfy the transition zone gradation; however, the gradation provided in Table 9 is recommended to provide a lower coefficient of uniformity (Cu = d60/d10) by limiting the percent finer than the #4 sieve to 16 percent and the percent passing the #200 sieve to 3 percent. This will provide more stable gradation for transport and placement. The filter design calculation for Base Soil to Filter and for Rock Infill to Filter is provided in Appendix F.

The Class III and Class V Riprap have a D_{50} of 12.5 and 18 inches respectively per the Caltrans new riprap classes. Using the Cistern-Ziems Chart from NCHRP Report 568 and assuming the riprap and riprap bedding layers have a Cu = 2.4 and 2.0, the A value of the D_{50r} to D_{50b} (riprap:base layer) is 10. The D_{50b} of the Class III and Class V riprap is 1.25 and 1.8 inches, respectively. Therefore, a coarser backing layer is needed. For both Class III and Class V riprap the D_{50} of the backing layer should be a minimum of 8 inches in thickness and a gradation $1.5 < D_{50} < 2$ and the backing layer can replace the upper 8 inches of the transition layer. If a geotextile is used in lieu of a transition/backing layer, a geotextile a fabric with an AOS of 0.2 mm (#70 sieve) would be sufficient permeability for placement over the filter layer. It should be a woven geotextile with sufficient strength to not be damaged by the proposed placement method.

Table 10: Filter Material Gradation:

Particle size (mm)	Sieve size	ASTM C33 - 02a	Table 2: Size # 9	AASHTO M 43-88	Table 1: Size # 57	Blended Mix	
		Percent Passing		Percent Passing		Percent Passing	
		Upper Bound	Lower Bound	Upper Bound	Lower Bound	Upper Bound	Lower Bound
50.0	--	(100.0%)	(100.0%)	(100.0%)	(100.0%)	100%	100%
37.5	--	--	--	100.0%	(100.0%)	--	--
25.0	--	(100.0%)	(100.0%)	95.0%	100.0%	97%	100%
12.5	--	(100.0%)	(100.0%)	25.0%	60.0%	55%	76%
9.5	--	100.0%	--	--	--	--	--
4.75	4	85.0%	100.0%	0.0%	10.0%	34%	46%
2.36	8	10.0%	40.0%	0.0%	5.0%	4%	19%
1.40	14	--	--	(0.0%)	(0.0%)	--	--
1.18	16	0.0%	10.0%	(0.0%)	(0.0%)	0%	4%
0.600	30	--	--	(0.0%)	(0.0%)	--	--
0.300	50	0.0%	5.0%	--	--	--	--
0.075	200	(0.0%)	(0.0%)	(0.0%)	(0.0%)	0%	0%
0.0001	--	0.0%	0.0%	0.0%	0.0%	--	--

6.10 Additional Geotechnical Services

A geotechnical representative should review the geotechnical aspects of the Project plans and specifications to check for conformance with the intent of the recommendations provided in this report. During construction, a geotechnical representative should observe the following:

- Soil conditions exposed by foundation excavations, to check for consistency with those encountered during explorations
- Subgrade preparation prior to placement of engineered fill
- Engineered fill placement and compaction
- Pile installation

7 Limitations

This report has been prepared by the design team for specific application to the Bellota Fish Passage Project in accordance with generally accepted geotechnical engineering practice. No warranty, express or implied, is made. The analyses and recommendations submitted are based on the data available to HDR at the time of this geotechnical investigation and on the data collected during the field investigation. This report does not reflect subsurface soil variations that may occur between the locations of the explorations or variations in groundwater conditions which may occur over a period of time. Variations in conditions may become evident during construction, at which time re-evaluation of the conclusions may be necessary. In the event the Project design changes

after the final report is submitted, the recommendations should be reviewed and possibly modified by HDR's .

Historical explorations and testing were not performed by HDR, and HDR cannot vouch for the accuracy of data and information obtained by others. The historic explorations and subsurface data by others are presented herein to include data relevant to this Project in one document. Data by others should not be relied upon unless the originator of that data is available to confirm its accuracy.

This geotechnical study did not include an investigation regarding the existence, location, or type of possible hazardous materials. If any hazardous materials are encountered during construction of the Project, the proper regulatory officials should be notified immediately.

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Appendix A. Figures

Figure 1. Site Location

Figure 2. Preferred Fish Passage Alternative

Figure 3. Boring Location Plan

Figure 4. Geologic Map

Figure 5. Seismic Hazard

Figure 6. FEMA Flood Map

Figure 7. Geotechnical Profiles – Plan View

Figure 8. Profile Section A-A'

Figure 9. Profile Section B-B'

Figure 10. Profile Figure C-C'

Figure 11. Slope Stability Model - Old Calaveras River

Figure 12. Static Stability Existing Conditions

Figure 13. Static Stability Slope Flattening

Figure 14. Seismic Stability (pseudo-Static K_h=0.15g)

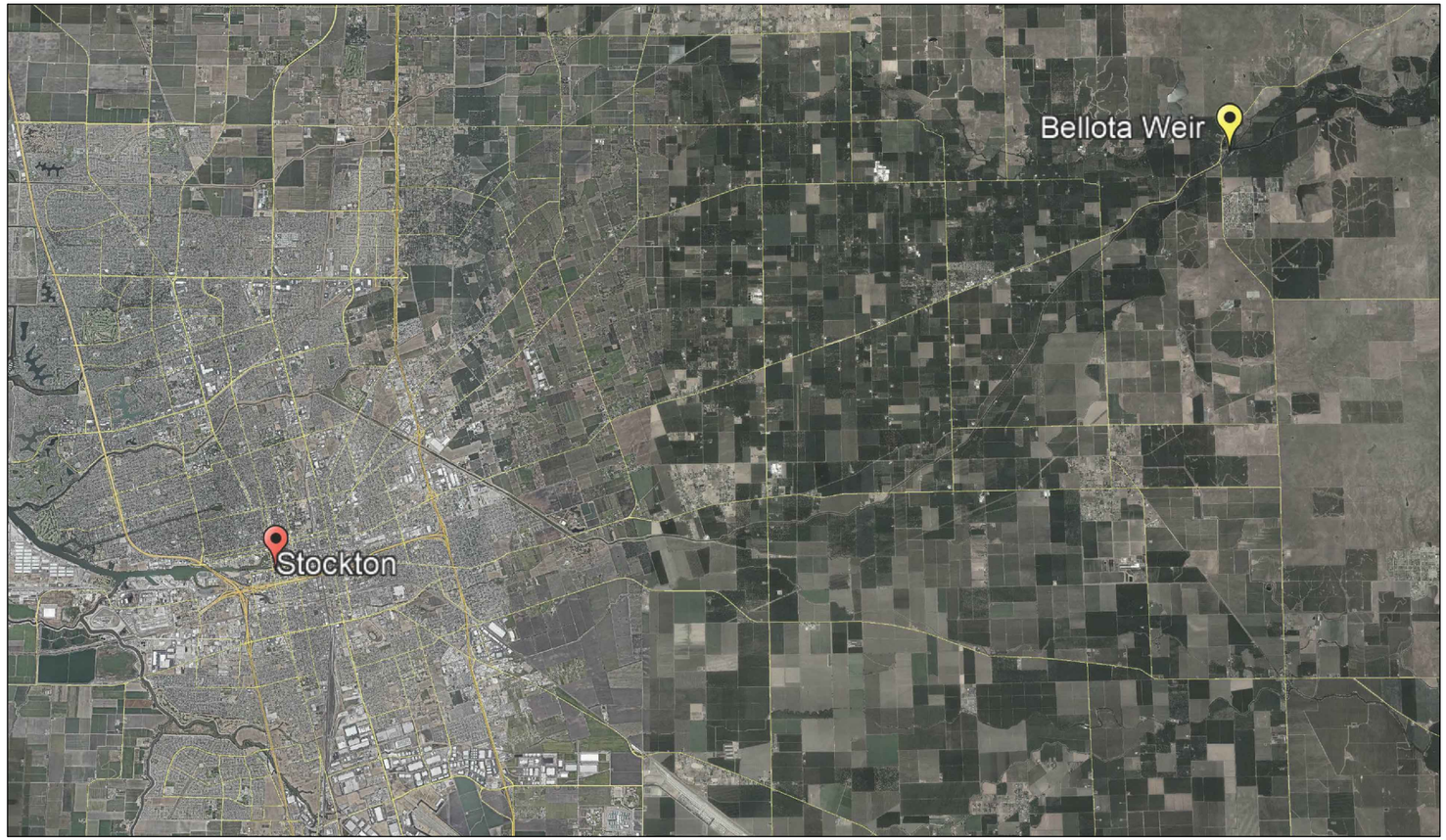
Figure 15. Ultimate Axial Pile Capacity Curve – 24-inch Pipe Pile 0.5-inch Thick

Figure 16. Ultimate Axial Pile Capacity Curve – 36-inch CIDH Pile

Figure 17. Static Lateral Earth Pressures on Structure Walls

Figure 18. Seismic Lateral Earth Pressures on Structure Walls

Figure 19. Roughened Channel Profile





12,000 0 12,000 24,000

SCALE: 1 INCH = 12,000 FEET

NOTE
Image from Google Earth Pro

Site Location

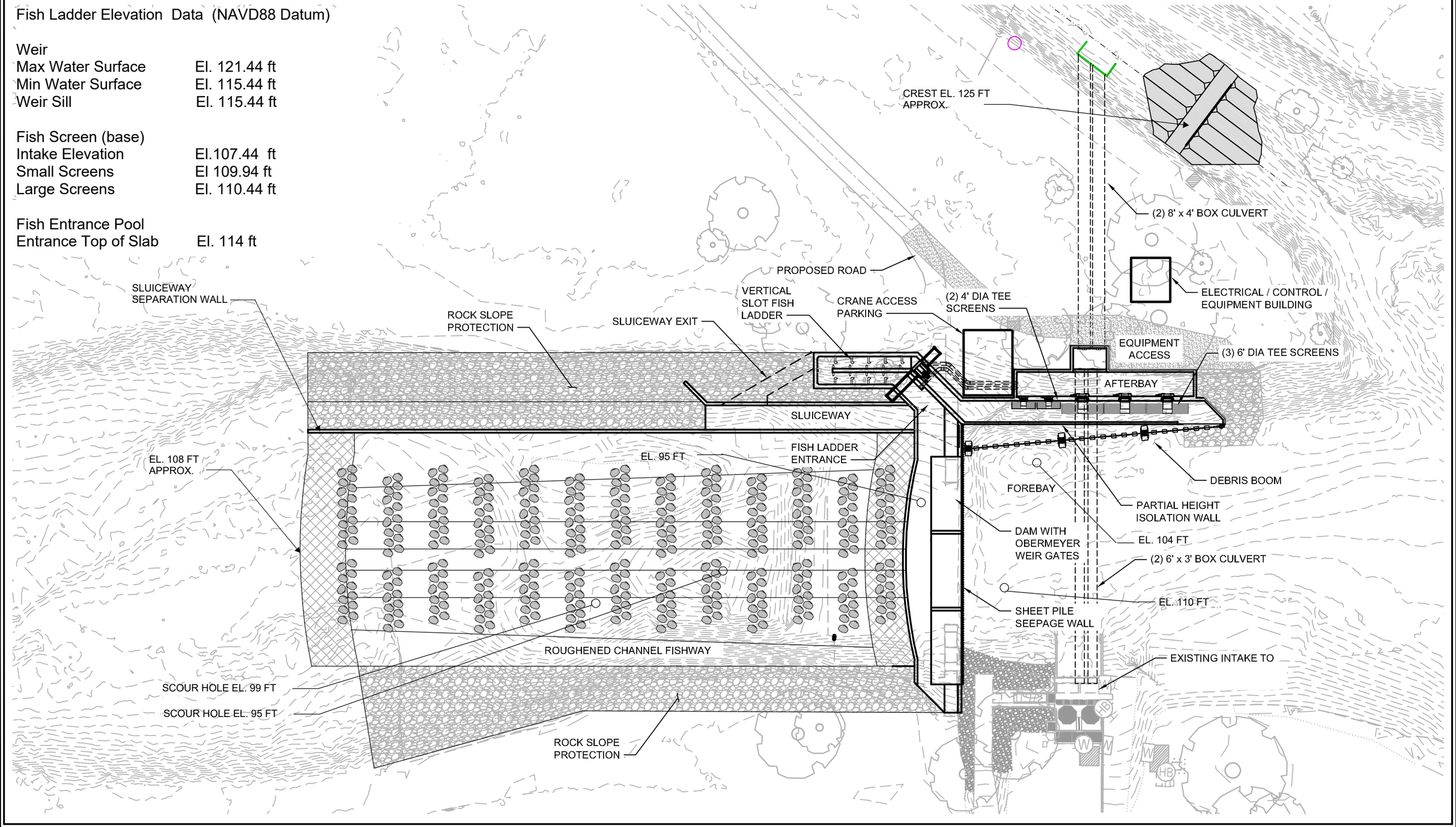
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APRIL 2021

Figure
1

Fish Ladder Elevation Data (NAVD88 Datum)

Weir	
Max Water Surface	El. 121.44 ft
Min Water Surface	El. 115.44 ft
Weir Sill	El. 115.44 ft
Fish Screen (base)	
Intake Elevation	El.107.44 ft
Small Screens	El 109.94 ft
Large Screens	El. 110.44 ft
Fish Entrance Pool	
Entrance Top of Slab	El. 114 ft



N

60 0 60 120

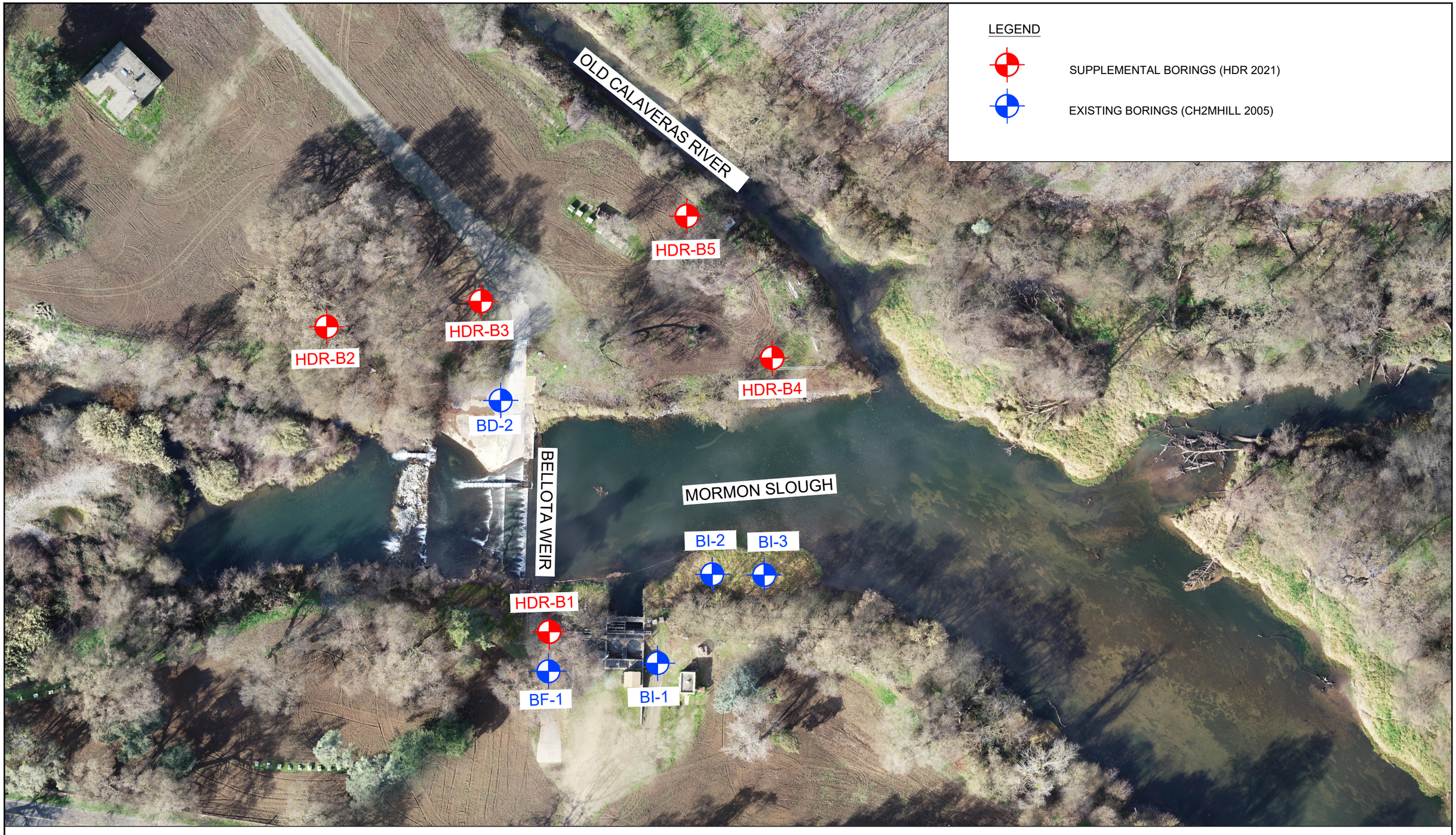
SCALE IN FEET

Preferred Fish Passage Alternative

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Figure
2



NOTE
EXPLORATION LOCATIONS SHOWN ARE APPROXIMATE.
EXPLORATION LOCATIONS WERE NOT SURVEYED.

75 0 75 150
SCALE: 1 INCH = 75 FEET

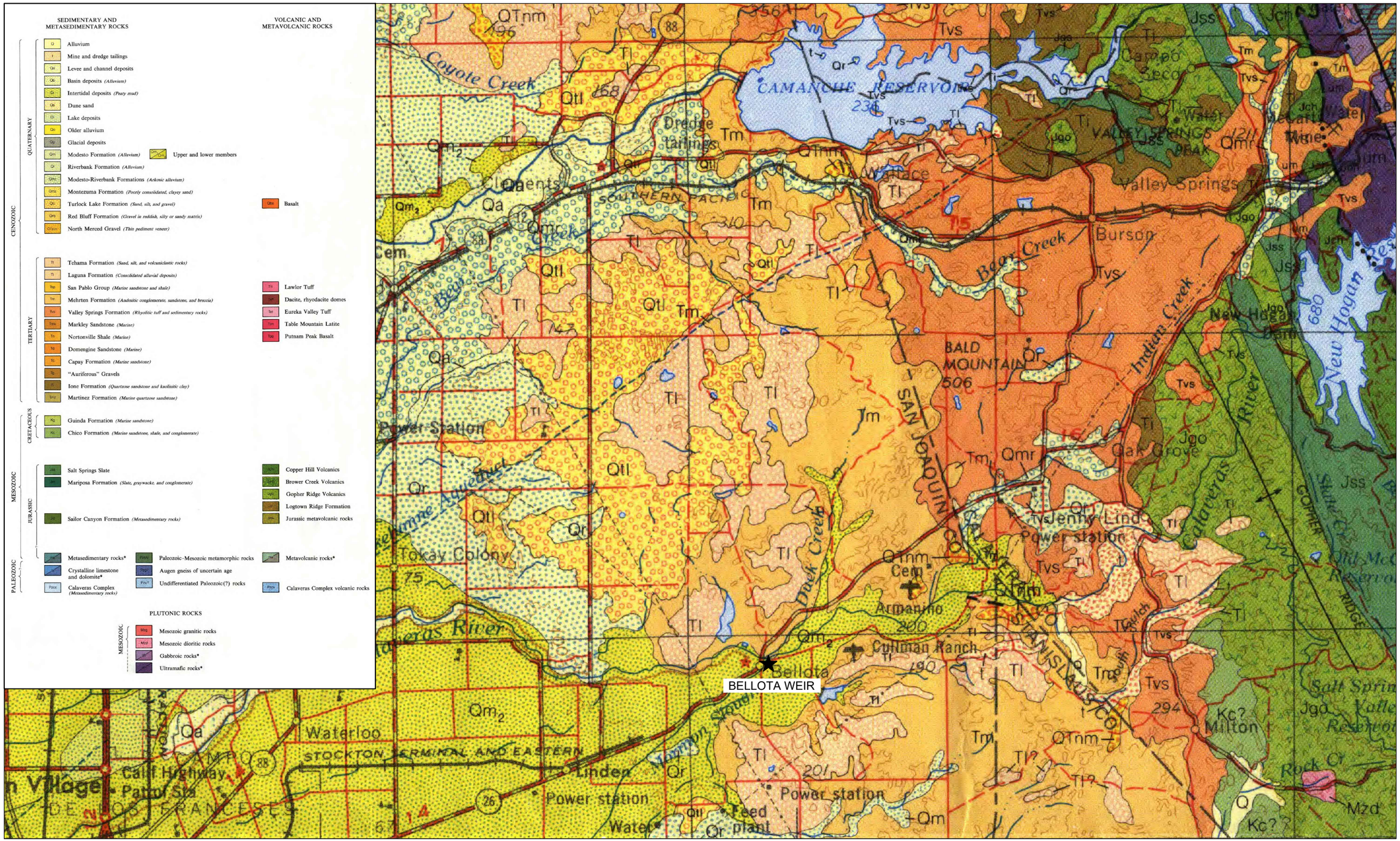
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BORING LOCATION

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Figure
3



NOTE

GEOLOGIC MAP SOURCE: GEOLOGIC MAP OF THE SACRAMENTO QUADRANGLE, CALIFORNIA, COMPILATION BY D.L. WAGNER, C.W. JENNINGS, AND E.J. BORTUGNO (1981)

10,000 0 10,000 20,000

SCALE: 1 INCH = 10,000 FEET

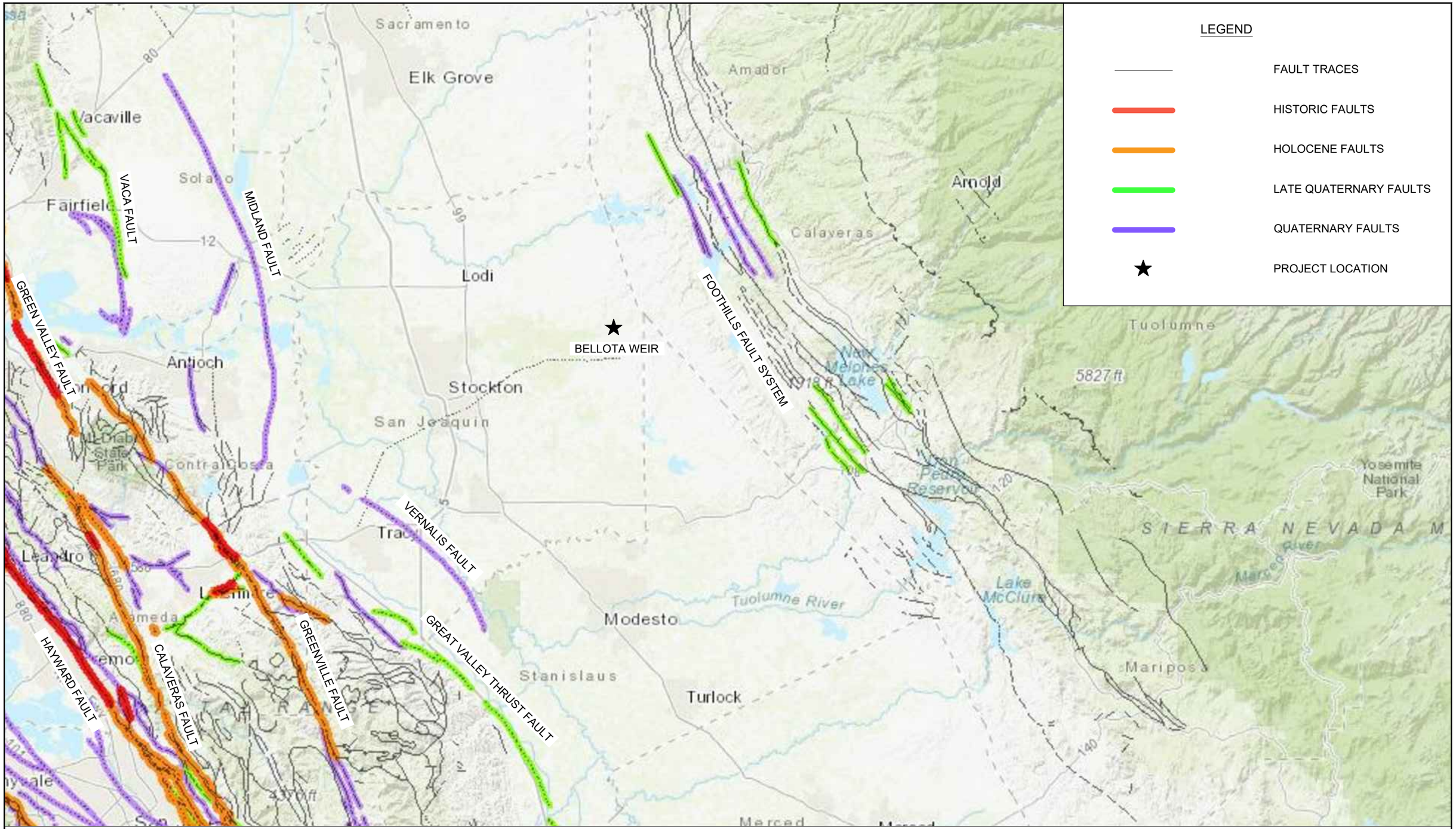
N

GEOLOGIC MAP

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Figure
4



NOTE

FAULT TRACES DATA FROM CALIFORNIA GEOLOGICAL SURVEY - FAULT ACTIVITY MAP OF CALIFORNIA.

50,000 0 50,000 100,000

SCALE: 1 INCH = 50,000 FEET

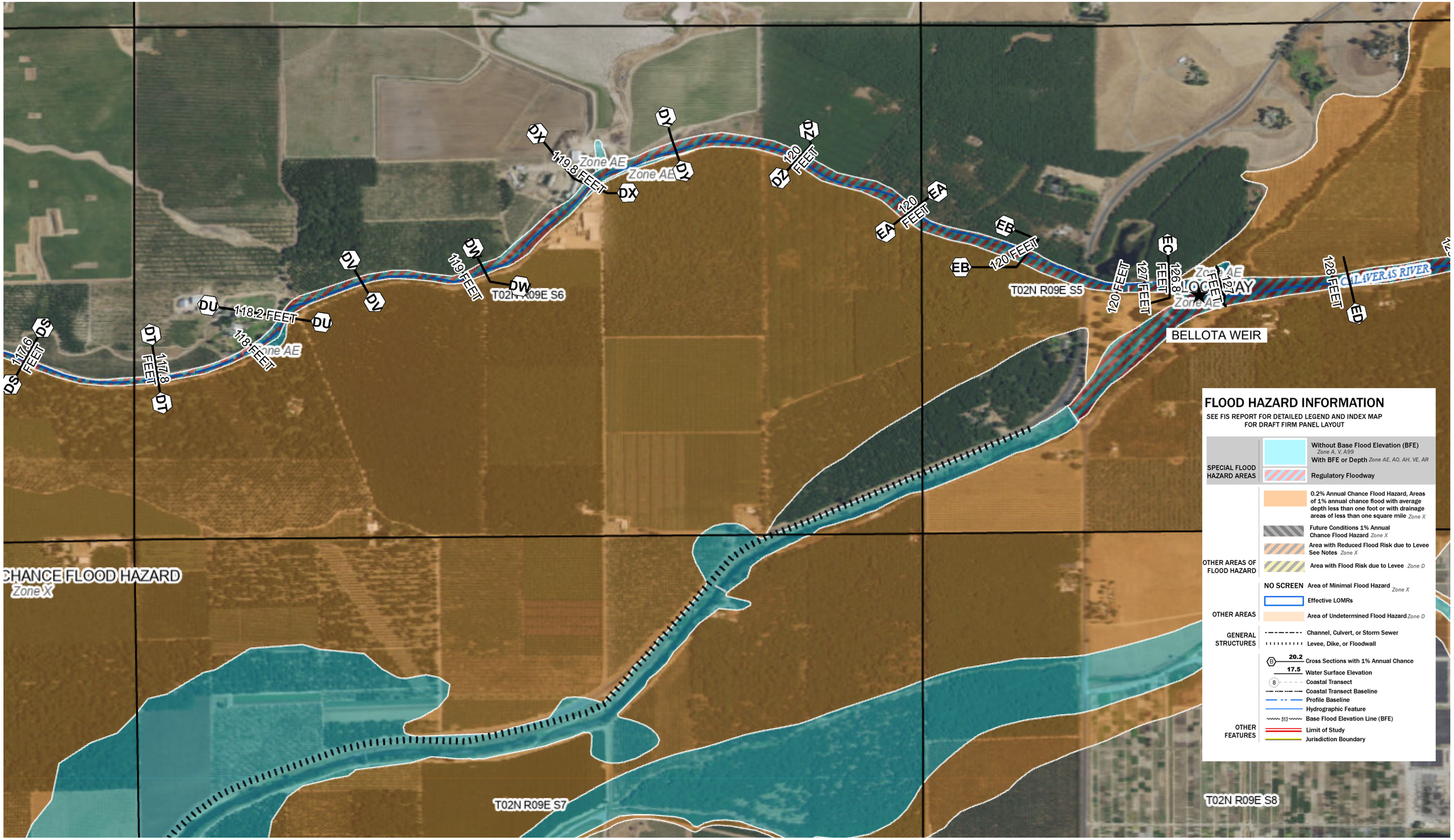
N

SEISMIC HAZARD

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Figure
5



NOTE

Flood Map from: FEMA's National Flood Hazard Layer (NFHL) Viewer

1000 0 1000 2000

SCALE: 1 INCH = 1000 FEET

N

FEMA Flood Map

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Figure
6

LEGEND

SUPPLEMENTAL BORINGS (HDR 2021)

EXISTING BORINGS (CH2MHILL 2005)

OLD CALAVERAS RIVER

A

B

C

C'

A'

B'

10+00

11+00

12+00

13+00

14+00

15+00

16+00

16+24.07

1+00

2+00

3+00

00

5+00

6+00

7+00

7+24.07

20+00

21+00

22+00

23+00

24+00

24+46.22

HDR-B1

HDR-B2

HDR-B3

HDR-B4

HDR-B5

BD-2

BF-1

BI-1

BI-2

BI-3

BELLOTA WEIR

MORMON SLOUGH

EXISTING WEIR AND FISH LADDER

EXISTING WEIR CENTERLINE

DIVERSION INTAKE STRUCTURE

NOTE

EXPLORATION LOCATIONS SHOWN ARE APPROXIMATE.
EXPLORATION LOCATIONS WERE NOT SURVEYED.

75 0 75 150

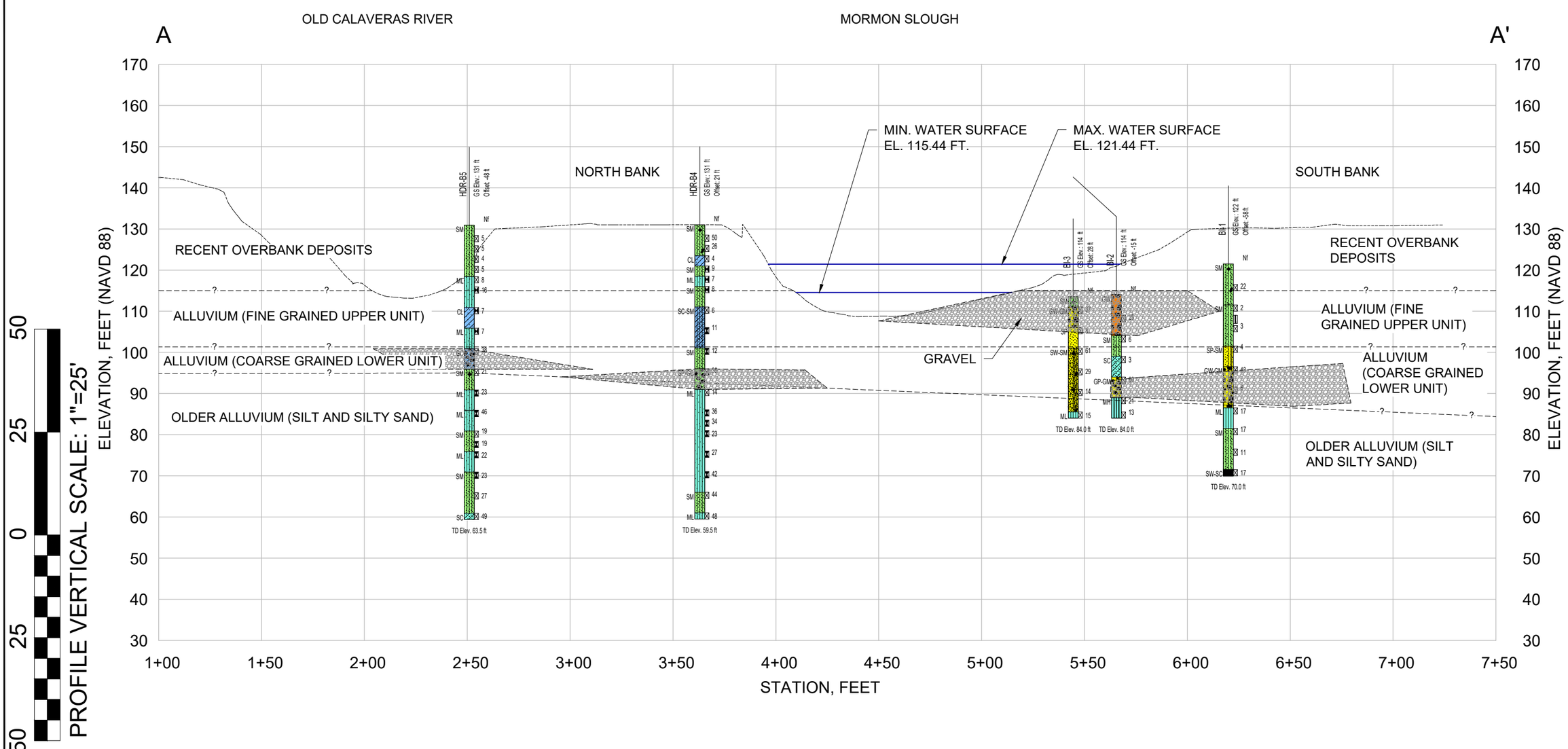
SCALE: 1 INCH = 75 FEET

GEOTECHNICAL PROFILES - PLAN VIEW

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Figure
7



NOTE

EXPLORATION LOCATIONS SHOWN ARE APPROXIMATE.
EXPLORATION LOCATIONS WERE NOT SURVEYED.

PROFILE HORIZONTAL SCALE: 1"=50'

100 50 0 50 100

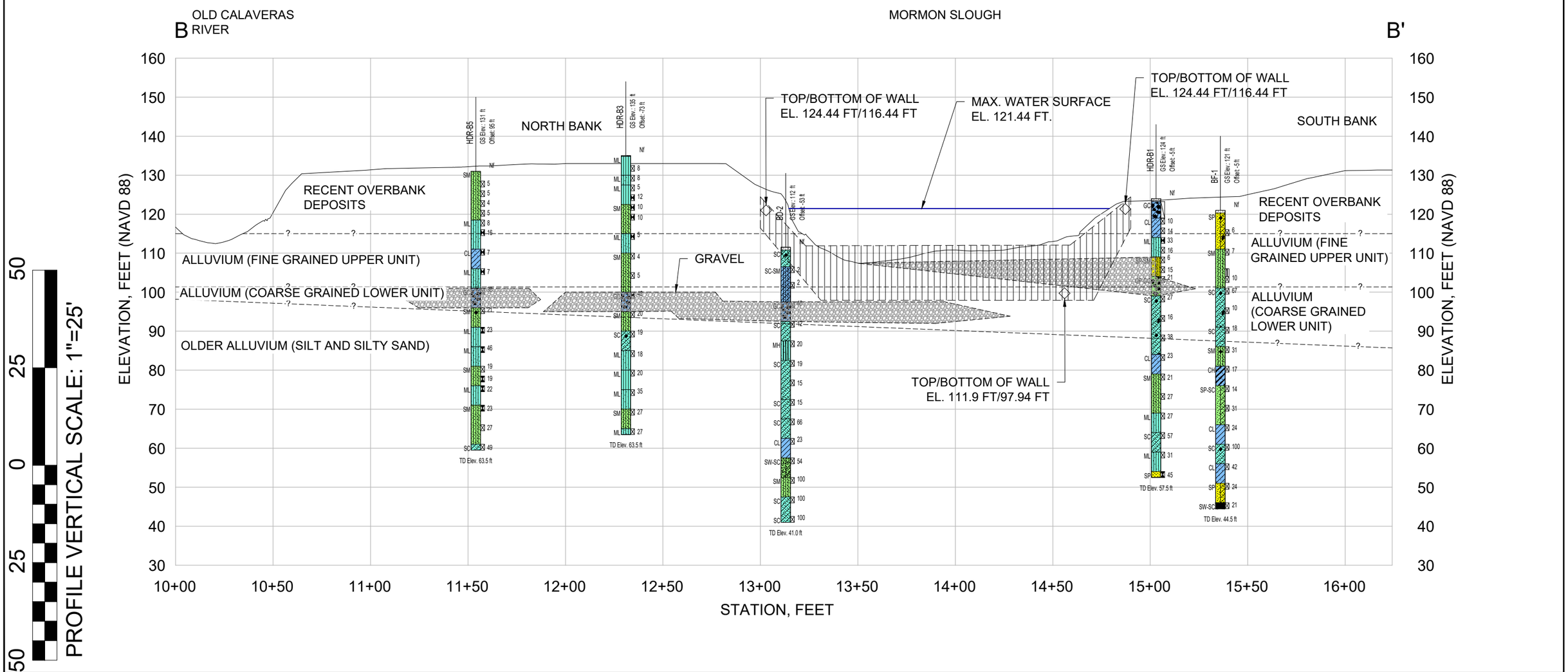
SCALE IN FEET

PROFILE A-A'
Looking Upstream 170 feet upstream of Weir

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Figure
8



PROFILE VERTICAL SCALE: 1"=25'

50 25 0 25 50

NOTE

EXPLORATION LOCATIONS SHOWN ARE APPROXIMATE.
EXPLORATION LOCATIONS WERE NOT SURVEYED.

PROFILE HORIZONTAL SCALE: 1"=50'

100 50 0 50

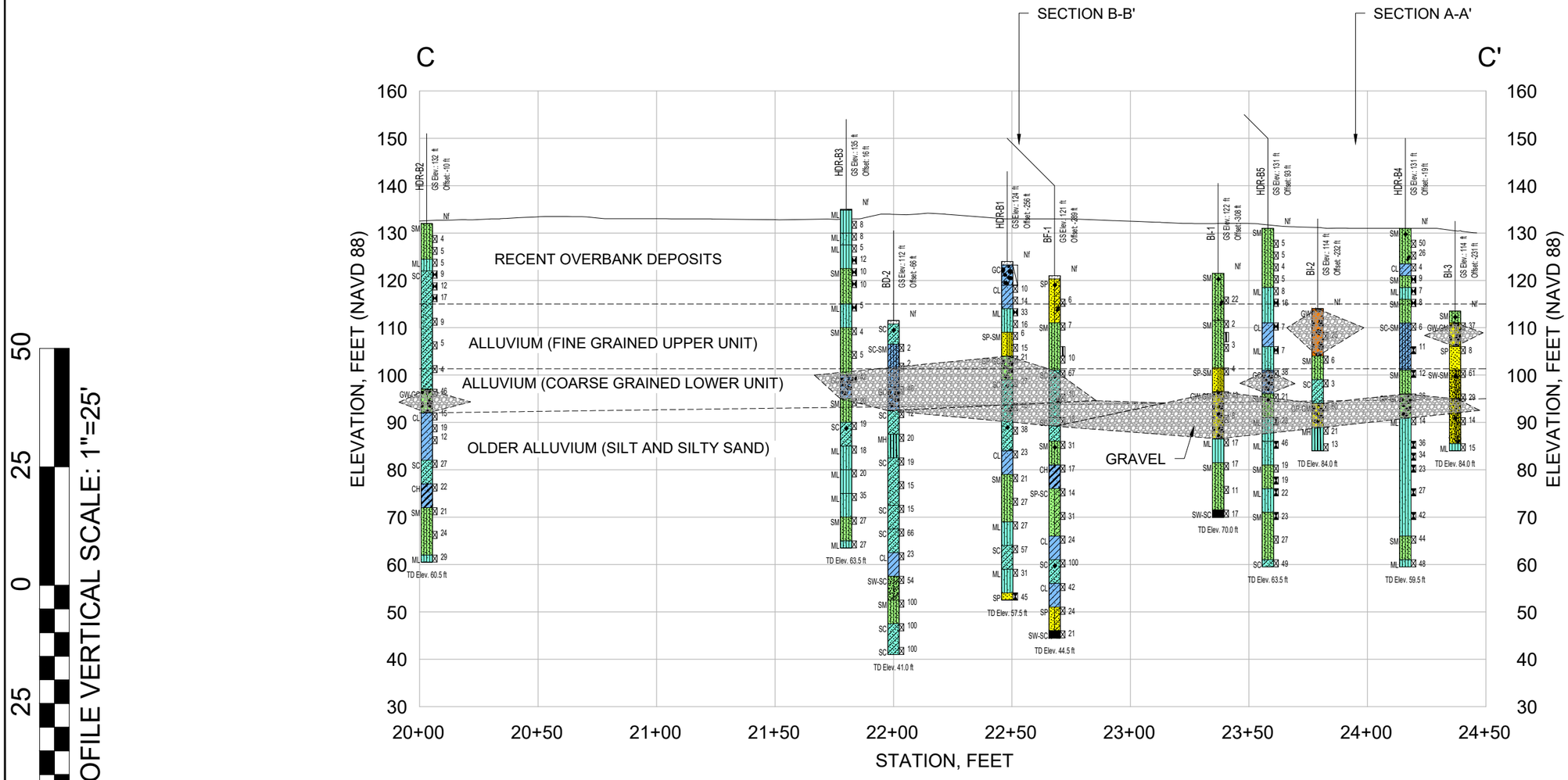
SCALE IN FEET

PROFILE B-B'
Looking upstream 28 Feet from Existing Weir

BELLOTA WEIR MODIFICATION PROJECT
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Figure
9



PROFILE VERTICAL SCALE: 1"=25'

50 25 0 25 50

NOTE

EXPLORATION LOCATIONS SHOWN ARE APPROXIMATE.
EXPLORATION LOCATIONS WERE NOT SURVEYED.

PROFILE HORIZONTAL SCALE: 1"=50'

100 50 0 50

SCALE IN FEET

PROFILE C-C'
NORTH BANK

BELLOTA WEIR MODIFICATION PROJECT
SAN JOAQUIN, CALIFORNIA

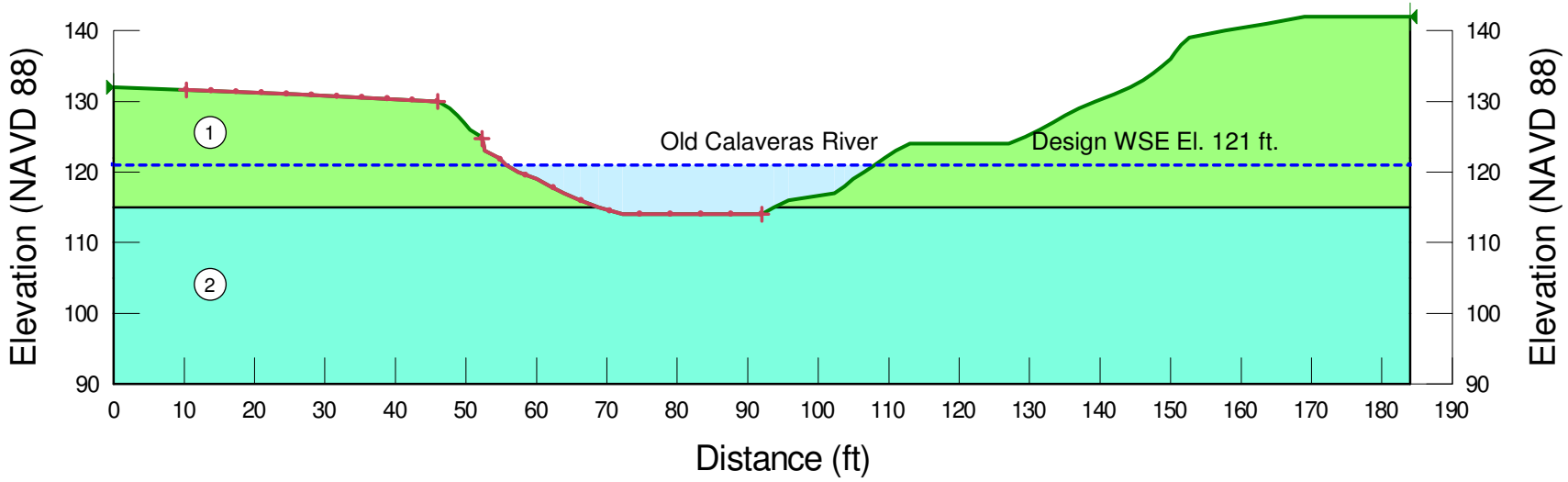
Date
APRIL 2021

Figure
10

Layer	Geology	Material Type	Total Unit Weight (pcf)	Effection friction angle (deg)	Effective Cohesion (psf)
1	Recent Overbank Deposits	SM/ML	115	30	-
2	Alluvium	SC/SM	120	32	-

Project Area Side

North Bank



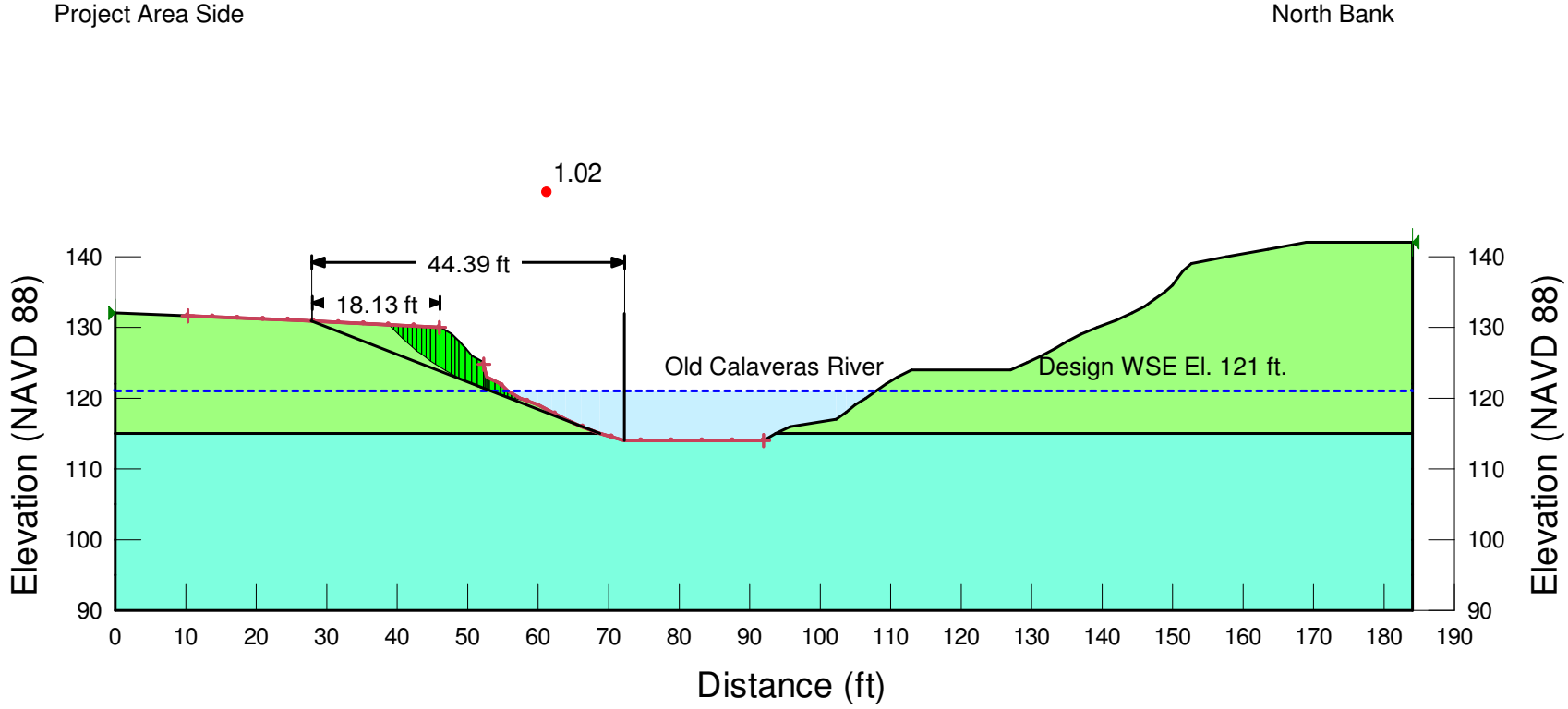
NOTES:

Bellota Weir Modification Project

Slope Stability Model – Old Calaveras River

April 2021

FIGURE 11



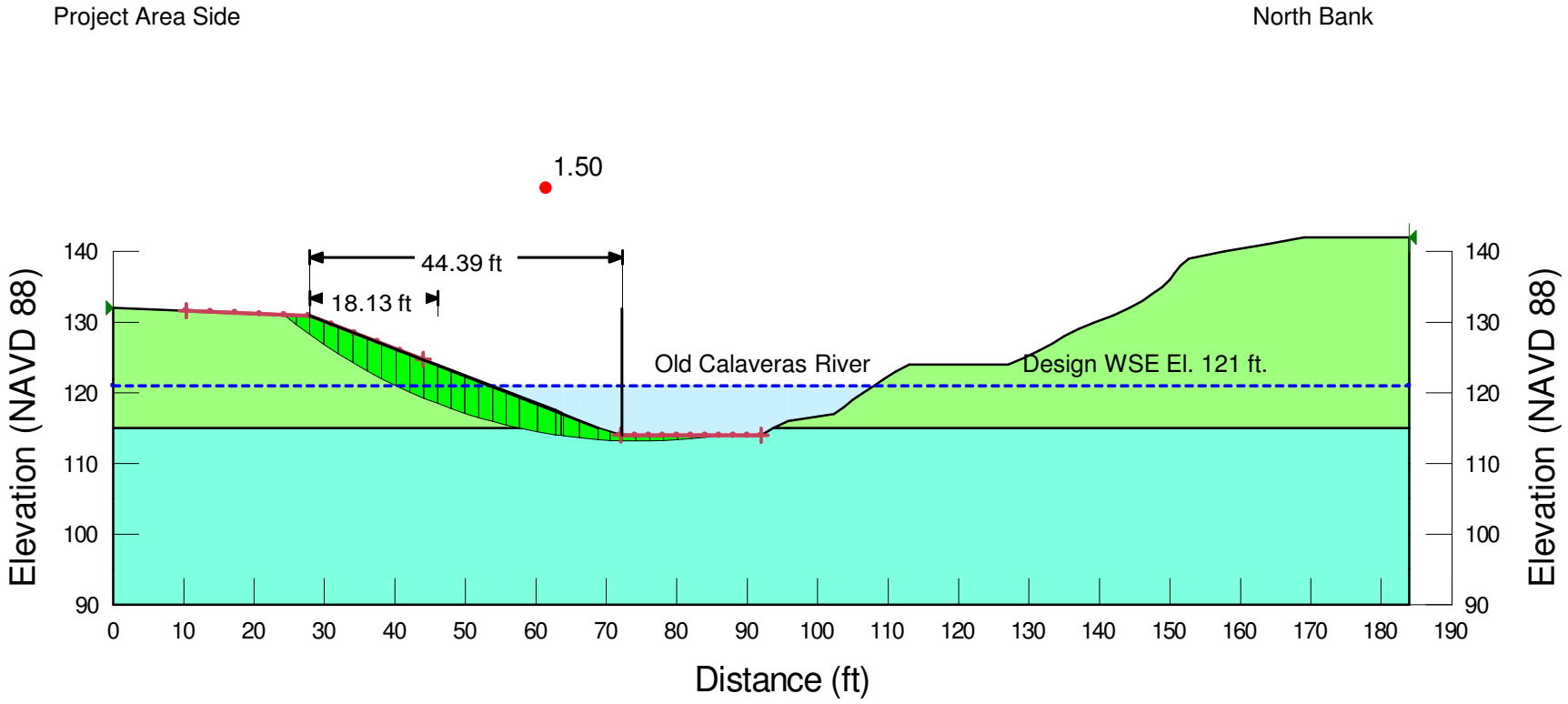
NOTES:

Bellota Weir Modification Project

Static Stability Existing Conditions

April 2021

FIGURE 12



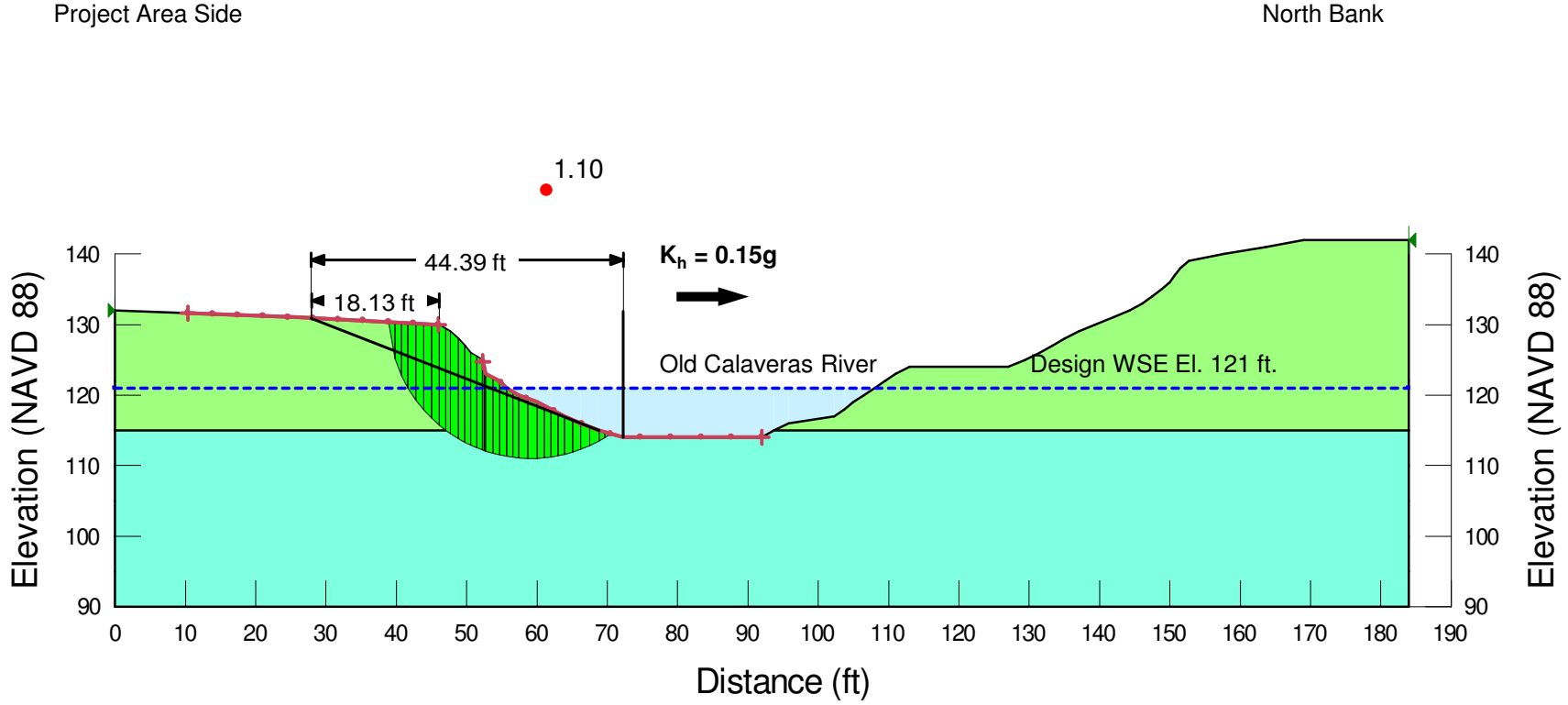
NOTES:

Bellota Weir Modification Project

Static Stability Slope Flattening

April 2021

FIGURE 13



NOTES:

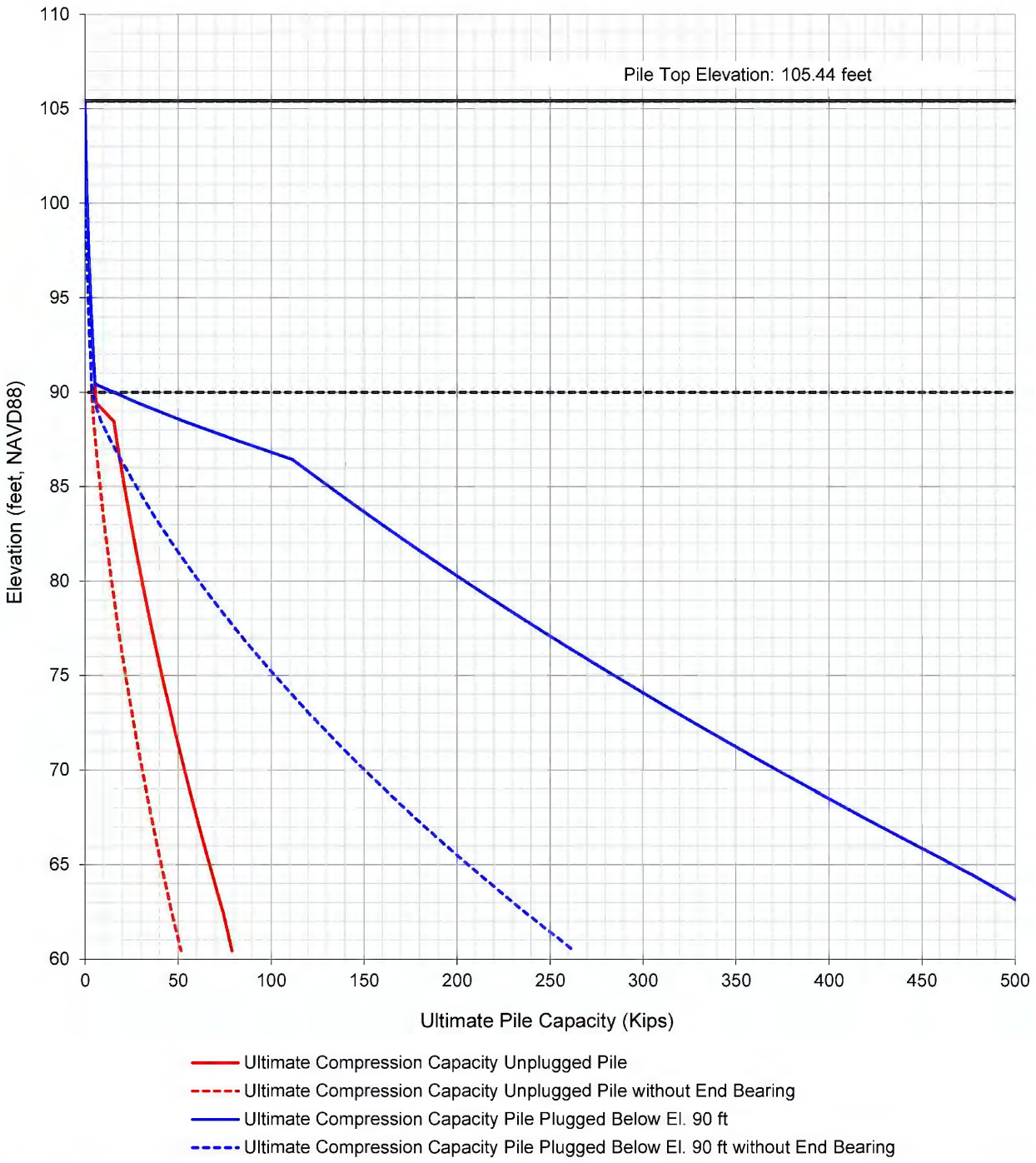
Bellota Weir Modification Project

Seismic Stability (Pseudo-Static K_h=0.15g)

April 2021

FIGURE 14

Bellota Weir Modification Project
24-inch 0.5-inch thick Pipe Pile
Ultimate Axial Capacity



Notes:
1) The allowable pile capacity for service load cases should be taken as the ultimate pile capacity shown divided by the required factor of safety of 2.0.
2) The allowable pile capacity can be increased by one-third for all load combinations that include wind or seismic.

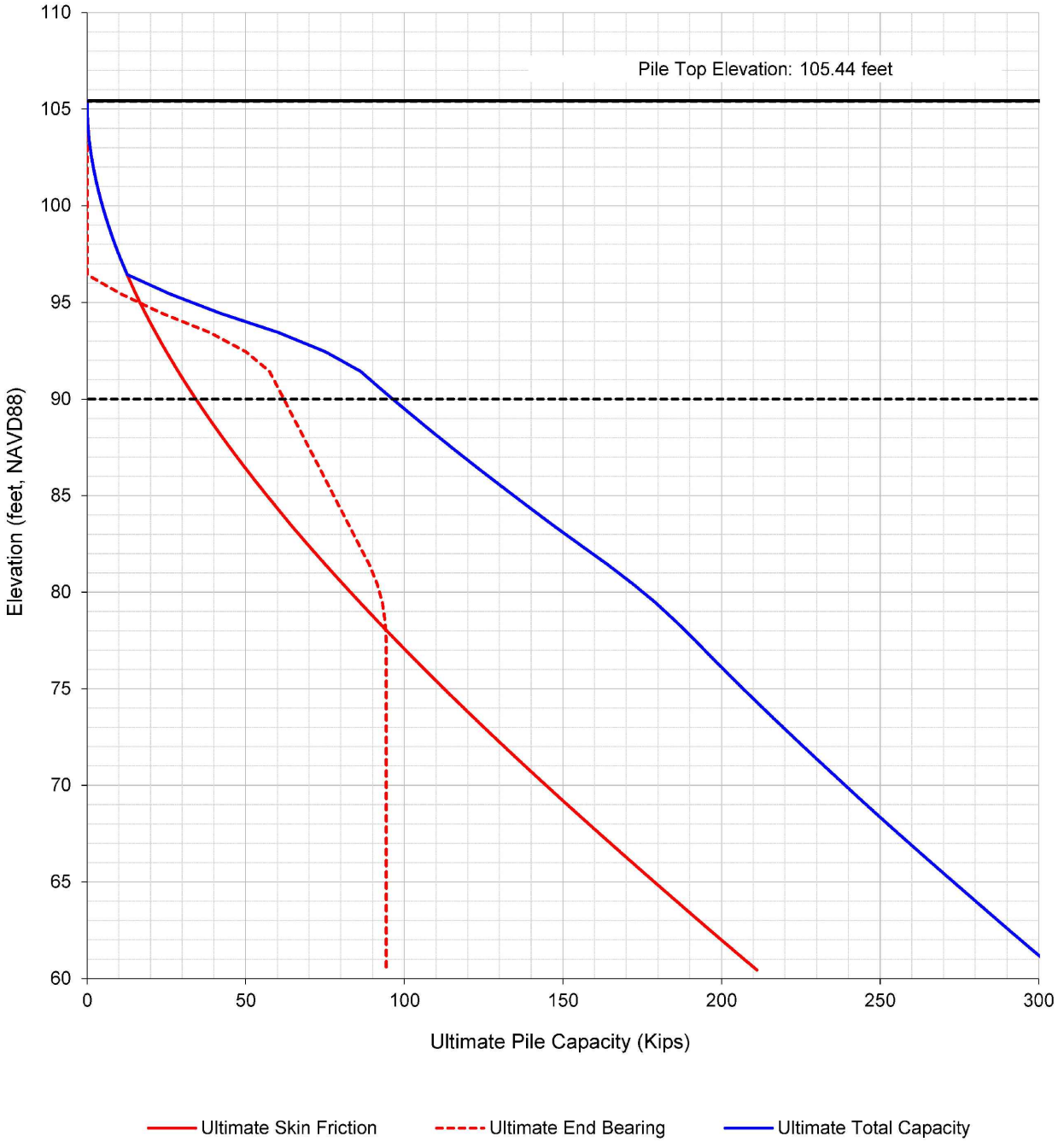
ULTIMATE AXIAL PILE CAPACITY
24 INCH PIPE PILE 0.5 INCH THICK

BELLOTA WEIR MODIFICATION PROJECT
SAN JOAQUIN, CALIFORNIA

Date
APRIL 2021

Figure
15

Bellota Weir Modification Project
36-inch CIDH Pile
Ultimate Axial Capacity



Notes:
1) The allowable pile capacity for service load cases should be taken as the ultimate pile capacity shown divided by the required factor of safety of 2.0.
2) The allowable pile capacity can be increased by one-third for all load combinations that include wind or seismic.

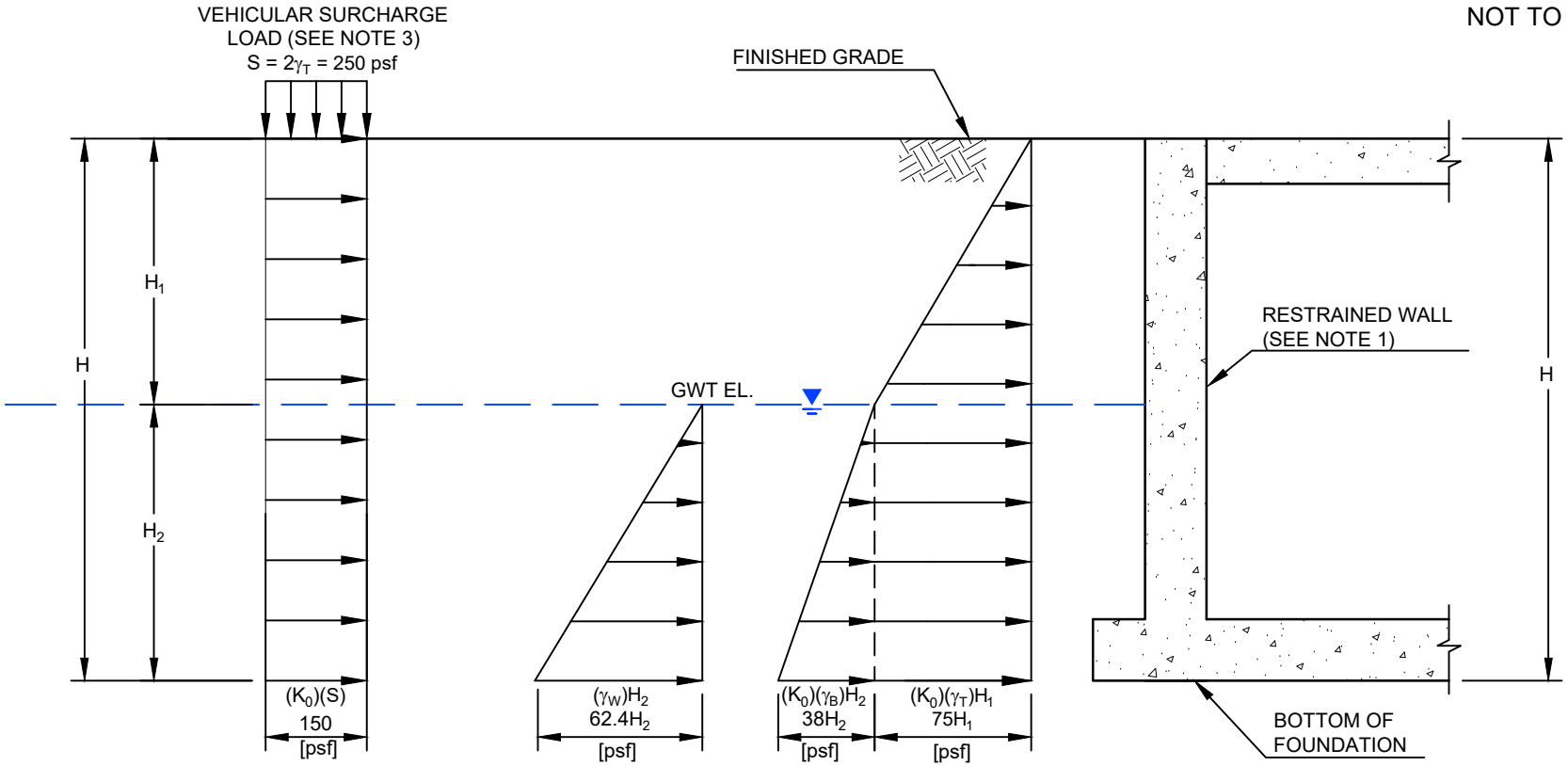
ULTIMATE AXIAL PILE CAPACITY
36 INCH CIDH PILE

BELLOTA WEIR MODIFICATION PROJECT
SAN JOAQUIN, CALIFORNIA

Date
APRIL 2021

Figure
16

NOT TO SCALE



PARAMETER	VALUE
AT-REST EARTH PRESSURE COEFFICIENT (K_0)	0.60
TOTAL UNIT WEIGHT (γ_T)	125 pcf
UNIT WEIGHT OF WATER (γ_W)	62.4 pcf
BUOYANT UNIT WEIGHT ($\gamma_B = \gamma_T - \gamma_W$)	62.6 pcf

NOTES:

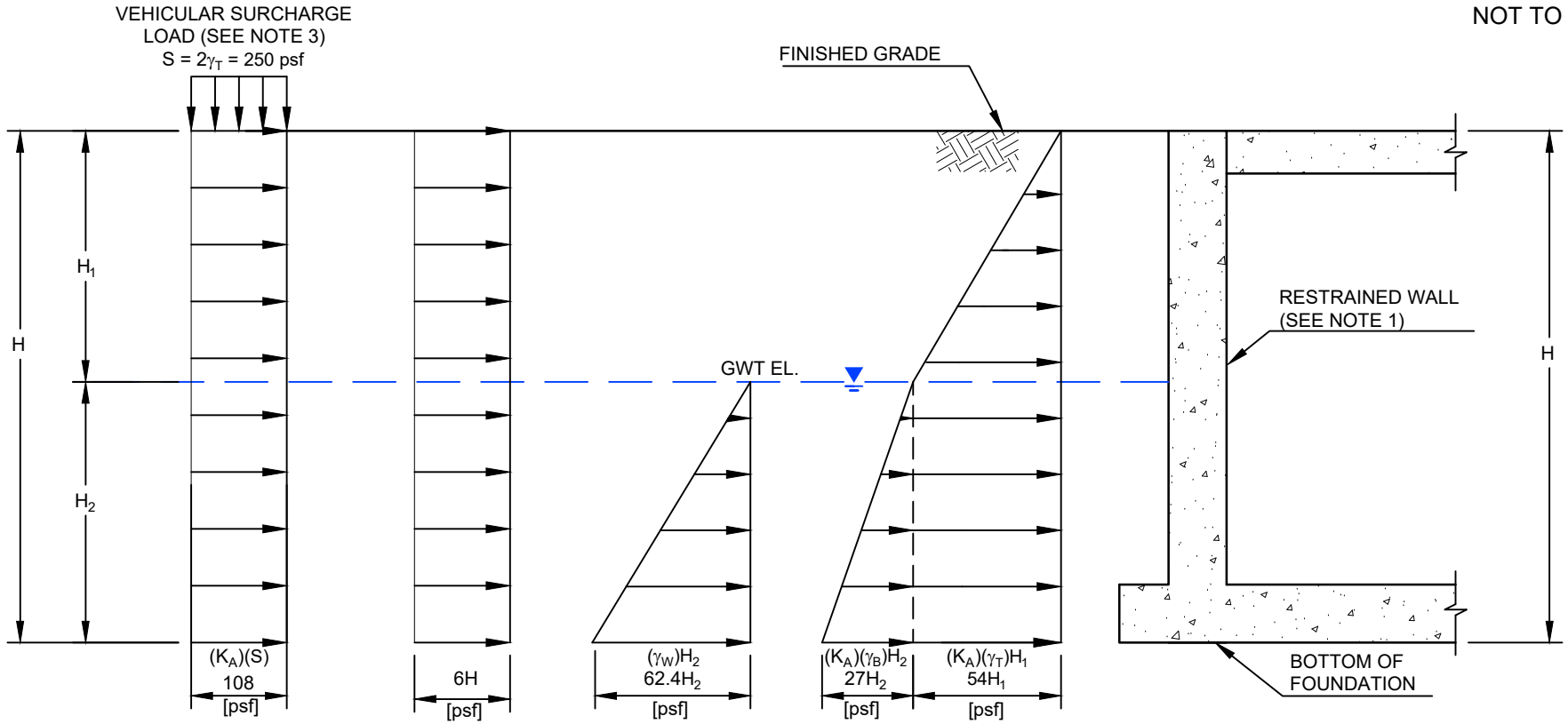
1. RESTRAINED WALLS ARE NOT FREE TO MOVE/ROTATE. RESTRAINED WALLS INCLUDE BRACED WALLS AND RIGID CANTILEVER WALLS.
2. THE HEIGHT OF THE WALL (H) IS FROM THE FINISHED GRADE TO THE BOTTOM OF THE FOUNDATION IN FEET (I.E. $H_1 + H_2$). H_1 REPRESENTS THE PORTION OF THE WALL ABOVE THE GROUNDWATER TABLE (GWT), AND H_2 REPRESENTS THE PORTION OF THE WALL BELOW THE GWT.
3. VEHICULAR SURCHARGE (S) SHOULD BE TAKEN AS EQUAL TO THE SURCHARGE LOAD FROM 2 FEET OF FILL. THE HORIZONTAL PRESSURE DUE TO VEHICULAR SURCHARGE IS THE SURCHARGE LOAD MULTIPLIED BY THE AT-REST EARTH PRESSURE COEFFICIENT (K_0) FOR THE FULL HEIGHT OF THE WALL.
4. FOR THE STATIC LOADING CASE FOR RESTRAINED WALLS, THE EARTH PRESSURE COMPONENTS USED ARE THE AT-REST VALUES.
5. THE EARTH PRESSURE SHOWN HEREIN IS BASED ON THE ASSUMPTION OF LEVEL BACKFILL.
6. THE HYDROSTATIC PRESSURE DISTRIBUTION SHOWN IS BASED ON THE ASSUMPTION THAT A PROPERLY DESIGNED FILTER AND BACK DRAIN IS PLACED BEHIND THE WALL.

STATIC LATERAL EARTH PRESSURES ON STRUCTURE WALLS

APRIL 2021

BELLOTA WEIR MODIFICATION PROJECT
SAN JOAQUIN, CALIFORNIA

NOT TO SCALE



PARAMETER	VALUE
ACTIVE EARTH PRESSURE COEFFICIENT (K_A)	0.43
TOTAL UNIT WEIGHT (γ_T)	125 pcf
UNIT WEIGHT OF WATER (γ_W)	62.4 pcf
BUOYANT UNIT WEIGHT ($\gamma_B = \gamma_T - \gamma_W$)	62.6 pcf

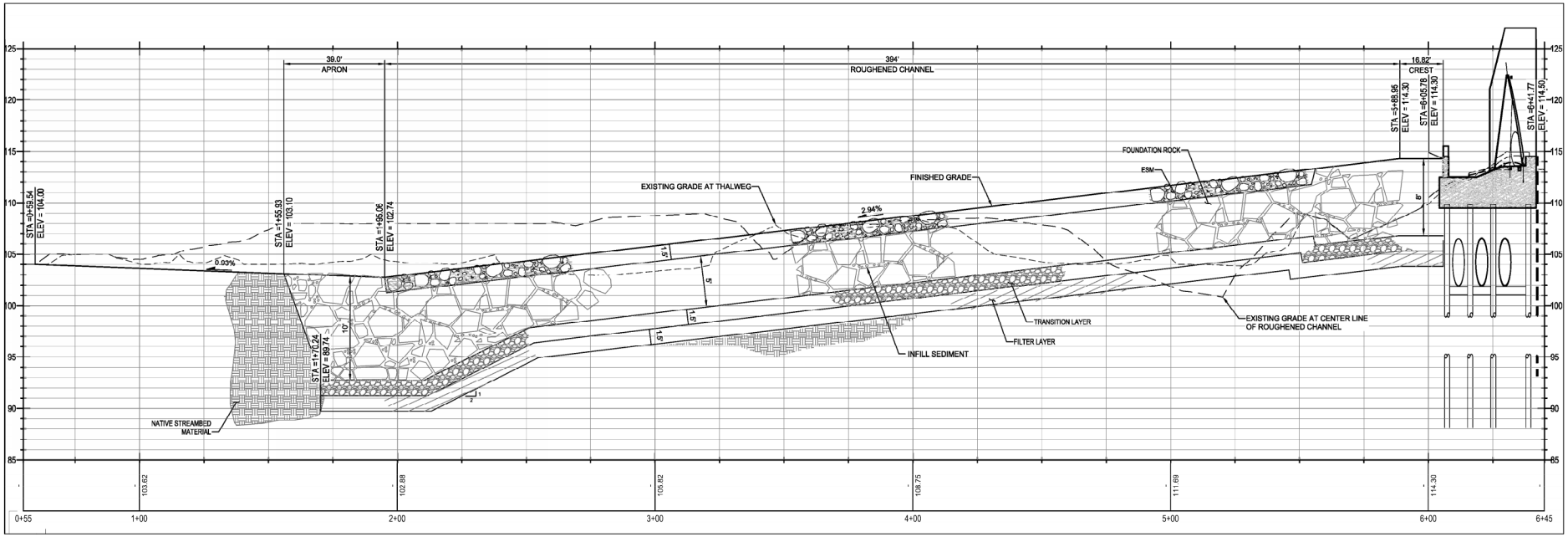
NOTES:

1. RESTRAINED WALLS ARE NOT FREE TO MOVE/ROTATE. RESTRAINED WALLS INCLUDE BRACED WALLS AND RIGID CANTILEVER WALLS.
2. THE HEIGHT OF THE WALL (H) IS FROM THE FINISHED GRADE TO THE BOTTOM OF THE FOUNDATION IN FEET (I.E. $H_1 + H_2$). H_1 REPRESENTS THE PORTION OF THE WALL ABOVE THE GROUNDWATER TABLE (GWT), AND H_2 REPRESENTS THE PORTION OF THE WALL BELOW THE GWT.
3. THE STRUCTURAL ENGINEER SHALL DETERMINE IF A VEHICULAR SURCHARGE SHOULD BE APPLIED FOR THE SEISMIC LOADING CASE. IF A VEHICULAR SURCHARGE (S) IS APPLIED, IT SHOULD BE TAKEN AS EQUAL TO THE SURCHARGE LOAD FROM 2 FEET OF FILL. THE HORIZONTAL PRESSURE DUE TO VEHICULAR SURCHARGE IS THE SURCHARGE LOAD MULTIPLIED BY THE ACTIVE EARTH PRESSURE COEFFICIENT (K_A) FOR THE FULL HEIGHT OF THE WALL.
4. FOR THE SEISMIC LOADING CASE FOR RESTRAINED WALLS, THE EARTH PRESSURE COMPONENTS USED ARE THE ACTIVE VALUES.
5. THE EARTH PRESSURE SHOWN HEREIN IS BASED ON THE ASSUMPTION OF LEVEL BACKFILL.
6. THE HYDROSTATIC PRESSURE DISTRIBUTION SHOWN IS BASED ON THE ASSUMPTION THAT A PROPERLY DESIGNED FILTER AND BACK DRAIN IS PLACED BEHIND THE WALL.

SEISMIC LATERAL EARTH PRESSURES ON STRUCTURE WALLS

APRIL 2021

BELLOTA WEIR MODIFICATION PROJECT
SAN JOAQUIN, CALIFORNIA



NOTES: Source: Sheet RI201 of Bellota Weir Modifications Project, 90% design submittal

Bellota Weir Modification Project

Bellota Weir Roughened Channel Downstream Profile

Feb 2022 FIGURE 19

Appendix B. 2004-2005 Historic Boring Logs

Appendix B - Historic Boring Logs
Bellota weir Modification Project

Background:

Bellota Weir is located where water flowing down the Calaveras River splits between the Mormon Slough and the Old Calaveras River channel. The project includes weir and intake improvements that will provide more reliable water delivery systems for Stockton East Water District (SEWD) customers, improve migration of anadromous fish, and reduce fish entrainment at the facilities, thereby enhancing populations of anadromous salmonids present in the Calaveras River.

Existing Geotechnical Explorations:

Existing geotechnical explorations includes previously conducted drilling at the site by CH2M Hill between October 2004 and January 2005. Five borings were visually observed and logged during the previous exploration program. These logs and laboratory testing indicate that the project site is generally underlain by riverine alluvial deposits to the depths explored (CH2M Hill 2005). The geologic map indicates presence of Holocene alluvium channel and levee deposits and Upper Modesto formation (Marchand and Bartow 1979).

Summary of Explorations:

The CH2M Hill borings were drilled in two phases to accommodate in-river work-window restrictions. Borings BI-2 and BI-3 were drilled within the river channel on October 25 and 26, 2004, using a Simco 2400 SK-1 drill rig. Drilling was advanced with a 6-inch-diameter hollow-stem and continuous-flight auger. The remaining borings were drilled on January 17, 18, and 19, 2005 using a Mobile B-80 drill rig. Drilling was advanced with a 5.5-inch-diameter tricone bit and mud-rotary drilling techniques. During both phases of the investigation, a 2-inch-outside-diameter split-spoon sampler was driven using a safety hammer of 140 pounds falling 30 inches, in accordance with the American Society for Testing Materials (ASTM) Standard Penetration Test (SPT; ASTM D1586). The hammer was lifted with a rope and cathead system for the initial borings, and an automatic trip hammer system for the latter borings. The number of blows required to drive the sampler 18 inches in 6-inch increments was recorded, and the number of blows required to drive the last two 6-inch intervals was summed to get the “N” value. Sampling was performed at approximate 5-foot intervals. Additional sampling was performed by hydraulically pushing thin-walled steel Shelby tubes (3-inch-diameter) into the soil at selected depths.

Select samples obtained during drilling were transported to a soil testing laboratory and tested for moisture content, dry density, gradation, and Atterberg limits.

Borings BI-1 through BI-3 were drilled near the proposed intake and fish screen structure. The presence of buried bank-protection riprap within the footprint of the proposed intake structure

necessitated that the borings be relocated away from the riverbank during drilling to a gravel bar within the river. Borings BF-1 and BD-2 were drilled on the south and north banks of the river, respectively, near the proposed inflatable crest-gate weir and fishway facilities. The presence of deep tailwater on the downstream limit of the existing apron prevented drill rig access to the portion of the apron south of the existing fish ladder structure. No borings were drilled within the river south of the fish ladder or on the sloping abutment.

Soil samples were classified using the Unified Soil Classification System in accordance with ASTM D2487. The relative compactness of the soil included in the soil description was determined from the SPT blow count obtained during the subsurface investigation. To determine the relative density and consistency of the soils encountered in the boring, an N60 blow count, or a corrected blow count, was calculated. The N60 blow count is standardized to account for samplers and hammers other than the standard 2-inch split spoon sampler and the standard safety hammer (140 pounds falling 30 inches). For classifying relative density and consistency to evaluate liquefaction potential, the uncorrected field blow counts shown on the logs were corrected for hammer type by multiplying the field blow count by the applicable correction factors (Attachment A.

Subsurface conditions were determined using the boring log information prepared during the investigation. Subsurface soil generally consists of very loose to loose silty or clayey sand underlain by medium-dense to dense sand interbedded by stiff clay, silt, and gravel lenses.

References

CH2M Hill

2005 Phase 2 Preliminary Design Report, Calaveras River Anadromous Fish Protection Project. April, 2005.

American Society for Testing and Material

2011 ASTM Standard D1586 “Standard Test Method for Standard Penetration Test (SPT) and Split-Barrel Sampling of Soils,” ASTM International, West Conshohocken, PA, DOI: 10.1520/D1586-11, www.astm.org.

2011 ASTM Standard D2487 “Standard Practice for Classification of Soils for Engineering Purposes (Unified Soil Classification System),” ASTM International, West Conshohocken, PA, DOI: 10.1520/D2487-11, www.astm.org.

Attachment A. Historic Boring Logs

CH2MHILL

PROJECT NUMBER:	BORING NUMBER:
175404.AA.P2.03	BD-2 SHEET 1 OF 3

SOIL BORING LOG

PROJECT : Calaveras River Fish Passage Improvement Project, Bellota, CA LOCATION : North side of Apron, 14 feet West of Existing Dam

ELEVATION : Approximately 111.5 feet DRILLING CONTRACTOR : Gregg Drilling

DRILLING METHOD AND EQUIPMENT : Mobile B-80 Drill Rig, Mud Rotary, Automatic Hammer

WATER LEVELS : --- START : END : LOGGER : A. Evans

DEPTH BELOW GROUND SURFACE (ft)	INTERVAL (ft)	RECOVERY (ft)	#TYPE	STANDARD PENETRATION TEST RESULTS 6"-6"-6" (N)	SOIL DESCRIPTION: SOIL NAME, USCS GROUP SYMBOL, COLOR, MOISTURE CONTENT, RELATIVE DENSITY OR CONSISTENCY, SOIL STRUCTURE, MINERALOGY	COMMENTS: DEPTH OF CASING, DRILLING RATE, DRILLING FLUID LOSS, TESTS, AND INSTRUMENTATION
5	5.0				0.75 feet - Concrete CLAYEY SAND WITH GRAVEL (SC), light brown, wet, with coarse gravel and fine cobble	9:00 am - Start time Encountered gravel and cobbles. Very hard to clear the hole. Drove 8-inch casing to 5 feet bgs.
	6.5	1.5	1-SS	0-0-2 (2)	SILTY CLAYEY SAND (SC-SM), gray with orange brown oxide staining, wet, very loose, fine-grained sand	Very soft. Sampler sank 1 foot with weight of hammer P200 = 45.0%; LL = 25, PL = 21, PI = 4
	9.0					
10	10.5	0.0	2-SS	1-1-1 (2)	SILTY CLAYEY SAND (SC-SM), similar to above	No recovery Very soft. Sampler tip plugged with gravel. Soil description was logged from soil encountered in cuttings.
	14.0					
15	15.5	0.5	3-SS	1-14-32 (46)	CLAYEY GRAVEL WITH SAND (GC), gray with reddish and yellowish gravel, wet, dense, coarse sand, with 1-inch diameter gravel in sample	Drill bouncing P200 = 29.8%; SA; %Gravel = 45.3%
	19.0					
20	20.5	1.0	4-SS	5-6-6 (12)	CLAYEY SAND (SC), light brown with horizontal orangish oxide stains in layers, wet, medium dense, medium- to fine-grained sand	
	24.0					
25	25.5	1.5	5-SS	7-8-12 (20)	ELASTIC SILT WITH SAND (MH), light brown with orangish stains, wet, medium dense, fine-grained sand	Unable to clear hole, lower down Modified California sampler to pull up gravels P200% = 76.2%; LL = 57, PL = 41, PI = 16
	29.0					
30		1.5	6-SS	6-8-11 (19)	CLAYEY SAND (SC), light gray with pinkish blue, moist, medium dense, fine-grained sand	

CH2MHILL

PROJECT NUMBER:	BORING NUMBER:
175404.AA.P2.03	BD-2 SHEET 2 OF 3

SOIL BORING LOG

PROJECT : Calaveras River Fish Passage Improvement Project, Bellota, CA LOCATION : North side of Apron, 14 feet West of Existing Dam

ELEVATION : Approximately 111.5 feet DRILLING CONTRACTOR : Gregg Drilling

DRILLING METHOD AND EQUIPMENT : Mobile B-80 Drill Rig, Mud Rotary, Automatic Hammer

WATER LEVELS : --- START : END : LOGGER : A. Evans

DEPTH BELOW GROUND SURFACE (ft)	INTERVAL (ft)	RECOVERY (ft)	#TYPE	STANDARD PENETRATION TEST RESULTS 6"-6"-6" (N)	SOIL DESCRIPTION: SOIL NAME, USCS GROUP SYMBOL, COLOR, MOISTURE CONTENT, RELATIVE DENSITY OR CONSISTENCY, SOIL STRUCTURE, MINERALOGY	COMMENTS: DEPTH OF CASING, DRILLING RATE, DRILLING FLUID LOSS, TESTS, AND INSTRUMENTATION
	30.5					
	34.0					
35	35.5	1.5	7-SS	5-8-7 (15)	CLAYEY SAND (SC), light brown, wet, medium dense, fine-grained sand, becoming gray medium-grained and less clayey in bottom 6 inches of the sample	
	39.0					
40	40.5	1.4	8-SS	7-8-7 (15)	CLAYEY SAND/WELL-GRADED SAND (SC/SW), gray, wet, medium dense	12:00 pm
	44.0					
45	45.5	1.5	9-SS	8-17-49 (66)	CLAYEY SAND/SANDY LEAN CLAY (SC/CL), light brown to light gray, moist, very dense, fine-grained sand	
	49.0					
50	50.5	0.5	10-SS	6-11-12 (23)	LEAN CLAY WITH SAND (CL), gray, moist, very stiff	12:30 pm P200 = 70.8%
	54.0					
55	55.5	1.5	11-SS	11-21-33 (54)	CLAYEY SAND/WELL-GRADED SAND (SC/SW), light brown with some pinkish brown zones, moist, very dense, some horizontal layering, very hard zones	
	59.0					
60		0.7	12-SS	37-50/6" (>50)	SILTY SAND (SM), brown, moist, very dense, fine-grained sand	Broken gravel in sample P200 = 35.0%; SA; %Gravel = 1.1%

CH2MHILL

PROJECT NUMBER:	BORING NUMBER:
175404.AA.P2.03	BD-2 SHEET 3 OF 3

SOIL BORING LOG

PROJECT : Calaveras River Fish Passage Improvement Project, Bellota, CA LOCATION : North side of Apron, 14 feet West of Existing Dam

ELEVATION : Approximately 111.5 feet DRILLING CONTRACTOR : Gregg Drilling

DRILLING METHOD AND EQUIPMENT : Mobile B-80 Drill Rig, Mud Rotary, Automatic Hammer

WATER LEVELS : --- START : END : LOGGER : A. Evans

DEPTH BELOW GROUND SURFACE (ft)	INTERVAL (ft)	RECOVERY (ft)	#TYPE	STANDARD PENETRATION TEST RESULTS 6"-6"-6" (N)	SOIL DESCRIPTION: SOIL NAME, USCS GROUP SYMBOL, COLOR, MOISTURE CONTENT, RELATIVE DENSITY OR CONSISTENCY, SOIL STRUCTURE, MINERALOGY	COMMENTS: DEPTH OF CASING, DRILLING RATE, DRILLING FLUID LOSS, TESTS, AND INSTRUMENTATION
60.5						
64.0						
65	65.5	0.5	13-SS	22-42-50/4" (>92)	CLAYEY SAND (SC), yellowish brown, moist, very dense, fine-grained sand, with some hard black nodules less than 1/8-inch diameter	1:38 pm Gravels in top of sample
69.0						
70	70.5	0.6	14-SS	17-50/6" (>50)	CLAYEY SAND (SC), similar to above, no nodules	2:00 pm Gravels in top of sample Hole grouted at 3:13 pm
75					Bottom of Hole at 70.5 ft below ground surface	
80						
85						
90						

CH2MHILL

PROJECT NUMBER:	BORING NUMBER:
175404.AA.P2.03	BF-1 SHEET 1 OF 3

SOIL BORING LOG

PROJECT : Calaveras River Fish Passage Improvement Project, Bellota, CA LOCATION : Slab at Roll Off Dumpster

ELEVATION : Approximately 121.0 feet DRILLING CONTRACTOR : Gregg Drilling

DRILLING METHOD AND EQUIPMENT : Mobile B-80 Drill Rig, Mud Rotary, Automatic Hammer

WATER LEVELS : --- START : 1/17/05 15:15 END : 1/18/05 12:55 LOGGER : A. Evans

DEPTH BELOW GROUND SURFACE (ft)	INTERVAL (ft)	RECOVERY (ft)	#TYPE	STANDARD PENETRATION TEST RESULTS 6"-6"-6" (N)	SOIL DESCRIPTION: SOIL NAME, USCS GROUP SYMBOL, COLOR, MOISTURE CONTENT, RELATIVE DENSITY OR CONSISTENCY, SOIL STRUCTURE, MINERALOGY	COMMENTS: DEPTH OF CASING, DRILLING RATE, DRILLING FLUID LOSS, TESTS, AND INSTRUMENTATION
					0.73 feet - Concrete with rebar	Very hard drilling with gravel and cobble
5	5.0					Transition at 4.5 feet bgs.
	6.5	0.0	1-SS	3-3-3 (6)	POORLY GRADED SAND WITH GRAVEL/CLAYEY SAND WITH GRAVEL (SP/SC), light brown, wet, loose, fine- to medium-grained sand	No recovery Very hard drilling. No catcher installed. Logged from rotary wash.
10	10.0					
	11.5	0.7	2-SS	3-4-3 (7)	SILTY SAND (SM), light orangish brown and gray, wet, loose, fine-grained sand	3:45 pm PP = 0.75 tsf.
15	15.0					
	17.0	1.6	3-ST	push	SILTY SAND (SM), similar to above	4:10 pm Pushed tube at 200 psi to refusal at 1.9 feet. Tube was possibly full. P200 = 19.3%; WC = 31.7%; UWD = 90 pcf; LL = 25, PL = NP, PI = NP
	18.5	0.7	4-SS	4-4-6 (10)	SILTY SAND (SM), similar to above, more medium-grained sand	PP = 0.25 tsf. Very wet
20	20.0					
	21.5	1.2	5-SS	6-32-35 (67)	CLAYEY SAND WITH GRAVEL (SC), orangish brown and gray, wet, very dense, coarse- to medium-grained sand, with fine- to medium-grained gravel	Stopped drilling at 4:48pm 1/17/05 Started drilling at 8:20am 1/18/05 Rough drilling, bouncing.
25	25.0					
	26.5	0.0	6-SS	7-5-5 (10)	CLAYEY SAND WITH GRAVEL (SC), similar to above, loose, with subrounded gravels (less than 1.5-inch diameter)	No recovery Gravel in sampler tip dropped from cuttings. Having trouble clearing gravels from hole.
30						

CH2MHILL

PROJECT NUMBER:	BORING NUMBER:
175404.AA.P2.03	BF-1 SHEET 2 OF 3

SOIL BORING LOG

PROJECT : Calaveras River Fish Passage Improvement Project, Bellota, CA LOCATION : Slab at Roll Off Dumpster

ELEVATION : Approximately 121.0 feet DRILLING CONTRACTOR : Gregg Drilling

DRILLING METHOD AND EQUIPMENT : Mobile B-80 Drill Rig, Mud Rotary, Automatic Hammer

WATER LEVELS : --- START : 1/17/05 15:15 END : 1/18/05 12:55 LOGGER : A. Evans

DEPTH BELOW GROUND SURFACE (ft)	INTERVAL (ft)	RECOVERY (ft)	#TYPE	STANDARD PENETRATION TEST RESULTS 6"-6"-6" (N)	SOIL DESCRIPTION: SOIL NAME, USCS GROUP SYMBOL, COLOR, MOISTURE CONTENT, RELATIVE DENSITY OR CONSISTENCY, SOIL STRUCTURE, MINERALOGY	COMMENTS: DEPTH OF CASING, DRILLING RATE, DRILLING FLUID LOSS, TESTS, AND INSTRUMENTATION
30.0						
	30.0 - 31.5	0.3	7-SS	10-11-7 (18)	CLAYEY SAND/SANDY LEAN CLAY (SC/CL), orange brown, moist, medium dense	Large gravels at bottom. Used Modified California sampler to try and remove rocks. 2" - 2.5" Max size and some soil PP = 2.0 tsf. Drilled without mud between 30 and 33 feet bgs.
35	35.0 - 36.5	1.5	8-SS	9-12-19 (31)	SILTY SAND WITH GRAVEL (SM), very dark brown with yellow and white and pink gravels, moist, dense, with fine-grained gravel	PP = 4.0 tsf. Started drilling with mud at 35 feet bgs.
40	40.0 - 41.5	1.5	9-SS	4-9-8 (17)	FAT CLAY WITH SAND (CH), light gray with pinkish blue, moist, very stiff	P200 = 75.9%; LL = 59, PL = 26, PI = 33
45	45.0 - 46.5	1.5	10-SS	5-6-8 (14)	POORLY GRADED SAND/CLAYEY SAND (SP/SC), grayish brown, wet, medium dense, medium- to fine-grained sand	PP = 1.5 tsf.
50	50.0 - 51.5	1.5	11-SS	6-15-16 (31)	POORLY GRADED SAND/CLAYEY SAND (SP/SC), similar to above, becoming CLAYEY SAND (SC), light gray, wet, dense at bottom 6 inches of sample	
55	55.0 - 56.5	1.5	12-SS	5-10-14 (24)	SANDY LEAN CLAY (CL), light brown transitioning to light gray, wet, very stiff, with coarse- to fine-grained sand	P200 = 69.3%
60						

CH2MHILL

PROJECT NUMBER:	BORING NUMBER:
175404.AA.P2.03	BF-1 SHEET 3 OF 3

SOIL BORING LOG

PROJECT : Calaveras River Fish Passage Improvement Project, Bellota, CA LOCATION : Slab at Roll Off Dumpster

ELEVATION : Approximately 121.0 feet DRILLING CONTRACTOR : Gregg Drilling

DRILLING METHOD AND EQUIPMENT : Mobile B-80 Drill Rig, Mud Rotary, Automatic Hammer

WATER LEVELS : --- START : 1/17/05 15:15 END : 1/18/05 12:55 LOGGER : A. Evans

DEPTH BELOW GROUND SURFACE (ft)	INTERVAL (ft)	RECOVERY (ft)	#TYPE	STANDARD PENETRATION TEST RESULTS 6"-6"-6" (N)	SOIL DESCRIPTION: SOIL NAME, USCS GROUP SYMBOL, COLOR, MOISTURE CONTENT, RELATIVE DENSITY OR CONSISTENCY, SOIL STRUCTURE, MINERALOGY	COMMENTS: DEPTH OF CASING, DRILLING RATE, DRILLING FLUID LOSS, TESTS, AND INSTRUMENTATION
60.0						
	60.0 – 61.5	1.0	13-SS	23-50/5.5" (>50)	CLAYEY SAND WITH GRAVEL/WELL-GRADED SAND (SC/SW), wet, very dense, well graded sand, with less than 5% fine-grained gravel	12:00 pm
65	65.0 – 66.5	1.5	14-SS	11-17-25 (42)	SANDY LEAN CLAY/LEAN CLAY (CL), very light brown with pinkish blue structure, dry, hard, soft-rock like, some orange oxidation veins	Drill bouncing
70	70.0 – 71.5	1.5	15-SS	8-12-12 (24)	POORLY GRADED SAND/CLAYEY SAND (SP/SC), grayish brown, wet, medium dense, fine-grained sand	Drill bouncing
75	75.0 – 76.5	1.2	16-SS	12-13-8 (21)	WELL-GRADED SAND/CLAYEY SAND (SW/SC), grayish brown, wet, medium dense	12:55 pm
					Bottom of Hole at 76.5 ft below ground surface 1/18/05 12:55	
80						
85						
90						

CH2MHILL

PROJECT NUMBER:	BORING NUMBER:	
175404.AA.P2.03	BI-1	SHEET 1 OF 2

SOIL BORING LOG

PROJECT : Calaveras River Fish Passage Improvement Project, Bellota, CA LOCATION : 25 feet East of Existing Intake

ELEVATION : Approximately 121.5 feet DRILLING CONTRACTOR : Gregg Drilling

DRILLING METHOD AND EQUIPMENT : Mobile B-80 Drill Rig, Mud Rotary, Automatic Hammer

WATER LEVELS : --- START : 1/17/05 10:00 END : 1/17/05 14:00 LOGGER : A. Evans

DEPTH BELOW GROUND SURFACE (ft)	INTERVAL (ft)	RECOVERY (ft)	#TYPE	STANDARD PENETRATION TEST RESULTS 6"-6"-6" (N)	SOIL DESCRIPTION: SOIL NAME, USCS GROUP SYMBOL, COLOR, MOISTURE CONTENT, RELATIVE DENSITY OR CONSISTENCY, SOIL STRUCTURE, MINERALOGY	COMMENTS: DEPTH OF CASING, DRILLING RATE, DRILLING FLUID LOSS, TESTS, AND INSTRUMENTATION
						Hard drilling, with small cobble/rip rap
5	5.0					
	6.5	0.8	1-SS	7-10-12 (22)	SILTY SAND WITH GRAVEL (SM), gray, wet, medium dense, well graded sand, with fine-grained gravel	PP = 1.0, 1.5 tsf.
						7.5 feet - Smooth drilling
10	10.0					
	11.5	1.2	2-SS	1-1-1 (2)	SILTY SAND (SM), light brown, wet, very loose, fine-grained sand	P200 = 46.3%
	12.5					
	14.5	0.0	3-ST	push	SILTY SAND (SM), similar to above	Pushed tube at 500 psi End crushed on gravel sluff No recovery, too sandy
15	15.0					
	16.5	0.0	4-SS	2-1-2 (3)	SILTY SAND (SM), similar to above	No recovery
20	20.0					
	21.5	0.3	5-SS	1-2-2 (4)	POORLY GRADED SAND/SILTY SAND (SP/SM), gray and white, wet, very loose, medium-grained sand	
						Rough drilling, bouncing on rocks
25	25.0					
	26.5		6-SS	17-22-21 (43)	WELL-GRADED GRAVEL WITH SILT AND SAND (GW-GM), gray and yellow brown gravels and quartz, wet, dense, approximately 1 inch diameter gravel	P200 = 8.0%; SA; %Gravel = 62.2%
30						

CH2MHILL

PROJECT NUMBER:	BORING NUMBER:
175404.AA.P2.03	BI-1 SHEET 2 OF 2

SOIL BORING LOG

PROJECT : Calaveras River Fish Passage Improvement Project, Bellota, CA LOCATION : 25 feet East of Existing Intake

ELEVATION : Approximately 121.5 feet DRILLING CONTRACTOR : Gregg Drilling

DRILLING METHOD AND EQUIPMENT : Mobile B-80 Drill Rig, Mud Rotary, Automatic Hammer

WATER LEVELS : --- START : 1/17/05 10:00 END : 1/17/05 14:00 LOGGER : A. Evans

DEPTH BELOW GROUND SURFACE (ft)	INTERVAL (ft)	RECOVERY (ft)	#TYPE	STANDARD PENETRATION TEST RESULTS 6"-6"-6" (N)	SOIL DESCRIPTION: SOIL NAME, USCS GROUP SYMBOL, COLOR, MOISTURE CONTENT, RELATIVE DENSITY OR CONSISTENCY, SOIL STRUCTURE, MINERALOGY	COMMENTS: DEPTH OF CASING, DRILLING RATE, DRILLING FLUID LOSS, TESTS, AND INSTRUMENTATION
	30.0					Probably gravel in tip of the sample
	31.5	0.0	7-SS	4-4-4 (8)		No recovery
						Smooth drilling between 33 and 33.5 feet bgs.
35	35.0					
	36.5	1.5	8-SS	7-9-8 (17)	SANDY SILT (ML), gray to light gray with orange oxidation stains, wet, very stiff	Rough drilling, not typical for what is in the sample P200 = 50.1%; LL = 45, PL = 42, PI = 3
40	40.0					
	41.5	0.4	9-SS	8-8-9 (17)	SILTY SAND (SM), light brown, wet, medium dense, fine-grained sand	12:49 pm Staff gage 1.4 feet PP = 1.5 tsf.
45	45.0					
	46.5	1.5	10-SS	2-5-6 (11)	SILTY SAND (SM), similar to above, light brown with thin white veins, with some medium-grained sand zones	PP = 2.0 tsf.
50	50.0					
	51.5	1.5	11-SS	4-9-8 (17)	WELL-GRADED SAND/CLAYEY SAND (SW/SC), light brown, wet, medium dense	2:00 pm - End drilling
55					Bottom of Hole at 51.5 ft below ground surface 1/17/05 14:00	
60						

CH2MHILL

PROJECT NUMBER:	BORING NUMBER:
175404.AA.P2.03	BI-2 SHEET 1 OF 1

SOIL BORING LOG

PROJECT : Calaveras River Fish Passage Improvement Project, Bellota, CA LOCATION :

ELEVATION : Approximately 114.0 feet DRILLING CONTRACTOR : Taber

DRILLING METHOD AND EQUIPMENT : Simco 2400 SK-1, Hollow Stem Auger, 6-inch, Safety Hammer with Rope and Cathead

WATER LEVELS : --- START : 10/26/04 10:15 END : 10/26/04 16:00 LOGGER : A. Finney

DEPTH BELOW GROUND SURFACE (ft)	INTERVAL (ft)	RECOVERY (ft)	#TYPE	STANDARD PENETRATION TEST RESULTS 6"-6"-6" (N)	SOIL DESCRIPTION: SOIL NAME, USCS GROUP SYMBOL, COLOR, MOISTURE CONTENT, RELATIVE DENSITY OR CONSISTENCY, SOIL STRUCTURE, MINERALOGY	COMMENTS: DEPTH OF CASING, DRILLING RATE, DRILLING FLUID LOSS, TESTS, AND INSTRUMENTATION
						Driller was unable to advance auger. Rip rap buried 12 inches below surface. Moved 3-4 feet to the north. Restarted drilling at 11 am. Moved and restarted 4 times to avoid rip rap.
5	5.0					
	6.5	1.0	1-SS	15-16-9 (25)	WELL-GRADED GRAVEL WITH SAND (GW), dark yellowish brown, wet, medium dense	1:30 pm P200 = 4.7%; SA
						Driller noted material changed at 7.5 feet bgs.
10	10.0					
	11.5	1.5	2-SS	1-3-3 (6)	SILTY SAND (SM), dark yellowish brown, wet, loose	LL = 24, PL = NP, PI = NP; P200 = 41.8%; SA; %Gravel = 53.3%
						Driller noted material changed at 12.5 feet bgs.
15	15.0					
	16.5	1.5	3-SS	1-1-2 (3)	CLAYEY SAND (SC), reddish brown, wet, very loose, very fine-grained sand	2:30 pm Driller added water to ensure material does not enter when plugs pulled
						Drilling much harder below 17.5 feet, rods chattering
20	20.0					
	21.5	1.1	4-SS	14-25-35 (60)	POORLY GRADED GRAVEL WITH SAND AND SILT (GP-GM), dark yellow brown, wet, very dense	
						Driller noted material changed at 22.5 feet bgs.
25	25.0					
	26.5	1.5	5-SS	5-9-12 (21)	SANDY ELASTIC SILT (MH), pinkish brown with rust and black mottling, moist, very stiff, grading to silty clayey sand	PP = 3.0 tsf. LL = 50, PL = 39, PI = 11; P200 = 66.2%
	28.5					
30	30.0		6-SS	5-5-8 (13)	SANDY ELASTIC SILT (MH), pinkish brown and gray, moist, stiff	4:00 pm

Bottom of Hole at 30.0 ft below ground surface
10/26/04 16:00

CH2MHILL

PROJECT NUMBER:	BORING NUMBER:
175404.AA.P2.03	BI-3 SHEET 1 OF 1

SOIL BORING LOG

PROJECT : Calaveras River Fish Passage Improvement Project, Bellota, CA LOCATION : Gravel Bar

ELEVATION : Approximately 113.5 feet DRILLING CONTRACTOR : Taber

DRILLING METHOD AND EQUIPMENT : Simco 2400 SK-1, Hollow Stem Auger, 6-inch, Safety Hammer with Rope and Cathead

WATER LEVELS : --- START : 10/25/04 14:45 END : 10/26/04 09:15 LOGGER : A. Finney

DEPTH BELOW GROUND SURFACE (ft)	INTERVAL (ft)	RECOVERY (ft)	#TYPE	STANDARD PENETRATION TEST RESULTS 6"-6"-6" (N)	SOIL DESCRIPTION: SOIL NAME, USCS GROUP SYMBOL, COLOR, MOISTURE CONTENT, RELATIVE DENSITY OR CONSISTENCY, SOIL STRUCTURE, MINERALOGY	COMMENTS: DEPTH OF CASING, DRILLING RATE, DRILLING FLUID LOSS, TESTS, AND INSTRUMENTATION
2.5					SILTY SAND WITH GRAVEL (SM), dark yellowish brown, wet, medium dense	Surface covered with cobbles
4.0	1.0	1	SS	10-17-20 (37)	WELL GRADED GRAVEL WITH SILT AND SAND (GW-GM), dark yellowish brown, wet, dense	
5						
7.5						
9.0	1.5	2	SS	2-1-7 (8)	POORLY GRADED SAND (SP), dark yellowish brown, wet, loose, with some fine gravel and wood fragments at 8.5 feet bgs.	3:20 pm Rock in tip of sampler P200 = 4.3%; SA; LL = 31, PL = NP, PI = NP
10						Gravel and harder drilling at 10 feet bgs.
12.5						
14.0	1.2	3	SS	45-32-29 (61)	WELL-GRADED SAND WITH SILT AND GRAVEL (SW-SM), gray, wet, dense	4:30 pm
15						
17.5						
19.0	1.2	4	SS	17-12-17 (29)	WELL-GRADED SAND WITH SILT AND GRAVEL (SW-SM), similar to above	P200 = 7.0%; SA; %Gravel = 33.4%
20						
22.5						
24.0	0.9	5	SS	7-9-5 (14)	WELL-GRADED SAND WITH SILT AND GRAVEL (SW-SM), similar to above, gray, medium dense	5:45 pm
25						Driller noted material changed at 24 feet bgs
28.0						
29.5	1.5	6	SS	7-6-9 (15)	SANDY SILT (ML), tan, moist, stiff	P200 = 53.0%; LL = 42, PL = 40, PI = 2 6:30 pm Too dark to continue. Left auger in ground. Returned at 8:00 am. Drillers on site at 9:15 am.
30						

Bottom of Hole at 29.5 ft below ground surface
10/26/04 09:15

Appendix C. 2021 Supplemental Boring Logs

Appendix C - Supplemental Boring Logs
Bellota weir Modification Project

Thursday, April 01, 2021

To: Mark Stanley, P.E., G.E., D.GE

From: Sagar Raj Satyal, EIT

Background

Bellota Weir is located where water flowing down the Calaveras River splits between the Mormon Slough and the Old Calaveras River channel. The project includes weir and intake improvements that will provide more reliable water delivery systems for Stockton East Water District (SEWD) customers, improve migration of anadromous fish, and reduce fish entrainment at the facilities, thereby enhancing populations of anadromous salmonids present in the Calaveras River.

Existing Geotechnical Explorations

Existing geotechnical explorations includes previously conducted drilling at the site by CH2M Hill between October 2004 and January 2005. Five borings were visually observed and logged during the previous exploration program. These logs and laboratory testing indicate that the project site is generally underlain by riverine alluvial deposits to the depths explored (CH2M Hill 2005). The geologic map indicates presence of Holocene alluvium channel and levee deposits and Upper Modesto formation (Marchand and Bartow 1979).

Supplemental Geotechnical Explorations

Exploration Preparation

HDR developed exploration and health and safety work plans prior to site mobilization. The exploration work plan included a description of the location and proposed depth of explorations and type of drilling method/equipment to use for advancing the boring.

As required by San Joaquin County, a well (drilling) permit was obtained for the explorations. As required prior to performing explorations, the proposed exploration locations were marked in the field and California 811 Service Alert was contacted to check for the presence of underground utilities in the proposed locations.

Summary of Explorations

The field investigation consisted of advancing a total of five exploratory borings to depths of 70 feet adjacent to the weir. Four mud-rotary borings were advanced to depths of 70 feet along the northern bank of Mormon Slough to provide a better understanding of the materials, help interpret the subsurface stratigraphy of the area, and characterize the soil layers. And one mud-rotary boring was advanced to 70 feet deep along the south bank of the Mormon Slough to confirm the subsurface condition.

The boring locations were determined by measuring the distance from existing site features by pacing and are accurate only to the degree implied by the method used. The boring locations were not surveyed, and boring coordinates and ground surface elevations were obtained from Google Earth imagery and an existing topographic survey map of the site.

The borings were drilled by Taber Drilling of West Sacramento California using mud-rotary drilling methods. Subsurface encountered in each boring were logged by an HDR engineer with the coordinates and depth for each boring included in the boring logs (Attachment A). T.

Drilling Methods

As previously discussed, Taber Drilling advanced the borings using track-mounted drill rig with mud-rotary drilling methods detailed on the boring logs. The mud-rotary method employs a rotating drill bit that grinds and pulverizes the soil as the bit advances. The drill cuttings are transported up to the ground surface using a circulating drilling fluid composed of water and bentonite, referred to as mud. This method can incorporate the use of casing to help reinforce the drilled borehole and prevent borehole collapse, while facilitating transportation of the cuttings. This method (either cased or open hole), is usually preferred to retrieve undisturbed samples below the ground water table as the drilling fluid (mud) serves to equalize the hydrostatic pressure between the open borehole and the surrounding area reducing the potential for heaving.

Sampling

Samples were collected by driving an 18-inch-long split spoon sampler into the subsurface material using an automatic trip 140-pound hammer with a 30-inch free fall in general accordance with ASTM D1586. After the sampler was driven, it was removed from the borehole and the exploration was advanced to the next sample interval. Samples were collected at 2.5-foot intervals for the first 15-foot depth and every 5-foot thereafter. Soil samplers included 2.5-inch inside diameter Modified California (Mod Cal) split barrel sampler with liners (6-inch-long stainless steel liners), 2-inch inside diameter California (Cal) split barrel sampler with liners (6-inch long stainless steel liners), and a 1.375-inch inside diameter Standard Penetration Test (SPT) split barrel sampler without liners.

The Mod Cal, Cal, and SPT samplers were used in general accordance with ASTM D1586. The type of sampler used at each sampling interval were based on the material encountered or expected to be encountered. Mod Cal and Cal samples were capped to retain moisture while

SPT samples were placed in sealed “Ziploc” bags. Each sample was labeled with the project number, date, sample number, and depth. If more than one sample was taken at each interval, an A/B designation was added with A being the lower sample and B the upper.

An HDR field engineer maintained a boring log and visually classified the soils encountered in the samples and cuttings in accordance with ASTM D2488. The field classifications were updated based on laboratory test results in accordance with ASTM D2487.

Upon completion of drilling, the samples were transported to the HDR Folsom, CA office for further review.

Hammer

Samplers were advanced by dropping a 140-pound, automatic hammer through a 30-inch free fall and the number of blows required to drive the sampler a total of 18 inches, or a shorter distance where the number of blows encountered for a 6-inch interval exceeded 50 (hard resistance). The number of blows were recorded for each 6 inches of penetration. The vertically-stacked blows recorded on the boring logs represent the individual blows for each 6 inches of driving. The blows per foot is the accumulated number of blows that were required to drive the last 12 inches, or the number of inches indicated where hard resistance was encountered. The field blow counts (Nf) are shown for each sample on the boring logs. The Nf has not been corrected for the sampler size.

The hammer energy was not measured at the site during the exploration program. The latest hammer calibration report from Taber Drilling was obtained to be used for the hammer efficiency. The calibration report dated 7/1/2020 indicated an average energy transfer ratio of 76.5 percent.

Drill Cuttings and Restoration

During each boring, drill cuttings were collected and stored in 55-gallon drums and the borings backfilled with neat cement grout in accordance with San Joaquin County well permit requirements, with an approximate ratio of 94 pounds of Portland cement for every 5 gallons of water. Grout inspector from the county was present to observe grouting of all boreholes. All equipment, materials, and trash was removed from the site and the area restored to its prior condition.

Laboratory Testing

Soil samples were selected from the samples obtained during the subsurface investigation for testing and were delivered to Gulf Shores Exploration and Testing of Rancho Cordova, California for geotechnical laboratory testing, which consisted of:

- Moisture Content and Dry Density (ASTM D2937)
- Moisture Content (ASTM D2216)
- Coarse Sieve (ASTM D6913, 3/4” to - #200 Sieve)

- Fine Sieve (ASTM D6913, #4 to #200 Sieve)
- #200 Wash (ASTM D1140)
- Atterberg Limits (ASTM D 4318 Wet Method)
- Triaxial Shear Unconsolidated Undrained (UU; ASTM D2850)

The results of the laboratory tests are presented on the boring logs at the appropriate sample depths (Attachment A).

Subsurface Condition

Four borings on the north bank of the Mormon Slough were advanced to the depth of 70 feet. Subsurface encountered recent overbank deposits consisting of approximately 15 feet of loose to medium stiff silty sands and silt layers overlying alternating layers of loose to medium dense silt, silty sands, and clayey sands to the depth of 35 feet. Coarse grained gravel was encountered at depths of 30 to 40 feet. Below a depth of 40 feet, older alluvial deposits consisting of medium dense to dense silty sands, silt, clay, and clayey sands were encountered to the depth of 70 feet. The borings indicated that the thickness of the gravel layer varies along the north bank and the gravel deposit is not continuous.

One boring on the south bank of the Mormon Slough was advanced to the depth of 70 feet. The boring was advanced by coring through a 9-inch-thick concrete slab overlying a 5-foot-thick clayey gravel layer. Underlaying the gravel layer, hard silty sand was encountered to the depth of 15 feet. Clayey sand, silty sand, and clayey gravel layers were encountered to the depth of 40 feet. Below 40 feet to a depth of 70 feet, older alluvial deposits consisting of medium dense to dense silty sands, silt, clay, and clayey sands were encountered.

The ground water table was encountered at depths ranging from 18 feet to 30 feet. Given the proximity to the river, the groundwater level would probably match closely with the river water level. Piezometers were not installed in the project area as part of this effort.

References

CH2M Hill

2005 Phase 2 Preliminary Design Report, Calaveras River Anadromous Fish Protection Project. April 2005.

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1979 Preliminary geologic map of Cenozoic deposits of the Bellota Quadrangle, California (No. 79-664).

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2017 ASTM Standard D2937 “Standard Test Method for Density of Soil in Place by the Drive-Cylinder Method,” ASTM International, West Conshohocken, PA, DOI: 10.1520/D2937-17E02, www.astm.org.

2019 ASTM D2216 “Standard Test Methods for Laboratory Determination of Water (Moisture) Content of Soil and Rock by Mass”, ASTM International, West Conshohocken, PA, 2019, www.astm.org.

2007 ASTM Standard D422 “Standard Test Method for Particle-Size Analysis of Soils,” ASTM International, West Conshohocken, PA, DOI: 10.1520/D0422-63R07E02, www.astm.org.

2010 ASTM Standard D4318 “Standard Test Methods for Liquid Limit, Plastic Limit and Plasticity Index of Soils,” ASTM International, West Conshohocken, PA, DOI: 10.1520/D4318, www.astm.org.

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2017 ASTM D1140 “Standard Test Methods for Determining the Amount of Material Finer than 75-µm (No. 200) Sieve in Soils by Washing”, ASTM International, West Conshohocken, PA, 2017, www.astm.org.

2015 ASTM D2850 “Standard Test Method for Unconsolidated-Undrained Triaxial Compression Test on Cohesive Soils”, ASTM International, West Conshohocken, PA, 2015, www.astm.org.

Attachment A. Supplemental Boring Logs

UNIFIED SOIL CLASSIFICATION SYSTEM (ASTM D-2487)

MATERIAL TYPES	CRITERIA FOR ASSIGNING SOIL GROUP NAMES			GROUP SYMBOL	SOIL GROUP NAMES & LEGEND
COARSE-GRAINED SOILS >50% RETAINED ON NO. 200 SIEVE	GRAVELS >50% OF COARSE FRACTION RETAINED ON NO 4. SIEVE	CLEAN GRAVELS <5% FINES	$C_u \geq 4$ AND $1 \leq C_c \leq 3$	GW	WELL-GRADED GRAVEL
			$C_u < 4$ AND/OR $1 > C_c > 3$	GP	POORLY-GRADED GRAVEL
		GRAVELS WITH FINES >12% FINES	FINES CLASSIFY AS ML OR MH	GM	SILTY GRAVEL
			FINES CLASSIFY AS CL OR CH	GC	CLAYEY GRAVEL
	SANDS >50% OF COARSE FRACTION PASSES NO 4. SIEVE	CLEAN SANDS <5% FINES	$C_u \geq 6$ AND $1 \leq C_c \leq 3$	SW	WELL-GRADED SAND
			$C_u < 6$ AND/OR $1 > C_c > 3$	SP	POORLY-GRADED SAND
		SANDS AND FINES >12% FINES	FINES CLASSIFY AS ML OR MH	SM	SILTY SAND
			FINES CLASSIFY AS CL OR CH	SC	CLAYEY SAND
FINE-GRAINED SOILS >50% PASSES NO. 200 SIEVE	SILTS AND CLAYS LIQUID LIMIT<50	INORGANIC	PI>7 AND PLOTS>"A" LINE	CL	LEAN CLAY
			PI<4 OR PLOTS<"A" LINE	ML	SILT
		ORGANIC	LL (oven dried)/LL (not dried)<0.75	OL	ORGANIC CLAY OR SILT
	SILTS AND CLAYS LIQUID LIMIT>50	INORGANIC	PI PLOTS >"A" LINE	CH	FAT CLAY
			PI PLOTS <"A" LINE	MH	ELASTIC SILT
		ORGANIC	LL (oven dried)/LL (not dried)<0.75	OH	ORGANIC CLAY OR SILT
HIGHLY ORGANIC SOILS		PRIMARILY ORGANIC MATTER, DARK IN COLOR, AND ORGANIC ODOR		PT	PEAT

OTHER SYMBOLS

MATERIALS

- Asphalt
- Aggregate Base
- Topsoil
- Bedrock

PIEZOMETER

- Grout Seal or Fill
- Bentonite Seal or Fill
- Sand Pack + Solid Pipe
- Sand Pack + Slotted Pipe

SAMPLERS

- SPT (2" OD)
- Modified California (3" OD)
- California (2.5" OD)
- Shelby Tube
- Pitcher Barrel
- HQ Core
- Grab/Bulk

- INITIAL WATER LEVEL MEASUREMENT(WITH DATE)
- STABILIZED WATER LEVEL MEASUREMENT(WITH DATE)

GRAIN SIZES

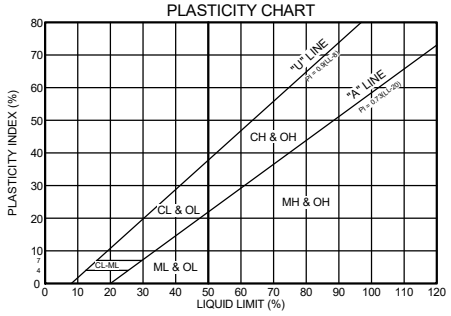
U.S. STANDARD SIEVE	200	40	10	4	3/4"	3"	12"
SILTS AND CLAYS	SAND			GRAVEL		COBBLES	BOULDERS
	FINE	MEDIUM	COARSE	FINE	COARSE		

PENETRATION RESISTANCE

SAND & GRAVEL		SILT & CLAY		
RELATIVE DENSITY	BLOWS/FOOT*	CONSISTENCY	BLOWS/FOOT*	UNC. COMP. STRENGTH (KSF)
VERY LOOSE	0 - 4	VERY SOFT	0 - 1	0 - 1/2
LOOSE	5 - 10	SOFT	2 - 4	1/2 - 1
MEDIUM DENSE	11 - 30	MEDIUM STIFF	5 - 8	1 - 2
DENSE	31 - 50	STIFF	9 - 15	2 - 4
VERY DENSE	OVER 50	VERY STIFF	16 - 30	4 - 8
		HARD	OVER 30	OVER 8

* NUMBER OF BLOWS OF 140 LB HAMMER FALLING 30 INCHES TO DRIVE A 2 INCH O.D. (1-3/8 INCH I.D.) SPLIT-BARREL SAMPLER THE LAST 12 INCHES OF AN 18-INCH DRIVE (ASTM-1586 STANDARD PENETRATION TEST).

PLASTICITY CHART



LABORATORY TESTS

AT	ATTERBERG LIMITS
CD	CONSOLIDATED DRAINED TRIAXIAL
CN	CONSOLIDATION
CR	CORROSIVITY
CU	CONSOLIDATED UNDRAINED TRIAXIAL
DS	DIRECT SHEAR
HY	HYDROMETER
PR	PERMEABILITY
RV	R-VALUE
SA	SIEVE ANALYSIS
TC	CYCLIC TRIAXIAL
UC	UNCONFINED COMPRESSION
UU	UNCONSOLIDATED UNDRAINED TRIAXIAL
-200	% PASSING NO. 200 SIEVE

PROPERTIES

c	COHESION
DD	DRY DENSITY
EI	EXPANSION INDEX
LL	LIQUID LIMIT
MC	MOISTURE CONTENT
N_f	FIELD BLOW COUNT
PI	PLASTICITY INDEX
Su	UNDRAINED STRENGTH

UNDRAINED SHEAR STRENGTH

V	FIELD VANE
P	POCKET PENETROMETER
T	TORVANE
Q	UNCONFINED COMPRESSION
U	UNCONSOLIDATED UNDRAINED TRIAXIAL

INCREASING VISUAL MOISTURE CONTENT

WET
MOIST
DRY

Boring and Test Pit Legend

Bellota Weir Modification Project
Linden, CA

Date
3/8/2021

Figure
A-1

LEGEND_JULY 2016; 10171169_KIEFER_LANDFILL_LEACHATE_PIPELINE.GPJ; HDR_FOLSOM_OAKLAND_MARCH 2017_WIP.GLB; 11/2/20

Project: Bellota Weir Modification Project		Boring ID: HDR-B1	Sheet 1 of 3 Sheets
Project Location: Linden, CA			
Project Number: 10209137			

Start Date:	End Date:	Logged By:	Checked By:	Date Checked:
1/21/2021	1/21/2021	Sagar Raj Satyal	Mark Stanley	04/02/2021

Drilling Company (Rig Type): Taber Drilling (CME 55 Track Mounted)	Inspector: Aaron - San Joaquin County	Weather Conditions: warm/cloudy
Drill Method: Auger / Mud	Drilled By: Toby/Ernesto/Jacob	Elevation Top of Boring: 124.0 ft. Vertical Datum: NAVD88
Drill Bit (Type/Size): Auger / 4" : Coring Bit / 94 mm	Total Depth Drilled: 71.5 ft.	Latitude: 38.052124° Longitude: -121.011166° Horizontal Datum: WGS84
Hammer Type: Automatic	Hole Backfill: Neat Cement Grout	Northing: Easting: Coordinate System:
Hammer Efficiency: 76.5 % / Rod Type: NWJ	Total Number of Samples: 18 Disturbed: 18 Undisturbed: 0	Initial Groundwater Depth: 18 ft Static Groundwater Depth:

ELEV	DEPTH	SAMPLE	Blows/6" or Press.	N_f	LEGEND	DESCRIPTION OF MATERIALS	% REC	Samp No.	Laboratory: Fines	LL	PI	DD	MC	Su (ksf)	REMARKS
						Light gray, 9" concrete.									Started at 10:45 AM
						CLAYEY GRAVEL with Sand (GC): medium dense, brown, moist, max. gravel size up to 1.5", sub-rounded, fine sand, less gravel with depth.		S1							Cored concrete slab with 6" bucket drill Hand Augered to 5 ft
125	5		5 4 6	10		LEAN CLAY with Sand (CL): stiff, brown, moist, 71% fines.	100	S2	71						4" Auger
			6 6 8	14		Approx. 30% sand.	100	S3		31	12		15		
120	10		11 14 19	33		SANDY SILT (ML): hard, grayish brown, moist, 61% non-plastic, fines.	78	S4	61			107	15		
			10 7 9	16			100	S5		NP	NP				
115	15		4 3 3	6		Poorly Graded SAND with Silt (SP-SM): loose, brown, moist.	11	S6							Low recovery, sampling at 17.5 ft. with catcher
			4 6 9	15		Medium dense, brownish gray, moist, 13% non-plastic fines and approx. 10% gravel, gravel up to 1", sub-rounded, fine to medium sand.	100	S7	13	NP	NP		28		Ground water at 18 ft Switched to mud-rotary
110	20		5 8 13	21		Poorly Graded GRAVEL with Clay and Sand (GP-GC): medium dense, gray, wet, 6% fines, 15% sand, sub-angular gravel, up to 1.5".	28	S8	6						Rig chattering
105	25		14 17 10	27		CLAYEY SAND with Gravel (SC): medium dense, brownish gray, wet, sub-angular gravel up to 1", approx. 10% fines, 20% gravel, medium to fine sub-angular sand.	33	S9					17		Cuttings contain sand and gravel up to 1"
100															

DRAFT

Project: Bellota Weir Modification Project	Boring ID: HDR-B1	Sheet 2 of 3 Sheets
Project Location: Linden, CA		
Project Number: 10209137		

ELEV	DEPTH	SAMPLE	Blows/6" or Press.	N_f	LEGEND	DESCRIPTION OF MATERIALS	% REC	Samp No.	Laboratory: Fines	LL	PI	DD	MC	Su (ksf)	REMARKS
			7 9 7	16			45	S10							
95	35		34 22 16	38		Dense, dark gray, medium to fine sand, sub-rounded gravel, up to 1", 31% gravel, 19% fines.	55	S11	19	52	23		28		
90	40		8 13 10	23		LEAN CLAY with Sand (CL): very stiff, light gray, wet, medium plasticity, very stiff, fine sub-rounded sand, approx. 20% sand.	28	S12							
85	45		7 10 11	21		SILTY SAND (SM): medium dense, light grayish brown, wet, medium to fine sand, approx. 30% fines.	100	S13							
80	50		10 12 15	27			100	S14		50	12				
75	55		6 14 13	27		SILT with Sand (ML): very stiff, light gray, wet, approx. 20% sand.	100	S15							
70	60		8 21 36	57		CLAYEY SAND (SC): very dense, gray, wet, medium to fine sand, sub-rounded, approx. 20% fines.	100	S16							
65	65		10 12 19	31		SILT with Sand (ML): hard, light gray, wet, dark gray sand, trace gravel up to 0.5", approx. 20% sand.	100	S17							

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HDR SOIL BORING LOG_2017_MARCH_R1; 10209137_BELLOTA_WEIR_EXPLORATIONS-SRS-V2.GPJ; HDR_FOLSOM_OAKLAND_FEB 2018_WIP.GLB; 3/8/21

Project: Bellota Weir Modification Project	Boring ID:	Sheet
Project Location: Linden, CA	HDR-B1	3 of 3 Sheets
Project Number: 10209137		

ELEV	DEPTH	SAMPLE	Blows/6" or Press.	N_f	LEGEND	DESCRIPTION OF MATERIALS	% REC	Samp No.	Laboratory: Fines	LL	PI	DD	MC	Su (ksf)	REMARKS
60	70		14 21 24	45		Poorly Graded SAND (SP): dense, gray, wet.	100	S18							Terminated at 71.5 ft. Grouted with neat cement grout using 10 bags of cement. Completed at 5:30 PM

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HDR SOIL BORING LOG_2017_MARCH_R1; 10209137_BELLOTA_WEIR_EXPLORATIONS-SRS-V2.GPJ; HDR_FOLSOM_OAKLAND_FEB 2018_WIP.GLB; 3/8/21

Project: Bellota Weir Modification Project		Boring ID: HDR-B2	Sheet 1 of 3 Sheets
Project Location: Linden, CA			
Project Number: 10209137			
Start Date: 1/19/2021	End Date: 1/19/2021	Logged By: Sagar Raj Satyal	Checked By: Mark Stanley / Date Checked: 04/02/2021
Drilling Company (Rig Type): Taber Drilling (CME 55 Track Mounted)		Inspector: Aaron - San Joaquin County	Weather Conditions: sunny/clear/windy
Drill Method: Auger / Mud		Drilled By: Toby/Ernesto/Jacob	Elevation Top of Boring: 132.0 ft. Vertical Datum: NAVD88
Drill Bit (Type/Size): Auger / 4" : Coring Bit / 94 mm		Total Depth Drilled: 71.5 ft.	Latitude: 38.052257° Longitude: -121.012052° Horizontal Datum: WGS84
Hammer Type: Automatic		Hole Backfill: Neat Cement Grout	Northing: Easting: Coordinate System:
Hammer Efficiency: 76.5 %	Rod Type: NWJ	Total Number of Samples: 18 Disturbed: 18 Undisturbed: 0	Initial Groundwater Depth: 30 ft Static Groundwater Depth:

ELEV	DEPTH	SAMPLE	Blows/6" or Press.	N_f	LEGEND	DESCRIPTION OF MATERIALS	% REC	Samp No.	Laboratory: Fines	LL	PI	DD	MC	Su (ksf)	REMARKS
						SILTY SAND (SM): loose, light brown, moist, fine sand, approx. 30% fines.									Started at 9:40 AM Started with 4" Auger
130			3 2 2	4			89	S1	34						
	5		3 2 3	5			94	S2					8		
125			3 2 3	5		SILT with Sand (ML): medium stiff, dark brown, moist, some roots, approx. 20% sand, fine sand.	100	S3							
	10		6 4 5	9		CLAYEY SAND (SC): loose, dark grayish brown, moist, fine sand, trace gravel up to 0.5", sub-rounded, approx. 30% fines.	83	S4		35	15	90	16	2.50 P	
120			5 4 8	12		Approx. 40% fines.	78	S5						2.50 P	
	15		6 7 10	17		Approx. 30% fines.	67	S6		35	14	97	16		
115															
	20		5 4 5	9		Dark brown, 46% fines.	100	S7	46						
110															
	25		2 2 3	5		Dark gray, sub-rounded sand, approx. 20% fines.	28	S8							Low recovery, gravel in shoe
105															

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Project: Bellota Weir Modification Project	Boring ID: HDR-B2	Sheet 2 of 3 Sheets
Project Location: Linden, CA		
Project Number: 10209137		

ELEV	DEPTH	SAMPLE	Blows/6" or Press.	N_f	LEGEND	DESCRIPTION OF MATERIALS	% REC	Samp No.	Laboratory: Fines	LL	PI	DD	MC	Su (ksf)	REMARKS
			2 2 2	4		Dark gray with mottled brown.	100	S9							Ground water at 30 ft Switched to mud-rotary
100	35		25 24 22	46		Well-Graded GRAVEL with Clay and Sand (GW-GC): dense, dark gray, wet, sub-rounded to sub-angular gravel up to 1", sub-rounded sand, 54% gravel and 37% sand.	39	S10	9						Rig chattering
95	40		10 8 7	15		LEAN CLAY with Sand (CL): very stiff, light gray with mottled brown, wet, trace gravel, sub-rounded gravel up to 1", coarse to medium sub-rounded sand.	0	S11							No recovery, sampling at 42.5 ft. after cleaning the hole
90			18 11 8	19			67	S12	82						
85	45		9 6 6	12			0	S13							No recovery, gravel in shoe
80	50		16 10 17	27		CLAYEY SAND (SC): medium dense, light gray, wet, fine sand, sub-rounded, low to medium plasticity fines, approx. 30% fines.	94	S14							
75	55		6 10 12	22		FAT CLAY with Sand (CH): very stiff, light gray, wet, fine sand, sub-rounded, 26% sand.	72	S15	74	71	39				
70	60		9 10 11	21		SILTY SAND (SM): medium dense, light gray, wet, fine sand, sub-rounded, approx. 40% fines.	100	S16							
	65		7 11 13	24		Approx. 20% fines.	39	S17							

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Project: Bellota Weir Modification Project	Boring ID: HDR-B2	Sheet 3 of 3 Sheets
Project Location: Linden, CA		
Project Number: 10209137		

ELEV	DEPTH	SAMPLE	Blows/6" or Press.	N_f	LEGEND	DESCRIPTION OF MATERIALS	% REC	Samp No.	Laboratory: Fines	LL	PI	DD	MC	Su (ksf)	REMARKS
	70		7 13 16	29		SILT with Sand (ML): very stiff, light gray, wet, fine, sub-rounded sand, approx. 20% sand.	39	S18							Terminated at 71.5 ft. Grouted with neat cement grout using 6 bags of cement. Completed at 2:30 PM

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Project: Bellota Weir Modification Project		Boring ID: HDR-B3	Sheet 1 of 3 Sheets
Project Location: Linden, CA			
Project Number: 10209137			

Start Date:	End Date:	Logged By:	Checked By:	Date Checked:
1/18/2021	1/18/2021	Sagar Raj Satyal	Mark Stanley	04/02/2021

Drilling Company (Rig Type): Taber Drilling (CME 55 Track Mounted)	Inspector: Aaron - San Joaquin County	Weather Conditions: sunny/clear/windy
Drill Method: Auger / Mud	Drilled By: Toby/Ernesto/Jacob	Elevation Top of Boring: 135.0 ft. Vertical Datum: NAVD88
Drill Bit (Type/Size): Auger / 4" : Coring Bit / 94 mm	Total Depth Drilled: 71.5 ft.	Latitude: 38.052581° Longitude: -121.011907° Horizontal Datum: WGS84
Hammer Type: Automatic	Hole Backfill: Neat Cement Grout	Northing: Easting: Coordinate System:
Hammer Efficiency: 76.5 % / Rod Type: NWJ	Total Number of Samples: 17 Disturbed: 17 Undisturbed: 0	Initial Groundwater Depth: 25 ft Static Groundwater Depth:

ELEV	DEPTH	SAMPLE	Blows/6" or Press.	N_f	LEGEND	DESCRIPTION OF MATERIALS	% REC	Samp No.	Laboratory: Fines	LL	PI	DD	MC	Su (ksf)	REMARKS
						Aggregate base 2" thick.									Started at 9:00 AM
			3 4 4	8		SILT (ML): loose, light brown, moist, trace gravel up to 0.5", approx. 10% sand.	89	S1							Started with 4" Auger
130	5		4 3 5	8		SILT with Sand (ML): loose, light brown, moist, approx. 20% fine sand, non-plastic fines.	89	S2		NP	NP				
			2 3 2	5		SILT (ML): loose, light brown, moist, trace fine sand.	67	S3							
125	10		6 6 6	12		Trace gravel up to 0.5".	67	S4						4.5+ P	
			5 5 5	10		SILTY SAND (SM): medium dense, light gray, 39% fines.	78	S5	39						
120	15		5 5 5	10		32% fines.	100	S6	32			99	15		
115	20		3 2 3	5		SANDY SILT (ML): loose, dark gray, moist, approx. 30% fine sand.	28	S7							
110	25		1 2 2	4		SILTY SAND (SM): loose, light gray, wet, fine, sub-rounded, sand, 33% fines.	100	S8	33						Ground water at 25 ft Switched to mud-rotary

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Project: Bellota Weir Modification Project	Boring ID: HDR-B3	Sheet 2 of 3 Sheets
Project Location: Linden, CA		
Project Number: 10209137		

ELEV	DEPTH	SAMPLE	Blows/6" or Press.	N_f	LEGEND	DESCRIPTION OF MATERIALS	% REC	Samp No.	Laboratory: Fines	LL	PI	DD	MC	Su (ksf)	REMARKS
			1 2 3	5		Light gray with mottled brown, approx. 40% fines.	83	S9							
100	35		26 20 22	42		CLAYEY GRAVEL with Sand (GC): dense, gray, wet, sub-rounded gravel up to 0.5", approx. 20% fines and 30% sand.	45	S10							Rig chattering
95	40		9 10 10	20		SILTY SAND (SM): medium dense, light brown, wet, sub-rounded sand, approx. 40% fines, trace gravel up to 0.5".	45	S11		49	17				
90	45		8 10 9	19		CLAYEY SAND with Gravel (SC): medium dense, light gray, wet, sub-rounded sand, approx. 30% gravel, up to 0.5", approx. 30% fines.	22	S12							Low recovery, sand catcher added
85	50		6 8 10	18		SILT with Sand (ML): very stiff, light gray, wet, fine sub-rounded sand, approx. 30% sand.	100	S13							
80	55		8 8 12	20		SILT (ML): very stiff, light gray, wet, approx. 10% fine sand.	89	S14		48	17				
75	60		15 20 15	35		SILT with Sand (ML): hard, light gray, wet, approx. 30% sub-rounded sand.	45	S15							
70	65		12 15 12	27		SILTY SAND (SM): medium dense, light gray, wet, fine sand, approx. 40% fines.	89	S16							

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HDR SOIL BORING LOG_2017_MARCH_R1; 10209137_BELLOTA_WEIR_EXPLORATIONS-SRS-V2.GPJ; HDR_FOLSOM_OAKLAND_FEB 2018_WIP.GLB; 3/8/21

Project: Bellota Weir Modification Project	Boring ID: HDR-B3	Sheet 3 of 3 Sheets
Project Location: Linden, CA		
Project Number: 10209137		

ELEV	DEPTH	SAMPLE	Blows/6" or Press.	N_f	LEGEND	DESCRIPTION OF MATERIALS	% REC	Samp No.	Laboratory: Fines	LL	PI	DD	MC	Su (ksf)	REMARKS
65	70		14 13 14	27		SANDY SILT (ML): medium dense, light gray, wet, 54% fines.	94	S17	54						Terminated at 71.5 ft. Grouted with neat cement grout using 8 bags of cement. Completed at 3:00 PM. Grouted next morning

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HDR SOIL BORING LOG_2017_MARCH_R1; 10209137_BELLOTA_WEIR_EXPLORATIONS-SRS-V2.GPJ; HDR_FOLSOM_OAKLAND_FEB 2018_WIP.GLB; 3/8/21

Project: Bellota Weir Modification Project		Boring ID: HDR-B4	Sheet 1 of 3 Sheets
Project Location: Linden, CA			
Project Number: 10209137			

Start Date:	End Date:	Logged By:	Checked By:	Date Checked:
1/20/2021	1/20/2021	Sagar Raj Satyal	Mark Stanley	04/02/2021

Drilling Company (Rig Type): Taber Drilling (CME 55 Track Mounted)	Inspector: Aaron - San Joaquin County	Weather Conditions: sunny/warm
Drill Method: Auger / Mud	Drilled By: Toby/Ernesto/Jacob	Elevation Top of Boring: 131.0 ft. Vertical Datum: NAVD88
Drill Bit (Type/Size): Auger / 4" : Coring Bit / 94 mm	Total Depth Drilled: 71.5 ft.	Latitude: 38.052946° Longitude: -121.011322° Horizontal Datum: WGS84
Hammer Type: Automatic	Hole Backfill: Neat Cement Grout	Northing: Easting: Coordinate System:
Hammer Efficiency: 76.5 %	Rod Type: NWJ	Total Number of Samples: 18 Disturbed: 18 Undisturbed: 0
		Initial Groundwater Depth: 21 ft Static Groundwater Depth:

ELEV	DEPTH	SAMPLE	Blows/6" or Press.	N_f	LEGEND	DESCRIPTION OF MATERIALS	% REC	Samp No.	Laboratory: Fines	LL	PI	DD	MC	Su (ksf)	REMARKS
130			22 25 25	50		SILTY SAND with Gravel (SM): dense, light brown, moist, approx. 30% fines, and 20% gravel, sub-rounded gravel up to 1", some roots.	83	S1					4		Started at 11:30 AM Started with 4" Auger
125	5		17 16 10	26			22	S2							
			2 2 2	4		SANDY LEAN CLAY (CL): soft, brown, moist, fine sand, trace gravel up to 0.5", sub-rounded, some roots, approx. 30% sand.	83	S3		28	9				
120	10		3 4 5	9		SILTY SAND (SM): loose, brown, moist, sub-rounded fine sand, 41% fines.	67	S4	41			98	14		
			3 4 3	7		SILT with Sand (ML): soft, brown, moist, fine sand, trace gravel up to 0.5", sub-rounded, some roots, approx. 30% sand.	78	S5							
115	15		3 4 4	8		SILTY SAND (SM): loose, brown, moist, sub-rounded fine sand, 28% fines.	67	S6	28						
110	20		3 3 3	6		SILTY CLAYEY SAND (SC-SM): loose, light brownish gray, moist, approx. 40% fines, fine sub-rounded sand.	94	S7		25	4				Ground water at 21 ft Switched to mud-rotary
105	25		5 6 5	11			0	S8							

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Project: Bellota Weir Modification Project	Boring ID: HDR-B4	Sheet 2 of 3 Sheets
Project Location: Linden, CA		
Project Number: 10209137		

ELEV	DEPTH	SAMPLE	Blows/6" or Press.	N_f	LEGEND	DESCRIPTION OF MATERIALS	% REC	Samp No.	Laboratory: Fines	LL	PI	DD	MC	Su (ksf)	REMARKS
100			5 4 8	12		SILTY SAND (SM): medium dense, brownish gray, wet, approx. 30% fines, medium to fine sand, trace, sub-rounded gravel up to 0.5".	72	S9							
	35		15 22 14	36		Poorly Graded GRAVEL with Clay and Sand (GP-GC): dense, dark gray, wet, sub-rounded gravel up to 1.5", 73% gravel, 20% sand.	45	S10	7						
95															
	40		11 7 7	14		SILT with Sand (ML): stiff, brownish gray, wet, approx. 20% fine sand.	72	S11							
90															
	45		14 18 18	36		Hard, light gray.	33	S12							Low recovery, sampling at 47.5 ft
85			19 15 19	34		Medium to coarse white sand, trace gravel up to 1".	33	S13							
	50		12 9 14	23		Fine sand, no gravel.	100	S14						0.87 U	
80															
	55		11 13 14	27		Coarser sand, approx. 20% sand.	33	S15							
75															
	60		11 20 22	42		Fine sand, approx. 20% sand.	67	S16						0.62 U	Sample S16 B highly disturbed
70															
	65		16 25 19	44		SILTY SAND (SM): dense, gray, wet, medium to fine sand, sub-angular to sub-rounded, 40% fines.	61	S17	40						
65															

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HDR SOIL BORING LOG_2017_MARCH_R1; 10209137_BELLOTA_WEIR_EXPLORATIONS-SRS-V2.GPJ; HDR_FOLSOM_OAKLAND_FEB 2018_WIP.GLB; 3/8/21

Project:	Bellota Weir Modification Project	Boring ID:	Sheet
Project Location:	Linden, CA	HDR-B4	3 of 3 Sheets
Project Number:	10209137		

ELEV	DEPTH	SAMPLE	Blows/6" or Press.	N_f	LEGEND	DESCRIPTION OF MATERIALS	% REC	Samp No.	Laboratory: Fines	LL	PI	DD	MC	Su (ksf)	REMARKS
60	70		9 18 30	48		SANDY SILT (ML): hard, dark brown, wet, approx. 30% sand, fine sand, trace gravel, sub-angular, up to 1".	100	S18							Terminated at 71.5 ft. Grouted with neat cement grout using 12 bags of cement. Completed at 4:40 PM. Grouted next morning

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HDR SOIL BORING LOG_2017_MARCH_R1; 10209137_BELLOTA_WEIR_EXPLORATIONS-SRS-V2.GPJ; HDR_FOLSOM_OAKLAND_FEB 2018_WIP.GLB; 3/8/21

Project: Bellota Weir Modification Project			Boring ID: HDR-B5	Sheet 1 of 3 Sheets
Project Location: Linden, CA				
Project Number: 10209137				
Start Date: 1/19/2021	End Date: 1/21/2021	Logged By: Sagar Raj Satyal	Checked By: Mark Stanley	Date Checked: 04/02/2021
Drilling Company (Rig Type): Taber Drilling (CME 55 Track Mounted)		Inspector: Aaron - San Joaquin County	Weather Conditions: sunny/warm/windy	
Drill Method: Auger / Mud		Drilled By: Toby/Ernesto/Jacob	Elevation Top of Boring: 131.0 ft. Vertical Datum: NAVD88	
Drill Bit (Type/Size): Auger / 4" : Coring Bit / 94 mm		Total Depth Drilled: 71.5 ft.	Latitude: 38.053055° Longitude: -121.011818° Horizontal Datum: WGS84	
Hammer Type: Automatic		Hole Backfill: Neat Cement Grout	Northing: Easting: Coordinate System:	
Hammer Efficiency: 76.5 %	Rod Type: NWJ	Total Number of Samples: 18 Disturbed: 18 Undisturbed: 0	Initial Groundwater Depth: 25 ft Static Groundwater Depth:	

ELEV	DEPTH	SAMPLE	Blows/6" or Press.	N_f	LEGEND	DESCRIPTION OF MATERIALS	% REC	Samp No.	Laboratory: Fines	LL	PI	DD	MC	Su (ksf)	REMARKS
			2 2 3	5		SILTY SAND (SM): loose, light brown, moist, fine sand, approx. 20% non-plastic fines, trace gravel, sub-rounded, up to 0.5", some roots.	78	S1							Started at 3:00 PM Started with 4" Auger
130	5		4 2 3	5			78	S2		NP	NP				
			2 2 2	4			72	S3							
125	10		3 2 3	5		Increased fines, approx. 20%.	83	S4							
			3 3 5	8		SILT with Sand (ML): medium stiff, brown, moist, approx. 20% fine sand, sub-rounded sand.	100	S5							
120	15		4 7 9	16			94	S6							
115	20		3 4 3	7		LEAN CLAY with Sand (CL): medium stiff, brownish gray, wet, approx. 20% fine sand, sub-rounded sand.	100	S7		31	10	89	32		
110	25		4 3 4	7		SANDY SILT (ML): medium stiff, dark gray, wet, approx. 40% fine sand.	94	S8							Ground water at 25 ft Switched to mud-rotary

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Project: Bellota Weir Modification Project	Boring ID: HDR-B5	Sheet 2 of 3 Sheets
Project Location: Linden, CA		
Project Number: 10209137		

ELEV	DEPTH	SAMPLE	Blows/6" or Press.	N_f	LEGEND	DESCRIPTION OF MATERIALS	% REC	Samp No.	Laboratory: Fines	LL	PI	DD	MC	Su (ksf)	REMARKS
			8 15 23	38		CLAYEY GRAVEL with Sand (GC): dense, dark gray, wet, sub-rounded gravel, up to 1", approx. 50% gravel and 20% fines.	33	S9							Rig chattering
100	35		15 9 12	21		SILTY SAND with Gravel (SM): dense, grayish brown, wet, approx. 50% sand, 30% fines, and 20% gravel, sub-rounded gravel up to 1".	67	S10							Stopped for the day Started at 7:30 AM 1/20/2021
95	40		16 9 14	23		SILT with Sand (ML): very stiff, light brownish gray, wet, 31% sand, trace gravel up to 0.5".	67	S11	68			72	59		
90	45		12 18 28	46		SANDY SILT (ML): hard, dark brownish gray, wet, approx. 30% fine sand.	33	S12							1.5" gravel at the shoe, low recovery, highly disturbed sample
85	50		9 9 10	19		SILTY SAND (SM): medium dense, light brownish gray, wet, approx. 40% fines, trace gravel, up to 1.5".	0	S13							
			9 11 8	19		43% fines.	67	S14	43			90	39		
80	55		8 9 13	22		SILT with Sand (ML): very stiff, light brownish gray, wet, approx. 20% fine sand, trace gravel up to 0.5".	100	S15							
75	60		9 6 17	23		SILTY SAND (SM): medium dense, light gray, wet, approx. 20% fines.	100	S16							
70	65		8 12 15	27			100	S17							

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Project: Bellota Weir Modification Project	Boring ID: HDR-B5	Sheet 3 of 3 Sheets
Project Location: Linden, CA		
Project Number: 10209137		

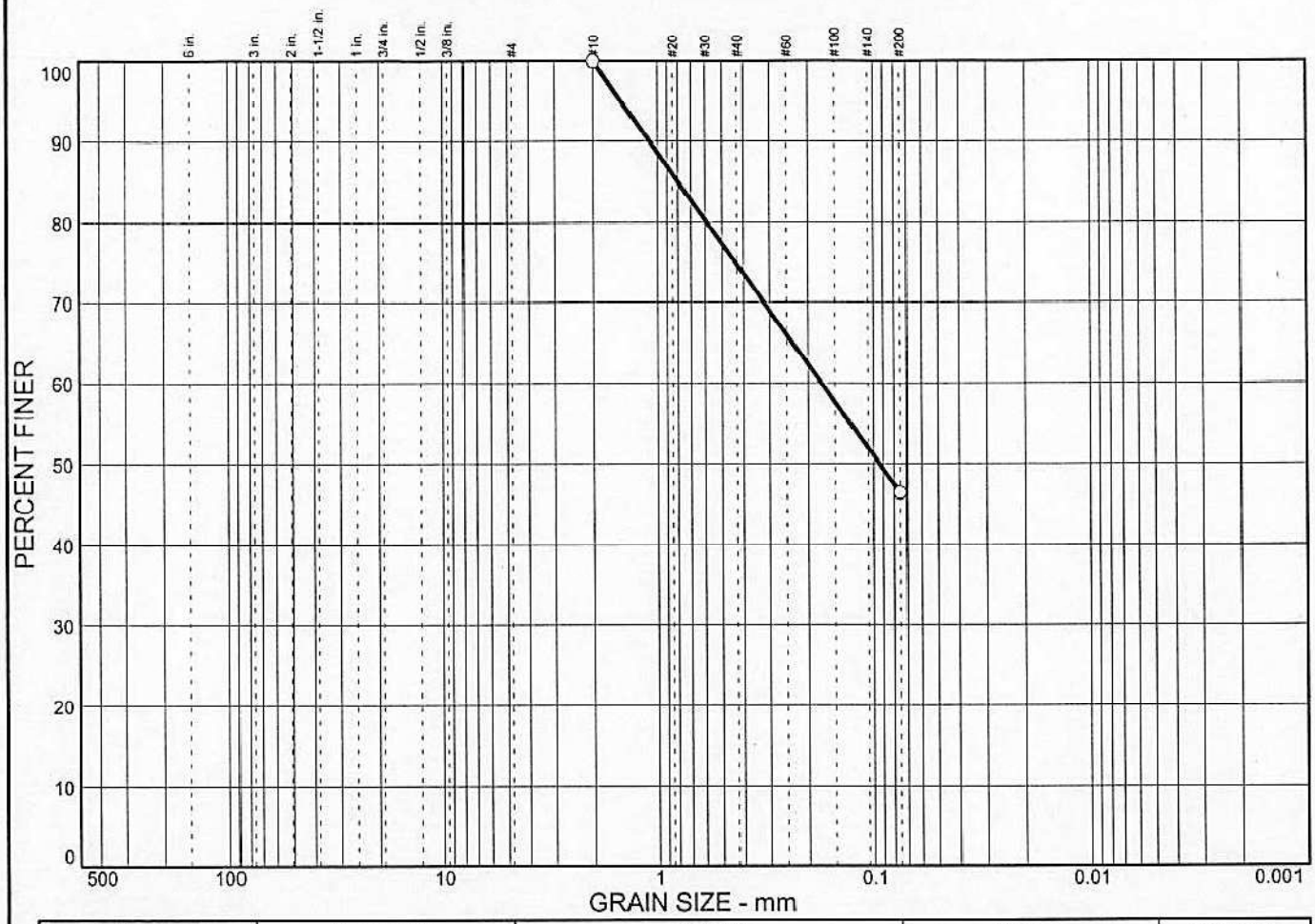
ELEV	DEPTH	SAMPLE	Blows/6" or Press.	N_f	LEGEND	DESCRIPTION OF MATERIALS	% REC	Samp No.	Laboratory: Fines	LL	PI	DD	MC	Su (ksf)	REMARKS
65	70		13 19 30	49		CLAYEY SAND (SC): dense, light gray, wet, approx. 15% fines, trace gravel up to 0.5".	100	S18							Terminated at 71.5 ft. Grouted with neat cement grout using 11 bags of cement. Copleted at 10:50 AM

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Appendix D. Laboratory Testing

Attachment A-2
Laboratory Test Results

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	0.0	53.7	46.3	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#10	100.0		
#200	46.3		

* (no specification provided)

Material Description

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= 0.799 D_{60}= 0.173 D_{50}= 0.0940
D_{30}= D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= AASHTO=

Remarks

Total dry weight tested= 480 grams.

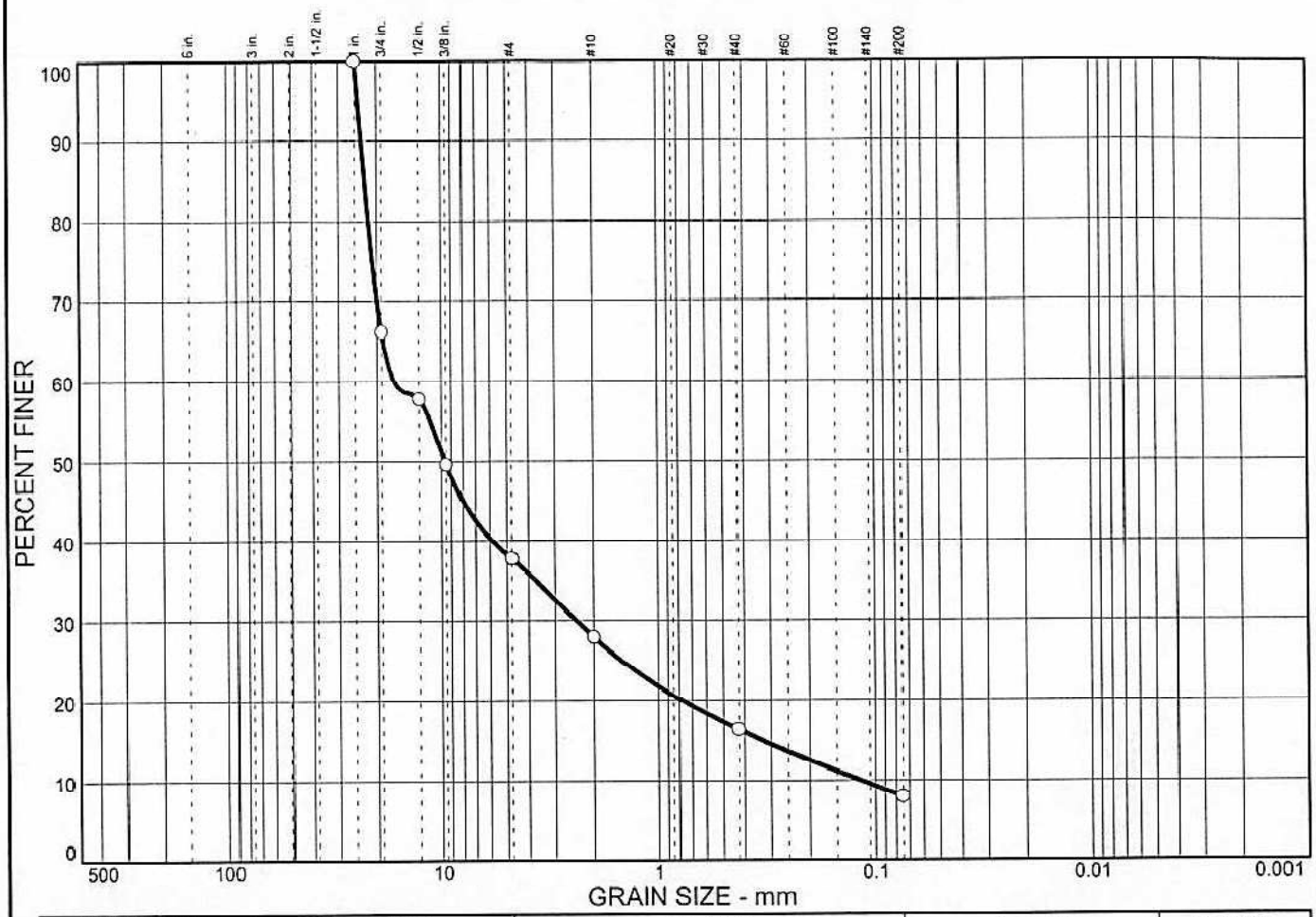
Sample No.: BI-1 **Source of Sample:** **Date:** 1-27-05
Location: **Elev./Depth:** 10'

Taber Since 1954

Client: CH2M Hill
Project: Bellota Weir / #175404.AA.P2.03

Project No: 2T2/305/19 **Figure:** 1

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	62.2	29.8	8.0	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1 in.	100.0		
0.75 in.	66.1		
0.5 in.	57.8		
0.375 in.	49.6		
#4	37.8		
#10	28.0		
#40	16.4		
#200	8.0		

* (no specification provided)

Material Description

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= 22.8 D_{60}= 16.6 D_{50}= 9.65
D_{30}= 2.40 D_{15}= 0.328 D_{10}= 0.117
C_u= 142.50 C_c= 2.97

Classification

USCS= AASHTO=

Remarks

Total dry weight tested= 463 grams.

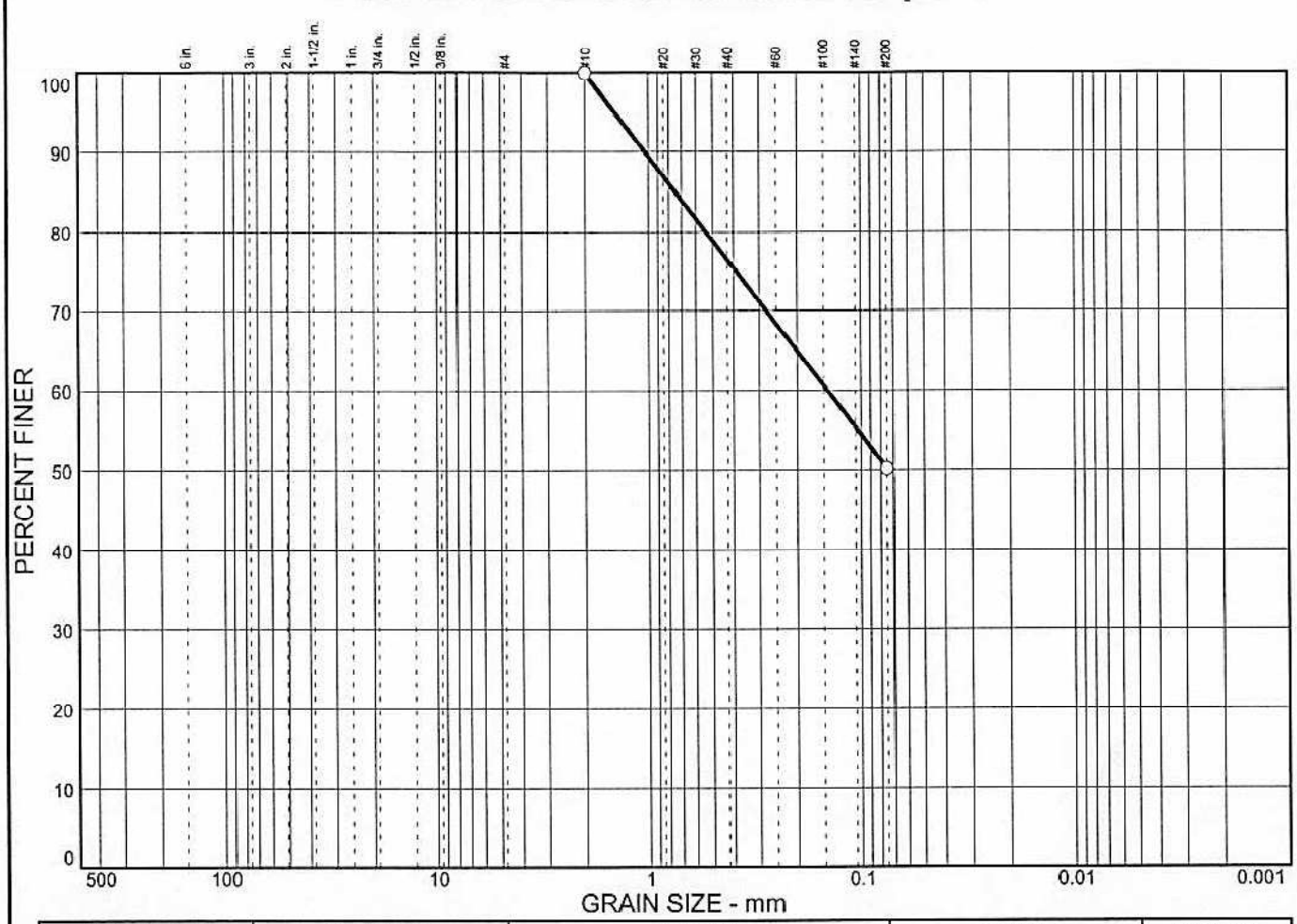
Sample No.: BI-1 **Source of Sample:** **Date:** 1-27-05
Location: **Elev./Depth:** 25'

Taber Since 1954

Client: CH2M Hill
Project: Bellota Weir / #175404.AA.P2.03

Project No: 2T2/305/19 **Figure:** 1

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	0.0	49.9	50.1	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#10	100.0		
#200	50.1		

* (no specification provided)

Material Description

Sandy silt

Atterberg Limits

PL= 42 LL= 45 PI= 3

Coefficients

D_{85}= 0.745 D_{60}= 0.144 D_{50}=
D_{30}= D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= ML AASHTO= A-5(1)

Remarks

Total dry weight tested= 491 grams.

Sample No.: BI-1 **Source of Sample:** **Date:** 1-27-05
Location: **Elev./Depth:** 35'

Taber
Since 1954

Client: CH2M Hill
Project: Bellota Weir / #175404.AA.P2.03

Project No: 2T2/305/19 **Figure:** 1

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	53.3	42.0	4.7	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
0.75 in.	100.0		
0.5 in.	78.2		
0.375 in.	64.9		
#4	46.7		
#10	33.3		
#40	15.0		
#200	4.7		

* (no specification provided)

Material Description

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= 14.5 D_{60}= 8.32 D_{50}= 5.65
D_{30}= 1.57 D_{15}= 0.425 D_{10}= 0.215
C_u= 38.68 C_c= 1.38

Classification

USCS= AASHTO=

Remarks

Total Dry weight tested= 493 grams.

Sample No.: BI-2 **Source of Sample:** **Date:** 1-27-05
Location: **Elev./Depth:** 5'

Taber Since 1954

Client: CH2M Hill
Project: Bellota Weir / #175404.AA.P2.03

Project No: 2T2/305/19 **Figure:** 1

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	2.9	55.3	41.8	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
0.375 in.	100.0		
#4	97.1		
#10	95.2		
#40	82.8		
#200	41.8		

* (no specification provided)

Material Description

Silty sand

Atterberg Limits

PL= NP LL= 24 PI= NP

Coefficients

D_{85}= 0.492 D_{60}= 0.147 D_{50}= 0.101
D_{30}= D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= SM AASHTO= A-4(0)

Remarks

Total dry weight tested= 532 grams.

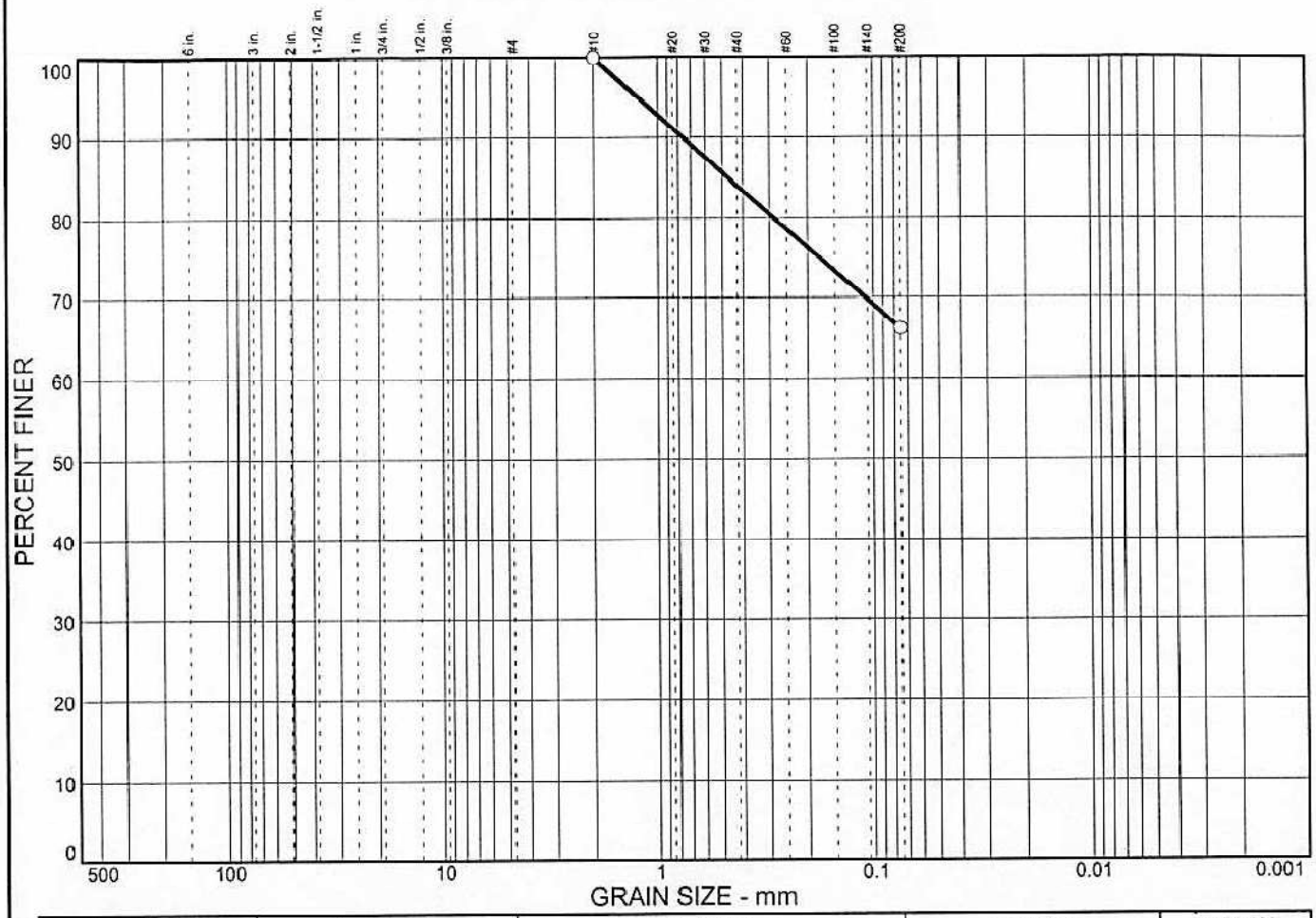
Sample No.: BI-2 **Source of Sample:** **Date:** 1-27-05
Location: **Elev./Depth:** 10'

Taber Since 1954

Client: CH2M Hill
Project: Bellota Weir / #175404.AA.P2.03

Project No: 2T2/305/19 **Figure:** 1

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	0.0	33.8	66.2	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#10	100.0		
#200	66.2		

* (no specification provided)

Material Description

Sandy elastic silt

Atterberg Limits

PL= 39 LL= 50 PI= 11

Coefficients

D_{85}= 0.466 D_{60}= D_{50}=
D_{30}= D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= MH AASHTO= A-7-5(8)

Remarks

Total dry weight tested= 406 grams.

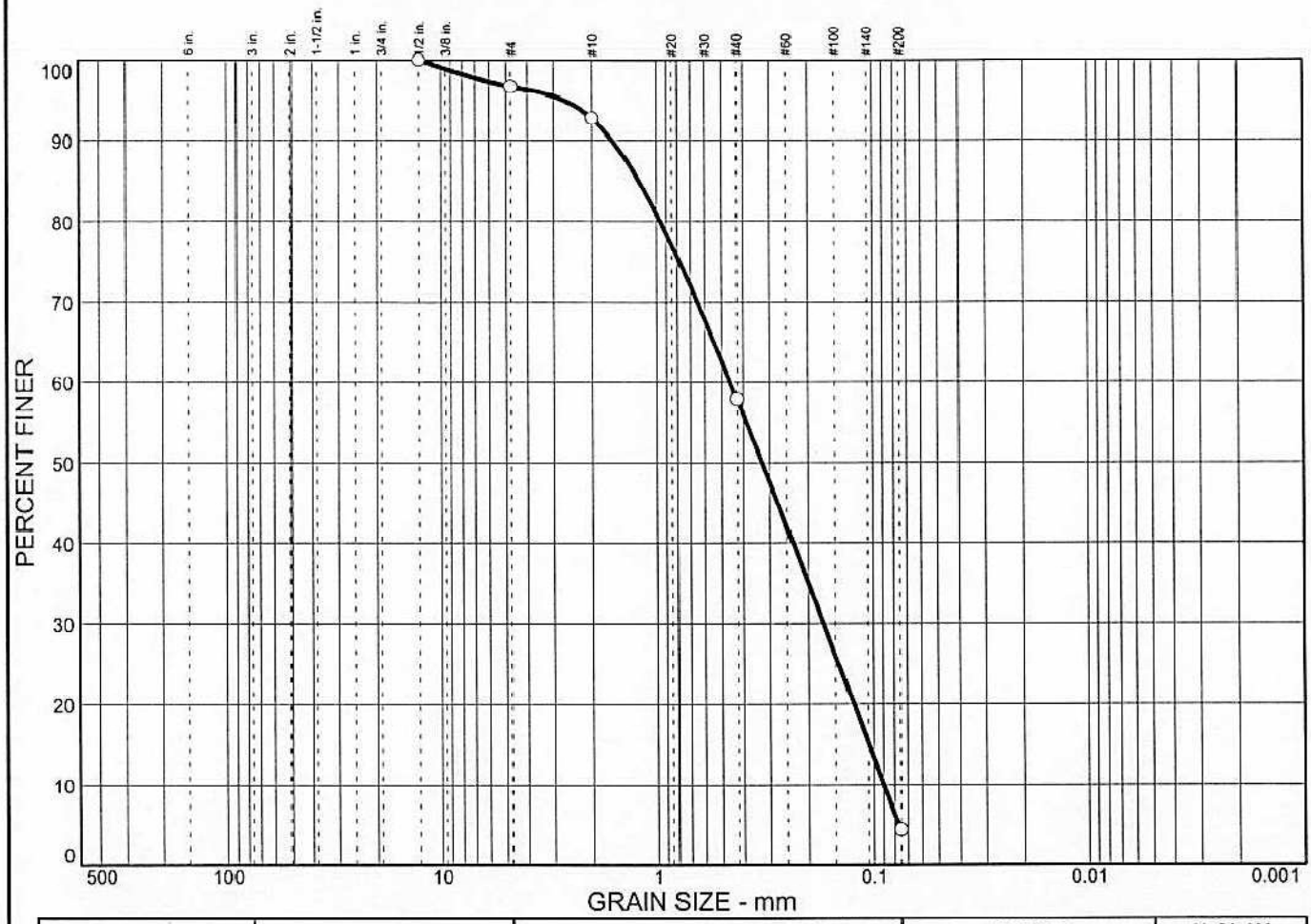
Sample No.: BI-2 **Source of Sample:** **Date:** 1-27-05
Location: **Elev./Depth:** 25'

Taber Since 1954

Client: CH2M Hill
Project: Bellota Weir / #175404.AA.P2.03

Project No: 2T2/305/19 **Figure:** 1

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	3.3	92.4	4.3	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
0.5 in.	100.0		
#4	96.7		
#10	92.8		
#40	57.9		
#200	4.3		

* (no specification provided)

Material Description

Poorly graded sand

Atterberg Limits

PL= NP LL= 31 PI= NP

Coefficients

D_{85}= 1.21 D_{60}= 0.456 D_{50}= 0.326
D_{30}= 0.170 D_{15}= 0.105 D_{10}= 0.0898
C_u= 5.08 C_c= 0.70

Classification

USCS= SP AASHTO= A-3

Remarks

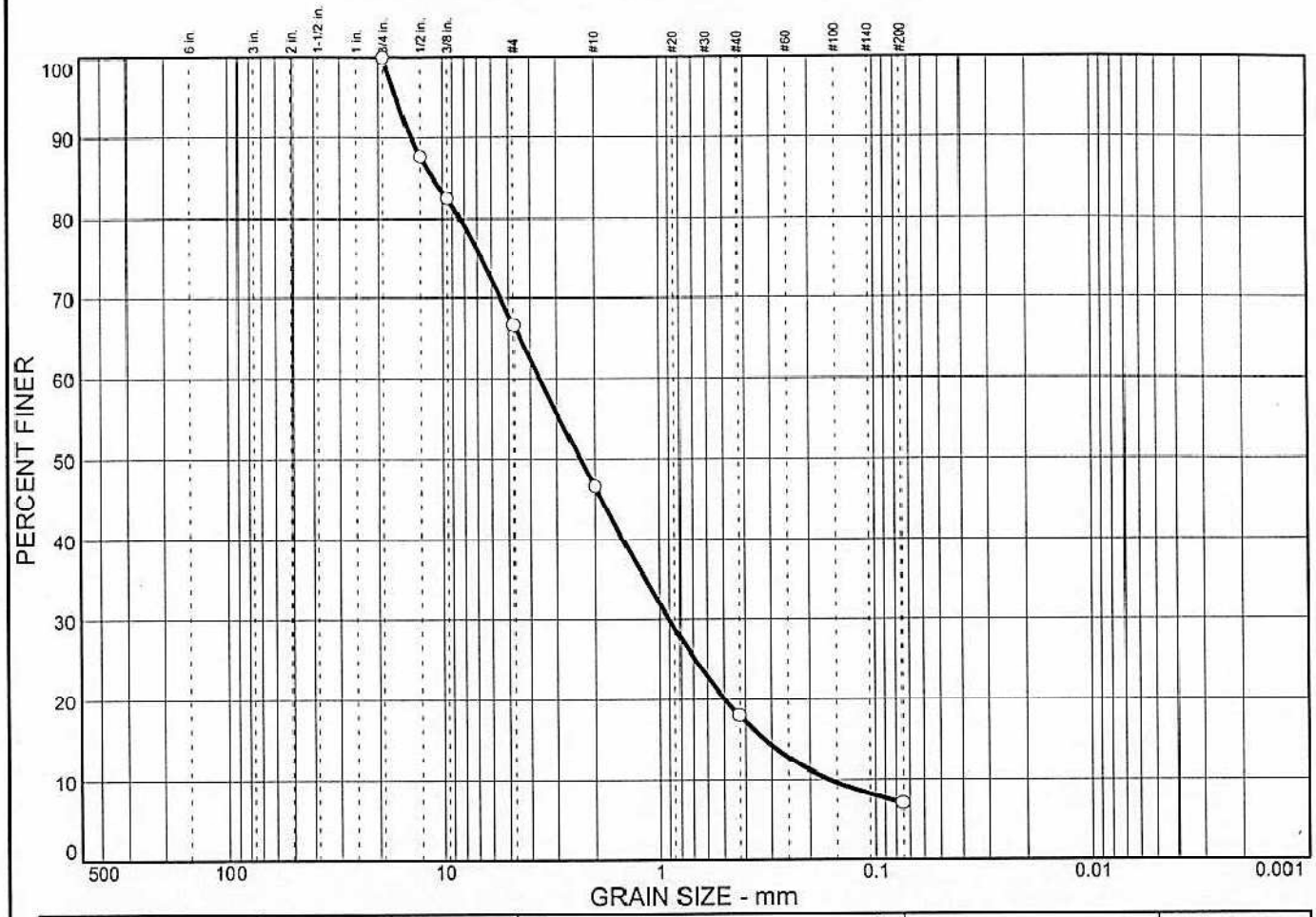
Total dry weight tested= 654 grams.

Sample No.: BI-3 **Source of Sample:** **Date:** 1-27-05
Location: **Elev./Depth:** 7.5'

Taber Since 1954

Client: CH2M Hill
Project: Bellota Weir / #175404.AA.P2.03
Project No: 2T2/305/19 **Figure:** 1

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	33.4	59.6	7.0	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
0.75 in.	100.0		
0.5 in.	87.7		
0.375 in.	82.5		
#4	66.6		
#10	46.5		
#40	18.0		
#200	7.0		

* (no specification provided)

Material Description

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= 11.0 D_{60}= 3.62 D_{50}= 2.34
D_{30}= 0.903 D_{15}= 0.324 D_{10}= 0.162
C_u= 22.28 C_c= 1.39

Classification

USCS= AASHTO=

Remarks

Total dry weight tested= 759 grams.

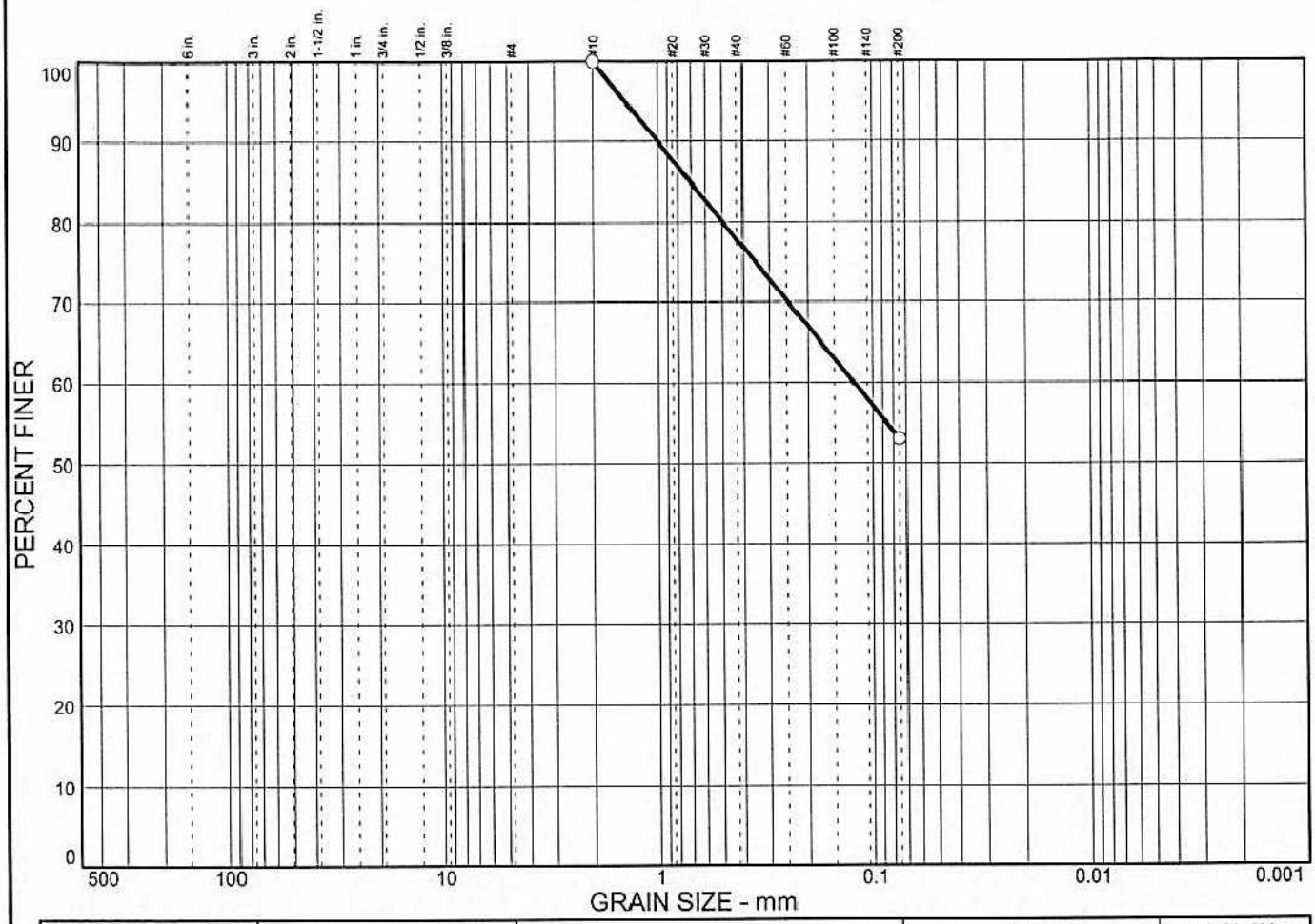
Sample No.: BI-3 **Source of Sample:** **Date:** 1-27-05
Location: **Elev./Depth:** 17.5'

Taber Since 1954

Client: CH2M Hill
Project: Bellota Weir / #175404.AA.P2.03

Project No: 2T2/305/19 **Figure:** 1

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	0.0	47.0	53.0	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#10	100.0		
#200	53.0		

* (no specification provided)

Material Description

Sandy silt

Atterberg Limits

PL= 40 LL= 42 PI= 2

Coefficients

D_{85}= 0.701 D_{60}= 0.122 D_{50}=
D_{30}= D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= ML AASHTO= A-5(1)

Remarks

Total dry weight tested= 532 grams.

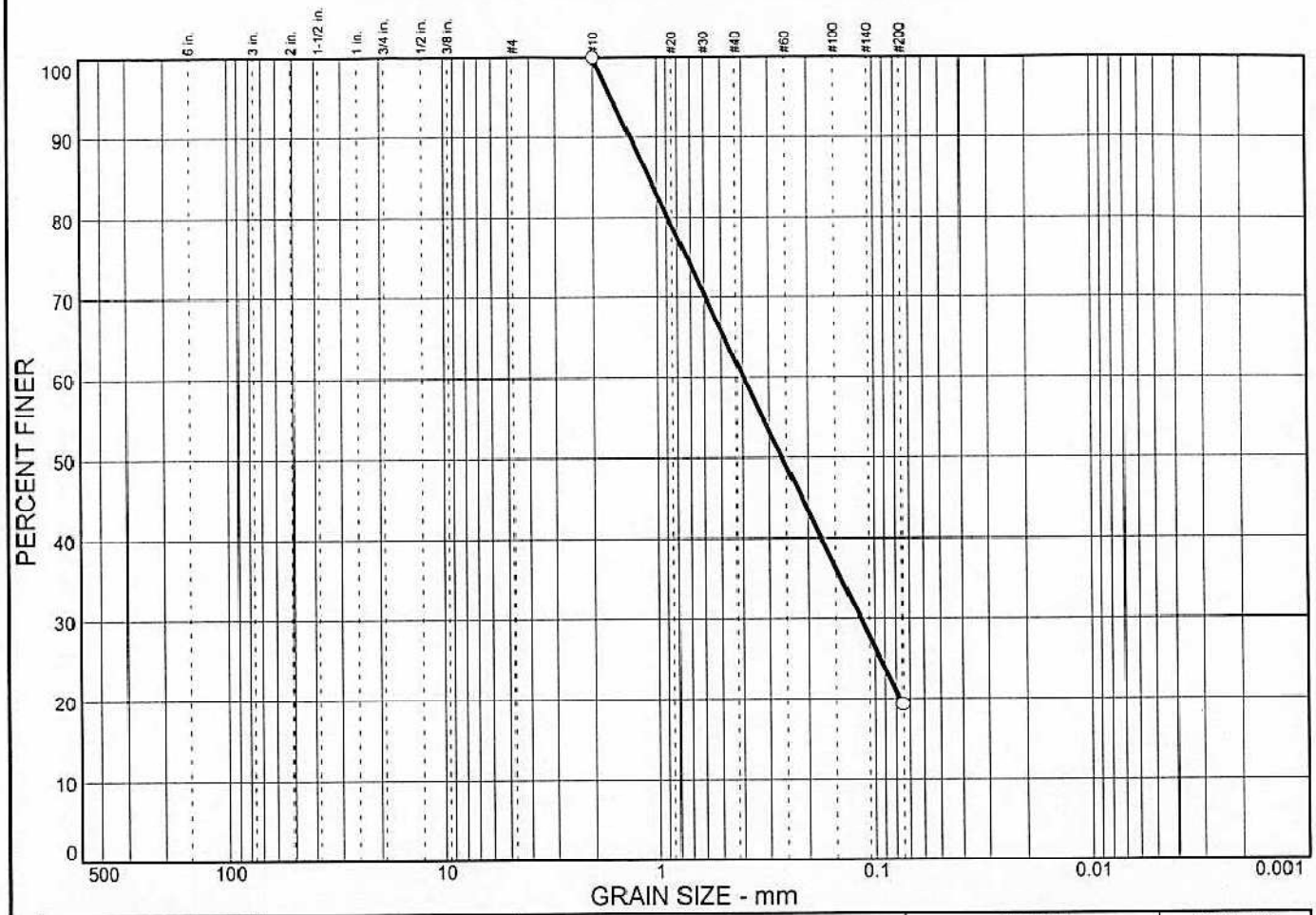
Sample No.: BI-3 **Source of Sample:** **Date:** 1-27-05
Location: **Elev./Depth:** 28.5'

Taber Since 1954

Client: CH2M Hill
Project: Bellota Weir / #175404.AA.P2.03

Project No: 2T2/305/19 **Figure:** 1

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	0.0	80.7	19.3	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#10	100.0		
#200	19.3		

* (no specification provided)

Material Description

Silty sand

Atterberg Limits

PL= NP LL= 25 PI= NP

Coefficients

D_{85}= 1.09 D_{60}= 0.393 D_{50}= 0.262
D_{30}= 0.116 D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= SM AASHTO= A-2-4(0)

Remarks

Total dry weight tested= 1042 grams.

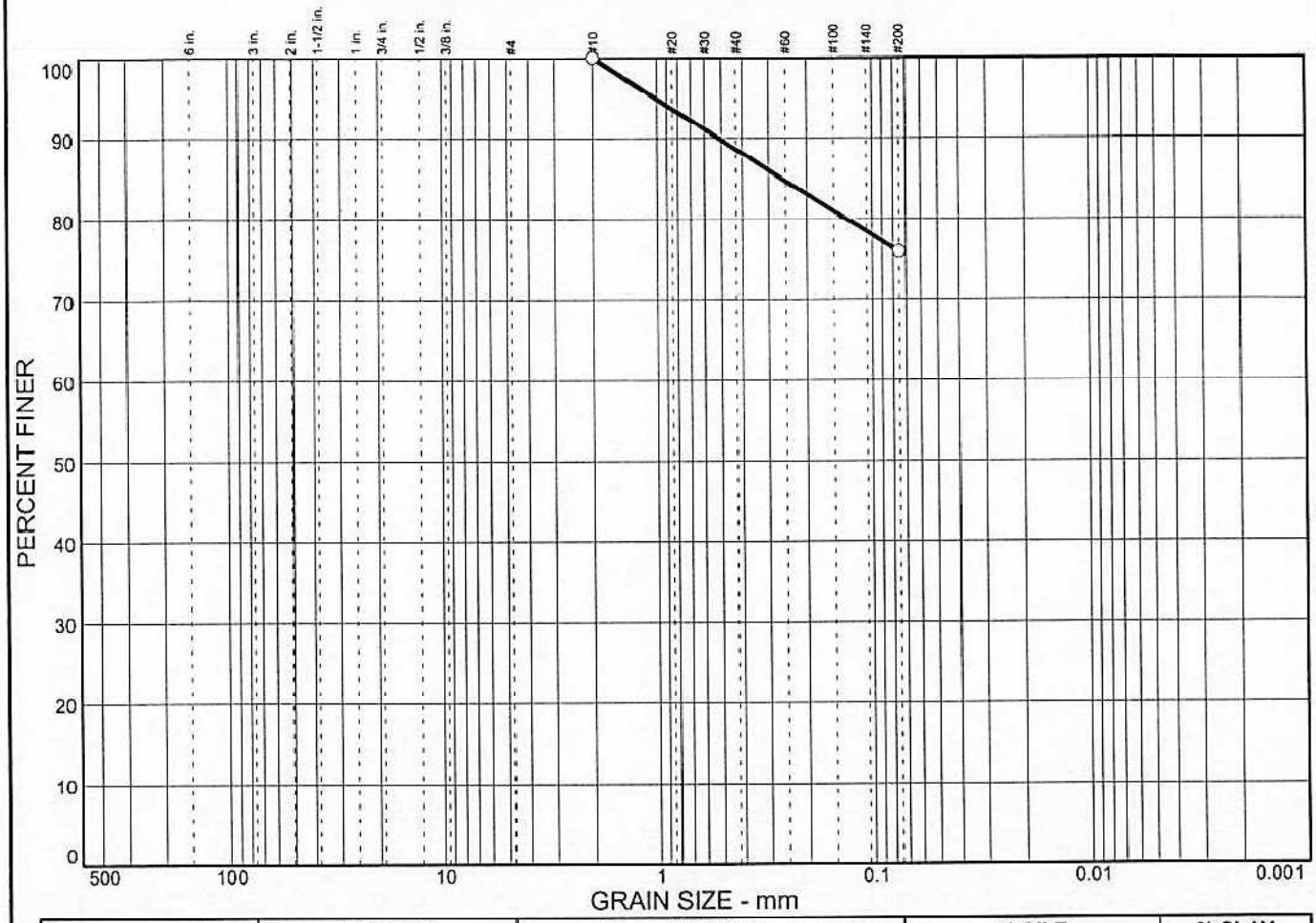
Sample No.: BF-1 (Shelby) **Source of Sample:** **Date:** 1-27-05
Location: **Elev./Depth:** 15'

Taber Since 1954

Client: CH2M Hill
Project: Bellota Weir / #175404.AA.P2.03

Project No: 2T2/305/19 **Figure:** 1

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	0.0	24.1	75.9	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#10	100.0		
#200	75.9		

* (no specification provided)

Material Description

Fat clay with sand

Atterberg Limits

PL= 26 LL= 59 PI= 33

Coefficients

D_{85}= 0.259 D_{60}= D_{50}=
D_{30}= D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= CH AASHTO= A-7-6(26)

Remarks

Total dry weight tested= 292 grams.

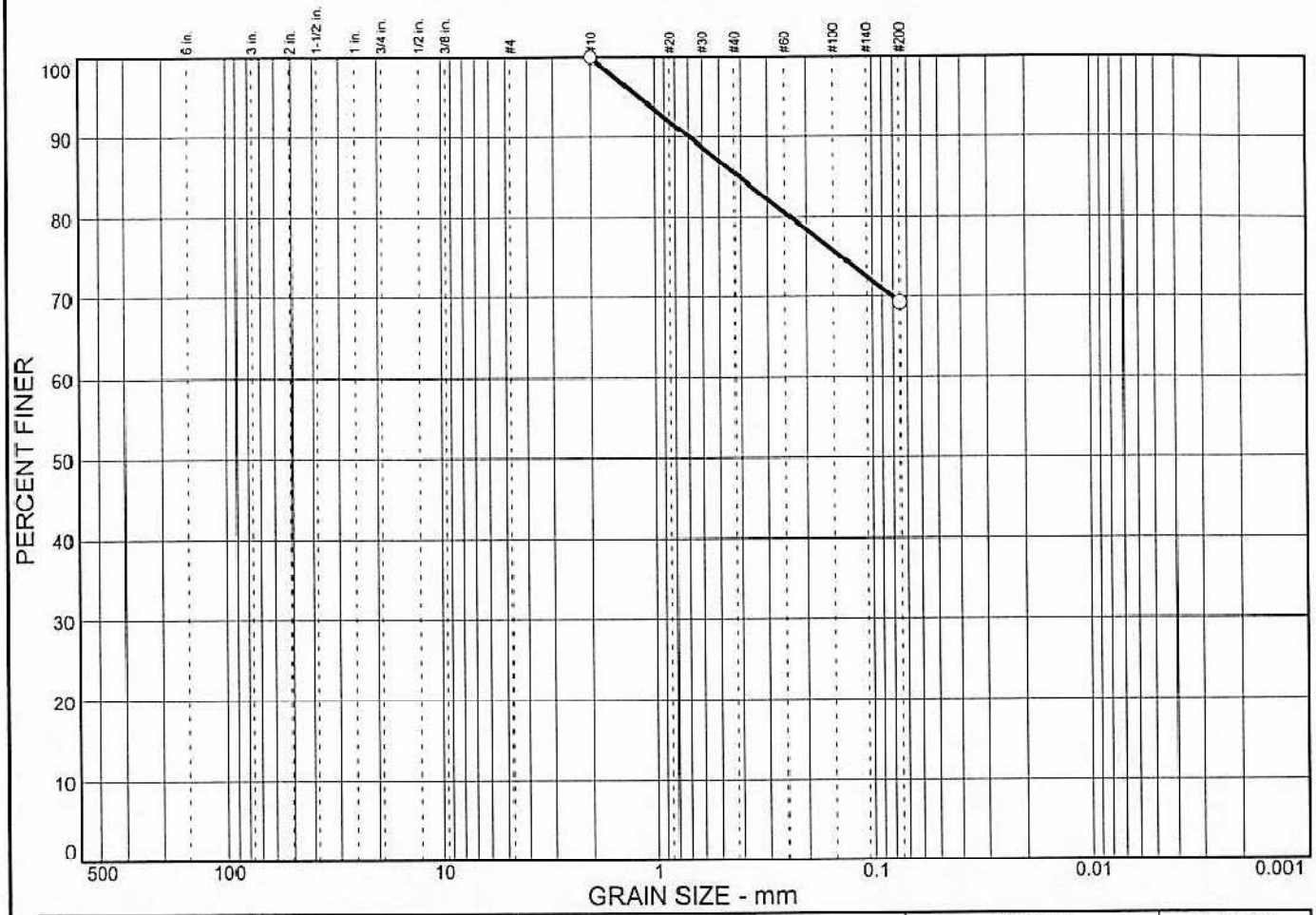
Sample No.: BF-1 **Source of Sample:** **Date:** 1-27-05
Location: **Elev./Depth:** 40'

Taber
Since 1954

Client: CH2M Hill
Project: Bellota Weir / #175404.AA.P2.03

Project No: 2T2/305/19 **Figure:** 1

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	0.0	30.7	69.3	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#10	100.0		
#200	69.3		

* (no specification provided)

Material Description

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= 0.402 D_{60}= D_{50}=
D_{30}= D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= AASHTO=

Remarks

Total dry weight tested= 444 grams.

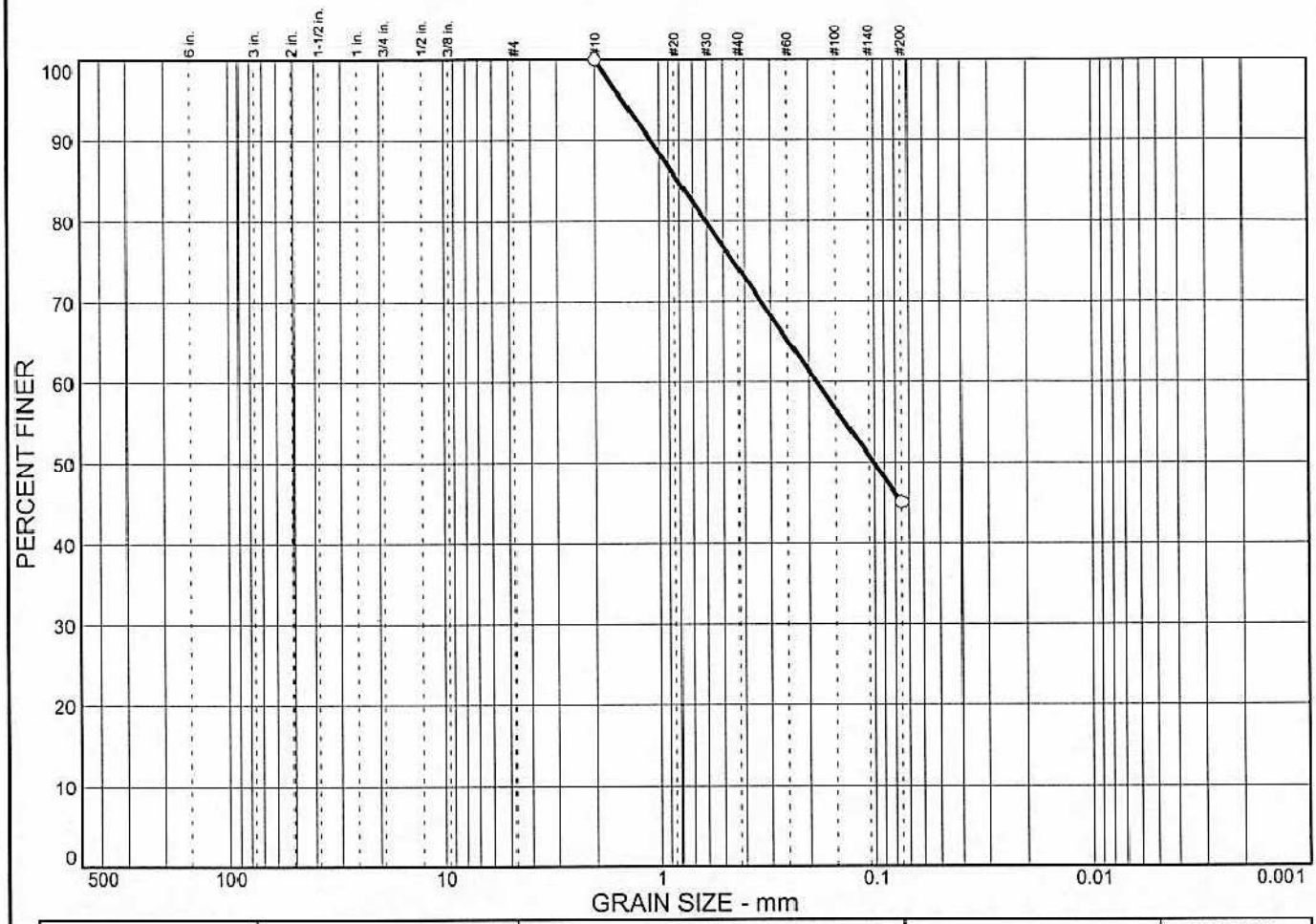
Sample No.: BF-1 **Source of Sample:** **Date:** 1-27-05
Location: **Elev./Depth:** 55'

Taber Since 1954

Client: CH2M Hill
Project: Bellota Weir / #175404.AA.P2.03

Project No: 2T2/305/19 **Figure:** 1

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	0.0	55.0	45.0	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#10	100.0		
#200	45.0		

* (no specification provided)

Material Description

Silty, clayey sand

Atterberg Limits

PL= 21 LL= 25 PI= 4

Coefficients

D_{85}= 0.817 D_{60}= 0.184 D_{50}= 0.101
D_{30}= D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= SC-SM AASHTO= A-4(0)

Remarks

Total dry weight tested= 807 grams.

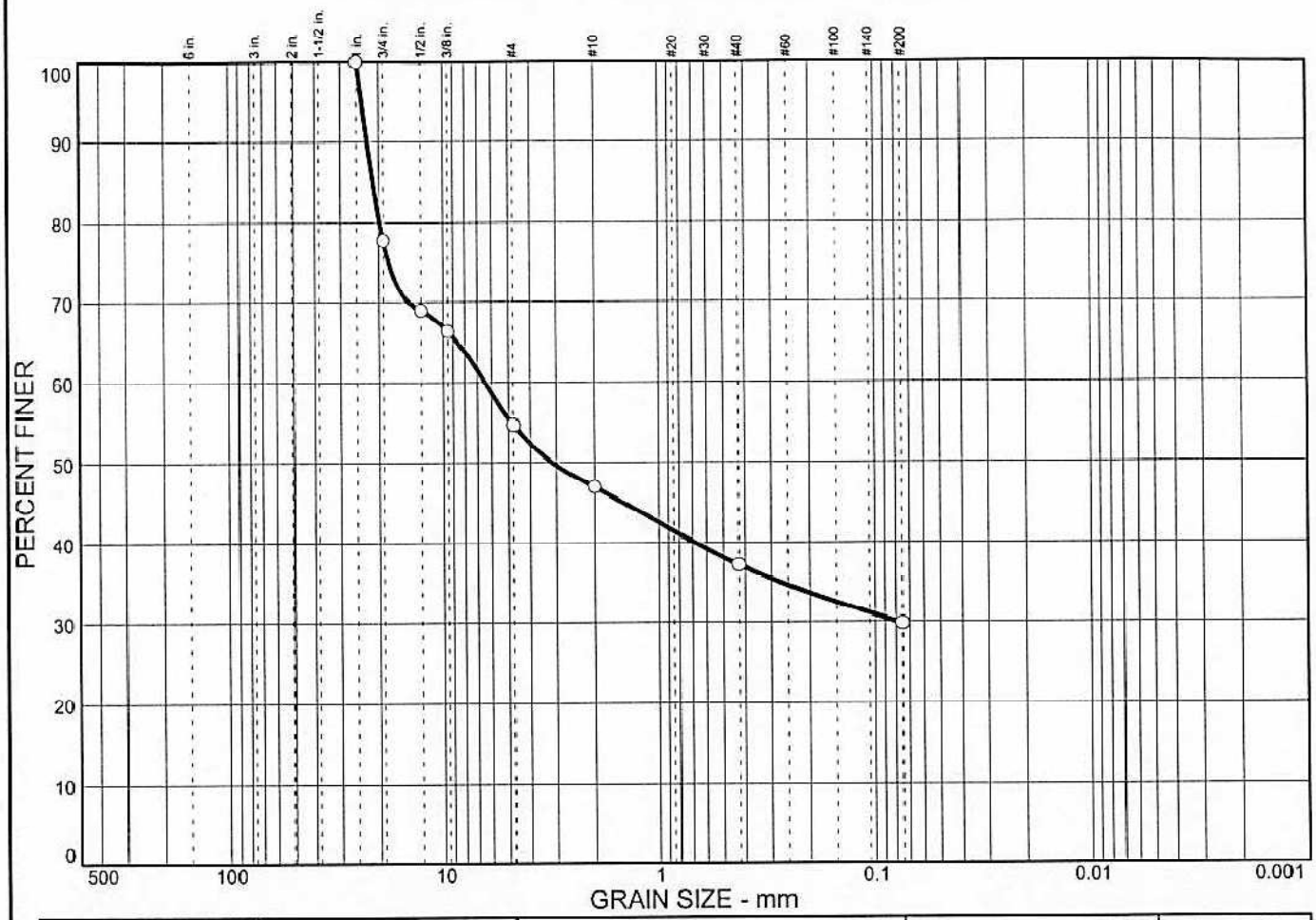
Sample No.: BD-2 **Source of Sample:** **Date:** 1-27-05
Location: **Elev./Depth:** 5'

Taber
Since 1954

Client: CH2M Hill
Project: Bellota Weir / #175404.AA.P2.03

Project No: 2T2/305/19 **Figure:** 1

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	45.3	24.9	29.8	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1 in.	100.0		
0.75 in.	77.7		
0.5 in.	69.0		
0.375 in.	66.4		
#4	54.7		
#10	46.9		
#40	37.2		
#200	29.8		

* (no specification provided)

Material Description

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= 21.3 D_{60}= 6.44 D_{50}= 3.15
D_{30}= 0.0792 D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= AASHTO=

Remarks

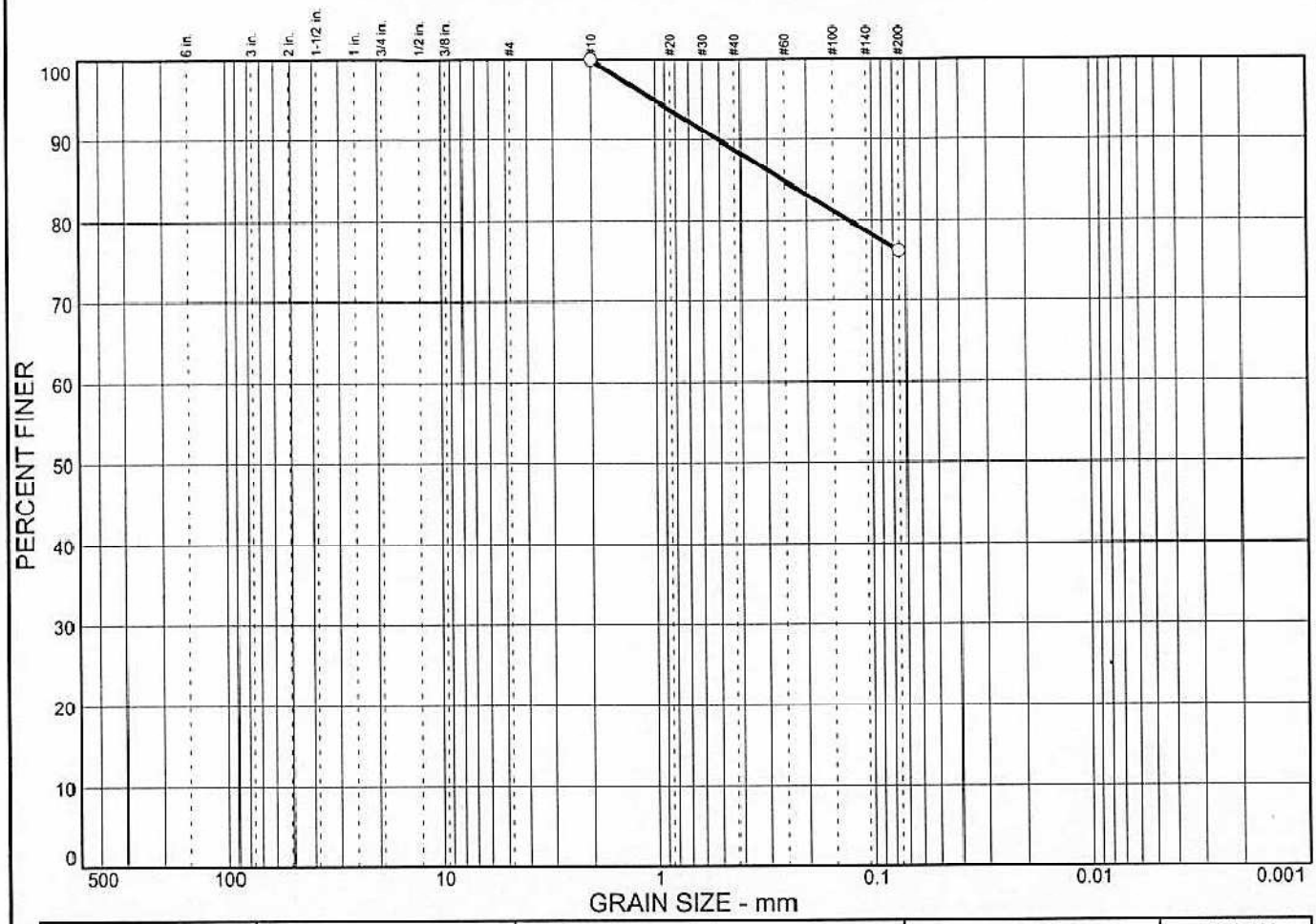
Total dry weight tested= 412 grams.

Sample No.: BD-2 **Source of Sample:** **Date:** 1-27-05
Location: **Elev./Depth:** 15'

Taber
Since 1954

Client: CH2M Hill
Project: Bellota Weir / #175404.AA.P2.03
Project No: 2T2/305/19 **Figure:** 1

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	0.0	23.8	76.2	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#10	100.0		
#200	76.2		

* (no specification provided)

Material Description

Elastic silt with sand

Atterberg Limits

PL= 41 LL= 57 PI= 16

Coefficients

D_{85}= 0.253 D_{60}= D_{50}=
D_{30}= D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= MH AASHTO= A-7-5(15)

Remarks

Total dry weight tested= 318 grams.

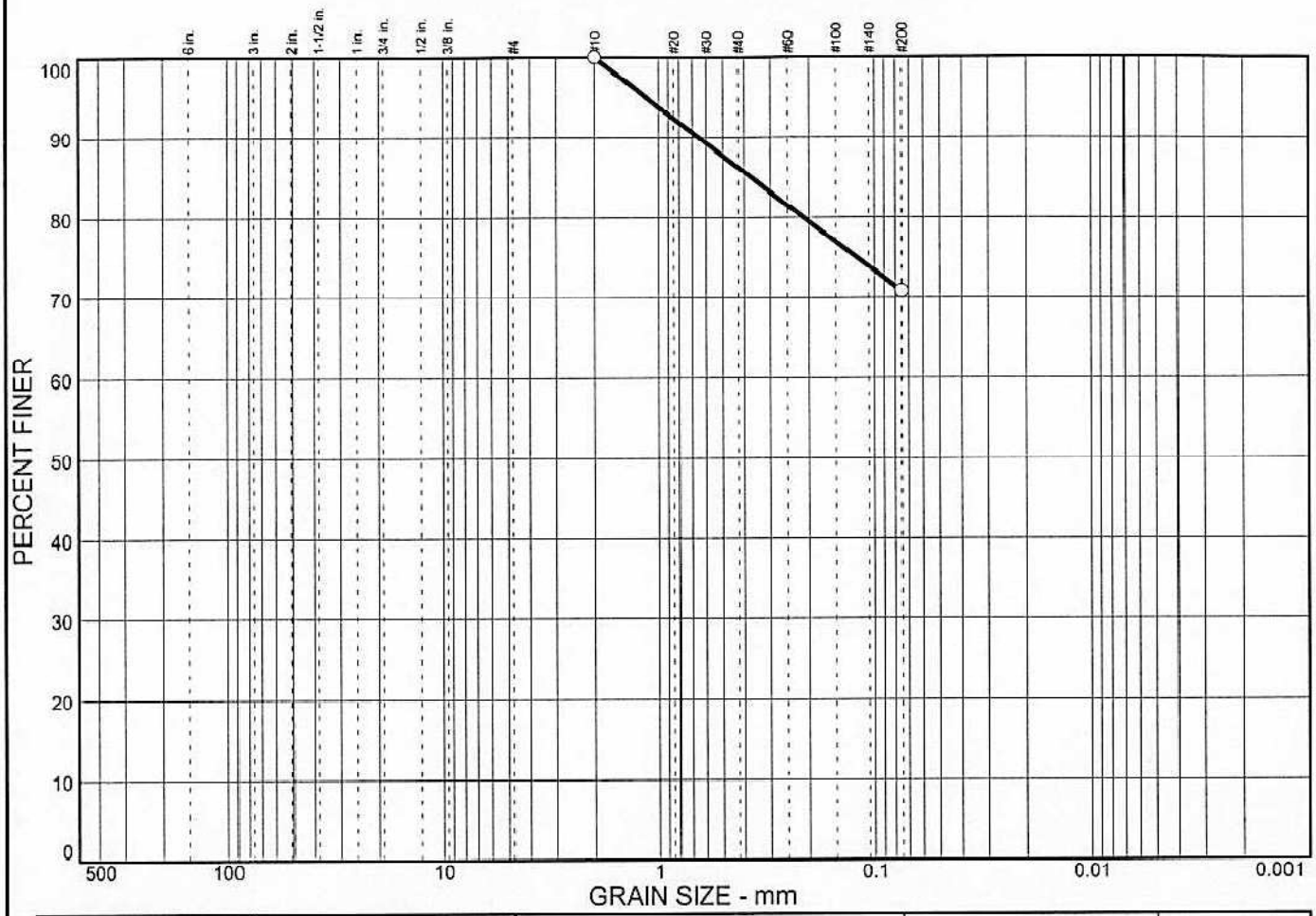
Sample No.: BD-2 **Source of Sample:** **Date:** 1-27-05
Location: **Elev./Depth:** 25'

Taber
Since 1954

Client: CH2M Hill
Project: Bellota Weir / #175404.AA.P2.03

Project No: 2T2/305/19 **Figure:** 1

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	0.0	29.2	70.8	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#10	100.0		
#200	70.8		

* (no specification provided)

Material Description

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= 0.370 D_{60}= D_{50}=
D_{30}= D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= AASHTO=

Remarks

Total dry weight tested= 141 grams.

Sample No.: BD-2 **Source of Sample:** **Date:** 1-27-05
Location: **Elev./Depth:** 50'

Taber Since 1954

Client: CH2M Hill
Project: Bellota Weir / #175404.AA.P2.03

Project No: 2T2/305/19 **Figure:** 1

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	1.1	63.9	35.0	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
0.375 in.	100.0		
#4	98.9		
#10	95.4		
#40	85.0		
#200	35.0		

* (no specification provided)

Material Description

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= 0.425 D_{60}= 0.157 D_{50}= 0.116
D_{30}= D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= AASHTO=

Remarks

Total dry weight tested= 241 grams.

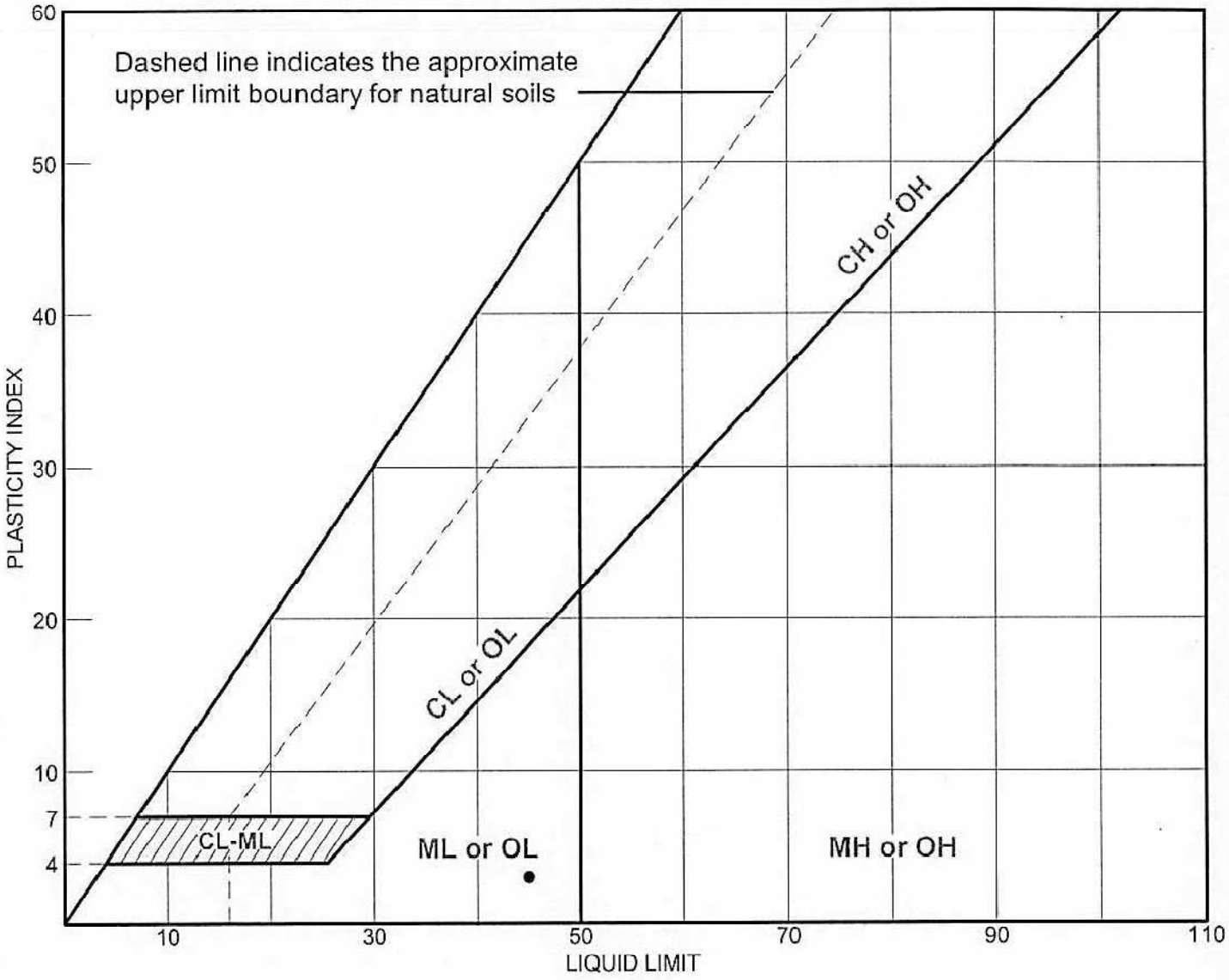
Sample No.: BD-2 **Source of Sample:** **Date:** 1-27-05
Location: **Elev./Depth:** 60'

Taber Since 1954

Client: CH2M Hill
Project: Bellota Weir / #175404.AA.P2.03

Project No: 2T2/305/19 **Figure:** 1

LIQUID AND PLASTIC LIMITS TEST REPORT



SOIL DATA								
SYMBOL	SOURCE	SAMPLE NO.	DEPTH (ft.)	NATURAL WATER CONTENT (%)	PLASTIC LIMIT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	USCS
●		BI-1	35'		42	45	3	ML

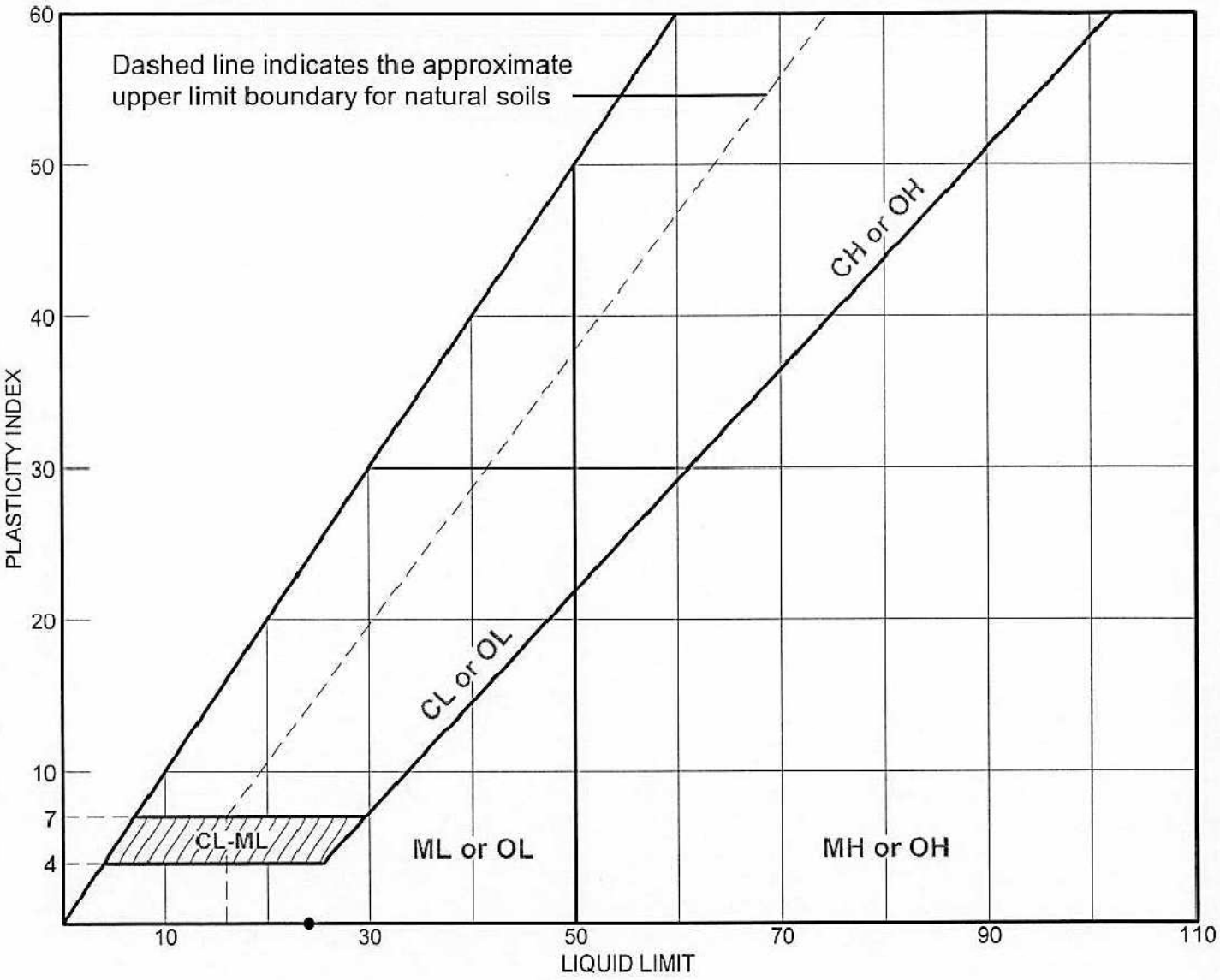
Taber Since 1954

Client: CH2M Hill

Project: Bellota Weir / #175404.AA.P2.03

Project No.: 2T2/305/19 **Figure:** 2

LIQUID AND PLASTIC LIMITS TEST REPORT

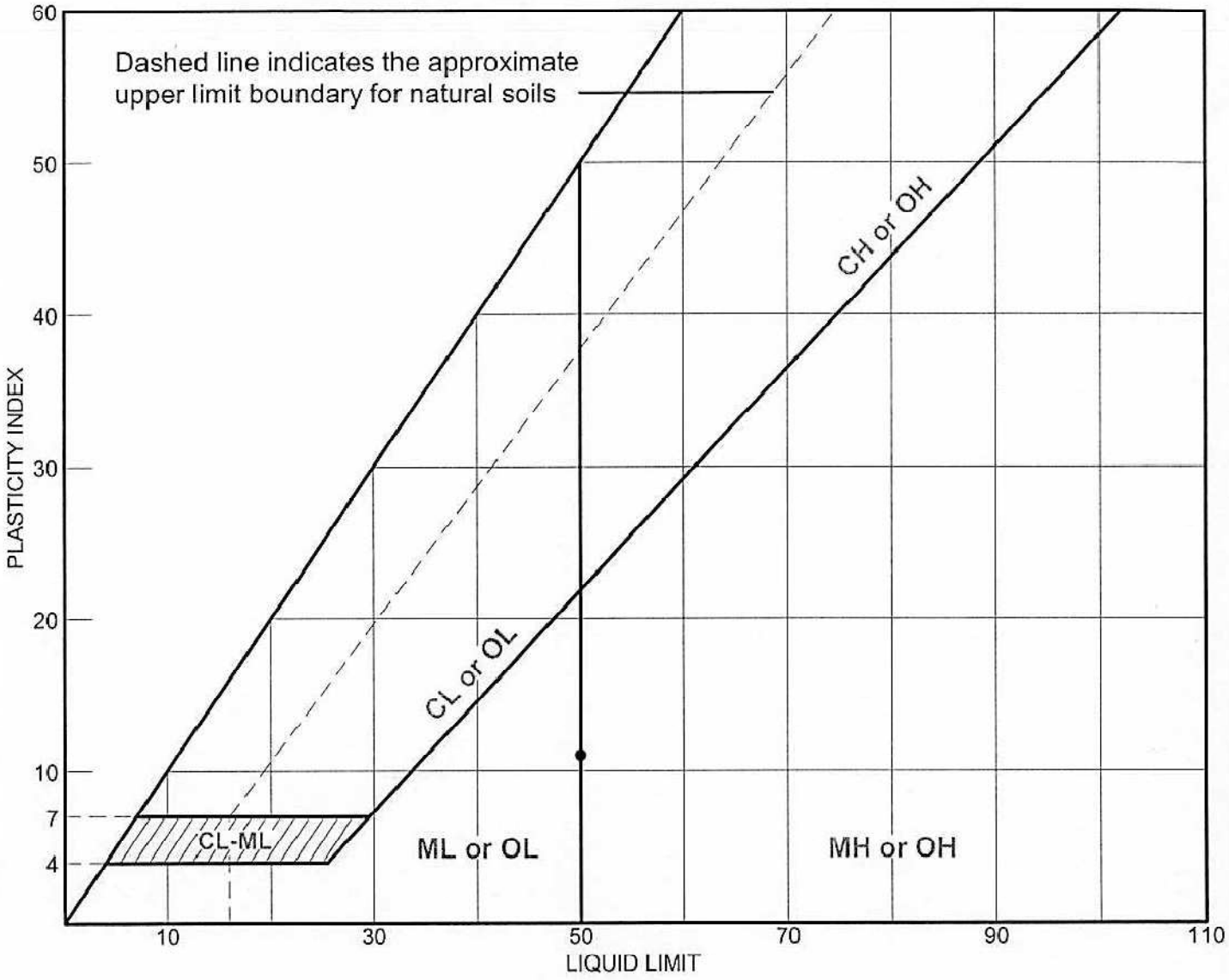


SOIL DATA								
SYMBOL	SOURCE	SAMPLE NO.	DEPTH (ft.)	NATURAL WATER CONTENT (%)	PLASTIC LIMIT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	USCS
●		BI-2	10'		NP	24	NP	SM

Taber
Since 1954

Client: CH2M Hill
Project: Bellota Weir / #175404.AA.P2.03
Project No.: 2T2/305/19 **Figure:** 2

LIQUID AND PLASTIC LIMITS TEST REPORT



SOIL DATA								
SYMBOL	SOURCE	SAMPLE NO.	DEPTH (ft.)	NATURAL WATER CONTENT (%)	PLASTIC LIMIT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	USCS
●		BI-2	25'		39	50	11	MH

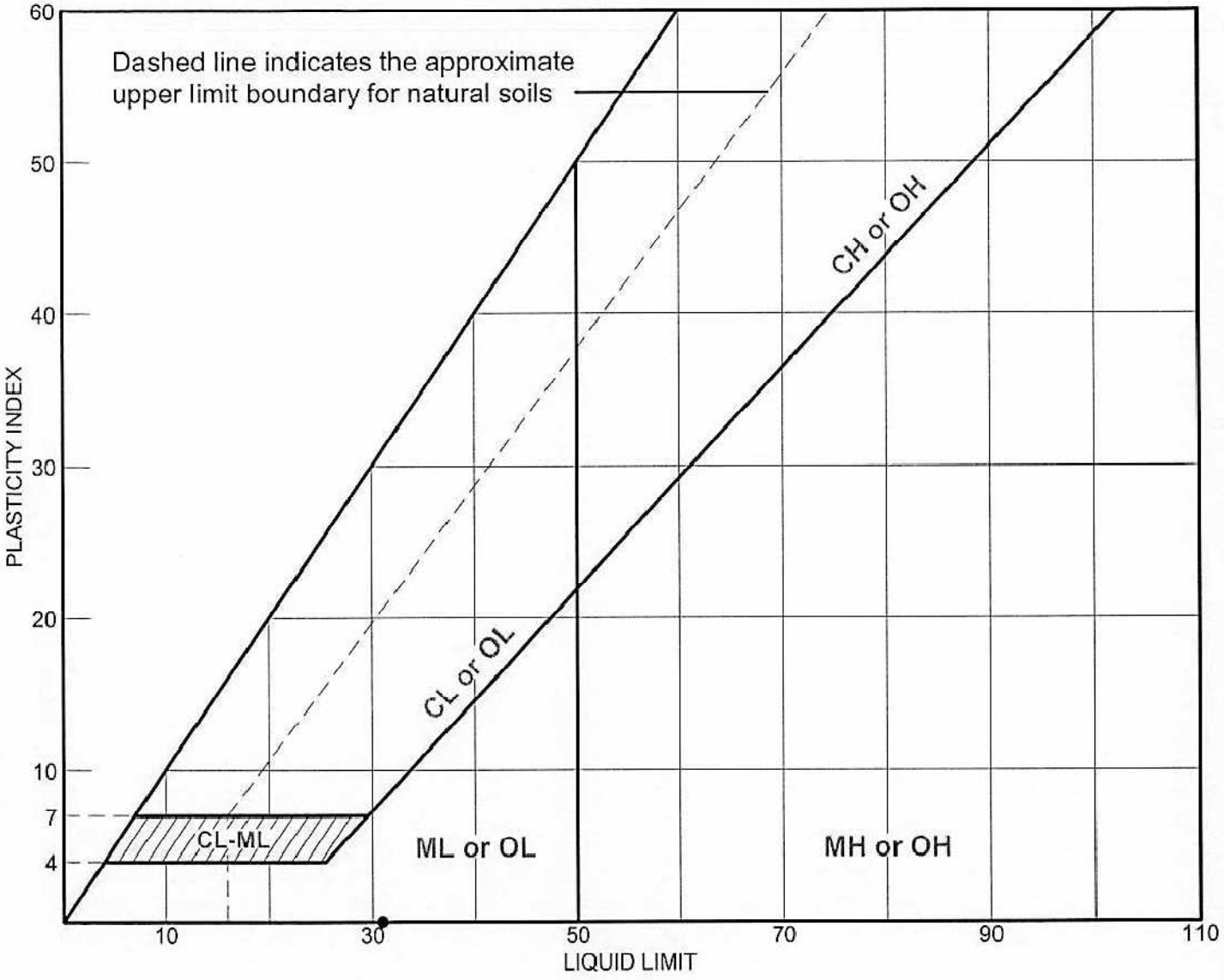
Taber Since 1954

Client: CH2M Hill

Project: Bellota Weir / #175404.AA.P2.03

Project No.: 2T2/305/19 **Figure:** 2

LIQUID AND PLASTIC LIMITS TEST REPORT



SOIL DATA								
SYMBOL	SOURCE	SAMPLE NO.	DEPTH (ft.)	NATURAL WATER CONTENT (%)	PLASTIC LIMIT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	USCS
●		BI-3	7.5'		NP	31	NP	SP

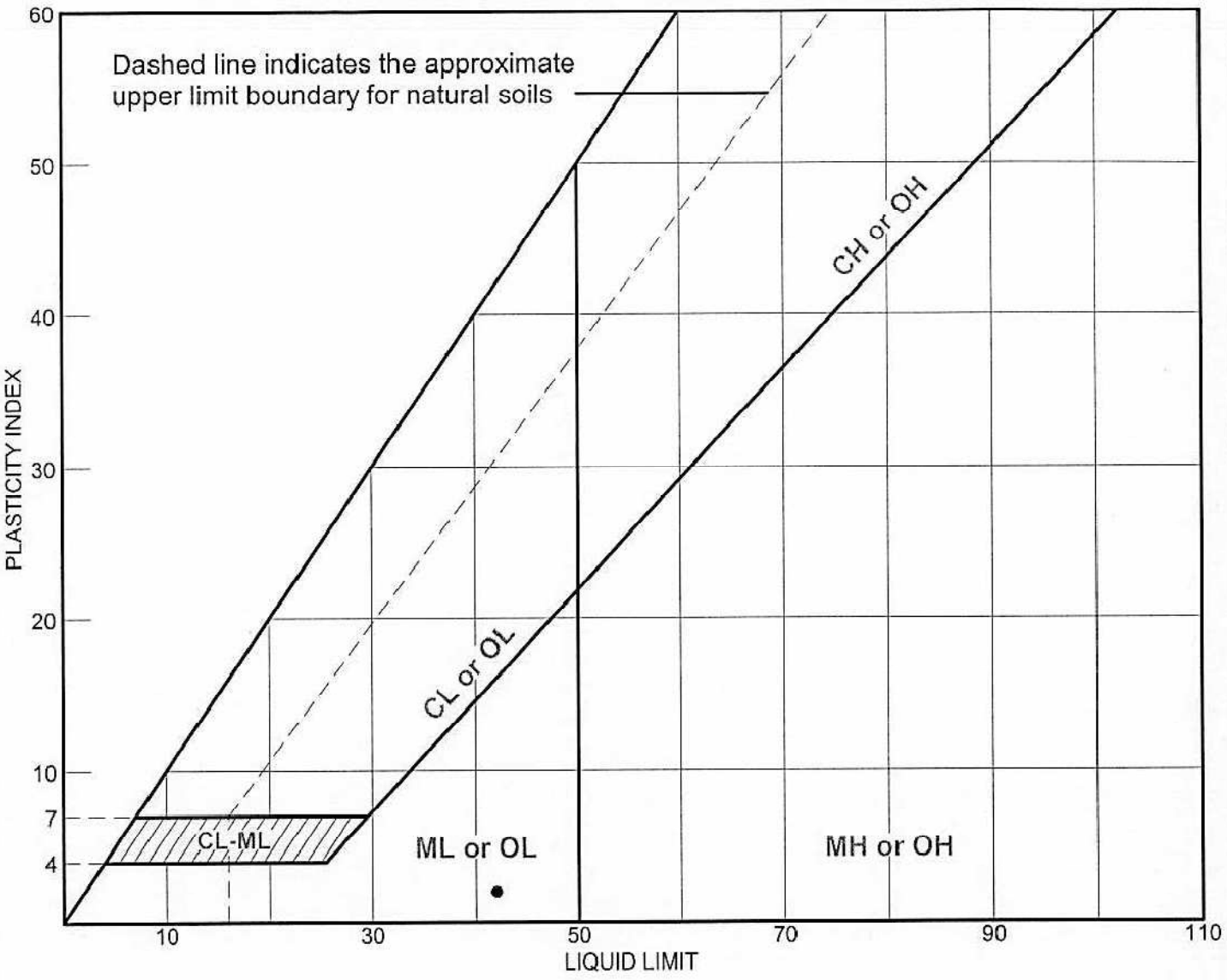
Taber Since 1954

Client: CH2M Hill

Project: Bellota Weir / #175404.AA.P2.03

Project No.: 2T2/305/19 **Figure:** 2

LIQUID AND PLASTIC LIMITS TEST REPORT



SOIL DATA								
SYMBOL	SOURCE	SAMPLE NO.	DEPTH (ft.)	NATURAL WATER CONTENT (%)	PLASTIC LIMIT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	USCS
●		BI-3	28.5'		40	42	2	ML

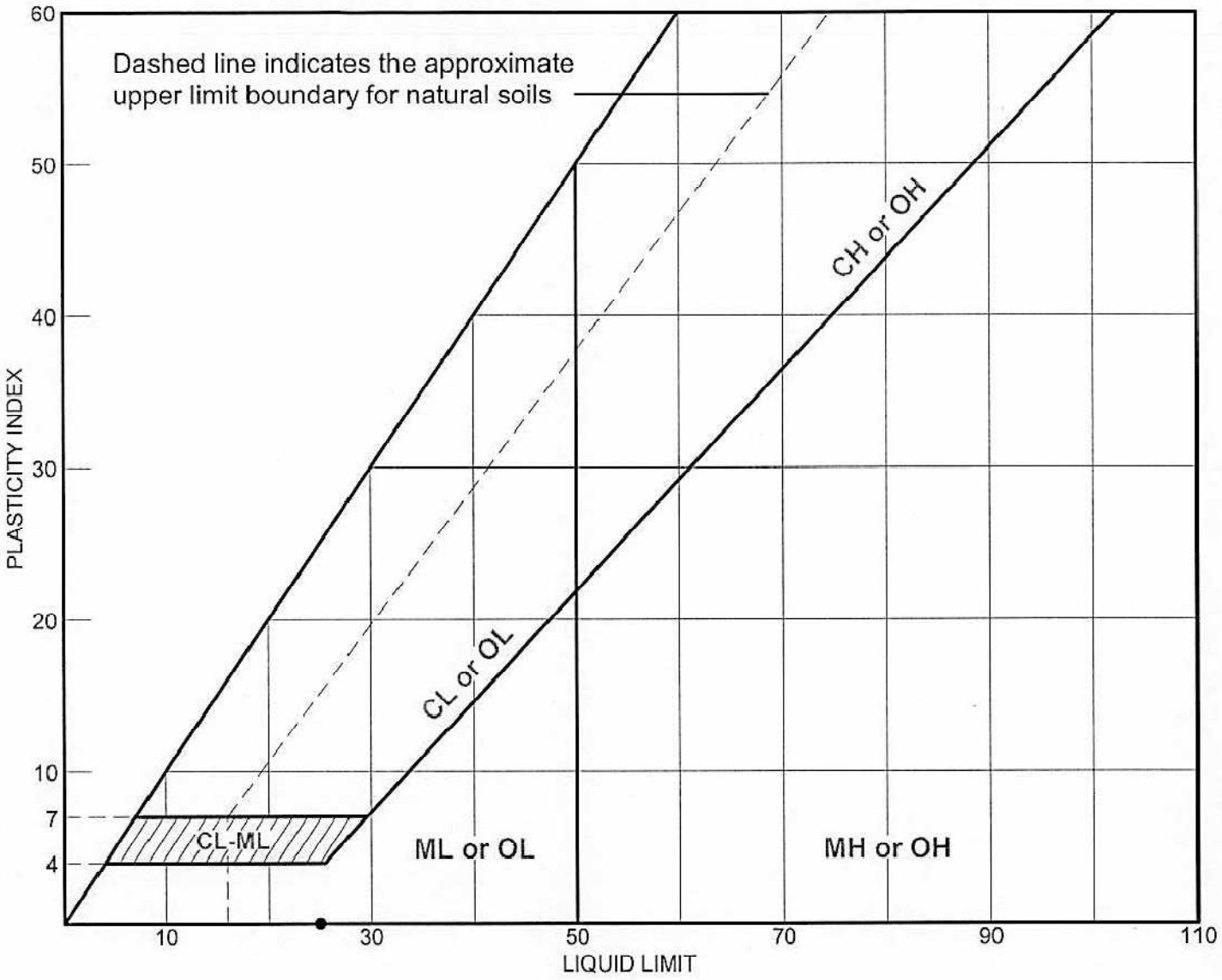
Taber Since 1954

Client: CH2M Hill

Project: Bellota Weir / #175404.AA.P2.03

Project No.: 2T2/305/19 **Figure:** 2

LIQUID AND PLASTIC LIMITS TEST REPORT

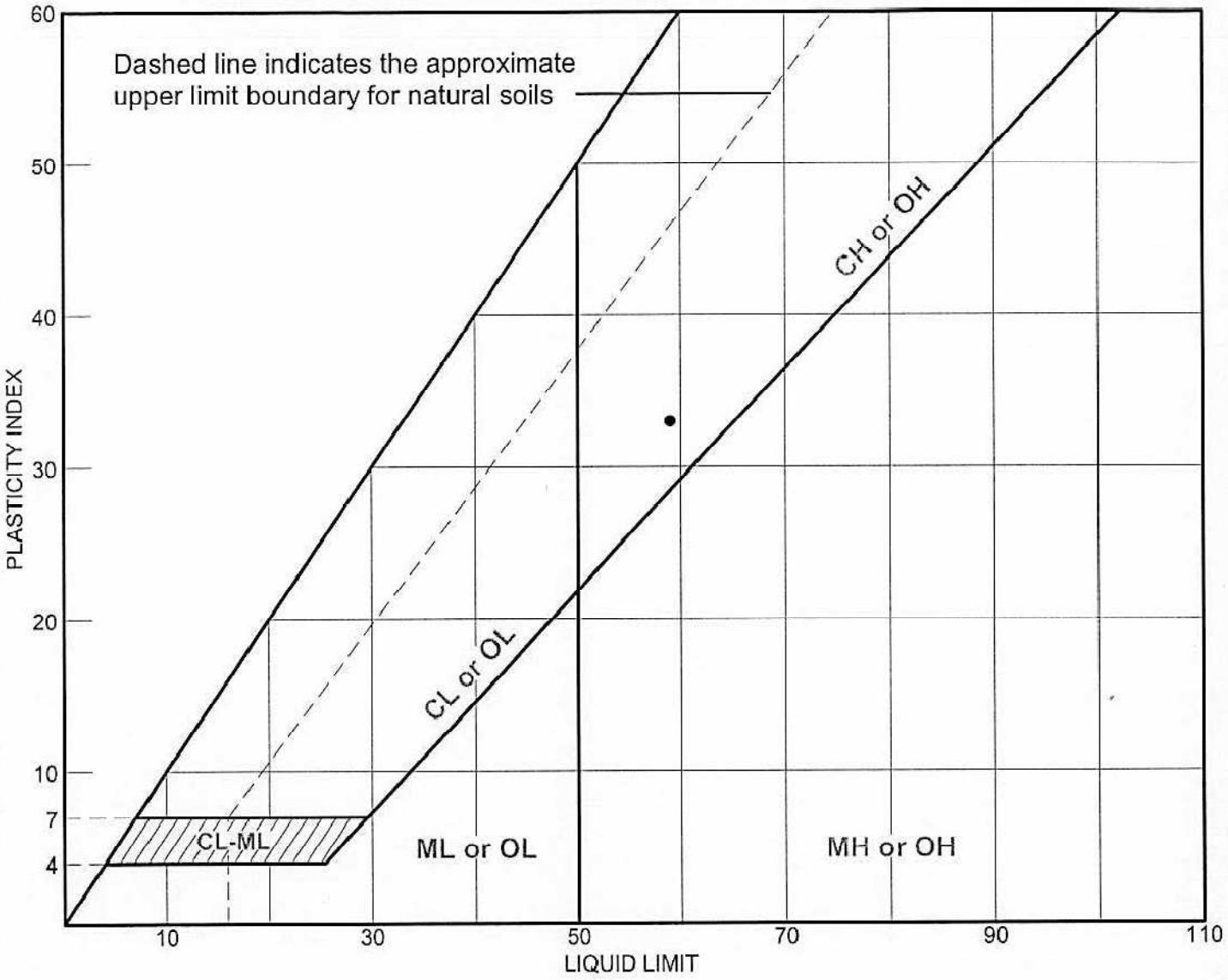


SOIL DATA								
SYMBOL	SOURCE	SAMPLE NO.	DEPTH (ft.)	NATURAL WATER CONTENT (%)	PLASTIC LIMIT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	USCS
●		BF-1	15'		NP	25	NP	SM

Taber
Since 1954

Client: CH2M Hill
Project: Bellota Weir / #175404.AA.P2.03
Project No.: 2T2/305/19 **Figure:** 2

LIQUID AND PLASTIC LIMITS TEST REPORT

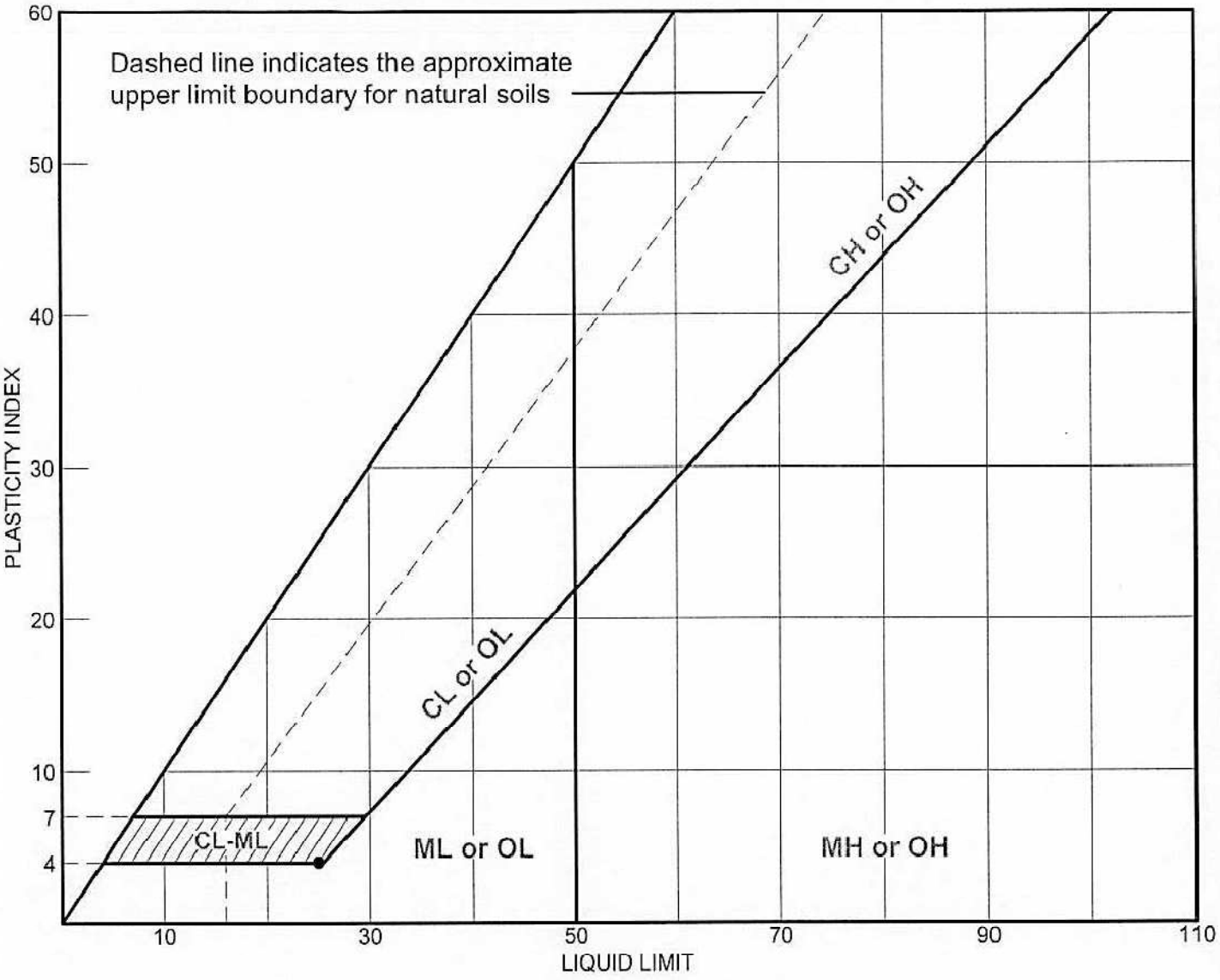


SOIL DATA								
SYMBOL	SOURCE	SAMPLE NO.	DEPTH (ft.)	NATURAL WATER CONTENT (%)	PLASTIC LIMIT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	USCS
●		BF-1	40'		26	59	33	CH

Taber
Since 1954

Client: CH2M Hill
Project: Bellota Weir / #175404.AA.P2.03
Project No.: 2T2/305/19 **Figure:** 2

LIQUID AND PLASTIC LIMITS TEST REPORT

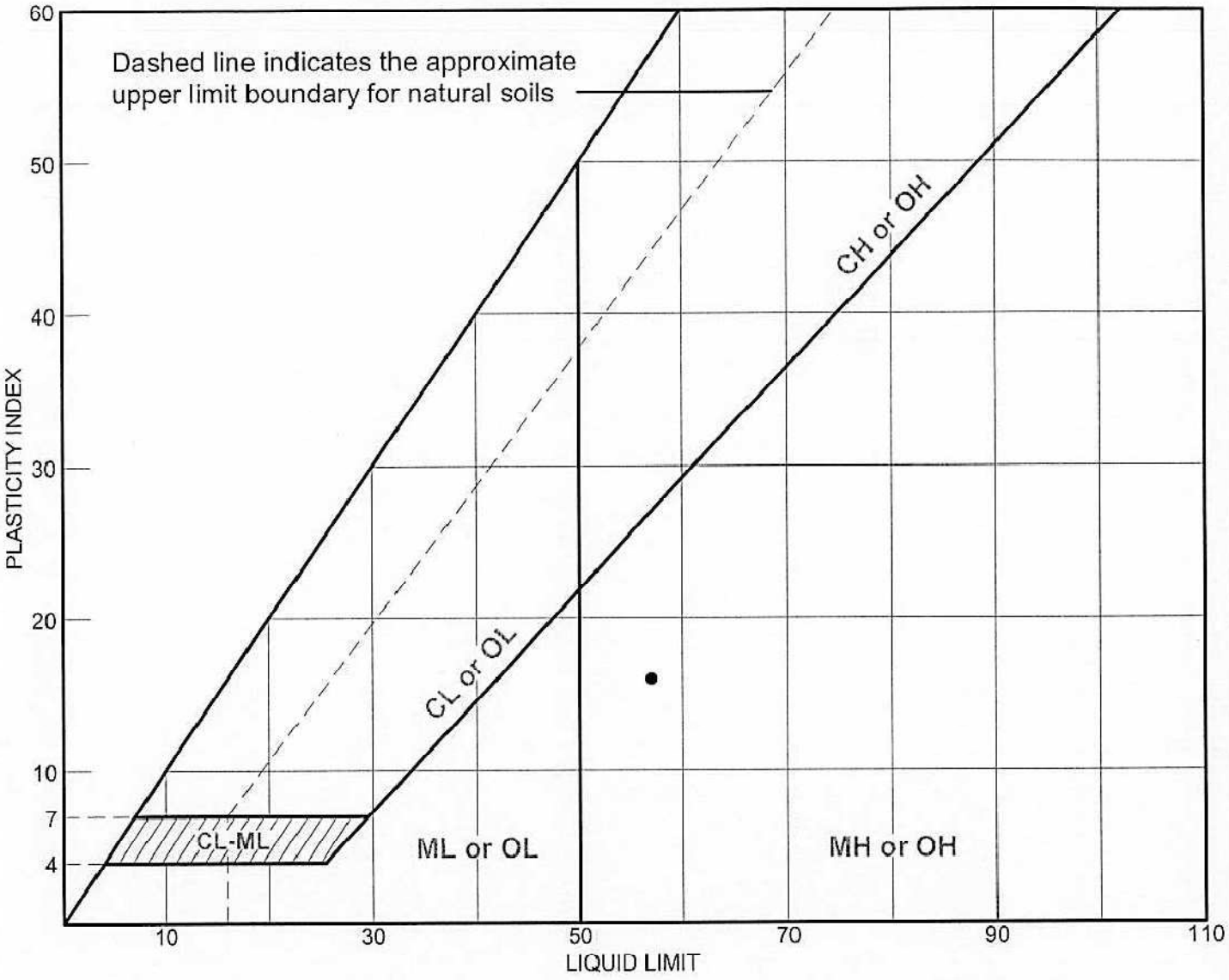


SOIL DATA								
SYMBOL	SOURCE	SAMPLE NO.	DEPTH (ft.)	NATURAL WATER CONTENT (%)	PLASTIC LIMIT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	USCS
●		BD-2	5'		21	25	4	SC-SM

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Client: CH2M Hill
Project: Bellota Weir / #175404.AA.P2.03
Project No.: 2T2/305/19 **Figure:** 2

LIQUID AND PLASTIC LIMITS TEST REPORT



SOIL DATA								
SYMBOL	SOURCE	SAMPLE NO.	DEPTH (ft.)	NATURAL WATER CONTENT (%)	PLASTIC LIMIT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	USCS
●		BD-2	25'		41	57	16	MH

Taber
Since 1954

Client: CH2M Hill
Project: Bellota Weir / #175404.AA.P2.03
Project No.: 2T2/305/19 **Figure:** 2

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3911 West Capitol Avenue West
Sacramento, CA 95691-2116

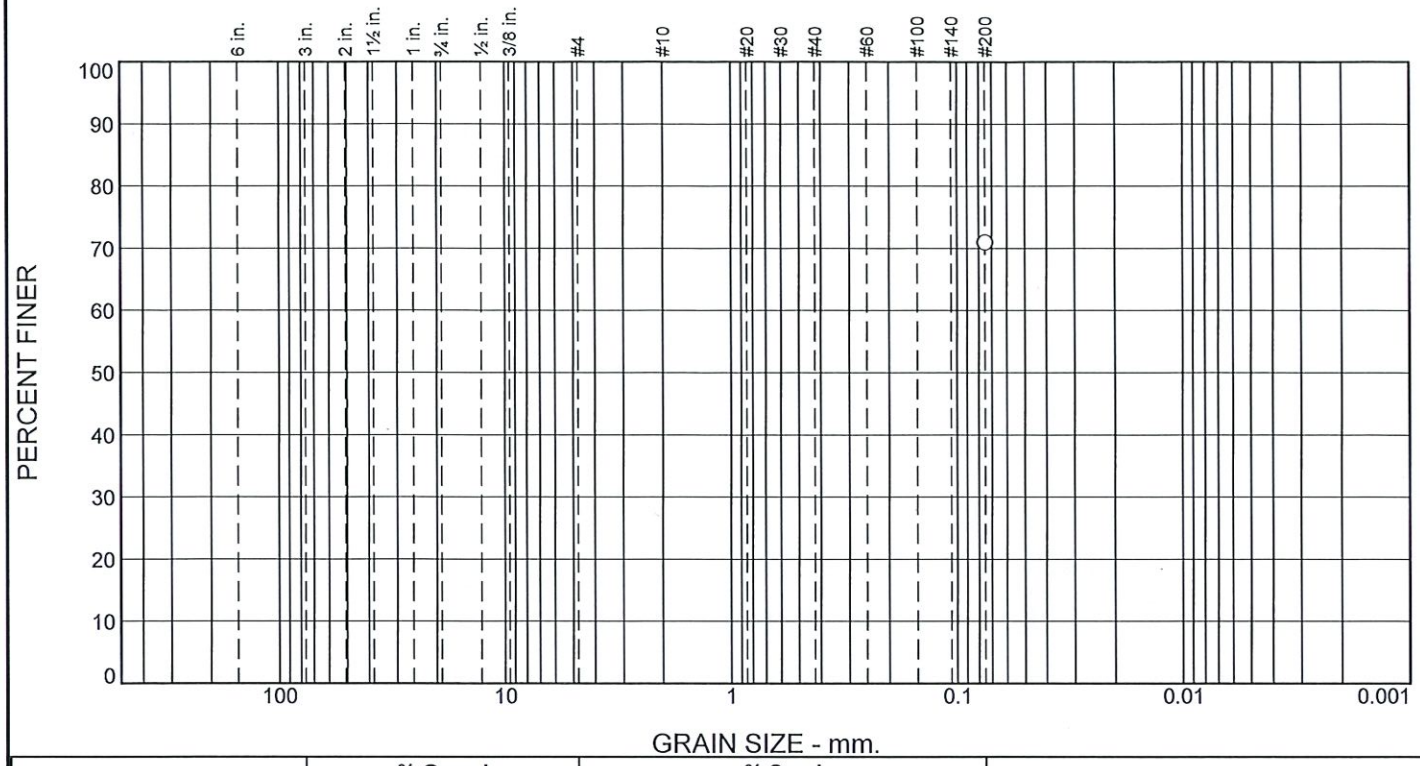
DENSITY TESTS

Client CH2M Hill / #175404.AA.P2.03 Date 1/25/2005 Job # 2T2/305/19

Sample No.	BF-1									
Sample Length (in)	8.00									
Sample Diameter (in)	2.870									
Sample weight	1608.6									
Tare										
Wet Weight	1608.6									
Corrected weight	1608.6									
Container #	C									
Wet Weight+tare	1822.6									
Dry Weight+tare	1435.0									
Water	387.6									
Tare	214.0									
Dry Weight	1221.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
% Water	**31.7%**									
Dry Density (pcf)	**90**									
Dry Density (kN/m^3)	**14.12**									
Unit Weight (pcf)	**118**									
Unit Weight (kN/m^3)	**19**									

Figure-3

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines
	Coarse	Fine	Coarse	Medium	Fine	
						71

Test Results (ASTM D6913 & ASTM D1140)

Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
#200	71		

* (no specification provided)

Material Description

Atterberg Limits (ASTM D 4318)

PL= LL= PI=

Classification

USCS (D 2487)= AASHTO (M 145)=

Coefficients

D_{90}= D_{85}= D_{60}=
D_{50}= D_{30}= D_{15}=
D_{10}= C_u= C_c=

Remarks

Date Received: 2/9/21 **Date Tested:** 2/22/21

Tested By: JB

Checked By: JML

Title: PM

Location: HDR-B1: S2 **Date Sampled:** -

Sample Number: 62174 **Depth:** 5.0'-7.5'

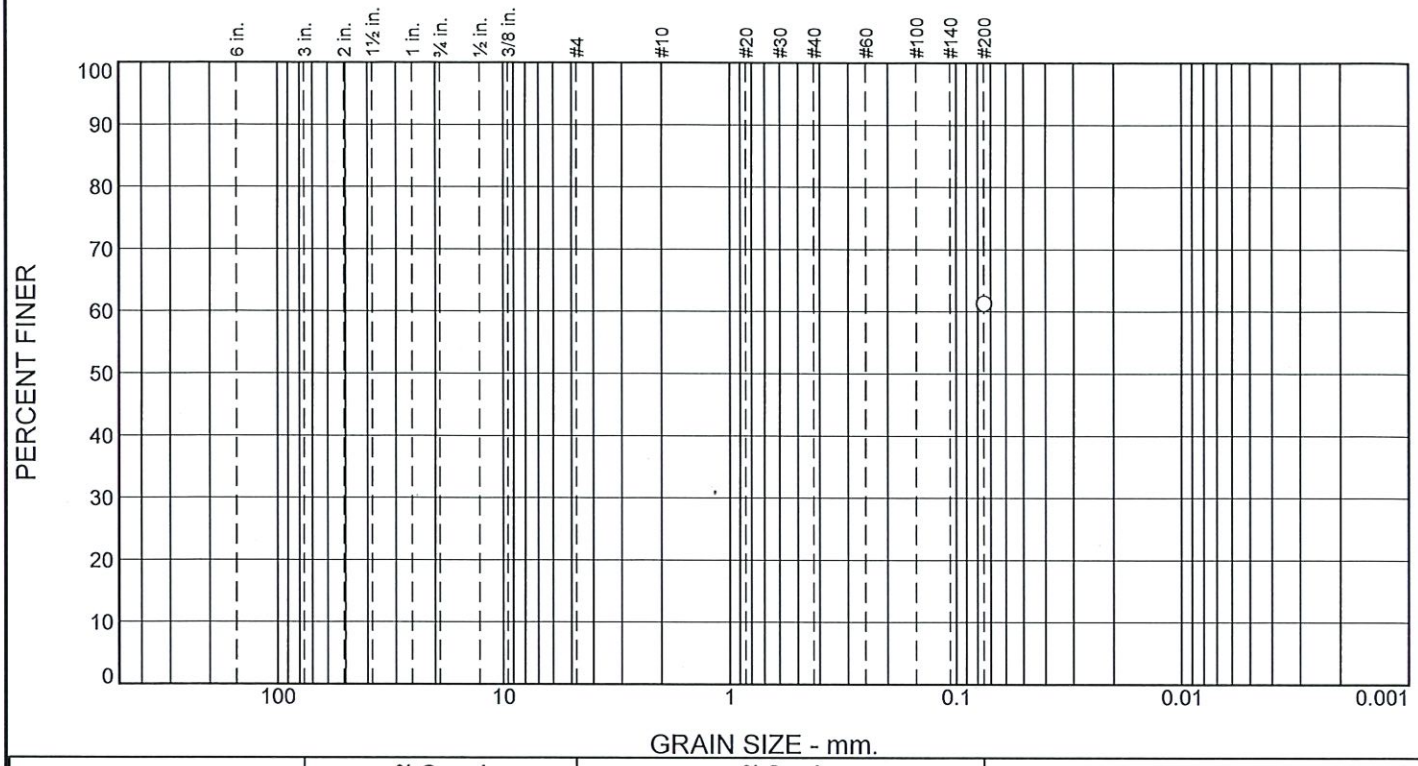
GULF SHORE EXPLORATION AND TESTING

Client: HDR, Inc.

Project: Bellotta Weir Modification

Project No: 21-105 **Figure**

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines
	Coarse	Fine	Coarse	Medium	Fine	
						61

Test Results (ASTM D6913 & ASTM D1140)

Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
#200	61		

* (no specification provided)

Material Description

Atterberg Limits (ASTM D 4318)

PL= LL= PI=

Classification

USCS (D 2487)= AASHTO (M 145)=

Coefficients

D_{90}= D_{85}= D_{60}=
D_{50}= D_{30}= D_{15}=
D_{10}= C_u= C_c=

Remarks

Date Received: 2/9/21 **Date Tested:** 2/22/21

Tested By: JB

Checked By: JML

Title: PM

Location: HDR-B1: S4
Sample Number: 62176 **Depth:** 10.0'-11.5'

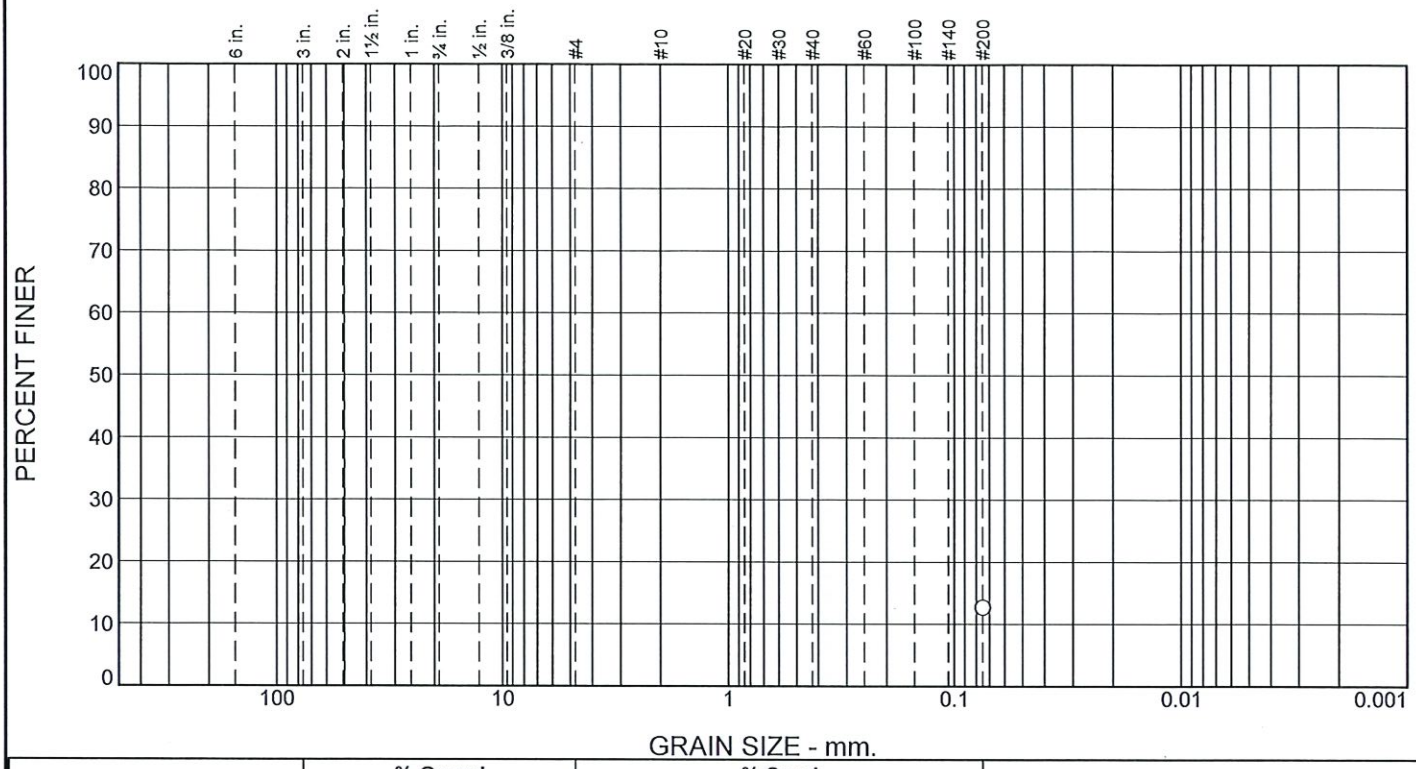
Date Sampled: -

GULF SHORE EXPLORATION AND TESTING

Client: HDR, Inc.
Project: Bellotta Weir Modification

Project No: 21-105 **Figure**

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines
	Coarse	Fine	Coarse	Medium	Fine	
						13

Test Results (ASTM D6913 & ASTM D1140)

Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
#200	13		

* (no specification provided)

Material Description

Atterberg Limits (ASTM D 4318)

PL= NP LL= NV PI= NP

Classification

USCS (D 2487)= AASHTO (M 145)=

Coefficients

D_{90}= D_{85}= D_{60}=
D_{50}= D_{30}= D_{15}=
D_{10}= C_u= C_c=

Remarks

Date Received: 2/9/21 **Date Tested:** 2/22/21

Tested By: JB

Checked By: JML

Title: PM

Location: HDR-B1: S7 **Date Sampled:** -

Sample Number: 62178 **Depth:** 17.5'-19.0'

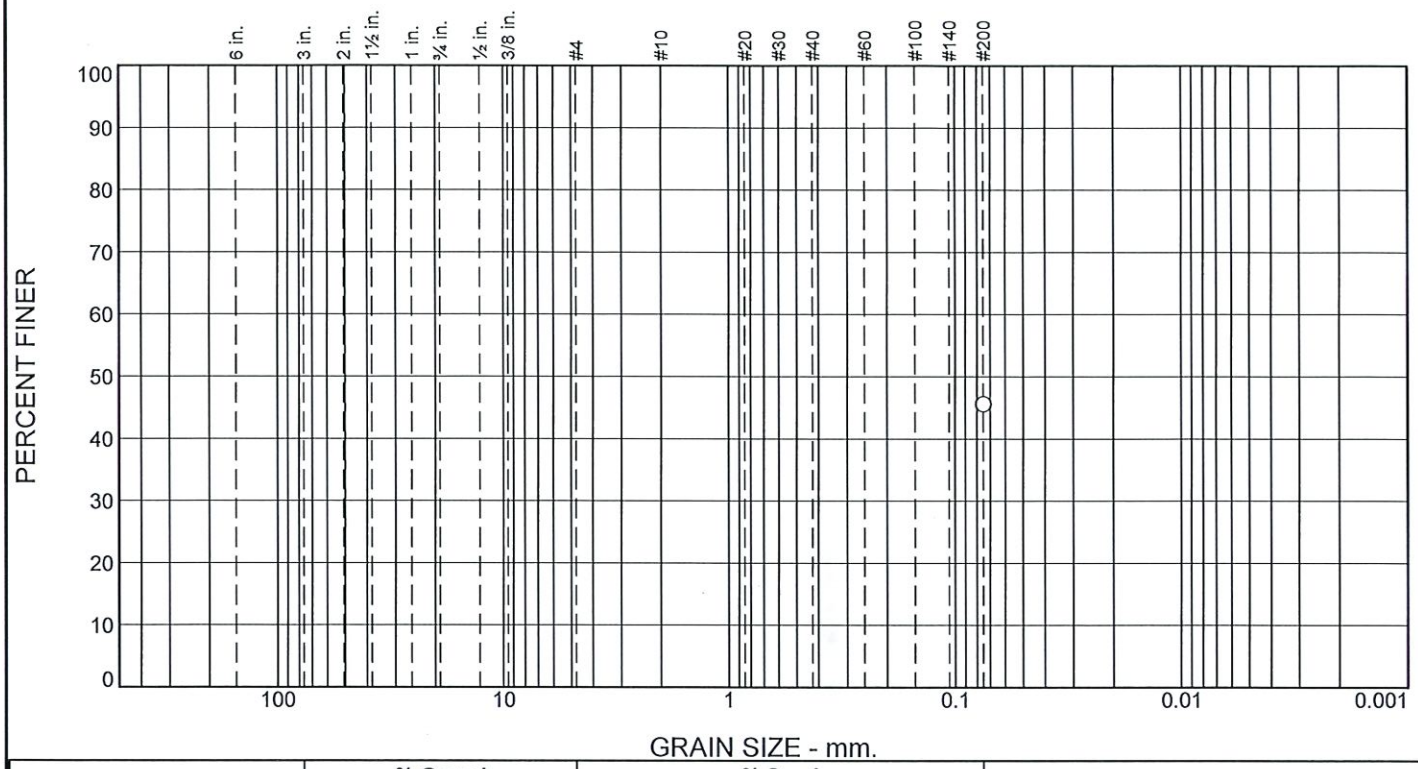
GULF SHORE EXPLORATION AND TESTING

Client: HDR, Inc.

Project: Bellotta Weir Modification

Project No: 21-105 **Figure**

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines
	Coarse	Fine	Coarse	Medium	Fine	
						46

Test Results (ASTM D6913 & ASTM D1140)

Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
#200	46		

* (no specification provided)

Material Description

Atterberg Limits (ASTM D 4318)

PL= LL= PI=

Classification

USCS (D 2487)= AASHTO (M 145)=

Coefficients

D_{90}= D_{85}= D_{60}=
D_{50}= D_{30}= D_{15}=
D_{10}= C_u= C_c=

Remarks

Date Received: 2/9/21 **Date Tested:** 2/22/21

Tested By: JB

Checked By: JML

Title: PM

Location: HDR-B2: S7 **Date Sampled:** -

Sample Number: 62187 **Depth:** 20.0'-21.5'

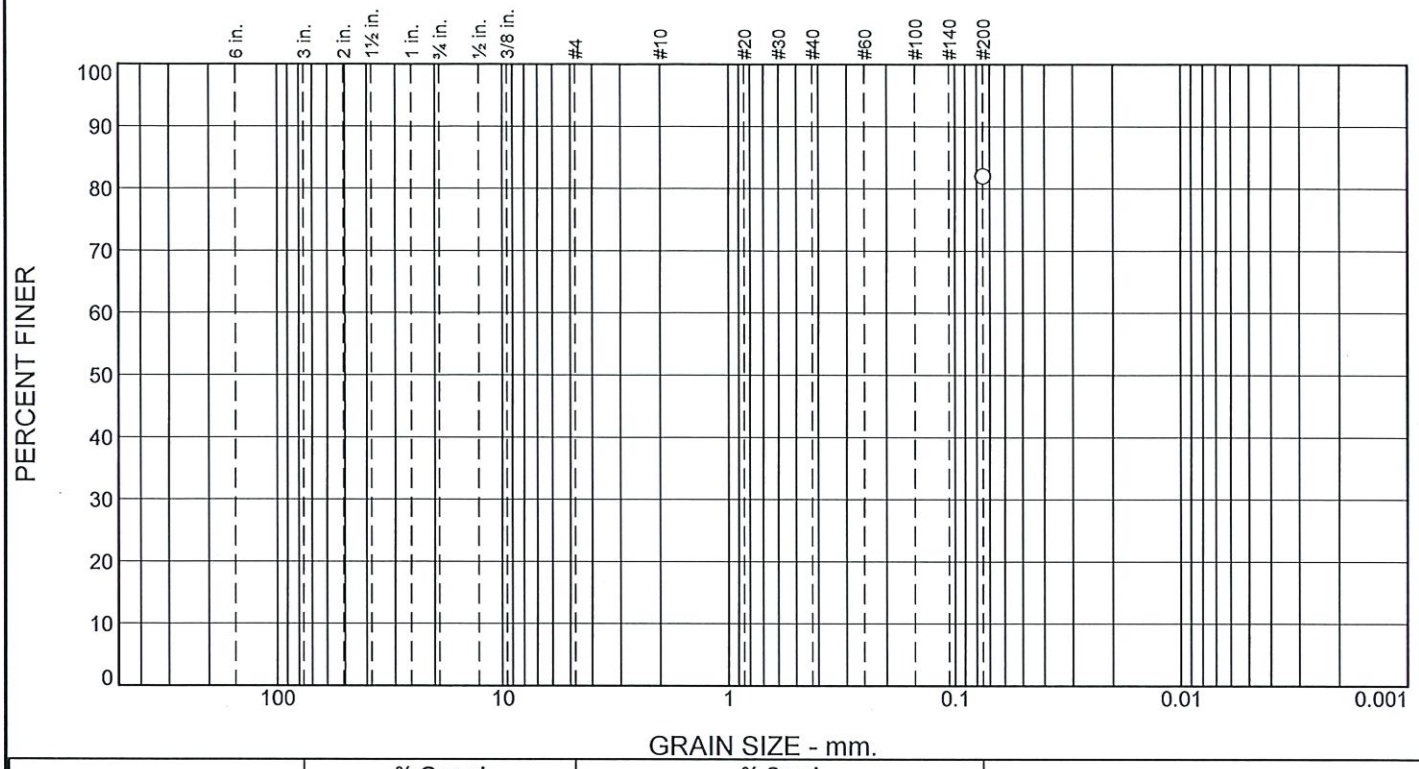
GULF SHORE EXPLORATION AND TESTING

Client: HDR, Inc.

Project: Bellotta Weir Modification

Project No: 21-105 **Figure**

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines
	Coarse	Fine	Coarse	Medium	Fine	
						82

Test Results (ASTM D6913 & ASTM D1140)

Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
#200	82		

* (no specification provided)

Material Description

Atterberg Limits (ASTM D 4318)

PL= LL= PI=

Classification

USCS (D 2487)= AASHTO (M 145)=

Coefficients

D_{90}= D_{85}= D_{60}=
D_{50}= D_{30}= D_{15}=
D_{10}= C_u= C_c=

Remarks

Date Received: 2/9/21 **Date Tested:** 2/22/21

Tested By: JB

Checked By: JML

Title: PM

Location: HDR-B2: S12 **Date Sampled:** -

Sample Number: 62189 **Depth:** 42.5'-44.0'

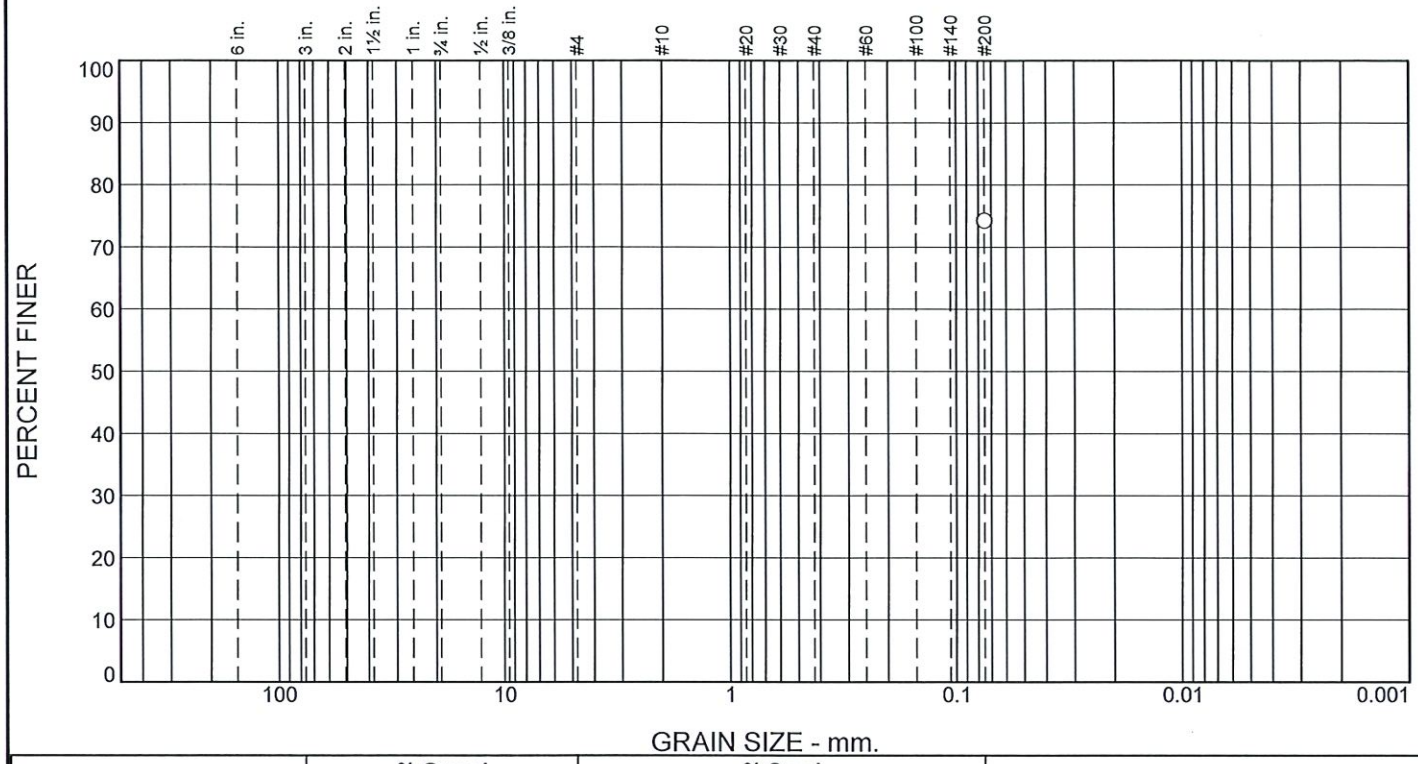
GULF SHORE EXPLORATION AND TESTING

Client: HDR, Inc.

Project: Bellotta Weir Modification

Project No: 21-105 **Figure**

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines
	Coarse	Fine	Coarse	Medium	Fine	
						74

Test Results (ASTM D6913 & ASTM D1140)			
Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
#200	74		

* (no specification provided)

Material Description

Atterberg Limits (ASTM D 4318)

PL= LL= PI=

Classification

USCS (D 2487)= AASHTO (M 145)=

Coefficients

D_{90}= D_{85}= D_{60}=
D_{50}= D_{30}= D_{15}=
D_{10}= C_u= C_c=

Remarks

Date Received: 2/9/21 **Date Tested:** 2/22/21

Tested By: JB

Checked By: JML

Title: PM

Location: HDR-B2: S15
Sample Number: 62190 **Depth:** 55.0'-56.5'

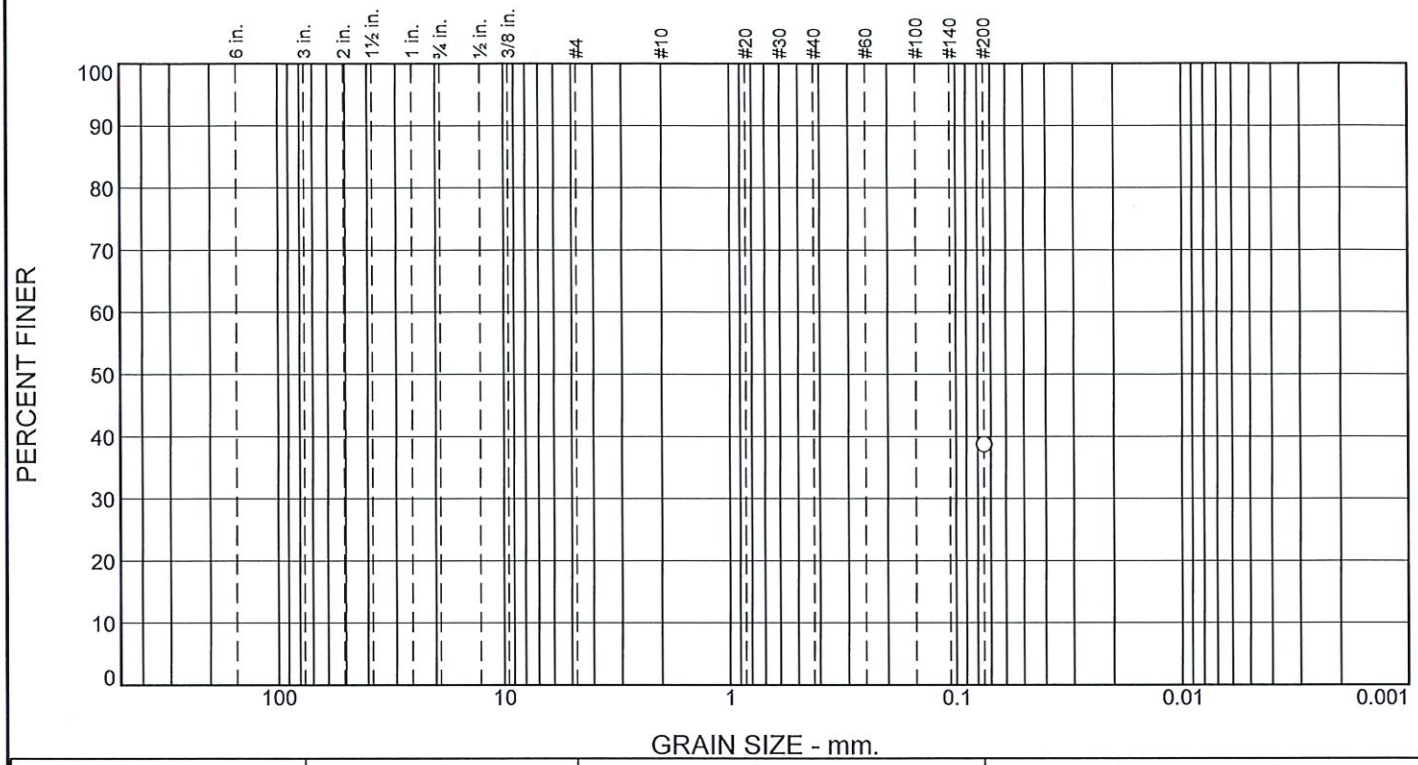
Date Sampled: -

GULF SHORE EXPLORATION AND TESTING

Client: HDR, Inc.
Project: Bellotta Weir Modification

Project No: 21-105 **Figure**

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines
	Coarse	Fine	Coarse	Medium	Fine	
						39

Test Results (ASTM D6913 & ASTM D1140)

Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
#200	39		

* (no specification provided)

Material Description

Atterberg Limits (ASTM D 4318)

PL= LL= PI=

Classification

USCS (D 2487)= AASHTO (M 145)=

Coefficients

D_{90}= D_{85}= D_{60}=
D_{50}= D_{30}= D_{15}=
D_{10}= C_u= C_c=

Remarks

Date Received: 2/9/21 **Date Tested:** 2/22/21

Tested By: JB

Checked By: JML

Title: PM

Location: HDR-B3: S5 **Date Sampled:** -

Sample Number: 62192 **Depth:** 12.5'-14.0'

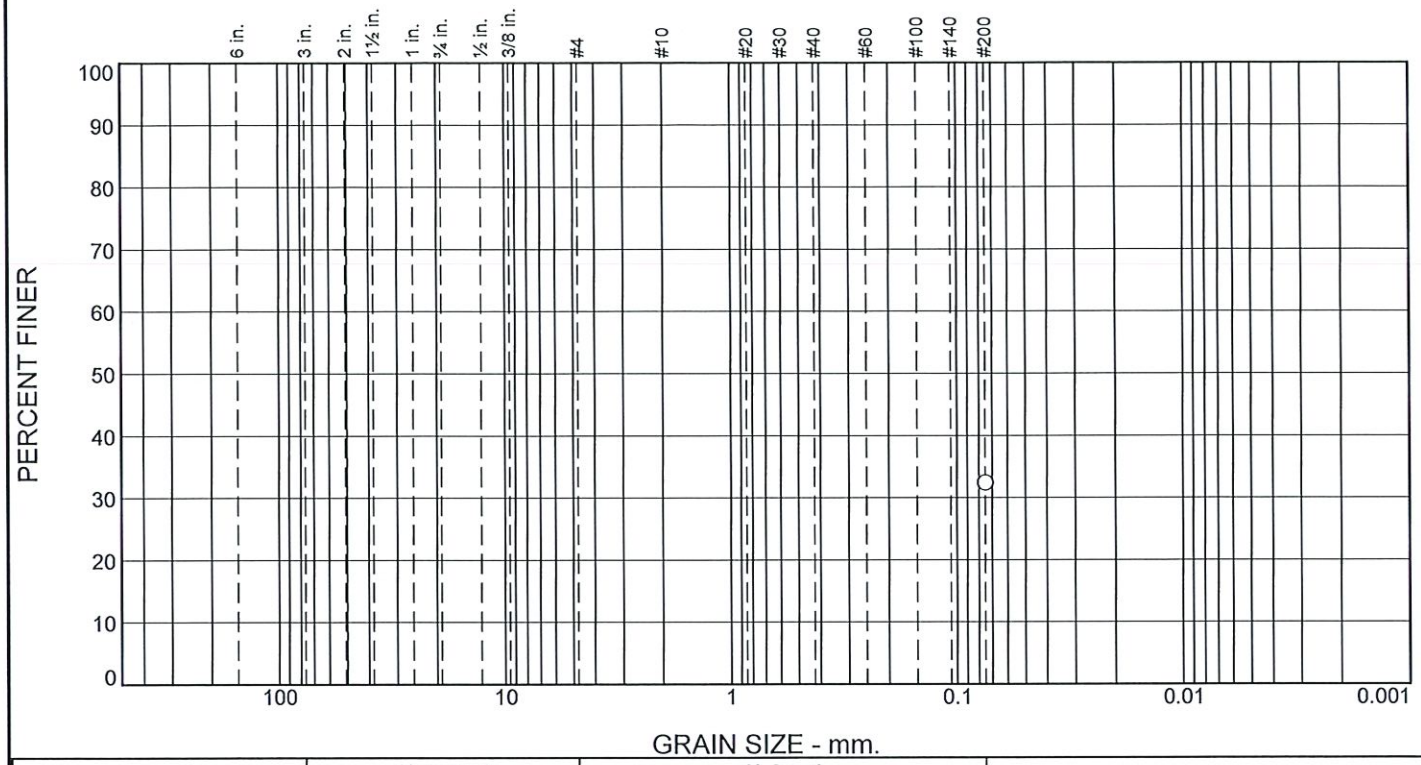
GULF SHORE EXPLORATION AND TESTING

Client: HDR, Inc.

Project: Bellotta Weir Modification

Project No: 21-105 **Figure**

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines
	Coarse	Fine	Coarse	Medium	Fine	
						32

Test Results (ASTM D6913 & ASTM D1140)

Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
#200	32		

* (no specification provided)

Material Description

Atterberg Limits (ASTM D 4318)

PL= LL= PI=

Classification

USCS (D 2487)= AASHTO (M 145)=

Coefficients

D_{90}= D_{85}= D_{60}=
D_{50}= D_{30}= D_{15}=
D_{10}= C_u= C_c=

Remarks

Date Received: 2//9/21 **Date Tested:** 2/22/21

Tested By: JB

Checked By: JML

Title: PM

Location: HDR-B3: S6 **Date Sampled:** -

Sample Number: 62193 **Depth:** 15.0'-16.5'

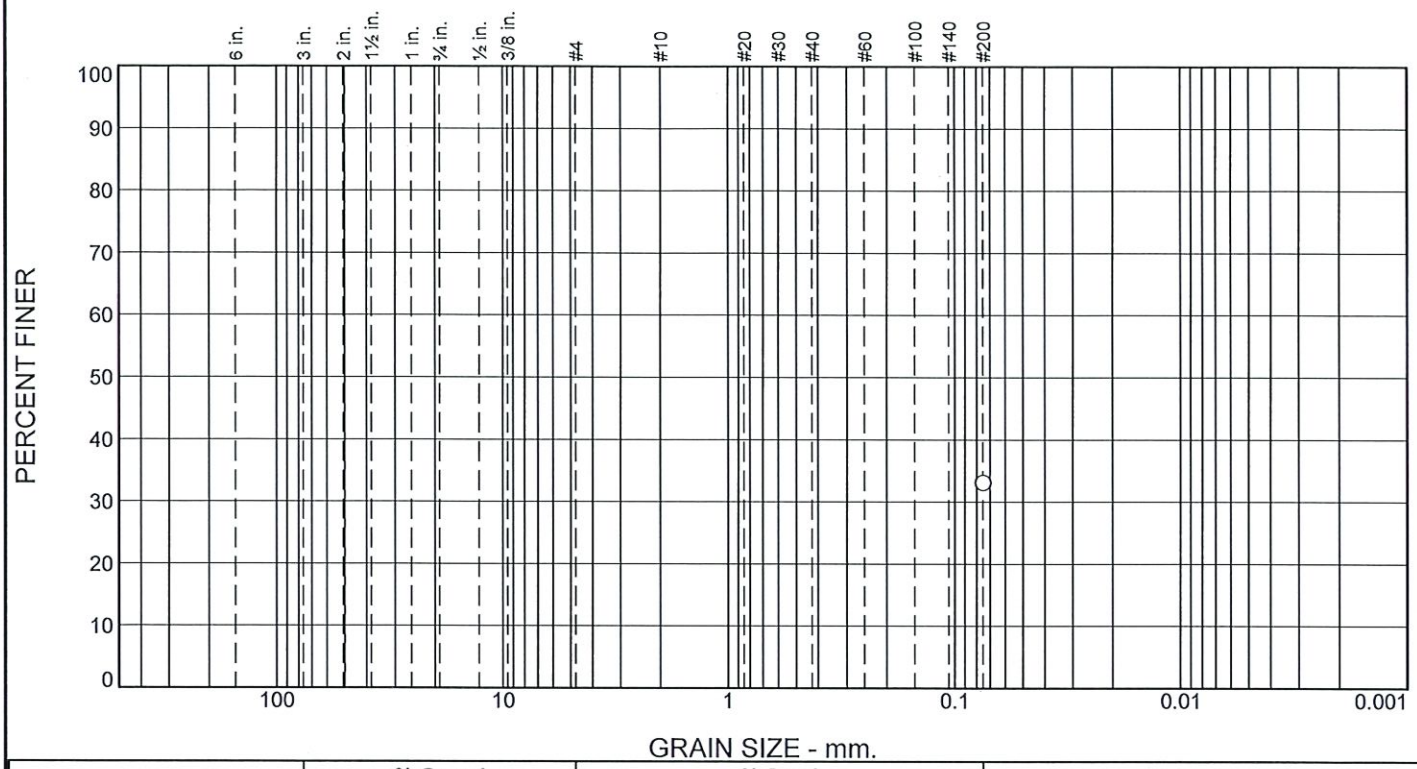
GULF SHORE EXPLORATION AND TESTING

Client: HDR, Inc.

Project: Bellotta Weir Modification

Project No: 21-105 **Figure**

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines
	Coarse	Fine	Coarse	Medium	Fine	
						33

Test Results (ASTM D6913 & ASTM D1140)			
Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
#200	33		

* (no specification provided)

Material Description

Atterberg Limits (ASTM D 4318)

PL= LL= PI=

Classification

USCS (D 2487)= AASHTO (M 145)=

Coefficients

D_{90}= D_{85}= D_{60}=
D_{50}= D_{30}= D_{15}=
D_{10}= C_u= C_c=

Remarks

Date Received: 2/9/21 **Date Tested:** 2/22/21

Tested By: JB

Checked By: JML

Title: PM

Location: HDR-B3: S8
Sample Number: 62194 **Depth:** 25.0'-26.5'

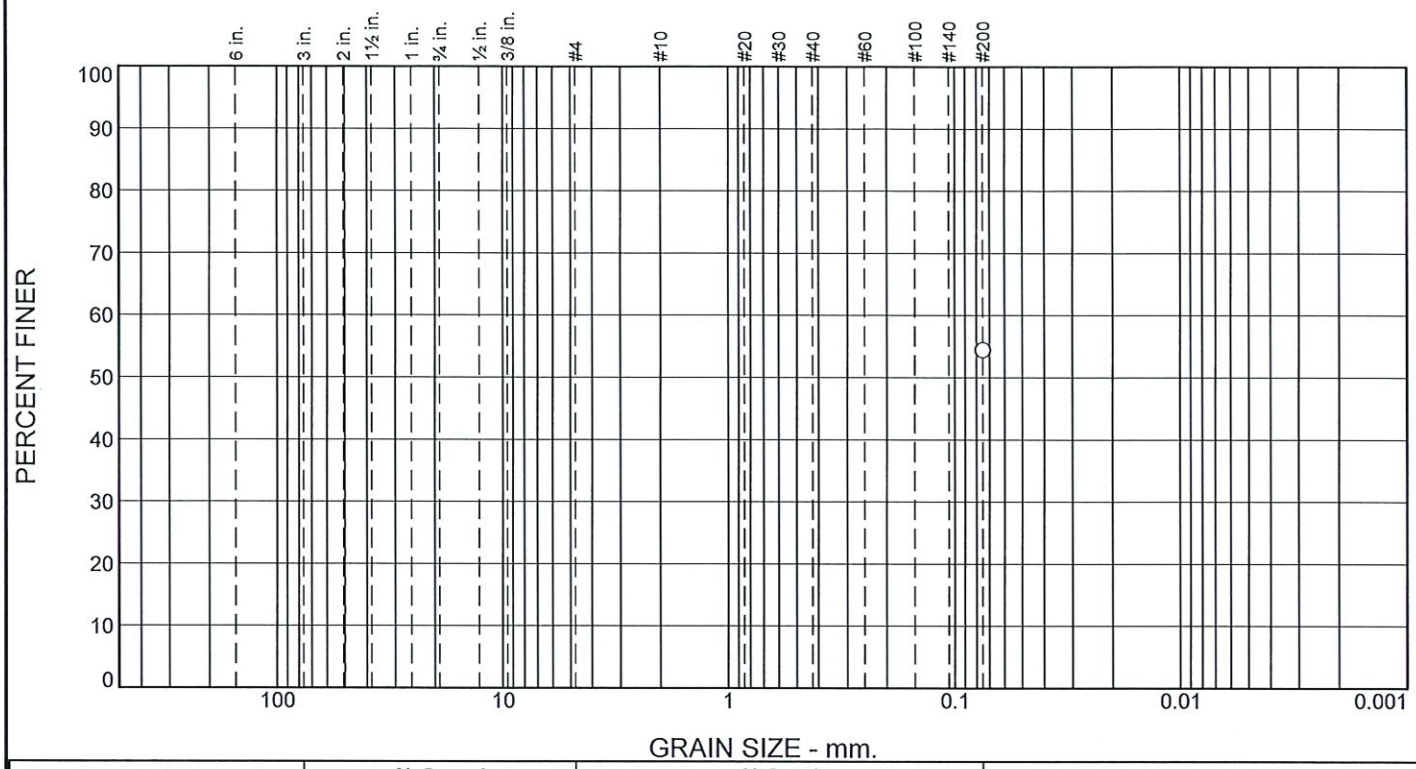
Date Sampled: -

GULF SHORE EXPLORATION AND TESTING

Client: HDR, Inc.
Project: Bellotta Weir Modification

Project No: 21-105 **Figure**

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines
	Coarse	Fine	Coarse	Medium	Fine	
						54

Test Results (ASTM D6913 & ASTM D1140)

Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
#200	54		

* (no specification provided)

Material Description

Atterberg Limits (ASTM D 4318)

PL= LL= PI=

Classification

USCS (D 2487)= AASHTO (M 145)=

Coefficients

D_{90}= D_{85}= D_{60}=
D_{50}= D_{30}= D_{15}=
D_{10}= C_u= C_c=

Remarks

Date Received: 2/9/21 **Date Tested:** 2/22/21

Tested By: JB

Checked By: JML

Title: PM

Location: HDR-B3: S17
Sample Number: 62198 **Depth:** 70.0'-71.5'

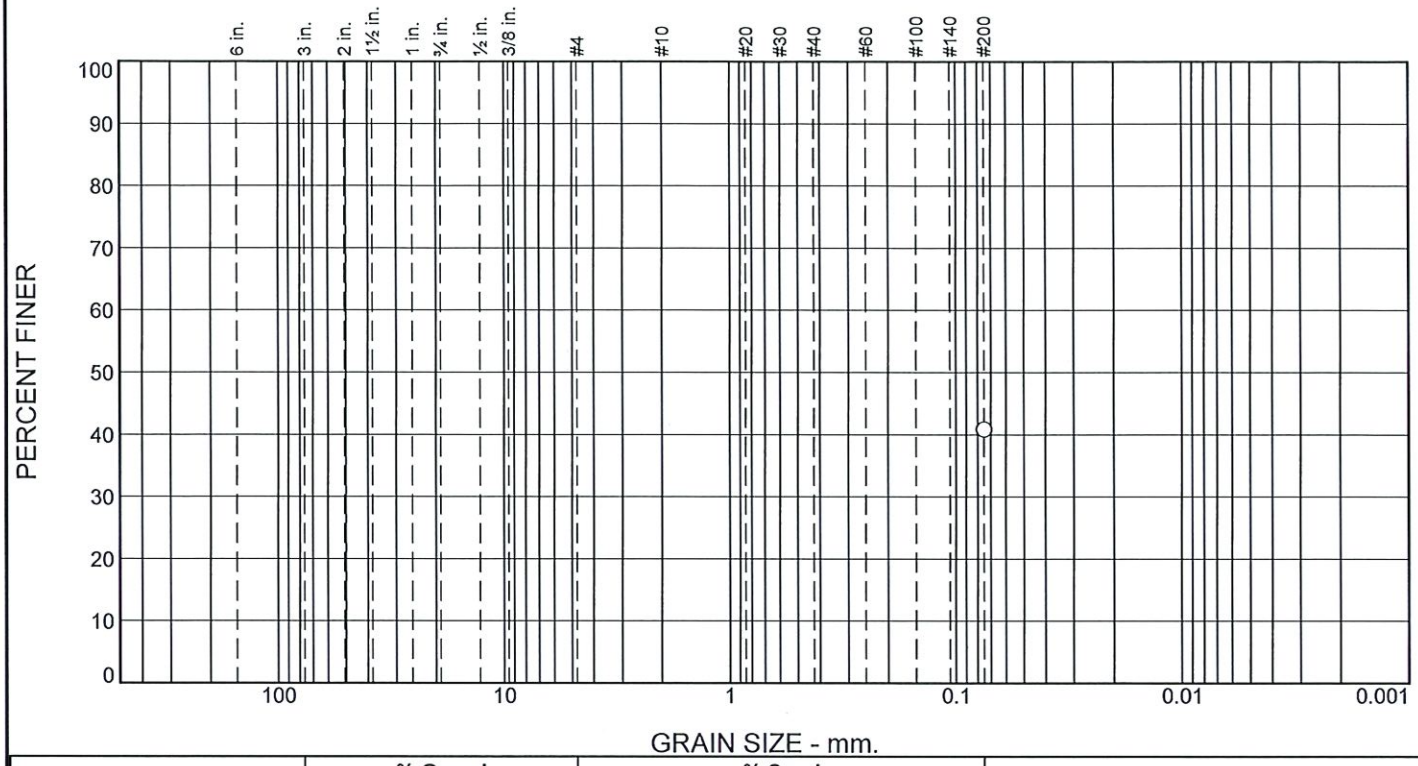
Date Sampled: -

GULF SHORE EXPLORATION AND TESTING

Client: HDR, Inc.
Project: Bellotta Weir Modification

Project No: 21-105 **Figure**

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines
	Coarse	Fine	Coarse	Medium	Fine	
						41

Test Results (ASTM D6913 & ASTM D1140)

Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
#200	41		

* (no specification provided)

Material Description

Atterberg Limits (ASTM D 4318)

PL= LL= PI=

Classification

USCS (D 2487)= AASHTO (M 145)=

Coefficients

D_{90}= D_{85}= D_{60}=
D_{50}= D_{30}= D_{15}=
D_{10}= C_u= C_c=

Remarks

Date Received: 2/9/21 **Date Tested:** 2/22/21

Tested By: JB

Checked By: JML

Title: PM

Location: HDR-B4: S4 **Date Sampled:** -

Sample Number: 62201 **Depth:** 10.0'-11.5'

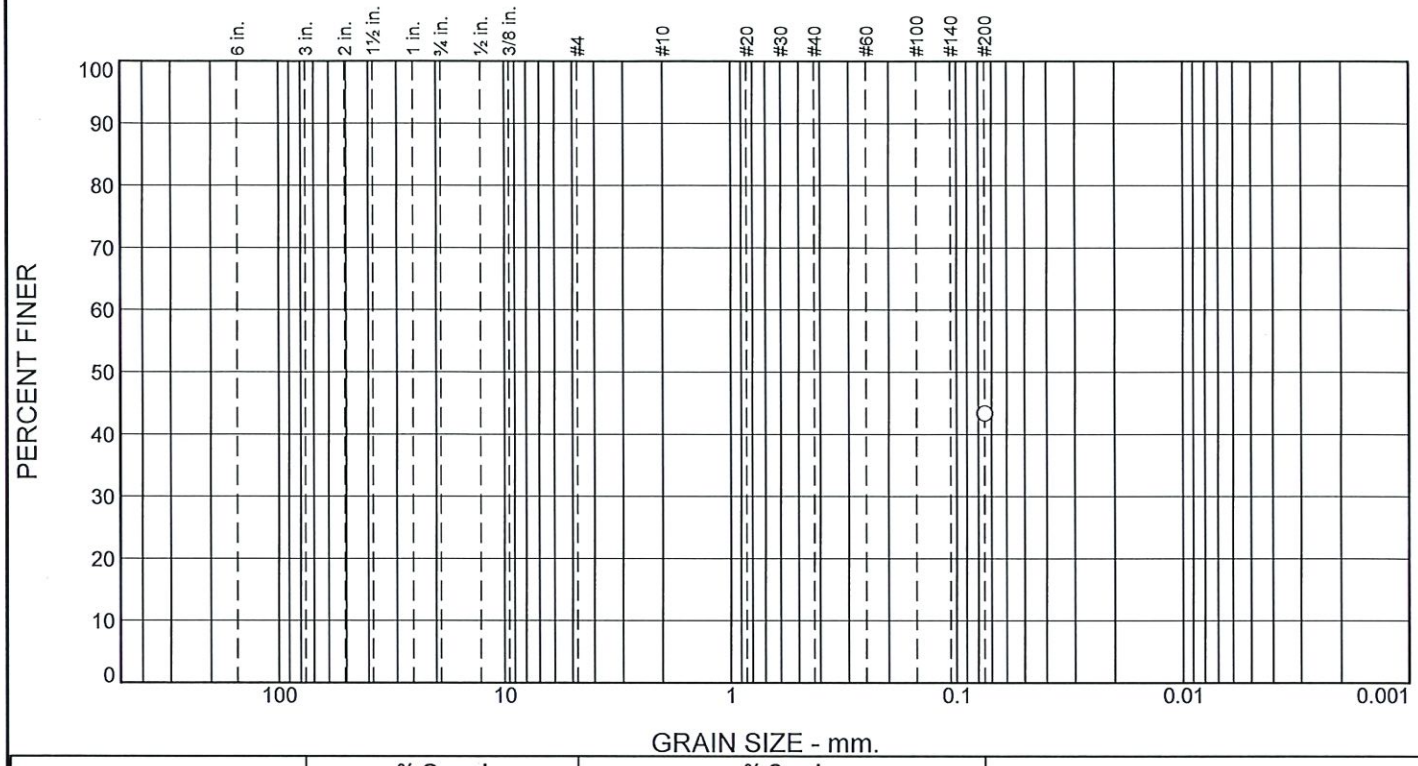
GULF SHORE EXPLORATION AND TESTING

Client: HDR, Inc.

Project: Bellotta Weir Modification

Project No: 21-105 **Figure**

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines
	Coarse	Fine	Coarse	Medium	Fine	
						43

Test Results (ASTM D6913 & ASTM D1140)

Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
#200	43		

* (no specification provided)

Material Description

Atterberg Limits (ASTM D 4318)

PL= LL= PI=

Classification

USCS (D 2487)= AASHTO (M 145)=

Coefficients

D_{90}= D_{85}= D_{60}=
D_{50}= D_{30}= D_{15}=
D_{10}= C_u= C_c=

Remarks

Date Received: 2/9/21 **Date Tested:** 2/22/21

Tested By: JB

Checked By: JML

Title: PM

Location: HDR-B5: S14
Sample Number: 62212 **Depth:** 52.5'-54.0'

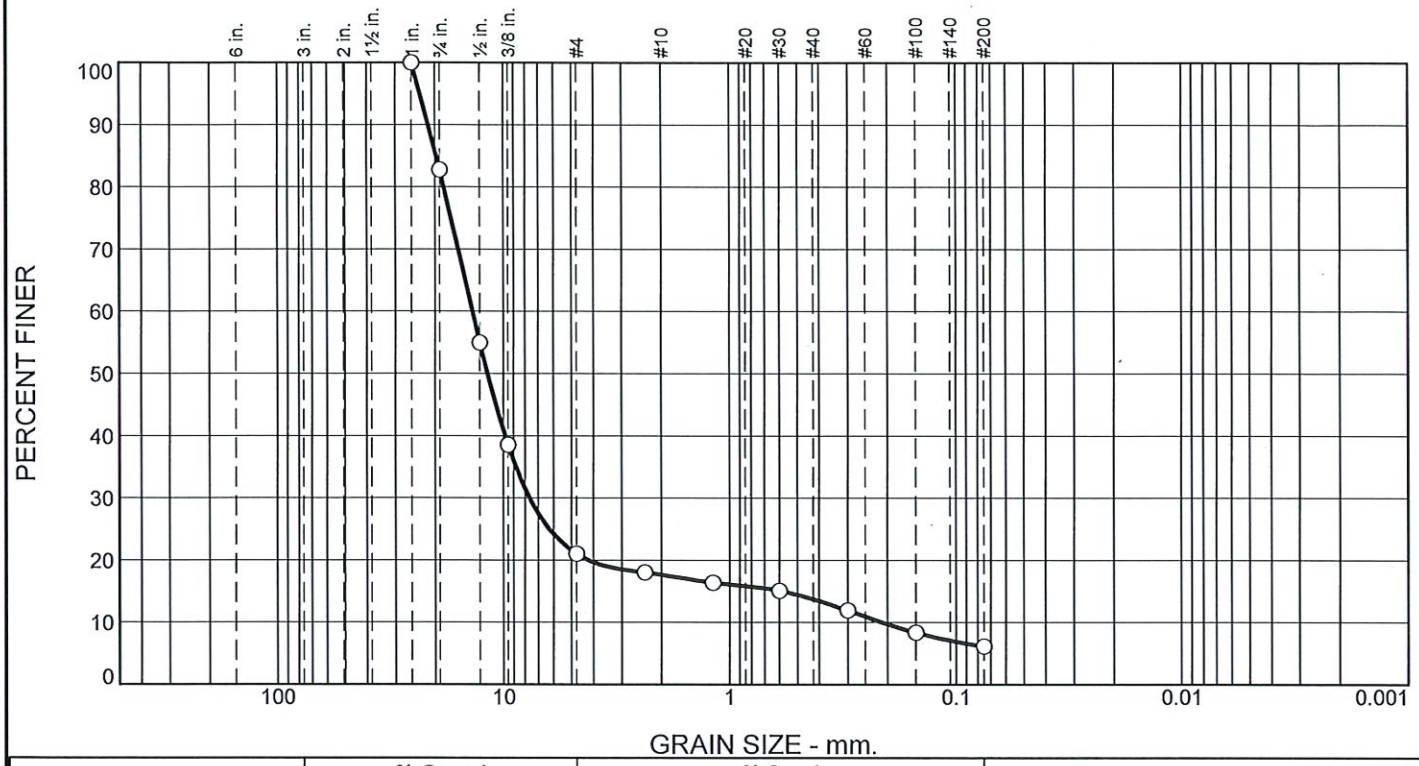
Date Sampled: -

GULF SHORE EXPLORATION AND TESTING

Client: HDR, Inc.
Project: Bellotta Weir Modification

Project No: 21-105 **Figure**

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines
	Coarse	Fine	Coarse	Medium	Fine	
0	17	62	3	4	8	6

Test Results (ASTM D6913 & ASTM D1140)

Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
1 Inch	100		
3/4 Inch	83		
1/2 Inch	55		
3/8 Inch	39		
#4	21		
#8	18		
#16	16		
#30	15		
#50	12		
#100	8		
#200	6.1		

* (no specification provided)

Material Description

Atterberg Limits (ASTM D 4318)

PL= LL= PI=

Classification

USCS (D 2487)= AASHTO (M 145)=

Coefficients

D_{90}= 21.4017 D_{85}= 19.7318 D_{60}= 13.6905
D_{50}= 11.7471 D_{30}= 7.6396 D_{15}= 0.5863
D_{10}= 0.2114 C_u= 64.77 C_c= 20.17

Remarks

Not enough sample for the max particle size (below the minimum mass requirement for specimen as required in ASTM). F.M.=5.88

Date Received: 2/9/21 **Date Tested:** 2/22/21

Tested By: JB

Checked By: JML

Title: PM

Location: HDR-B1: S8 **Date Sampled:** -

Sample Number: 62179 **Depth:** 20.0'-21.5'

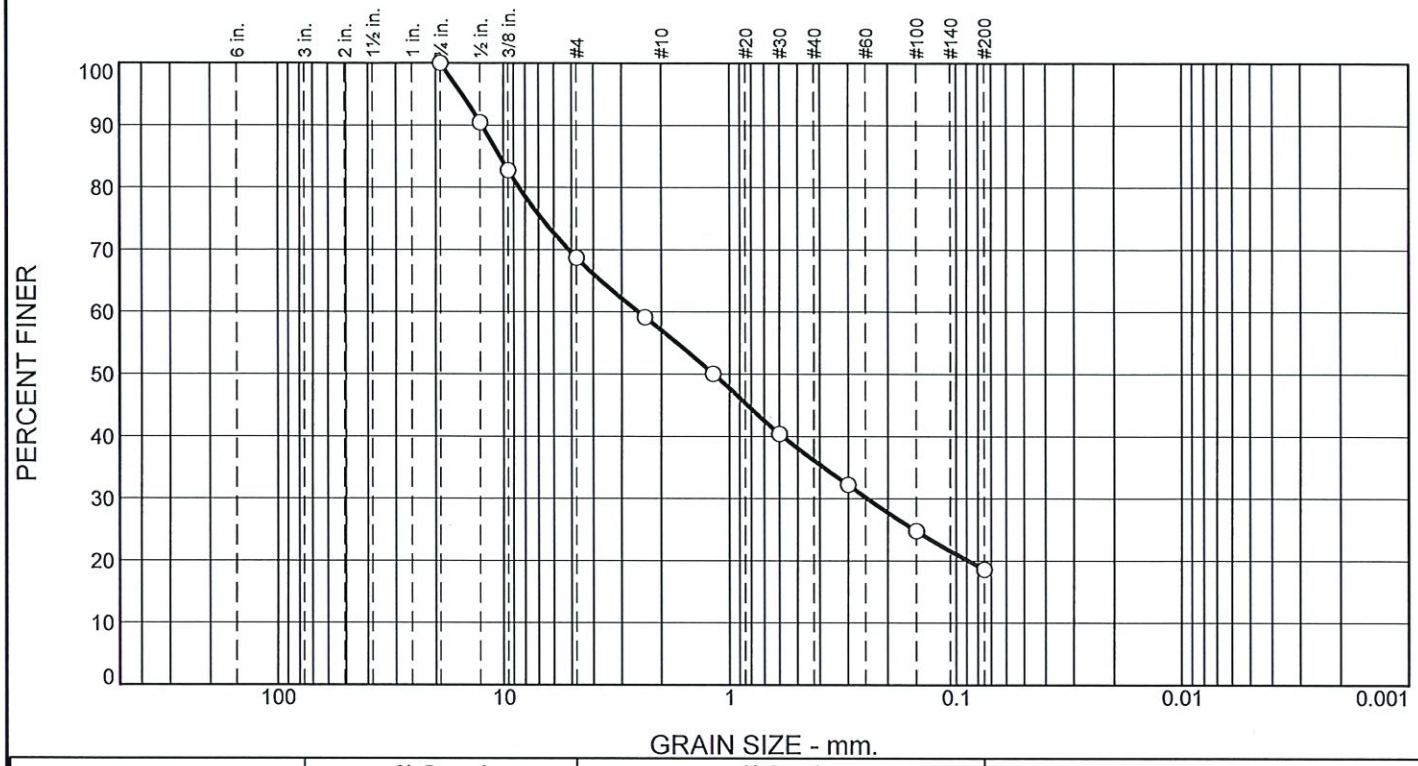
GULF SHORE EXPLORATION AND TESTING

Client: HDR, Inc.

Project: Bellotta Weir Modification

Project No: 21-105 **Figure**

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines
	Coarse	Fine	Coarse	Medium	Fine	
0	0	31	12	21	17	19

Test Results (ASTM D6913 & ASTM D1140)

Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
3/4 Inch	100		
1/2 Inch	90		
3/8 Inch	83		
#4	69		
#8	59		
#16	50		
#30	40		
#50	32		
#100	25		
#200	19		

* (no specification provided)

Material Description

Atterberg Limits (ASTM D 4318)

PL= 29 LL= 52 PI= 23

Classification

USCS (D 2487)= SC AASHTO (M 145)= A-2-7(1)

Coefficients

D_{90}= 12.4880 D_{85}= 10.3703 D_{60}= 2.5212
D_{50}= 1.1736 D_{30}= 0.2457 D_{15}=
D_{10}= C_u= C_c=

Remarks

Not enough sample for the max particle size (below the minimum mass requirement for specimen as required in ASTM). F.M.=3.42

Date Received: 2/9/21 **Date Tested:** 2/22/21

Tested By: JB

Checked By: JML

Title: PM

Location: HDR-B1: S11 **Date Sampled:** -

Sample Number: 62181 **Depth:** 35.0'-36.5'

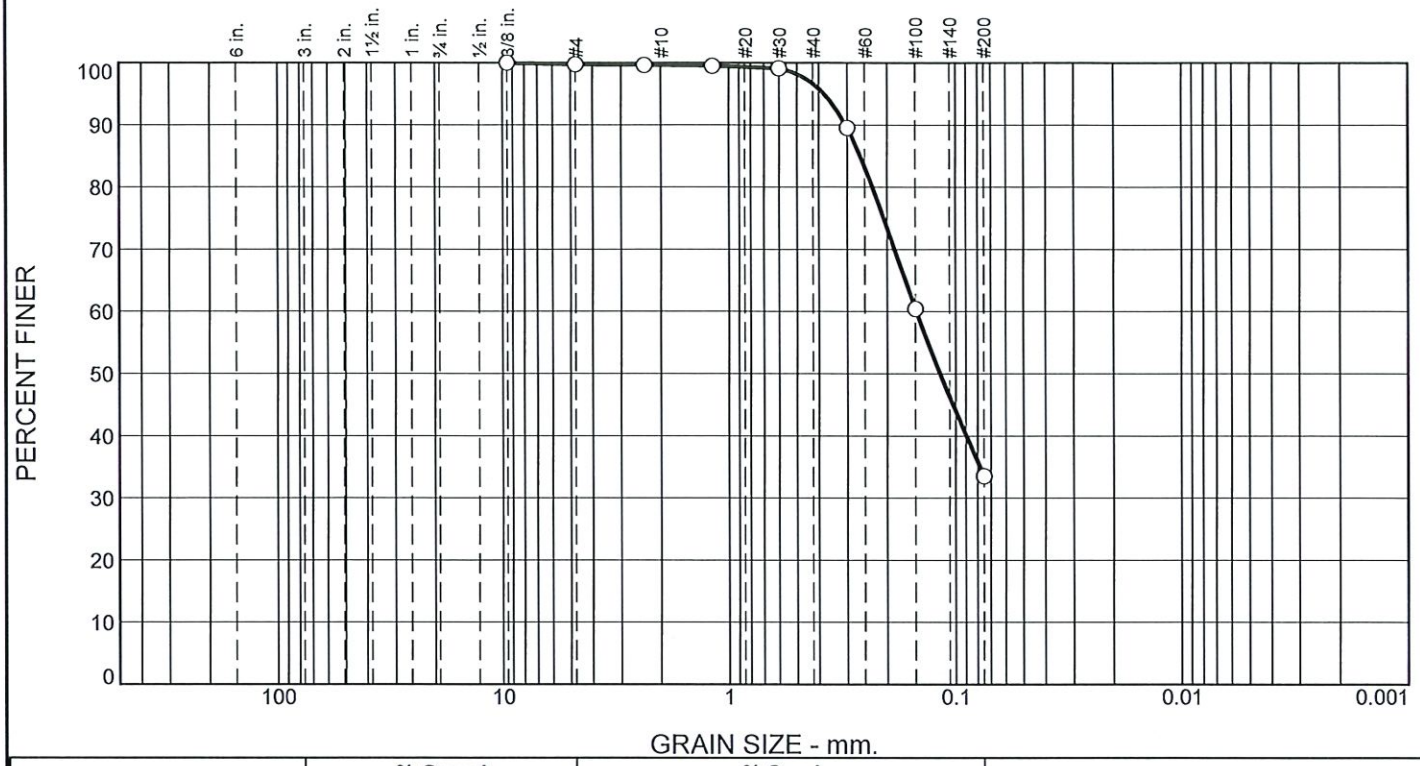
GULF SHORE EXPLORATION AND TESTING

Client: HDR, Inc.

Project: Bellotta Weir Modification

Project No: 21-105 **Figure**

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines
	Coarse	Fine	Coarse	Medium	Fine	
0	0	0	0	3	63	34

Test Results (ASTM D6913 & ASTM D1140)

Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
3/8 Inch	100		
#4	100		
#8	100		
#16	100		
#30	99		
#50	90		
#100	60		
#200	34		

* (no specification provided)

Material Description

Atterberg Limits (ASTM D 4318)

PL= LL= PI=

Classification

USCS (D 2487)= AASHTO (M 145)=

Coefficients

D_{90}= 0.3048 D_{85}= 0.2624 D_{60}= 0.1486
D_{50}= 0.1167 D_{30}= D_{15}=
D_{10}= C_u= C_c=

Remarks

F.M.=0.52

Date Received: 2/9/21 **Date Tested:** 2/22/21

Tested By: JB

Checked By: JML

Title: PM

Location: HDR-B2: S1 **Date Sampled:** -

Sample Number: 62183 **Depth:** 2.5'-3.0'

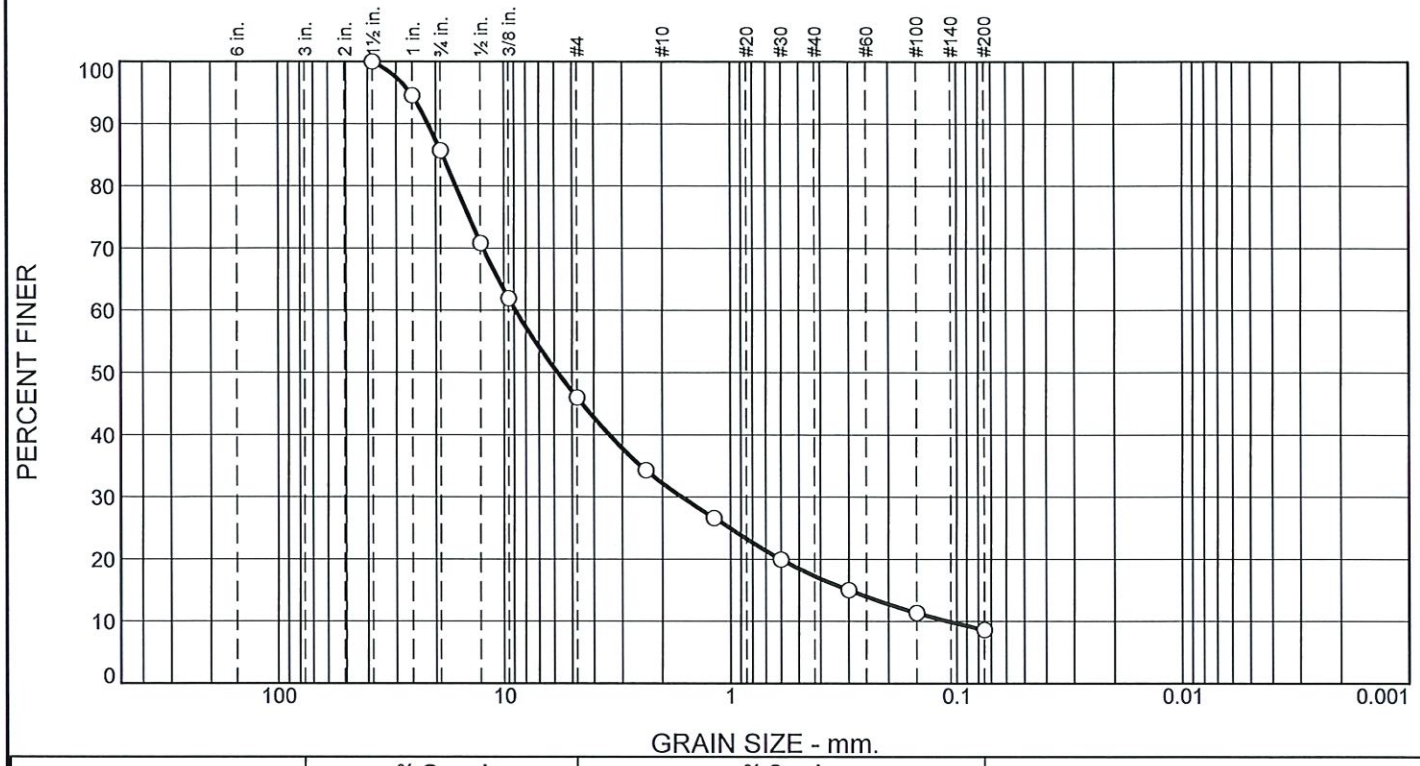
GULF SHORE EXPLORATION AND TESTING

Client: HDR, Inc.

Project: Bellotta Weir Modification

Project No: 21-105 **Figure**

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines
	Coarse	Fine	Coarse	Medium	Fine	
0	14	40	14	15	8	9

Test Results (ASTM D6913 & ASTM D1140)			
Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
1 1/2 Inch	100		
1 Inch	95		
3/4 Inch	86		
1/2 Inch	71		
3/8 Inch	62		
#4	46		
#8	34		
#16	27		
#30	20		
#50	15		
#100	11		
#200	8.6		

* (no specification provided)

Material Description

Atterberg Limits (ASTM D 4318)

PL= LL= PI=

Classification

USCS (D 2487)= AASHTO (M 145)=

Coefficients

D_{90}= 21.5995 D_{85}= 18.6961 D_{60}= 8.8784
D_{50}= 5.8017 D_{30}= 1.6425 D_{15}= 0.2991
D_{10}= 0.1087 C_u= 81.69 C_c= 2.80

Remarks

Not enough sample for the max particle size (below the minimum mass requirement for specimen as required in ASTM). F.M.=4.99

Date Received: 2/9/21 **Date Tested:** 2/23/21

Tested By: JB

Checked By: JML

Title: PM

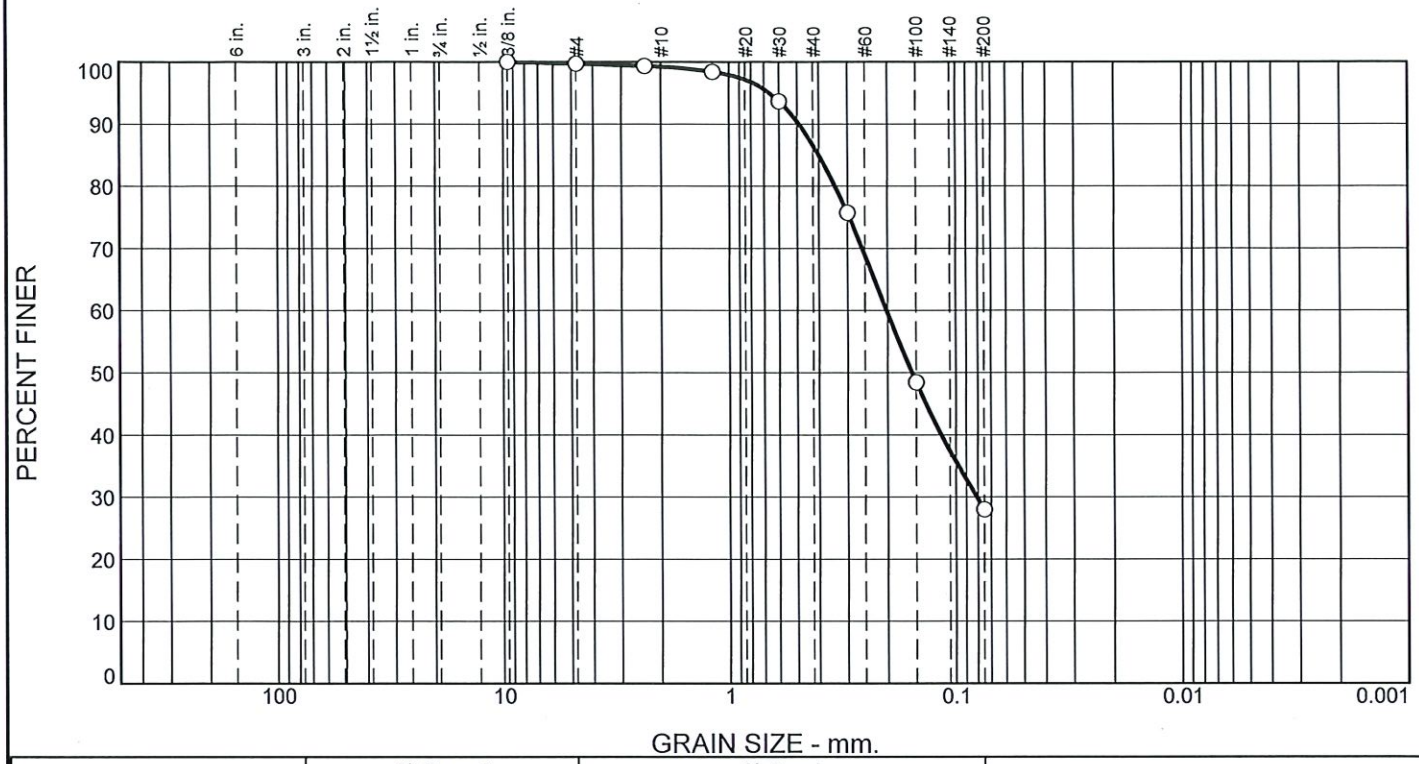
Location: HDR-B2: S10 **Date Sampled:** -
Sample Number: 62188 **Depth:** 35.0'-36.5'

GULF SHORE EXPLORATION AND TESTING

Client: HDR, Inc.
Project: Bellotta Weir Modification

Project No: 21-105 **Figure**

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines
	Coarse	Fine	Coarse	Medium	Fine	
0	0	0	1	12	59	28

Test Results (ASTM D6913 & ASTM D1140)

Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
3/8 Inch	100		
#4	100		
#8	99		
#16	98		
#30	94		
#50	76		
#100	48		
#200	28		

* (no specification provided)

Material Description

Atterberg Limits (ASTM D 4318)

PL= LL= PI=

Classification

USCS (D 2487)= AASHTO (M 145)=

Coefficients

D_{90}= 0.4907 D_{85}= 0.4008 D_{60}= 0.2016
D_{50}= 0.1564 D_{30}= 0.0809 D_{15}=
D_{10}= C_u= C_c=

Remarks

F.M.=0.85

Date Received: 2/9/21 **Date Tested:** 2/22/21

Tested By: JB

Checked By: JML

Title: PM

Location: HDR-B4: S6 **Date Sampled:** -

Sample Number: 62202 **Depth:** 15.0'-16.5'

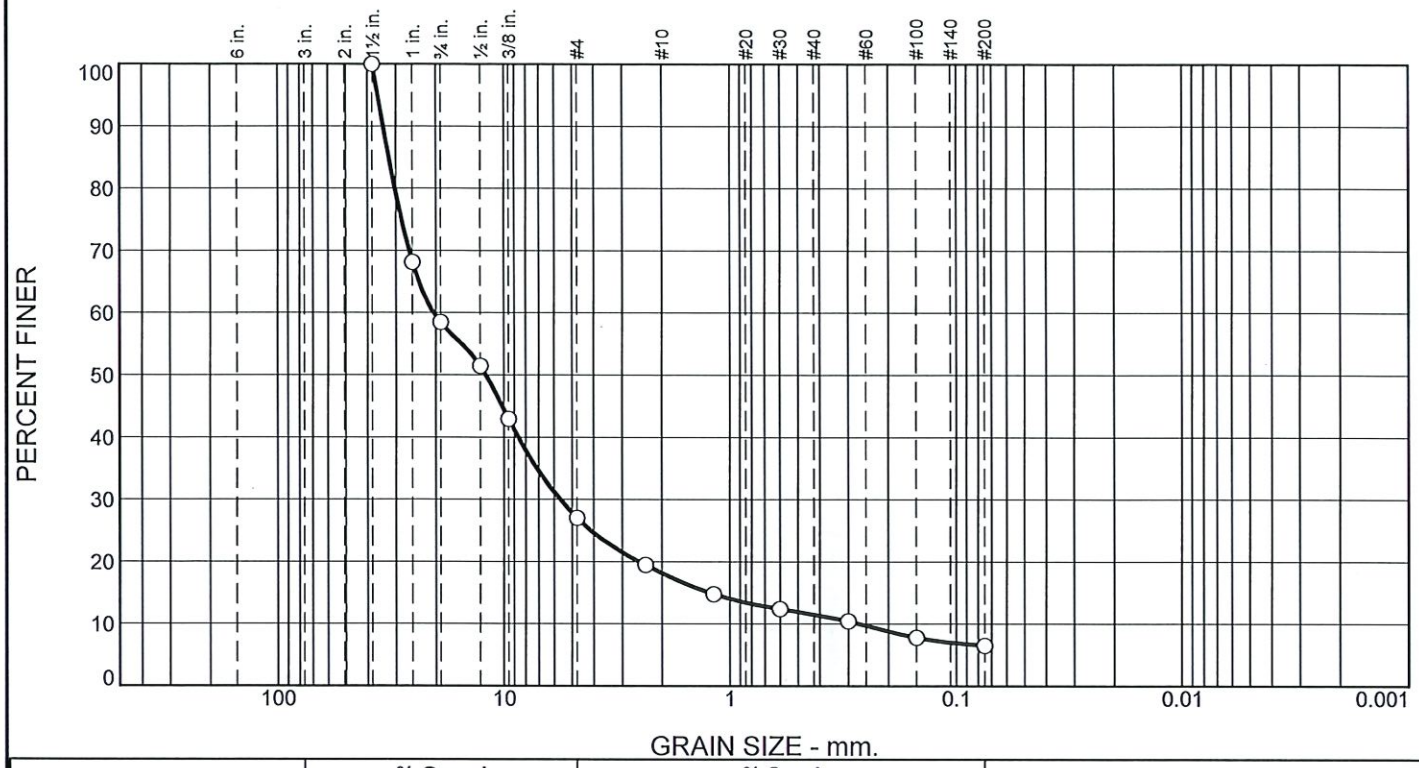
GULF SHORE EXPLORATION AND TESTING

Client: HDR, Inc.

Project: Bellotta Weir Modification

Project No: 21-105 **Figure**

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines
	Coarse	Fine	Coarse	Medium	Fine	
0	42	31	9	7	4	7

Test Results (ASTM D6913 & ASTM D1140)

Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
1 1/2 Inch	100		
1 Inch	68		
3/4 Inch	58		
1/2 Inch	51		
3/8 Inch	43		
#4	27		
#8	19		
#16	15		
#30	12		
#50	10		
#100	8		
#200	6.5		

* (no specification provided)

Material Description

Atterberg Limits (ASTM D 4318)

PL= LL= PI=

Classification

USCS (D 2487)= AASHTO (M 145)=

Coefficients

D_{90}= 34.0913 D_{85}= 32.1742 D_{60}= 20.4269
D_{50}= 12.0038 D_{30}= 5.6494 D_{15}= 1.2289
D_{10}= 0.2685 C_u= 76.07 C_c= 5.82

Remarks

Not enough sample for the max particle size (below the minimum mass requirement for specimen as required in ASTM). F.M.=6.07

Date Received: 2/9/21 **Date Tested:** 2/22/21

Tested By: JB

Checked By: JML

Title: PM

Location: HDR-B4: S10 **Date Sampled:** -

Sample Number: 62204 **Depth:** 35.0'-36.5'

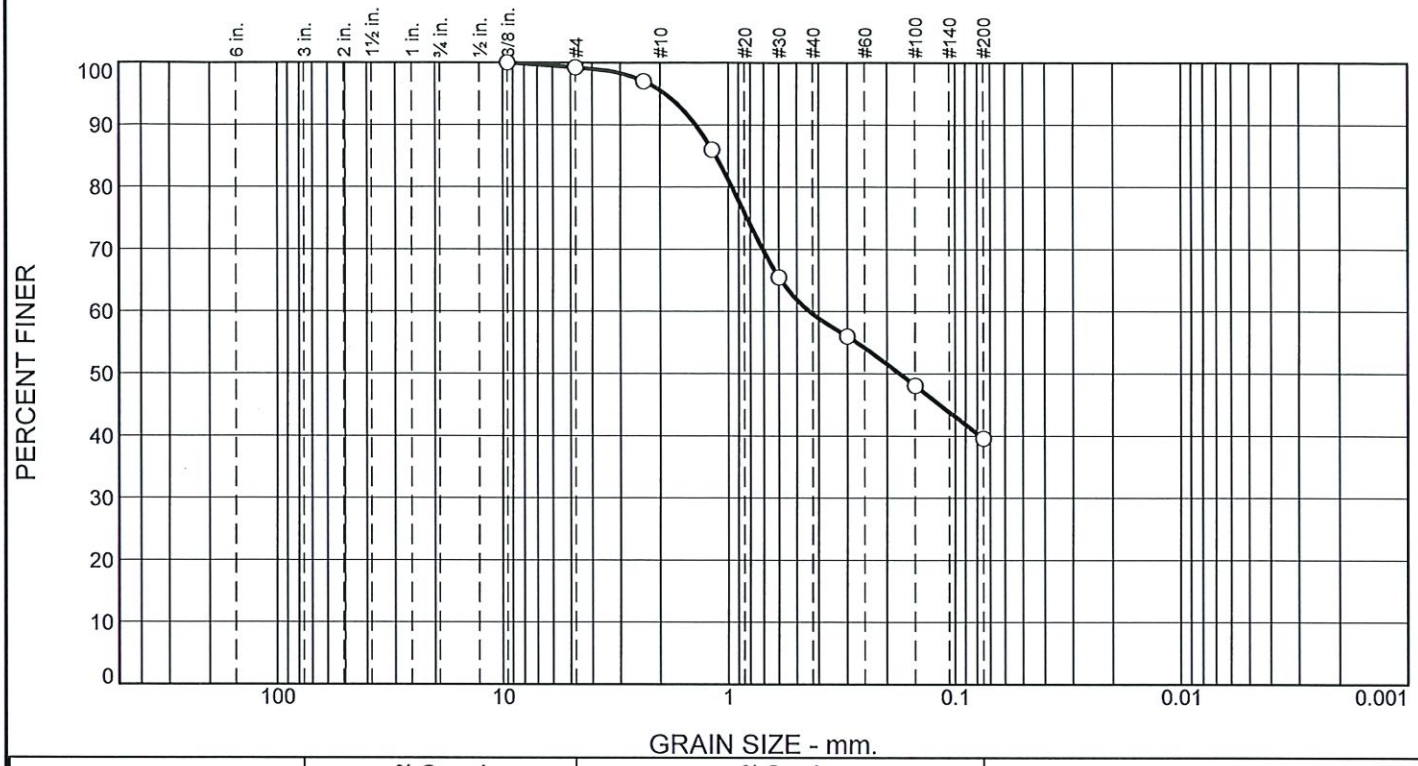
GULF SHORE EXPLORATION AND TESTING

Client: HDR, Inc.

Project: Bellotta Weir Modification

Project No: 21-105 **Figure**

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines
	Coarse	Fine	Coarse	Medium	Fine	
0	0	1	3	36	20	40

Test Results (ASTM D6913 & ASTM D1140)

Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
3/8 Inch	100		
#4	99		
#8	97		
#16	86		
#30	66		
#50	56		
#100	48		
#200	40		

* (no specification provided)

Material Description

Atterberg Limits (ASTM D 4318)

PL= LL= PI=

Classification

USCS (D 2487)= AASHTO (M 145)=

Coefficients

D_{90}= 1.3937 D_{85}= 1.1366 D_{60}= 0.4400
D_{50}= 0.1754 D_{30}= D_{15}=
D_{10}= C_u= C_c=

Remarks

F.M.=1.48

Date Received: 2/9/21 **Date Tested:** 2/22/21

Tested By: JB

Checked By: JML

Title: PM

Location: HDR-B4: S17 **Date Sampled:** -

Sample Number: 62207 **Depth:** 65.0'-66.5'

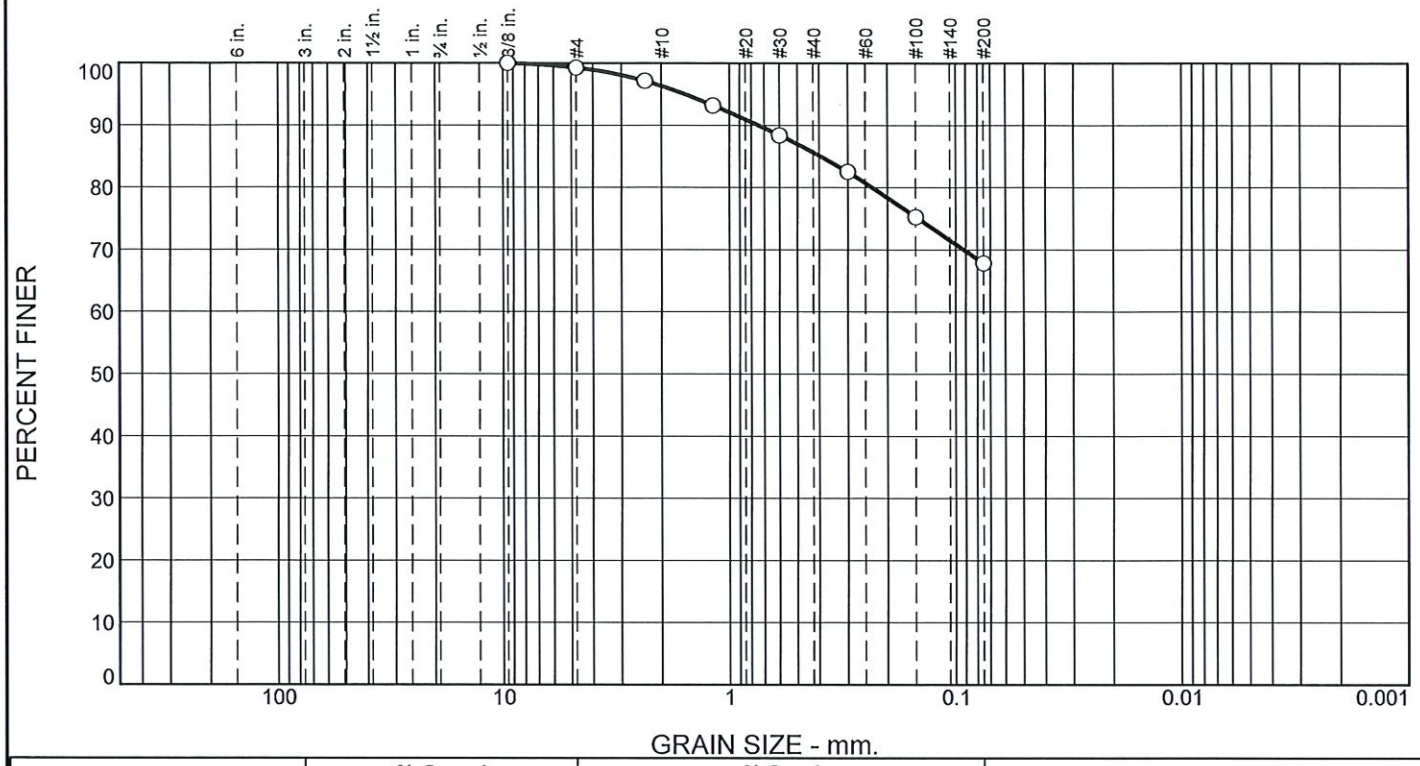
GULF SHORE EXPLORATION AND TESTING

Client: HDR, Inc.

Project: Bellotta Weir Modification

Project No: 21-105 **Figure**

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines
	Coarse	Fine	Coarse	Medium	Fine	
0	0	1	3	10	18	68

Test Results (ASTM D6913 & ASTM D1140)

Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
3/8 Inch	100		
#4	99		
#8	97		
#16	93		
#30	88		
#50	83		
#100	75		
#200	68		

* (no specification provided)

Material Description

Atterberg Limits (ASTM D 4318)

PL= LL= PI=

Classification

USCS (D 2487)= AASHTO (M 145)=

Coefficients

D_{90}= 0.7435 D_{85}= 0.3940 D_{60}=
D_{50}= D_{30}= D_{15}=
D_{10}= C_u= C_c=

Remarks

F.M.=0.64

Date Received: 2/9/21 **Date Tested:** 2/23/21

Tested By: JB

Checked By: JML

Title: PM

Location: HDR-B5: S11 **Date Sampled:** -

Sample Number: 62211 **Depth:** 40.0'-41.5'

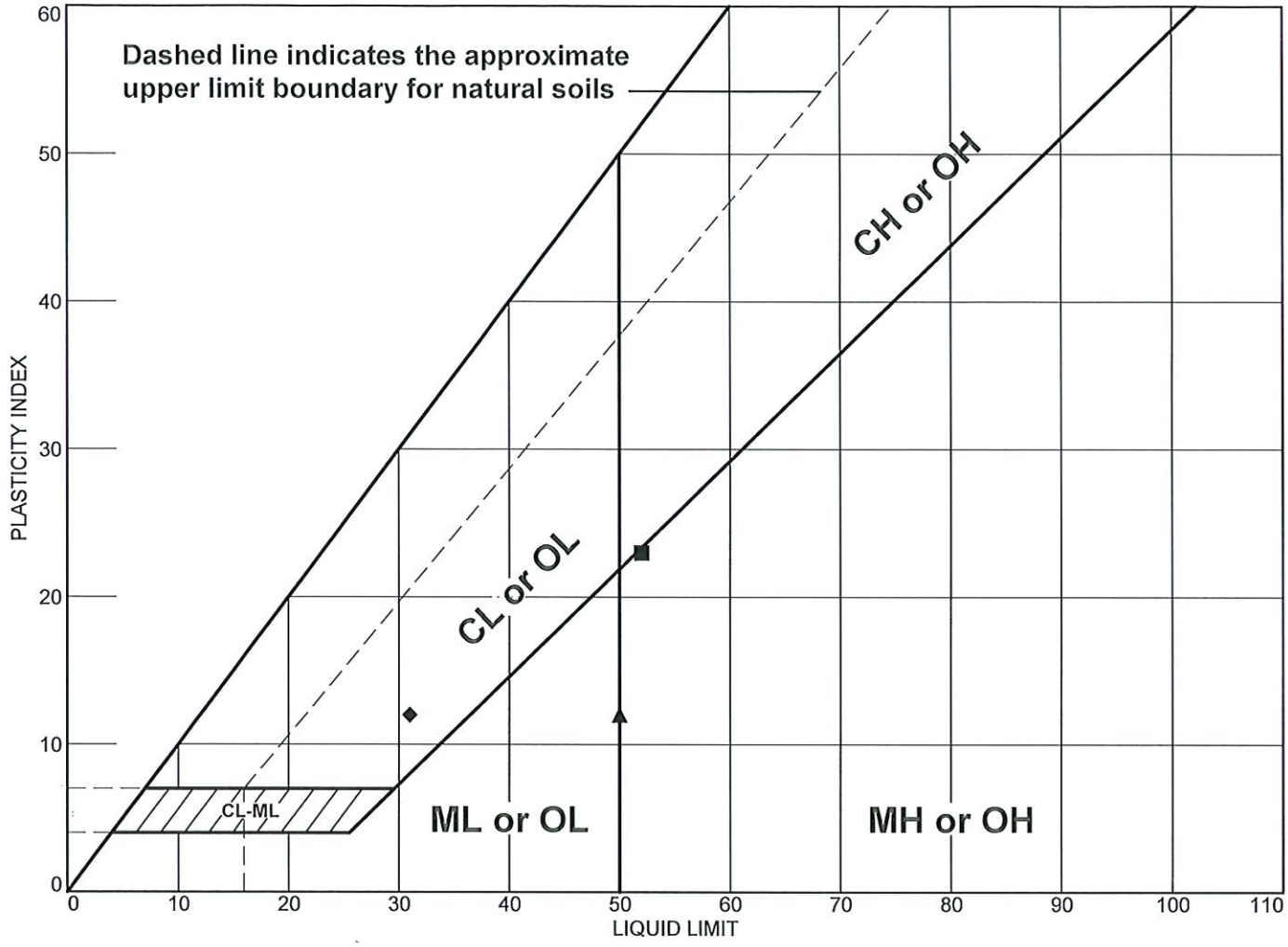
GULF SHORE EXPLORATION AND TESTING

Client: HDR, Inc.

Project: Bellotta Weir Modification

Project No: 21-105 **Figure**

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●		NV	NP	NP			
■		52	29	23	36	19	SC
▲		50	38	12			
◆		31	19	12			

Project No. 21-105 **Client:** HDR, Inc.

Project: Bellotta Weir Modification

● **Location:** HDR-B1: S5 **Depth:** 12.5'-14.0' **Sample Number:** 62177

■ **Location:** HDR-B1: S11 **Depth:** 35.0'-36.5' **Sample Number:** 62181

▲ **Location:** HDR-B1: S14 **Depth:** 50.0'-51.5' **Sample Number:** 62182

◆ **Location:** HDR-B1: S3 **Depth:** 7.5'-9.0' **Sample Number:** 62175

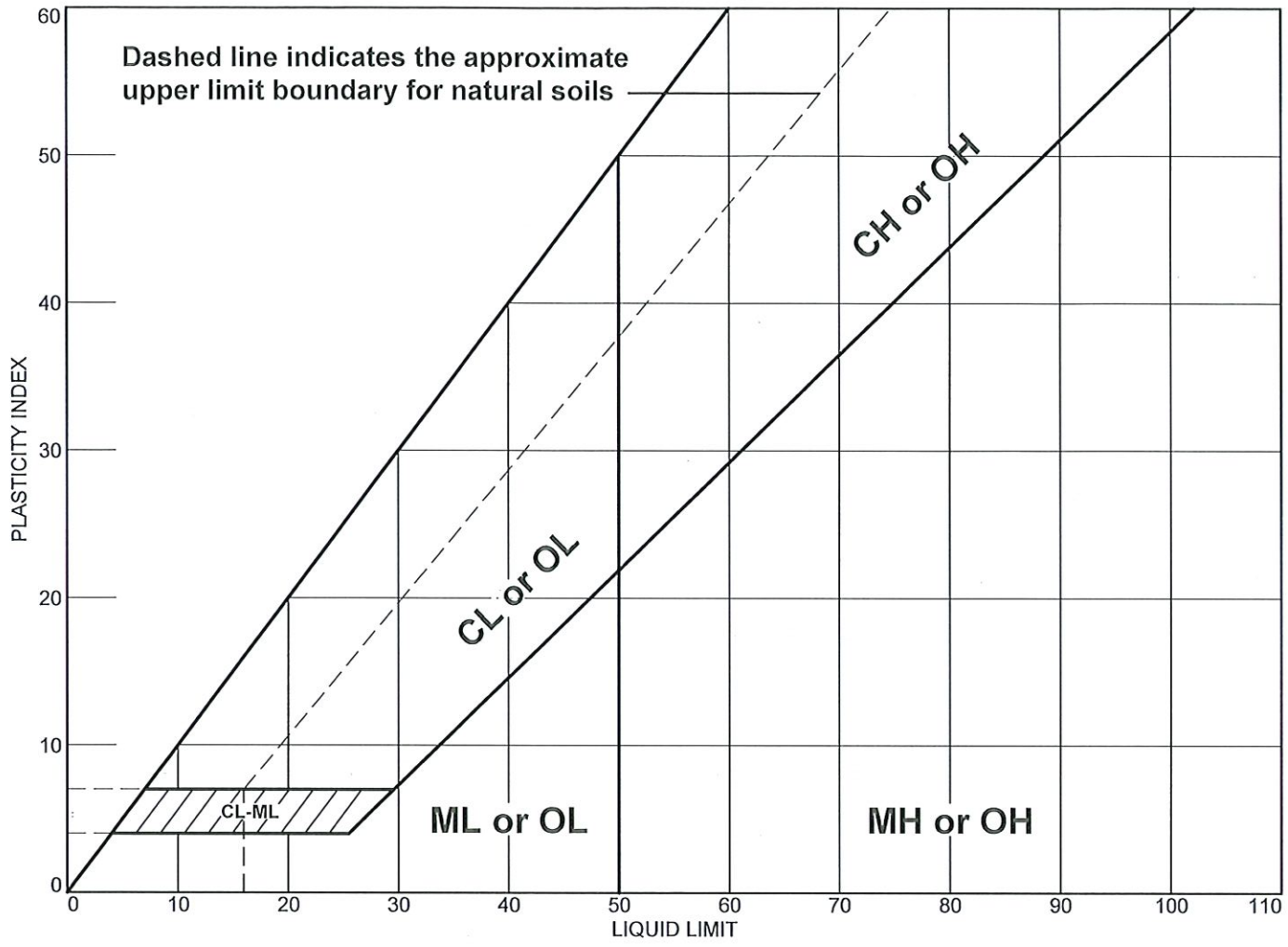
GS GULF SHORE EXPLORATION AND TESTING

Remarks:

Figure

Tested By: BGS **Checked By:** JML

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●		NV	NP	NP		13	

Project No. 21-105 **Client:** HDR, Inc.

Project: Bellotta Weir Modification

● **Location:** HDR-B1: S7 **Depth:** 17.5'-19.0' **Sample Number:** 62178

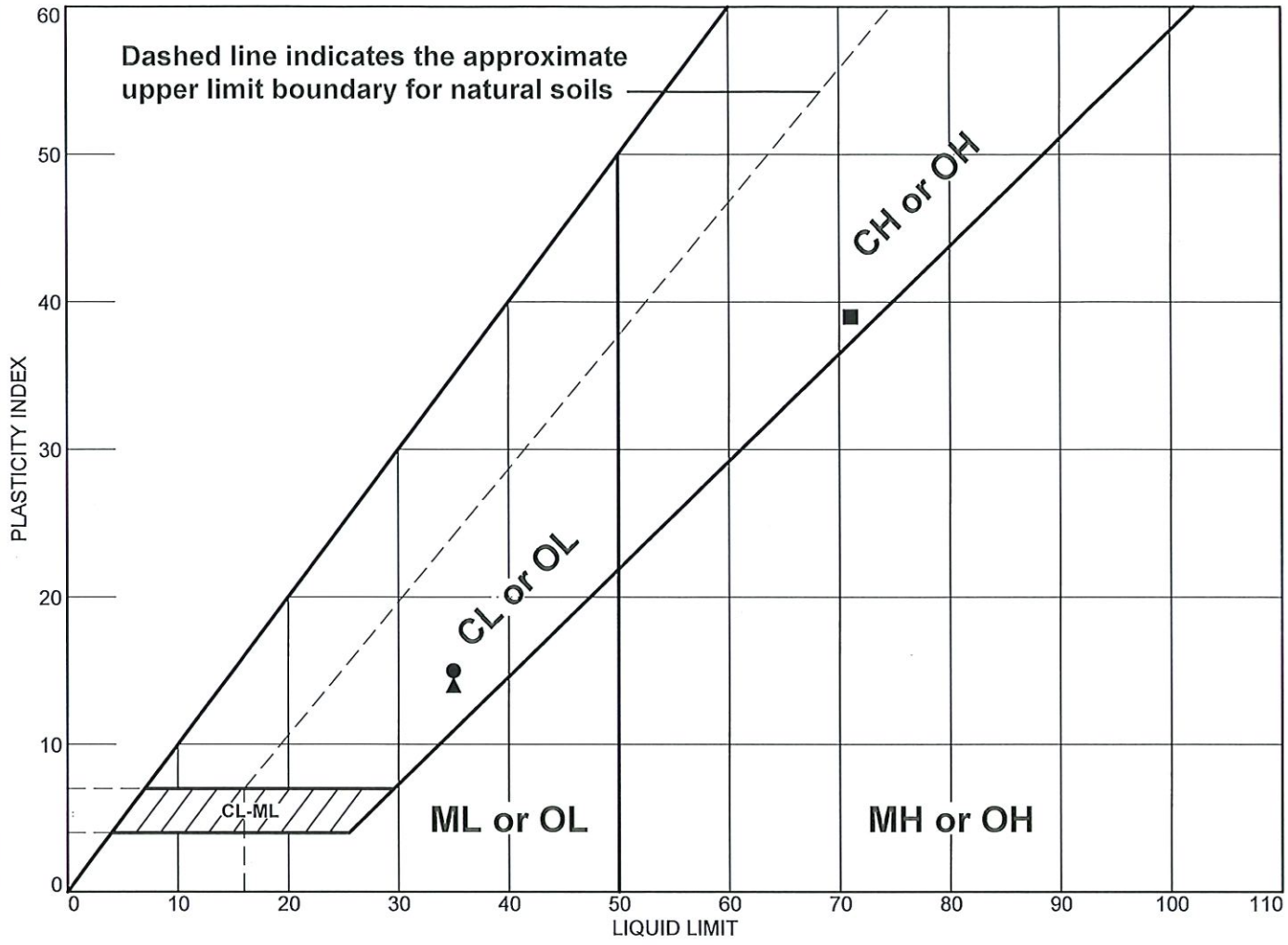
Remarks:

GULF SHORE EXPLORATION AND TESTING

Figure

Tested By: BGS **Checked By:** JML

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●		35	20	15			
■		71	32	39		74	
▲		35	21	14			

Project No. 21-105 **Client:** HDR, Inc.

Project: Bellotta Weir Modification

● **Location:** HDR-B2: S4 **Depth:** 10.0'-11.5' **Sample Number:** 62185

■ **Location:** HDR-B2: S15 **Depth:** 55.0'-56.5' **Sample Number:** 62190

▲ **Location:** HDR-B2: S6 **Depth:** 15.0'-16.5' **Sample Number:** 62186

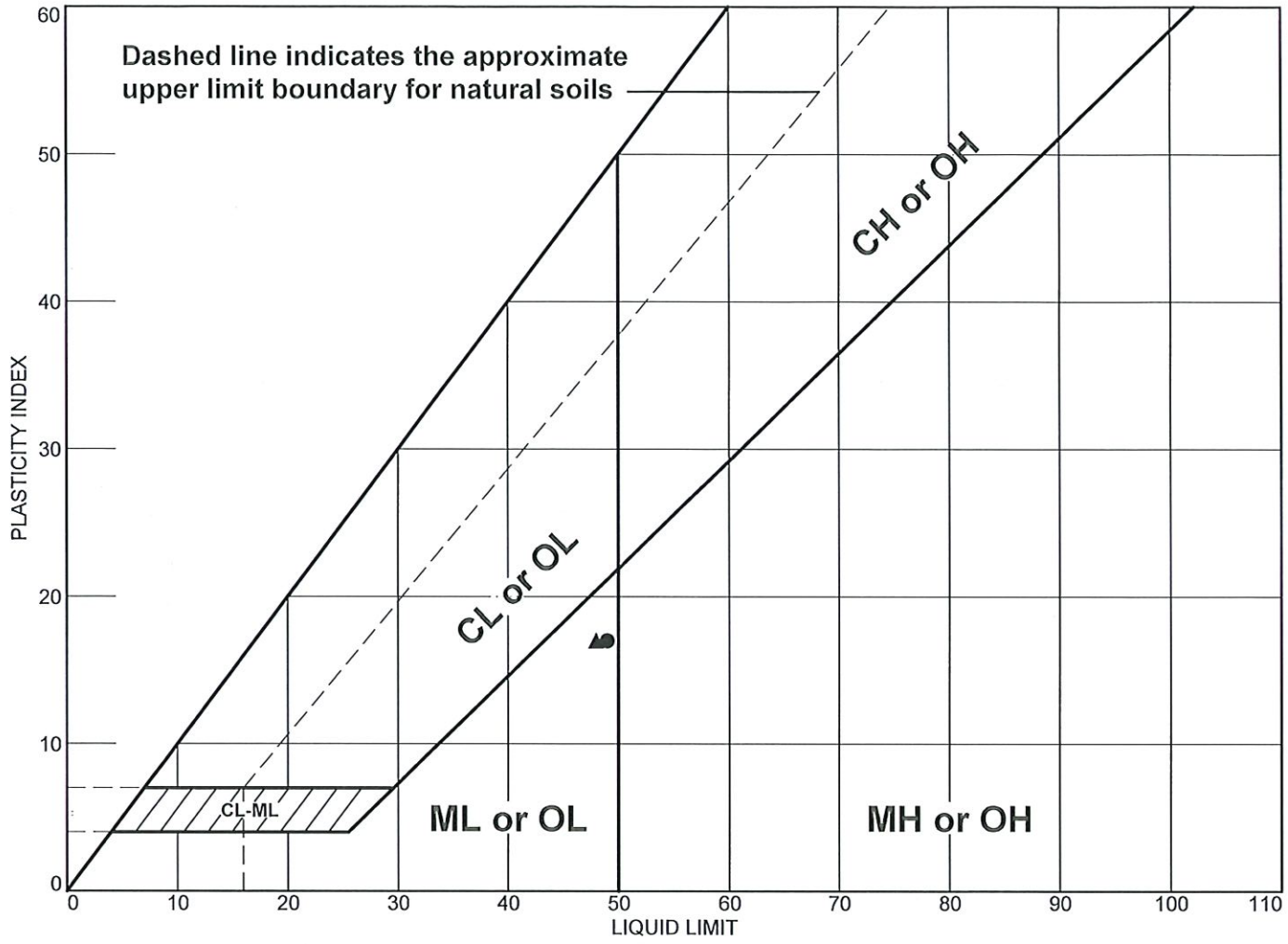
Remarks:

GS GULF SHORE EXPLORATION AND TESTING

Figure

Tested By: BGS **Checked By:** JML

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●		49	32	17			
■		NV	NP	NP			
▲		48	31	17			

Project No. 21-105 **Client:** HDR, Inc.

Project: Bellota Weir Modification

● **Location:** HDR-B3: S11 **Depth:** 40.0'-41.5' **Sample Number:** 62196

■ **Location:** HDR-B3: S2 **Depth:** 5.0'-6.5' **Sample Number:** 62191

▲ **Location:** HDR-B3: S14 **Depth:** 55.0'-56.5' **Sample Number:** 62197

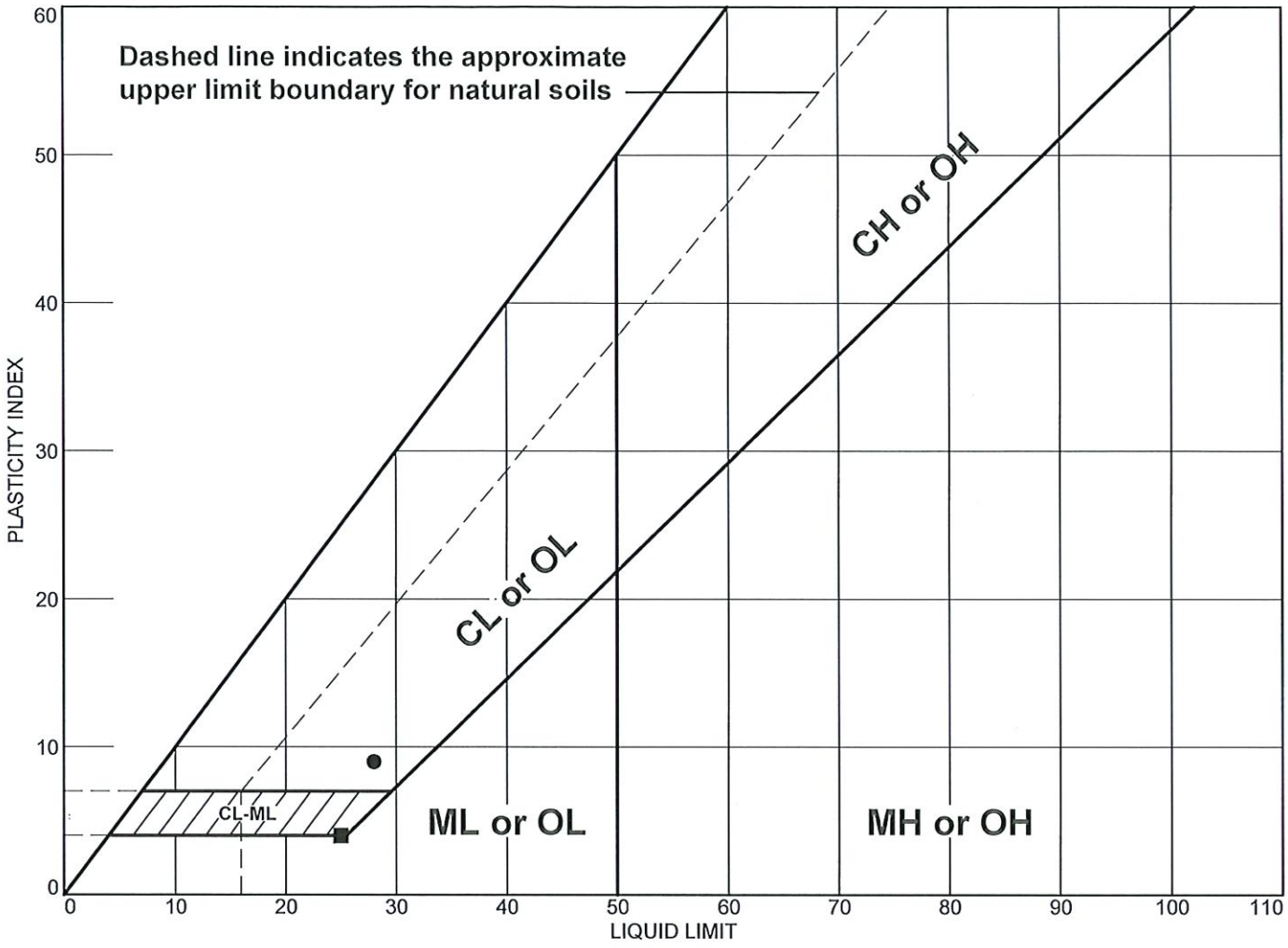
GS GULF SHORE EXPLORATION AND TESTING

Remarks:

Figure

Tested By: BGS **Checked By:** JML

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●		28	19	9			
■		25	21	4			

Project No. 21-105 **Client:** HDR, Inc.

Project: Bellotta Weir Modification

● **Location:** HDR-B4: S3 **Depth:** 7.5'-9.0' **Sample Number:** 62200

■ **Location:** HDR-B4: S7 **Depth:** 20.0'-21.5' **Sample Number:** 62203

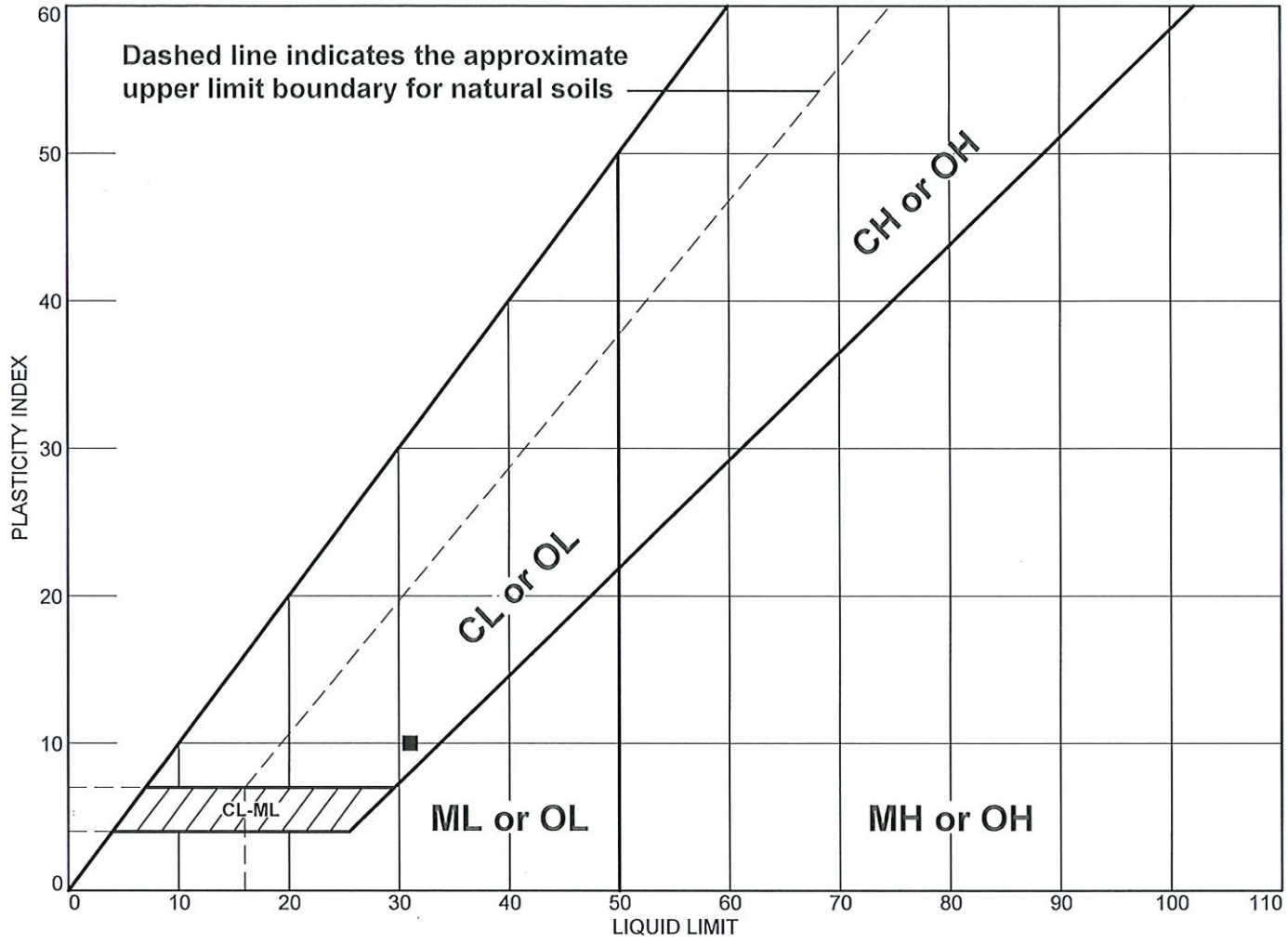
Remarks:

GULF SHORE EXPLORATION AND TESTING

Figure

Tested By: BGS **Checked By:** JML

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●		NV	NP	NP			
■		31	21	10			

Project No. 21-105 **Client:** HDR, Inc.

Project: Bellotta Weir Modification

● **Location:** HDR-B5: S2 **Depth:** 5.0'-6.5' **Sample Number:** 62209

■ **Location:** HDR-B5: S7 **Depth:** 20.0'-21.5' **Sample Number:** 62210

Remarks:

GULF SHORE EXPLORATION AND TESTING

Figure

Tested By: BGS **Checked By:** JML

MOISTURE CONTENT & UNIT WEIGHT TEST RESULTS

Sample Identification	Depth, ft.	Wet Unit Weight, lb/ft.^{3}	Dry Unit Weight, lb/ft.^{3}	Moisture Content, %
HDR-B1: S3	7.5'-9.0'			15.0
HDR-B1: S4	10.0'-11.5'	122.7	106.6	15.1
HDR-B1: S7	17.5'-19.0'			27.6
HDR-B1: S9	25.0'-26.5'			16.9
HDR-B1: S11	35.0'-36.5'			28.1
HDR-B2: S2	5.0'-6.5'			8.1
HDR-B2: S4	10.0'-11.5'	104.9	90.3	16.2
HDR-B2: S6	15.0'-16.5'	112.6	97.3	15.7
HDR-B3: S6	15.0'-16.5'	114.0	98.9	15.2

Test Method: ASTM D2216, ASTM D2937

PROJECT NUMBER:	21-105	February 18, 2021	Bellota Weir Modification

GULF SHORE EXPLORATION AND TESTING
3362 Fitzgerald Road
Rancho Cordova, CA 95742
Phone: (916) 939-4117
FAX: (916) 635-4315

MOISTURE CONTENT & UNIT WEIGHT TEST RESULTS

Sample Identification	Depth, ft.	Wet Unit Weight, lb/ft.^{3}	Dry Unit Weight, lb/ft.^{3}	Moisture Content, %
HDR-B4: S4	10.0'-11.5'	111.8	98.0	14.1
HDR-B5: S7	20.0'-21.5'	117.9	89.4	31.9
HDR-B5: S11	40.0'-41.5'	114.6	71.9	59.4
HDR-B5: S14	52.5'-54.0'	124.9	89.7	39.2

Test Method: ASTM D2216, ASTM D2937

PROJECT NUMBER:	21-105	February 18, 2021	Bellota Weir Modification

GULF SHORE EXPLORATION AND TESTING
3362 Fitzgerald Road
Rancho Cordova, CA 95742
Phone: (916) 939-4117
FAX: (916) 635-4315

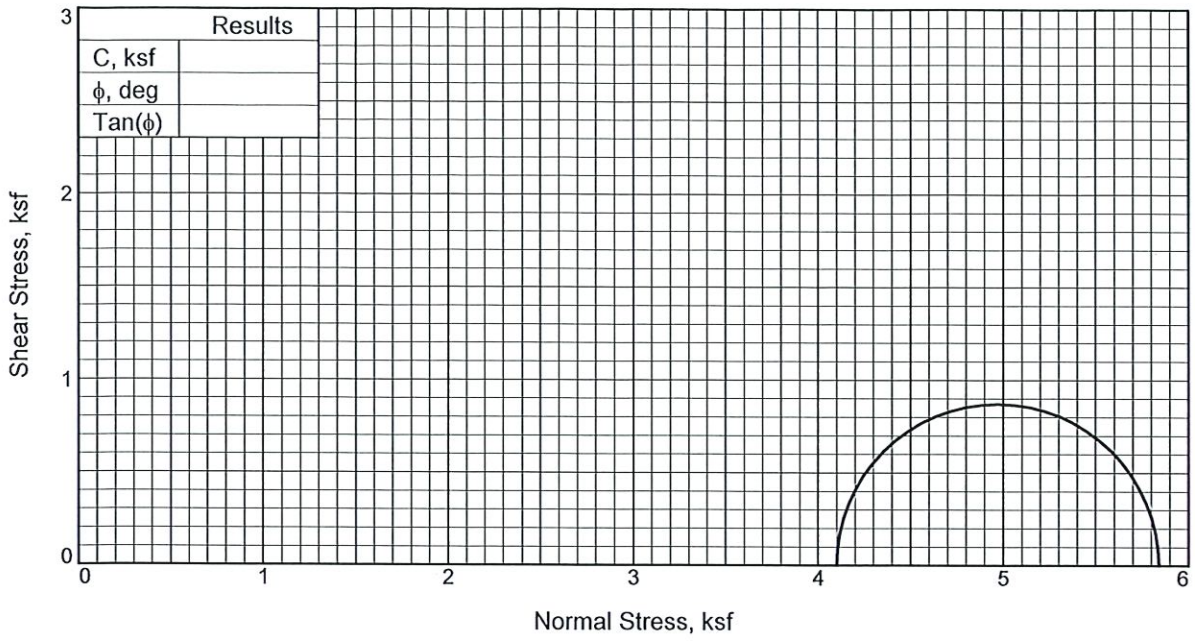
MOISTURE AND ORGANIC CONTENT TEST RESULTS

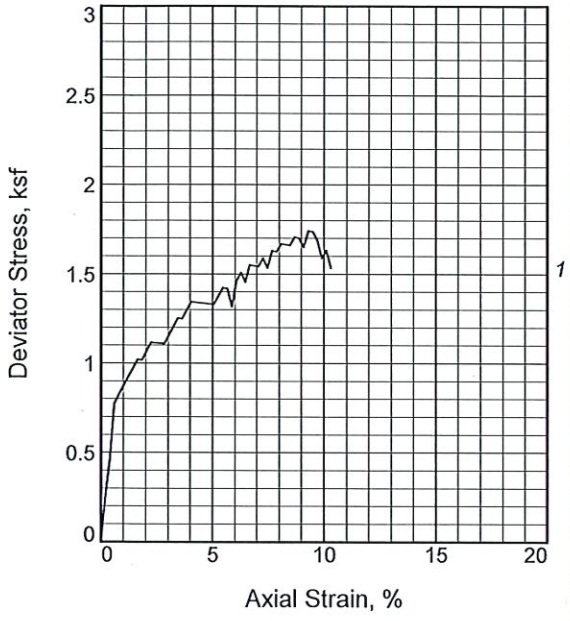
Sample Identification	Depth, ft.	Organic Content, %	Moisture Content, %
HDR-B4: S1	2.5'-4.0'	1.5	3.8

Test Method: ASTM D2974

PROJECT NUMBER:	21-105	February 18, 2021	Bellota Weir Modification

GULF SHORE EXPLORATION AND TESTING
3362 Fitzgerald Road
Rancho Cordova, CA 95742
Phone: (916) 939-4117
FAX: (916) 635-4315





Sample No.		1
Initial	Water Content, %	41.9
	Dry Density, pcf	78.9
	Saturation, %	99.6
	Void Ratio	1.1354
	Diameter, in.	1.88
	Height, in.	4.95
At Test	Water Content, %	42.1
	Dry Density, pcf	78.9
	Saturation, %	100.0
	Void Ratio	1.1354
	Diameter, in.	1.88
	Height, in.	4.95
Strain rate, in./min.		0.008
Back Pressure, psi		0.00
Cell Pressure, psi		28.47
Fail. Stress, ksf		1.74
Ult. Stress, ksf		
σ_1 Failure, ksf		5.84
σ_3 Failure, ksf		4.10

Type of Test:
Unconsolidated Undrained

Sample Type: -

Description:

Assumed Specific Gravity= 2.70

Remarks:

Figure _____

Client: HDR, Inc.

Project: Bellotta Weir Modification

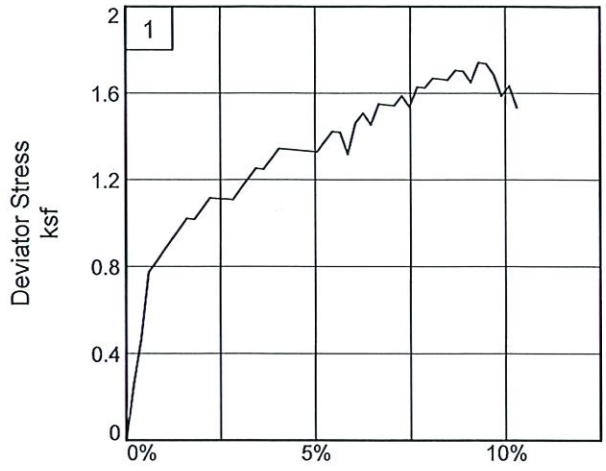
Location: HDR-B4: S14

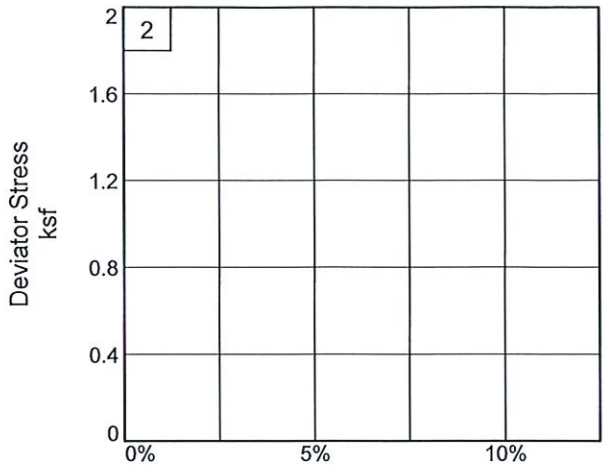
Sample Number: 62205 **Depth:** 50.0'-51.5'

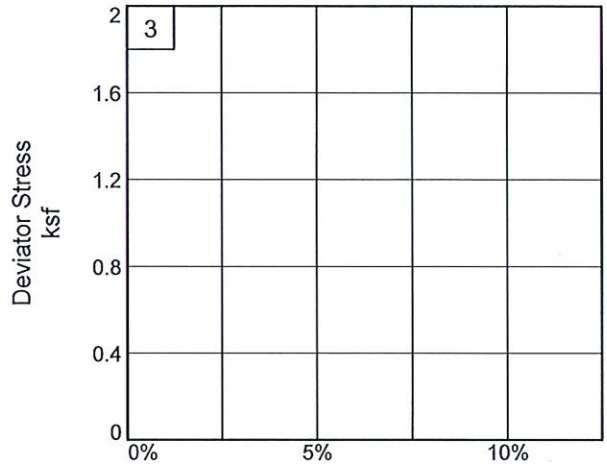
Proj. No.: 21-105 **Date Sampled:** -

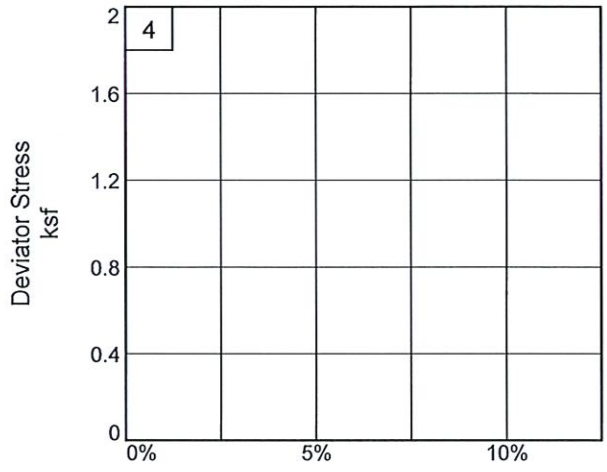
GS GULF SHORE EXPLORATION AND TESTING

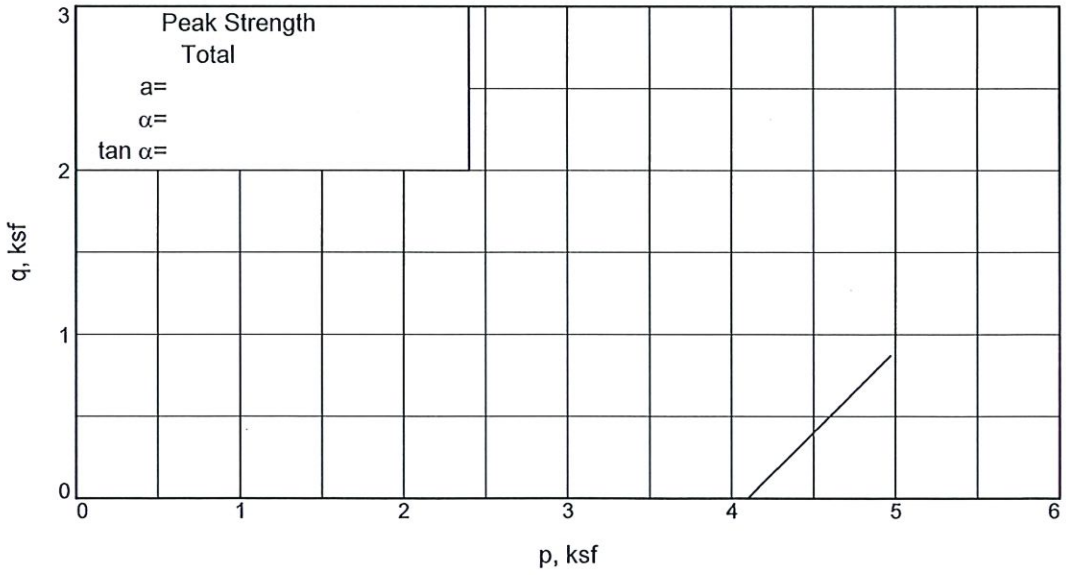
Tested By: MPW **Checked By:** JML







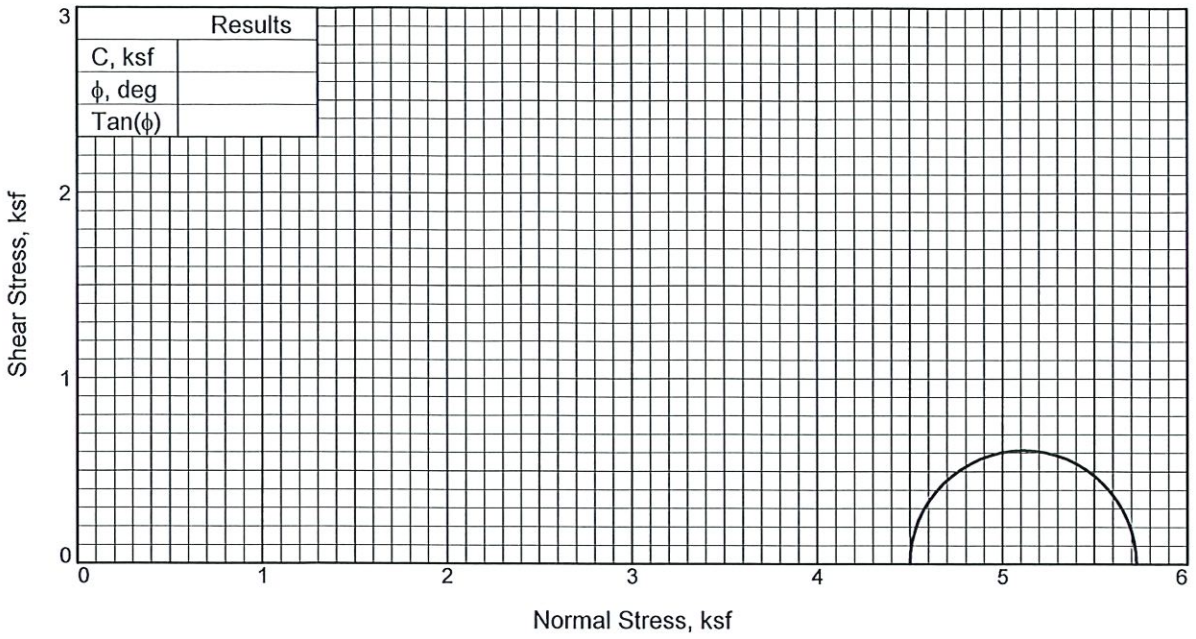


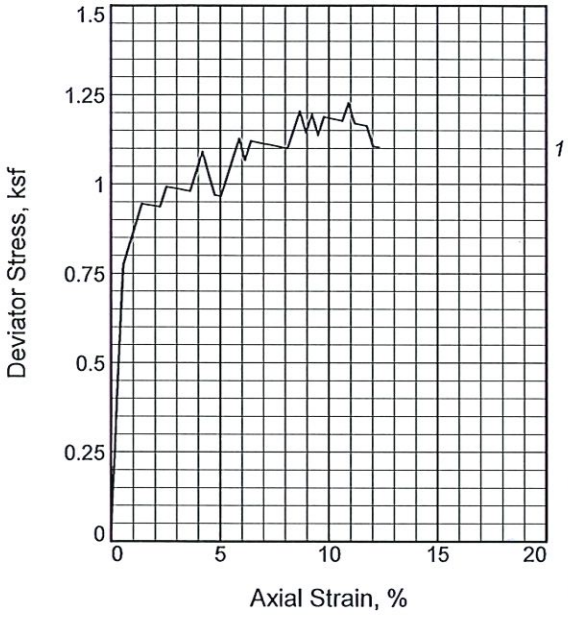


Client: HDR, Inc.
Project: Bellotta Weir Modification
Location: HDR-B4: S14 **Depth:** 50.0'-51.5' **Sample Number:** 62205
Project No.: 21-105 **Figure** _____

Gulf Shore Construction Services, LLC

Tested By: MPW **Checked By:** JML





Sample No.		1
Initial	Water Content, %	32.3
	Dry Density, pcf	89.9
	Saturation, %	99.6
	Void Ratio	0.8750
	Diameter, in.	1.75
	Height, in.	3.58
At Test	Water Content, %	32.4
	Dry Density, pcf	89.9
	Saturation, %	100.0
	Void Ratio	0.8750
	Diameter, in.	1.75
	Height, in.	3.58
Strain rate, in./min.		0.008
Back Pressure, psi		0.00
Cell Pressure, psi		31.25
Fail. Stress, ksf		1.23
Ult. Stress, ksf		
σ_1 Failure, ksf		5.73
σ_3 Failure, ksf		4.50

Type of Test:
Unconsolidated Undrained

Sample Type: -

Description:

Assumed Specific Gravity= 2.70

Remarks:

Figure _____

Client: HDR, Inc.

Project: Bellotta Weir Modification

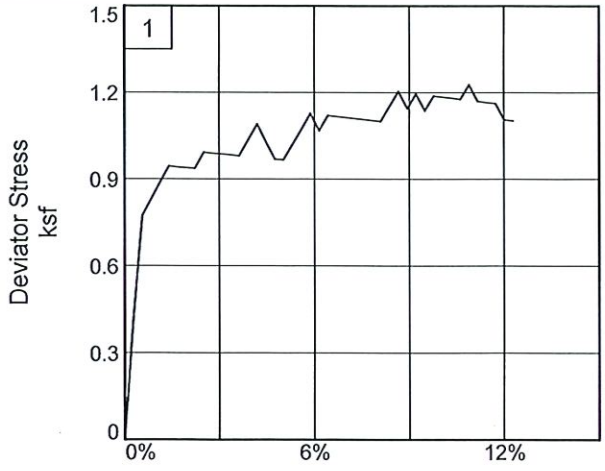
Location: HDR-B4: S16

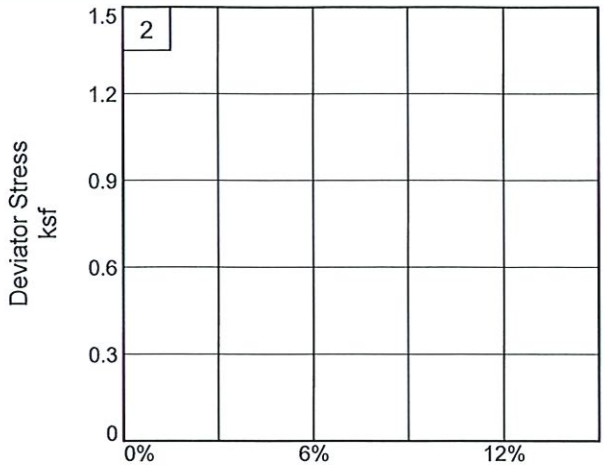
Sample Number: 62206 **Depth:** 60.0'-61.5'

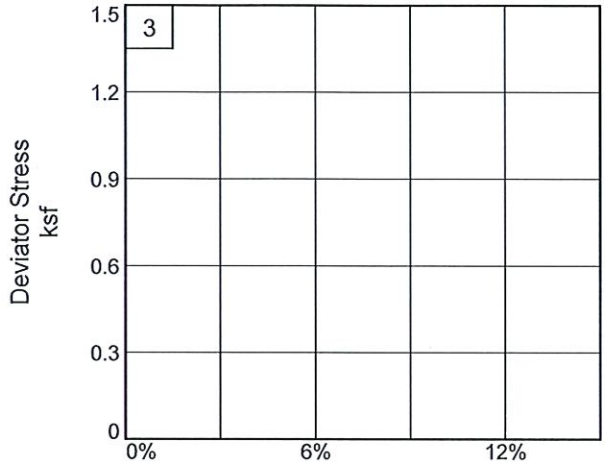
Proj. No.: 21-105 **Date Sampled:** -

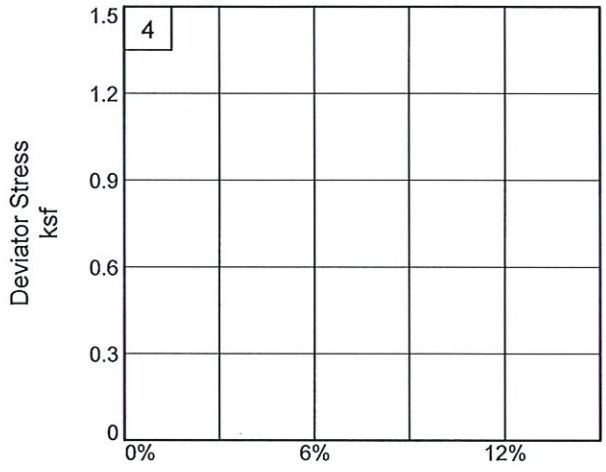
GS GULF SHORE EXPLORATION AND TESTING

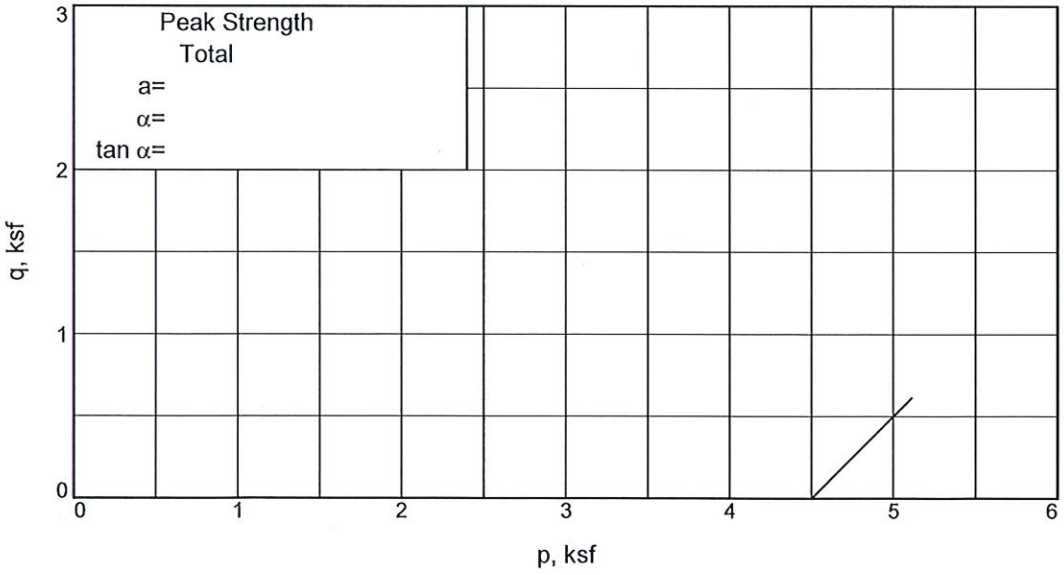
Tested By: MPW **Checked By:** JML









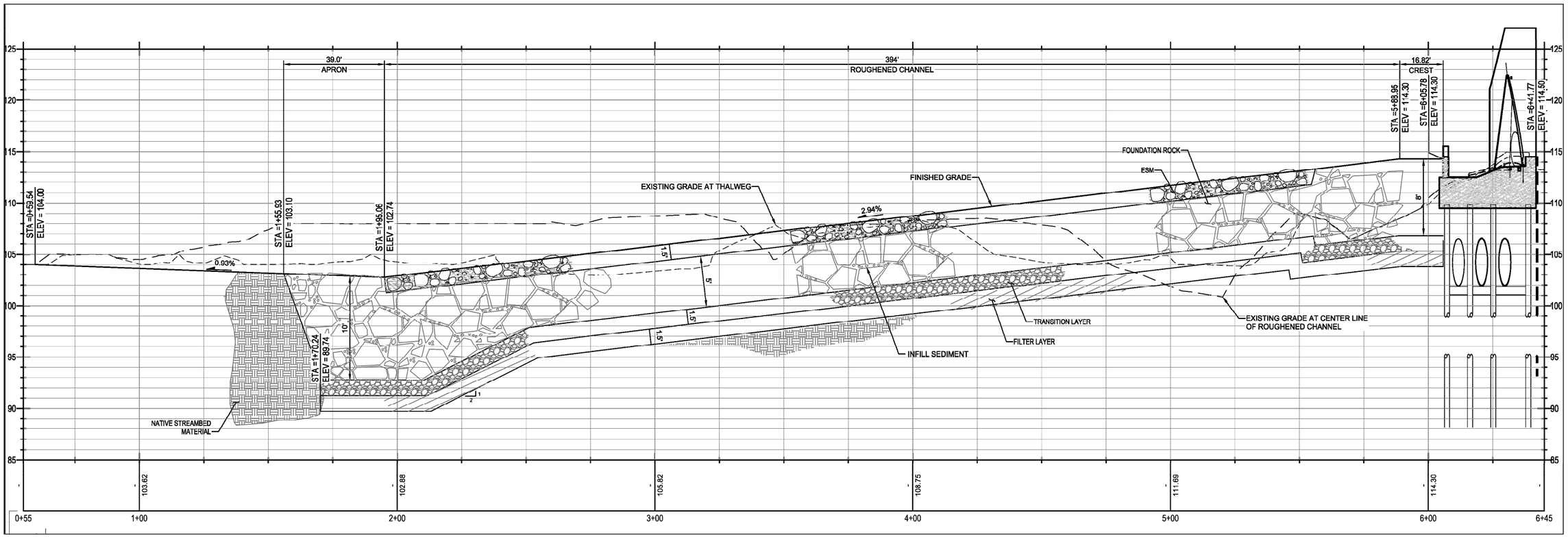


Client: HDR, Inc.
Project: Bellotta Weir Modification
Location: HDR-B4: S16 **Depth:** 60.0'-61.5' **Sample Number:** 62206
Project No.: 21-105 **Figure** _____

Gulf Shore Construction Services, LLC

Tested By: MPW **Checked By:** JML

Appendix E. Weir Seepage and Stability Analysis



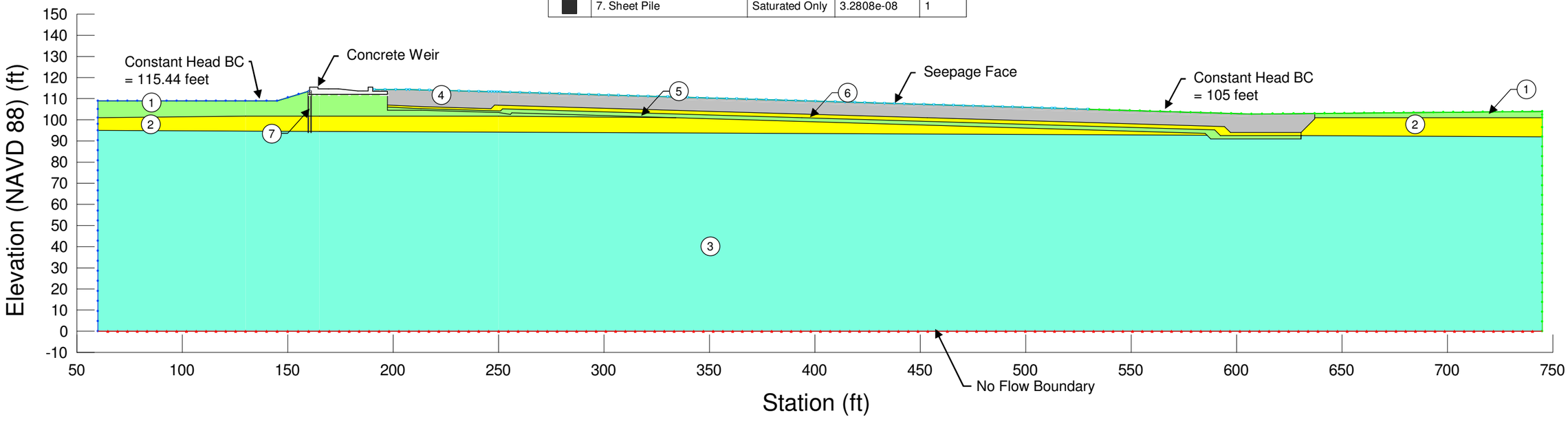
NOTES: Source: Sheet RI201 of Bellota Weir Modifications Project, 90% design submittal

Bellota Weir Modification Project

Bellota Weir Roughened Channel Downstream Profile

Feb 2022 FIGURE E-1

Color	Name	Material Model	Sat Kx (ft/sec)	Ky'/Kx' Ratio
	1. Alluvium (Upper Unit) SM	Saturated Only	3.28084e-06	0.25
	2. Alluvium (Lower Unit) SP	Saturated Only	0.000328084	0.25
	3. Older Alluvium ML/SM	Saturated Only	3.2808e-07	0.25
	4. Foundation Rocks Cobbles	Saturated Only	0.328084	1
	5. Filter SP	Saturated Only	0.0164042	1
	6. Transition SM	Saturated Only	3.28084e-06	0.25
	7. Sheet Pile	Saturated Only	3.2808e-08	1



NOTES: BC = Boundary Condition

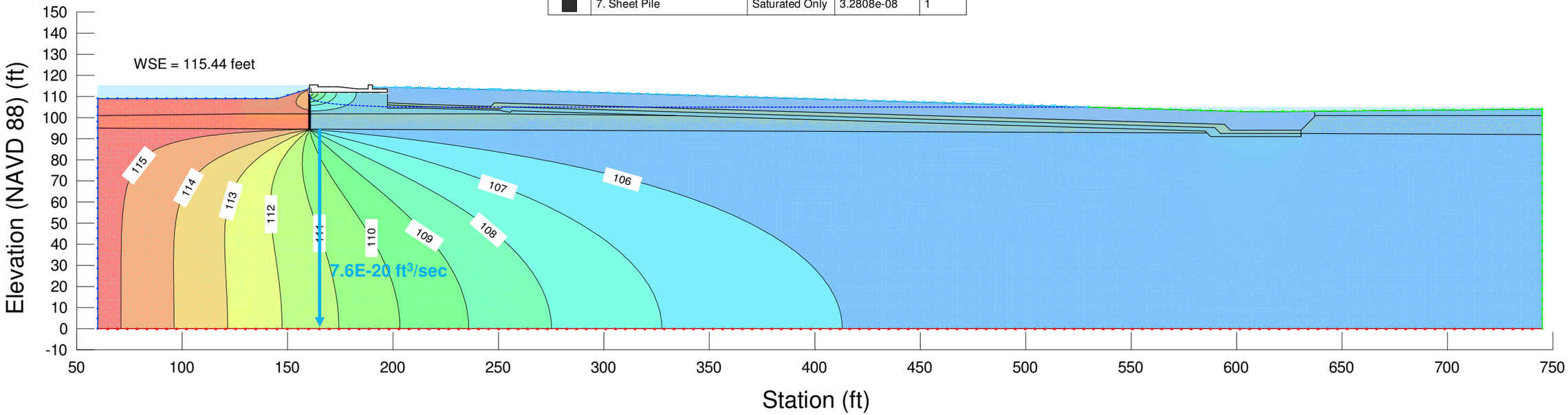
Bellota Weir Modification Project

Steady State Seepage

Bellota Weir Steady State Seepage Model D/S WSE 105 ft

Feb 2022 FIGURE E-2

Color	Name	Material Model	Sat Kx (ft/sec)	Ky'/Kx' Ratio
	1. Alluvium (Upper Unit) SM	Saturated Only	3.28084e-06	0.25
	2. Alluvium (Lower Unit) SP	Saturated Only	0.000328084	0.25
	3. Older Alluvium ML/SM	Saturated Only	3.2808e-07	0.25
	4. Foundation Rocks Cobbles	Saturated Only	0.328084	1
	5. Filter SP	Saturated Only	0.0164042	1
	6. Transition SM	Saturated Only	3.28084e-06	0.25
	7. Sheet Pile	Saturated Only	3.2808e-08	1



NOTES:

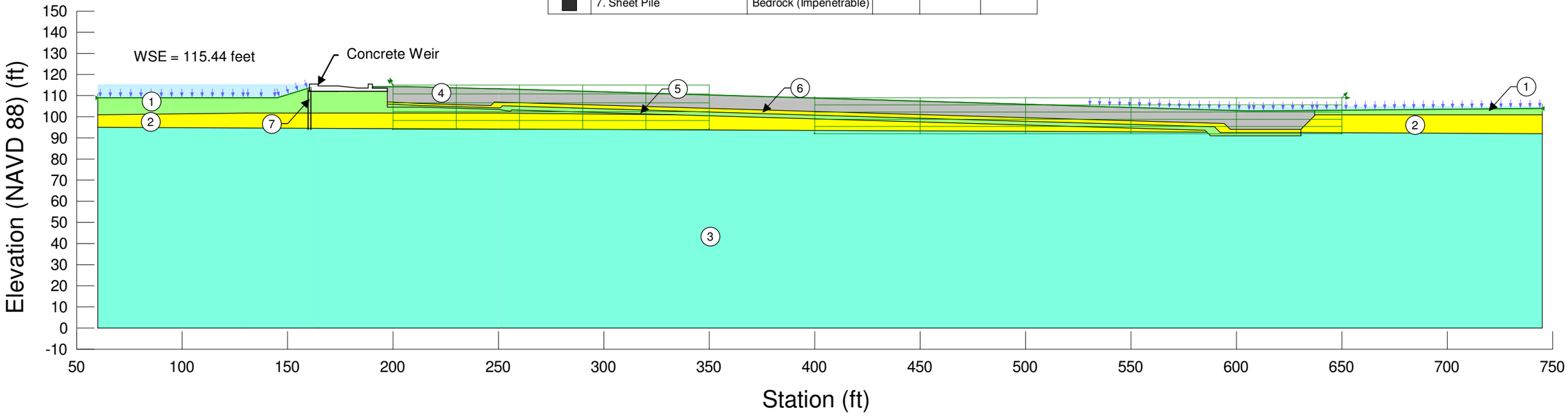
Bellota Weir Modification Project

Steady State Seepage

Bellota Weir Steady State Seepage Result D/S WSE 105 ft

Feb 2022 FIGURE E-3

Color	Name	Material Model	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)
	1. Alluvium (Upper Unit) SM	Mohr-Coulomb	120	50	30
	2. Alluvium (Lower Unit) SP	Mohr-Coulomb	120	0	32
	3. Older Alluvium ML/SM	Mohr-Coulomb	120	100	30
	4. Foundation Rocks Cobbles	Mohr-Coulomb	140	0	40
	5. Filter SP	Mohr-Coulomb	120	0	33
	6. Transition SM	Mohr-Coulomb	120	0	33
	7. Sheet Pile	Bedrock (Impenetrable)			



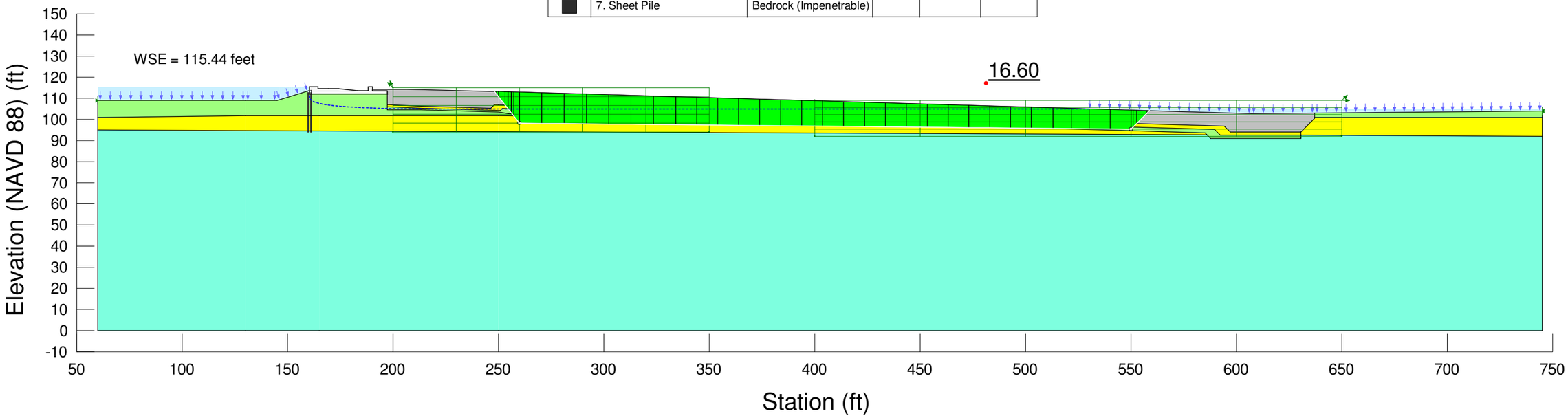
NOTES: Blocks used for Slip Surface search

Bellota Weir Modification Project

Bellota Weir Steady State Stability Model D/S WSE 105 ft

Feb 2022 FIGURE E-4

Color	Name	Material Model	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)
	1. Alluvium (Upper Unit) SM	Mohr-Coulomb	120	50	30
	2. Alluvium (Lower Unit) SP	Mohr-Coulomb	120	0	32
	3. Older Alluvium ML/SM	Mohr-Coulomb	120	100	30
	4. Foundation Rocks Cobbles	Mohr-Coulomb	140	0	40
	5. Filter SP	Mohr-Coulomb	120	0	33
	6. Transition SM	Mohr-Coulomb	120	0	33
	7. Sheet Pile	Bedrock (Impenetrable)			



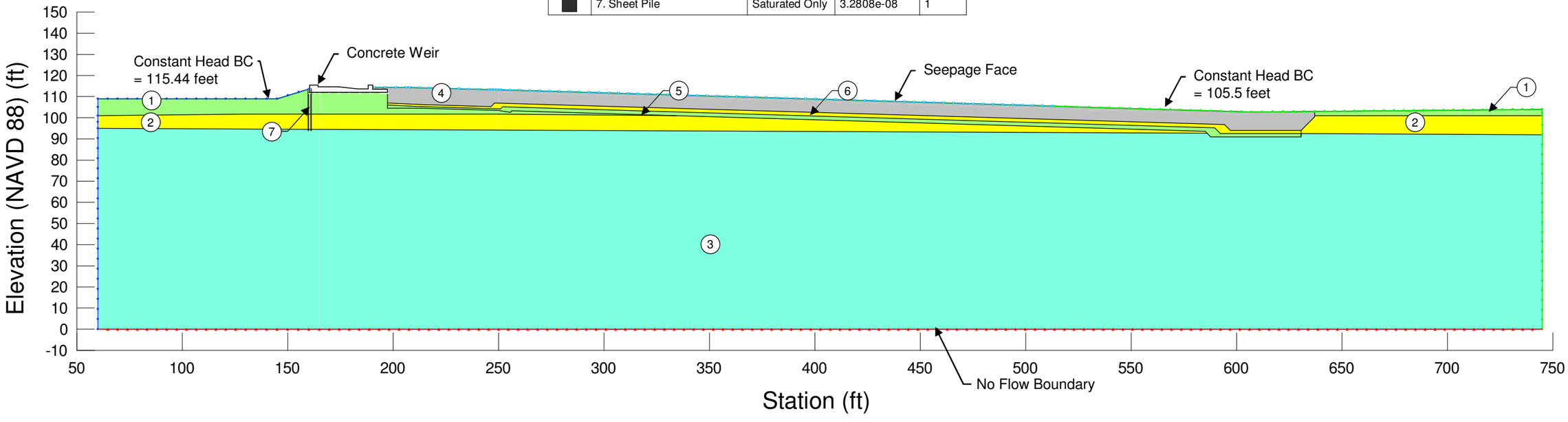
NOTES:

Bellota Weir Modification Project

Bellota Weir Steady State Stability Result D/S WSE 105 ft

Feb 2022 FIGURE E-5

Color	Name	Material Model	Sat Kx (ft/sec)	Ky'/Kx' Ratio
	1. Alluvium (Upper Unit) SM	Saturated Only	3.28084e-06	0.25
	2. Alluvium (Lower Unit) SP	Saturated Only	0.000328084	0.25
	3. Older Alluvium ML/SM	Saturated Only	3.2808e-07	0.25
	4. Foundation Rocks Cobbles	Saturated Only	0.328084	1
	5. Filter SP	Saturated Only	0.0164042	1
	6. Transition SM	Saturated Only	3.28084e-06	0.25
	7. Sheet Pile	Saturated Only	3.2808e-08	1



NOTES: BC = Boundary Condition

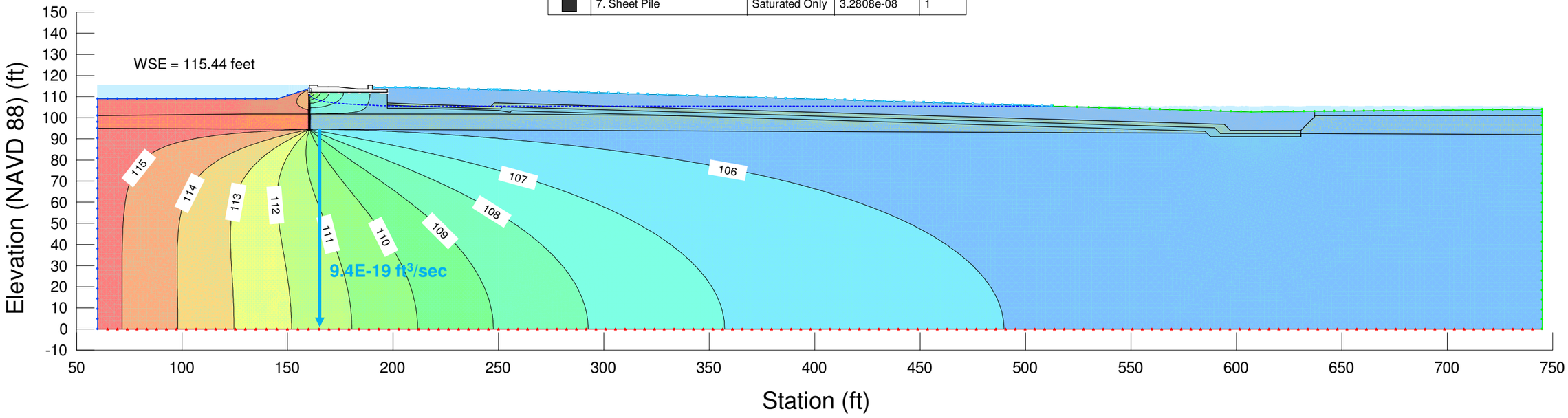
Bellota Weir Modification Project

Steady State Seepage

Bellota Weir Steady State Seepage Model D/S WSE 105.5 ft

Feb 2022 FIGURE E-6

Color	Name	Material Model	Sat Kx (ft/sec)	Ky'/Kx' Ratio
	1. Alluvium (Upper Unit) SM	Saturated Only	3.28084e-06	0.25
	2. Alluvium (Lower Unit) SP	Saturated Only	0.000328084	0.25
	3. Older Alluvium ML/SM	Saturated Only	3.2808e-07	0.25
	4. Foundation Rocks Cobbles	Saturated Only	0.328084	1
	5. Filter SP	Saturated Only	0.0164042	1
	6. Transition SM	Saturated Only	3.28084e-06	0.25
	7. Sheet Pile	Saturated Only	3.2808e-08	1



NOTES:

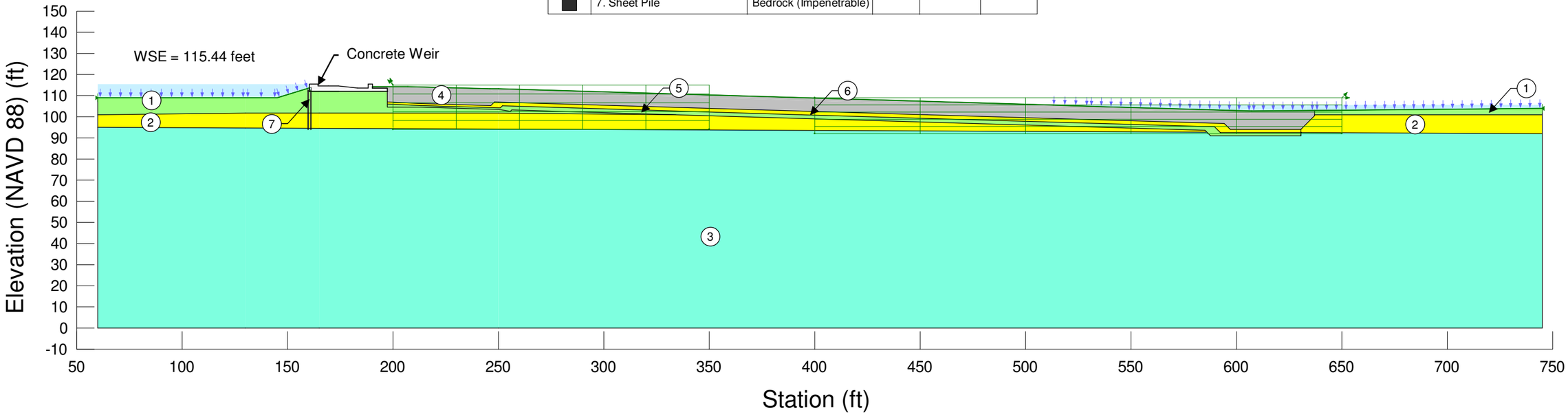
Bellota Weir Modification Project

Steady State Seepage

Bellota Weir Steady State Seepage Result D/S WSE 105.5 ft

Feb 2022 FIGURE E-7

Color	Name	Material Model	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)
	1. Alluvium (Upper Unit) SM	Mohr-Coulomb	120	50	30
	2. Alluvium (Lower Unit) SP	Mohr-Coulomb	120	0	32
	3. Older Alluvium ML/SM	Mohr-Coulomb	120	100	30
	4. Foundation Rocks Cobbles	Mohr-Coulomb	140	0	40
	5. Filter SP	Mohr-Coulomb	120	0	33
	6. Transition SM	Mohr-Coulomb	120	0	33
	7. Sheet Pile	Bedrock (Impenetrable)			



NOTES: Blocks used for Slip Surface search

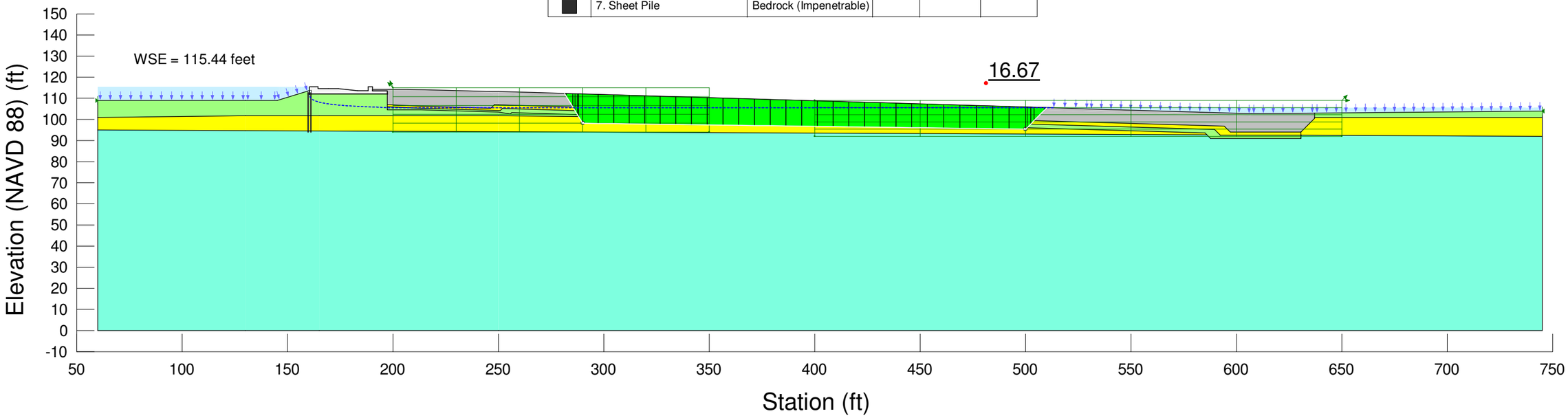
Bellota Weir Modification Project

Bellota Weir Steady State Stability Model D/S WSE 105.5 ft

Feb 2022

FIGURE E-8

Color	Name	Material Model	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)
	1. Alluvium (Upper Unit) SM	Mohr-Coulomb	120	50	30
	2. Alluvium (Lower Unit) SP	Mohr-Coulomb	120	0	32
	3. Older Alluvium ML/SM	Mohr-Coulomb	120	100	30
	4. Foundation Rocks Cobbles	Mohr-Coulomb	140	0	40
	5. Filter SP	Mohr-Coulomb	120	0	33
	6. Transition SM	Mohr-Coulomb	120	0	33
	7. Sheet Pile	Bedrock (Impenetrable)			



NOTES:

Bellota Weir Modification Project

Bellota Weir Steady State Stability Result D/S WSE 105.5 ft

Feb 2022 FIGURE E-9

Appendix F. Filter Analysis

Base Material

Determine the gradation curves of the base soil. Use enough samples to define the range of grain size for the base soil. Design the filter gradation based on the base soil that requires the smallest D_{15F} size. If soil has particles larger than the #4 sieve, an adjusted gradation is calculated. Input values below for the base soil (original) gradation (in red):

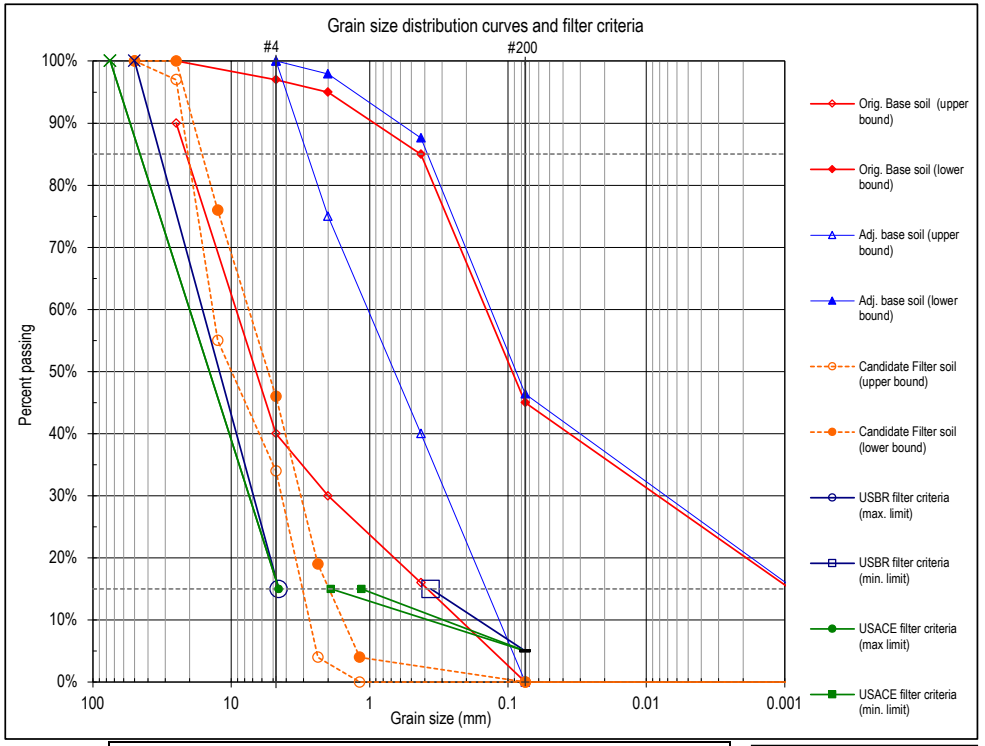
Project: Bellota Weir
Base Soil to Filter

Name: SRS
Date: 12/23/2021

Particle size (mm)	Sieve #	Base soil (original), % passing (upper bound)	(lower bound)	Adjusted gradation, % passing upper bound	lower bound
25	-	90.0%	100.0%		
19.05	-				
12.5	-				
9.5	-				
4.75	4	40.0%	97.0%	100.0%	100.0%
4.00	5				
3.35	6				
2.80	7				
2.36	8				
2.00	10	30.0%	95.0%	75.0%	97.9%
1.70	12				
1.40	14				
1.18	16				
1.00	18				
0.850	20				
0.710	25				
0.600	30				
0.500	35				
0.425	40	16.0%	85.0%	40.0%	87.6%
0.300	50				
0.250	60				
0.212	70				
0.180	80				
0.150	100				
0.125	120				
0.106	140				
0.090	170				
0.075	200	0.0%	45.0%	0.0%	46.4%
0.053	270				
0.037	-				
0.017	-				
0.009	-				
0.005	-				
0.002	-				
0.001	-				
0.0001	-	0.0%	0.0%	0.0%	0.0%

Required entry values are defined below. *

Average % passing #200 after regarding (if any) = A = 23.2%



Properties of base soil	Upper bound	Lower bound
D_{85B} of original base soil =	21.175 mm	0.425 mm
D_{85B} of adjusted base soil =	2.827 mm	0.381 mm
D_{15B} of original base soil =	0.3813 mm	0.0009 mm
D_{15B} of adjusted base soil =	0.1437 mm	0.0009 mm
D_{60B} of adjusted base soil =	1.030 mm	0.133 mm
D_{10B} of adjusted base soil =	0.1157 mm	0.0004 mm
C_u of adjusted base soil =	8.9	319.1

Fixed points on graph:
- <5% passing #200
- 50mm max. grain size (USBR)
- 75mm max. grain size (USACE)

Filter Material

Filter criteria required by the USBR as published in *Design Standards - Embankment Dams No. 13 (1994):*

D_{85B} used in filter design	1.604
Average Passing #200 sieve of base soil (from adjusted gradation)	23.2%
Base soil category	3
Base soil description	Silty and clayey sands and gravels
Filter criteria (mm)	Maximum: $D_{15F} \leq$ 4.54 To ensure sufficient permeability: Minimum: $D_{15F} \geq$ 0.36
Maximum particle size of filter (mm)	50
Maximum % passing # 200 sieve	5%
PI of material passing #40	0 when tested in accordace with USBR 5360, Earth Manual, on material passing #40

Filter criteria required by the US Army Corps of Engineers as published in EM 1110-2-2300 (31 Jul 94):

D_{85B} used in filter design	1.604
Average Passing #200 sieve of base soil (from adjusted gradation)	23.2%
Base soil category	3
Filter criteria (mm)	Maximum: $D_{15F} \leq$ 4.54 to 4.54 To ensure sufficient permeability: Minimum: $D_{15F} \geq$ 1.14 to 1.91
Maximum particle size of filter (mm)	75
Maximum % passing # 200 sieve	5%
PI of material passing #40	0 when tested in accordance with EM 1110-2-1906

**If the base soil is in category 4, use the lower of the two 'max. D_{15F}' values when the filter is beneath riprap subject to wave action or beneath drains which may be subject to violent surging and/or vibration.

*Required entry values for base soil & candidate filter gradations:

1. Particle size for 100% passing.
2. % Passing the #4 sieve.
3. % Passing the #200 sieve.
4. One point in the 85% - 90% range and another point in the 80% - 85% range, or the 85% point.
5. One point in the 15% - 20% range and another point in the 10% - 15% range, or the 15% point.
6. No duplicate entries; if D100<#4, enter 101% for #4 and 100% for appropriate size.

USBR filter gradation limits:

Maximum limit	
Grain size (mm)	% Passing
50.00	100.0%
4.54	15.0%

Minimum limit	
Grain size (mm)	% Passing
0.36	15.0%
0.075	5.0%

USACE filter gradation limits:

Maximum limit	
Grain size (mm)	% Passing
75.00	100.0%
4.54	15.0%
4.54	15.0%

Minimum limit	
Grain size (mm)	% Passing
1.14	15.0%
1.91	15.0%
0.075	5.0%

Candidate filter soil gradation. Values shown in red in the left column, and all values in the two right columns, can be changed.

Particle size *mm*	Sieve #	% Passing (upper bound)	% Passing (lower bound)
150.0	-		
100.0	-		
90.0	-		
75.0	-		
63.0	-		
50.0	-	(100.0%)	(100.0%)
37.5	-		
25.0	-	(97.0%)	(100.0%)
19.0	-		
12.5	-	55.0%	(76.0%)
9.5	-		
4.75	4	34.0%	46.0%
3.35	6		
2.36	8	4.0%	19.0%
2.00	10		
1.70	12		
1.40	14		
1.18	16	0.0%	4.0%
0.850	20		
0.600	30		
0.425	40		
0.300	50		
0.250	60		
0.212	70		
0.180	80		
0.150	100		
0.125	120		
0.106	140		
0.090	170		
0.075	200	(0.0%)	(0.0%)
0.053	270		
0.037	-		
0.019	-		
0.009	-		
0.0001	-	0.0%	0.0%

Required entry values are defined above. *

Properties of candidate filter soil (CF). D sizes are in mm:

	D_{85CF}	D_{15CF}	D_{60CF}	D_{10CF}	C_u
upper bound	20.51	3.05	13.58	2.71	5.00
lower bound	16.21	1.96	7.46	1.56	4.79

Acceptibility of candidate filter (CF) soil:

USBR criteria	Upper bound	Lower bound
Max % passing #200:	OK	OK
Max particle size (mm):	OK	OK
Maximum D_{15CF}:	OK	OK
Minimum D_{15CF}:	OK	OK
To minimize segregation (from *Table 2*)***		
Max allowable D_{90CF} =	30	OK
Max D_{90CF} =	22.27	

USACE criteria	Upper bound	Lower bound
Max % passing #200:	OK	OK
Max particle size (mm):	OK	OK
Maximum D_{15CF}:	OK	OK
Minimum D_{15CF} ($3\times D_{15B}$):	OK	OK
Minimum D_{15CF} ($5\times D_{15B}$):	OK	OK
To minimize segregation (from *Table B-3*)***		
Max allowable D_{90CF} =	30	OK
Max D_{90CF} =	22.27	

Filters should be relatively uniform (see the C_U value of the candidate filter soil.). Also, filters should not be gap-graded.

*** Generally, this requirement is only necessary for coarse filters and gravel zones that serve as both filters and drains. For sand filters with D_{90} < ~20mm, these limitations are usually not necessary.

Base Material

Determine the gradation curves of the base soil. Use enough samples to define the range of grain size for the base soil. Design the filter gradation based on the base soil that requires the smallest D_{15F} size. If soil has particles larger than the #4 sieve, an adjusted gradation is calculated. Input values below for the base soil (original) gradation (in red):

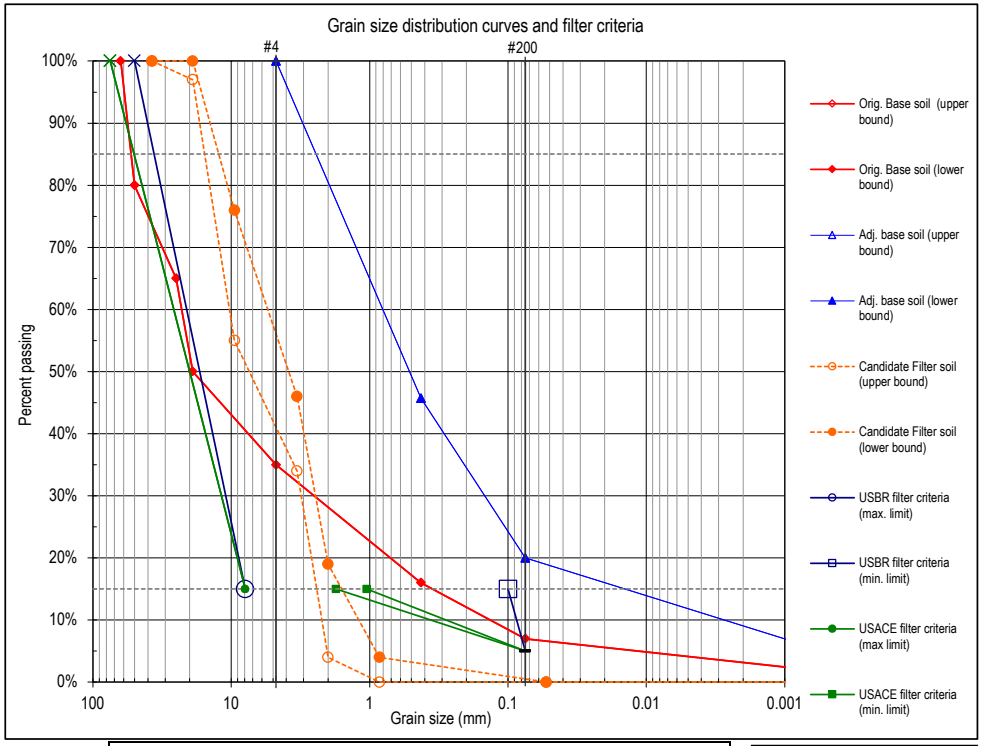
Project: Bellota Weir
Rock Infill to Filter

Name: SRS
Date: 12/23/2021

Particle size (mm)	Sieve #	Base soil (original), % passing (upper bound)	(lower bound)	Adjusted gradation, % passing upper bound	lower bound
63	-	100.0%	100.0%		
50	-	80.0%	80.0%		
25.0	-	65.0%	65.0%		
19.0	-	50.0%	50.0%		
4.75	4	35.0%	35.0%	100.0%	100.0%
4.00	5				
3.35	6				
2.80	7				
2.36	8				
2.00	10				
1.70	12				
1.40	14				
1.18	16				
1.00	18				
0.850	20				
0.710	25				
0.600	30				
0.500	35				
0.425	40	16.0%	16.0%	45.7%	45.7%
0.300	50				
0.250	60				
0.212	70				
0.180	80				
0.150	100				
0.125	120				
0.106	140				
0.090	170				
0.075	200	7.0%	7.0%	20.0%	20.0%
0.053	270				
0.037	-				
0.017	-				
0.009	-				
0.005	-				
0.002	-				
0.001	-				
0.0001	-	0.0%	0.0%	0.0%	0.0%

Required entry values are defined below. *

Average % passing #200 after regarding (if any) = A = 20.0%



Properties of base soil	Upper bound	Lower bound
D_{85B} of original base soil =	52.974 mm	52.974 mm
D_{85B} of adjusted base soil =	2.438 mm	2.438 mm
D_{15B} of original base soil =	0.3505 mm	0.3505 mm
D_{15B} of adjusted base soil =	0.0143 mm	0.0143 mm
D_{60B} of adjusted base soil =	0.802 mm	0.802 mm
D_{10B} of adjusted base soil =	0.0027 mm	0.0027 mm
C_u of adjusted base soil =	292.9	292.9

Fixed points on graph:
- <5% passing #200
- 50mm max. grain size (USBR)
- 75mm max. grain size (USACE)

Filter Material

Filter criteria required by the USBR as published in *Design Standards - Embankment Dams No. 13 (1994):*

D_{85B} used in filter design	2.438
Average Passing #200 sieve of base soil (from adjusted gradation)	20.0%
Base soil category	3
Base soil description	Silty and clayey sands and gravels
Filter criteria (mm)	Maximum: $D_{15F} \leq$ 7.94 To ensure sufficient permeability: Minimum: $D_{15F} \geq$ 0.10
Maximum particle size of filter (mm)	50
Maximum % passing # 200 sieve	5%
PI of material passing #40	0 when tested in accordace with USBR 5360, Earth Manual, on material passing #40

Filter criteria required by the US Army Corps of Engineers as published in EM 1110-2-2300 (31 Jul 94):

D_{85B} used in filter design	2.438
Average Passing #200 sieve of base soil (from adjusted gradation)	20.0%
Base soil category	3
Filter criteria (mm)	Maximum: $D_{15F} \leq$ 7.94 to 7.94 To ensure sufficient permeability: Minimum: $D_{15F} \geq$ 1.05 to 1.75
Maximum particle size of filter (mm)	75
Maximum % passing # 200 sieve	5%
PI of material passing #40	0 when tested in accordance with EM 1110-2-1906

**If the base soil is in category 4, use the lower of the two 'max. D_{15F}' values when the filter is beneath riprap subject to wave action or beneath drains which may be subject to violent surging and/or vibration.

*Required entry values for base soil & candidate filter gradations:

1. Particle size for 100% passing.
2. % Passing the #4 sieve.
3. % Passing the #200 sieve.
4. One point in the 85% - 90% range and another point in the 80% - 85% range, or the 85% point.
5. One point in the 15% - 20% range and another point in the 10% - 15% range, or the 15% point.
6. No duplicate entries; if D100<#4, enter 101% for #4 and 100% for appropriate size.

USBR filter gradation limits:

Maximum limit	
Grain size (mm)	% Passing
50.00	100.0%
7.94	15.0%

Minimum limit	
Grain size (mm)	% Passing
0.10	15.0%
0.075	5.0%

USACE filter gradation limits:

Maximum limit	
Grain size (mm)	% Passing
75.00	100.0%
7.94	15.0%
7.94	15.0%

Minimum limit	
Grain size (mm)	% Passing
1.05	15.0%
1.75	15.0%
0.075	5.0%

Candidate filter soil gradation. Values shown in red in the left column, and all values in the two right columns, can be changed.

Particle size *mm*	Sieve #	% Passing (upper bound)	% Passing (lower bound)
150.0	-		
100.0	-		
90.0	-		
75.0	-		
63.0	-		
50.0	-		
37.5	-	(100.0%)	(100.0%)
25.0	-		
19.0	-	(97.0%)	(100.0%)
12.5	-		
9.5	-	55.0%	76.0%
4.75	4		
3.35	6	(34.0%)	(46.0%)
2.36	8		
2.00	10	(4.0%)	(19.0%)
1.70	12		
1.40	14		
1.18	16		
0.850	20	(0.0%)	(4.0%)
0.600	30		
0.425	40		
0.300	50		
0.250	60		
0.212	70		
0.180	80		
0.150	100		
0.125	120		
0.106	140		
0.090	170		
0.075	200		
0.053	270	(0.0%)	(0.0%)
0.037	-		
0.019	-		
0.009	-		
0.0001	-	0.0%	0.0%

Required entry values are defined above. *

Properties of candidate filter soil (CF). D sizes are in mm:

	D_{85CF}	D_{15CF}	D_{60CF}	D_{10CF}	C_u
upper bound	15.59	2.42	10.32	2.22	4.65
lower bound	15.90	1.59	5.45	1.20	4.55

Acceptibility of candidate filter (CF) soil:

USBR criteria	Upper bound	Lower bound
Max % passing #200:	OK	OK
Max particle size (mm):	OK	OK
Maximum D_{15CF}:	OK	OK
Minimum D_{15CF}:	OK	OK
To minimize segregation (from *Table 2*)***		
Max allowable D_{90CF} =	30	OK
Max D_{90CF} =	16.93	

USACE criteria	Upper bound	Lower bound
Max % passing #200:	OK	OK
Max particle size (mm):	OK	OK
Maximum D_{15CF}:	OK	OK
Minimum D_{15CF} ($3\times D_{15B}$):	OK	OK
Minimum D_{15CF} ($5\times D_{15B}$):	OK	NOT MET
To minimize segregation (from *Table B-3*)***		
Max allowable D_{90CF} =	30	OK
Max D_{90CF} =	16.93	

Filters should be relatively uniform (see the C_U value of the candidate filter soil.). Also, filters should not be gap-graded.

*** Generally, this requirement is only necessary for coarse filters and gravel zones that serve as both filters and drains. For sand filters with D_{90} < ~20mm, these limitations are usually not necessary.



DRAFT 90% Hydraulic Modeling Summary Report

Bellota Weir Modifications Project

Mormon Slough, Calaveras River, CA

February 4, 2022

Prepared for the Stockton East Water District

Prepared by HDR with technical assistance from KSN.

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Appendix D. 2D HEC-RAS Results

Appendix E. Rock and ESM Sizing Calculations

Acronyms and Abbreviations

1D	one-dimensional
2D	two-dimensional
CCWD	Calaveras County Water District
CDFW	California Department of Fish and Wildlife
CDWR	California Department of Water Resources
cfs	cubic foot/feet per second
CHCP	Calaveras River Habitat Conservation Plan
CVFED	Central Valley Floodplain Evaluation and Delineation Program (
District	Stockton East Water District
FEMA	Federal Emergency Management Agency
FHWA	Federal Highway Administration
FIRM	Flood Insurance Rate Map
FIS	Flood Insurance Study
ITP	Incidental Take Permit
H:V	horizontal:vertical
HEC-RAS	Hydrologic Engineering Center - River Analysis System
LiDAR	light detection and ranging
NAVD88	North American Vertical Datum of 1988
NCHRP	National Cooperative Highway Research Program
NMFS	National Marine Fisheries Service
NSSDA	National Standards for Spatial Data Accuracy
Project	Bellota Weir Modifications Project
Reclamation	U.S. Bureau of Reclamation
SEWD	Stockton East Water District
USACE	U.S. Army Corps of Engineers
USGS	U.S. Geological Survey
WSE	water surface elevation

1 Introduction

The Stockton East Water District (SEWD or District) is developing designs for the Bellota Weir Modifications Project (Project) located at the fork of the Mormon Slough and the Old Calaveras River, about 17 miles downstream of the New Hogan Dam. It is a continuation of the Calaveras River Anadromous Fish Protection Project. The Project includes removal of the existing weir and construction of several new elements intended to provide more reliable water delivery systems for SEWD and Calaveras County Water District (CCWD) customers, improve migration of both juvenile and adult fish, and reduce fish entrainment at the facilities, thereby enhancing populations of anadromous salmonids using the Calaveras River (CH2MHILL 2003). The following Project actions will be taken to achieve these goals:

- Replace the existing profile control infrastructure
- Provide fish passage facility for fish species throughout various life stages at Project site
- Provide fish exclusion and protection measures for SEWD intake at Bellota Weir
- Exclude fish from migrating downstream into the Old Calaveras River
- Maintain existing flood conveyance

1.1 Purpose

This report documents and summarizes the hydrologic, hydraulic, and scour analyses conducted for the Project. It also summarizes key background conditions, characteristics, analysis input, and criteria that shaped the hydraulic approach, as well as results and recommendations to meet the following hydraulic objectives:

- Support regulatory permitting efforts associated with Project improvements at Bellota Weir
- Inform design by analysis of proposed hydraulics and scour potential
- Determine Project impacts on flood conveyance

1.2 Approach

To address the hydraulic objectives, the following were conducted:

- **Define Study Area:** Defined the study area extent and described the major features, facilities, and regulatory conditions as needed to provide context to study efforts.
- **Review Existing Data and Past Studies:** Summarized the available and pertinent existing data and studies as data sources for this study and identified data gaps.
- **Establish Hydrology:** Summarized the Calaveras River and Mormon Slough flow conditions used in the hydraulic analyses for this study.
- **Establish Existing Hydraulic Conditions:** Described the hydraulic analysis approach, input, and results of current conditions within the study area.

- **Analyze Proposed Hydraulic Conditions:** Described the input, modeling, and results of proposed hydraulic conditions within the study area. Reported hydraulic results as they relate to established design and regulatory criteria and compared to existing conditions.
- **Perform Scour Analysis of the Proposed Conditions:** Described the scour analysis approach, input, calculations, and results under proposed conditions.
- **Recommendations:** Provided recommendations to proposed facility design, detailed recommendations and inputs into scour countermeasure design, and demonstrated compliance with established design and regulatory criteria.

1.3 Background

The existing Bellota Weir facility on the Mormon Slough/Calaveras River is owned and operated by SEWD and provides water to urban and agricultural users. The Bellota Intake provides municipal and industrial flow year-round for the City of Stockton, and also supplies irrigation water to local farmers during the irrigation season (generally between mid-April and mid-October). The Old Calaveras Headworks (Calaveras Headworks), provides flow control and prevents flooding to downstream landowners on the Old Calaveras River by routing flood waters down the Mormon Slough. During most flow regimes, both the Bellota Weir and Calaveras Headworks are complete fish barriers to upstream migrating adult salmonids.

Continued operation of the Bellota Weir and diversion will be guided by the final Calaveras River Habitat Conservation Plan (CHCP; National Oceanic and Atmospheric Administration 2020). The CHCP provides operational criteria to support the biological goals of maintaining a viable population of threatened California Central Valley steelhead, *Oncorhynchus mykiss,* within the CHCP boundaries, and maintains adequate habitat conditions upstream of the Bellota Weir for fall-, late fall-, spring-, or winter-run Chinook Salmon, *Oncorhynchus tshawytscha*, that may opportunistically migrate into the conservation area. While the CHCP intends to provide conditions that support Chinook Salmon should they migrate into the conservation area, these salmon are not expected to maintain a viable population based both on pre-dam and current conditions. The CHCP enables SEWD to comply with the Endangered Species Act, protecting and managing fishery resources and habitat while maintaining reliable water delivery to its constituents. Following National Marine Fisheries Service (NMFS) approval on August 11, 2020, the District is authorized for a 50-year Incidental Take Permit (ITP #23264), for Endangered Species Act-listed species under NMFS authority.

Upgrade or replacement of the Bellota Diversion, Weir, and Calaveras Headworks is a required compliance measure specified in the CHCP, as part of a seven target fish passage objective. Specifically, the CHCP's target states:

> FP1 and AE1. Avoid migration delays and blockage, and entrainment within the Old Calaveras River Channel by constructing a non-entraining barrier at the Old Calaveras River Headworks Facility and at the downstream end of the channel near the confluence with the [Stockton Diverting Canal] within the first ten years of the ITP.

> FP2/AE3. Construct and implement a combined crest gate/fishway/fish screen at the Bellota Diversion Facility to improve [salmonid] passage into/out of the 18-mile spawning and rearing reach between Bellota and New Hogan Dam and to prevent fish entrainment; target completion within first five years, but no later than 10 years of [issuance of] the ITP.

Based on the CHCP targets summarized above and SEWD's water supply needs, the three primary Project objectives are to:

- **Improve fish passage** by designing and constructing a new crest gate dam, fishway, and fish screens that include passage for *Oncorhynchus mykiss* and opportunistic migration for fall-, late fall-, spring-, and winter-run Chinook Salmon.
- **Reduce fish entrainment** by constructing a new non-entraining fish barrier at the Old Calaveras River.
- **Provide more reliable water delivery** through weir and intake improvements.

2 Site Conditions

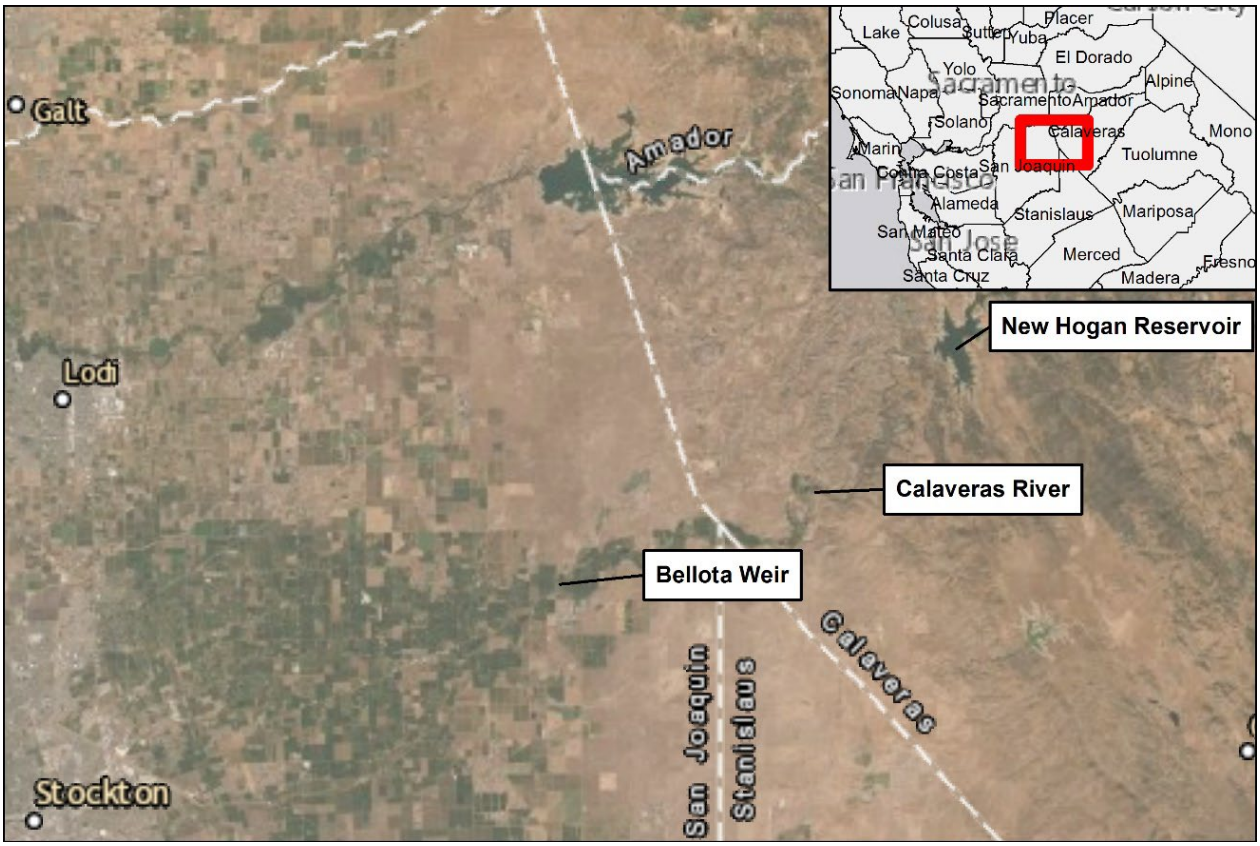
Physical considerations related to the existing facilities' hydrologic, hydraulic, and geotechnical conditions are presented in this section.

2.1 Calaveras River

The Project is located within the Calaveras watershed at the divergence of the Calaveras River Channel and Mormon Slough. Upstream of the Project site, the Calaveras River has one reach upstream of the New Hogan Reservoir and Dam and the other between the dam and Mormon Slough. The hydrology is dominated by rain events due to the low elevation of the upper watershed area. The basin tributary to the New Hogan Reservoir is 363 square miles. The Calaveras watershed between New Hogan Reservoir and the Project is an additional 107 square miles. The Calaveras River flow downstream of New Hogan Dam are metered by reservoir operation and influenced by diversion operations during irrigation season. Figure 2-1 shows the general Project location and surrounding area.

At the Project Site, the Calaveras River channel below Mormon Slough is controlled by the Calaveras Headworks. Compared to the channel upstream of Mormon Slough, the channel capacity is significantly reduced due to excessive vegetation caused by irregular flows.

Figure 2-1. Project Location Map



2.2 Existing Facilities

Artificial structures located in the watershed can become impediments to salmonids at various times due to flow conditions. Fish entering the Stockton Diverting Canal or Mormon Slough can become stranded in pools to die or become prey. This report is most concerned with:

- Bellota Weir, built in the late 1940s
- Bellota Intake for the Dr. Joe Waidhofer Water Treatment Plant built in 1978
- Old Calaveras Headworks, built in 1933
- Mormon Slough, built in 1971

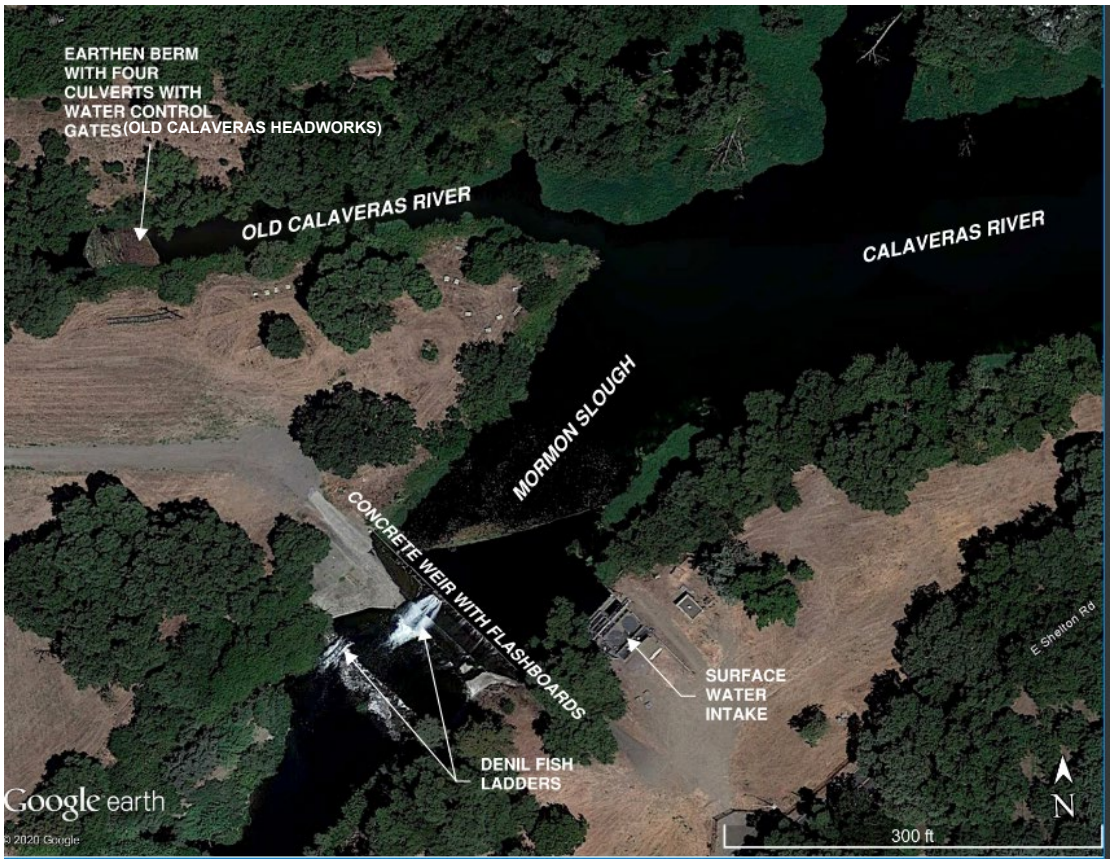
2.2.1 Mormon Slough

The Mormon Slough is a flood control facility completed in 1971 to divert flood flows from the Upper Calaveras River and re-introduce them closer to the City of Stockton. The Project included widening the Mormon Slough, levee construction, and bank protection. The design discharge for the Mormon Slough is 12,500 cubic feet per second (cfs) and is intended to accept the full flow of the Calaveras River for the 100-year event.

2.2.2 Bellota Weir

The Bellota Weir is the largest check dam on the Calaveras River system. The weir is located on the Mormon Slough approximately 350 feet downstream of its confluence with the Old Calaveras River. The weir is an in-river concrete structure designed to accommodate installation of a flashboard dam; during irrigation season, the check dam crest with the flashboards installed is 8 feet above the channel invert (invert elevation of 121.44 feet North American Vertical Datum of 1988 [NAVD88]). During non-irrigation seasonal (flood season) periods of the year, shorter flashboards are installed 2 feet above the channel invert. Existing features of the Project are identified in the Figure 2-2.

Figure 2-2. Prominent Features of the Bellota Weir Modifications Project Site



The Bellota Weir includes two Denil fish ladders, installed in 1999 and 2001. During the irrigation season, only the downstream Denil fish ladder is in place, while during non-irrigation times, both Denil fish ladder sections are in place. Employees from California Department of Fish and Wildlife (CDFW) and Fishery Foundation evaluated conditions at the ladders in 2007. The upper ladder is longer than the maximum recommended ladder length of 30 feet. The lower ladder has a 3 horizontal to 1 vertical (3H:1V) slope ratio, double the maximum recommended slope of 6H:1V. Finally, water does not flow through the lower ladder as intended but instead flows around the sides resulting in an inadequate water supply to the ladder that confuses fish looking for the ladder entrance (California Department of Water Resources [CDWR] 2007).

2.2.3 Bellota Intake

The Bellota Intake is a gravity diversion constructed in 1978 on the south riverbank about 50 feet upstream of the Bellota Weir. Water diverted through the intake is conveyed to the City of Stockton and the Dr. Joe Waidhofer Water Treatment Plant (SEWD, HDR, and KSN 2020). The structure possesses bulkhead slots for installation of stop logs, a trashrack facility, two 14-foot-wide rectangular intake channels perpendicular to the river, two intakes with fish screens, a bypass pipeline, a sediment trap, and two slide gates to regulate flow.

2.2.4 Old Calaveras Headworks

The Old Calaveras Headworks is located approximately 600 feet downstream from the Old Calaveras River, Mormon Slough confluence (Figure 2-2). The existing structure includes an earthen berm, four culverts with control gates, a trashrack, and concrete slope lining.

The earthen berm crest is 68 feet long and 10 feet wide. The 4-foot-wide, concrete culverts penetrate the berm and control flow to the Old Calaveras River. There is a 21-foot-wide trashrack on the upstream side of the berm protecting the culvert inlets, their concrete headwall, and wing walls. Flow is controlled with slide gates on the upstream side – two are operated manually and two are electrically actuated. The slide gates are operated between April and November to deliver water to downstream agricultural users, intermittently operated between November and April for groundwater recharge on the Old Calaveras River, and open during irrigation season to provide flow to downstream diverters (SEWD and FISHBIO 2019).

The headworks is a barrier to upstream adult fish migration and does not comply with juvenile downstream passage or fish protection guidelines provided by CDFW and NMFS.

In 2003, SEWD placed a small mesh net across the Old Calaveras River upstream of the Old Calaveras Headworks structure (CH2MHILL 2003) to encourage juvenile fish to reside in the Bellota Pool and migrate upstream to suitable habitat or downstream to the Mormon Slough.

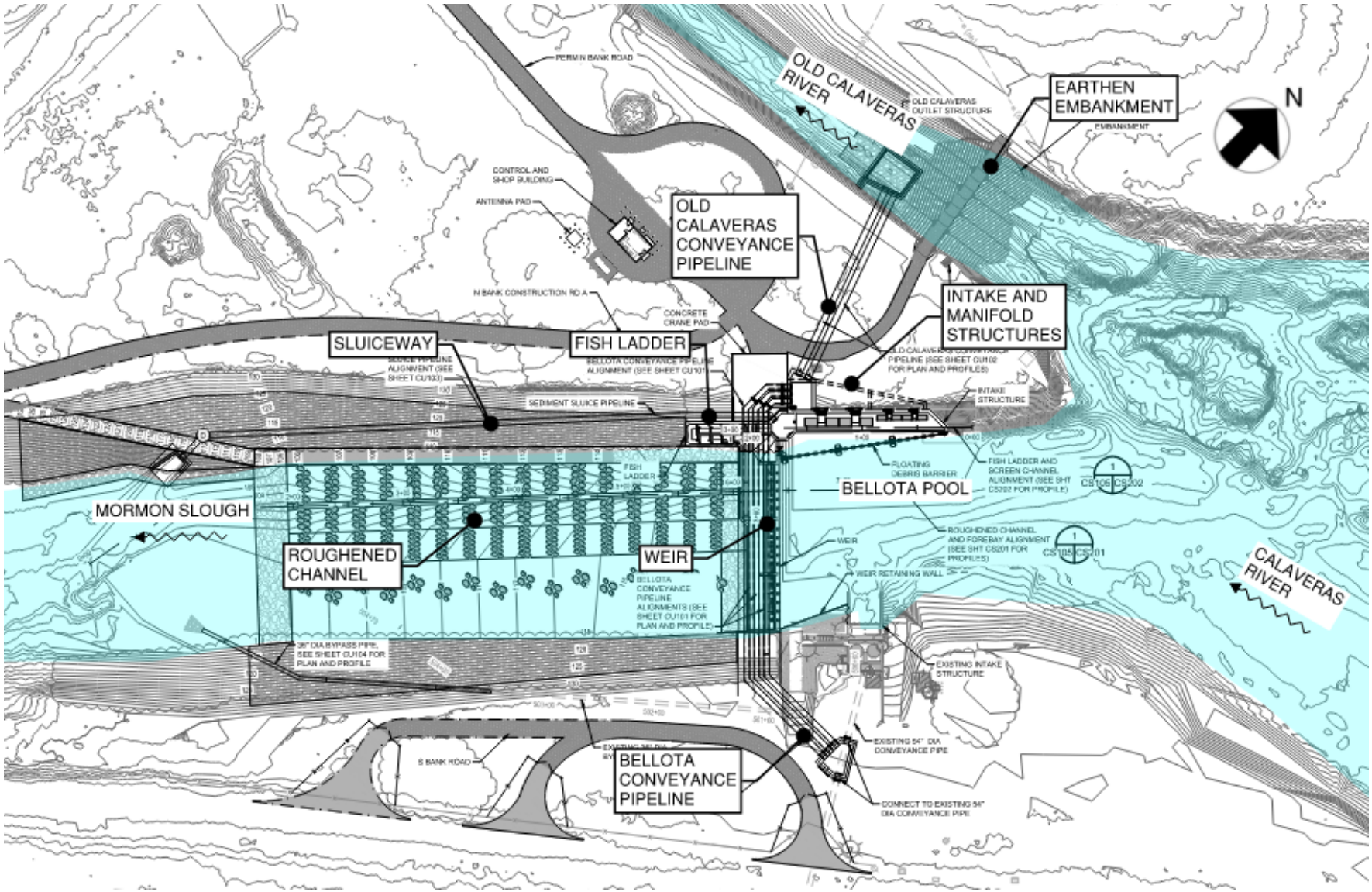
2.3 Proposed Facility

The proposed Project facility combines several operational and engineered elements to achieve its intended objectives: intake and weir replacement and improvements for effective fish passage, reduced fish entrainment, and water supply reliability. The proposed Project elements are briefly summarized below and illustrated in Figure 2-3:

- Construct a concrete weir with a series of weir gates designed to modulate forebay pool elevations based on operational needs. The proposed weir consists of a 150-foot-wide gated concrete weir with three gate openings. A concrete sill (elevation 115.5 feet) just downstream of the weir gates regulates the upstream water surface during normal operation. A low flow notch (invert elevation 114.3 feet) in the weir sill allows passage of low flows (Figure 4-5.).

- Improve sediment conveyance by adding a vertical slide gate at the dam, operated to facilitate bypass and sluicing functions.
- Provide reliable fish passage using a roughened channel in combination with a vertical slot fishway to accommodate the full range of flows and river stages at the site. The fish passage ramp with a low flow channel extends from weir sill approximately 360 feet downstream to where it terminates at an approximately 2.5-foot-deep pool that regulates the water surface during low flows, and then ties in with the existing channel invert. Roughness features (boulder weirs and rock clusters) along the fish passage ramp will provide sufficient depth during low flows and velocity refugia during high flows. Boulder weirs will be located at approximately 8-foot intervals along the low flow channel; the boulders will protrude from 1.5 to 2.5 feet from the low flow channel invert. Rock clusters will be placed at a similar interval along the southern portion of the fish passage ramp and protrude approximately 1.5 feet from the ramp invert.
- Provide reliable fish screening, both for the Bellota Intake and the Old Calaveras River, to prevent fish entrainment.
- Provide a non-entraining fish barrier to the Old Calaveras River.

Figure 2-3. Overview of Proposed Project Elements



3 River Hydrology

3.1 Calaveras River Hydrology

This section describes the Calaveras River hydrology, including historic and ongoing flow regulation, flood recurrence, fish passage design flows, and Project-specific flow uncertainties.

Hydrology in the region was analyzed using desktop research, a U.S. Army Corps of Engineers (USACE)-installed gage downstream of the weir, and data provided by the Federal Emergency Management Agency (FEMA).

3.1.1 Historic and Ongoing Flow Regulation

Calaveras River/Mormon Slough flows are regulated by operation of New Hogan Dam about 17 miles upstream of the Mormon Slough and Old Calaveras River bifurcation. Discharge from the dam is altered throughout the year. Additionally, diversions throughout the reach between Bellota Weir and New Hogan Dam contribute to water withdrawals that must be considered when evaluating low flow conditions.

Flow release from New Hogan Dam is regulated by the USACE; rights to storage releases are held by the U.S. Bureau of Reclamation (Reclamation) and contracted to SEWD and CCWD. Streamflow records are available using a USACE gage for both pre- and post-construction of the dam according to the CH2MHILL reports, but gage information prior to 1997 was not found online and multiple information requests to USACE yielded no results. There is no U.S. Geological Survey (USGS) gage nearby.

Releases to the river are lowest between October and December. Because much of the upper watershed is below elevation 4,000 feet (typical winter snowline), flow in the Calaveras River is heavily reliant on precipitation instead of snow melt unlike many other rivers nearby. The dam occasionally releases flood flows between January and March, but releases become more consistent during the irrigation season (Mid-April to Mid-October). These flows supply most of the irrigation demand for diversions between the dam and weir (CH2MHILL 2003). In a February 27, 2020 workshop, SEWD stated 20 to 100 cfs of flow is released downstream of Bellota Weir during the irrigation season (SEWD, KSN, and HDR 2020).

3.1.2 Flood Recurrence

Previously established flood events were used to determine water surface profiles at the Project location for alternative development, comparison, and measuring performance. Flood recurrence information for the Calaveras River and Mormon Slough at Bellota was obtained from two sources: the Flood Insurance Study for San Joaquin County conducted by the FEMA (2016), and the Lower San Joaquin River Feasibility Study conducted by the USACE (2017). FEMA flow values were used to inform the design of the proposed facility as it relates to potential impacts on the regulatory floodplain. The USACE study includes flood recurrence values that account for flow regulation by New Hogan Dam (regulated values) and have been used in flood risk management planning for the Stockton area. A combination of FEMA and USACE regulated values informed the proposed design as it relates to channel and structural stability. FEMA and USACE

flow values for selected recurrence intervals are listed in Table 3-1. The FEMA 50-year and 100-year flows are similar due to the 12,500-cfs annual objective release flow target from New Hogan Dam (Table 3-1).

Table 3-1. Flood Frequency and Magnitude of Calaveras River Flow at Mormon Slough at Bellota USACE Gage ID MRS

Recurrence Interval	Annual Exceedance Probability (%)	FEMA (2016) (cfs)	USACE, Regulated (2017) (cfs)
1-year	99.9	-	-
2-year	50	-	3,520
5-year	20	-	9,520
10-year	10	12,500	9,530
25-year	4	-	10,640
50-year	2	12,500	12,500
100-year	1	12,690	12,500
500-year	0.2	27,100	16,000

3.1.3 Fish Passage Design Flows

Selection of site-specific fish passage design flow criteria influences fish passage facility size, complexity, and anticipated performance. Guidelines presented by the NMFS and CDFW are based on exceedance analysis of mean daily flows during salmonid migration season, but can be modified to suit site-specific requirements.

An exceedance analysis was conducted to determine annual and seasonal low and high fish passage design flows (HDR 2021). The analysis included mean daily streamflow data obtained from USACE Gage ID MRS (USACE 1997) for the period of 1997 to 2021. Seasonal periods analyzed included irrigation season and migration seasons for salmonid species covered by the CHCP. Per NMFS (2011) guidelines, low and high fish passage design flows were defined as the 95 percent and 5 percent exceedance values, respectively.

The maximum high flow value across all of the analyzed migration seasons was selected as a high fish passage design flow. The irrigation season high flow and the annual high flow were also considered during the design process to verify adequate fish passage across a range of flows for which hydraulics in the roughened channel may vary. Due to upstream flow regulation and diversion, the Calaveras River near the Bellota Weir periodically runs dry throughout the year, resulting in a calculated low fish passage design flow equal to 0 cfs. CDFW guidelines recommend that 3 cfs be used as an alternative low fish passage design flow in instances where the calculated low flow value is 0 cfs; however in a February 27, 2020 workshop, SEWD stated 20 to 100 cfs of flow is released downstream of Bellota Weir during the irrigation season (SEWD, KSN, and HDR 2020). Additionally, 20 cfs was assessed as the likely lower threshold of discharge

for passable hydraulics within the anticipated roughened channel concept, which relies on a low flow notch and rock weirs to concentrate flows to support a swim through condition. This configuration must be formed by rock elements large enough to remain stable under the high flow condition and the size of these elements constrains the physical dimensions of the low flow section. Based on this information, 20 cfs was selected as the low fish passage design flow.

Table 3-2 provides fish passage design flows included in hydraulic analysis.

Table 3-2. Low Fish Passage and High Fish Passage Design Flows for Annual, Irrigation, and Salmonid Migration Seasons

Season	Period	Low Fish Passage Design Flow (cfs)	High Fish Passage Design Flow (cfs)
Annual	All year	3.0*	**545.3**
Irrigation	Mid-April to Mid-Oct	7.2	**150.9**
Fall Chinook Migration	Nov 1 to Dec 31	3.0*	420.6
Steelhead Migration	Nov 1 to March 31	3.0*	1,728.6
Late Fall Chinook Migration	Nov 1 to Feb 28	3.0*	**1,735.2**
Spring Chinook Migration	March 1 to May 31	3.0*	908.1
Winter Chinook Migration	Jan 1 to May 31	3.0*	1,710.0

Notes: Low and High Fish Passage Design Flow are defined as the 95 percent and 5 percent exceedance value for the specified migration season, respectively.
Bolded values indicate the selected high fish passage design flows
* Calculated value is 0 cfs. Alternate low design flow selected per California Department of Fish and Game 2002 (Table 4) and 2004 (Table IX-5)

3.1.4 Project-Specific Flow Uncertainties

Numerous hydrologic inputs and withdrawals in the Project reach of the Calaveras River system create high flow variability during seasonal low flow conditions. The Fish Passage Barriers Assessment report shows there are 29 diversions between the weir and New Hogan Dam on the Calaveras River (CDWR 2007).

In addition, New Hogan Lake influences river flow through reservoir releases, which may devalue the exceedance analysis performed. High or low flows may result from reservoir releases rather than natural hydrology.

The following assumptions have been adopted as part of the design process:

- Mean daily flow statistics will be based on data available after 1997 (USACE 1997). Use of data recorded between December 10, 1997 and July 28, 2021 provides an 18-year period of record that gives an adequate representation of current and future operational conditions in the Project reach. This information will be used herein to make reasonable assumptions for development of design criteria used for alternative assessment.

- The exceedance analysis is valid considering generalities and seasonality; it is influenced by New Hogan Dam releases and diversion of flow. Any adjustment in dam releases or diversion intakes will influence flow magnitudes at the Project site.

4 Hydraulics

A combination of one-dimensional (1D) and two-dimensional (2D) models were used to inform the design process and confirm design compliance with regulatory requirements. Specifically, the objectives of the combined modeling efforts are to:

- Support regulatory permitting efforts associated with Project improvements
- Confirm that the range of hydraulic conditions anticipated after Project implementation will meet the fish passage, sediment passage, and water supply and flood resilience/stability criteria established during preliminary and final design
- Refine sediment continuity and particle mobility thresholds to inform sediment transport and sluicing targets
- Provide hydraulic conditions such as shear stress to verify the following:
    - Sizing and stability of channels
    - Boulder matrices of the roughened channel
    - Supporting rock filters for boulder structure and low flow channel
    - Potential for scour holes downstream of structure and forces on existing structures

Two distinct modeling efforts were completed to meet these objectives. A 1D Hydrologic Engineering Center - River Analysis System (HEC-RAS) model was used to verify compliance with the FEMA No-Rise condition. A 2D HEC-RAS model was used to inform the proposed weir structure and fish passage ramp design.

4.1 One-Dimensional Model – No-Rise Analysis

The Project is currently within a regulatory floodway designated by the FEMA National Flood Hazard Layer and Flood Insurance Rate Map (FIRM) (FIRM Panel 06077C0370F effective October 20, 2016. For new construction to occur, the Project design must demonstrate, through appropriate hydraulic analyses, that the proposed improvements will not result in any increase in base flood elevation for a 100-year event (1:100 chance of occurrence in a given year for the 100-year event).

4.1.1 Effective FEMA Mapping

The Calaveras River was first subject to detailed study for FEMA flood mapping purposes in 1994 (FEMA 2002). Per the FEMA Flood Insurance Study (FIS), a restudy was completed in 1998 (San Joaquin Area Flood Control Agency 1998) for both the Calaveras River and Mormon Slough, however, the restudy limits did not extend upstream to the Project. For the Calaveras River, the restudy extended to Solari Ranch Road. For the Mormon Slough, the restudy extended to Jack Tone Road. Both locations

are miles downstream of the Project and do not serve as the basis for mapped conditions near the Project.

Near the Project, the Calaveras River includes a stream centerline, lettered cross sections, mapped base flood elevations, designated floodway, and areas of floodway fringe. Figure 4-1 provides a portion of the October 20, 2016 FIRM map. In the figure, an area of floodway is mapped in the upstream most 1,600 feet of the Mormon Slough, starting at the channel bifurcation and ending at the Bellota Road Bridge. While mapped as a floodway, there are no lettered cross sections, base flood elevations, or centerline in this reach. This designated floodway transitions to a FEMA Zone A floodplain downstream. A detailed analysis likely was not used to map the Mormon Slough reach immediately downstream.

4.1.2 FEMA Flood Insurance Study

The FEMA FIS report (2016) details information on the existence, severity, and mapping basis of flood hazards within the County of San Joaquin for use under the National Flood Insurance Program. The FIS was last revised in 2016 as FIS Number 06077CV001B. The FIS provides summaries of included flood sources, associated discharges with these flood sources, floodway data, and flood profiles, among other information. The Calaveras River is included in these summaries, but with some inconsistencies. The flood source table lists the Calaveras River, but places the upstream limit at Solari Ranch Road (approximately 13 miles downstream of the Old Calaveras Headworks), instead of perhaps New Hogan Dam or the Stanislaus County line. The discharge summary table provides the most upstream discharge location at Jack Tone Road, with a discharge of 1,450 cfs and a contributing area of 14.4 miles. This discharge matches the re-study hydraulics from 1998 (San Joaquin Area Flood Control Agency 1998).

By contrast, the floodway data table, which details floodway widths and surcharge elevations at each lettered cross section, includes data for sections to the upstream limit at the Stanislaus County line. Additionally, the FIS includes flood elevation profiles for the 10-, 100-, and 500-year events from the county line to the Calaveras Headworks (labeled Diversion Structure). The 50-year profile overlaps the 10-year due to shared discharge values. No profile overtops the Diversion Structure and each is considerably reduced in depth downstream of the structure. The 50-year profile begins at the Calaveras Headworks and extends downstream.

The following conclusions were made from the FIS study:

- The 100-year event from the Calaveras River is diverted entirely to the Mormon Slough.
- No discharge passes the Calaveras Headworks. The Calaveras River discharge at Jack Tone Road is based on tributary areas downstream of the Calaveras Headworks.
- The original detailed study extended to the Stanislaus County line, but was not revisited during the 1998 restudy.

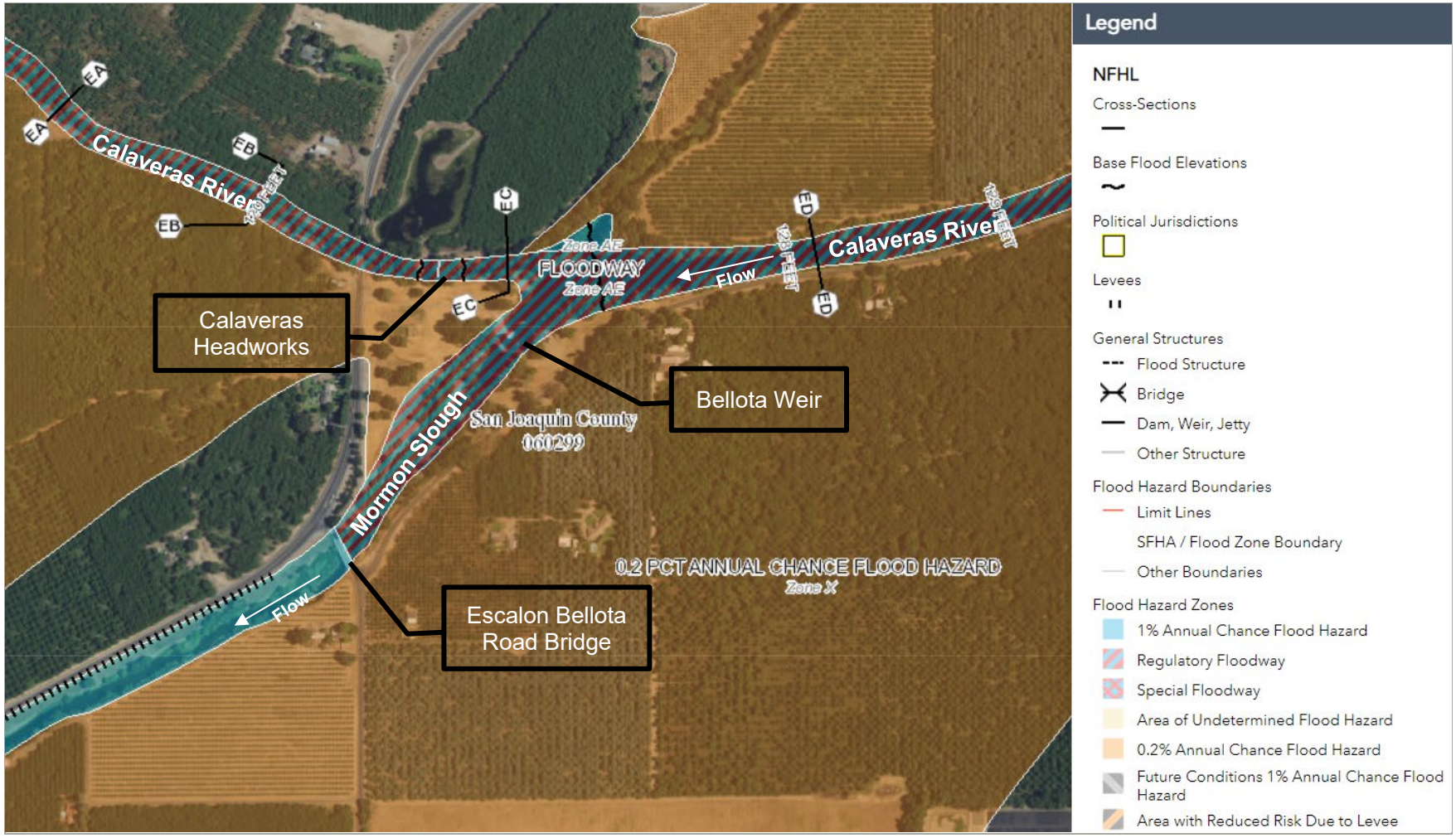
The Mormon Slough is also listed as a flood source and included in the various summary tables. The Summary Discharge table indicates that Mormon Slough starts at Bellota Weir and conveys diverted flow from the Calaveras River (FEMA 2016). The listed

drainage area is 480 square miles. However, the Floodway Data table only includes the Mormon Channel, a downstream facility, but not the Mormon Slough. No flood profiles are included for the Mormon Channel, which is typical for a FEMA Zone A.

In the absence of additional information, the following uncertainties exist:

- The basis of the designated floodway in the upper reach of the Mormon Channel. Potentially, the floodway was arbitrarily mapped. No Effective elevation data is present from which to compare a proposed condition (e.g., surcharge values).
- The influence of Mormon Channel conveyance on the flood profiles for the Calaveras River upstream of the Calaveras Headworks. In this area, the flood profiles are flat, which is anticipated for slack or ponded water. However, the backwater elevation should be based on the normal depth at the Mormon Channel entrance.

Figure 4-1. Effective Floodplain Exhibit



Source: FEMA (2016)

4.1.3 FEMA Data Request and Effective Model

A 1D USACE HEC-RAS model was likely developed to support the original mapping effort and would be classified as the Effective model. As a matter of general practice, the Effective model should be used as the basis for determining compliance with FEMA requirements and is typically obtained from FEMA or the local community floodplain administrator. However, through extensive effort and communication with FEMA and representatives from local flood control authorities, the Effective modeling has not been located. The FEMA Engineering Library has no record of this model, nor does the community floodplain administrator for San Joaquin County.

4.1.4 No-Rise Approach

For new construction to occur in the floodway, the Project must demonstrate, through appropriate hydraulic analyses, that the proposed floodway encroachment will not result in an increase in base flood elevation for a 100-year event (1:100 chance of occurrence in a given year for the 100-year event). In the absence of the Effective model, FEMA guidance, as confirmed in correspondence with San Joaquin County (Shayan Rehman, Senior Engineer, personal communications, 2021), stipulates new existing conditions modeling will serve as the basis to evaluate a No-Rise condition. A No-Rise condition will be accepted without the Effective model if the proposed flood elevation is below the existing conditions flood elevation and if the proposed flood profile is equal to or less than the current Effective base flood elevations.

4.1.5 Existing Condition Model

A 1D HEC-RAS model was developed for existing conditions to serve as a benchmark to evaluate the No-Rise condition for the proposed facility. The existing condition represented in the 1D model was the existing weir structure with flashboards implemented to raise the weir invert by 2 feet, from 113.44 feet to 115.44 feet.

Study Area

The study area covers an approximate 2-mile reach that begins on the Calaveras River approximately 1.6 miles upstream of where the Calaveras River is diverted into the Mormon Slough and ends on the Mormon Slough approximately 0.4 mile downstream of the diversion. Duck Creek confluences with the Calaveras River just upstream of the diversion. The Bellota Weir is approximately 300 feet downstream of the diversion. Apart from the Bellota Weir, Escalon Bellota Road Bridge approximately one-quarter mile downstream of the weir, is the only other structure within the study reach.

Model Geometry

One-dimensional model geometry consists of a channel centerline that defines the length of the flow path; cross-sections that define the hydraulic characteristics of the channel, including shape, roughness, and slope; and hydraulic structure elements that represent structures obstructing or affecting flow within the channel.

Terrain Development

Topographical data was aggregated within HEC-RAS RASMapper to develop modeling terrain surfaces from which cross-section geometry was extracted. The primary source of topographic or terrain data for construction of the HEC-RAS models was light detection and ranging (LiDAR) data compiled by CDWR under the Central Valley Floodplain Evaluation and Delineation Program (CVFED) and Delta Risk Management Study from March to April 2008. This served as the base layer for a composite modeling surface. The LiDAR data comprises points that densely cover the entire region, with a point spacing of 18 to 30 inches, depending on vegetation. The minimum expected horizontal accuracy was tested to meet or exceed a 3.5-foot horizontal accuracy at 95 percent confidence level using RMSE(r) x 1.7308 as defined by the National Standards for Spatial Data Accuracy (NSSDA). Final ground surface LiDAR point elevation data in areas other than open terrain meet or exceed NSSDA standards of 0.6 feet RMSE vertical (Accuracy z = 1.2 feet at the 95 percent confidence level). Accuracy was tested to meet a 0.6-foot fundamental vertical accuracy at 95 percent confidence level using RMSE(z) x 1.9600 as defined by the NSSDA. The horizontal datum of the stations is North American Datum of 1983 (2007) and the vertical datum is NAVD88.

Project survey data conducted by KSN and provided on June 04, 2020 to support site design was the best available data and used to enhance the base LiDAR data to provide correlation to proposed design elements.

The Calaveras River channel was supplemented by two survey datasets. In the area of the Bellota Weir, the Calaveras Headworks and approximately 4,600 feet upstream of the weir, bathymetric survey was completed as part of the Project survey dataset. Upstream of this bathymetric survey, channel cross-sections collected by CDWR for use in the CVFED effort and connecting three-dimensional polylines was used to develop simple bathymetry data in the upper areas of the study reach.

Cross-Section Development

The study reach was discretized into 96 cross-sections spaced along the channel centerline at approximately 150-foot intervals, with finer spacing near the weir. Each cross-section was extended across the study reach perpendicular to the direction of flow, ensuring that high points on either side of the study reach were included within the cross-section limits. Cross-section geometry was extracted from the composite terrain surface described above. Ineffective flow areas were used to reduce cross-section area available for conveyance at locations where structures or channel orientation obstructed flow. In addition to geometry, definition of bank stations and Manning's roughness values for each cross-section are required to perform flow calculations.

Bank stations partition the cross-section into three conveyance areas—main channel, left overbank, and right overbank—for which flow calculations are performed. Bankfull flow stage is typically used to estimate the extents of main channel. Grade breaks in the bank slope based on the terrain and changes in vegetation cover from aerial imagery were used for preliminary demarcation of the bankfull flow boundary. Channel banks within the study reach were generally steep and elevations at which grade breaks occurred were

inconsistent. Additionally, dense, bushy vegetation generally obscured the entire bank, making it difficult to differentiate the extents of overbank vegetation from vegetation overhanging into the main channel. Relatively frequent flood events (1- to 10-year recurrence intervals) are often used to estimate bankfull discharge in the absence of clear visual indicators (Natural Resources Conservation Service 2001). Thus, the 2-year regulated flow value (3,520 cfs) estimated by USACE (2017) was input into the 1D model to approximate bankfull flow stage and inform bank station placement. Preliminary bank stations that did not align with the 2-year water surface elevations (WSE) were adjusted accordingly.

Manning's roughness coefficients (*n*) are used to represent energy loss in the model. Manning's *n* values used in the model were associated with land cover types delineated within the study area based aerial imagery and available photos. Three land cover types were identified within the existing channel:

Alluvial channel with light to moderate vegetation land cover type was assigned to portions of the channel (primarily the channel invert) where either water was present or minimal vegetation growth was observed in the aerial imagery.

Alluvial channel/overbank with dense vegetation land cover type was assigned to the channel banks and portions of the channel where dense brush and reeds that would be submerged during high flows were observed in the aerial imagery.

Concrete land cover type was assigned to the concrete weir structure.

Each land cover type was assigned a Manning's *n* value based on descriptions presented by Chow (1959). Table 4-1 shows the land cover types and their associated Manning's *n* values and description.

Table 4-1. Manning's Roughness Values Used for the Existing Channel in the 1D HEC-RAS Model

Land Cover Type	Source	Description	Manning's *n*
Channel, earthen with light to moderate vegetation	Chow (1959)	Main channel at full stage; clean, straight, no rifts or deep pools, with some stones and weeds (normal value)	0.035
Alluvial channel/overbank with dense vegetation	Chow (1959)	Channels not maintained; weeds and brush uncut, highest stages of flow (normal value)	0.070
Concrete	Chow (1959)	Concrete; trowel finish (normal value)	0.013

Hydraulic Structure Geometry

Bellota Weir – The Bellota weir was represented within the terrain, rather than as an inline structure element. In HEC-RAS, flow over an inline structure element can be modeled using a weir flow equation. However, the depth of flow over the weir for the 100-year flow event considered in this study is large (approximately 6 feet) and flow is more likely to resemble flow through a channel than over a weir. Four cross-sections were used to represent the existing weir: two cross-sections each to represent the upstream

and downstream extents of the weir (invert elevation = 113.44 feet; Figure 4-2) and the flashboards (invert elevation = 115.44 feet; Figure 4-3), respectively. Cross-section geometry was modified as necessary based as-built drawings (R.B. Welty and Associates 1967).

Figure 4-2. HEC-RAS Cross-Section for Existing Weir (Invert Elevation 113.44 feet)

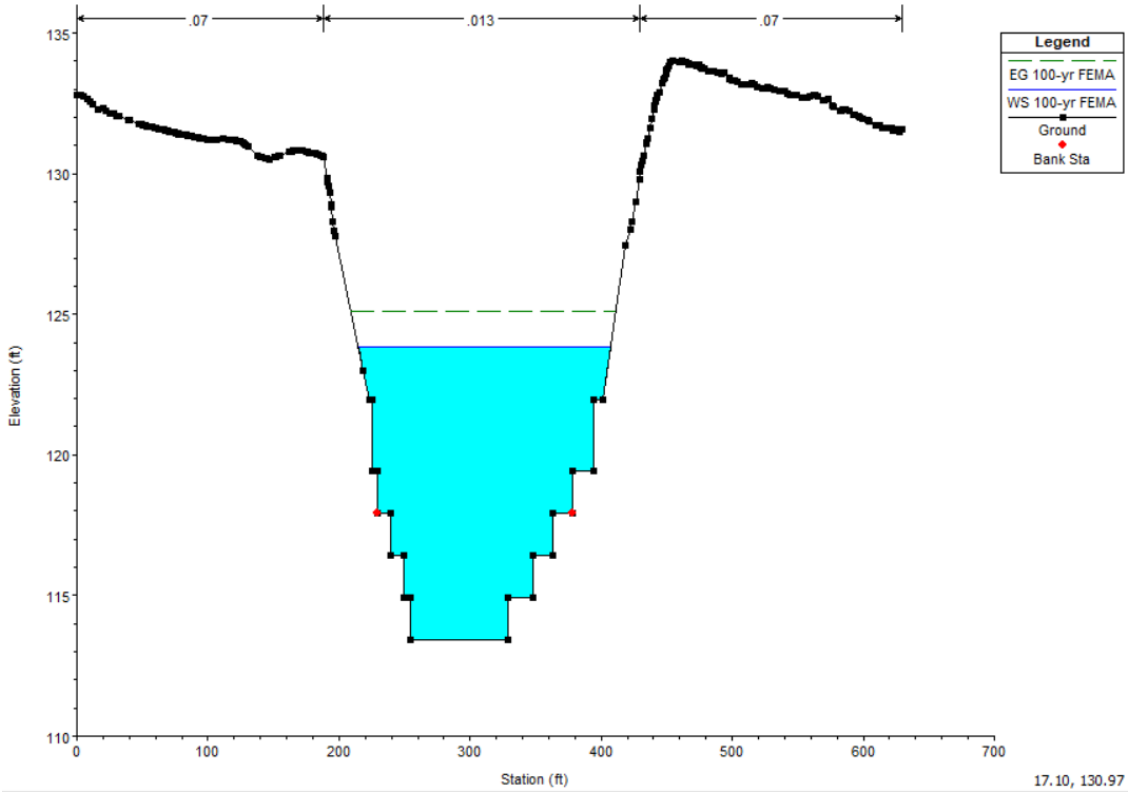
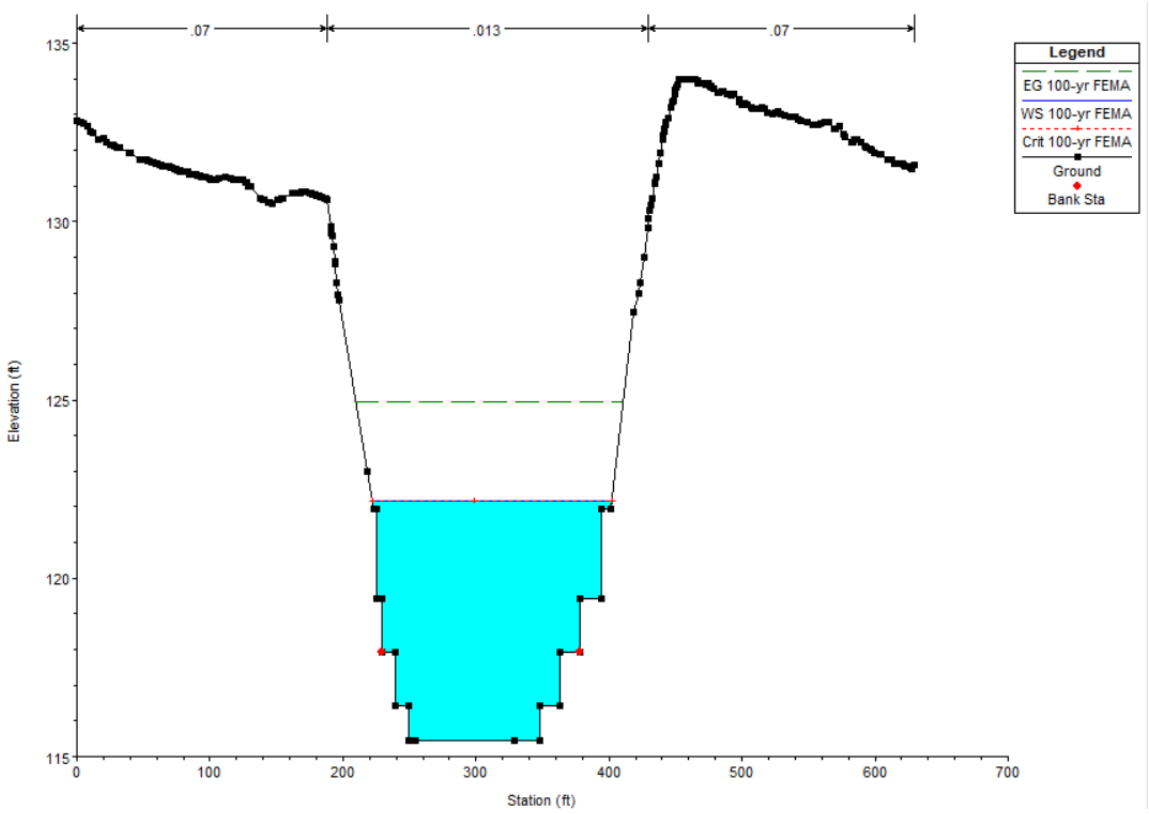


Figure 4-3. HEC-RAS Cross-Section for Existing Weir with Flashboards Implemented (Invert Elevation = 115.44 feet)



Escalon Bellota Road Bridge– Bridge geometry was obtained from the CVFED model. The source of the data used in development of the CVFED bridge geometry is unknown.

Steady Flow Data

Steady flow data is required for the model to compute a water surface profile. Data required includes specification of flow regime, boundary conditions, and channel discharge.

Flow Regime

Hydraulic calculations in the 1D model were performed under the assumption of a subcritical flow regime to provide a conservative WSE.

Boundary Conditions

Boundary conditions provide the information required to estimate the initial WSE used to initiate hydraulic profile calculations. Calculation of a subcritical profile begins at the downstream cross-section; thus, a normal depth boundary condition was set at the downstream cross-section. Normal depth boundary conditions require specification of the energy grade slope at the cross-section. The channel invert slope was used to

approximate energy grade slope; an invert slope of 0.002 feet/feet was estimated at the boundary using the terrain surface.

Channel Discharge

The 100-year FEMA flow value (12,690 cfs; Table 3-1) was the channel discharge specified for the existing conditions model.

4.1.6 Proposed Condition Model

A 1D HEC-RAS model was developed for the proposed facility, including a new weir structure with fish passage ramp and fish ladder.

The proposed condition model is the same as the existing conditions model, with modified geometry and manning's roughness coefficients in cross-sections at the weir and along the proposed roughened channel (cross-section 2375 through 1474):

- Cross-section geometry was updated to reflect the proposed weir and roughened channel dimensions. Assumes all Obermeyer weir gates are at elevation 115.5 feet NAVD88.
- Ineffective flow areas were updated to reflect areas obstructed by the new weir structure, including the fish ladder, the exit for which is likely to be closed under high flow conditions.
- Manning's *n* values were updated to represent the proposed concrete weir and fish passage ramp, riprap-lined slopes, and weir access road:
    - **Fish Passage Ramp** with boulder weirs and rock clusters. Limitations of 1D hydraulic models make it difficult to adequately capture the hydraulic impact of detailed features like boulder weirs and rock clusters, thus, the roughness features on the fish passage ramp were represented by Manning's *n* rather than as features in the terrain. The fish passage ramp was divided into two land cover regions:
        - **Engineered stream bed material (ESM) with boulder weirs** land cover type was assigned to the low flow section of the ramp. This section is comprised of ESM with boulder weirs (maximum 2.5 feet exposed above the low flow channel invert) spaced longitudinally along the fish passage ramp at approximately 8-foot intervals. The boulder weirs generally span the width of the low flow channel. Manning's *n* was calculated using the Limerinos relationship between the 84th percentile bed-material diameter (D_{84}), hydraulic radius, and Manning's *n* described in Equation 5 of USGS (1989). The D_{84} calculated for the engineered streambed material (1.6 feet; Section 5.3.4) was used to estimate of Manning's *n* throughout the boulder weir section of the fish passage ramp.
        - **Fish Passage Ramp, engineered stream bed material with rock clusters** land cover type was assigned to the overbank region of the ramp. This region is comprised of primarily engineered streambed material, with rock clusters

(maximum 1.5 feet exposed above the overbank invert) located at approximately 8-foot intervals longitudinally along the fish passage ramp**.** As with the low flow region, Manning's *n* in the overbank was calculated using the Limerinos relationship between the 84th percentile bed-material diameter (D_{84}) and Manning's *n* described in Equation 5 of USGS (1989). The D_{84} calculated for the portion of the engineered streambed material gradation not used in construction of the boulder weirs (1.2 feet; Section 5.3.4) was used to estimate Manning's *n* throughout the overbank region of the fish passage ramp.

- **Riprap** land cover type was assigned to the riprap-lined banks of the proposed fish passage ramp. Manning's *n* was calculated using the Manning-Strickler equation presented in Chang (1992), which relates D_{50} to Manning's *n*. The maximum D_{50} for 1/2-ton riprap (2.3 feet) based on NCHRP (2006) riprap gradations was used to calculate Manning's *n*.
- **Access road** land cover type was assigned to the proposed foundation rock access road that runs along the north bank of the channel and provides access to the proposed weir. The access road will be comprised of material similar to the riprap slope lining, and, thus, was assigned the same Manning's *n*.

Manning's *n* values used for the roughened ramp and their associated descriptions are provided in Table 4-1.

Table 4-2. Manning's Roughness Values Used for Proposed Condition 1D HEC-RAS Model Only

Land Cover Type	Source	Description	Manning's *n*
Riprap/Access Road	Chang (1992)	Calculated value for 1/2-ton riprap (d_{50} = 2.3 ft) using Manning-Strickler equation.	0.045
Roughened Channel, low flow region – engineered streambed material with boulder weirs (maximum 2.5-foot protrusion from low flow channel invert)	USGS (1989)	Calculated *n* using Limerinos relationship with D_{84} = 1.6 ft based on D_{100} calculated for engineered streambed material.	0.052
Roughened Channel, overbank region– engineered streambed material with rock clusters (maximum 1.5-foot protrusion from overbank invert)	USGS (1989)	Calculated *n* using Limerinos relationship with D_{84} = 1.2 ft based on D_{84} calculated for portion of the engineered streambed material gradation not used in construction of the boulder weirs.	0.048

Figure 4-4 through Figure 4-6 show representative cross-sections for the proposed weir and roughened channel. Cross-sections for existing and proposed conditions are provided in Appendix A.

Figure 4-4. HEC-RAS Cross-Section for Proposed Weir Opening

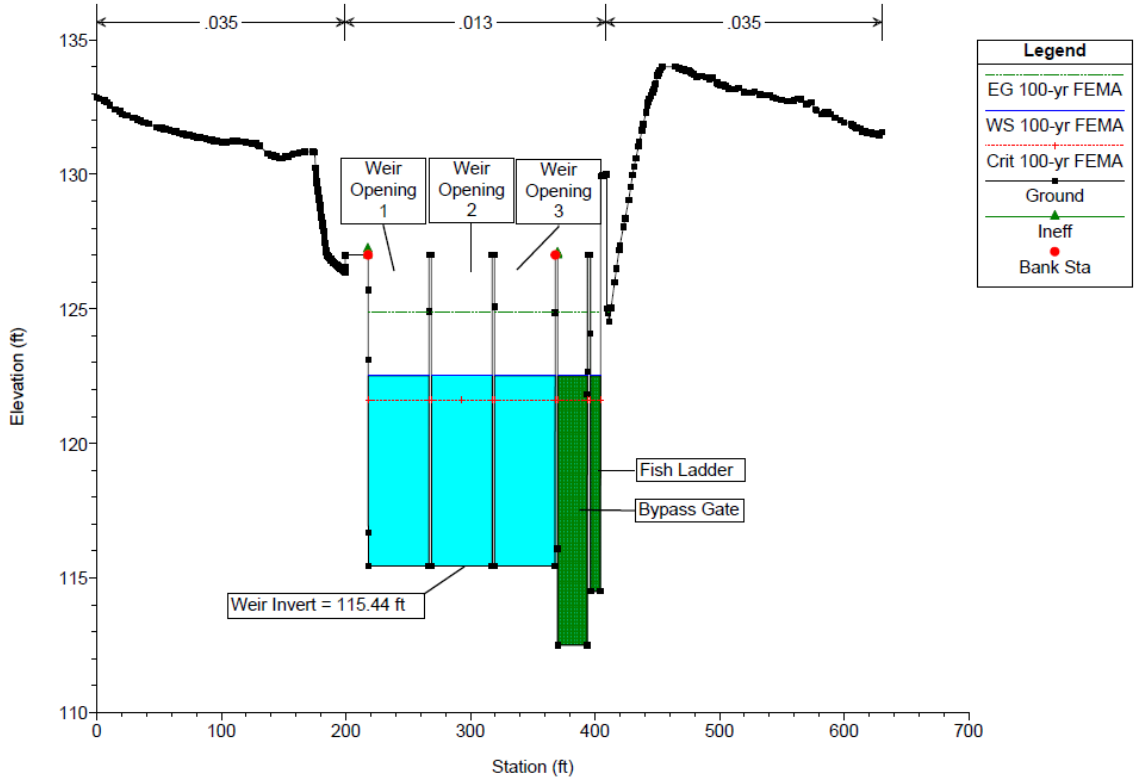


Figure 4-5. HEC-RAS Cross-Section for Proposed Weir Sill

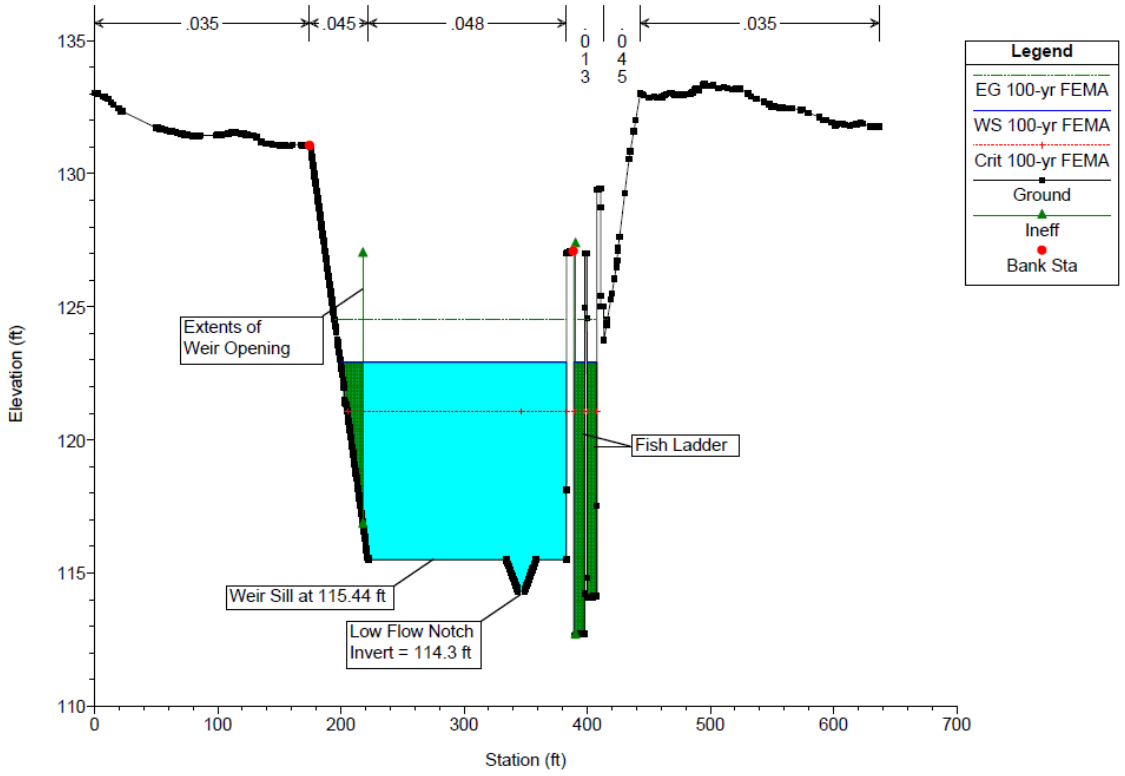
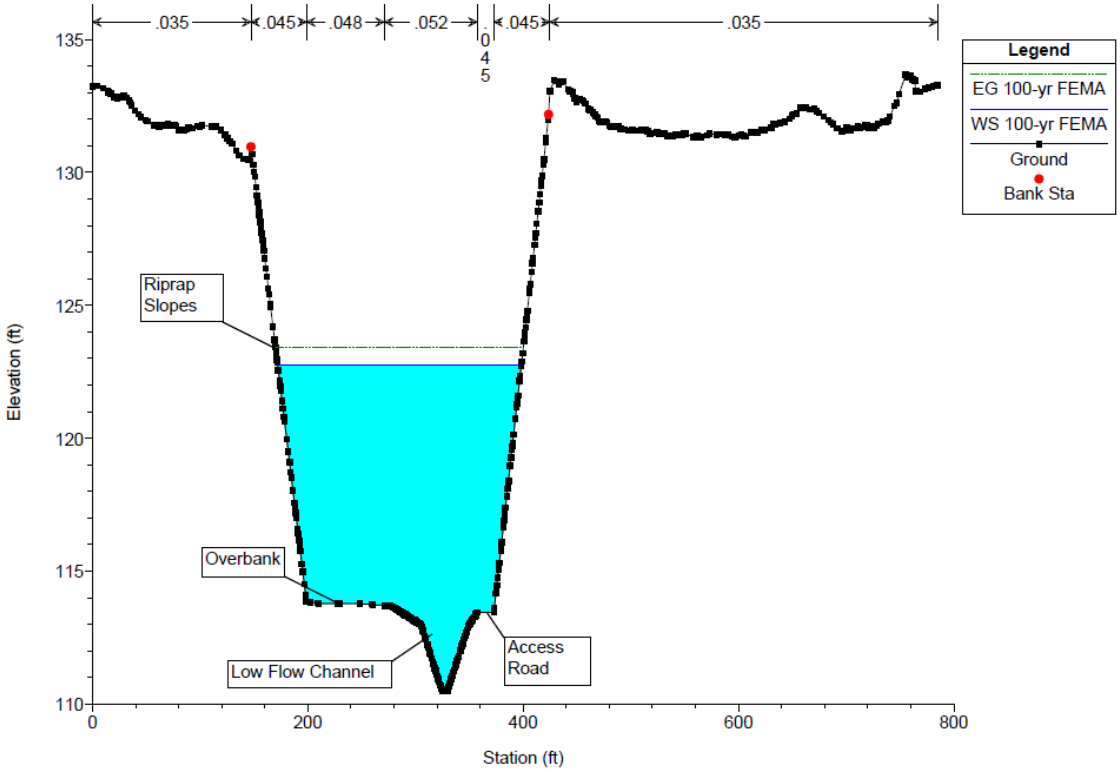


Figure 4-6. Representative HEC-RAS Cross-Section for Proposed Roughened Channel



4.1.7 No-Rise Results

A No-Rise condition is achieved along most of the study reach, except for some localized increases observed around the weir and roughened fish passage ramp. Localized increases were confined to an approximately 590-foot reach extending from the proposed weir (HEC-RAS reach station 2171) to the downstream limit of proposed fish passage ramp (HEC-RAS reach station 1582). Increases in the water surface along the roughened fish passage ramp were generally between 0.01 – 0.3 foot. Larger increases ranging from approximately 0.5 - 1.1 feet were observed through the weir structure and within the upstream 20 feet of the fish passage ramp. The larger increases through the weir are likely due to proposed weir extending approximately 20 feet downstream of the end of the existing weir, resulting in overlap between the dammed water surface upstream of the proposed weir sill and the existing weir tailwater. Localized increases downstream of the weir are a result of the increased tailwater elevation caused by the proposed roughened channel. At all other cross-sections, the proposed water surface is lower than, or equal to the existing water surface profile. Additionally, both the existing and proposed water surface profiles are below the FEMA effective water surface within the portion of the study reach for which FEMA effective base flood elevations were available (Figure 4-7). WSEs from cross-sections at and surrounding the existing and proposed structures are provided in Figure 4-8 and Table 4-3.

Figure 4-7. Existing and Proposed FEMA 100-year Water Surface Profiles and FEMA Effective Water Surface Elevation at FEMA Cross-Sections within the Study Reach

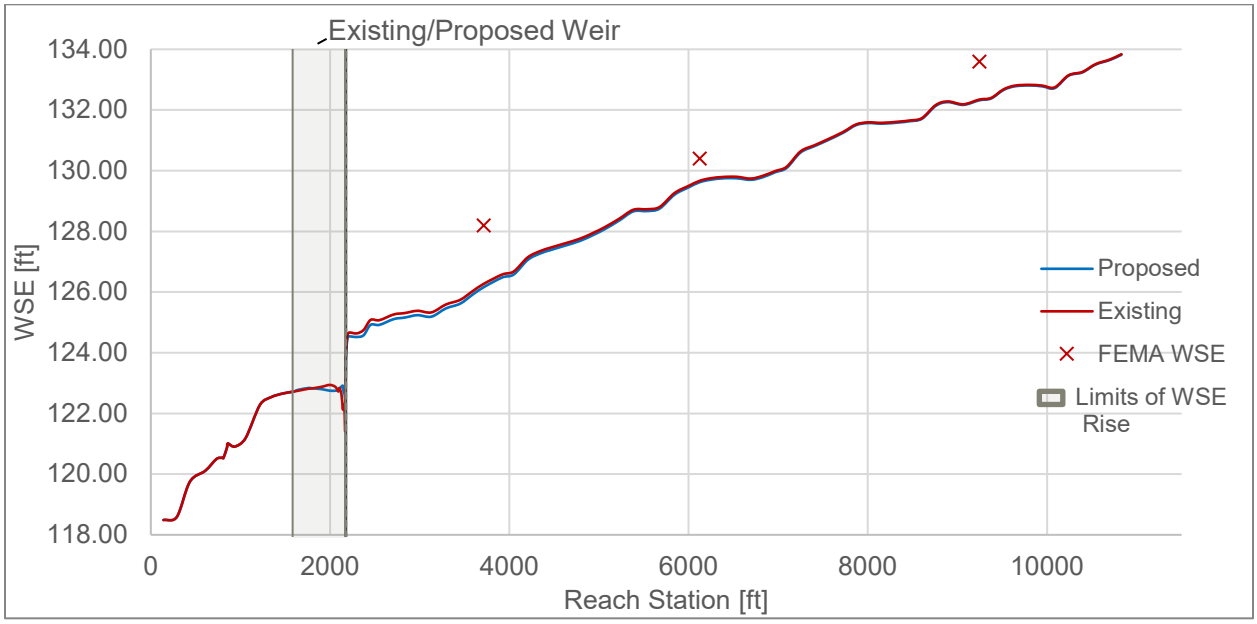


Figure 4-8. Existing and Proposed FEMA 100-year Water Surface Profiles near the Weir

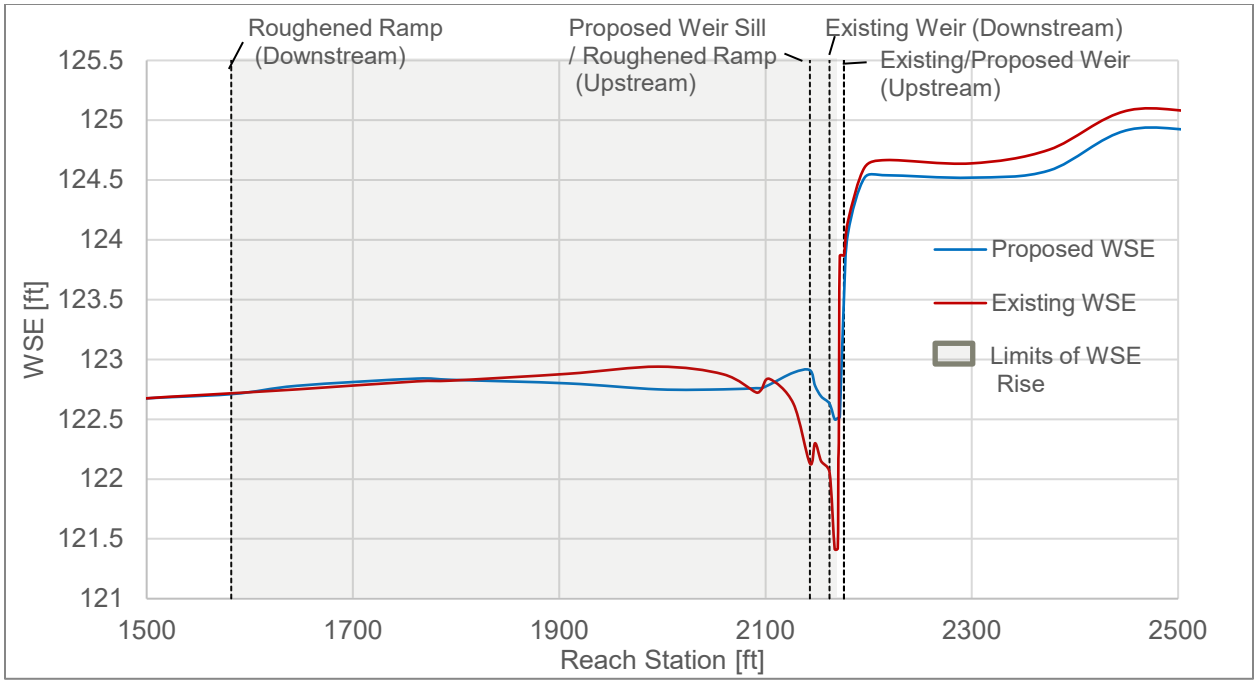


Table 4-3. Hydraulic Analysis Results, Cross-Sections 2550 - 1219

Cross-Section Location	Existing Station (ft)	Existing WSE (ft)	Proposed WSE (ft)	WSE Difference (Proposed - Existing) (ft)
	2550	125.07	124.92	-0.15
	2452	125.08	124.92	-0.16
Upstream Limit of Proposed Channel Modifications	2375	124.75	124.58	-0.17
	2299	124.64	124.52	-0.12
	2217	124.67	124.54	-0.13
	2196	124.61	124.52	-0.09
	2180	124.16	124.06	-0.10
Existing Weir - Upstream; Proposed Weir - Upstream	2176	123.87	123.51	-0.36
	2172	123.87	122.52	-1.35
	2171	122.30	122.51	0.21
	2170.6	122.17	122.51	0.34
	2170	121.42	122.51	1.09
	2167	121.42	122.50	1.08
Existing Weir - Downstream	2162	122.06	122.63	0.57
	2154	122.15	122.69	0.54
	2148	122.30	122.78	0.48
Weir Sill/Upstream limit of Roughened Fish Ramp	2143	122.13	122.91	0.78
	2127	122.63	122.89	0.26
	2104	122.84	122.79	-0.05
	2096	122.75	122.76	0.01
	2091	122.72	122.76	0.04
	2059	122.88	122.75	-0.13
	1996	122.94	122.75	-0.19
	1908	122.88	122.80	-0.08
	1795	122.82	122.83	0.01
	1762	122.82	122.84	0.02
	1646	122.75	122.78	0.03
Downstream Limit of Roughened Fish Ramp	1582	122.72	122.71	-0.01
Downstream Limit of Proposed Channel Modification	1474	122.66	122.66	0.00
	1343	122.54	122.54	0.00
	1219	122.29	122.29	0.00

Notes: Light gray shading indicates a WSE increase from existing condition.

4.1.8 No-Rise Discussion

Results from the 1D analyses indicate that the water surface for the FEMA 100-year flood event are lower than the FEMA effective base flood elevations (Appendix C). A No-Rise condition is generally achieved along the entire study reach, except for a few localized increases in WSE at the proposed weir and along the proposed fish passage ramp. The water surface was not affected downstream of the proposed channel modifications, and the water surface upstream of the proposed weir was lower under proposed conditions than under existing conditions. Within the reach that experienced an increase in the 100-year water surface under proposed conditions, the proposed WSE was contained within the channel and did not exceed the vertical extents of the riprap lining. Furthermore, there are no insurable structures at these locations and localized increases in water surface are unlikely to pose increased flooding risk along the study reach. A Conditional Letter of Map Revision may be required if the San Joaquin County Floodplain Administrator considers the localized increases in WSE within the Project area to be unacceptable. Hydraulic results will be refined as the design progresses.

4.2 Two-Dimensional Hydraulic Analysis

A 2D hydraulic analysis was performed for the proposed condition using the USACE's HEC-RAS version 6.1. Results from the 2D analysis were used to inform design, including evaluation of fish passage performance, and design of the rock material that will line the slopes of and serve as the foundation for the fish passage ramp. The 2D hydraulic model consisted of several components:

- A model domain to define the area for which hydraulic calculations are performed
- A computational mesh to define the hydraulic properties and spatial resolution of calculations within the model domain
- Boundary conditions to provide the necessary information to initiate hydraulic calculations across the computational mesh

4.2.1 Model Domain

The model domain extends from approximately 0.4 mile upstream of the Bellota Weir to approximately 360 feet downstream of the Escalon Bellota Road Bridge (0.3 mile downstream of the Bellota Weir). The surrounding area is comprised primarily of agricultural land use. Exhibit B5, in Appendix B, shows the boundary of the model domain.

All elements within the HEC-RAS 2D model domain were georeferenced to the North American Datum of 1983, California State Plane coordinate system, Zone III. Vertical data is referenced to NAVD88. Horizontal and vertical units are in U.S. survey feet.

4.2.2 Computational Mesh

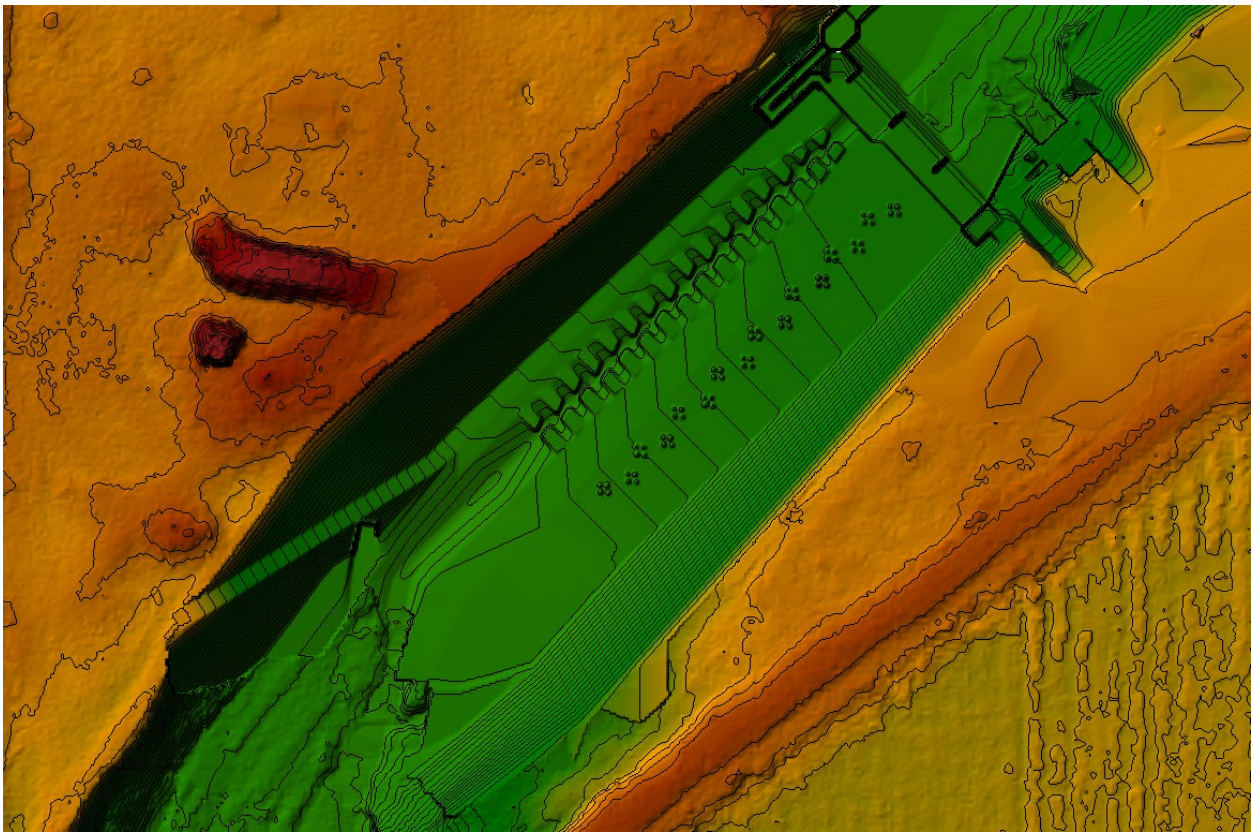
HEC-RAS 6.1 uses a finite-volume scheme to numerically solve the Full Momentum Saint-Venant Equations for flow depths and velocities across an irregular computational

grid (mesh). The mesh divides the model domain into discrete areas (cells) for which hydraulic properties based on terrain and Manning's roughness data are calculated and hydraulic computations performed. A flow depth is calculated at the center of each cell, while flow velocities are calculated at each cell face. The grid size (resolution) of the mesh defines the spatial (horizontal) interval at which calculations are performed. A 20-foot mesh resolution was generally used within the channel; resolution was reduced to 5 feet near the weir (approximately 50 feet upstream). A mesh resolution of 3 to 5 feet was used to represent features at the weir and along the fish passage ramp. Mesh cells were oriented to align with the general direction of flow and define significant features (e.g., channel banks, weir gates, weir sill, fish ladder, boulder weirs, rock clusters). Computations were performed at a variable time-step (maximum time-step = 4 seconds; minimum time-step = 0.063 second) governed by the Courant number, which is the ratio of the computational time-step to the flow residence time for a given computational grid-cell. Exhibit B6, in Appendix B, shows the computation mesh within the Project area.

Terrain

Terrain data defines the elevation of each mesh cell and cross-sectional geometry at each cell face. This information is used to calculate relationships between depth and the area and volume at each cell face. The existing conditions terrain developed for the 1D HEC-RAS model (see Section 4.1.5) was used as the base terrain for the 2D HEC-RAS model. A separate terrain was developed for the proposed weir and fish passage features and layered on top of the existing terrain to create a composite proposed terrain for the 2D model domain. Unlike with the 1D HEC-RAS model, the weir boulders and rock clusters were represented as terrain features. Boulder weirs were represented as simple blocks of raised terrain rather than as individual boulders to produce the desired hydraulic impact of the weirs without needing to determine currently unknown details such as number of boulders per weir, lateral boulder spacing, gradation of materials between boulder spaces. Figure 4-9 shows the terrain developed for the proposed weir and fish passage features used in the 2D hydraulic analysis.

Figure 4-9. Proposed Terrain Developed for the Project Area



Manning's *n* values

Manning's *n* values associated with land cover are used to calculate friction losses throughout the model domain. The same land cover types used for the 1D proposed HEC-RAS model were used for the 2D model; however the Manning's *n* values associated with each land cover type were adjusted to accommodate for energy loss caused by turbulence that is accounted for in the 2D hydraulic calculations, and to produce sufficiently conservative velocities and depths for riprap sizing and evaluation of fish passage performance. The following adjustments were made to Manning's *n* values in the 2D model:

- **Existing channel and overbanks** were assigned Manning's *n* values from the 1D model; however, Manning's *n* values were lowered to the minimum, and average of minimum and normal values for the same channel type from Chow (1959).
- **Concrete Weir** was assigned the lowest Manning's *n* value listed for Lined or Constructed Channels in Chow (1959).
- **Fish Passage Ramp, engineered stream bed material with boulder weirs/rock clusters** were assigned Manning's *n* values based on a Depth-Manning's *n* rating curve developed by NHC (2020) for a similar structure designed as part of the Nelson Dam Removal Project. NHC (2020) calibrated the rating curve based on the results of a physical model and 2D numerical hydraulic model; it is applicable for the

engineered streambed material (without boulder weirs or rock clusters) at depths from 0 to 10 feet. Roughness along the fish passage ramp was divided into the same two regions used for the proposed 1D HEC-RAS model. Due to significant longitudinal variation in depth along the fish passage ramp, the depth halfway down the fish passage ramp (approximately 200 feet downstream of the weir sill) was selected as the representative depth to determine Manning's *n* for each roughness region. The lowest Manning's *n* provided by the rating curve (*n* = 0.035) was assigned across the entire fish passage ramp for scenarios used in design of foundation rock and riprap (100-year, 50-year, 25-year, and 10-year discharges) to produce conservative velocities. The boulder weirs and rock clusters impact on energy loss was assumed to be accounted for in the 2D hydraulic calculations.

A summary of the Manning's roughness coefficients used for this analysis is provided in Table 4-4. Table 4-5 provides the rating curve developed by NHC (2020) for the Nelson Dam Removal Project.

Table 4-4. Manning's Roughness Values Used for 2D Proposed Condition Model

Land Cover Type	Source	Description	Manning's *n*
Channel, earthen with minor/moderate vegetation	Chow (1959)	Main channel at full stage; clean, straight, no rifts or deep pools, with some stones and weeds (minimum value)	0.025
Channel and overbanks, heavy brush/grass/trees	Chow (1959)	Excavated or Dredged Channels: not maintained; weeds and brush uncut, clean bottom with brush on sides, highest stages of flow (average of minimum and normal value)	0.060
Concrete	Chow (1959)	Minimum value for any lined channel	0.010
Riprap	Chang (1992)	Calculated value for 1/2-ton riprap (dumped rock, d_{50} = 2.3 feet) using Manning-Strickler equation.	0.045
Roughened Channel – Boulder Weirs with maximum 2.5 feet protrusion from channel bed	NHC (2020)	Depth-Manning's rating curve developed using a physical model for a structure similar to the Bellota Weir Fish Passage ramp	0.035 – 0.060
Roughened Channel – Rock Clusters with maximum 1.5 feet protrusion from channel bed	NHC (2020)	Depth-Manning's rating curve developed using a physical model for a structure similar to the Bellota Weir Fish Passage ramp	0.035 – 0.060

Table 4-5. Depth-Manning's *n* Rating Curve developed by NHC (2020) for the Nelson Dam Removal Project

Depth (ft)	Manning's *n*
< 1.0	0.060
2.0	0.054
4.5	0.044
7.0	0.037
10.0	0.035

Table 4-6. Manning's Roughness Values for the Roughened Ramp based on NHC (2020) depth-varied Manning's *n* rating curve

Scenario	Discharge (cfs)	ESM with Boulder Weirs		ESM with Rock Clusters	
		Representative Depth (feet)	Manning's *n*	Representative Depth (feet)	Manning's *n*
FEMA 100-year	12,690	>10	0.035	8.0	0.035
FEMA 50-year	12,500	>10	0.035	8.0	0.035
USACE 25-year	10,640	9.0	0.035	6.5	0.035
USACE 10-year	9,530	8.5	0.035	6.0	0.035
High Fish Passage (migration)	1,735.2	4.5	0.041	1.2	0.054
5% Annual Exceedance	545.3	3.5	0.046	<1.0	0.060
High Fish Passage (irrigation)	150.9	3.0	0.049	<1.0	0.060
Low Fish Passage	20	<1.0	0.060	<1.0	0.060

ESM = engineered streambed material

4.2.3 Boundary Conditions

The finite-volume scheme used to solve for flow depths and velocities requires user-defined boundary conditions (i.e., hydrograph, normal depth) at the upstream and downstream boundaries of the model domain to initiate iterations and produce a final solution. A discharge hydrograph condition was selected at the upstream boundary, which extended from the high point on the southern channel bank to the high point on the eastern channel bank at the upstream end of the model domain. Discharge was distributed across the upstream boundary based on conveyance. A normal depth condition was selected at the downstream boundary, which extended from the high point on the southern channel bank to the high point on the northern channel bank at the downstream end of the model domain.

Discharge Data

Throughout the design process, numerous flow scenarios were considered for the fish passage ramp. Design flows regarding design of the low flow channel and weir sill, and fish passage ramp foundation rock and riprap are described in Section 5.2.

Normal Depth

HEC-RAS requires a user-defined energy slope to calculate normal depth for a given discharge. The energy slope at the downstream boundary was approximated as the slope of the channel invert. An average invert slope of 0.003 feet/feet was estimated from an elevation profile extracted from the terrain at the downstream boundary.

5 Roughened Channel Hydraulic Design

5.1 Design Overview

Hydraulic modeling results were used to design the proposed roughened channel. Through an iterative process, the slope, shape, and configuration of the roughened channel will be developed to meet the Project fish passage hydraulic objectives. The roughened channel is also integrated with adjacent Project features and proposed improvements to the channel area. During 35 percent design, the roughened channel sill and low flow channel were configured using normal depth calculation and 1D modeling. The initial slope, shape, and configuration were confirmed and iterated with 2D modeling for 65 percent design. Design parameters for the roughened channel foundation material and slope protection were also developed using 2D hydraulic modeling results for 65 percent design. For 90 percent design, foundation rock and slope protection design were refined, and a gradation was developed for the engineered streambed material layer based on 2D hydraulic modeling results revised to reflect an updated representation of the boulder weir geometry.

The roughened channel fishway and channel spanning weir crest was designed to conform to several performance and engineering criteria. Engineering criteria target the overall stability and resilience of the structure. Performance criteria target the biological performance and periodic costs associated with maximum acceptable damages that require modification or repairs over time. The various criteria and performance targets used to establish the overall design of this feature are detailed in the Draft Design Criteria Report (HDR 2021). Below are some of the key criteria:

- The structures (weir, channels, and high flow sections) will be designed to withstand, and provide sufficient conveyance for flood events up to the 100-year flood event (12,690 cfs).
- Maintain the minimum operational water surface of 115.4 feet upstream of the sill for the low flow discharge during irrigation season (HDR 2021).

- Limited damage that may occur during low frequency events (high flows) will not impede the structures' ability to achieve fish passage or surface water withdrawal performance criteria.
- Damage will be limited to surface scour of rock matrix, including loss of fine materials, and minor degradation of tailwater control (i.e., facility will be designed such that periodic adjustments in surface composition do not inhibit irrigation supply or fish passage, or negatively affect the Escalon Bellota Road Bridge).
- Dislodgment of large structural rocks will be limited to small areas where storm events create conditions that cause isolated areas of instability.
- Accumulation and removal of finer sediments and cobbles on the surface layer above the armor layer will be transitional and vary over time.
- The structure configuration and composition will be designed to meet the Project's fish passage performance objectives.

Fish passage performance objectives of the roughened channel will be continuous and met throughout the range of anticipated migration flows identified for various target species and life stages. Specific fish passage design criteria include:

- Provide a minimum depth of 0.9 feet for the low fish passage design discharge through the high fish passage design discharge (HDR 2021).
- Provide adequate velocity refugia for the high fish passage design discharge (HDR 2021).

5.2 Roughened Channel Low Flow and Concrete Sill Design

The low flow channel of the roughened ramp was designed to meet fish passage and operational hydraulic criteria. The low flow channel is characterized by exposed structural rock, engineered streambed material, boulder clusters, and natural debris deposits. The low flow notch at the concrete sill was designed with the same geometry as the low flow channel. The concrete sill at the upstream end of the roughened channel serves to retain the roughened channel material and provide a minimum depth of 24 inches above the adjacent weir crest. The water surface through the notch during low flows is controlled by the low flow channel and boulder weirs.

5.2.1 Design Discharges

A range of discharges were used to evaluate the proposed design as it relates to fish passage and operational criteria. As previously mentioned, fish passage design discharges were defined per NMFS (2011) guidelines (Section 3.1.3). However, given the range of fish passage design discharges and the magnitude of the flood discharges, a roughened ramp is not suited to meet fish passage hydraulic criteria at flows as low as 3 cfs. For the 65 percent design, the low flow section was sized to meet fish passage criteria for 20 cfs. According to SEWD, the irrigation season low flow is also 20 cfs (SEWD, KSN, and HDR 2020). Therefore, the sill, roughened ramp, and low flow

channel were configured to meet design criteria for discharges from 20 cfs to 1,735.2 cfs (Table 5-1).

Table 5-1. Roughened Channel, Low Flow Channel, and Notch Design Discharges

Scenario	Discharge (cfs)
High Fish Passage (migration season)	1,735.2
5% Annual Exceedance	545.3
High Fish Passage (irrigation season)	150.9
Low Fish Passage (irrigation season low flow)	20

5.2.2 Final Configuration

Multiple sill, low flow channel, and boulder weir configurations were tested using the 2D HEC-RAS model (Section 4.2) to meet the hydraulic criteria. The final configuration includes a notch depth of 1.2 feet (invert elevation = 114.3 feet), a notch/low flow channel bottom width of 5 feet, and notch/low flow channel side slopes of 8H:1V (Figure 5-1). The roughened channel and low flow channel extend approximately 17 feet downstream from the sill at a constant elevation of 114.3 feet prior to sloping downstream at an approximate 3 percent slope. This initial horizontal portion of the roughened channel provides a protective buffer against degradation at the downstream end of the weir by distancing the invert grade break beyond the sill transition. Weir boulders are located longitudinally along the both sides of the low flow channel at approximate 8-foot intervals. The weirs on the left and right side of the low flow channel are spaced to create an effective low flow bottom width of 3 feet; the weirs are laterally staggered for the 3-foot opening alternate between the left and right sides of the low flow channel invert. Figure 5-2 depicts the water surface at the sill for fish passage design discharges. Appendix D provides the HEC-RAS 2D velocities where fish passage depth criteria is met within the Project area for all design discharges.

Figure 5-1. Typical Roughened Channel Section with Weir Boulders

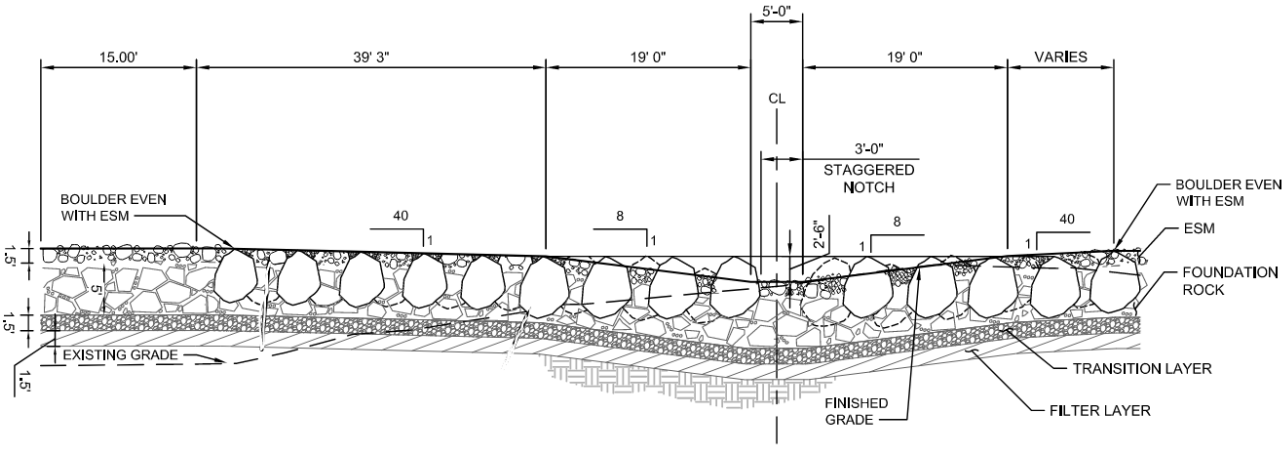
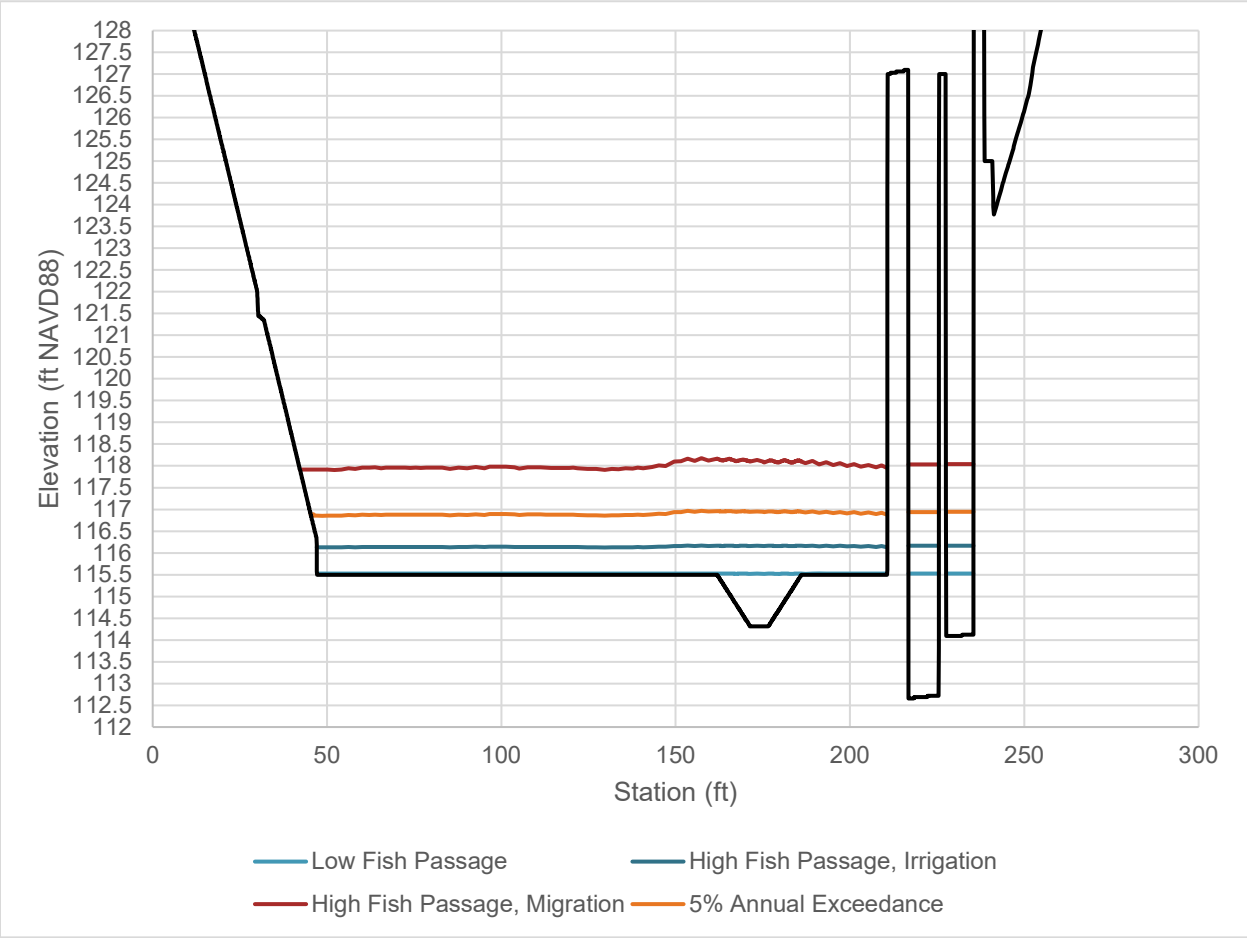


Figure 5-2. HEC-RAS 2D Water Surface for Fish Passage Design Flows at Roughened Channel Sill



5.3 Rock Material Sizing

The roughened channel slope, shape, and configuration must remain intact to function as intended. If the material comprising the roughened ramp is displaced or mobilized in an unintended manner, the hydraulic performance may be degraded and require maintenance beyond that identified as acceptable in Section 5.1. There are three primary mechanisms for degradation of the roughened ramp. First, the foundation rock, which constitutes the primary material of the channel, is displaced from the surface due to high flow hydraulics. This can affect the shape, configuration, and grade. Second, the underlaying layer, consisting of the natural streambed or placed fill, can be degraded by the winnowing or piping of fines up through the structural rock. This loss of substrate can lead to settling of the roughened channel, affecting the shape and grade. Finally, degradation at the downstream end, due to a variety of potential scour mechanisms, could cause mobilization of the roughened channel material and/or result in a perched condition not conducive to swim-through migration.

To guard against these three degradation mechanisms and support desired passage hydraulics, the typical roughened channel section was designed to include three individual layers installed above the existing riverbed material: a rock filter layer, structural foundation rock layer, and mobile bed layer composed of rounded river cobbles (engineered streambed material). Additionally, the roughened channel embankments will be protected with riprap to prevent degradation of the side slope.

Foundation rock and embankment protection sizing calculations are provided in Appendix E.

5.3.1 Design Discharges

The proposed fish passage ramp structures should be designed to withstand the 100-year discharge event. However, due to large variation in the distribution of flows across the fish passage ramp between the 100-year discharge and more frequent events, the 50-, 25-, and 10-year discharges were also evaluated to verify that the most conservative hydraulic variables are used to design the foundation and slope protection. Design discharges are summarized in Table 5-2.

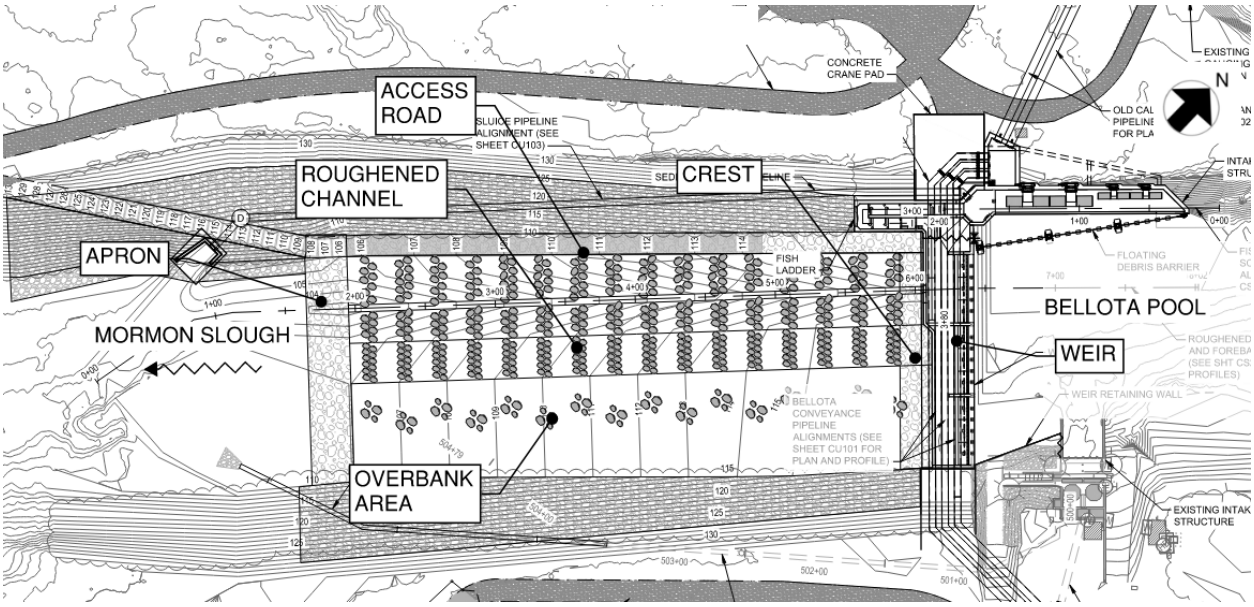
Table 5-2. Foundation Rock and Embankment Protection Design Discharges

Scenario	Discharge (cfs)
FEMA 100-year	12,690
FEMA 50-year	12,500
USACE 25-year	10,640
USACE 10-year	9,530

5.3.2 Fish Passage Ramp Foundation Rock

Results from the 2D HEC-RAS model were used to size the foundation rock for the fish passage ramp, including the crest, main channel, overbank, access road, and apron. Figure 5-3 shows a plan view of the different rock sizing zones.

Figure 5-3. Roughened Channel Foundation Rock Layer Zones



Approach

Two methodologies are recommended in the United States Bureau of Reclamation's *Rock Ramp Design Guidelines* (Reclamation 2007) for sizing riprap on rock ramps or, in this case, the foundation rock for the fish passage ramp:

- Steep Slope Riprap Design (USACE 1991)
- Abt and Johnson (1991)

All design flow scenarios were evaluated using both of the above rock sizing methods.

USACE (1991)

The USACE (1991) method for sizing riprap on steep slopes relates the bed slope and unit discharge to rock size using the following equation:

$$D_{30} = \frac{1.95\, S^{0.555} q^{2/3}}{g^{1/3}}$$

Where,

D_{30} = rock diameter for which 30 percent is smaller by mass; ft

S = slope of bed; ft/ft

q = design unit discharge; cfs/ft (recommended to multiply by safety factor of 1.25 to account for flow channelization)

g = acceleration due to gravity; ft/s^2

The equation is applicable for angular rock with a unit weight of 167 lbs/ft^3 , D_{85}/D_{15} ranging from 1.7 to 2.7, revetment thickness equal to 1.5*D_{100}, bed slopes ranging from 2 to 20 percent, and no tailwater.

Abt and Johnson (1991)

The Abt and Johnson (1991) method for sizing ramp bed material is based on empirical relationships developed from flume studies. Similar to USACE (1991) the method relates bed slope and unit discharge to rock size, but also includes additional coefficients to account for uncertainties related to regression fit, flow concentration, and material type:

$$D_{50} = \phi_e \cdot \phi_c \cdot a \cdot 5.23 \cdot S_0^{0.43} \cdot q_{design}^{0.56}$$

Where,

D_{50} = median diameter of the riprap layer; ft

ϕ_e = coefficient for the empirical envelope on the regression relationship (equal to 1.2)

ϕ_c = coefficient for flow concentration due to channelization within the revetment; equal to $r_q^{0.56}$, where r_q is the ratio of unit discharge in a channel versus uniform depth unit discharge (maximum value equal to 3)

a = shape factor for rounded material (equal to 1 for angular material)

S_0 = profile slope of the rock ramp; ft/ft

q_{design} = design unit discharge; cfs/ft (recommended to multiply by safety factor of 1.35)

The flume experiments used to develop the equation included bed slopes ranging from 1 to 20 percent, with stones having coefficient of uniformity ($C_u = D_{60}/D_{10}$) ranging from 1.62 to 2.15, and geometric standard deviations ($\sigma_g = (D_{84}/D_{16})^{0.5}$) ranging from 1.86 to 5.70.

Application

The primary inputs required for both of the methods described above are unit discharge and slope.

Unit discharges were estimated at 10 cross-sections along the fish passage ramp profile. The total discharge at each cross-section along the fish passage ramp invert was obtained from the 2D HEC-RAS model results. Unit discharge was then calculated by dividing the total discharges by the width of the fish passage ramp invert. The maximum unit discharge across all 10 locations was then selected for calculation of rock material size; unit discharge was multiplied by the recommended safety factors for their respective equations. Additionally, the maximum channelization factor ($r_q = 3$) was applied for the Abt and Johnson (1991) method to account for a significant preferential pathway down the center of the ramp during high flows (Appendix D provides 2D HEC-RAS results for design discharges).

The fish passage ramp has a bed slope of approximately 3 percent; however, a backwater condition exists at the downstream end of the fish passage ramp that limits the influence of the bed slope on hydraulics over the ramp. Because both rock sizing

methods were developed under the assumption of relatively uniform flow along the profile with no backwater, energy slope was considered to be an appropriate substitute for bed slope. Energy slope was determined for eight profiles across the fish passage ramp using water surface and velocity results produced by the 2D HEC-RAS model. The maximum energy slope across all eight profiles was used for the rock sizing calculations.

This method of estimating inputs for the rock sizing equations was repeated for all design discharges.

Results

The 100-year discharge event and the Abt and Johnson (1991) method produced the most conservative D_{50} (1.8 feet) for the fish passage ramp foundation rock. Although unit discharge was generally lower near upstream end of the ramp and would allow for a smaller rock size, the maximum rock size was conservatively selected for the entire ramp (main channel, overbank, crest, access road, apron). Rock gradations were selected based on NCHRP (2006) gradations. The United States Bureau of Reclamation (2007) recommends that the thickness of the foundation rock layer be calculated as the maximum of D_{100} and $1.5*D_{50}$. The resulting thickness given the selected riprap class would be 42 inches (3.5 feet). However, a conservative thickness of 60 inches (5 feet) was selected for the main channel, overbank, crest, and access road to ensure sufficient depth for boulder embedment. For the apron, a thickness of 108 inches (9 feet) was selected to account for increased erosion potential at the downstream end of the ramp, per the long-term degradation analysis (Section 6).

Table 5-3 provides a summary of the selected D_{50} and associated NCHRP (2006) gradation.

Table 5-3. Proposed Foundation Rock Gradation and Layer Thickness

Sub-Region	Nom. D50 (in)	D15 (in)		D50 (in)		D85 (in)		D100 (in)	Blanket Thickness (in)	Weight Class	Description
		Min	Max	Min	Max	Min	Max				
Access Road, Crest, Channel, Overbank	21	13	18.5	20	24	27.5	32.5	42	60	3/8-ton	NCHRP Class VI
Apron	21	13	18.5	20	24	27.5	32.5	42	108	3/8-ton	NCHRP Class VI

5.3.3 Embankment Protection

Approach

Riprap revetment protection of the channel embankments was sized and designed according to the USACE Engineering Manual 1110-2-1601, Hydraulic Design of Flood Control Channels (1994). This method is specifically used to size rock along a channel embankment, where the embankment side slope is considered. The USACE (1994) method for sizing riprap equates stone size with local velocity using empirical relationships using the following equation:

$$D_{30} = S_f C_s C_V C_T d \left[\left(\frac{\gamma_w}{\gamma_s - \gamma_w} \right)^{1/2} \frac{V}{\sqrt{K_1 g d}} \right]^{2.5}$$

Where,

D_{30} = riprap size for which 30 percent is smaller by weight, length

S_f = safety factor (1.25); ft/ft

C_s = stability coefficient for incipient failure (0.30 for angular rock); ft/ft

C_V = vertical velocity distribution coefficient (1.0 for straight channels)

C_T = thickness coefficient (1.0 for thickness = $1D_{100}$ (max) or $1.5D_{50}$ (max)

d = local depth of flow; ft

γ_w = unit weight of water; lbs/ft^2

γ_s = unit weight of riprap; lbs/ft^2

V = local depth averaged velocity; ft/sec

K_1 = side slope correction factor

g = gravitational constant; ft/s^2

Application

The primary input is the local depth average velocity. Similar to the approach for the foundation rock, velocity was evaluated in distinct channel zones, with the right and left channel embankments identified separately. The highest velocity result within these embankment zones was identified for the 100-year and more frequent events. Under the proposed channel configuration, the 100-year event produced the highest velocities, with a peak velocity of 11.7 ft/sec. Using the highest velocity provides a conservative rock size to accommodate changes to the local hydraulic condition during a large flood event. Angular rock is assumed for increased stability on the embankment slopes, which proposes 3H:1V.

Results

The NCHRP Class V riprap gradation was selected for design based on the rock size computations (Table 5-4). This gradation is recommended for both channel embankments bounding the roughened channel area. The recommended revetment thickness on slope is typically the greater of D_{100} or $1.5D_{50}$, which is 36 inches in this instance. The embankment revetments are to be tied into the foundation rock of the roughened channel. As the foundation rock gradation is larger in size, the embankment rock can terminate at the surface of the roughened channel. The roughened channel rock will span channel bank to channel bank.

Table 5-4. Proposed Embankment Rock Gradation and Layer Thickness

Location	Nom. D50 (in)	D15 (in)		D50 (in)		D85 (in)		D100 (in)	Blanket Thickness (in)	Weight Class	Description
		Min	**Max**	**Min**	**Max**	**Min**	**Max**				
Channel Embankments	18	11	15.5	17	20.5	23.5	27.5	36	26	1/4-ton	NCHRP Class V

5.3.4 Engineered Streambed Material

Approach

The engineered streambed material (ESM) was designed based on the D_{30} calculated using the Steep Slope Riprap Design equation (USACE 1991) discussed in Section 5.3.2. The resulting D_{30} (12.8 inches; 1.1 feet) was then used to determine an ESM gradation using the following relationships recommended by the California Department of Fish and Game (2009):

$$D_{84\text{-}ESM} = 1.5\ D_{30\text{-}USACE}$$

$$D_{50\text{-}ESM} = 0.4\ D_{84\text{-}ESM}$$

$$D_{100\text{-}ESM} = 2.5\ D_{84\text{-}ESM}$$

$$D_{16\text{-}ESM} = 0.32^{1/n}\ D_{50\text{-}ESM}$$

$$D_{8\text{-}ESM} = 0.16^{1/n}\ D_{50\text{-}ESM}$$

Where,

$D_{30\text{-}USACE}$ = the D_{30} calculated using the USACE (1991) Steep Slope Riprap Design equation

$D_{x\text{-}ESM}$ = Particle diameter at which the ESM mix is x-percent finer

n = density parameter that defines the coarseness of the mix

Results

This resulting gradation consisted of a D_{100}, D_{84}, D_{50}, D_{16}, and D_8 equal to 48.0, 19.2, 7.7, 0.4, and 0.08 inches, respectively. The California Department of Fish and Game recommends that the D_{84} – D_{100} should be separated from the ESM mix and reserved for construction of large stable rock features, where applicable. Since the proposed design includes large stable rock features (boulder weirs, rock clusters), a revised ESM gradation was developed for the remaining portion of the ESM, where the D_{100} was equal to the initially calculated D_{84}.

The Washington State Department of Transportation Standard Specifications (2021) for streambed aggregates were used to determine appropriate proportions of sediment and cobbles to create a well-graded size distribution that was reasonably consistent with the calculated gradation. The resulting mixture consists of 1 part streambed sediment and 1 parts cobble with gradations as presented in Table 5-5 and Table 5-6. Calculations for the ESM are provided in Appendix E.

Table 5-5. ESM - Streambed Sediment Gradation

Size Diameter	Percent Passing
2.5-inch	100
2.0-inch	80
1.5-inch	73
1.0-inch	65
0.75-inch	50
0.187-inch	35
No. 40	16
No. 200	7

Table 5-6. ESM - Cobbles Gradation

Size Diameter	
18-inch	100
15-inch	75
12-inch	50
10-inch	40
8-inch	34
6-inch	28
5-inch	23
4-inch	20
3-inch	17
2.5-inch	14
2-inch	11
1-inch	5.4

6 Scour Analysis

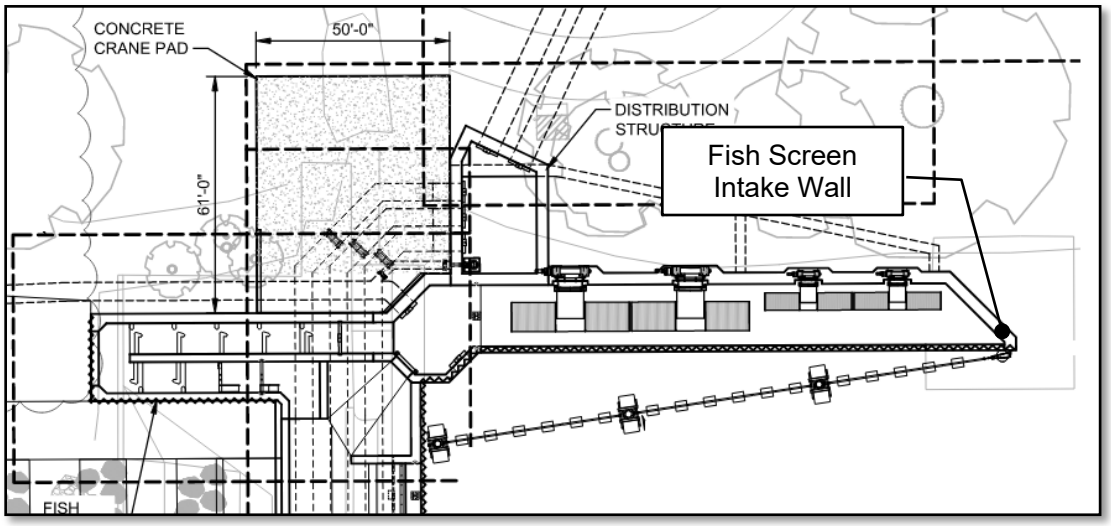
Contraction, impingement, and other applicable scour mechanisms were evaluated at appropriate locations, along with risk of bank erosion and long-term degradation. Long-term and local scour were assessed primarily by, but not limited to, the following guidelines:

- U.S. Department of Agriculture Technical Supplement 14B, Scour Calculations (2007).
- Federal Highway Administration (FHWA) Hydraulic Engineering Circular 18, Evaluating Scour at Bridges (2012).
- FHWA Hydraulic Engineering Circular 23, Bridge Scour and Stream Instability Countermeasures (2009).

6.1 Concrete Fish Screen Intake Wall

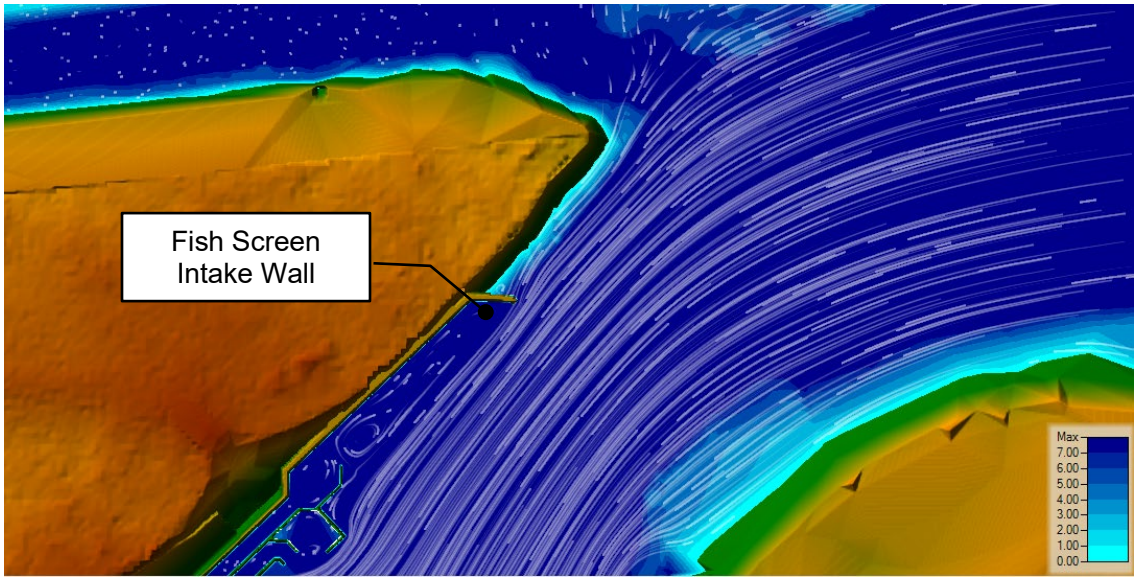
The proposed intake includes a concrete wall behind the screen assemblies, which wraps around the upstream extent of the intake apron (Figure 6-1). The upstream end of the wall extends into the inundated area of the 100-year discharge and is potentially subject to scour. The intake apron and wall are supported on piles. Sheet piling in place for construction purposes will be cut to final grade elevations and will remain along the wall face and in front of the apron, providing protection from undermining. Anticipated sheet pile depths below finish grade are approximately 20 feet. However, given the protruding nature of the wall (Figure 6-2), scour risk at the wall terminus was evaluated to assess the need for additional scour countermeasures elements.

Figure 6-1. Concrete Fish Screen Intake Wall



There were three scour mechanisms evaluated at this location. First, the channel is horizontally contracted from its normal upstream width to the width of the existing weir (Figure 6-3). This contraction of flow can cause general scour across the channel during a large flood event. Contraction scour was assessed through a comparison of hydraulics characteristics upstream of the channel contraction and at the point of interest within the contraction itself. While there is some contraction that occurs, results indicated that no scour is anticipated.

Figure 6-2. Fish Screen Intake Wall and 100-year Inundation Limits



Second, the wall terminus is located at the downstream end of a bend in the river, along the outer bank. Channel bends experience higher velocities along the outer bend, which can cause scour along the toe of the bank. Bendway scour was computed, as the effects of the channel bend could impact the wall terminus. A total bendway scour of 1.6 feet was computed.

Finally, impingement scour was assessed. Impingement occurs where flows are directed into a vertical wall or structure. The vertical Fish Screen Intake Wall intercepts upstream flow and has the potential to force flows downward in the water column, creating a localized scour condition. The scour depth is highly sensitive to the approach angle of flow, which can be variable. A 5-degree angle of approach was assumed, which yielded 10.3 feet of scour. Instead of designing the wall foundation to accommodate an anticipated scour depth, it's typical to counter the scouring effects with the use of a riprap apron.

The total scour depth anticipated is 11.9 feet (Table 6-1). While the sheet piling will remain well below this limit, a scour countermeasure at the upstream end of the intake, where flow impingement and bendway effects would be the most severe, was recommended.

Figure 6-3. Scour Mechanisms and 100-year Velocity

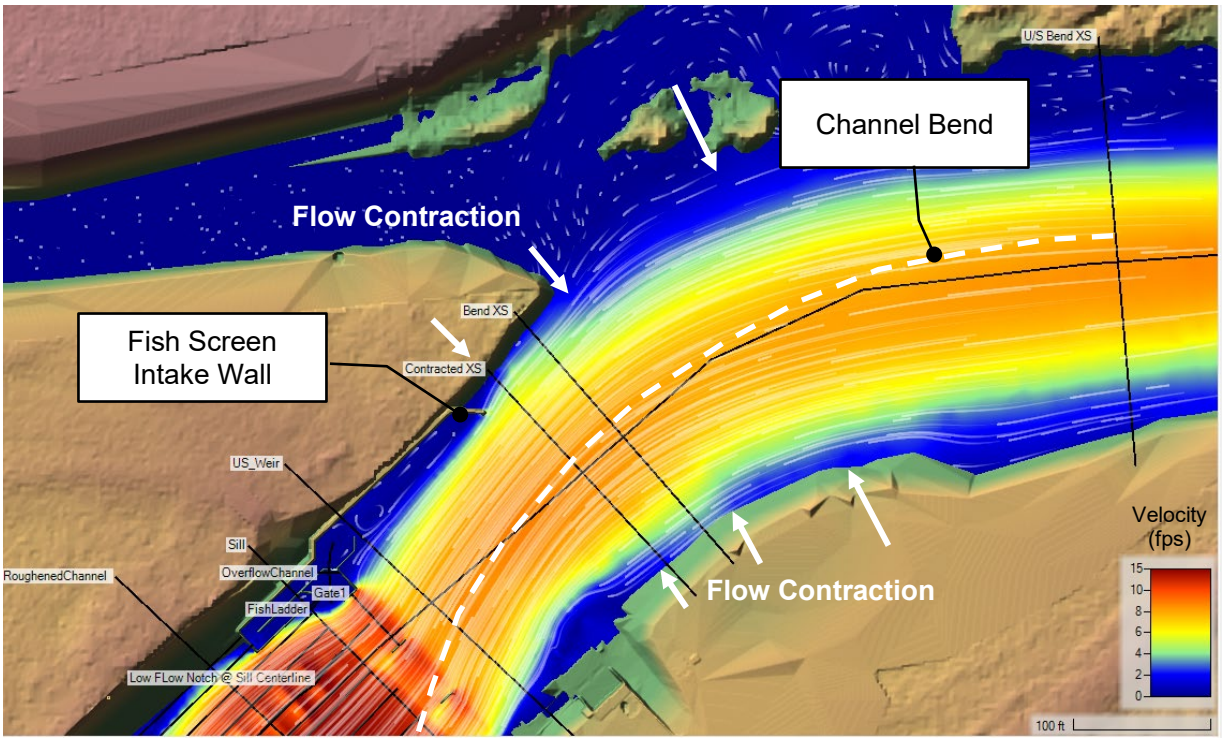


Table 6-1. Fish Screen Intake Wall Scour Potential (100-year)

Scour Mechanism	Scour Depth (ft)
Horizontal Contraction	0.0
Bendway	1.6
Impingement	10.3
Total	11.9

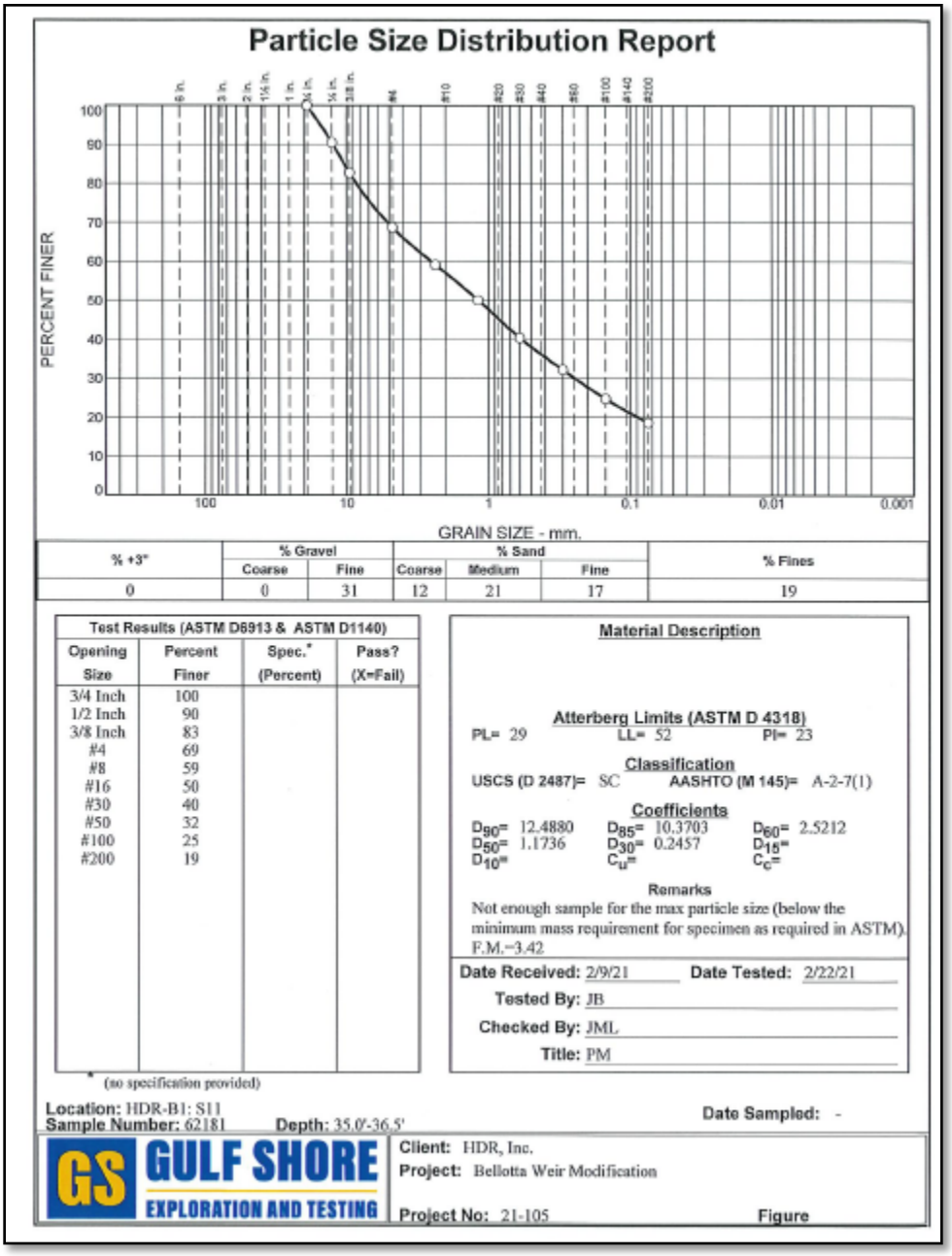
6.2 Roughened Channel Toe Down

The roughened channel is designed to be a permanent and stable feature within the main channel. The upstream end of the channel is protected by the proposed weir and sheet pile cutoff walls. The downstream end does not tie into a similar hardened feature, but rather transitions to the natural channel section of Mormon Slough. Degradation at the downstream end of any channel spanning structure is a likely mode of failure. Downstream degradation and the localized effects of slope or channel material transitions can result in a scour hole capable of undermining the structure. Therefore, the downstream end needs to be designed to accommodate the depth of this potential scour. With an in-stream, rock structure, this is typically done with a toe-down, where rock is extended to the depth of anticipated scour to prevent undermining. Two scour mechanisms were evaluated: long-term degradation, and the plunging effects of an exposed toe. These were evaluated using a1D sediment transport model based on the existing conditions 1-D HEC-RAS model.

The sediment model was run in a long-term scenario (approximately five months) with a channel-forming discharge on the order of the two-year flood event (Q_2 = 3,520 cfs), until no further bed change was observed. The bed material gradation data was obtained from boring HDR-B1 (HDR, 2021b) in the vicinity of the existing weir. Relatively fine sandy material sample (S11), with a median grain size D_{50} = 1 mm (Figure 6-4) was used to represent the entire bed gradation below the surface. Harder (clay) and coarser (gravel) layers of soil present in boring HDR-B1 above the sandy material were neglected to provide conservative scour results.

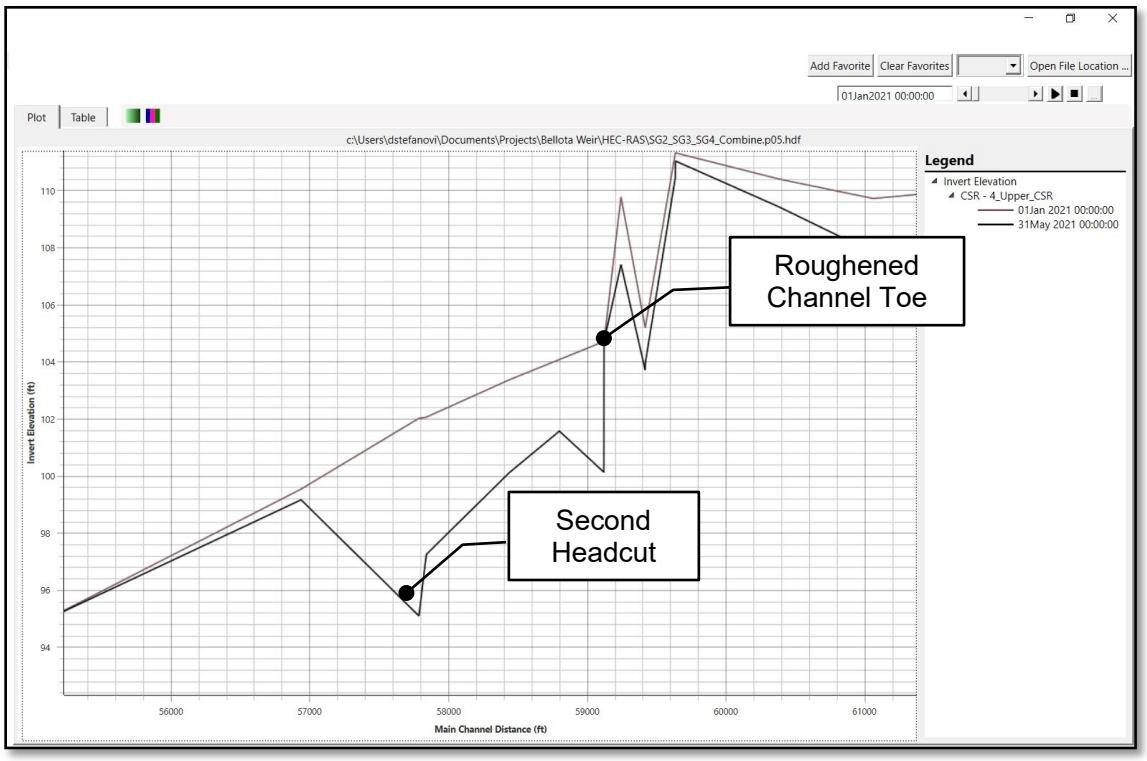
The upstream sediment load was approximated by a relatively low sediment concentration of 50 milligrams per liter (mg/l) to also maximize bed degradation. Sediment runs were based on the Yang transport function (recommended for sandy bed material) and Copeland (Exner 7) sorting method, which is known to be conservative for scour evaluation in finer systems.

Figure 6-4. Geotechnical Boring B1 with Sample S11



The long-term sediment transport results indicate maximum 5 ft of bed degradation immediately downstream of the weir (Figure 6-5) for the conservative modeling assumptions. Approximately 1000 feet downstream of the maximum bed degradation near the weir toe there is another potential headcut that may propagate farther upstream. This would cause a maximum bed degradation less than 10 feet in this worst-case scenario. Therefore, a toe down depth of 10-feet is recommended for the downstream end of the roughened channel.

Figure 6-5. Channel Bed Profile with Long-Term Bed Degradation



7 Scour Countermeasure Design

Scour countermeasures to the proposed structures were designed using the following guidelines:

- USACE Engineering Manual 1110-2-1601, Hydraulic Design of Flood Control Channels (1994).
- FHWA Hydraulic Engineering Circular 14, Hydraulic Design of Energy Dissipators for Culverts and Channels (2006).
- FHWA Hydraulic Engineering Circular 23, Bridge Scour and Stream Instability Countermeasures (2009).
- Reclamation Rock Ramp Design Guidelines (2007).
- FEMA Filters for Embankment Dams – Best Practices for Design and Construction (2011).

7.1 Fish Screen Intake Wall Riprap Apron

Given the assessed depth of potential scour at the end of the Fish Screen Intake Wall, a designed countermeasure in the form of a riprap apron has been incorporated into the proposed project. This rock apron is intended to resist the localized hydraulic effects of

the primary scour mechanism, impingement scour, and to handle any degradation due to a mild bendway scour condition.

Figure 7-1 depicts a plan view of the proposed riprap apron. During the phased construction process, the entire bank will be excavated to the depth of the intake floor (El.110' NAVD 88). The channel slope will be replaced and compacted to a 1.5H: 1V slope, per the darkened grading contours. Prior to this, the riprap apron will be constructed at the level of the intake floor and wall base. Subsequently, the apron will be partially buried below the reconstructed channel embankment. The apron will exist as a safeguard against further degradation in the event the new constructed embankment is scoured away during a high flow event.

The apron will extend 25-feet downstream and upstream of the wall terminus to ensure adequate coverage to mitigate for the local hydraulic effect of flow impingement. The upstream edge of the apron will include a thickened edge of twice the apron thickness per USACE Engineering Manual 1110-2-1601 (USACE 1994). This thickened end treatment will safeguard against the effects of bendway scour.

Rock sizing was developed utilizing methods presented in the USACE Engineering Manual 1110-2-1601 (USACE 1994). Table 7-1 summarizes the recommended riprap design parameters.

Figure 7-1. Fish Screen Intake Wall Apron

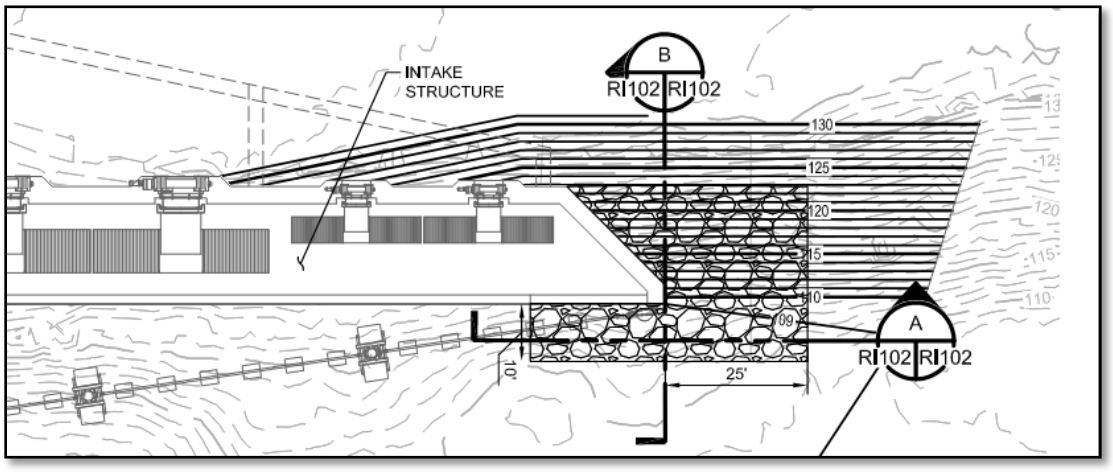


Table 7-1. Proposed Apron Rock Gradation and Layer Thickness

Location	Nom. D50 (in)	D15 (in)		D50 (in)		D85 (in)		D100 (in)	Apron Thickness (in)	Weight Class	Description
		Min	**Max**	**Min**	**Max**	**Min**	**Max**				
Fish Screen Intake Wall	12	7.3	10.5	11.5	14.0	15.5	18.5	24.0	24.0	150 lb	NCHRP Class III

7.2 Old Calaveras Outlet Structure Riprap Apron

Given the orientation of the Old Calaveras River pipelines to the Old Calaveras River, scouring velocities are expected and energy dissipation required. The Old Calaveras Outlet Structure was designed with a length to combat the expected scour length of the bend from the pipeline outlet orientation to the orientation of the Old Calaveras River (see DDR Section 7.7.2). Following this outlet structure, to provide further energy dissipation a countermeasure in the form of a riprap apron has been incorporated in the proposed project. This rock apron is intended to resist the effects of excess energy from the outlet of the Old Calaveras River pipelines.

Figure 7-2 depicts a plan view of the proposed riprap apron. During the construction process, the channel will be excavated to a depth of approximately 4 feet to a final elevation of 118.0 on each bank. This height accommodates 2 feet of freeboard over the expected channel depth. There will be a filter layer of 18-inch depth installed above existing fill, with a geotextile on top. Riprap will be installed above the geotextile with a 30-inch depth. The apron will extend 22.5 downstream of the structure.

Apron sizing was developed utilizing methods presented in the FHWA HEC 14 manual (FHWA 2006). Table 7-2 summarizes the recommended riprap design parameters.

Figure 7-2. Old Calaveras Outlet Structure Apron

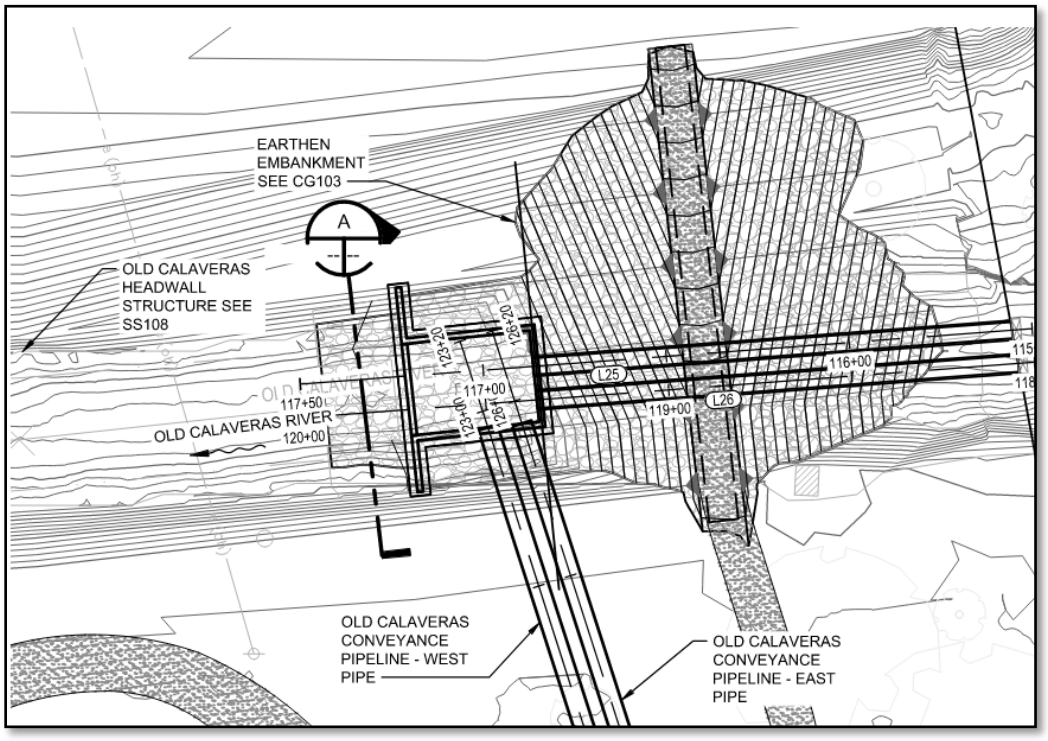


Table 7-2. Proposed Old Calaveras Outlet Structure Apron Rock Gradation and Layer Thickness

Location	Nom. D50 (in)	D15 (in)		D50 (in)		D85 (in)		D100 (in)	Apron Thickness (in)	Weight Class	Description
		Min	**Max**	**Min**	**Max**	**Min**	**Max**				
Old Calaveras River Outlet Structure	12	7.3	10.5	11.5	14.0	15.5	18.5	24.0	24.0	150 lb	NCHRP Class III

8 Next Steps and Recommendations

This report documents the current progress of the ongoing hydraulic modeling and analysis effort. The following actions are pending:

- Confirm with the San Joaquin County Floodplain Administrator that localized water surface increases within the Project area are acceptable. Continue to refresh the model based on design and surface changes.
- Complete the FEMA No-Rise Certificate, or Conditional Letter of Map Revision, if required, in preparation for delivery to the local floodplain administrator.

The Project may require authorization from USACE under 33 United States Code Section 408. A Section 408 authorization is mandated for any use or potential alteration of a USACE project. The Mormon Slough system is a federally authorized flood control facility constructed by the USACE and falls under USACE jurisdiction. USACE's Operation and Maintenance manual for the Mormon Slough flood control system is unclear as to whether USACE's jurisdiction extends to the Bellota Weir, therefore, an early determination of a potential Section 408 nexus is recommended.

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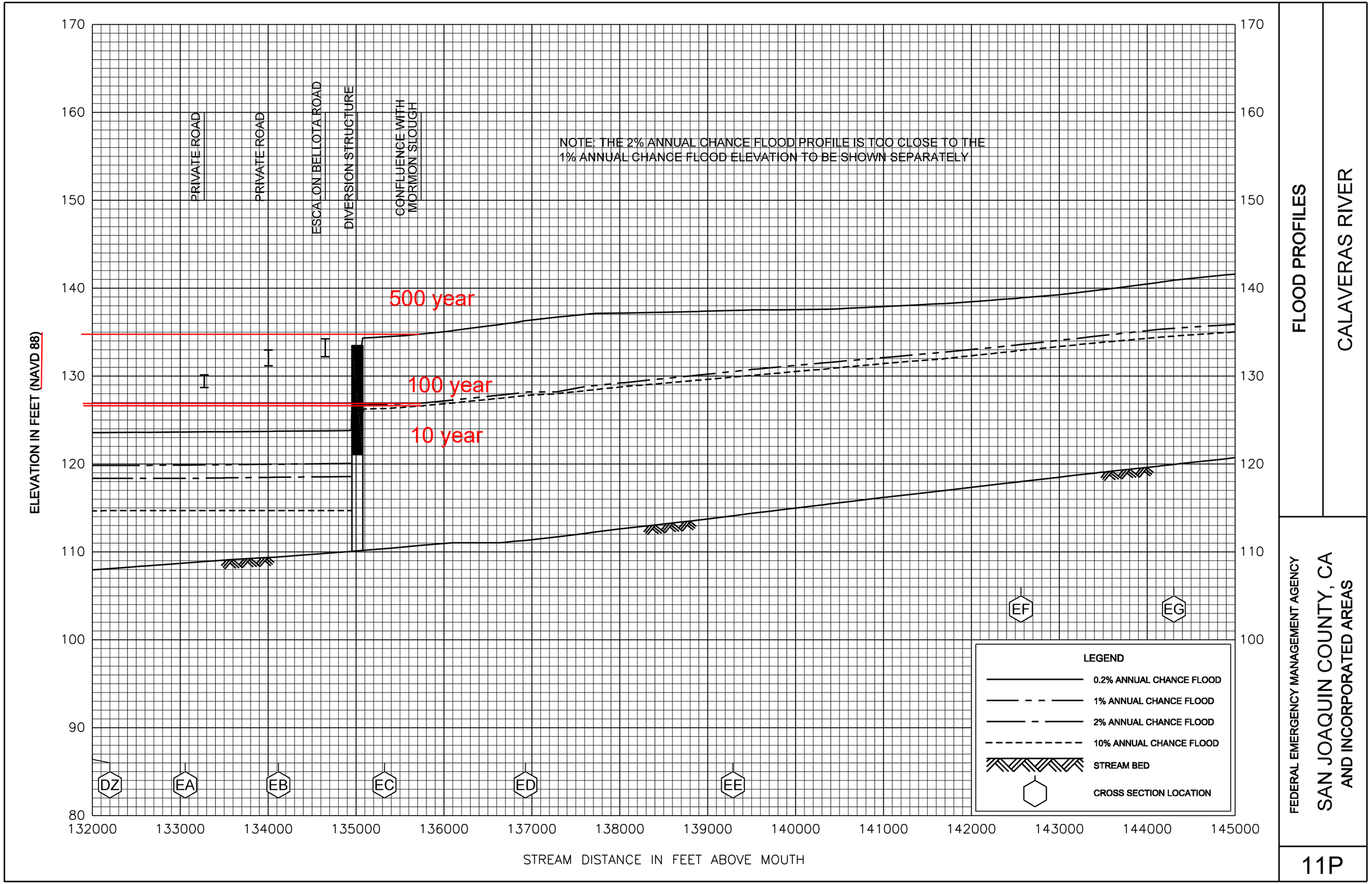
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Appendix A. FEMA FIS Data and FIRM Panels

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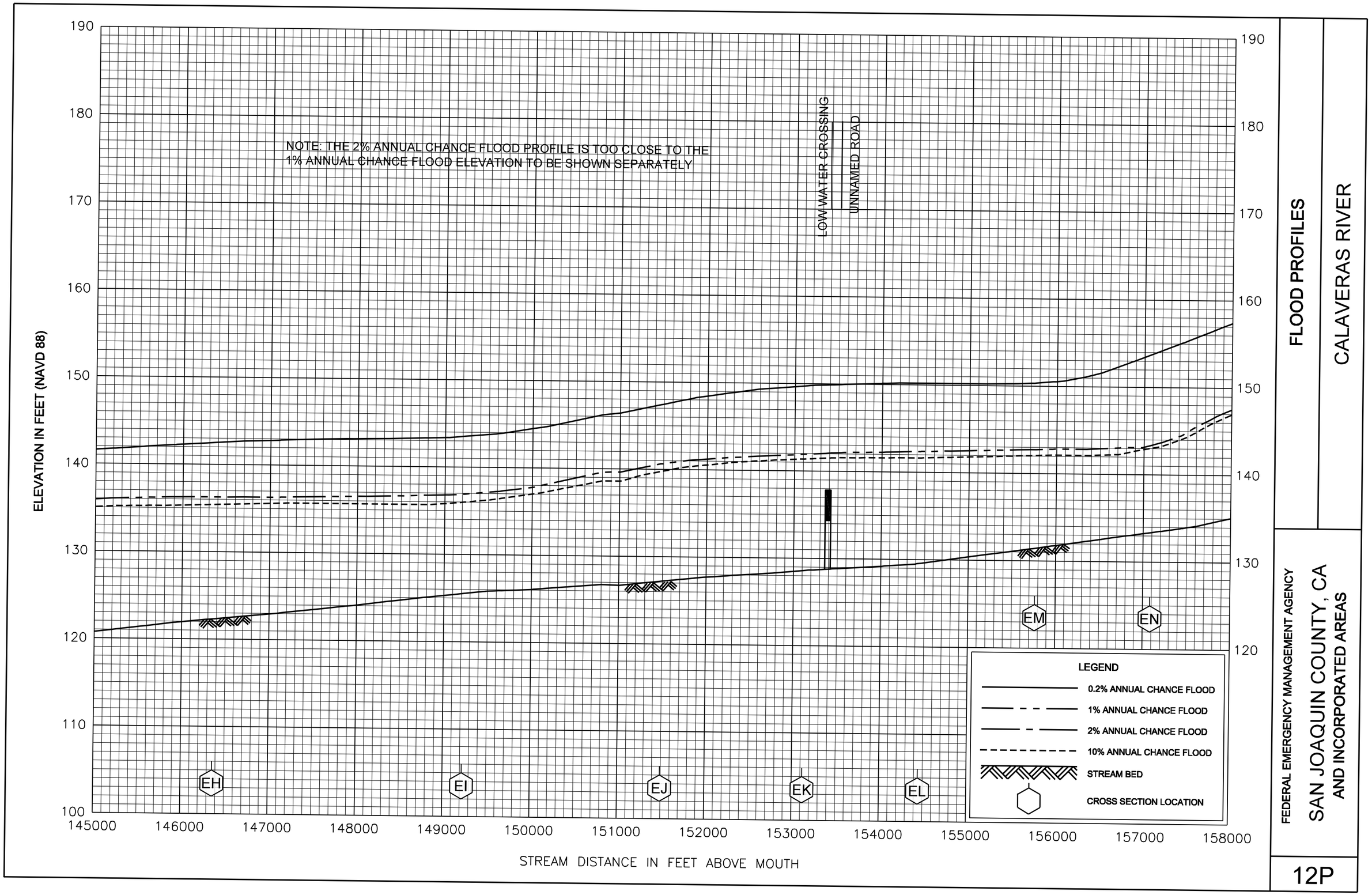
FLOOD PROFILES

CALAVERAS RIVER

FEDERAL EMERGENCY MANAGEMENT AGENCY

SAN JOAQUIN COUNTY, CA
AND INCORPORATED AREAS

11P



FLOOD PROFILES

CALAVERAS RIVER

FEDERAL EMERGENCY MANAGEMENT AGENCY

SAN JOAQUIN COUNTY, CA
AND INCORPORATED AREAS

12P

NOTE: THE 2% ANNUAL CHANCE FLOOD PROFILE IS TOO CLOSE TO THE 1% ANNUAL CHANCE FLOOD ELEVATION TO BE SHOWN SEPARATELY

SHELTON ROAD

COUNTY BOUNDARY

ELEVATION IN FEET (NAVD 88)

200
190
180
170
160
150
140
130
120
110

158000 159000 160000 161000 162000 163000 164000 165000 166000 167000

STREAM DISTANCE IN FEET ABOVE MOUTH

EO EP EQ ER ES ET EU

LEGEND

- 0.2% ANNUAL CHANCE FLOOD
- 1% ANNUAL CHANCE FLOOD
- 2% ANNUAL CHANCE FLOOD
- 10% ANNUAL CHANCE FLOOD
- STREAM BED
- CROSS SECTION LOCATION

FLOOD PROFILES

CALAVERAS RIVER

FEDERAL EMERGENCY MANAGEMENT AGENCY

SAN JOAQUIN COUNTY, CA
AND INCORPORATED AREAS

Table 10: Summary of Discharges

Flooding Source	Location	Drainage Area (Square Miles)	Peak Discharges (cfs)				
			10% Annual Chance	4% Annual Chance	2% Annual Chance	1% Annual Chance	0.2% Annual Chance
Bear Creek	Interstate Highway 5	102.5	*	*	*	7,630	*
Bear Creek	At confluence of Pixley Slough	100.7	*	*	*	7,540	*
Bear Creek	State Highway 99	84.2	*	*	*	6,730	*
Bear Creek	At confluence of Mosher Diversion	84.1	*	*	*	6,370	*
Bear Creek	At confluence of Paddy Creek	79.4	*	*	*	5,490	*
Bear Creek	At Lockeford gage	48.4	*	*	*	3,580	*
Calaveras River	Downstream of Stockton Diverting Canal[2]	614.2	*	*	*	15,818	*
Calaveras River	Upstream of Stockton Diverting Canal	23.7	*	*	*	1,159[3]	*
Calaveras River	Jack Tone Road	14.4	*	*	*	1,450	*
Corral Hollow Creek[1]	Chrisman Road	65.0	375	*	835	840	845
Corral Hollow Creek[1]	McArthur Road	64.0	400	*	1,200	1,300	1,380

Table 10: Summary of Discharges (continued)

[2]Flow is diverted from the Calaveras River to Mormon Slough at Bellota, carried by Mormon Slough to the Stockton Diverting Canal, and returned to the Calaveras River.
[3]Discharge decrease due to channel routing.
[4]Decrease in volume of 10-, 2-, and 1-percent annual chance floodflows in a downstream direction results from channel routing losses. Fluctuation in 0.2-percent annual chance floodflow results from channel routing losses and overbank gains (principally from Mormon Slough) and overbank losses.
[5]Flows for 10-, 2-, 1-, and 0.2-percent annual chance floods reflect overbank gains and losses. Source of slough is the confluence of the North and South Forks of South Littlejohns Creek.
[6]Decrease in volume of 1- and 0.2-percent annual chance floods in a downstream direction results from overbank and channel routing losses.
[7]Outflow will vary according to local conditions affecting project operation.
[8]Anabranch Channel of South Littlejohns Creek.
[9]Flows of 10-, 2-, and 1-percent annual chance floods at the upper end of the channel are estimated. Variations in flood flow downstream from Diamond Street result from storm drain diversions that return to the channel downstream and (for the 0.2-percent annual chance flood) from overbank gains and losses. The large increase in 0.2-percent annual chance flow between the upper end of the channel and State Highway 99 results from Mormon Slough inflow via the Duck Creek system.
[10]Volume of flow decreases in the downstream direction because of channel routing losses (1-percent annual chance flood) or overbank losses (0.2-percent annual chance flood).
[11]Controlled release.
[12]Decrease in 2-percent annual chance floodflow results from overbank losses. Fluctuation in 1- and 0.2-percent annual chance floodflows reflects overbank losses and gains, including overland inflow from South Fork South Littlejohns Creek.
[13]Volume of flow decreases in a downstream direction because of overbank losses and distributary-type flows from the study reach.
[14]Some of the drainage areas are estimates.
[15]Flows shown reflect overbank gains and losses and (for 0.2-percent annual chance flood) tributary inflow.
[16]Discharge increase due to incoming flow from overtopping right bank.
[17]Discharge decrease due to channel routing.
[18]Also known as Tempo Creek.
[19]Fluctuations in flows shown result from overbank losses and gains, including overland inflow from the upper reaches of Lone Tree Creek.

Table 10: Summary of Discharges (continued)

Flooding Source	Location	Drainage Area (Square Miles)	Peak Discharges (cfs)				
			10% Annual Chance	4% Annual Chance	2% Annual Chance	1% Annual Chance	0.2% Annual Chance
Lower Mosher Creek (Mosher Slough)	State Highway 99	3.9	*	*	*	430	*
Lower Mosher Creek (Mosher Slough)	Alpine Road	2.0	*	*	*	240	*
Middle Paddy Creek	Jack Tone Road	7.8	*	*	*	990	*
Mormon Channel[9]	Terminus (Upper Stockton Channel)	8.4	245	*	430	520	2,835
Mormon Channel[9]	Southern Pacific Railroad	6.4	100	*	195	240	2,615
Mormon Channel[9]	Diamond Street	4.5	175	*	300	365	2,720
Mormon Channel[9]	State Highway 99	2.5	175	*	300	365	3,525
Mormon Channel[9]	Upper end of channel	0.0	5	*	10	20	325
Mormon Slough	Terminus (head of Diverting Canal)	538.0	*	*	*	15,020[2]	*
Mormon Slough	Confluence of Potter Creek A	536.4	*	*	*	15,140	*

Table 10: Summary of Discharges (continued)

Flooding Source	Location	Drainage Area (Square Miles)	Peak Discharges (cfs)				
			10% Annual Chance	4% Annual Chance	2% Annual Chance	1% Annual Chance	0.2% Annual Chance
Mormon Slough	Confluence of Potter Creek B	513.7	*	*	*	13,240	*
Mormon Slough	At Bellota[1]	480.0	12,500	*	12,500	12,690	27,100
Mokelumne River[10]	Below Dry Creek	1,000	*	*	*	28,200	*
Mokelumne River[10]	Mouth of Dry Creek	696	4,000[11]	*	5,000[11]	14,500	34,400
Mokelumne River[10]	Peltier Road	680	4,000[11]	*	5,000[11]	15,400	35,200
Mokelumne River[10]	Woolbridge Dam	661	4,000[11]	*	5,000[11]	15,900	40,700
Mokelumne River[10]	Lodi Lake	658	4,000[11]	*	5,000[11]	16,000	41,800
Mokelumne River[10]	Southern Pacific Railroad Bridge	656	4,000[11]	*	5,000[11]	16,000	41,800
Mokelumne River[10]	State Highway 99	653	4,000[11]	*	5,000[11]	16,100	47,900
Mokelumne River[10]	Central Highway Traction Co. Railroad	650	4,000[11]	*	5,000[11]	16,100	49,400
Mokelumne River[10]	Elliot Road	644	4,000[11]	*	5,000[11]	18,100	57,300
Mokelumne River[10]	At Camanche Dam	627	4,000[11]	*	5,000[11]	19,000	60,200
Mosher Creek	Mouth (Bear Creek)	10.0	*	*	*	851	*

Table 24: Floodway Data

LOCATION		FLOODWAY			1% ANNUAL CHANCE FLOOD WATER SURFACE ELEVATION (FEET NAVD88)			
CROSS SECTION	DISTANCE[1]	WIDTH (FEET)	SECTION AREA (SQ. FEET)	MEAN VELOCITY (FEET/SEC)	REGULATORY	WITHOUT FLOODWAY	WITH FLOODWAY	INCREASE
A-AY[2]								
AZ	49,632	50	245	3.4	43.5	43.5	43.5	0.0
BA	50,688	50	250	3.3	44.8	44.8	44.8	0.0
BB	51,744	45	250	3.3	45.8	45.8	45.8	0.0
BC	52,800	45	260	3.2	47.3	47.3	47.3	0.0
BD	53,856	50	255	3.2	48.4	48.4	48.4	0.0
BE	54,912	50	270	3.4	49.8	49.8	49.8	0.0
BF	55,968	60	335	2.7	50.6	50.6	50.6	0.0
BG	57,024	60	335	2.4	51.4	51.4	51.4	0.0
BH	58,080	55	295	2.8	52.0	52.0	52.0	0.0
BI	59,136	55	285	2.9	53.0	53.0	53.0	0.0
BJ	60,192	55	265	3.1	54.1	54.1	54.1	0.0
BK	61,248	50	245	3.4	55.4	55.4	55.4	0.0
BL	62,304	50	275	3.0	56.6	56.6	56.6	0.0
BM	63,360	55	280	2.9	57.6	57.6	57.6	0.0
BN	64,416	55	290	2.9	58.6	58.6	58.6	0.0
BO	65,472	55	300	2.7	59.5	59.5	59.5	0.0
BP	66,528	60	315	2.6	60.3	60.3	60.3	0.0
BQ	67,584	55	315	2.6	61.1	61.1	61.1	0.0
BR	68,640	50	260	3.2	62.1	62.1	62.1	0.0
BS	69,696	50	270	3.1	63.2	63.2	63.2	0.0
BT	70,752	55	285	2.9	64.3	64.3	64.3	0.0
BU	71,808	55	295	2.8	65.2	65.2	65.2	0.0

[1]Feet above mouth
[2]Cross sections A-AY not included. Floodway superseded by revised analysis.

Table 24: Floodway Data (continued)

LOCATION		FLOODWAY			1% ANNUAL CHANCE FLOOD WATER SURFACE ELEVATION (FEET NAVD88)			
CROSS SECTION	DISTANCE[1]	WIDTH (FEET)	SECTION AREA (SQ. FEET)	MEAN VELOCITY (FEET/SEC)	REGULATORY	WITHOUT FLOODWAY	WITH FLOODWAY	INCREASE
BV	72,864	60	335	2.5	65.9	65.9	65.9	0.0
BW	73,920	55	320	2.6	66.6	66.6	66.6	0.0
BX	74,976	55	315	2.6	67.1	67.1	67.1	0.0
BY	76,032	55	315	2.6	67.9	67.9	67.9	0.0
BZ	76,982	55	315	2.6	68.5	68.5	68.5	0.0
CA	78,144	55	290	2.8	69.5	69.5	69.5	0.0
CB	79,200	55	275	3.0	70.2	70.2	70.2	0.0
CC	80,256	55	270	3.0	71.3	71.3	71.3	0.0
CD	81,312	60	300	2.8	72.5	72.5	72.5	0.0
CE	82,368	55	300	2.8	73.7	73.7	73.7	0.0
CF	83,424	55	310	2.7	74.7	74.7	74.7	0.0
CG	84,480	55	305	2.7	75.7	75.7	75.7	0.0
CH	85,536	55	295	2.8	76.5	76.5	76.5	0.0
CI	86,592	55	295	2.8	78.1	78.1	78.1	0.0
CJ	87,648	60	325	2.5	79.9	79.9	79.9	0.0
CK	88,704	60	340	2.4	81.5	81.5	81.5	0.0
CL	89,760	60	360	2.3	82.9	82.9	82.9	0.0
CM	90,816	65	370	2.2	84.0	84.0	84.0	0.0
CN	91,872	65	370	2.2	85.1	85.1	85.1	0.0
CO	92,928	65	380	2.2	86.1	86.1	86.1	0.0
CP	93,984	65	330	2.5	87.0	87.0	87.0	0.0
CQ	95,040	60	330	2.5	87.4	87.4	87.4	0.0
CR	96,096	60	310	2.6	88.0	88.0	88.0	0.0
CS	97,152	60	300	2.7	88.9	88.9	88.9	0.0
CT	98,208	55	285	2.9	89.8	89.8	89.8	0.0

[1]Feet above mouth

Table 24: Floodway Data (continued)

LOCATION		FLOODWAY			1% ANNUAL CHANCE FLOOD WATER SURFACE ELEVATION (FEET NAVD88)			
CROSS SECTION	DISTANCE[1]	WIDTH (FEET)	SECTION AREA (SQ. FEET)	MEAN VELOCITY (FEET/SEC)	REGULATORY	WITHOUT FLOODWAY	WITH FLOODWAY	INCREASE
CU	99,264	55	285	2.9	90.8	90.8	90.8	0.0
CV	100,320	55	260	3.1	91.8	91.8	91.8	0.0
CW	101,376	50	260	3.1	93.0	93.0	93.0	0.0
CX	102,432	55	255	3.1	94.1	94.1	94.1	0.0
CY	103,488	55	270	2.9	95.4	95.4	95.4	0.0
CZ	104,544	55	300	2.7	97.1	97.1	97.1	0.0
DA	105,600	50	255	3.1	98.4	98.4	98.4	0.0
DB	106,656	55	270	1.6	99.6	99.6	99.6	0.0
DC	107,712	50	240	1.8	100.2	100.2	100.2	0.0
DD	108,768	45	195	2.2	101.2	101.2	101.2	0.0
DE	109,824	40	205	2.1	102.2	102.2	102.2	0.0
DF	110,880	40	205	2.1	103.2	103.2	103.2	0.0
DG	111,936	40	205	2.1	103.9	103.9	103.9	0.0
DH	112,992	40	150	2.8	105.7	105.7	105.7	0.0
DI	114,048	40	170	2.5	107.3	107.3	107.3	0.0
DJ	115,104	45	230	3.6	109.5	109.5	109.5	0.0
DK	116,160	55	285	2.9	111.5	111.5	111.5	0.0
DL	117,216	60	320	2.6	113.3	113.3	113.3	0.0
DM	118,272	55	345	2.5	114.4	114.4	114.4	0.0
DN	119,328	55	320	2.4	115.5	115.5	115.5	0.0
DO	120,384	60	390	2.1	116.3	116.3	116.3	0.0
DP	121,440	65	480	1.7	116.9	116.9	116.9	0.0
DQ	122,496	75	625	1.3	117.2	117.2	117.2	0.0

[1]Feet above mouth

Table 24: Floodway Data (continued)

LOCATION		FLOODWAY			1% ANNUAL CHANCE FLOOD WATER SURFACE ELEVATION (FEET NAVD88)			
CROSS SECTION	DISTANCE[1]	WIDTH (FEET)	SECTION AREA (SQ. FEET)	MEAN VELOCITY (FEET/SEC)	REGULATORY	WITHOUT FLOODWAY	WITH FLOODWAY	INCREASE
DR	123,605	75	550	1.5	117.3	117.3	117.3	0.0
DS	124,608	75	565	1.5	117.6	117.6	117.6	0.0
DT	125,664	75	555	1.5	117.9	117.9	117.9	0.0
DU	126,878	70	520	1.6	118.3	118.3	118.3	0.0
DV	127,776	65	415	2.0	118.7	118.7	118.7	0.0
DW	128,832	60	425	2.0	119.3	119.3	119.3	0.0
DX	130,099	60	435	1.9	119.8	119.8	119.8	0.0
DY	130,944	60	435	0.0	120.0	120.0	120.0	0.0
DZ	132,000	50	335	0.0	120.0	120.0	120.0	0.0
EA	133,056	45	295	0.0	120.1	120.1	120.1	0.0
EB	134,112	50	350	0.0	120.1	120.1	120.1	0.0
EC	135,326	60	400	0.0	126.9	126.9	126.9	0.0
ED	136,910	120	1,745	8.1	128.2	128.2	128.2	0.0
EE	139,286	360	2,340	5.6	130.5	130.5	130.7	0.2
EF	142,560	125	1,815	7.0	133.6	133.6	134.0	0.4
EG	144,302	210	2,750	4.6	135.5	135.5	136.0	0.5
EH	146,362	820	8,130	1.6	136.4	136.4	136.7	0.3
EI	149,213	230	2,025	6.3	137.0	137.0	137.1	0.1
EJ	151,483	280	2,285	5.6	140.7	140.7	140.9	0.2
EK	153,120	340	3,930	3.3	141.9	141.9	142.1	0.2
EL	154,440	515	5,530	2.4	142.4	142.4	142.5	0.1
EM	155,760	325	3,320	4.0	142.7	142.7	142.8	0.1
EN	157,080	135	1,060	12.0	143.4	143.4	143.8	0.4
EO	158,400	200	2,135	6.0	148.6	148.6	148.6	0.0
EP	159,720	160	1,750	7.3	150.6	150.6	150.7	0.1

[1]Feet above mouth

Table 24: Floodway Data (continued)

LOCATION		FLOODWAY			1% ANNUAL CHANCE FLOOD WATER SURFACE ELEVATION (FEET NAVD88)			
CROSS SECTION	DISTANCE[1]	WIDTH (FEET)	SECTION AREA (SQ. FEET)	MEAN VELOCITY (FEET/SEC)	REGULATORY	WITHOUT FLOODWAY	WITH FLOODWAY	INCREASE
EQ	161,040	340	2,720	4.7	153.0	153.0	153.1	0.1
ER	162,360	370	1,835	7.9	155.4	155.4	155.4	0.0
ES	163,680	170	1,560	8.1	158.4	158.4	159.0	0.6
ET	165,000	180	1,980	6.4	161.6	161.6	162.2	0.6
EU	166,320	180	1,630	7.8	164.2	164.2	164.7	0.5

[1]Feet above mouth

NOTES TO USERS

This map is for use in administering the National Flood Insurance Program. It does not necessarily identify all areas subject to flooding, particularly from local drainage sources of small size. The **community map repository** should be consulted for possible updated or additional flood hazard information.

To obtain more detailed information in areas where **Base Flood Elevations** (BFEs) and/or **floodways** have been determined, users are encouraged to consult the Flood Profiles and Floodway Data and/or Summary of Stillwater Elevations tables contained within the Flood Insurance Study (FIS) report that accompanies this FIRM. Users should be aware that BFEs shown on the FIRM represent rounded whole-foot elevations. These BFEs are intended for flood insurance rating purposes only and should not be used as the sole source of flood elevation information. Accordingly, flood elevation data presented in the FIS report should be utilized in conjunction with the FIRM for purposes of construction and/or floodplain management.

Coastal Base Flood Elevations shown on this map apply only landward of 0.0' North American Vertical Datum of 1988 (NAVD 88). Users of this FIRM should be aware that coastal flood elevations are also provided in the Summary of Stillwater Elevations table in the Flood Insurance Study Report for this jurisdiction. Elevations shown in the Summary of Stillwater Elevations table should be used for construction, and/or floodplain management purposes when they are higher than the elevations shown on this FIRM.

Boundaries of the **floodways** were computed at cross sections and interpolated between cross sections. The floodways were based on hydraulic considerations with regard to requirements of the National Flood Insurance Program. Floodway widths and other pertinent floodway data are provided in the Flood Insurance Study report for this jurisdiction.

Certain areas not in Special Flood Hazard Areas may be protected by **flood control structures**. Refer to Section 2.4 "Flood Protection Measures" of the Flood Insurance Study report for information on flood control structures in this jurisdiction.

The **projection** used in the preparation of this map was Universal Transverse Mercator (UTM) Zone 10N. The **horizontal datum** was NAD83, GRS80 spheroid. Differences in datum, spheroid, projection or UTM zones used in the production of FIRMs for adjacent jurisdictions may result in slight positional differences in map features across jurisdiction boundaries. These differences do not affect the accuracy of this FIRM.

Flood elevations on this map are referenced to the North American Vertical Datum of 1988. These flood elevations must be compared to structure and ground elevations referenced to the same **vertical datum**. For information regarding conversion between the National Geodetic Vertical Datum of 1929 and the North American Vertical Datum of 1988, visit the National Geodetic Survey website at http://www.ngs.noaa.gov or contact the National Geodetic Survey at the following address:

NGS Information Services
NOAA, N/NGS12
National Geodetic Survey, SSMC-3, #9202
1315 East-West Highway
Silver Spring, Maryland 20910-3202
(301) 713-3242

To obtain current elevation, description, and/or location information for **bench marks** shown on this map, please contact the Information Services Branch of the National Geodetic Survey at **(301) 713-3242**, or visit their website at http://www.ngs.noaa.gov/.

Base map information shown on this FIRM was derived from multiple sources. This information was compiled from the U.S. Geological Survey, 2002, National Geodetic Survey, 2002, City of Lathrop, 1997, City of Manteca, Department of Public Works, 2001, and San Joaquin County Community Development Department, 2008. Additional information was photogrammetrically compiled at a scale of 1:12,000 from U.S. Geological Survey aerial photography dated 1989, 1993 and 2002.

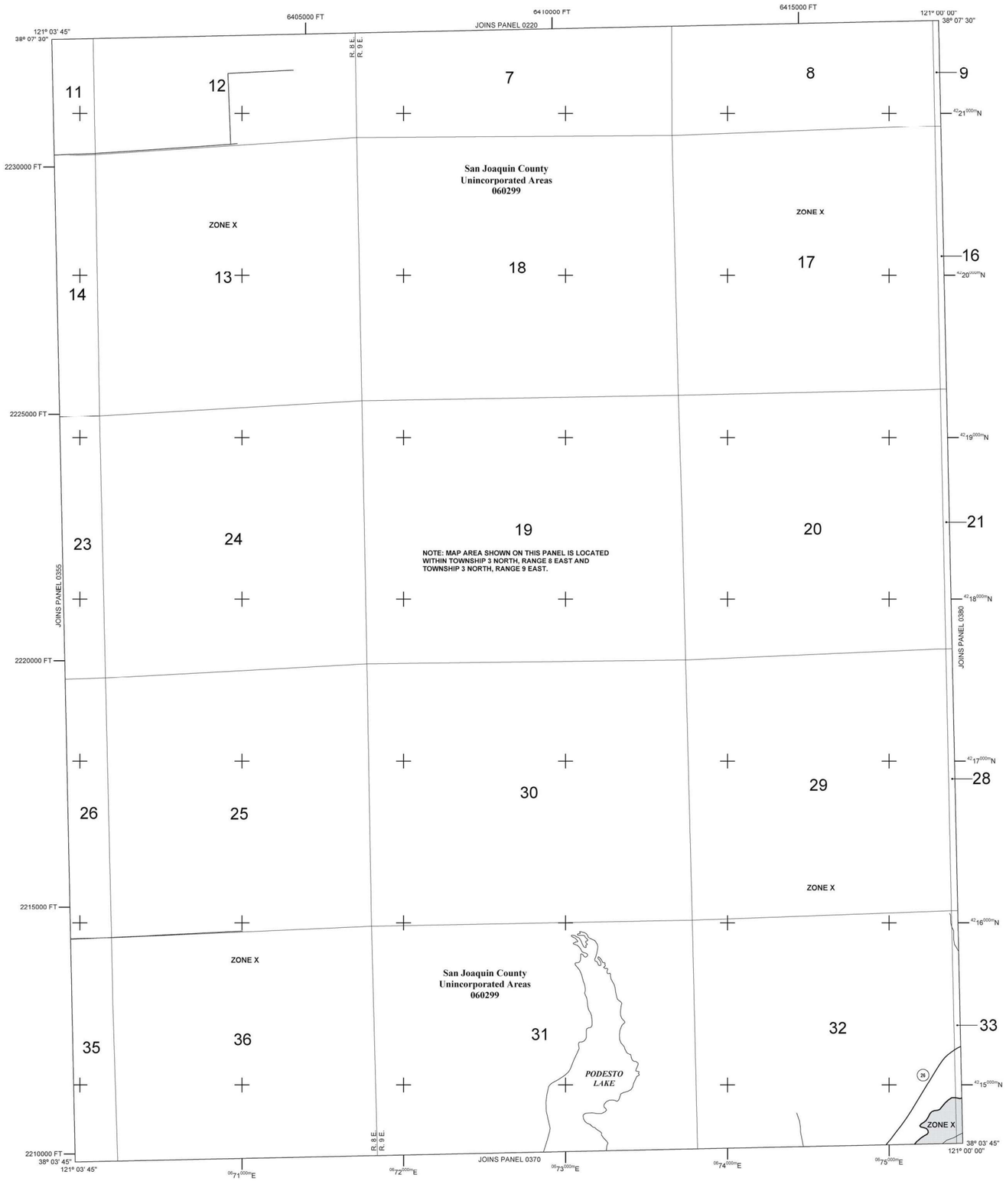
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Corporate limits shown on this map are based on the best data available at the time of publication. Because changes due to annexations or de-annexations may have occurred after this map was published, map users should contact appropriate community officials to verify current corporate limit locations.

Please refer to the separately printed **Map Index** for an overview map of the county showing the layout of map panels; community map repository addresses; and a Listing of Communities table containing National Flood Insurance Program dates for each community as well as a listing of the panels on which each community is located.

Contact the **FEMA Map Service Center** at 1-800-358-9616 for information on available products associated with this FIRM. Available products may include previously issued Letters of Map Change, a Flood Insurance Study report, and/or digital versions of this map. The FEMA Map Service Center may also be reached by Fax at 1-800-358-9620 and their website at http://www.msc.fema.gov/.

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LEGEND

SPECIAL FLOOD HAZARD AREAS (SFHAs) SUBJECT TO INUNDATION BY THE 1% ANNUAL CHANCE FLOOD

The 1% annual chance flood (100-year flood), also known as the base flood, is the flood that has a 1% chance of being equaled or exceeded in any given year. The Special Flood Hazard Area is the area subject to flooding by the 1% annual chance flood. Areas of Special Flood Hazard include Zones A, AE, AH, AO, AR, A99, V, and VE. The Base Flood Elevation is the water-surface elevation of the 1% annual chance flood.

- **ZONE A** No Base Flood Elevations determined.
- **ZONE AE** Base Flood Elevations determined.
- **ZONE AH** Flood depths of 1 to 3 feet (usually areas of ponding); base flood Elevations determined.
- **ZONE AO** Flood depths of 1 to 3 feet (usually sheet flow on sloping terrain); average depths determined. For areas of alluvial fan flooding, velocities also determined.
- **ZONE AR** Special Flood Hazard Area formerly protected from the 1% annual chance flood by a flood control system that was subsequently decertified. Zone AR indicates that the former flood control system is being restored to provide protection from the 1% annual chance or greater flood.
- **ZONE A99** Area to be protected from 1% annual chance flood by a Federal flood protection system under construction; no Base Flood Elevations determined.
- **ZONE V** Coastal flood zone with velocity hazard (wave action); no Base Flood Elevations determined.
- **ZONE VE** Coastal flood zone with velocity hazard (wave action); Base Flood Elevations determined.

FLOODWAY AREAS IN ZONE AE

The floodway is the channel of a stream plus any adjacent floodplain areas that must be kept free of encroachment so that the 1% annual chance flood can be carried without substantial increases in flood heights.

OTHER FLOOD AREAS

- **ZONE X** Areas of 0.2% annual chance flood; areas of 1% annual chance flood with average depths of less than 1 foot or with drainage areas less than 1 square mile; and areas protected by levees from 1% annual chance flood.

OTHER AREAS

- **ZONE X** Areas determined to be outside the 0.2% annual chance floodplain.
- **ZONE D** Areas in which flood hazards are undetermined, but possible.

COASTAL BARRIER RESOURCES SYSTEM (CBRS) AREAS

OTHERWISE PROTECTED AREAS (OPAs)

CBRS areas and OPAs are normally located within or adjacent to Special Flood Hazard Areas.

- Floodplain boundary
- Floodway boundary
- Zone D Boundary
- CBRS and OPA Boundary
- Boundary dividing Special Flood Hazard Areas of different Base Flood Elevations, flood depths or flood velocities.
- 513 (EL 987) Base Flood Elevation line and value; elevation in feet* / Base Flood Elevation value where uniform within zone; elevation in feet*

*Referenced to the North American Vertical Datum of 1988

- A — A Cross section line
- 23 — 23 Transect line
- 97° 07' 30", 32° 22' 30" Geographic coordinates referenced to the North American Datum of 1983 (NAD 83), Western Hemisphere
- 6276000mE 1000-meter Universal Transverse Mercator grid values, zone 10
- 600000 FT 5000-foot grid ticks: California State Plane coordinate system, zone III (FIPSZONE 0403), Lambert Conformal Conic Projection
- DX5510 × Bench mark (see explanation in Notes to Users section of this FIRM panel)
- M1.5 River Mile

MAP REPOSITORIES
Refer to Map Repositories list on Map Index.

EFFECTIVE DATE OF COUNTYWIDE FLOOD INSURANCE RATE MAP PANEL
October 16, 2009

EFFECTIVE DATE(S) OF REVISION(S) TO THIS PANEL

For community map revision history prior to countywide mapping, refer to the Community Map History table located in the Flood Insurance Study report for this jurisdiction.

To determine if flood insurance is available in this community, contact your insurance agent or call the National Flood Insurance Program at 1-800-638-6620.

MAP SCALE 1" = 1000'

500 0 1000 2000 FEET

300 0 300 600 METERS

NFIP

NATIONAL FLOOD INSURANCE PROGRAM

PANEL 0360F

FIRM

FLOOD INSURANCE RATE MAP

SAN JOAQUIN COUNTY, CALIFORNIA AND INCORPORATED AREAS

PANEL 360 OF 950

(SEE MAP INDEX FOR FIRM PANEL LAYOUT)

CONTAINS:

COMMUNITY	NUMBER	PANEL	SUFFIX
SAN JOAQUIN COUNTY	060299	0360	F

Notice to User: The **Map Number** shown below should be used when placing map orders; the **Community Number** shown above should be used on insurance applications for the subject community.

MAP NUMBER
06077C0360F

EFFECTIVE DATE
OCTOBER 16, 2009

Federal Emergency Management Agency

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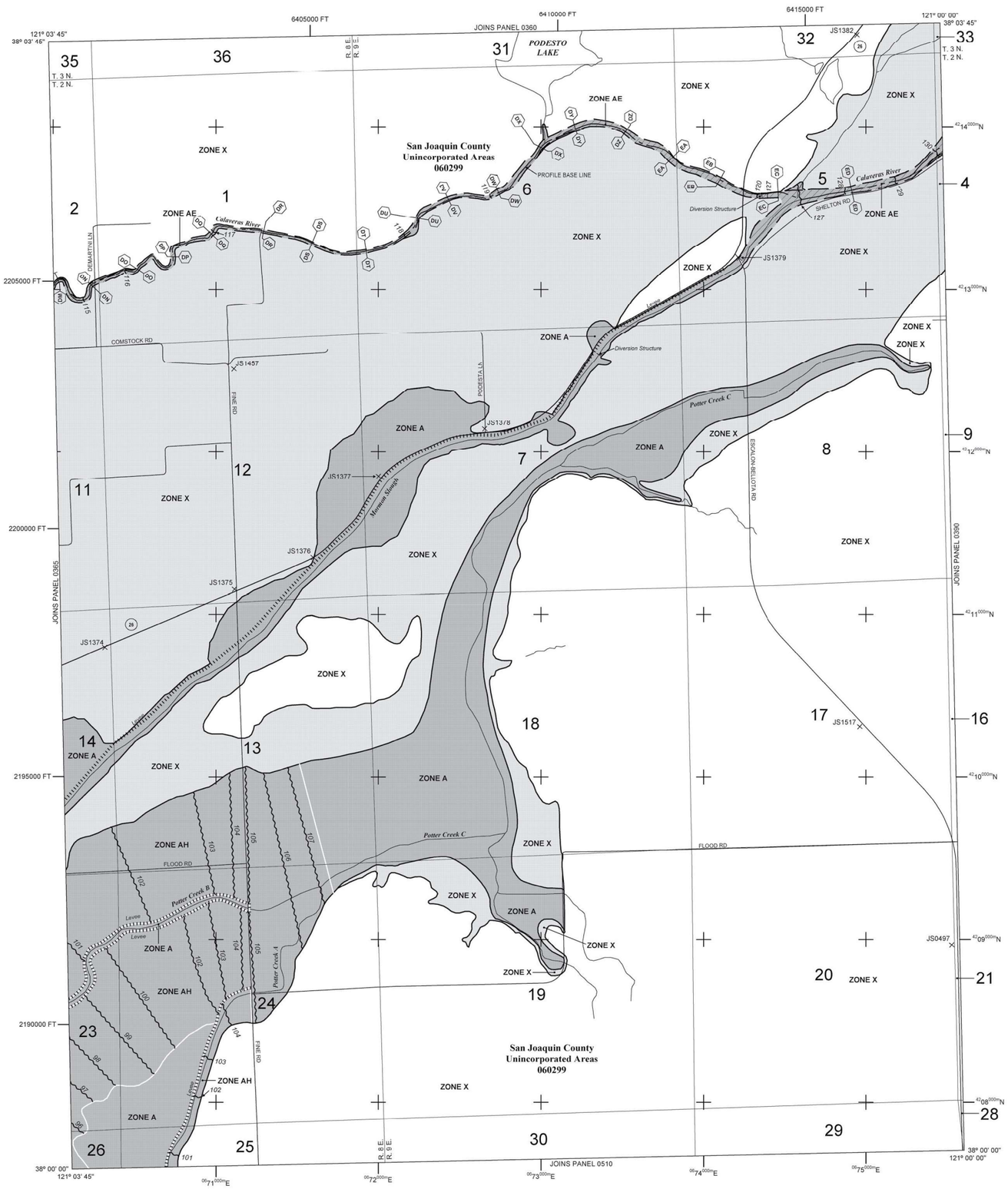
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- Boundary dividing Special Flood Hazard Areas of different Base Flood Elevations, flood depths or flood velocities.
- 513 (EL 987) — Base Flood Elevation line and value; elevation in feet* / Base Flood Elevation value where uniform within zone; elevation in feet*

*Referenced to the North American Vertical Datum of 1988

- A — A — Cross section line
- 23 — 23 — Transect line
- 97° 07' 30", 32° 22' 30" — Geographic coordinates referenced to the North American Datum of 1983 (NAD 83), Western Hemisphere
- 6276000mE — 1000-meter Universal Transverse Mercator grid values, zone 10
- 600000 FT — 5000-foot grid ticks: California State Plane coordinate system, zone III (FIPSZONE 0403), Lambert Conformal Conic Projection
- DX5510 — Bench mark (see explanation in Notes to Users section of this FIRM panel)
- M1.5 — River Mile

MAP REPOSITORIES
Refer to Map Repositories list on Map Index.

EFFECTIVE DATE OF COUNTYWIDE FLOOD INSURANCE RATE MAP PANEL
October 16, 2009

EFFECTIVE DATE(S) OF REVISION(S) TO THIS PANEL

For community map revision history prior to countywide mapping, refer to the Community Map History table located in the Flood Insurance Study report for this jurisdiction.

To determine if flood insurance is available in this community, contact your insurance agent or call the National Flood Insurance Program at 1-800-638-6620.

MAP SCALE 1" = 1000'

500 0 1000 2000 FEET

300 0 300 600 METERS

NFIP

NATIONAL FLOOD INSURANCE PROGRAM

PANEL 0370F

FIRM

FLOOD INSURANCE RATE MAP

SAN JOAQUIN COUNTY, CALIFORNIA AND INCORPORATED AREAS

PANEL 370 OF 950

(SEE MAP INDEX FOR FIRM PANEL LAYOUT)

CONTAINS:

COMMUNITY	NUMBER	PANEL	SUFFIX
SAN JOAQUIN COUNTY	060299	0370	F

Notice to User: The **Map Number** shown below should be used when placing map orders; the **Community Number** shown above should be used on insurance applications for the subject community.

MAP NUMBER
06077C0370F

EFFECTIVE DATE
OCTOBER 16, 2009

Federal Emergency Management Agency

NOTES TO USERS

This map is for use in administering the National Flood Insurance Program. It does not necessarily identify all areas subject to flooding, particularly from local drainage sources of small size. The **community map repository** should be consulted for possible updated or additional flood hazard information.

To obtain more detailed information in areas where **Base Flood Elevations** (BFEs) and/or **floodways** have been determined, users are encouraged to consult the Flood Profiles and Floodway Data and/or Summary of Stillwater Elevations tables contained within the Flood Insurance Study (FIS) report that accompanies this FIRM. Users should be aware that BFEs shown on the FIRM represent rounded whole-foot elevations. These BFEs are intended for flood insurance rating purposes only and should not be used as the sole source of flood elevation information. Accordingly, flood elevation data presented in the FIS report should be utilized in conjunction with the FIRM for purposes of construction and/or floodplain management.

Coastal Base Flood Elevations shown on this map apply only landward of 0.0' North American Vertical Datum of 1988 (NAVD 88). Users of this FIRM should be aware that coastal flood elevations are also provided in the Summary of Stillwater Elevations table in the Flood Insurance Study Report for this jurisdiction. Elevations shown in the Summary of Stillwater Elevations table should be used for construction, and/or floodplain management purposes when they are higher than the elevations shown on this FIRM.

Boundaries of the **floodways** were computed at cross sections and interpolated between cross sections. The floodways were based on hydraulic considerations with regard to requirements of the National Flood Insurance Program. Floodway widths and other pertinent floodway data are provided in the Flood Insurance Study report for this jurisdiction.

Certain areas not in Special Flood Hazard Areas may be protected by **flood control structures**. Refer to Section 2.4 "Flood Protection Measures" of the Flood Insurance Study report for information on flood control structures in this jurisdiction.

The **projection** used in the preparation of this map was Universal Transverse Mercator (UTM) Zone 10N. The **horizontal datum** was NAD83, GRS80 spheroid. Differences in datum, spheroid, projection or UTM zones used in the production of FIRMs for adjacent jurisdictions may result in slight positional differences in map features across jurisdiction boundaries. These differences do not affect the accuracy of this FIRM.

Flood elevations on this map are referenced to the North American Vertical Datum of 1988. These flood elevations must be compared to structure and ground elevations referenced to the same **vertical datum**. For information regarding conversion between the National Geodetic Vertical Datum of 1929 and the North American Vertical Datum of 1988, visit the National Geodetic Survey website at http://www.ngs.noaa.gov or contact the National Geodetic Survey at the following address:

NGS Information Services
NOAA, N/NGS12
National Geodetic Survey, SSMC-3, #9202
1315 East-West Highway
Silver Spring, Maryland 20910-3202
(301) 713-3242

To obtain current elevation, description, and/or location information for **bench marks** shown on this map, please contact the Information Services Branch of the National Geodetic Survey at **(301) 713-3242**, or visit their website at http://www.ngs.noaa.gov/.

Base map information shown on this FIRM was derived from multiple sources. This information was compiled from the U.S. Geological Survey, 2002, National Geodetic Survey, 2002, City of Lathrop, 1997, City of Manteca, Department of Public Works, 2001, and San Joaquin County Community Development Department, 2008. Additional information was photogrammetrically compiled at a scale of 1:12,000 from U.S. Geological Survey aerial photography dated 1989, 1993 and 2002.

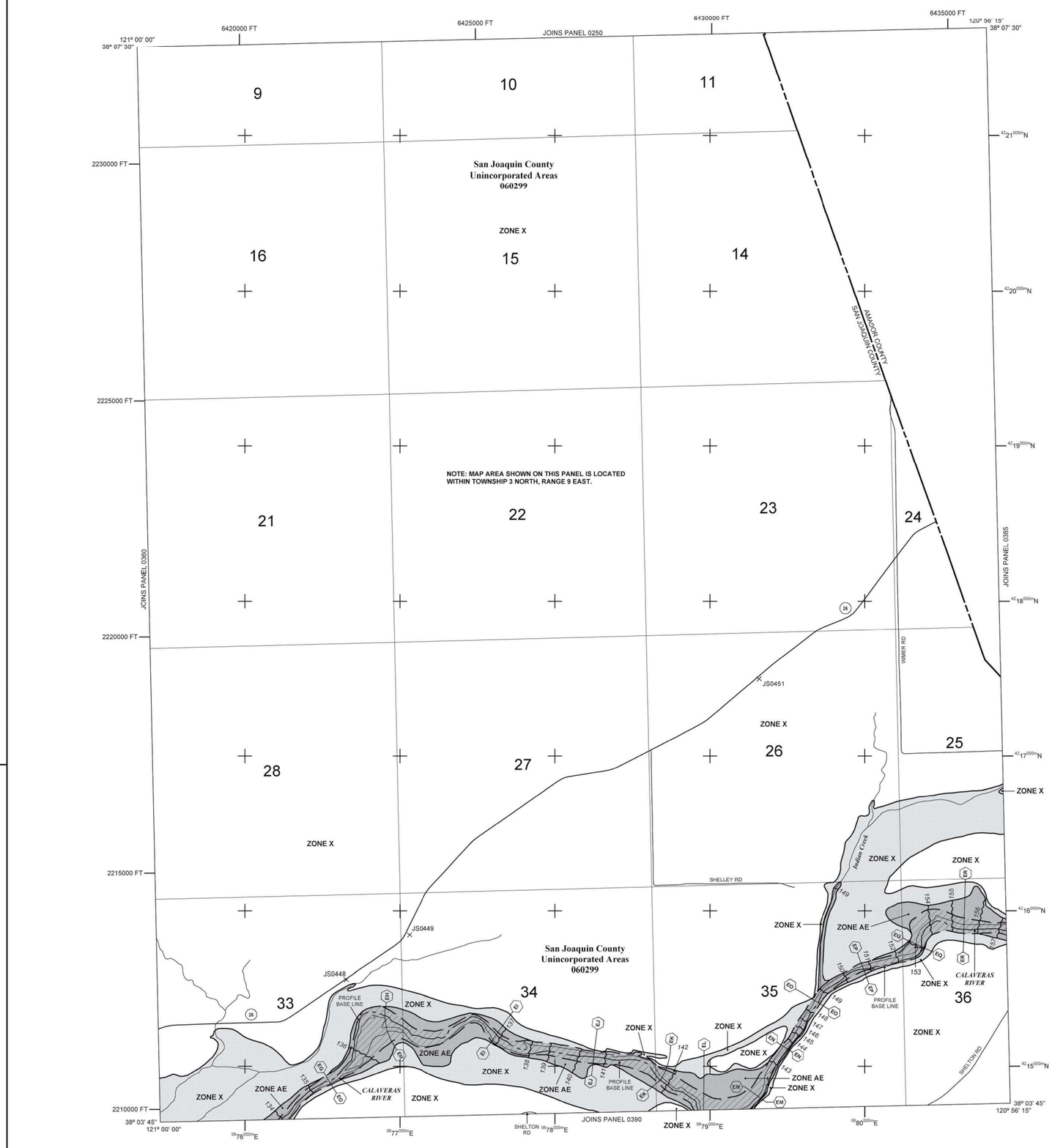
This map reflects more detailed and up-to-date **stream channel configurations** than those shown on the previous FIRM for this jurisdiction. The floodplains and floodways that were transferred from the previous FIRM may have been adjusted to conform to these new stream channel configurations. As a result, the Flood Profiles and Floodway Data tables in the Flood Insurance Study report (which contains authoritative hydraulic data) may reflect stream channel distances that differ from what is shown on this map.

Corporate limits shown on this map are based on the best data available at the time of publication. Because changes due to annexations or de-annexations may have occurred after this map was published, map users should contact appropriate community officials to verify current corporate limit locations.

Please refer to the separately printed **Map Index** for an overview map of the county showing the layout of map panels; community map repository addresses; and a Listing of Communities table containing National Flood Insurance Program dates for each community as well as a listing of the panels on which each community is located.

Contact the **FEMA Map Service Center** at 1-800-358-9616 for information on available products associated with this FIRM. Available products may include previously issued Letters of Map Change, a Flood Insurance Study report, and/or digital versions of this map. The FEMA Map Service Center may also be reached by Fax at 1-800-358-9620 and their website at http://www.msc.fema.gov/.

If you have **questions about this map** or questions concerning the National Flood Insurance Program in general, please call **1- 877- FEMA MAP** (1-877-336-2627) or visit the FEMA website at http://www.fema.gov/.



LEGEND

SPECIAL FLOOD HAZARD AREAS (SFHAs) SUBJECT TO INUNDATION BY THE 1% ANNUAL CHANCE FLOOD

The 1% annual chance flood (100-year flood), also known as the base flood, is the flood that has a 1% chance of being equaled or exceeded in any given year. The Special Flood Hazard Area is the area subject to flooding by the 1% annual chance flood. Areas of Special Flood Hazard include Zones A, AE, AH, AO, AR, A99, V, and VE. The Base Flood Elevation is the water-surface elevation of the 1% annual chance flood.

- **ZONE A** — No Base Flood Elevations determined.
- **ZONE AE** — Base Flood Elevations determined.
- **ZONE AH** — Flood depths of 1 to 3 feet (usually areas of ponding); base flood Elevations determined.
- **ZONE AO** — Flood depths of 1 to 3 feet (usually sheet flow on sloping terrain); average depths determined. For areas of alluvial fan flooding, velocities also determined.
- **ZONE AR** — Special Flood Hazard Area formerly protected from the 1% annual chance flood by a flood control system that was subsequently decertified. Zone AR indicates that the former flood control system is being restored to provide protection from the 1% annual chance or greater flood.
- **ZONE A99** — Area to be protected from 1% annual chance flood by a Federal flood protection system under construction; no Base Flood Elevations determined.
- **ZONE V** — Coastal flood zone with velocity hazard (wave action); no Base Flood Elevations determined.
- **ZONE VE** — Coastal flood zone with velocity hazard (wave action); Base Flood Elevations determined.

FLOODWAY AREAS IN ZONE AE

The floodway is the channel of a stream plus any adjacent floodplain areas that must be kept free of encroachment so that the 1% annual chance flood can be carried without substantial increases in flood heights.

OTHER FLOOD AREAS

- **ZONE X** — Areas of 0.2% annual chance flood; areas of 1% annual chance flood with average depths of less than 1 foot or with drainage areas less than 1 square mile; and areas protected by levees from 1% annual chance flood.

OTHER AREAS

- **ZONE X** — Areas determined to be outside the 0.2% annual chance floodplain.
- **ZONE D** — Areas in which flood hazards are undetermined, but possible.

COASTAL BARRIER RESOURCES SYSTEM (CBRS) AREAS

OTHERWISE PROTECTED AREAS (OPAs)

CBRS areas and OPAs are normally located within or adjacent to Special Flood Hazard Areas.

- Floodplain boundary
- Floodway boundary
- Zone D Boundary
- CBRS and OPA Boundary
- Boundary dividing Special Flood Hazard Areas of different Base Flood Elevations, flood depths or flood velocities.
- 513 (EL 987) — Base Flood Elevation line and value; elevation in feet* / Base Flood Elevation value where uniform within zone; elevation in feet*

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MAP REPOSITORIES
Refer to Map Repositories list on Map Index.

EFFECTIVE DATE OF COUNTYWIDE FLOOD INSURANCE RATE MAP PANEL
October 16, 2009

EFFECTIVE DATE(S) OF REVISION(S) TO THIS PANEL

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MAP SCALE 1" = 1000'

500 0 1000 2000 FEET

300 0 300 600 METERS

NATIONAL FLOOD INSURANCE PROGRAM

PANEL 0380F

FIRM

FLOOD INSURANCE RATE MAP

SAN JOAQUIN COUNTY, CALIFORNIA AND INCORPORATED AREAS

PANEL 380 OF 950

(SEE MAP INDEX FOR FIRM PANEL LAYOUT)

CONTAINS:

COMMUNITY	NUMBER	PANEL	SUFFIX
SAN JOAQUIN COUNTY	060299	0380	F

Notice to User: The **Map Number** shown below should be used when placing map orders; the **Community Number** shown above should be used on insurance applications for the subject community.

MAP NUMBER
06077C0380F

EFFECTIVE DATE
OCTOBER 16, 2009

Federal Emergency Management Agency

possible updated or additional flood hazard information.

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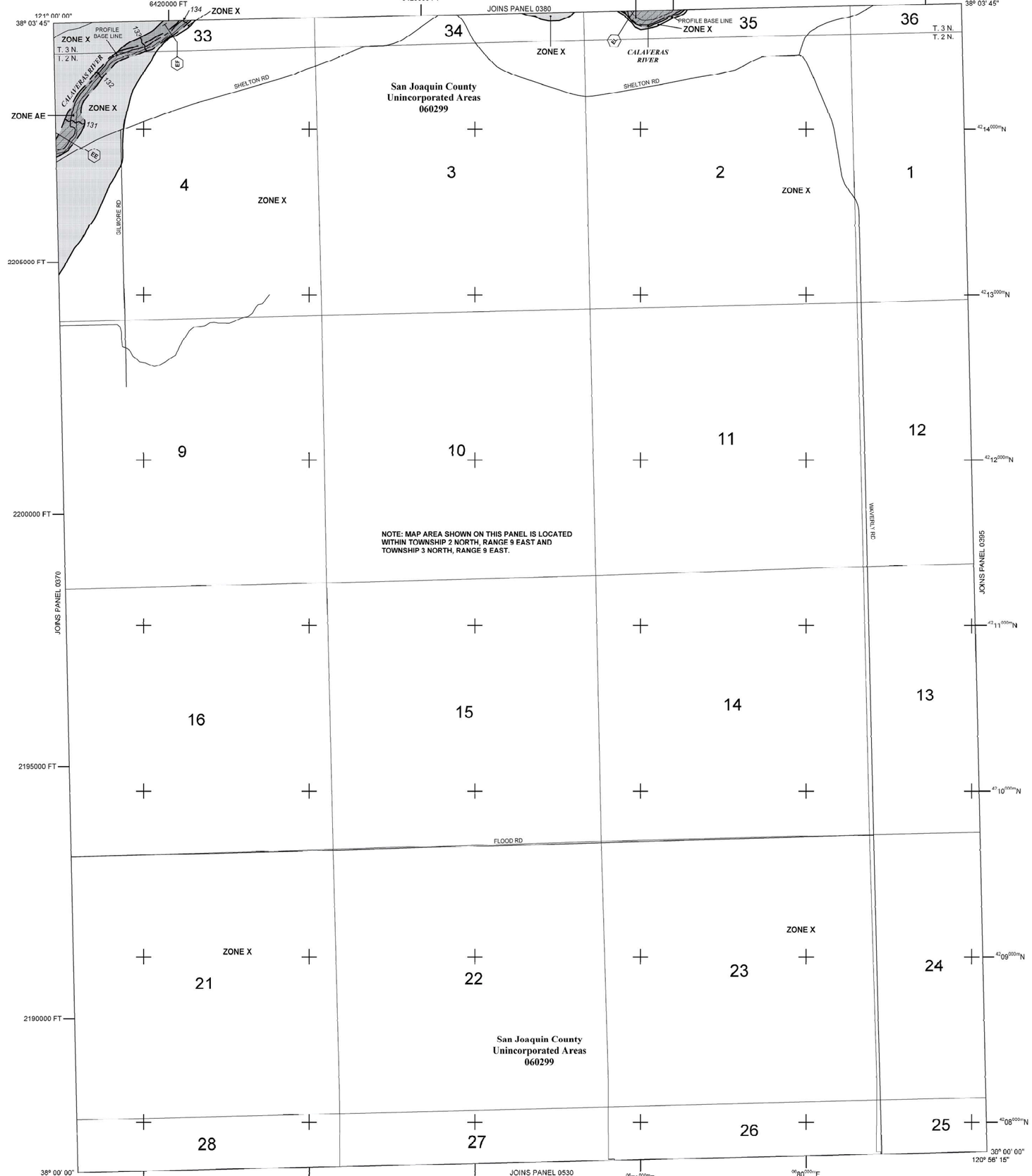
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ZONE A99 Area to be protected from 1% annual chance flood by a Federal flood protection system under construction; no Base Flood Elevations determined.

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OTHER FLOOD AREAS

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Floodway boundary
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Boundary dividing Special Flood Hazard Areas of different Base Flood Elevations, flood depths or flood velocities.
513 — Base Flood Elevation line and value; elevation in feet*
(EL 987) — Base Flood Elevation value where uniform within zone; elevation in feet*

*Referenced to the North American Vertical Datum of 1988

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600000 FT — 5000-foot grid ticks: California State Plane coordinate system, zone III (FIPSZONE 0403), Lambert Conformal Conic Projection
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M1.5 — River Mile

MAP REPOSITORIES
Refer to Map Repositories list on Map Index.

EFFECTIVE DATE OF COUNTYWIDE FLOOD INSURANCE RATE MAP PANEL
October 16, 2009

EFFECTIVE DATE(S) OF REVISION(S) TO THIS PANEL

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MAP SCALE 1" = 1000'
500 0 1000 2000 FEET
300 0 300 600 METERS

NFIP
NATIONAL FLOOD INSURANCE PROGRAM

PANEL 0390F

FIRM

FLOOD INSURANCE RATE MAP

SAN JOAQUIN COUNTY, CALIFORNIA AND INCORPORATED AREAS

PANEL 390 OF 950

(SEE MAP INDEX FOR FIRM PANEL LAYOUT)

CONTAINS:

COMMUNITY	NUMBER	PANEL	SUFFIX
SAN JOAQUIN COUNTY	060299	0390	F

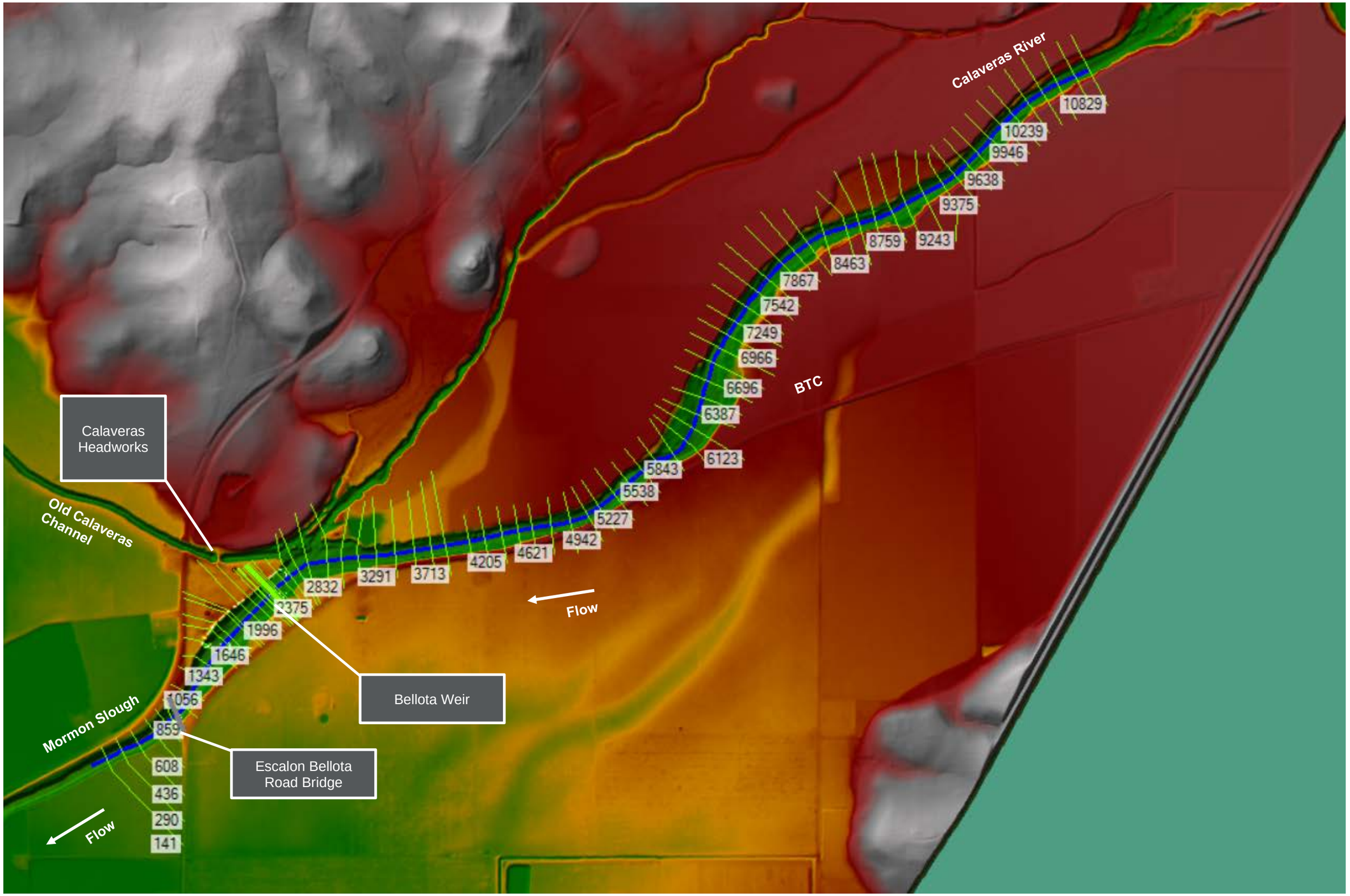
Notice to User: The **Map Number** shown below should be used when placing map orders; the **Community Number** shown above should be used on insurance applications for the subject community.

MAP NUMBER
06077C0390F

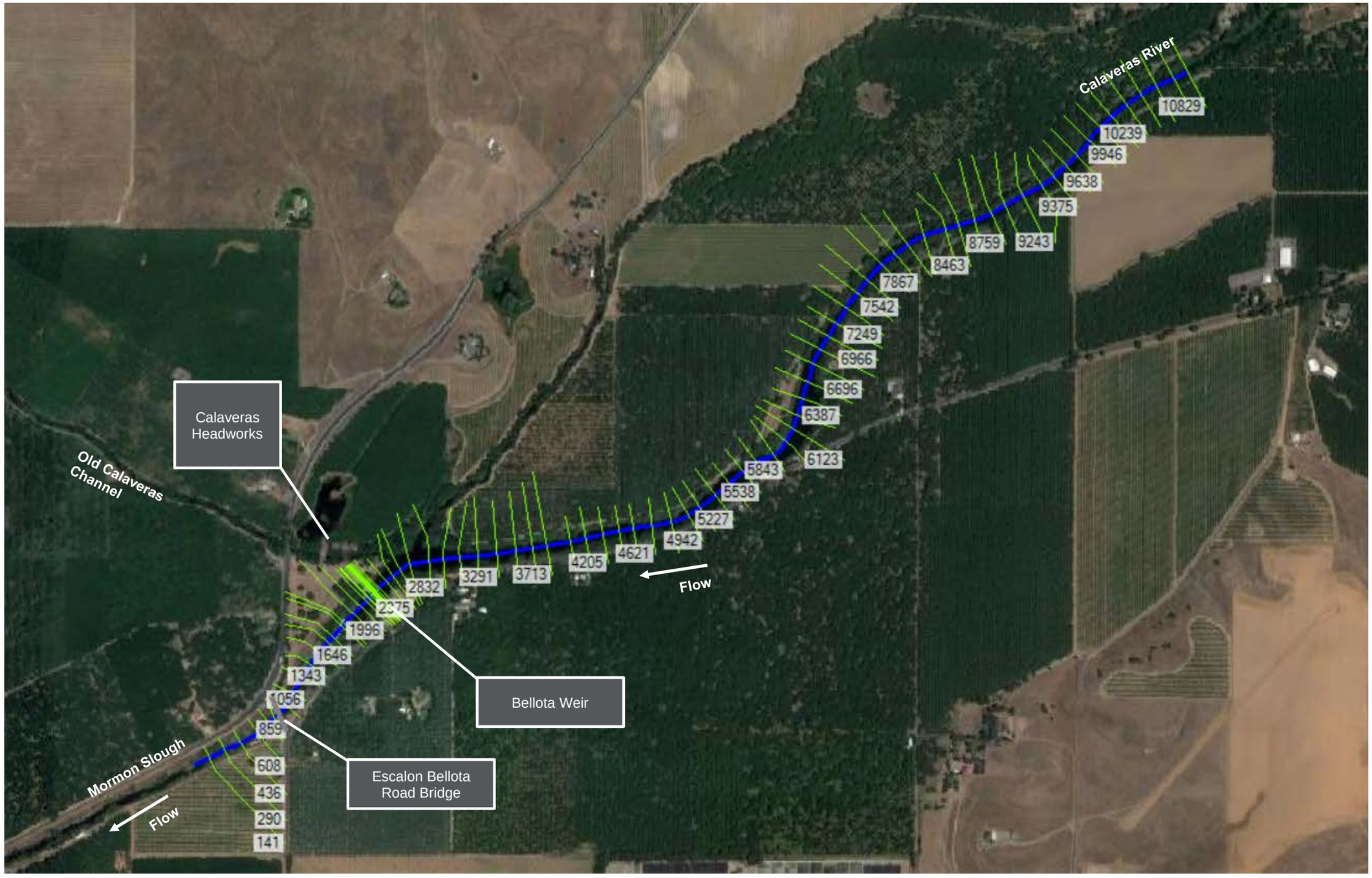
EFFECTIVE DATE
OCTOBER 16, 2009

Appendix B. Hydraulic Support Exhibits

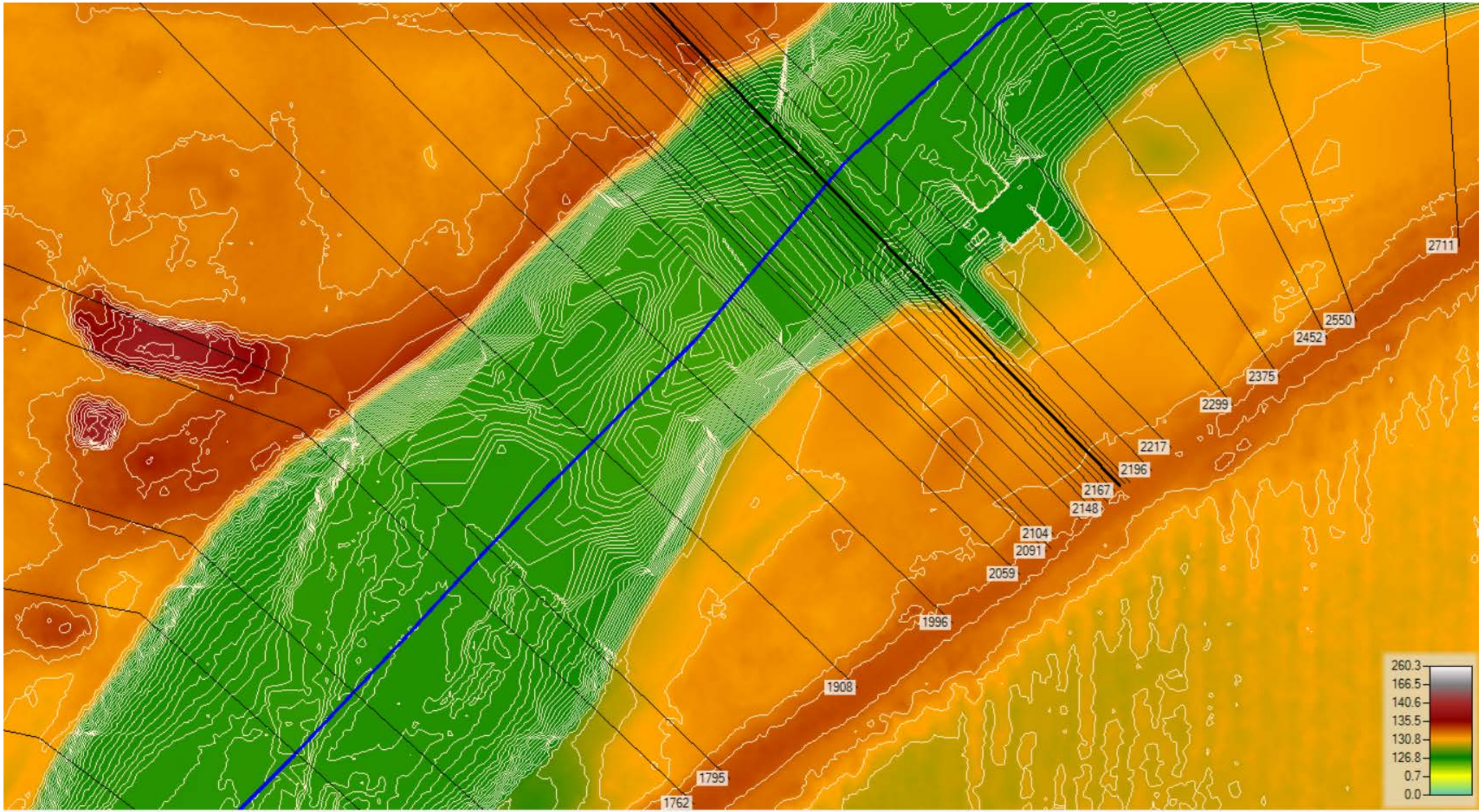
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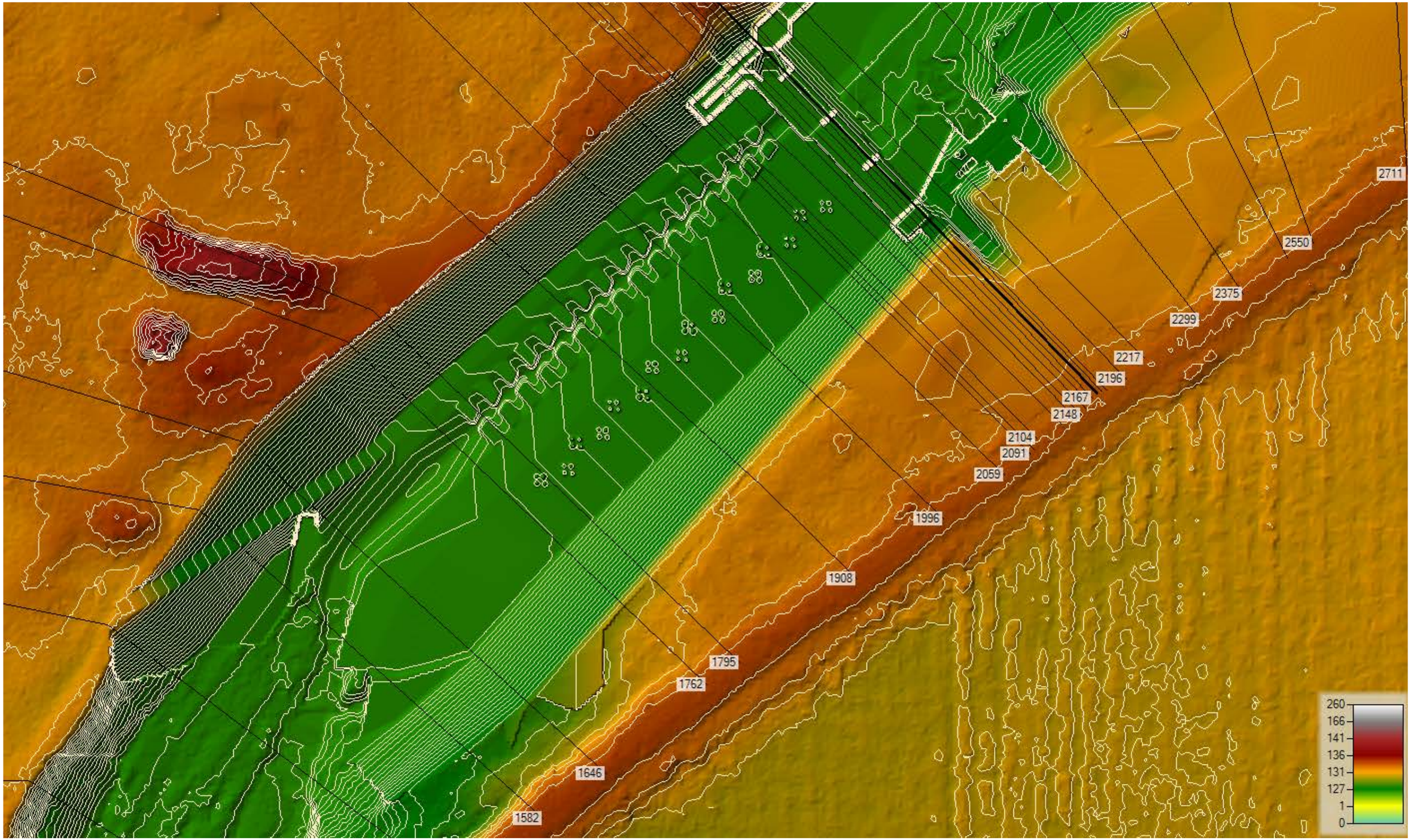
1D Modeling Layout and Existing Terrain Surface
EXHIBIT B1



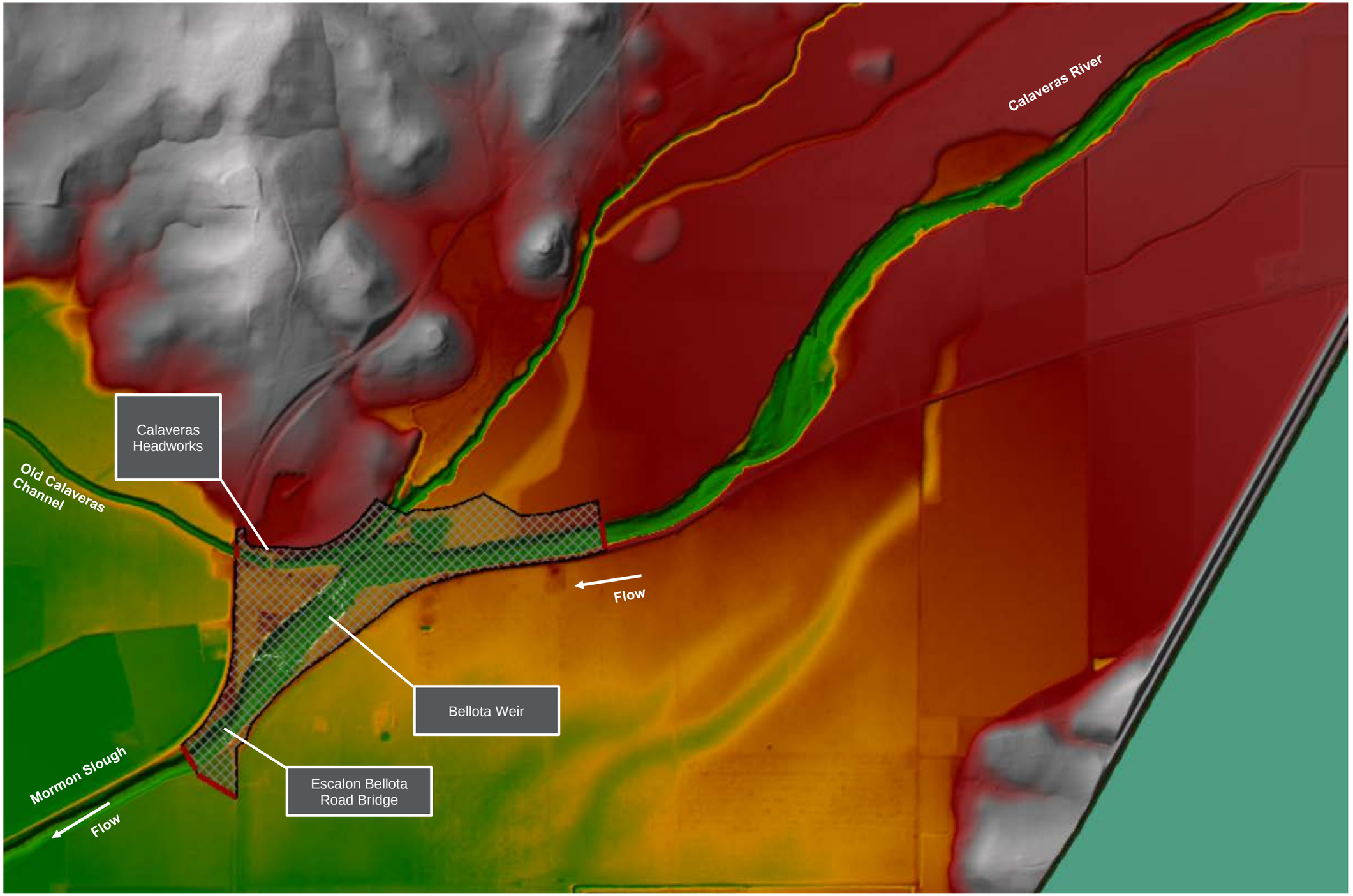
1D Modeling Layout and Aerial Image
EXHIBIT B2



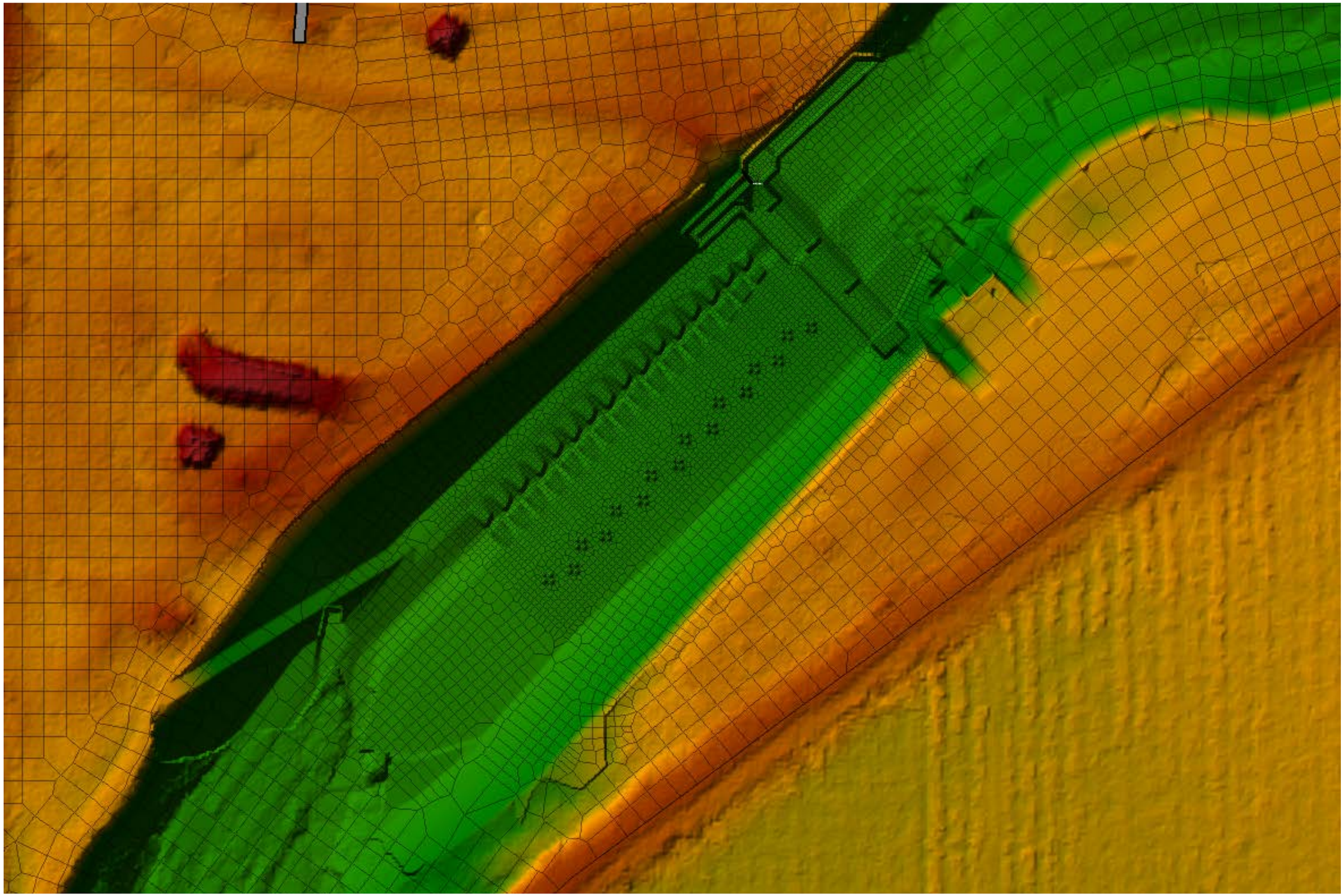
Project Area 1D Modeling Configuration, Existing Terrain
EXHIBIT B3



Project Area 1D Modeling Configuration, Proposed Terrain
EXHIBIT B4



2D Model Domain EXHIBIT B5



2D Computational Mesh in Project Area EXHIBIT B6

Appendix C. 1D HEC-RAS Results

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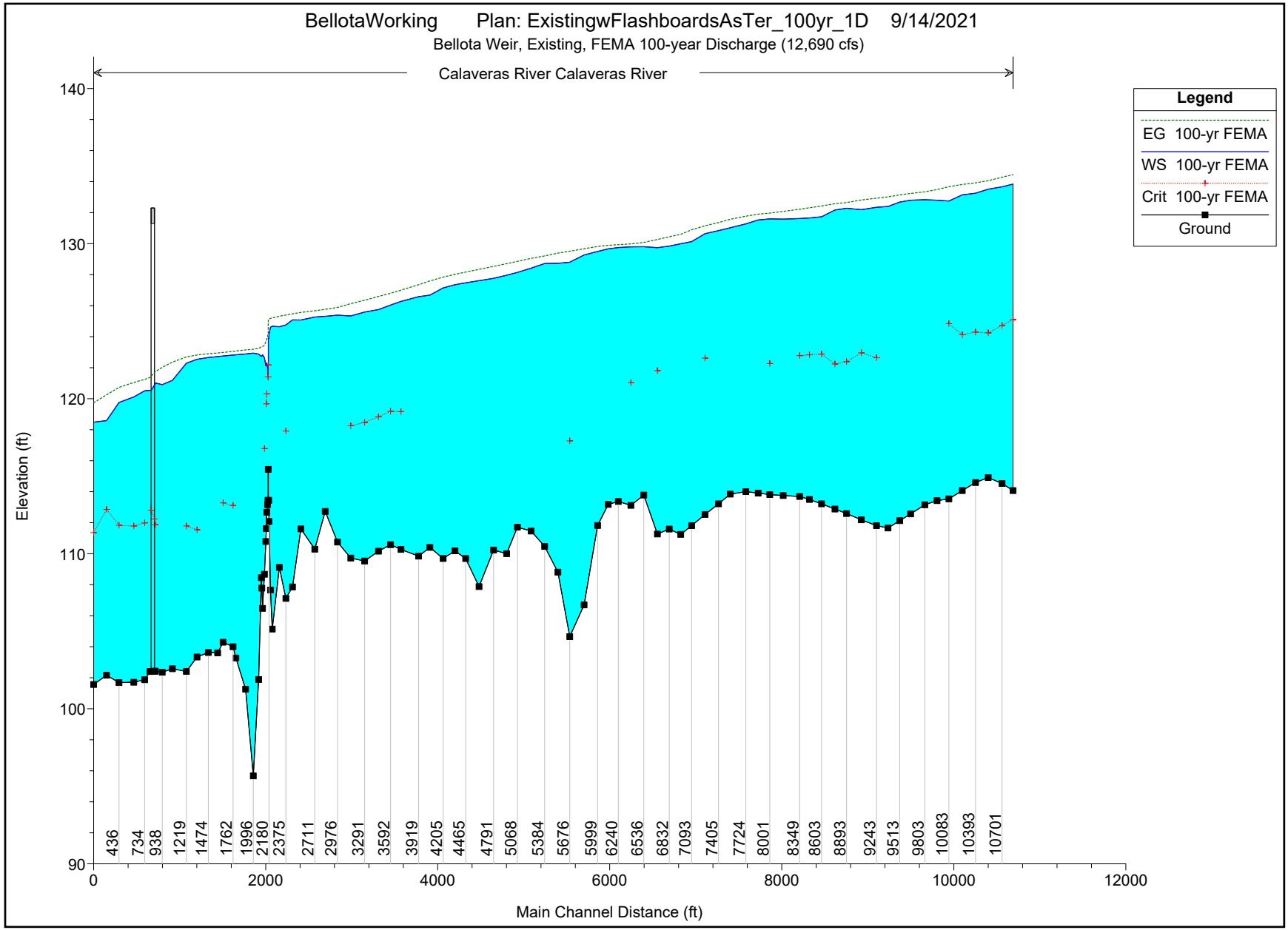
Existing Conditions: Hydraulic Output Table, Water Surface Profile, and Cross-Sections

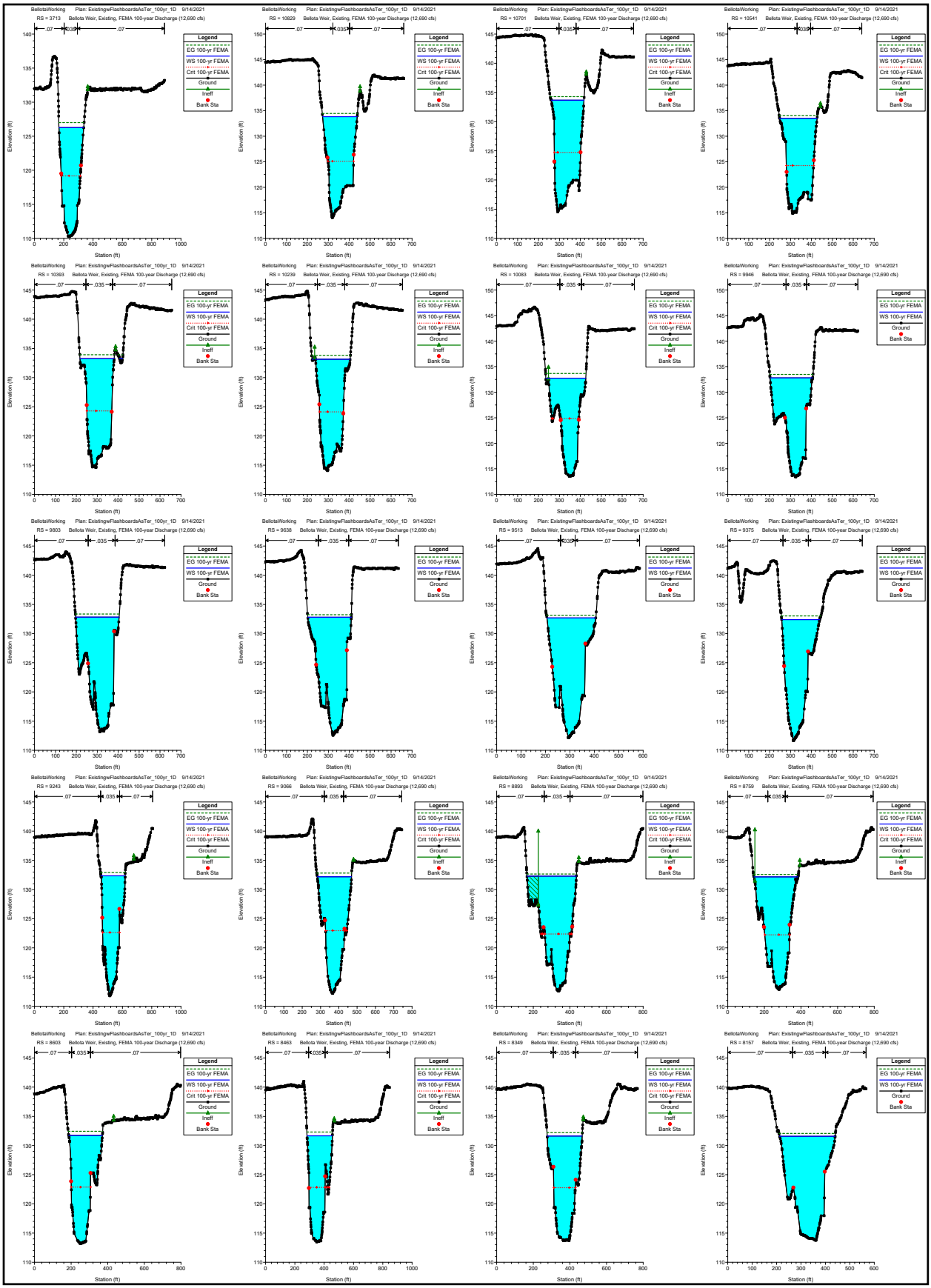
HEC-RAS Plan: ExistingwFlashboardsAsTer_100yr_1D River: Calaveras River Reach: Calaveras River Profile: 100-yr FEMA

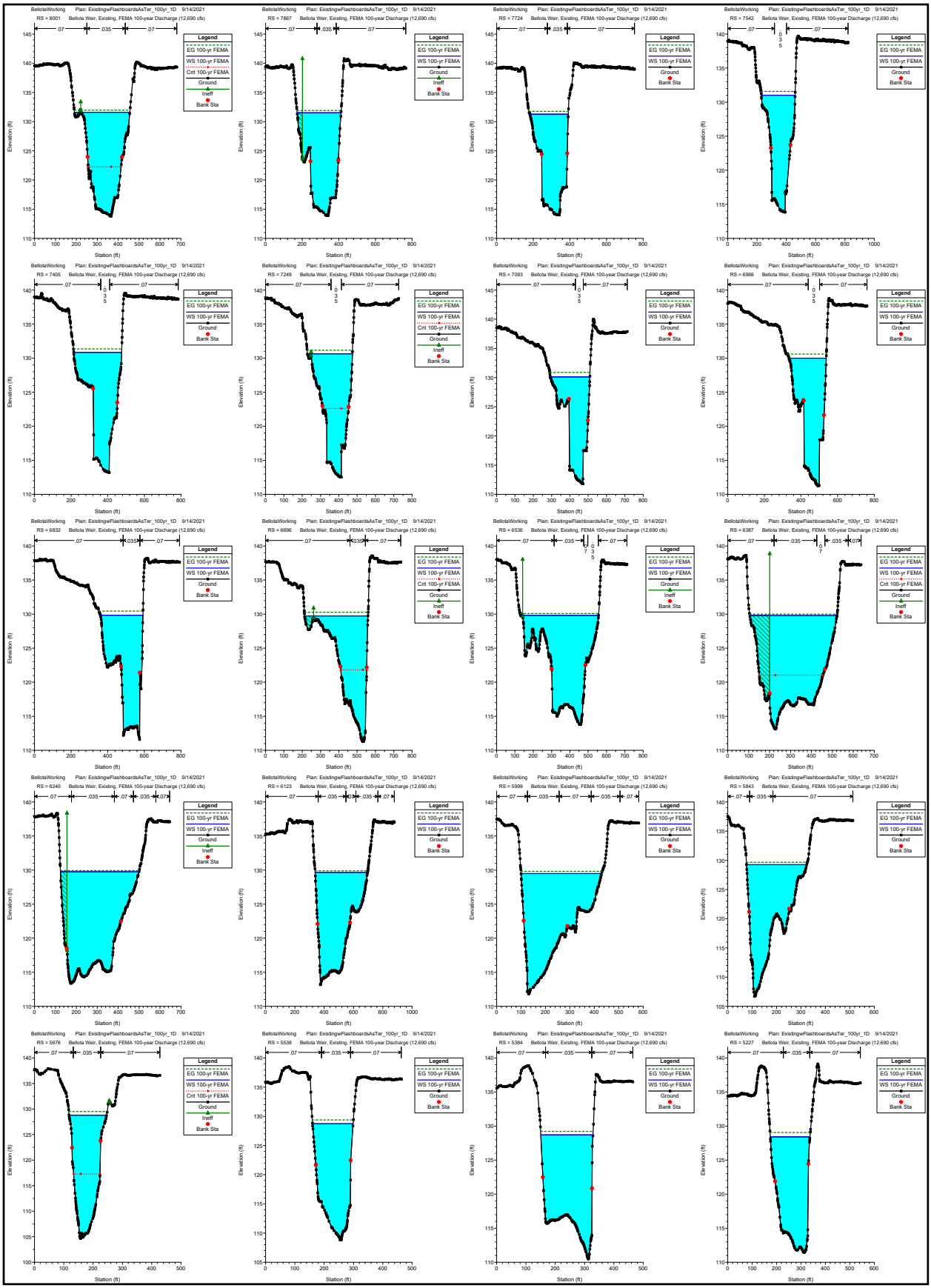
Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
Calaveras River	10829	100-yr FEMA	12690.00	114.07	133.839	125.09	134.45	0.001314	6.35	2124.25	165.34	0.28
Calaveras River	10701	100-yr FEMA	12690.00	114.53	133.665	124.72	134.29	0.001235	6.36	2074.13	156.17	0.28
Calaveras River	10541	100-yr FEMA	12690.00	114.91	133.512	124.24	134.06	0.001305	6.00	2238.73	179.23	0.27
Calaveras River	10393	100-yr FEMA	12690.00	114.59	133.254	124.30	133.92	0.000644	6.57	2009.46	177.49	0.29
Calaveras River	10239	100-yr FEMA	12690.00	114.07	133.145	124.12	133.82	0.000645	6.65	2013.05	171.63	0.29
Calaveras River	10083	100-yr FEMA	12690.00	113.54	132.745	124.84	133.67	0.000940	8.05	1911.10	190.89	0.35
Calaveras River	9946	100-yr FEMA	12690.00	113.42	132.804		133.48	0.000733	6.92	2272.82	203.68	0.30
Calaveras River	9803	100-yr FEMA	12690.00	113.16	132.829		133.34	0.000571	5.92	2480.30	207.96	0.26
Calaveras River	9638	100-yr FEMA	12690.00	112.57	132.803		133.23	0.000448	5.31	2548.49	208.35	0.23
Calaveras River	9513	100-yr FEMA	12690.00	112.13	132.679		133.14	0.001224	5.55	2446.19	199.94	0.24
Calaveras River	9375	100-yr FEMA	12690.00	111.66	132.398		133.02	0.000582	6.41	2190.78	181.37	0.28
Calaveras River	9243	100-yr FEMA	12690.00	111.81	132.343	122.64	132.93	0.000576	6.24	2218.77	165.40	0.27
Calaveras River	9066	100-yr FEMA	12690.00	112.19	132.189	122.96	132.82	0.000688	6.58	2275.29	186.73	0.28
Calaveras River	8893	100-yr FEMA	12690.00	112.59	132.282	122.39	132.65	0.000381	4.97	2798.14	269.54	0.22
Calaveras River	8759	100-yr FEMA	12690.00	112.88	132.167	122.23	132.58	0.000775	5.27	2710.12	240.15	0.23
Calaveras River	8603	100-yr FEMA	12690.00	113.22	131.733	122.87	132.43	0.000784	6.86	2143.78	183.73	0.30
Calaveras River	8463	100-yr FEMA	12690.00	113.50	131.654	122.83	132.31	0.000703	6.67	2168.60	173.20	0.29
Calaveras River	8349	100-yr FEMA	12690.00	113.69	131.618	122.77	132.22	0.000621	6.35	2265.47	191.15	0.28
Calaveras River	8157	100-yr FEMA	12690.00	113.75	131.577		132.08	0.000537	5.89	2547.21	227.41	0.26
Calaveras River	8001	100-yr FEMA	12690.00	113.81	131.593	122.26	131.97	0.000390	4.99	2689.11	257.17	0.23
Calaveras River	7867	100-yr FEMA	12690.00	113.91	131.523		131.90	0.000720	5.05	2743.15	239.59	0.23
Calaveras River	7724	100-yr FEMA	12690.00	114.00	131.277		131.77	0.000900	5.76	2416.90	208.90	0.26
Calaveras River	7542	100-yr FEMA	12690.00	113.84	131.020		131.57	0.001292	6.12	2328.30	230.39	0.28
Calaveras River	7405	100-yr FEMA	12690.00	113.22	130.839		131.35	0.001784	6.00	2439.96	261.54	0.28
Calaveras River	7249	100-yr FEMA	12690.00	112.53	130.637	122.62	131.15	0.000991	5.89	2410.00	245.60	0.28
Calaveras River	7093	100-yr FEMA	12690.00	111.81	130.128		130.90	0.002447	7.27	2013.34	217.11	0.32
Calaveras River	6966	100-yr FEMA	12690.00	111.25	129.988		130.60	0.001571	6.56	2234.40	203.92	0.29
Calaveras River	6832	100-yr FEMA	12690.00	111.59	129.836		130.44	0.000931	6.67	2458.57	232.17	0.29
Calaveras River	6696	100-yr FEMA	12690.00	111.27	129.741	121.81	130.27	0.001330	5.95	2437.84	346.13	0.28
Calaveras River	6536	100-yr FEMA	12690.00	113.78	129.799		130.07	0.000469	4.43	3500.82	419.28	0.21
Calaveras River	6387	100-yr FEMA	12690.00	113.12	129.794	121.03	129.99	0.000382	3.57	3647.77	416.62	0.18
Calaveras River	6240	100-yr FEMA	12690.00	113.37	129.748		129.93	0.000322	3.49	3862.18	375.60	0.17
Calaveras River	6123	100-yr FEMA	12690.00	113.18	129.666		129.89	0.000374	3.92	3587.20	357.41	0.19
Calaveras River	5999	100-yr FEMA	12690.00	111.82	129.498		129.82	0.000754	4.77	3114.14	329.48	0.23
Calaveras River	5843	100-yr FEMA	12690.00	106.70	129.264		129.67	0.000990	5.22	2652.75	246.27	0.24
Calaveras River	5676	100-yr FEMA	12690.00	104.65	128.804	117.28	129.51	0.000666	6.74	1939.52	128.05	0.27
Calaveras River	5538	100-yr FEMA	12690.00	108.81	128.730		129.38	0.000942	6.51	2002.05	138.38	0.28
Calaveras River	5384	100-yr FEMA	12690.00	110.47	128.714		129.21	0.000742	5.64	2272.86	177.91	0.27
Calaveras River	5227	100-yr FEMA	12690.00	111.46	128.419		129.05	0.001166	6.41	2046.30	162.34	0.30
Calaveras River	5068	100-yr FEMA	12690.00	111.71	128.145		128.86	0.001156	6.79	1918.31	140.60	0.31
Calaveras River	4942	100-yr FEMA	12690.00	110.00	127.960		128.70	0.001211	6.94	1879.04	136.13	0.31
Calaveras River	4791	100-yr FEMA	12690.00	110.24	127.763		128.53	0.001062	7.08	1878.06	143.75	0.32
Calaveras River	4621	100-yr FEMA	12690.00	107.88	127.606		128.34	0.001051	6.91	1889.73	142.94	0.31
Calaveras River	4465	100-yr FEMA	12690.00	109.69	127.469		128.17	0.001032	6.73	1944.34	152.39	0.31
Calaveras River	4342	100-yr FEMA	12690.00	110.19	127.349		128.02	0.001234	6.63	2008.94	165.95	0.31
Calaveras River	4205	100-yr FEMA	12690.00	109.69	127.144		127.84	0.001376	6.79	2018.94	186.52	0.32
Calaveras River	4049	100-yr FEMA	12690.00	110.41	126.679		127.60	0.001499	7.89	1815.87	178.91	0.38
Calaveras River	3919	100-yr FEMA	12690.00	109.84	126.578		127.35	0.001655	7.15	1912.52	169.78	0.34
Calaveras River	3713	100-yr FEMA	12690.00	110.28	126.269	119.17	127.00	0.001627	6.89	1924.14	168.65	0.33
Calaveras River	3592	100-yr FEMA	12690.00	110.58	126.033	119.19	126.80	0.001526	7.08	1880.73	163.80	0.34
Calaveras River	3452	100-yr FEMA	12690.00	110.16	125.745	118.83	126.59	0.001324	7.49	1822.52	156.23	0.35
Calaveras River	3291	100-yr FEMA	12690.00	109.53	125.589	118.47	126.35	0.001406	7.01	1866.74	161.01	0.34
Calaveras River	3130	100-yr FEMA	12690.00	109.72	125.330	118.26	126.13	0.001284	7.17	1808.37	152.37	0.34
Calaveras River	2976	100-yr FEMA	12690.00	110.75	125.383		125.88	0.000808	5.68	2263.38	217.57	0.28
Calaveras River	2832	100-yr FEMA	12690.00	112.73	125.308		125.76	0.000700	5.40	2365.26	265.62	0.28
Calaveras River	2711	100-yr FEMA	12690.00	110.29	125.263		125.67	0.000581	5.14	2547.25	404.72	0.27
Calaveras River	2550	100-yr FEMA	12690.00	111.60	125.074		125.56	0.000707	5.65	2404.79	346.35	0.30
Calaveras River	2452	100-yr FEMA	12690.00	107.86	125.083		125.47	0.000524	5.00	2577.47	352.18	0.25
Calaveras River	2375	100-yr FEMA	12690.00	107.12	124.754	117.91	125.39	0.000985	6.45	2075.19	286.92	0.32
Calaveras River	2299	100-yr FEMA	12690.00	109.12	124.639		125.32	0.000925	6.61	1944.17	205.36	0.33
Calaveras River	2217	100-yr FEMA	12690.00	105.14	124.667		125.20	0.000836	5.88	2216.23	204.56	0.28
Calaveras River	2196	100-yr FEMA	12690.00	107.66	124.605		125.18	0.000745	6.09	2154.91	228.66	0.30
Calaveras River	2180	100-yr FEMA	12690.00	112.08	124.160		125.12	0.001768	7.92	1690.54	203.02	0.44
Calaveras River	2176	100-yr FEMA	12690.00	113.44	123.868		125.09	0.000391	8.91	1447.69	192.56	0.54
Calaveras River	2172	100-yr FEMA	12690.00	113.44	123.865		125.09	0.000391	8.91	1447.13	192.54	0.54
Calaveras River	2171	100-yr FEMA	12690.00	115.44	122.300	122.17	124.95	0.001239	13.20	995.25	180.74	0.93
Calaveras River	2170.6	100-yr FEMA	12690.00	115.44	122.170	122.17	124.95	0.001333	13.50	971.87	179.76	0.96
Calaveras River	2170	100-yr FEMA	12690.00	113.44	121.421	121.40	124.20	0.001111	14.16	1002.80	169.01	0.91
Calaveras River	2167	100-yr FEMA	12690.00	113.44	121.415	121.40	124.20	0.001114	14.17	1001.81	169.01	0.91
Calaveras River	2162	100-yr FEMA	12690.00	113.17	122.058		123.91	0.000604	11.20	1211.20	181.45	0.70
Calaveras River	2154	100-yr FEMA	12690.00	112.67	122.147	120.32	123.87	0.000509	10.80	1276.19	180.18	0.65
Calaveras River	2148	100-yr FEMA	12690.00	111.62	122.300	119.67	123.71	0.000391	9.82	1399.33	179.92	0.56
Calaveras River	2143	100-yr FEMA	12690.00	110.79	122.131		123.67	0.000359	10.05	1457.63	178.11	0.56
Calaveras River	2127	100-yr FEMA	12690.00	108.67	122.631	116.78	123.45	0.000175	7.46	1861.14	181.17	0.37
Calaveras River	2104	100-yr FEMA	12690.00	106.48	122.837		123.36	0.000093	5.88	2289.67	183.20	0.27
Calaveras River	2096	100-yr FEMA	12690.00	107.78	122.749		123.35	0.000773	6.29	2174.25	183.61	0.30
Calaveras River	2091	100-yr FEMA	12690.00	108.45	122.724		123.34	0.001005	6.41	2147.42	184.67	0.31
Calaveras River	2059	100-yr FEMA	12690.00	101.89	122.876		123.25	0.000346	4.92	2614.93	187.21	0.20
Calaveras River	1996	100-yr FEMA	12690.00	95.67	122.940		123.19	0.000430	4.02	3252.38	190.67	0.16
Calaveras River	1908	100-yr FEMA	12690.00	101.26	122.880		123.14	0.000576	4.14	3143.89	207.78	0.18

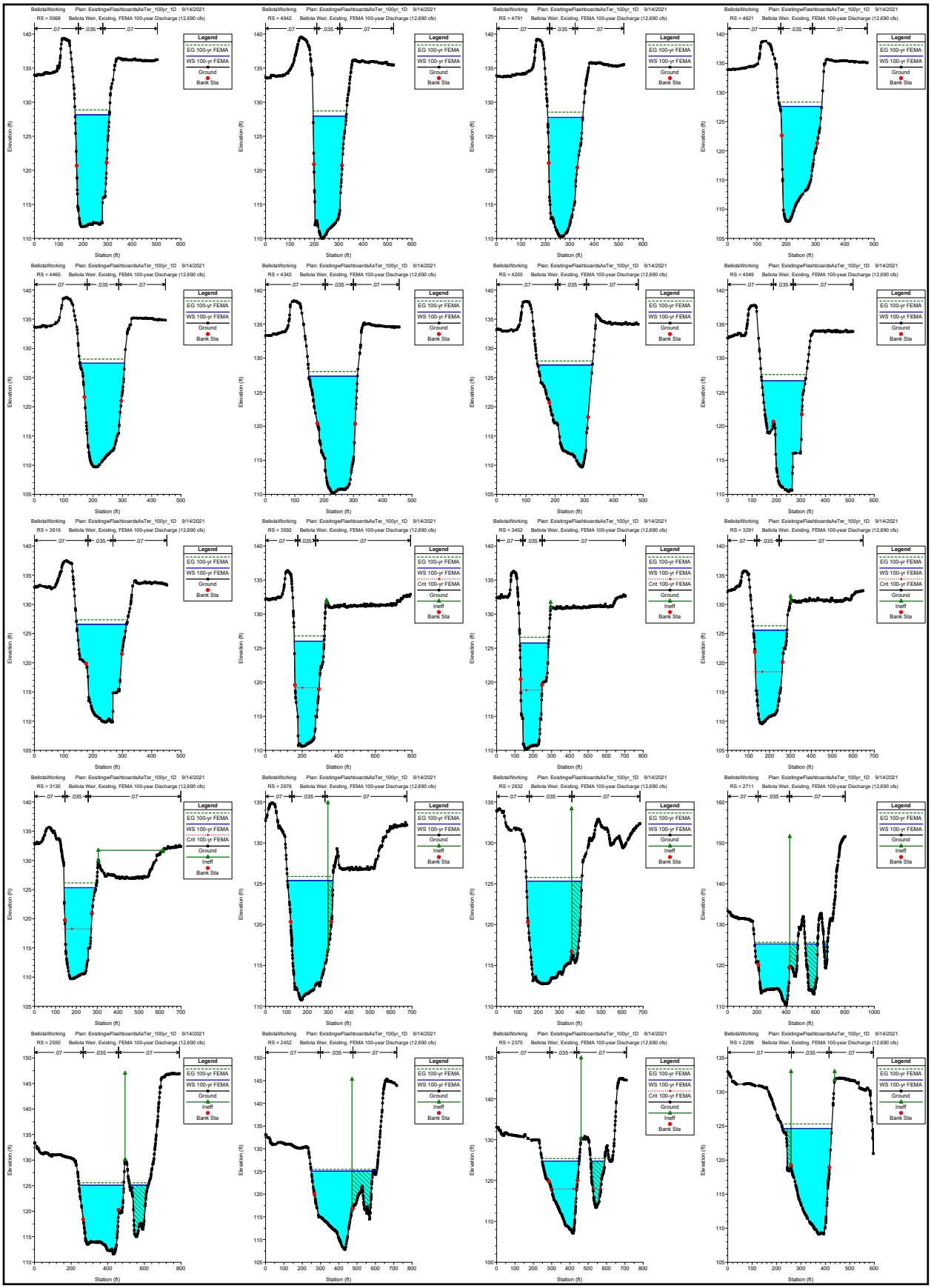
HEC-RAS Plan: ExistingwFlashboardsAsTer_100yr_1D River: Calaveras River Reach: Calaveras River Profile: 100-yr FEMA (Continued)

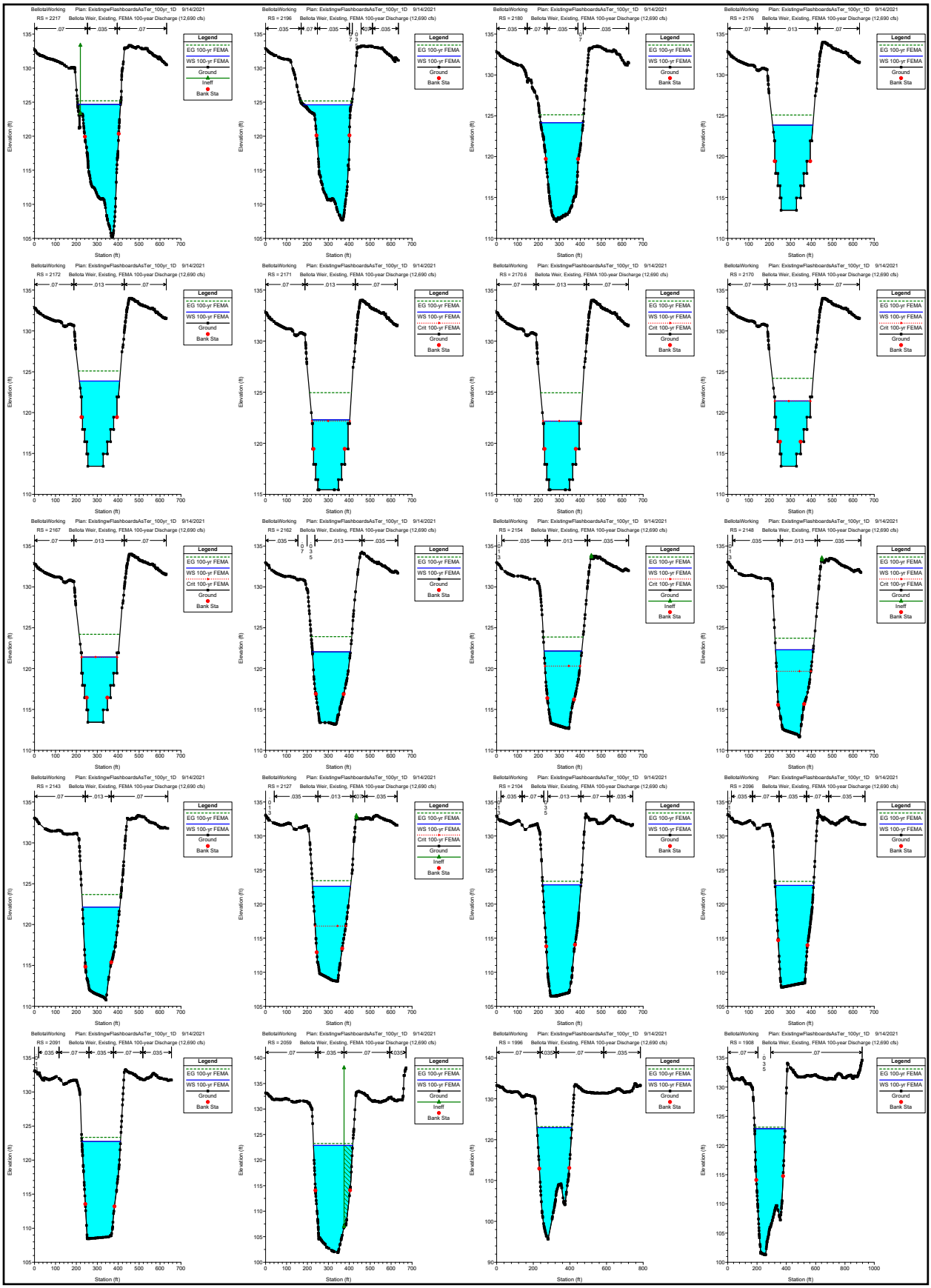
Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
Calaveras River	1795	100-yr FEMA	12690.00	103.28	122.823		123.07	0.000576	4.09	3292.97	255.84	0.19
Calaveras River	1762	100-yr FEMA	12690.00	104.00	122.817	113.11	123.05	0.000702	3.92	3390.04	267.57	0.18
Calaveras River	1646	100-yr FEMA	12690.00	104.29	122.750	113.28	122.98	0.000501	3.90	3446.61	277.62	0.18
Calaveras River	1582	100-yr FEMA	12690.00	103.61	122.717		122.95	0.000533	3.88	3377.75	261.48	0.18
Calaveras River	1474	100-yr FEMA	12690.00	103.63	122.658		122.89	0.000405	3.99	3308.95	261.97	0.18
Calaveras River	1343	100-yr FEMA	12690.00	103.34	122.537	111.55	122.82	0.000803	4.31	3104.78	253.25	0.19
Calaveras River	1219	100-yr FEMA	12690.00	102.41	122.291	111.79	122.69	0.001051	5.13	2570.77	196.98	0.22
Calaveras River	1056	100-yr FEMA	12690.00	102.58	121.182		122.36	0.002617	8.89	1564.85	126.21	0.39
Calaveras River	938	100-yr FEMA	12690.00	102.35	120.907		122.02	0.002795	8.59	1576.70	122.12	0.38
Calaveras River	859	100-yr FEMA	12690.00	102.42	121.012	111.87	121.75	0.001527	6.94	1926.24	141.77	0.30
Calaveras River	850		Bridge									
Calaveras River	796	100-yr FEMA	12690.00	102.40	120.538		121.37	0.001294	7.42	1811.56	145.90	0.33
Calaveras River	734	100-yr FEMA	12690.00	101.87	120.505	111.99	121.24	0.001360	6.97	1923.75	145.94	0.31
Calaveras River	608	100-yr FEMA	12690.00	101.71	120.114	111.79	121.05	0.001455	7.78	1671.00	111.49	0.34
Calaveras River	436	100-yr FEMA	12690.00	101.69	119.752	111.84	120.73	0.002317	7.98	1643.66	114.28	0.35
Calaveras River	290	100-yr FEMA	12690.00	102.16	118.588	112.86	120.25	0.003541	10.44	1283.00	100.13	0.47
Calaveras River	141	100-yr FEMA	12690.00	101.57	118.487	111.37	119.73	0.002001	9.00	1457.33	103.29	0.40

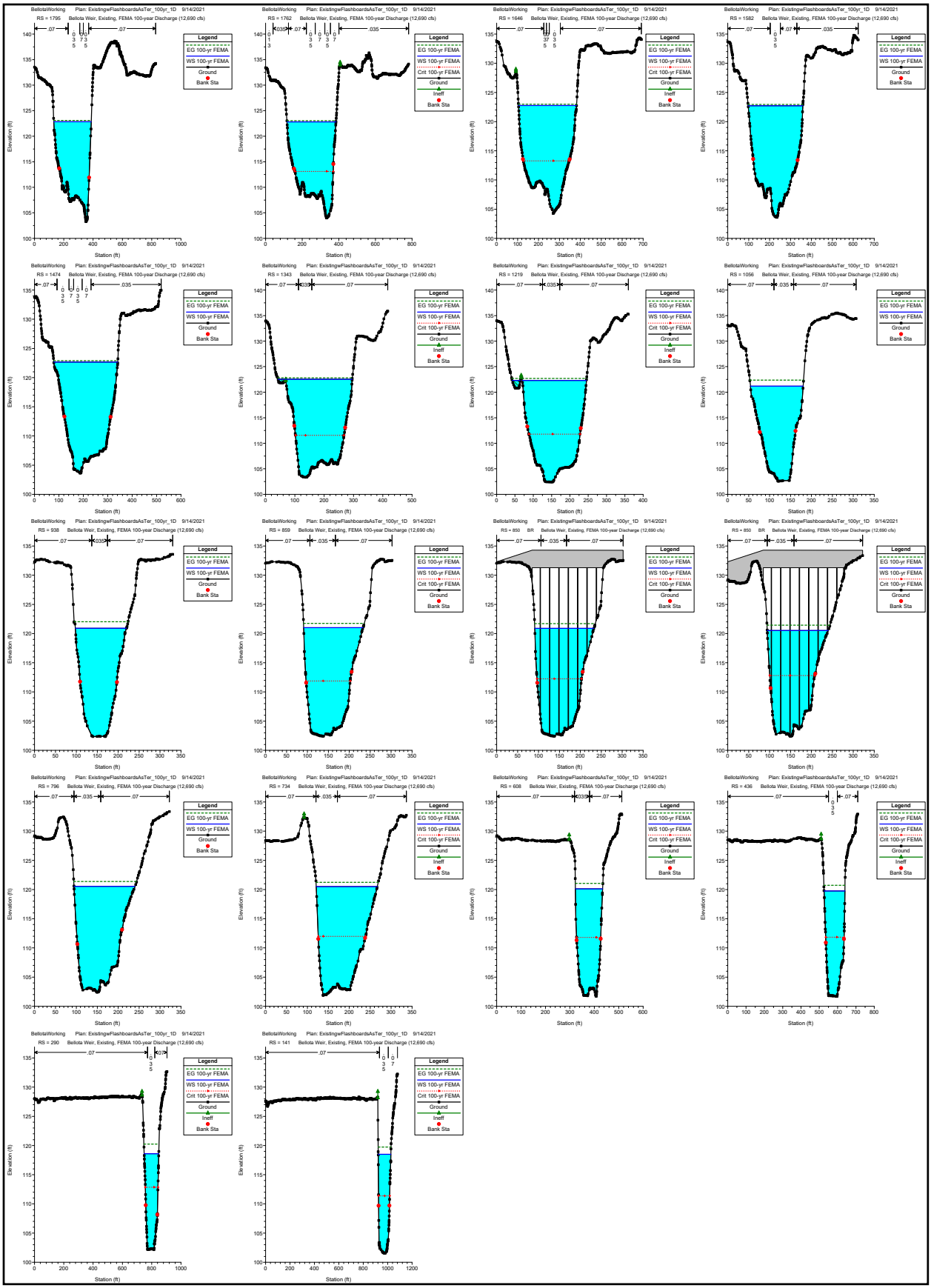

BellotaWorking Plan: ExistingwFlashboardsAsTer_100yr_1D 9/14/2021
Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
Calaveras River Calaveras River
Legend
EG 100-yr FEMA
WS 100-yr FEMA
Crit 100-yr FEMA
Ground
Elevation (ft)
Main Channel Distance (ft)
140
130
120
110
100
90
0
2000
4000
6000
8000
10000
12000
436
734
938
1219
1474
1762
1996
2180
2375
2711
2976
3291
3592
3919
4205
4465
4791
5068
5384
5676
5999
6240
6536
6832
7093
7405
7724
8001
8349
8603
8893
9243
9513
9803
10083
10393
10701


BellotaWorking Plan: ExistingwFlashboardsAsTer_100yr_1D 9/14/2021
RS = 3713 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 10829 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 10701 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 10541 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 10393 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 10239 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 10083 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 9946 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 9803 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 9638 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 9513 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 9375 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 9243 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 9066 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 8893 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 8759 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 8603 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 8463 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 8349 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 8157 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
.07
.035
.07
Legend
EG 100-yr FEMA
WS 100-yr FEMA
Crit 100-yr FEMA
Ground
Ineff
Bank Sta
Elevation (ft)
Station (ft)


BellotaWorking Plan: ExistingwFlashboardsAsTer_100yr_1D 9/14/2021
RS = 8001 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 7867 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 7724 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 7542 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 7405 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 7249 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 7093 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 6966 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 6832 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 6696 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 6536 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 6387 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 6240 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 6123 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 5999 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 5843 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 5676 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 5538 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 5384 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 5227 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
Legend
EG 100-yr FEMA
WS 100-yr FEMA
Crit 100-yr FEMA
Ground
Ineff
Bank Sta
Elevation (ft)
Station (ft)


BellotaWorking Plan: ExistingwFlashboardsAsTer_100yr_1D 9/14/2021
RS = 5068 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 4942 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 4791 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 4621 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 4465 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 4342 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 4205 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 4049 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 3919 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 3592 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 3452 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 3291 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 3130 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 2976 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 2832 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 2711 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 2550 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 2452 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 2375 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 2299 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
.07 .035 .07
Legend
EG 100-yr FEMA
WS 100-yr FEMA
Crit 100-yr FEMA
Ground
Ineff
Bank Sta
Elevation (ft)
Station (ft)


BellotaWorking Plan: ExistingwFlashboardsAsTer_100yr_1D 9/14/2021
RS = 2217 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 2196 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 2180 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 2176 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 2172 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 2171 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 2170.6 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 2170 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 2167 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 2162 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 2154 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 2148 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 2143 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 2127 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 2104 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 2096 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 2091 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 2059 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 1996 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 1908 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
Legend
EG 100-yr FEMA
WS 100-yr FEMA
Crit 100-yr FEMA
Ground
Ineff
Bank Sta
Elevation (ft)
Station (ft)


BellotaWorking Plan: ExistingwFlashboardsAsTer_100yr_1D 9/14/2021
RS = 1795 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 1762 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 1646 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 1582 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 1474 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 1343 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
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RS = 1056 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 938 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 859 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 850 BR Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 850 BR Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
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RS = 734 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 608 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 436 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 290 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 141 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
Legend
EG 100-yr FEMA
WS 100-yr FEMA
Crit 100-yr FEMA
Ground
Ineff
Bank Sta
Elevation (ft)
Station (ft)

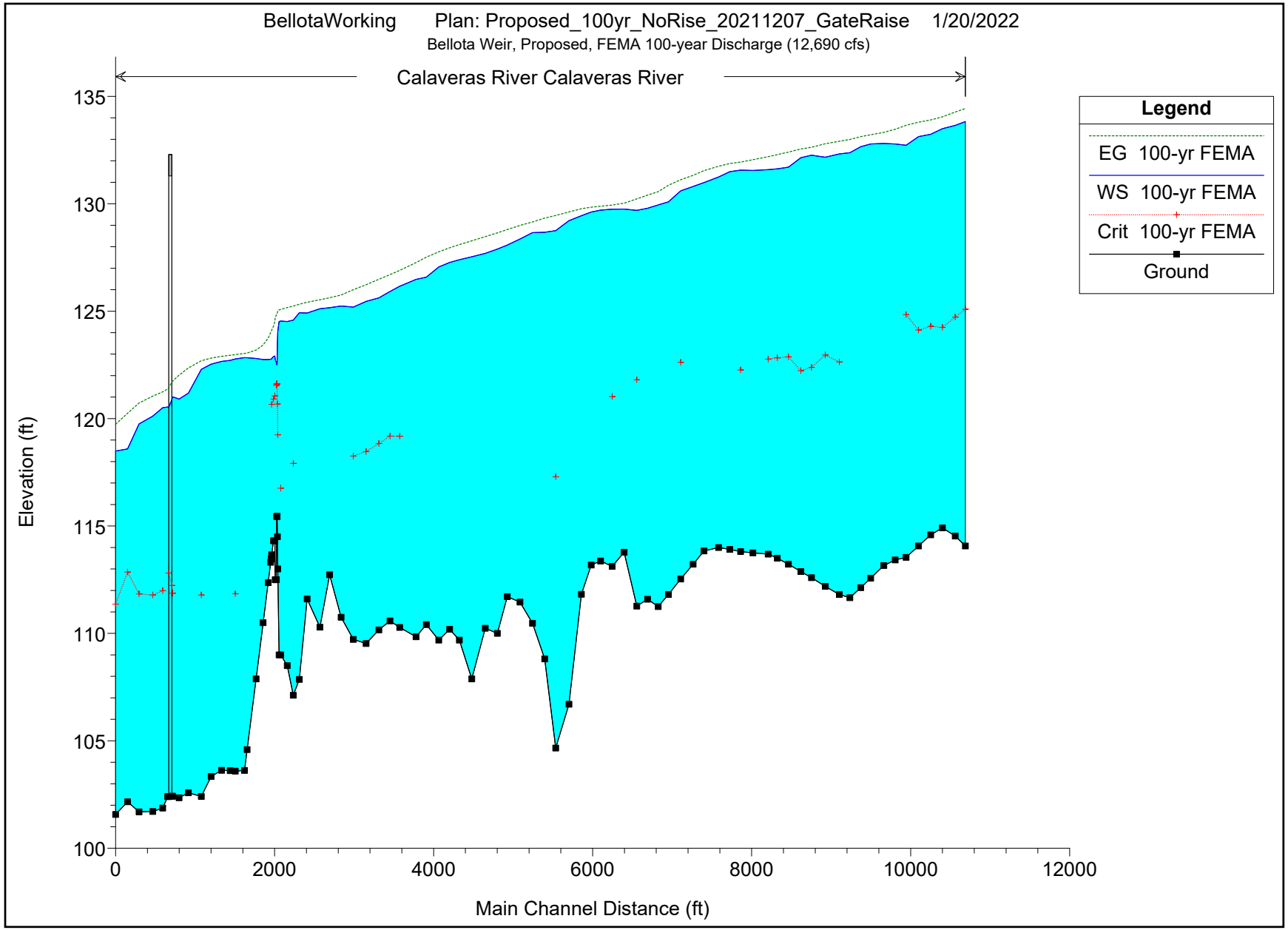
Proposed Conditions: Hydraulic Output Table, Water Surface Profile, and Cross-sections

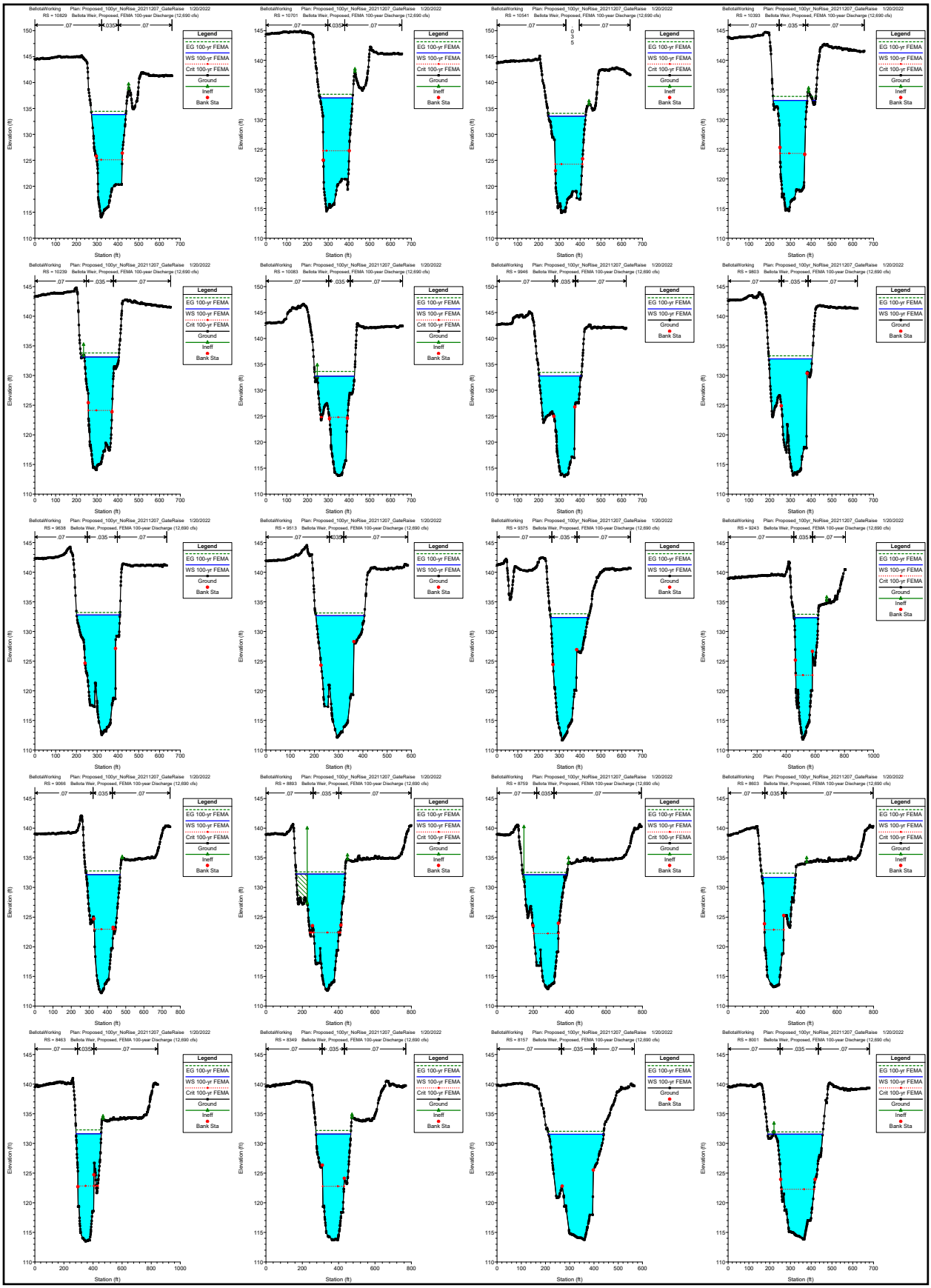
HEC-RAS Plan: Proposed_100yr_NoRise_20211207_GateRaise River: Calaveras River Reach: Calaveras River Profile: 100-yr FEMA

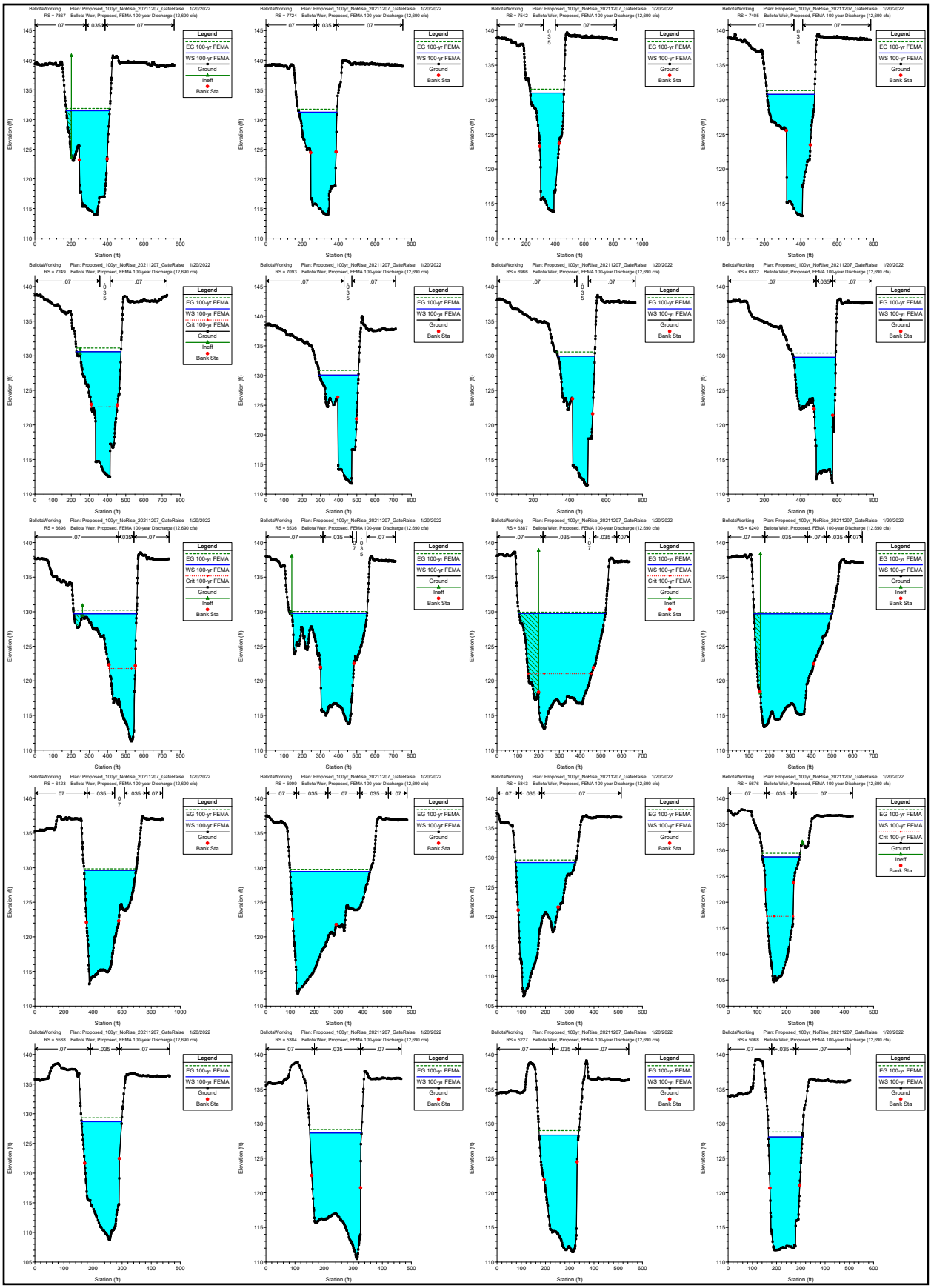
Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
Calaveras River	10829	100-yr FEMA	12690.00	114.07	133.82	125.09	134.44	0.001319	6.35	2121.69	165.28	0.28
Calaveras River	10701	100-yr FEMA	12690.00	114.53	133.65	124.72	134.27	0.001240	6.36	2071.63	156.08	0.28
Calaveras River	10541	100-yr FEMA	12690.00	114.91	133.50	124.24	134.04	0.001310	6.00	2235.67	179.12	0.27
Calaveras River	10393	100-yr FEMA	12690.00	114.59	133.24	124.30	133.90	0.000646	6.58	2006.54	177.27	0.29
Calaveras River	10239	100-yr FEMA	12690.00	114.07	133.13	124.12	133.80	0.000648	6.66	2010.00	171.21	0.29
Calaveras River	10083	100-yr FEMA	12690.00	113.54	132.72	124.84	133.65	0.000945	8.07	1907.43	190.83	0.35
Calaveras River	9946	100-yr FEMA	12690.00	113.42	132.78		133.47	0.000736	6.93	2268.67	203.55	0.30
Calaveras River	9803	100-yr FEMA	12690.00	113.16	132.81		133.32	0.000573	5.93	2476.07	207.88	0.26
Calaveras River	9638	100-yr FEMA	12690.00	112.57	132.78		133.22	0.000450	5.32	2544.22	208.26	0.23
Calaveras River	9513	100-yr FEMA	12690.00	112.13	132.66		133.12	0.001230	5.56	2441.96	199.84	0.24
Calaveras River	9375	100-yr FEMA	12690.00	111.66	132.38		133.00	0.000585	6.42	2186.75	181.20	0.28
Calaveras River	9243	100-yr FEMA	12690.00	111.81	132.32	122.64	132.91	0.000578	6.25	2215.07	165.15	0.27
Calaveras River	9066	100-yr FEMA	12690.00	112.19	132.17	122.96	132.79	0.000692	6.59	2270.92	186.61	0.28
Calaveras River	8893	100-yr FEMA	12690.00	112.59	132.26	122.39	132.63	0.000383	4.98	2793.31	269.46	0.22
Calaveras River	8759	100-yr FEMA	12690.00	112.88	132.14	122.23	132.55	0.000779	5.28	2704.49	239.82	0.23
Calaveras River	8603	100-yr FEMA	12690.00	113.22	131.71	122.87	132.40	0.000788	6.88	2139.04	183.47	0.30
Calaveras River	8463	100-yr FEMA	12690.00	113.50	131.63	122.83	132.29	0.000707	6.68	2164.04	173.11	0.29
Calaveras River	8349	100-yr FEMA	12690.00	113.69	131.59	122.77	132.19	0.000625	6.37	2260.35	191.07	0.28
Calaveras River	8157	100-yr FEMA	12690.00	113.75	131.55		132.05	0.000540	5.90	2541.02	227.23	0.26
Calaveras River	8001	100-yr FEMA	12690.00	113.81	131.57	122.26	131.94	0.000393	5.00	2682.78	256.51	0.23
Calaveras River	7867	100-yr FEMA	12690.00	113.91	131.50		131.87	0.000724	5.06	2737.29	239.46	0.23
Calaveras River	7724	100-yr FEMA	12690.00	114.00	131.25		131.75	0.000906	5.77	2410.77	208.69	0.26
Calaveras River	7542	100-yr FEMA	12690.00	113.84	130.99		131.54	0.001302	6.13	2320.99	230.23	0.28
Calaveras River	7405	100-yr FEMA	12690.00	113.22	130.81		131.32	0.001801	6.02	2431.10	261.40	0.28
Calaveras River	7249	100-yr FEMA	12690.00	112.53	130.60	122.62	131.12	0.000999	5.91	2402.02	245.43	0.28
Calaveras River	7093	100-yr FEMA	12690.00	111.81	130.09		130.86	0.002473	7.30	2004.46	216.86	0.33
Calaveras River	6966	100-yr FEMA	12690.00	111.25	129.95		130.57	0.001586	6.58	2225.80	203.56	0.29
Calaveras River	6832	100-yr FEMA	12690.00	111.59	129.79		130.40	0.000941	6.69	2448.31	231.80	0.30
Calaveras River	6696	100-yr FEMA	12690.00	111.27	129.70	121.81	130.23	0.001347	5.97	2424.08	345.89	0.28
Calaveras River	6536	100-yr FEMA	12690.00	113.78	129.75		130.03	0.000475	4.45	3481.61	416.63	0.21
Calaveras River	6387	100-yr FEMA	12690.00	113.12	129.75	121.03	129.94	0.000387	3.59	3632.99	416.25	0.18
Calaveras River	6240	100-yr FEMA	12690.00	113.37	129.70		129.89	0.000326	3.51	3846.03	375.13	0.17
Calaveras River	6123	100-yr FEMA	12690.00	113.18	129.62		129.84	0.000379	3.93	3570.16	356.94	0.19
Calaveras River	5999	100-yr FEMA	12690.00	111.82	129.45		129.77	0.000765	4.79	3097.60	329.08	0.23
Calaveras River	5843	100-yr FEMA	12690.00	106.70	129.21		129.62	0.001004	5.24	2639.76	245.91	0.25
Calaveras River	5676	100-yr FEMA	12690.00	104.66	128.75	117.29	129.46	0.000673	6.76	1932.25	127.79	0.27
Calaveras River	5538	100-yr FEMA	12690.00	108.81	128.67		129.33	0.000953	6.53	1994.20	138.22	0.29
Calaveras River	5384	100-yr FEMA	12690.00	110.47	128.66		129.15	0.000751	5.67	2262.62	177.77	0.27
Calaveras River	5227	100-yr FEMA	12690.00	111.46	128.36		128.99	0.001184	6.44	2036.24	162.06	0.30
Calaveras River	5068	100-yr FEMA	12690.00	111.71	128.08		128.80	0.001173	6.82	1909.08	140.44	0.31
Calaveras River	4942	100-yr FEMA	12690.00	110.00	127.89		128.64	0.001229	6.98	1869.90	135.96	0.31
Calaveras River	4791	100-yr FEMA	12690.00	110.24	127.69		128.47	0.001079	7.11	1867.96	143.53	0.32
Calaveras River	4621	100-yr FEMA	12690.00	107.88	127.53		128.27	0.001068	6.94	1879.39	142.54	0.32
Calaveras River	4465	100-yr FEMA	12690.00	109.69	127.39		128.10	0.001050	6.77	1932.69	152.05	0.31
Calaveras River	4342	100-yr FEMA	12690.00	110.19	127.27		127.95	0.001254	6.67	1995.66	165.73	0.31
Calaveras River	4205	100-yr FEMA	12690.00	109.69	127.06		127.77	0.001406	6.84	2003.12	185.79	0.33
Calaveras River	4049	100-yr FEMA	12690.00	110.41	126.58		127.52	0.001536	7.95	1798.77	178.28	0.39
Calaveras River	3919	100-yr FEMA	12690.00	109.84	126.48		127.26	0.001698	7.21	1895.71	169.31	0.34
Calaveras River	3713	100-yr FEMA	12690.00	110.28	126.16	119.18	126.90	0.001673	6.95	1905.64	168.14	0.33
Calaveras River	3592	100-yr FEMA	12690.00	110.58	125.92	119.19	126.70	0.001572	7.15	1861.85	163.47	0.34
Calaveras River	3452	100-yr FEMA	12690.00	110.16	125.62	118.84	126.49	0.001366	7.56	1803.14	155.78	0.36
Calaveras River	3291	100-yr FEMA	12690.00	109.53	125.46	118.47	126.23	0.001454	7.09	1845.63	160.48	0.34
Calaveras River	3130	100-yr FEMA	12690.00	109.72	125.19	118.26	126.00	0.001329	7.25	1787.03	151.69	0.35
Calaveras River	2976	100-yr FEMA	12690.00	110.75	125.24		125.75	0.000840	5.75	2236.06	217.09	0.29
Calaveras River	2832	100-yr FEMA	12690.00	112.73	125.16		125.63	0.000732	5.47	2333.05	265.01	0.29
Calaveras River	2711	100-yr FEMA	12690.00	110.29	125.11		125.53	0.000607	5.21	2511.45	403.19	0.27
Calaveras River	2550	100-yr FEMA	12690.00	111.60	124.92		125.42	0.000742	5.73	2365.86	343.70	0.30
Calaveras River	2452	100-yr FEMA	12690.00	107.86	124.92		125.32	0.000546	5.07	2542.14	348.97	0.26
Calaveras River	2375	100-yr FEMA	12690.00	107.12	124.58	117.92	125.24	0.001034	6.55	2040.50	276.99	0.33
Calaveras River	2299	100-yr FEMA	12690.00	108.50	124.52		125.17	0.000807	6.48	1961.68	203.48	0.32
Calaveras River	2217	100-yr FEMA	12690.00	109.00	124.54	116.76	125.08	0.000619	5.90	2168.49	212.38	0.28
Calaveras River	2196	100-yr FEMA	12690.00	109.00	124.52		125.07	0.000615	5.92	2143.27	233.88	0.28
Calaveras River	2180	100-yr FEMA	12690.00	113.00	124.06	119.25	125.01	0.001440	7.83	1621.32	204.64	0.42
Calaveras River	2176	100-yr FEMA	12690.00	114.50	123.51	120.68	124.96	0.000558	9.66	1313.77	165.97	0.57
Calaveras River	2172	100-yr FEMA	12690.00	115.44	122.52	121.61	124.86	0.001192	12.29	1032.88	179.00	0.81
Calaveras River	2171	100-yr FEMA	12690.00	115.44	122.51	121.61	124.86	0.001196	12.30	1031.91	178.23	0.82
Calaveras River	2170.6	100-yr FEMA	12690.00	115.44	122.51	121.62	124.86	0.001194	12.29	1032.82	177.96	0.81
Calaveras River	2170	100-yr FEMA	12690.00	115.44	122.51	121.61	124.86	0.001196	12.29	1032.21	178.12	0.82
Calaveras River	2167	100-yr FEMA	12690.00	115.44	122.50	121.61	124.86	0.001200	12.31	1031.01	178.63	0.82
Calaveras River	2162	100-yr FEMA	12690.00	112.50	122.63	121.54	124.80	0.000845	11.82	1074.00	184.53	0.78
Calaveras River	2154	100-yr FEMA	12690.00	112.50	122.69		124.76	0.000799	11.57	1097.03	177.71	0.76
Calaveras River	2148	100-yr FEMA	12690.00	112.50	122.78		124.67	0.000712	11.03	1150.92	184.54	0.72
Calaveras River	2143	100-yr FEMA	12690.00	114.31	122.91	121.06	124.51	0.007357	10.13	1251.75	200.18	0.65
Calaveras River	2127	100-yr FEMA	12690.00	114.31	122.89	120.91	124.30	0.007138	9.53	1330.91	206.43	0.62
Calaveras River	2104	100-yr FEMA	12690.00	113.66	122.79	120.66	124.11	0.006773	9.22	1377.09	210.14	0.61
Calaveras River	2096	100-yr FEMA	12690.00	113.47	122.76		124.05	0.006625	9.12	1391.06	210.33	0.60
Calaveras River	2091	100-yr FEMA	12690.00	113.31	122.76		124.00	0.006275	8.93	1420.88	212.13	0.58
Calaveras River	2059	100-yr FEMA	12690.00	112.36	122.75		123.75	0.004985	8.03	1579.41	218.08	0.53
Calaveras River	1996	100-yr FEMA	12690.00	110.50	122.75		123.43	0.002764	6.61	1919.56	226.35	0.40
Calaveras River	1908	100-yr FEMA	12690.00	107.88	122.80		123.19	0.001189	5.02	2527.33	239.94	0.27

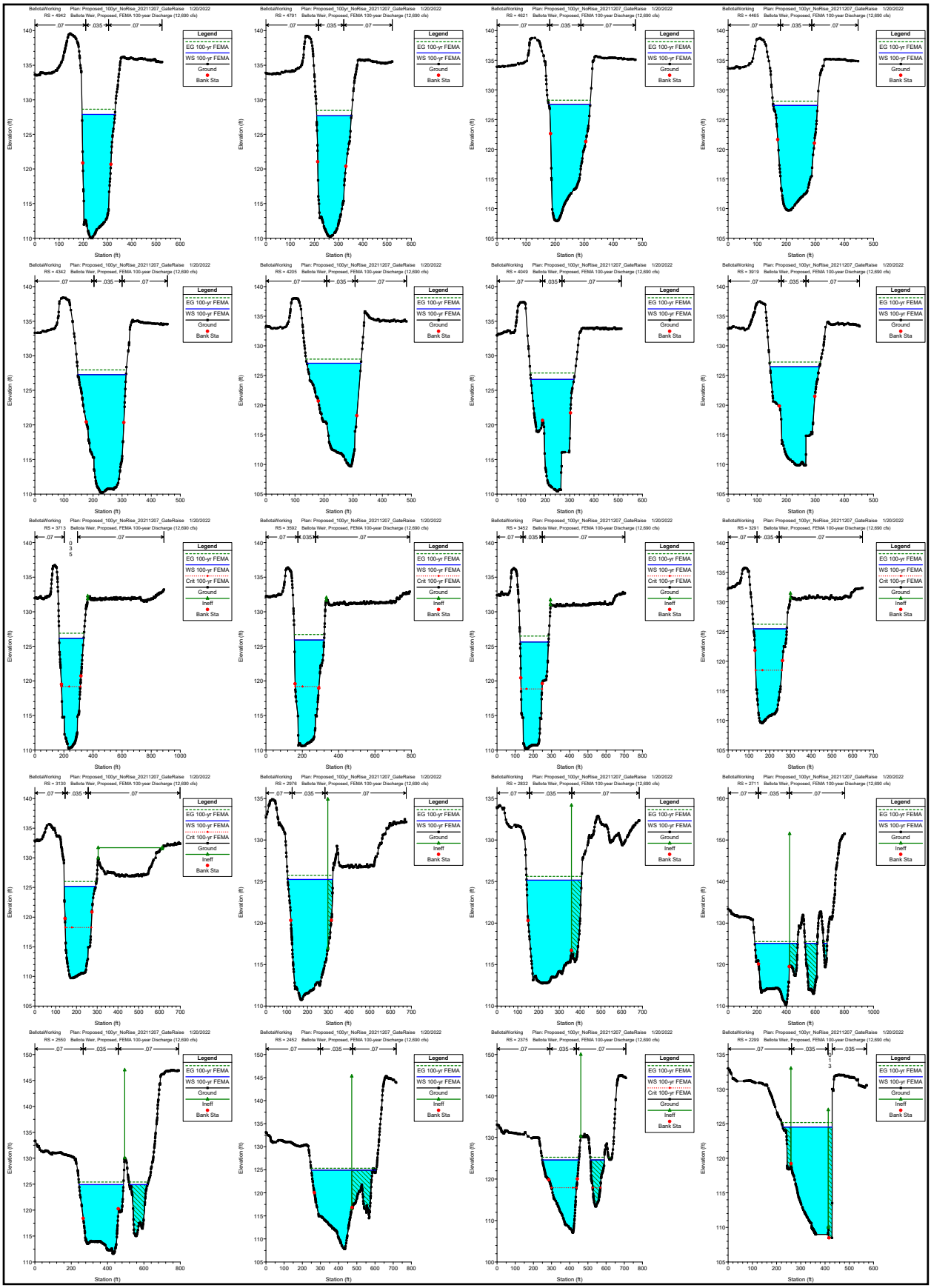
HEC-RAS Plan: Proposed_100yr_NoRise_20211207_GateRaise River: Calaveras River Reach: Calaveras River Profile: 100-yr FEMA (Continued)

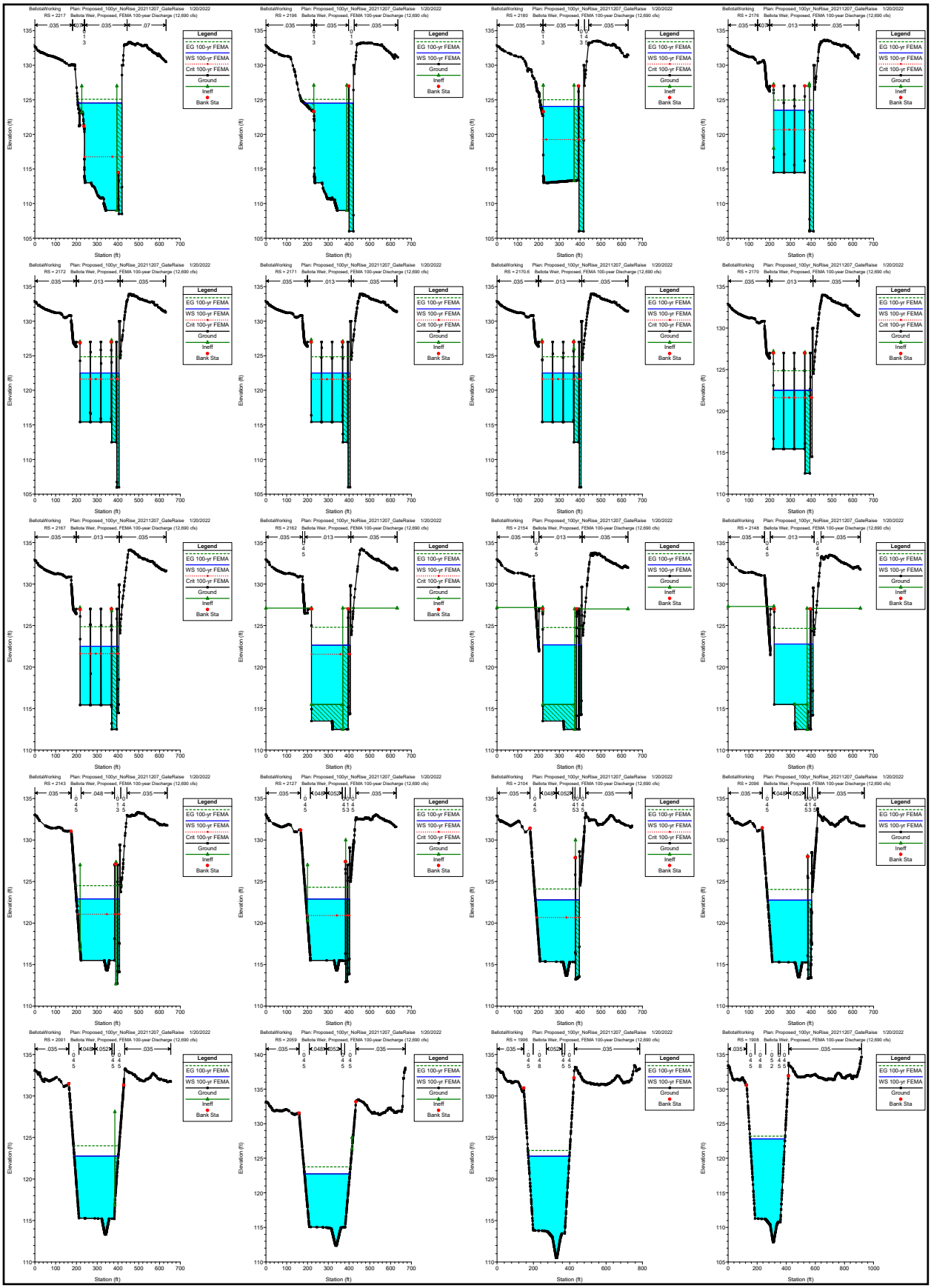
Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
Calaveras River	1795	100-yr FEMA	12690.00	104.59	122.83		123.05	0.000506	3.78	3361.16	258.94	0.18
Calaveras River	1762	100-yr FEMA	12690.00	103.62	122.84		123.03	0.000408	3.51	3615.26	264.64	0.17
Calaveras River	1646	100-yr FEMA	12690.00	103.59	122.78	111.84	122.98	0.000458	3.62	3506.36	269.65	0.18
Calaveras River	1582	100-yr FEMA	12690.00	103.61	122.71		122.95	0.000464	3.89	3263.03	255.27	0.19
Calaveras River	1474	100-yr FEMA	12690.00	103.63	122.66		122.90	0.000429	4.00	3315.63	261.05	0.18
Calaveras River	1343	100-yr FEMA	12690.00	103.34	122.54		122.82	0.000809	4.33	3119.19	253.24	0.19
Calaveras River	1219	100-yr FEMA	12690.00	102.41	122.29	111.79	122.69	0.001051	5.13	2570.77	196.98	0.22
Calaveras River	1056	100-yr FEMA	12690.00	102.58	121.18		122.36	0.002617	8.89	1564.85	126.21	0.39
Calaveras River	938	100-yr FEMA	12690.00	102.35	120.91		122.02	0.002795	8.59	1576.70	122.12	0.38
Calaveras River	859	100-yr FEMA	12690.00	102.42	121.01	111.87	121.75	0.001527	6.94	1926.24	141.77	0.30
Calaveras River	850		Bridge									
Calaveras River	796	100-yr FEMA	12690.00	102.40	120.54		121.37	0.001294	7.42	1811.56	145.90	0.33
Calaveras River	734	100-yr FEMA	12690.00	101.87	120.50	111.99	121.24	0.001360	6.97	1923.75	145.94	0.31
Calaveras River	608	100-yr FEMA	12690.00	101.71	120.11	111.79	121.05	0.001455	7.78	1671.00	111.49	0.34
Calaveras River	436	100-yr FEMA	12690.00	101.69	119.75	111.84	120.73	0.002317	7.98	1643.66	114.28	0.35
Calaveras River	290	100-yr FEMA	12690.00	102.16	118.59	112.86	120.25	0.003541	10.44	1283.00	100.13	0.47
Calaveras River	141	100-yr FEMA	12690.00	101.57	118.49	111.37	119.73	0.002001	9.00	1457.33	103.29	0.40

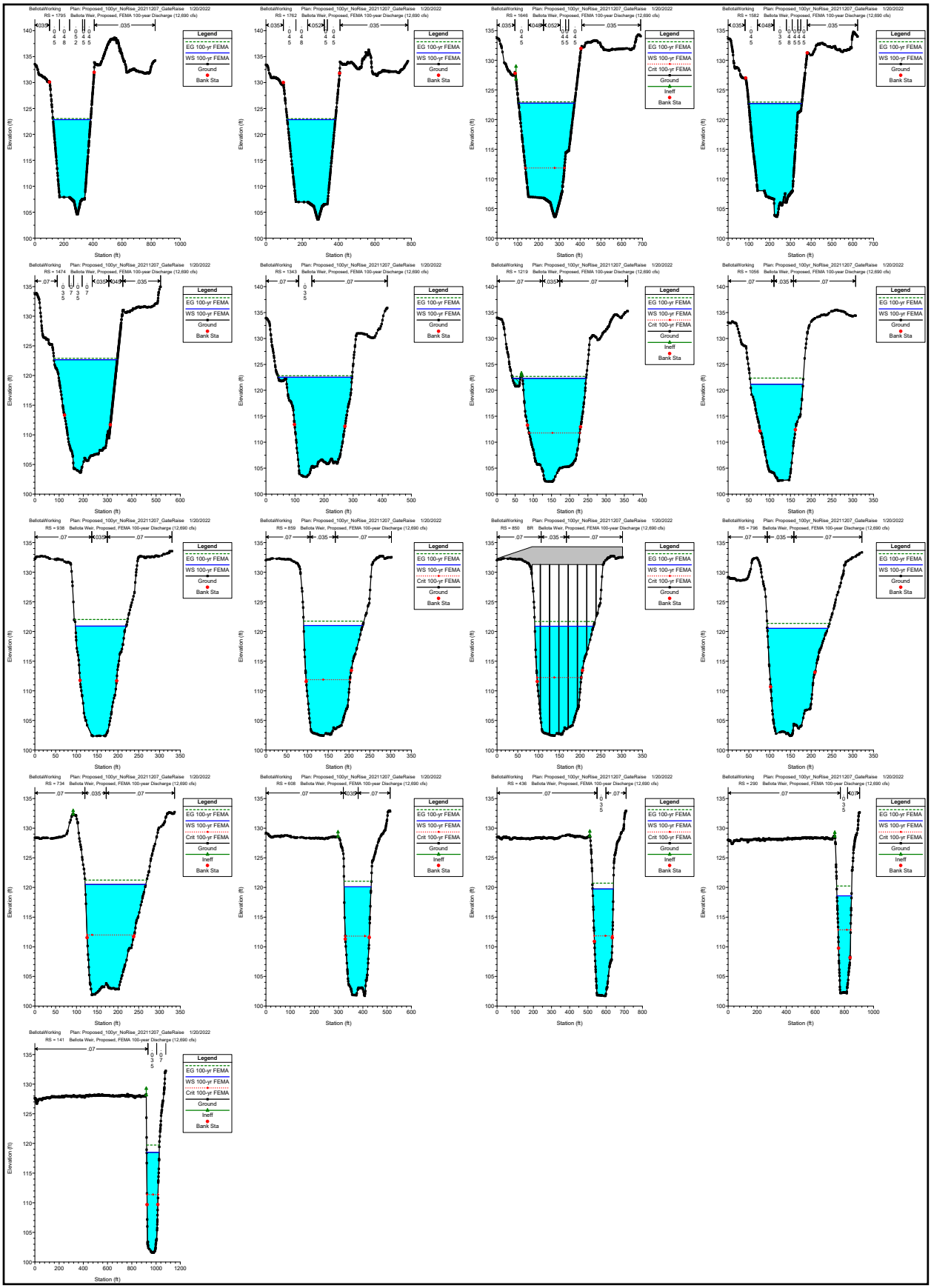

BellotaWorking Plan: Proposed_100yr_NoRise_20211207_GateRaise 1/20/2022
Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
Calaveras River Calaveras River
Legend
EG 100-yr FEMA
WS 100-yr FEMA
Crit 100-yr FEMA
Ground
Elevation (ft)
135
130
125
120
115
110
105
100
0
2000
4000
6000
8000
10000
12000
Main Channel Distance (ft)


BellotaWorking Plan: Proposed_100yr_NoRise_20211207_GateRaise 1/20/2022
RS = 10829 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 10701 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 10541 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 10393 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 10239 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 10083 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 9946 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 9803 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 9638 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 9513 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 9375 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 9243 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 9066 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 8863 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 8759 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 8603 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 8463 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 8349 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 8157 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 8001 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
.07
.035
Legend
EG 100-yr FEMA
WS 100-yr FEMA
Crit 100-yr FEMA
Ground
Ineff
Bank Sta
Elevation (ft)
Station (ft)


BellotaWorking Plan: Proposed_100yr_NoRise_20211207_GateRaise 1/20/2022
RS = 7867 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 7724 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 7542 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 7405 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 7249 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 7093 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
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RS = 6832 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 6696 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 6536 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 6387 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 6240 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 6123 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 5999 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 5843 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 5676 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 5538 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 5384 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 5227 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 5068 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
Legend
EG 100-yr FEMA
WS 100-yr FEMA
Crit 100-yr FEMA
Ground
Ineff
Bank Sta
Elevation (ft)
Station (ft)


BellotaWorking Plan: Proposed_100yr_NoRise_20211207_GateRaise 1/20/2022
RS = 4942 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 4791 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 4621 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 4465 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 4342 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 4205 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 4049 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 3919 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 3713 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 3592 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 3452 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 3291 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 3130 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 2976 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 2852 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 2711 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 2550 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 2452 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 2375 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 2299 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
.07
.035
.05
Legend
EG 100-yr FEMA
WS 100-yr FEMA
Crit 100-yr FEMA
Ground
Ineff
Bank Sta
Elevation (ft)
Station (ft)

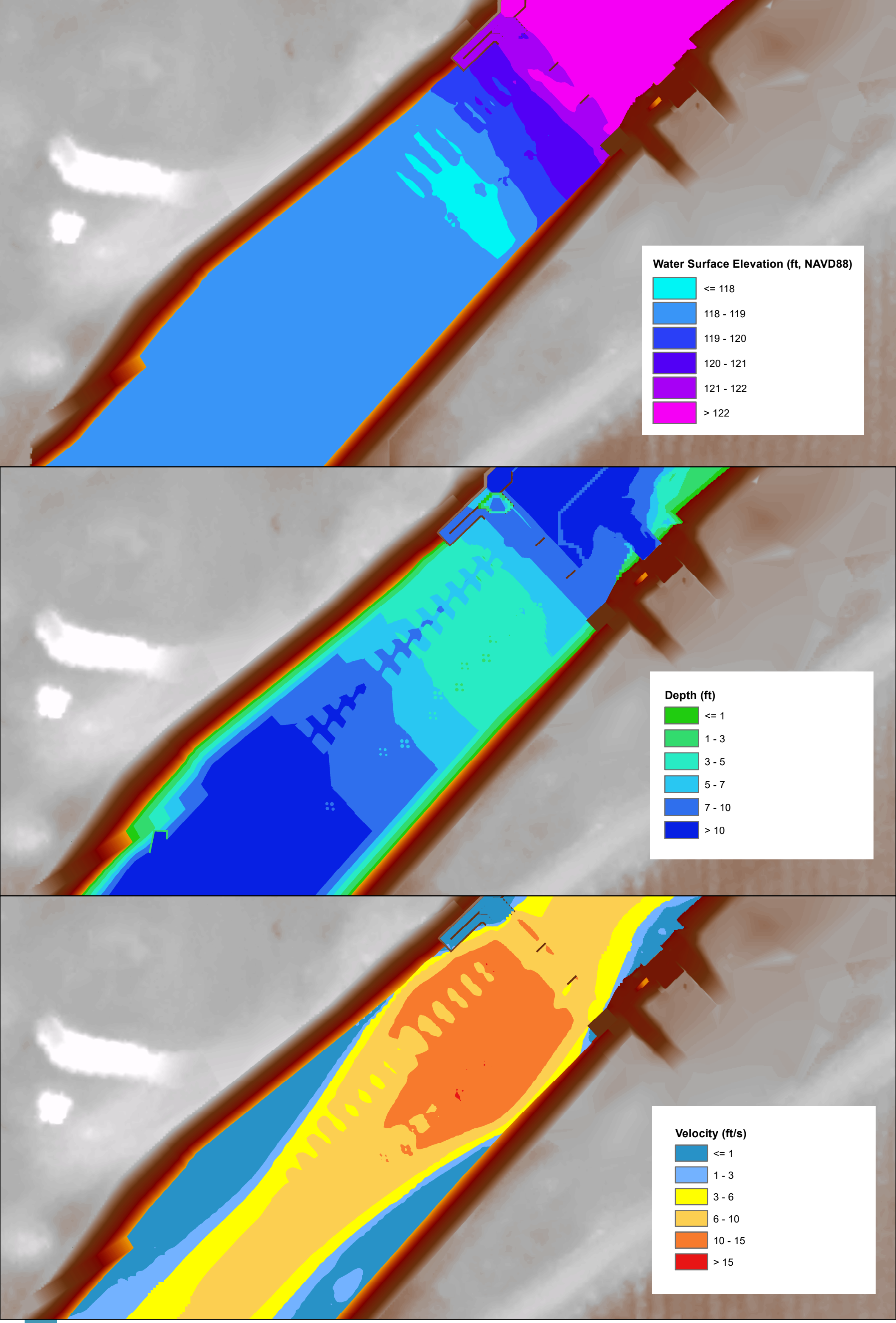

BellotaWorking Plan: Proposed_100yr_NoRise_20211207_GateRaise 1/20/2022
RS = 2217 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 2196 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 2180 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 2176 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 2172 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 2171 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 2170.6 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 2170 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 2167 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 2162 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 2154 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 2148 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 2143 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 2127 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 2104 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 2066 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 2091 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 2059 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 1996 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 1908 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
Legend
EG 100-yr FEMA
WS 100-yr FEMA
Crit 100-yr FEMA
Ground
Ineff
Bank Sta
Elevation (ft)
Station (ft)


BellotaWorking Plan: Proposed_100yr_NoRise_20211207_GateRaise 1/20/2022
RS = 1795 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 1762 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 1646 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 1582 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 1474 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 1343 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 1219 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 1056 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 938 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 859 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 850 BR Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 796 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 734 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 608 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 436 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 290 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 141 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
Legend
EG 100-yr FEMA
WS 100-yr FEMA
Crit 100-yr FEMA
Ground
Ineff
Bank Sta
Elevation (ft)
Station (ft)

Appendix D. 2D HEC-RAS Results

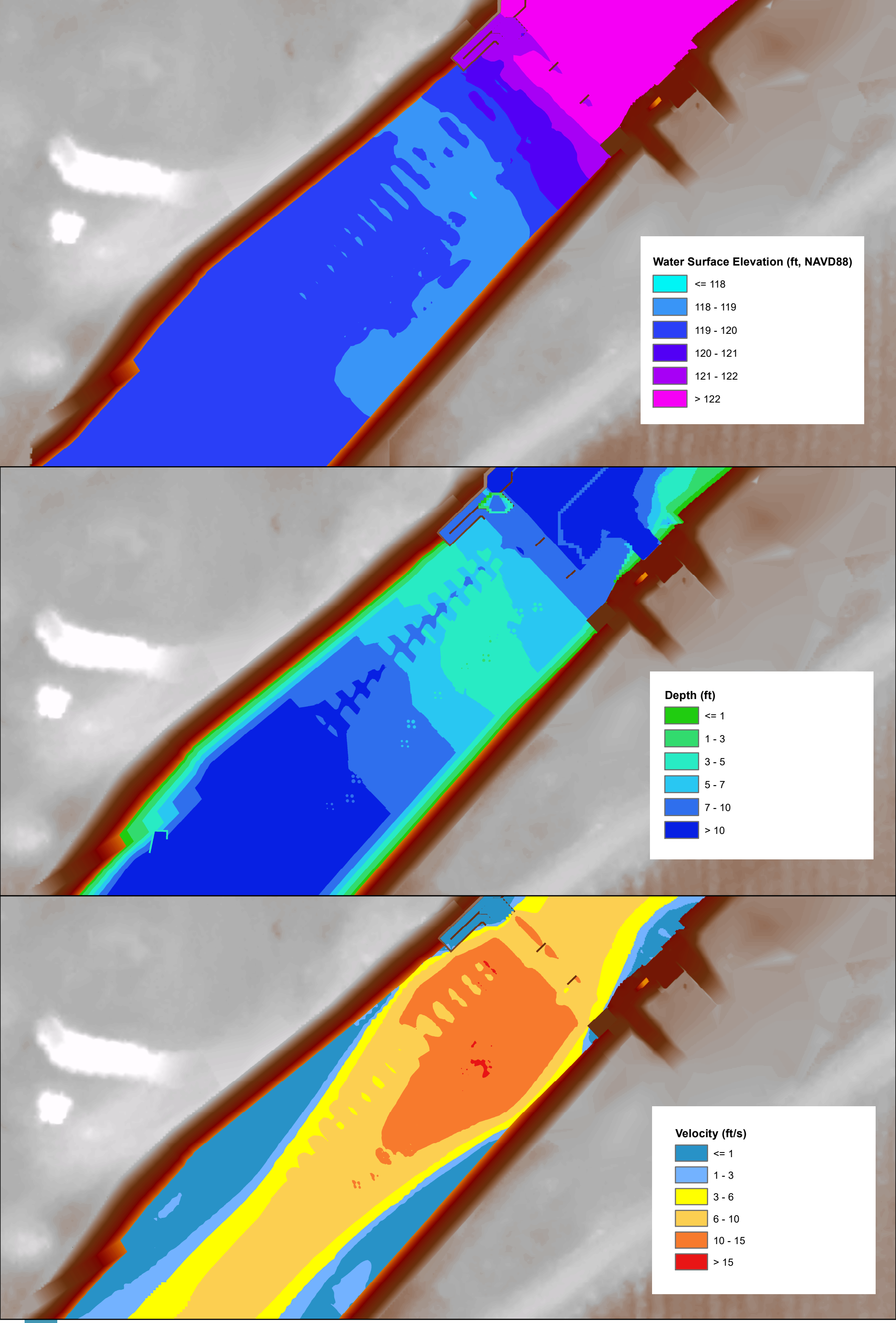
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Water Surface Elevations, Depths, and Velocities for the 10-, 25-, 50-, and 100-year Discharge Events



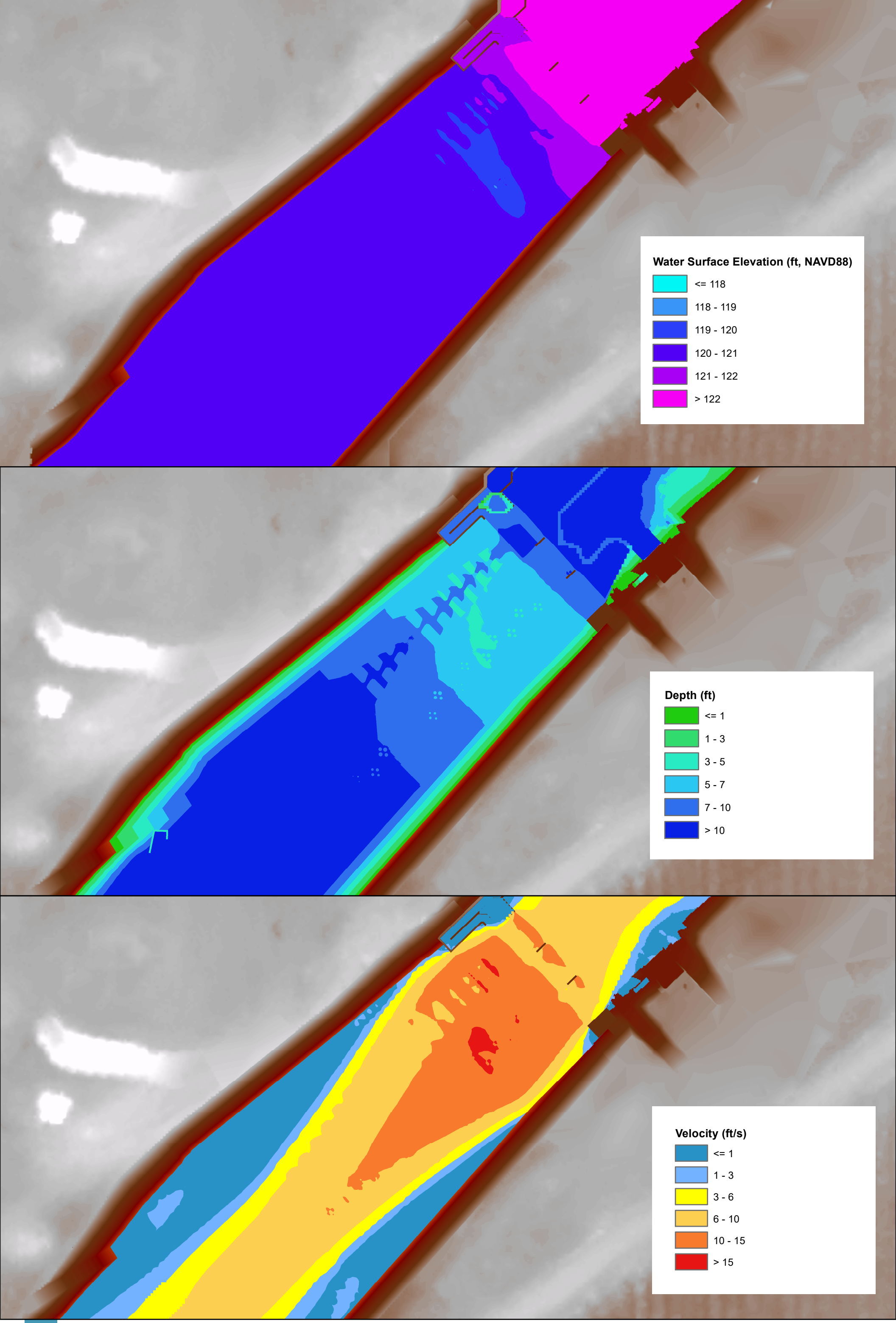
0 Feet 100

EXHIBIT D1
HEC-RAS 2D MODEL RESULTS
USACE 10-YEAR DISCHARGE (9,530 CFS)



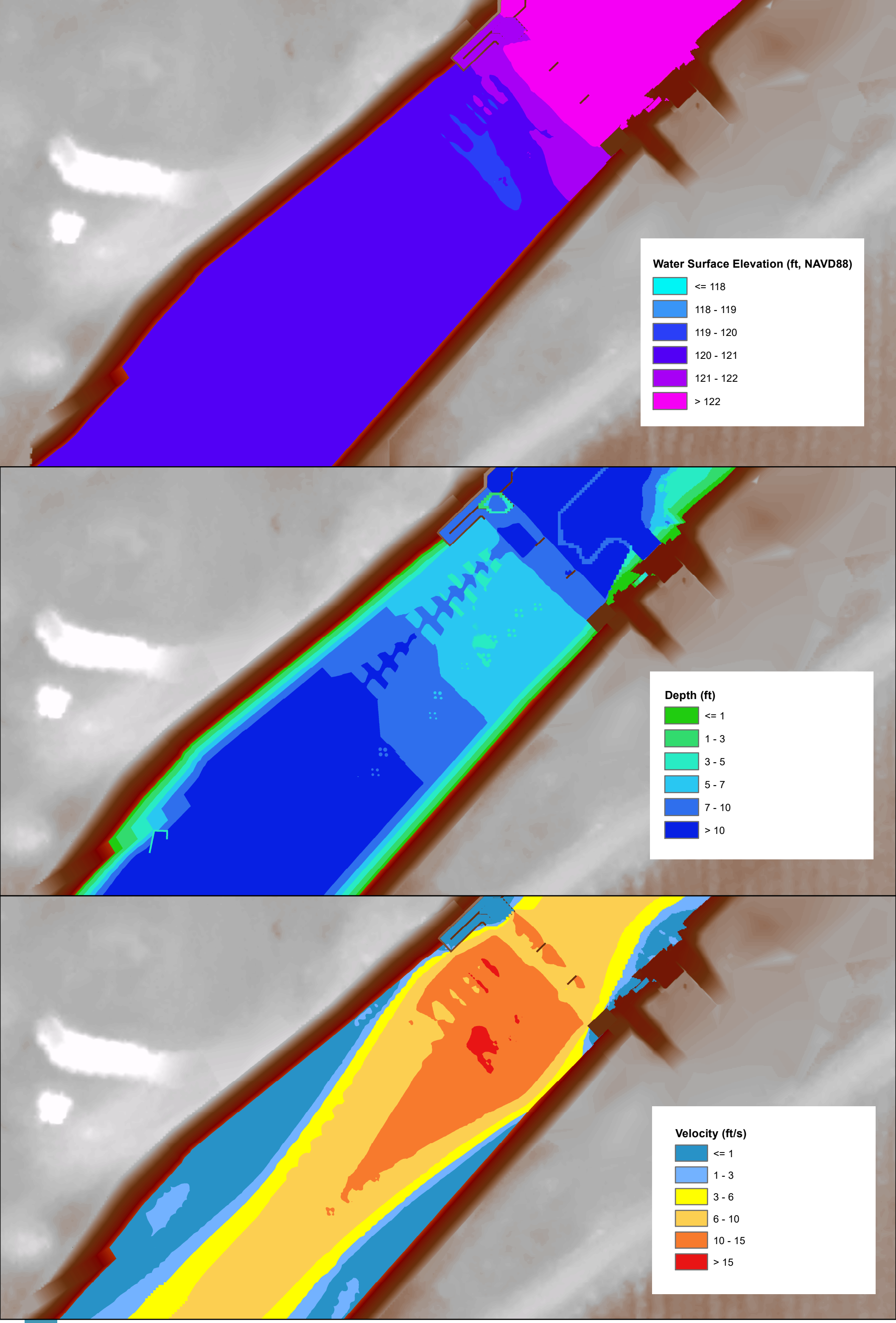
0 Feet 100

EXHIBIT D2
HEC-RAS 2D MODEL RESULTS
USACE 25-YEAR DISCHARGE (10,640 CFS)



0 Feet 100

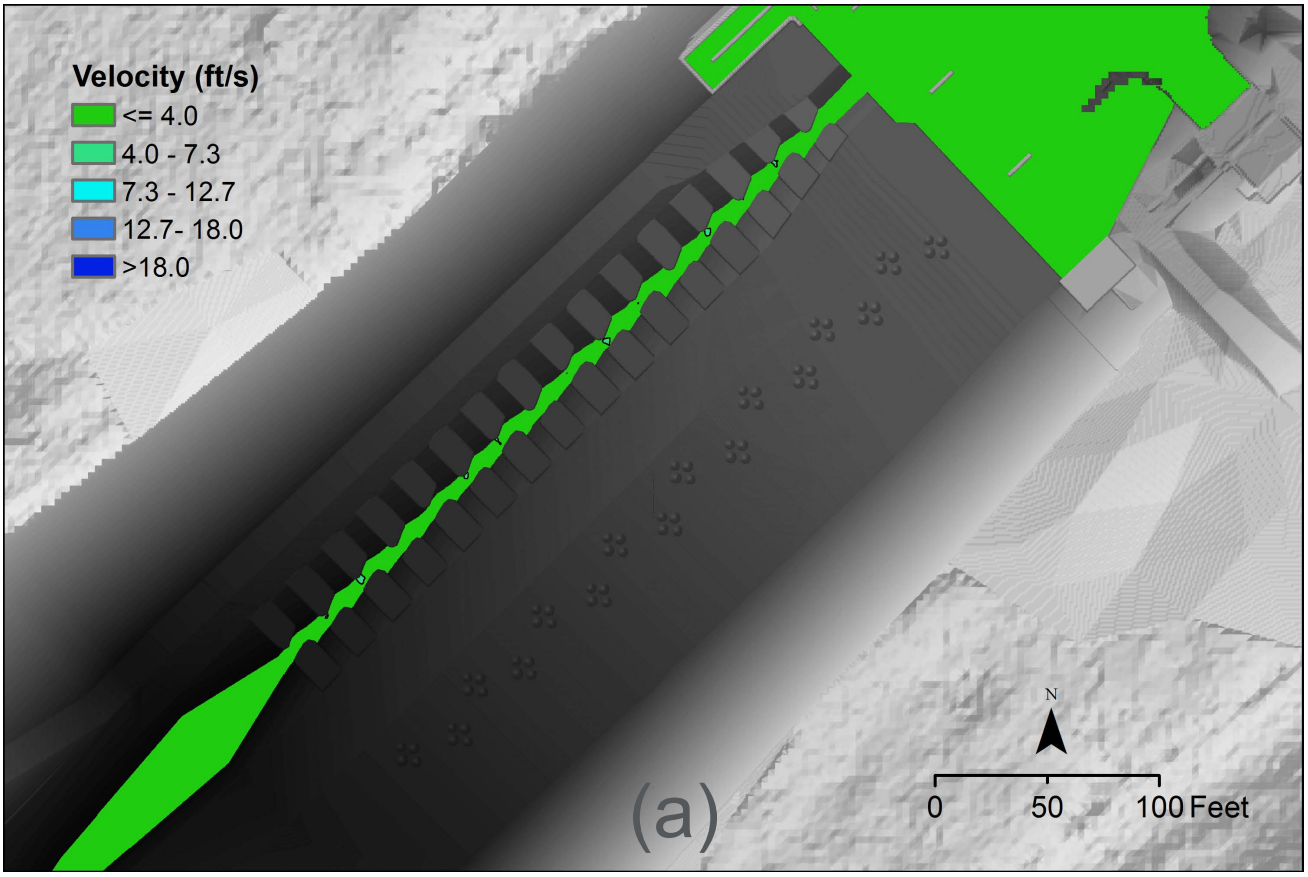
EXHIBIT D3
HEC-RAS 2D MODEL RESULTS
FEMA 50-YEAR DISCHARGE (12,500 CFS)

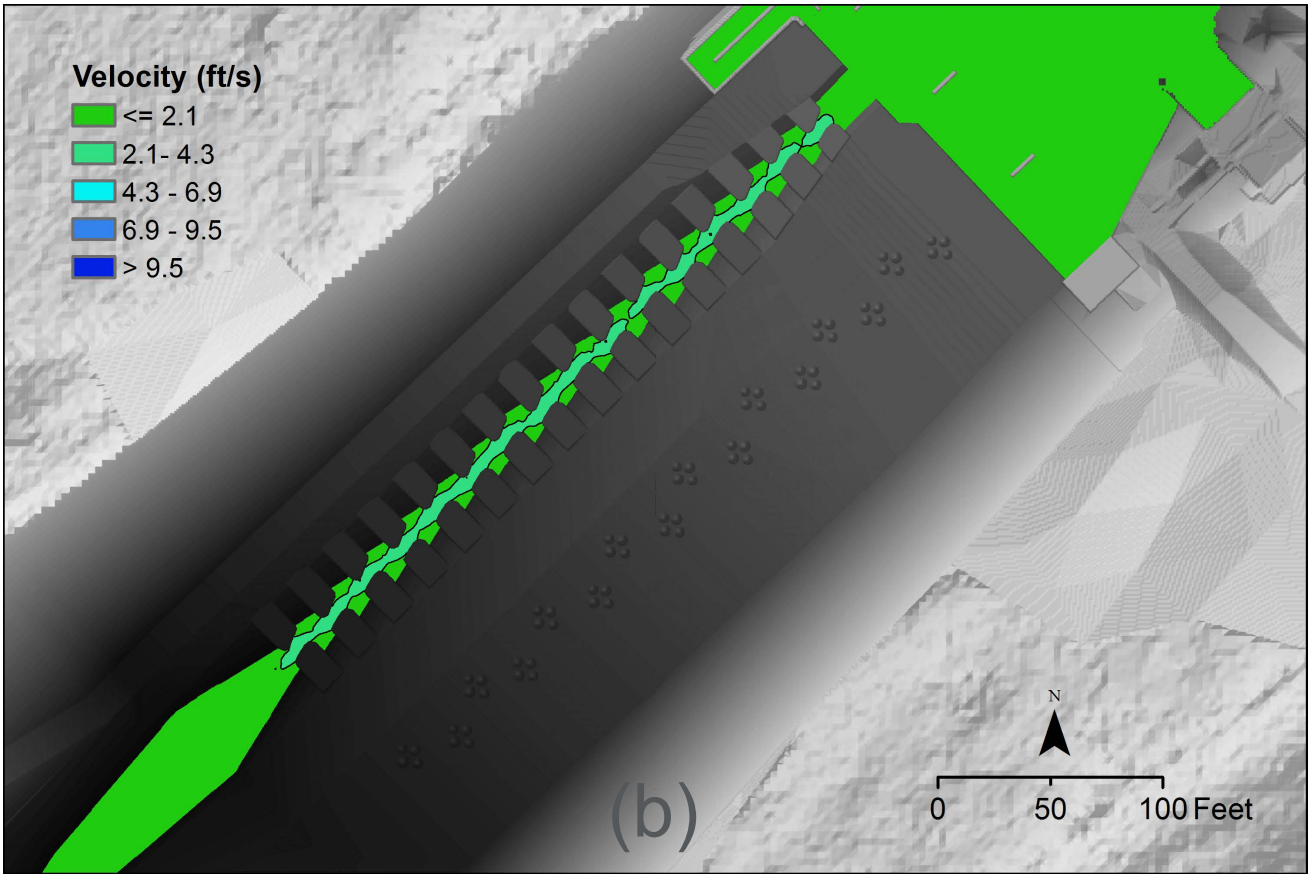


0 Feet 100

EXHIBIT D4
HEC-RAS 2D MODEL RESULTS
FEMA 100-YEAR DISCHARGE (12,690 CFS)

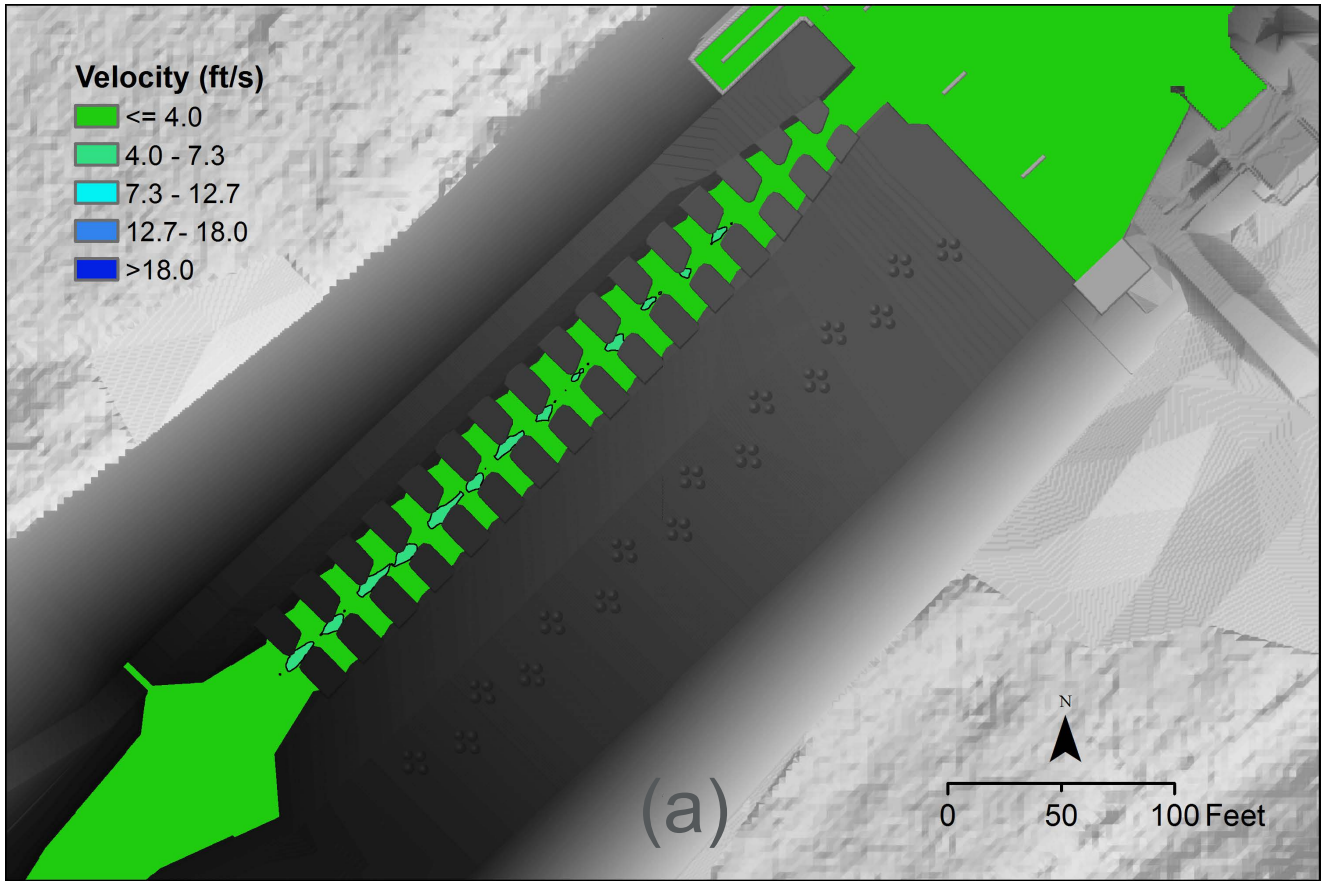
Velocities where Flow Meets Depth Criteria for Fish Passage for Fish Passage Design Flows

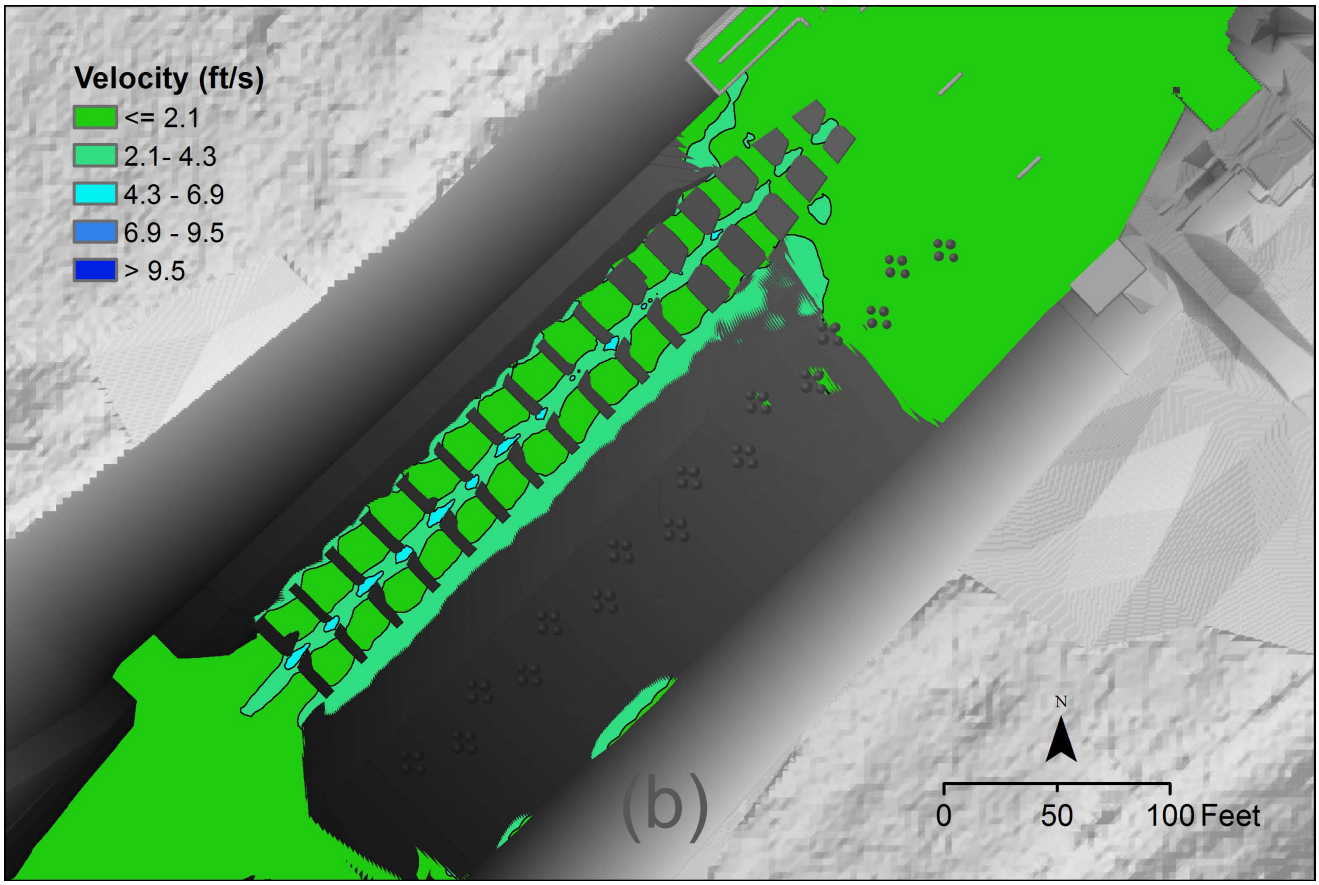




Panel (a) shows Katopodis and Gervais (2016) adult salmonid flow velocity categories where depth meets adult salmonid depth criteria (depth ≥ 0.9 ft); Panel (b) shows Katopodis and Gervais (2016) julvenile salmonid flow velocity categoeis where depth meets juvenile salmonid depth critiera (depth ≥ 0.4 ft)

EXHIBIT D5

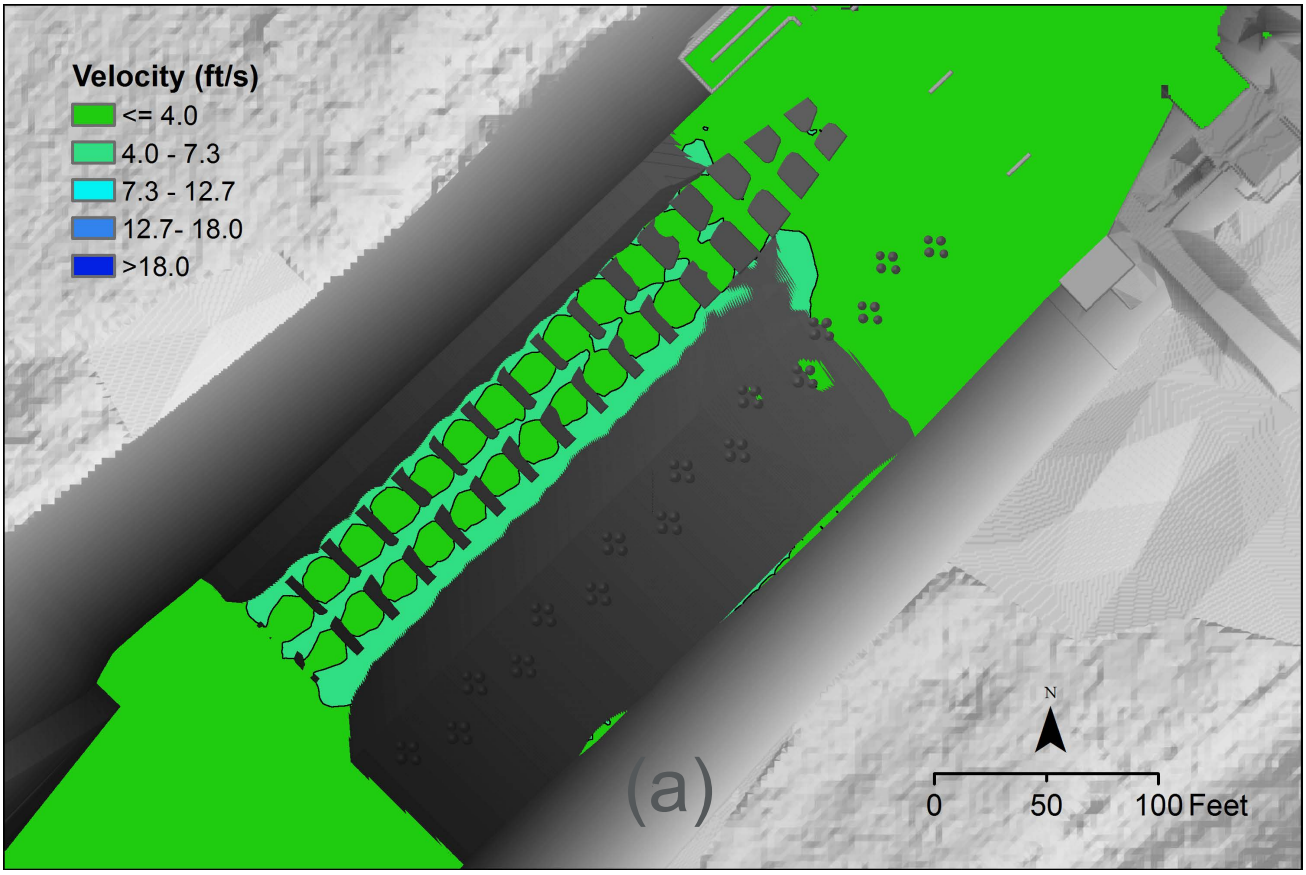


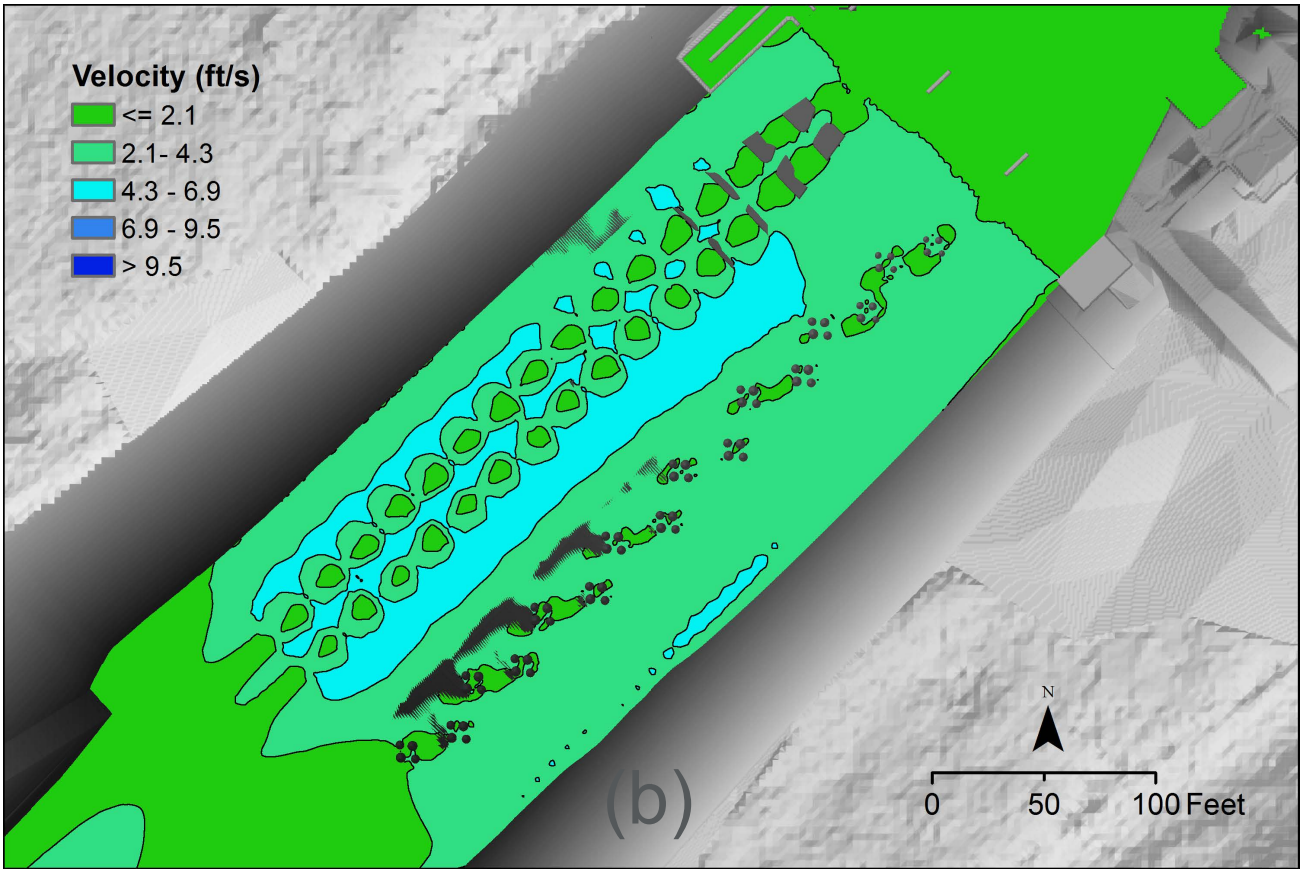


Panel (a) shows Katopodis and Gervais (2016) adult salmonid flow velocity categories where depth meets adult salmonid depth criteria (depth ≥ 0.9 ft); Panel (b) shows Katopodis and Gervais (2016) julvenile salmonid flow velocity categoeis where depth meets juvenile salmonid depth critiera (depth ≥ 0.4 ft)

EXHIBIT D6

HEC-RAS 2D MODEL RESULTS,
FISFISH PASSAGE DESIGN FLOW, HIGH FLOW - IRRIGATION SEASON (150.9 CFS)



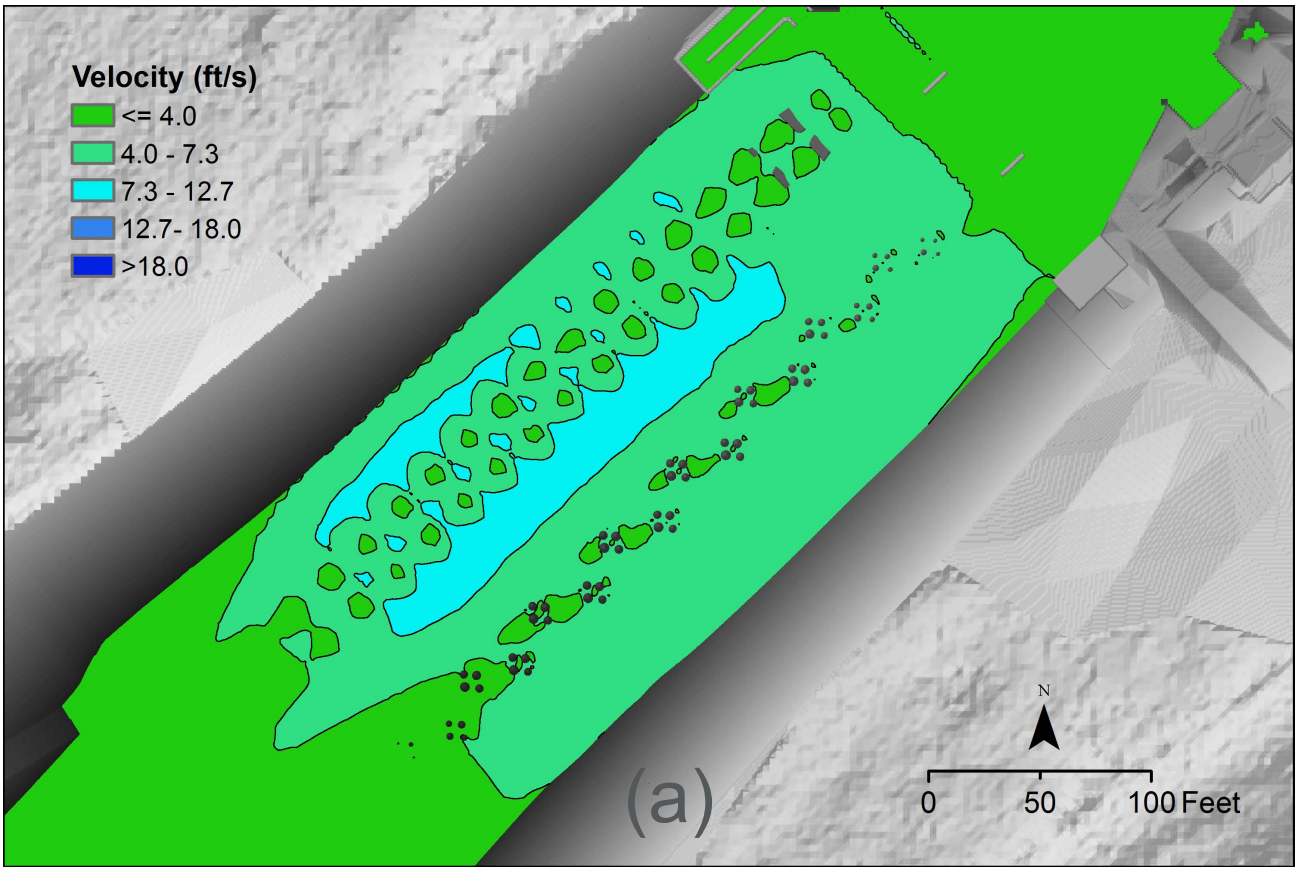


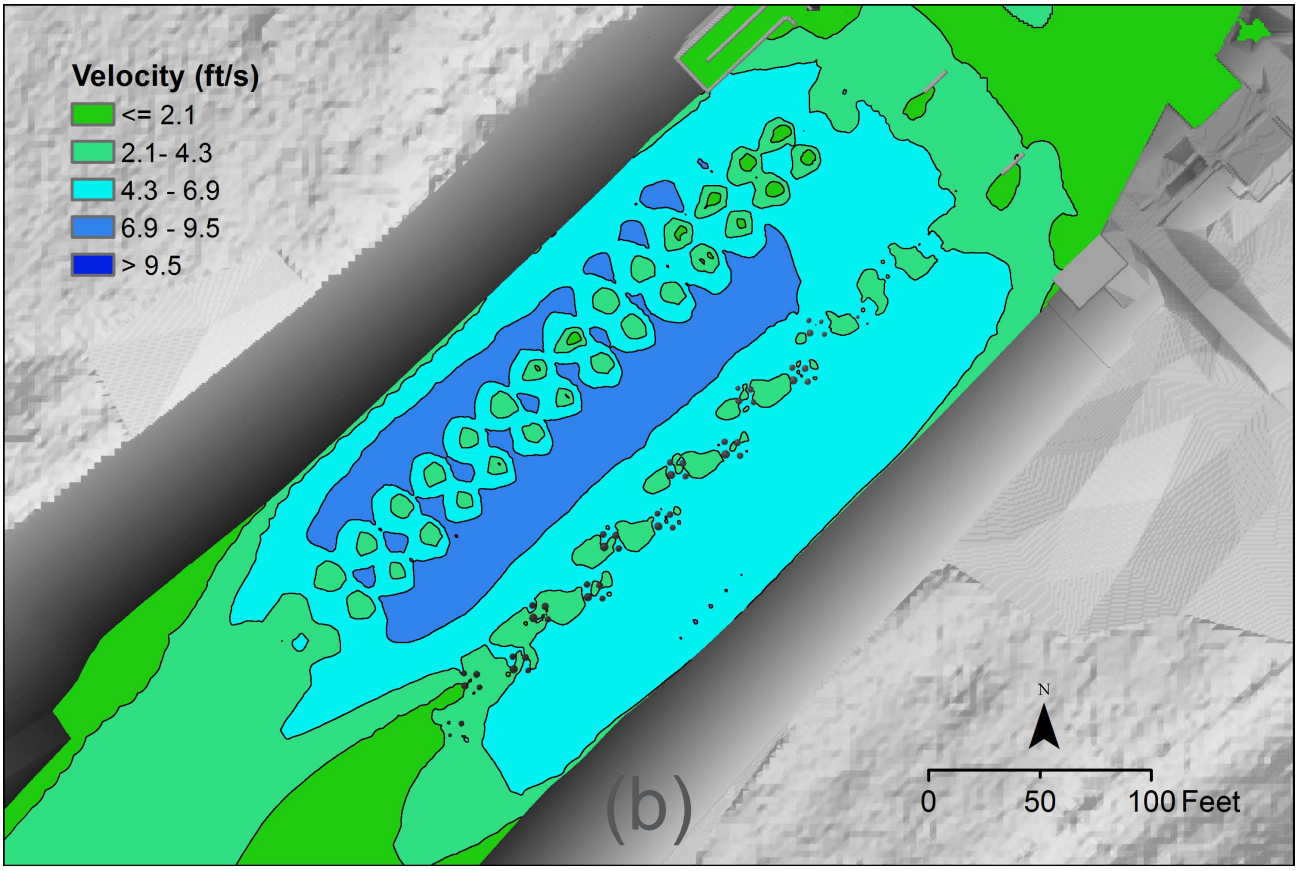
Panel (a) shows Katopodis and Gervais (2016) adult salmonid flow velocity categories where depth meets adult salmonid depth criteria (depth ≥ 0.9 ft); Panel (b) shows Katopodis and Gervais (2016) julvenile salmonid flow velocity categoeis where depth meets juvenile salmonid depth critiera (depth ≥ 0.4 ft)

EXHIBIT D7

HEC-RAS 2D MODEL RESULTS

, FISHFISH PASSAGE DESIGN FLOW, 5% ANNUAL EXCEEDANCE (545.3 CFS)





Panel (a) shows Katopodis and Gervais (2016) adult salmonid flow velocity categories where depth meets adult salmonid depth criteria (depth ≥ 0.9 ft); Panel (b) shows Katopodis and Gervais (2016) julvenile salmonid flow velocity categoeis where depth meets juvenile salmonid depth critiera (depth ≥ 0.4 ft)

EXHIBIT D8

HEC-RAS 2D MODEL RESULTS

, FISH FISH PASSAGE DESIGN FLOW, HIGH FLOW - MIGRATION SEASON (1,735.2 CFS)

Appendix E. Rock and ESM Sizing Calculations

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Roughened Ramp Slope Protection Calculations

EM 1110-2-1601: RIPRAP STONE SIZE CALCULATION

General Notes:

Calculations are based on EM 1110-2-1601 Equation 3-3

Calculations are for the Bellota Roughened Ramp bank riprap (Ramp Zones 3 and 4) design based on FEMA 100yr Discharge

SPREADSHEET LEGEND
HEADING
CONSTANT
USER FILL IN
AUTO-CALCULATES

$$D_{30} = S_f C_s C_V C_T d \left[\left(\frac{\gamma_w}{\gamma_s - \gamma_w} \right)^{1/2} \frac{V}{\sqrt{K_1 g d}} \right]^{2.5} \qquad (3\text{-}3)$$

where

D_{30} = riprap size of which 30 percent is finer by weight, length

S_f = safety factor (see *c* below)

* C_s = stability coefficient for incipient failure, D_{85}/D_{15} = 1.7 to 5.2

= 0.30 for angular rock

* = 0.375 for rounded rock

C_V = vertical velocity distribution coefficient

= 1.0 for straight channels, inside of bends

= 1.283 - 0.2 log (R/W), outside of bends (1 for (R/W) > 26)

= 1.25, downstream of concrete channels

= 1.25, ends of dikes

C_T = thickness coefficient (see d(1) below)

* = 1.0 for thickness = $1D_{100}$(max) or 1.5 D_{50}(max), whichever is greater

* d = local depth of flow, length (same location as *V*)

γ_w = unit weight of water, weight/volume

* V = local depth-averaged velocity, V_{SS} for side slope riprap, length/time

K_1 = side slope correction factor (see *d*(1) below)

g = gravitational constant, length/time^2

Safety Factor, *S_f*

-- Minimum SF is 1.1
-- SF may be increased for imposed impact forces resulting from logs, trees, loose vessels, ice, etc…
-- It is unlikely that an added SF should be used when blanket thickness exceeds 15"
-- SF may be increased to compensate for inaccuracies in velocity, unit weight of rock, and depth
-- SF should be increased where severe freeze-thaw is anticipated

Safety Factor, S_f =	1.25
Reasoning: SF of 1.1 is standard as recommended in EM 1110-2-1601	

Stability Coefficient for Incipient Failure, C_s

D_{85}/D_{15} > 1.7 for FHWA gradation	
Rock Shape: Angular	
Stability Coefficient C_s =	0.300

Vertical Velocity Distribution Coefficient, C_v

Channel Type: Downstream of Concrete Char	
Radius of Bend, Rc =	NA ft
Channel Width, W =	NA ft
Stability Coefficient C_v =	1.25

Blanket Thickness Coefficient, C_T

Thickness Coefficient C_T =	1.00	<-- Plate B-40, 1.00 is the most conservative C_T

Additional Inputs

Gravatational Constant, g =	32.20	ft/s^2
Unit weight of water, γ_w =	62.40	lb/ft^3
Unit weight of stone, γ_s =	165.00	lb/ft^3
Local depth of flow, d =	4.92	ft
Local depth averaged velocity, V =	11.67	ft/s
Side Slope (xH:1V) =	3.00	
Angle of Repose =	40.00	Degrees
Side slope correction factor, K_1 =	0.87	(Calculated by Eq. 3-4 in EM-1601)
Side slope correction factor, K_1 =	0.97	(Recommended for Design, Plate B-39)
Selected side slope correction factor, K_1 =	0.87	

Result

Computed D_{30} =	1.22 ft	
Computed D_{50} =	1.46 ft	(D_{50} = 1.2 D_{30} for FHWA gradation per NCHRP 568)
Computed D_{50} =	17.56 in	
Selected D_{50} =	18 in	(Table 4.1 from HEC-23 or Table 3.6 from NCHRP 568)

Blanket Thickness

1xD_{100}=	36.00	in
1.5xD_{50} =	27.00	in
Calculated Minimum Thicknesss =	36.00	in
Selected Minimum Thickness =	36.00	in

Gradation (Based on FHWA HEC-23 Table 4.1 in Design Guideline 4)

Nominal Riprap Class			D_{15}		D_{50}		D_{85}		D_{100}	
Class	Weight	Size	Min	Max	Min	Max	Min	Max	Max	
V	1/4 ton	18	11.0	15.5	17.0	20.5	23.5	27.5	36.0	(inch)

Roughened Ramp Foundation
Rock Calculations

Project:	**Bellota Weir Modifications**	Prepared:	KMF	Date: 12/22/2021
Subject:	**90% Design**	Checked:	SB	Date: 12/27/2021
Task:	**Fish Ramp Foundation Rock Design**			
Job#:				

Reference:

*Abt, S.R. and Johnson T.L. 1991. Riprap design for overtopping flow.

*U.S. Bureau of Reclamation (USBR), 2007. Rock Ramp Design Guidelines. Technical Service Center, Denver, CO. September.

*National Resource Conservation Service (NRCS). 2007. Stone Sizing Criteria. National Engineering Handbook, Park 654. Technical Supplement 14C.

Method 1: Abt and Johnson (1991)

EQUATION:

$$D_{50} = \phi_e \cdot \phi_c \cdot a \cdot 5.23 \cdot S_0^{0.43} \cdot q_{design}^{0.56}$$ **Equation 4-2**

INPUTS

ϕe =	1.2	coefficient for empirical envelope on the regression relationship
ϕc =	1.85	ϕc = rq 0.56 (USBR 2007, Eqn 4.3)
a =	1	angular material, a = 1; rounded material a = 1.40
r_q =	3	ratio of unit discharge in channel vs uniform depth; Max value = 3
Safety Factor	1.35	recommended 35% increase in design q (USBR 2007, Eqn. 4-1)
S_0	-0.03	channel invert slope

APPLICABILITY/ASSUMPTIONS

Embankment slopes:	1 - 20	%
Stone Sizes:	1 - 6	in
C_u (D60/D10):	1.62 - 2.15	
σ_g (D84/D16)$^{0.5}$:	1.86 - 5.70	
Unit Weight	167	pcf

DESIGN UNIT DISCHARGES, q, cfs/ft (MAX IN SUB-REGIONS)

Revised: 12/27/2021	**90% Proposed Conditions**			
Discharge	**USACE 10yr**	**USACE 25yr**	**FEMA 50yr**	**FEMA 100yr**
Ramp Location	**HEC-RAS 2D**	**HEC-RAS 2D**	**HEC-RAS 2D**	**HEC-RAS 2D**
Overbank	0	0	0	95
Main Channel	0	0	0	77
Main Channel & Overbank	0	0	0	82

DESIGN HYDRAULIC SLOPE (MAX IN CHANNEL)

Revised: 12/27/2021	**90% Proposed Conditions**			
Discharge	**USACE 10yr**	**USACE 25yr**	**FEMA 50yr**	**FEMA 100yr**
Ramp Location	**HEC-RAS 2D**	**HEC-RAS 2D**	**HEC-RAS 2D**	**HEC-RAS 2D**
Overbank	#DIV/0!	#DIV/0!	#DIV/0!	-0.002
Main Channel	#DIV/0!	#DIV/0!	#DIV/0!	-0.002
Main Channel & Overbank	#DIV/0!	#DIV/0!	#DIV/0!	-0.002

Note*: 65% design showed that 100-yr event produced most conservative rock sizing, thus only the 100-yr event was re-evaluated for 90% design

DESIGN ENERGY SLOPE (MAX IN CHANNEL)				
Revised: 12/27/2021	90% Proposed Conditions			
Discharge	USACE 10yr	USACE 25yr	FEMA 50yr	FEMA 100yr
Ramp Location	HEC-RAS 2D	HEC-RAS 2D	HEC-RAS 2D	HEC-RAS 2D
Overbank	#DIV/0!	#DIV/0!	#DIV/0!	-0.009
Main Channel	#DIV/0!	#DIV/0!	#DIV/0!	-0.009
Main Channel & Overbank	#DIV/0!	#DIV/0!	#DIV/0!	-0.009

***Note: Energy slope selected as design slope due to influence of backwater at the downstream end of the ramp; and energy slope produces more conservative rock sizing than hydraulic slope**

DESIGN D50, Equation 4-2 (USBR 2007); BASED ON HYDRAULIC SLOPE											
						See *Exhibit E.X* tab for gradation information					
Revised: 12/27/2021	90% Proposed Conditions				*Max of calculated D50*	*Min D50 based on NCHRP (2006) Table 3.6/3.7*	*Max D50 based on NCHRP (2006) Table 3.6/3.7*	*Max D100 based on NCHRP (2006) Table 3.6/3.7*	*Max of 1.5*D50, D100*	*Riprap Class based on NCHRP (2006) table 3.7*
Discharge	USACE 10yr	USACE 25yr	FEMA 50yr	FEMA 100yr	Calculated D50	MIN D50 (NCHRP 2006)	MAX D50 (NCHRP 2006)	MAX D100 (NCHRP 2006)	THICKNESS (NCHRP 2006)	WEIGHT (NCHRP 2006)
Ramp Location	HEC-RAS 2D	HEC-RAS 2D	HEC-RAS 2D	HEC-RAS 2D						
Overbank	#DIV/0!	#DIV/0!	#DIV/0!	0.95	0.95	0.71	0.88	1.50	1.50	60 lb
Main Channel	#DIV/0!	#DIV/0!	#DIV/0!	0.84	0.84	0.71	0.88	1.50	1.50	60 lb
Main Channel & Overbank	#DIV/0!	#DIV/0!	#DIV/0!	0.87	0.87	0.71	0.88	1.50	1.50	60 lb

DESIGN D50, Equation 4-2 (USBR 2007); BASED ON ENERGY SLOPE										
Revised: 12/27/2021	90% Proposed Conditions									
Discharge	USACE 10yr	USACE 25yr	FEMA 50yr	FEMA 100yr	Calculated D50	MIN D50 (NCHRP 2006)	MAX D50 (NCHRP 2006)	MAX D100 (NCHRP 2006)	THICKNESS (NCHRP 2006)	WEIGHT (NCHRP 2006)
Ramp Location	HEC-RAS 2D	HEC-RAS 2D	HEC-RAS 2D	HEC-RAS 2D						
Overbank	#DIV/0!	#DIV/0!	#DIV/0!	1.91	1.91	1.67	2.00	3.50	3.50	3/8 ton
Main Channel	#DIV/0!	#DIV/0!	#DIV/0!	1.69	1.69	1.67	2.00	3.50	3.50	3/8 ton
Main Channel & Overbank	#DIV/0!	#DIV/0!	#DIV/0!	1.76	1.76	1.67	2.00	3.50	3.50	3/8 ton

***Note: 6ft selected as conservative thickness to ensure adequate depth for embedment of weir**

DESIGN D50, Equation 4-2 (USBR 2007); BASED ON RAMP INVERT SLOPE										
Revised: 12/27/2021	90% Proposed Conditions									
Discharge	USACE 10yr	USACE 25yr	FEMA 50yr	FEMA 100yr	Calculated D50	MIN D50 (NCHRP 2006)	MAX D50 (NCHRP 2006)	MAX D100 (NCHRP 2006)	THICKNESS (NCHRP 2006)	WEIGHT (NCHRP 2006)
Ramp Location	HEC-RAS 2D	HEC-RAS 2D	HEC-RAS 2D	HEC-RAS 2D						
Overbank	0.00	0.00	0.00	3.24	3.24	2.83	3.46	6.00	6.00	2 ton
Main Channel	0.00	0.00	0.00	2.88	2.88	2.83	3.46	6.00	6.00	2 ton
Main Channel & Overbank	0.00	0.00	0.00	3.00	3.00	2.83	3.46	6.00	6.00	2 ton

Roughened Ramp Engineered Streambed Material Calculations

Project:	**Bellota Weir Modifications**	Prepared: KMF	Date:	20211215
Subject:	**90% Design**	Checked: SPB	Date:	12/27/2021
Task:	**Fish Ramp Engineered Streambed Material (ESM) Design**			
Job#:		No:		

FOR ENGINEERED STREAMBED MATERIAL SIZING

USACE. 1994. Hydraulic Design of Flood Control Channels (page 3-8)

CDFG. 2009. California Salmonid Stream Habitat Restoration Manual Part XII (page 67-70)

US Army Corps of Engineers Rock Sizing Equations

$D_{30\text{-ACOE}} = 1.95S^{0.555}(1.25q)^{0.667}/g^{0.333}$

Where:

Fill in yellow cells

$D_{30\text{-ACOE}}$ =			D_{30} stable particle size based on rock gradation provided in USACE (1994) (ft)
S (ft/ft)=	0.0087		S = Bed Slope (ft/ft) (EGL slope used due to presence of backwater/tailwater influence; See Foundation Rock Sizing Calculations for EG Slope data)
unit q (cfs/ft)=	95		q = unit discharge within the active channel at the stable bed design flow (cfs/ft) (Unit discharge is equal to the maximum 100-yr (12,960 cfs total) unit discharge in the roughened channel based on HEC-RAS 2D results)
g=	32.2		g = gravitational acceleration (32.2 ft/s^2)
n=	0.4	0.5	n should be between 0.45 and 0.70, and chosen such that the D8 is 2mm (0.0065')
$D_{30\text{-ACOE (ft)}}$ =	1.07		

CDFG (CFFW) ESM Ratios

	No Rock Structures	With Rock structures
$D_{84\ ESM\ (ft)}$ =	1.60	0.64
$D_{50\ ESM\ (ft)}$ =	0.64	0.26
$D_{100\ ESM\ (ft)}$ =	4.00	1.60
$D_{16\ ESM\ (ft)}$ =	0.037	0.026
$D_{8\ ESM\ (ft)}$ =	0.0066	0.0066

Bates

$$D_{84\text{-ESM}} = 1.5\ D_{30\text{-ACOE}}$$

$$D_{50\text{-ESM}} = 0.4\ D_{84\text{-ESM}}$$

$$D_{100\text{-ESM}} = 2.5\ D_{84\text{-ESM}}$$

Fuller-Thompson

$$D_{16-ESM} = 0.32^{\frac{1}{n}} D_{50-ESM}$$

$$D_{8-ESM} = 0.16^{\frac{1}{n}} D_{50-ESM}$$

Guidance for inclusion of large rock structures

Sizing of Rock Steps and other Large Rock Structures

For roughened channel steps, chutes or cascades, the larger rocks in the composite engineered streambed material, from $D_{84\text{-ESM}}$ to $D_{100\text{-ESM}}$ in size, are separated from the gradation and used to construct steps and other stable large rock features. The remainder of the material ($\leq D_{84\text{-ESM}}$) is used to form the bed between the steps. Although these large rock features are formed by material ranging in size ranging between the $D_{84\text{-ESM}}$ to $D_{100\text{-ESM}}$, the actual volume of material needed to construct these features should be calculated independently.

Engineered Streambed Material Mix

Calculated Mix					Material Mix (excluding D84 - D100)				
					From (ft)	To (ft)	Incremental %	Cumulative %	
D84 - D100	16%	1.60	to	4.00 (ft)					
D50 - D84	34%	0.64	to	1.60 (ft)	0.64	1.60	40.5%	100.0%	
D16 - D50	34%	0.037	to	0.64 (ft)	0.037	0.64	40.5%	59.5%	
D8 - D16	8%	0.0066	to	0.037 (ft)	0.0066	0.037	9.5%	19.0%	
to D8	8%	Fines			0.0000	0.0066	9.5%	9.5%	

D100 (ft)	D84 (ft)	D50 (ft)	D16 (ft)	D8 (ft)
1.60	1.22	0.50	0.027	0.002

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Summary - Stream Simulation Bed Material Design

Reference WSDOT. 2022. *Standard Specifications*. Division 9-03.11 Streambed Aggregates.

Project:	
By:	

Design Gradation:				
Location:	Streambed Design			
	D_{100}	D_{84}	D_{50}	D_{16}
ft	**1.60**	**1.22**	**0.50**	**0.03**
in	**19.2**	**14.6**	**6.0**	**0.3**
mm	**488**	**372**	**151.9**	**8.3**

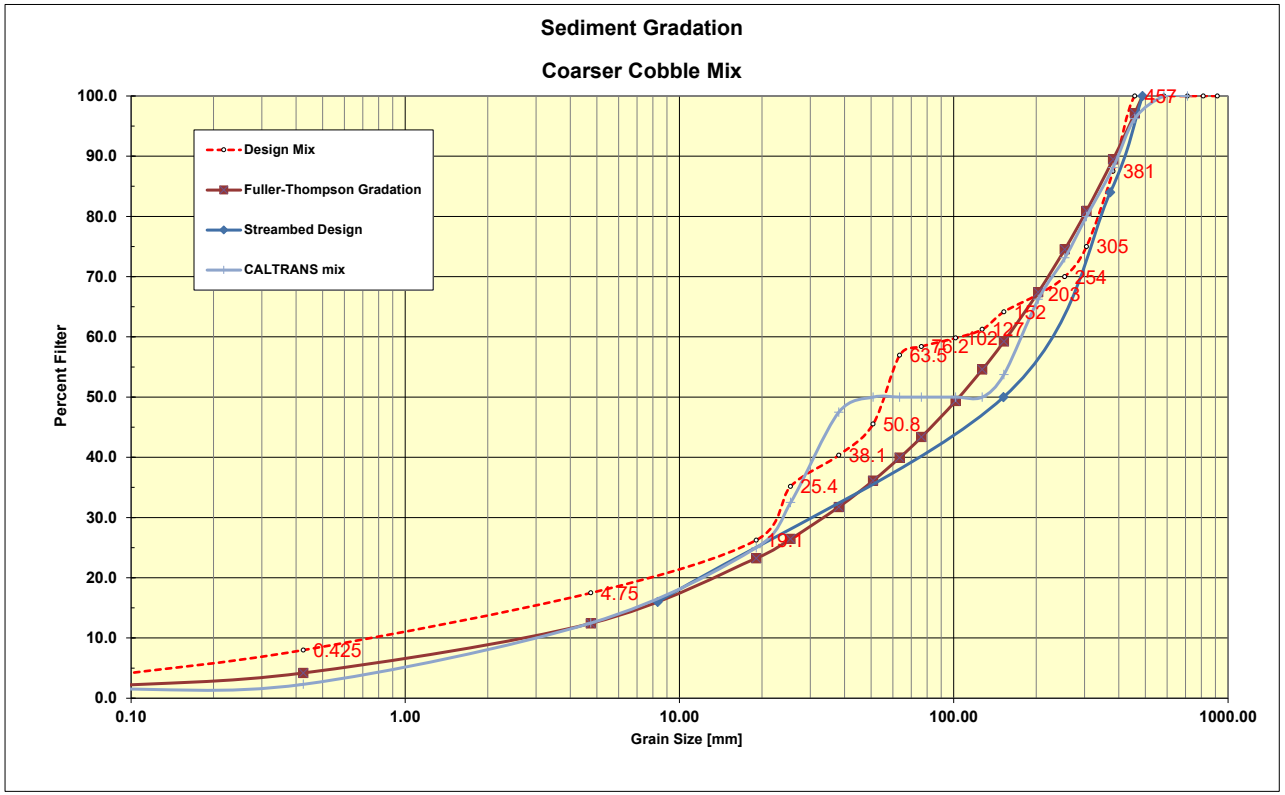
Design Gradation:				
Location:				
	D_{100}	D_{84}	D_{50}	D_{16}
ft				
in				
mm				

Determining Aggregate Proportions

Per WSDOT Standard Specifications 9-03.11

Rock Size		Streambed	Streambed Cobbles					Streambed Boulders			D_{size}
[in]	[mm]	Sediment	4"	6"	8"	10"	12"	12"-18"	18"-28"	28"-36"	
36.0	**914**									**100**	**100.0**
32.0	**813**	100	100	100	100	100	100	100	100	50	**100.0**
28.0	**711**	100							**100**		**100.0**
23.0	**584**	100							50		**100.0**
18.0	**457**	100						**100**			**100.0**
15.0	**381**	100						50			**87.5**
12.0	**305**	100					**100**				**75.0**
10.0	**254**	100				**100**	**80**				**70.0**
8.0	**203**	100			**100**	**80**	68				**67.1**
6.0	**152**	100		**100**	**80**	68	57				**64.2**
5.0	**127**	100		**80**	68	57	**45**				**61.3**
4.0	**102**	100	**100**	71	57	**45**	39				**59.8**
3.0	**76.2**	100	**80**	63	**45**	38	34				**58.4**
2.5	**63.5**	**100**	65	54	37	32	28				**57.0**
2.0	**50.8**	**80**	50	**45**	29	25	22				**45.5**
1.5	**38.1**	73	**35**	32	21	18	16				**40.4**
1.0	**25.4**	**65**	20	18	13	12	11				**35.2**
0.75	**19.1**	50	**5**	**5**	**5**	**5**	**5**				**26.3**
0.187	**4.75**	**35**									**17.5**
No. 40 =	**0.425**	**16**									**8.0**
No. 200 =	**0.0750**	**7**									**3.5**
% per category		**50**					**25**	**25**			*--> 100%*

	D95	D84	D50	D35	D16
Mix Size Interpolation	95	84	50	35	16
y1	87.5	75.0	45.5	26.3	8.0
y2	100.0	87.5	57.0	35.2	17.5
x1	381.0	304.8	50.8	19.1	0.4
x2	457.2	381.0	63.5	25.4	4.8
(mm)	**425**	**358**	**55**	**25**	**3**
(inches)	**16.7**	**14.1**	**2.2**	**1.0**	**0.1**
(feet)	**1.39**	**1.17**	**0.18**	**0.08**	**0.01**



Fuller-Thompson Gradation	
Dmax =	19.2
D[in]	
36.0	132.68
32.0	125.83
28.0	118.49
23.0	108.45
18.0	97.13
15.0	89.48
12.000	80.93
10.000	74.55
8.000	67.43
6.000	59.24
5.000	54.58
4.000	49.36
3.000	43.37
2.500	39.95
2.000	36.14
1.500	31.75
1.000	26.45
0.750	23.24
0.187	12.44
No. 40 =	4.20
No. 200 =	1.92

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Streambed Sediment (50% of Total ESM Mix)

Rock Size		Streambed Sediment	Streambed Cobbles					Streambed Boulders			D_{size}
[in]	[mm]		4"	6"	8"	10"	12"	12"-18"	18"-28"	28"-36"	
36.0	**914**									**100**	**100.0**
32.0	**813**	100	100	100	100	100	100	100	100	50	**100.0**
28.0	**711**	100							**100**		**100.0**
23.0	**584**	100							50		**100.0**
18.0	**457**	100						**100**			**100.0**
15.0	**381**	100						50			**100.0**
12.0	**305**	100					**100**				**100.0**
10.0	**254**	100				**100**	**80**				**100.0**
8.0	**203**	100			**100**	**80**	68				**100.0**
6.0	**152**	100		**100**	**80**	68	57				**100.0**
5.0	**127**	100		**80**	68	57	**45**				**100.0**
4.0	**102**	100	**100**	71	57	**45**	39				**100.0**
3.0	**76.2**	100	**80**	63	**45**	38	34				**100.0**
2.5	**63.5**	**100**	65	54	37	32	28				**100.0**
2.0	**50.8**	**80**	50	**45**	29	25	22				**80.0**
1.5	**38.1**	73	**35**	32	21	18	16				**72.5**
1.0	**25.4**	**65**	20	18	13	12	11				**65.0**
0.75	**19.1**	50	**5**	**5**	**5**	**5**	**5**				**50.0**
0.187 (No 4)	**4.75**	**35**									**35.0**
No. 40 =	**0.425**	**16**									**16.0**
No. 200 =	**0.0750**	**7**									**7.0**
Percent Per Category		**100**									

Sediment Mix Size Interpolation

	D95	D84	D50	D35	D16
% cum.	95	84	50	35	16
y1	80.0	80.0	50.0	35.0	16.0
y2	100.0	100.0	50.0	35.0	16.0
x1	50.8	50.8	19.1	4.8	0.4
x2	63.5	63.5	19.1	4.8	0.4
(mm)	**60**	**53**	**19**	**5**	**0.43**
(inches)	**2.4**	**2.1**	**0.8**	**0.2**	**0.017**
(feet)	**0.20**	**0.17**	**0.06**	**0.02**	**0.0014**

Streambed Sediment Size Lookup Tables

% cum.	Diameter (mm)	% cum.	Diameter (mm)
100.0	63.5	0.0	0.000
80.0	50.8	7.0	0.075
72.5	38.1	16.0	0.425
65.0	25.4	35.0	4.75
50.0	19.1	50.0	19.1
35.0	4.75	65.0	25.4
16.0	0.425	72.5	38.1
7.0	0.075	80.0	50.8
0.0	0.000	100.0	63.5

Streambed Cobbles (50% of Total ESM Mix)											
Rock Size		Streambed Cobbles					Streambed Boulders			D_{size}	$D_{size,}$ ESM 50% sed, 50% cobble
[in]	[mm]	4"	6"	8"	10"	12"	12"-18"	18"-28"	28"-36"		
36.0	914	100	100	100	100	100	100	100	**100**	100.0	50.0
32.0	813	100	100	100	100	100	100	100	**50**	100.0	50.0
28.0	711	100	100	100	100	100	100	**100**		100.0	50.0
23.0	584	100	100	100	100	100	100	**50**		100.0	50.0
18.0	457	100	100	100	100	100	**100**			100.0	100.0
15.0	381	100	100	100	100	100	**50**			75.0	87.5
12.0	305	100	100	100	100	**100**				50.0	75.0
10.0	254	100	100	100	**100**	**80**				40.0	70.0
8.0	203	100	100	**100**	**80**	**68**				34.2	67.1
6.0	152	100	**100**	**80**	**68**	**57**				28.3	64.2
5.0	127	100	**80**	**68**	**57**	**45**				22.5	61.3
4.0	102	**100**	**71**	**57**	**45**	**39**				19.6	59.8
3.0	76.2	**80**	**63**	**45**	**38**	**34**				16.8	58.4
2.5	63.5	**65**	**54**	**37**	**32**	**28**				13.9	57.0
2.0	50.8	**50**	**45**	**29**	**25**	**22**				11.1	45.5
1.5	38.1	**35**	**32**	**21**	**18**	**16**				8.2	40.4
1.0	25.4	**20**	**18**	**13**	**12**	**11**				5.4	35.2
0.75	19.1	**5**	**5**	**5**	**5**	**5**				2.5	26.3
0.187 (No 4)	4.75									0.0	17.5
No. 40 =	0.425									0.0	8.0
No. 200 =	0.0750									0.0	3.5
Percent Per Category						50	50				

Cobble Mix Size Interpolation

	D95	D84	D50	D35	D16	D10
% cum.	95	84	50	35	16	10
y1	75.0	75.0	50.0	34.2	13.9	8.2
y2	100.0	100.0	50.0	40.0	16.8	11.1
x1	381.0	381.0	304.8	203.2	63.5	38.1
x2	457.2	457.2	304.8	254.0	76.2	50.8
(mm)	**441**	**407**	**305**	**210**	**72**	**46**
(inches)	**17.4**	**16.0**	**12.0**	**8.3**	**2.9**	**1.8**
(feet)	**1.45**	**1.33**	**1.00**	**0.69**	**0.24**	**0.15**

Cobble Size Lookup Tables

% cum.	Diameter (mm)	% cum.	Diameter (mm)
100.0	457	0.0	0
75.0	381	0.0	0
50.0	305	0.0	5
40.0	254	2.5	19
34.2	203	5.4	25
28.3	152	8.2	38
22.5	127	11.1	51
19.6	102	13.9	64
16.8	76	16.8	76
13.9	64	19.6	102
11.1	51	22.5	127
8.2	38	28.3	152
5.4	25	34.2	203
2.5	19	40.0	254
0.0	5	50.0	305
0.0	0	75.0	381
0.0	0	100.0	457



Draft (90%) Design Documentation Report

Bellota Weir Modifications Project

Mormon Slough, Calaveras River, CA

February 7, 2022

Prepared for

STOCKTON EAST WATER DISTRICT

PROVIDING SERVICE SINCE 1948

Prepared by HDR Inc & KSN, Inc.

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Appendices

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Appendix F. Mechanical Calculations

Appendix G. Structural Calculations

Appendix H. Electrical Calculations

Abbreviations

1D	one-dimensional
2D	two-dimensional
ACI	American Concrete Institute
ASCE	American Society of Civil Engineers
ATS	automatic transfer switch
CCRTC	Central California Railroad Traction Crossing
CDFG	California Department of Fish and Game
CDFW	California Department of Fish and Wildlife
CDWR	California Department of Water Resources
CESA	California Endangered Species Act
cfs	cubic foot/feet per second
CHCP	Calaveras River Habitat Conservation Plan
DDR	Design Documentation Report
District	Stockton East Water District
EL	elevation
EM	Engineer Manual
ER	Engineering Regulation
ESA	Endangered Species Act
FEMA	Federal Emergency Management Agency
FESA	Federal Endangered Species Act
fps	foot/feet per second
HDR	HDR Engineering, Inc.
H:V	horizontal:vertical
ITP	Incidental Take Permit
kA	kiloamperes
lb(s)	pound(s)
M	million
MCC	motor control center
NAVD88	North American Vertical Datum of 1988
NCHRP	National Cooperative Highway Research Program
NEC	National Electric Code
NEMA	National Electrical Manufacturers Association
NFPA	National Fire Protection Association
NMFS	National Marine Fisheries Service
NOAA	National Oceanic and Atmospheric Administration
O&M	operation and maintenance
OIT	operator interface terminal
OPCC	opinion of probable construction costs
pcf	pound(s) per cubic foot

PG&E	Pacific Gas & Electric
PLC	programmable logic controller
Project	Bellota Weir Modifications Project
psf	pound(s) per square foot
psi	pound(s) per square inch
PVC	polyvinyl chloride
RCP	reinforced concrete pipe
Reclamation	U.S. Bureau of Reclamation
RSP	rock slope protection
SCADA	supervisory control and data acquisition
SEWD	Stockton East Water District
USACE	U.S. Army Corps of Engineers
USGS	U.S. Geological Survey
V	volt
VFD	variable frequency drive
WSE	water surface elevation
WTP	water treatment plant

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Executive Summary

The Bellota Weir Modifications Project (Project) will provide a more effective volitional fish passage route at the existing Bellota Weir location in Mormon Slough, exclude fish from the Old Calaveras River, improve sediment continuity in Mormon Slough, and provide a more reliable surface water intake for the Stockton East Water District (District).

Specifically, the Project proposes removal of the existing weir and replacement with a new weir and roughened channel; consolidation of diversion withdrawals; and construction of several new Project elements. The proposed Project incorporates numerous integrated, multi-disciplinary improvements to achieve these goals.

Construction Sequencing

The Project construction is proposed to occur in five sequential phases to minimize environmental impacts and optimize constructability, cost effectiveness, and operational continuity. The current sequencing strategy and level of effort is based on a minimum 2-year construction period; however, project elements may be constructed in a different order or over a longer period of time to accommodate costs or permitting restrictions.

A summary is provided in this section; more detail regarding the potential construction sequencing concept can be found in Section 11.

Sequence 1A (Non-Irrigation season, out of water work)

- Complete construction of access roads
- Remove trees

Sequence 1B-1D (Irrigation season, in-water work window)

Construction of sequences 1B-1D will take place during the irrigation season (mid-April to mid-October), during the in-water work window of mid-June to early October. Stream diversion will be provided using the proposed 54-inch sluiceway on the north bank and the existing 36-inch bypass on the south bank. Temporary fish passage will not be needed during the in-water work window because steelhead and fall run Chinook salmon (known to opportunistically utilize the river) are not present and there is no adult migration or juvenile outmigration occurring during this time (CHCP 2019). This is the same case for winter, spring, and late-fall run Chinook, though each of these is considered extirpated from the area and not likely to use the Calaveras River.

Sequence 1B

- Complete demolition of existing weir
- Complete removal and salvage of existing riprap
- Install sheet pile walls
- Install 54-inch sluiceway on north bank and extend 36-inch bypass pipeline on south bank

Sequence 1C

- Complete in-water grading in Mormon Slough
- Partial construction of Bellota conveyance pipelines (under concrete weir structure)

- Complete construction of weir slab
- Complete pile installation for the weir and debris boom
- Complete construction of the roughened channel

Sequence 1D

- Complete Installation of debris boom components
- Remove temporary access road to roughened channel
- Install sheet pile wall S-3A and remove sheet pile wall S2
- Complete installation of riverbank armoring
- Remove in-river dewatering systems

Sequence 2 (Non-irrigation season, out-of-water work)

- Complete construction of the concrete fish ladder structure
- Complete the downstream section of the intake used for sediment sluicing and install sluice gates. Connect 54-inch sluiceway pipeline to structure
- Complete construction of the diversion structure
- Complete construction of the control building

Sequence 3 (Irrigation season, in-water work window)

- Complete construction of the intake structure
- Complete construction of weir superstructure and installation of Obermeyer weirs

Sequence 4 (Non-irrigation season, out-of-water work)

- Complete construction of the Old Calaveras River conveyance pipelines
- Complete construction of the Bellota conveyance pipelines
- Complete remaining site work

Sequence 5 (Irrigation season, in-water work window)

- Complete in-water grading in the Old Calaveras River

Summary of Proposed Project Elements and Performance

Removal of Existing Infrastructure: Prior to implementing the proposed elements, several existing features that impair the construction or intended function of the intake or in-water work at the Project site will be demolished and removed. Overhead power and communication utility relocations will also be performed during these demolition activities. Existing facilities to be decommissioned and removed include:

- Existing weir including sheet piling
- Existing buildings on site

The existing intake will remain operable in the event of an emergency that renders the proposed facilities inoperable, per the District's request.

Roughened Channel: The roughened-channel fishway will be designed to conform to several performance and engineering criteria, specifically targeting the overall stability and resiliency of the

structure, ecohydraulic performance, and targeted periodic costs associated with maximum acceptable damages that require modification or repairs over time.

The primary roughened-channel fishway will slope downstream at 3 percent for about 400 feet until it catches the existing channel grade downstream. Additional scour and structural countermeasures are incorporated immediately below the roughened channel and weir crest to accommodate anticipated general channel scour and structure scour that may occur at both transitions. The channel geometry varies in cross-section with a primary flow channel extending downstream and an overbank area along the southern bank.

Proposed conditions provide continuous navigable hydraulic pathways throughout the Project area. These hydraulic pathways present opportunities for sub-adult and adult fish to navigate upstream through the Project reach. The different navigational pathways and migration opportunities vary across the range of anticipated fish passage flows.

Weir: The proposed weir will consist of a concrete structure anchored to a sheet pile cutoff wall with a weir-gate span of 150 feet, integrating three Obermeyer weir gates designed to modulate forebay pool elevations based on operational needs. The weir will be anchored by a series of pipe piles for seismic considerations.

The concrete structure and weir gate height will be designed to provide a minimum operating pool elevation of 121.44 feet North American Vertical Datum of 1988 (NAVD88) during the irrigation season and lie flat when needed to reduce the pool elevation to 115.44 feet NAVD88. The minimum operating pool elevation of 121.44 feet NAVD88 during the irrigation season is anticipated to supply the District with 70 cfs at present and 200 cfs in the future; the minimum required water surface to supply 60 cfs will be115.44 feet NAVD88.

Consolidated Surface Water Intake: The surface water intake will be designed to promote reliable water supply while meeting the fish protection guidelines provided by California Department of Fish and Wildlife (CDFW) and National Marine Fisheries Service (NMFS). Fish screening will be accomplished by integrating four cylindrical tee screen systems mounted on retractable rails along the southern wall intake structure within the forebay. There are two tee-screen sizes in the intake structure. The smaller tee screens will be composed of two, 48-inch-diameter, 6-foot-long cylinders wrapped with type 316 stainless steel wedge wire with 0.069-inch (1.75-mm) slots each with 50-cfs capacity. The larger tee screens will be composed of two, 83-inch-diameter,10-foot-long cylinders with type 316 stainless steel wedge wire with 0.069-inch (1.75-mm) slots each with 144-cfs capacity. The two cylinders are mounted in-line on a single manifold with a total fabricated length of approximately 18 feet, 3 inches for the smaller tee screens and approximately 27 feet, 7 inches for the larger tee screens. In the down position, the cylinders are typically submerged below the water surface with a minimum submergence of one radius (24 inches for the smaller tee screen and 41.5 inches for the larger tee screen). During diversionary periods, water passes through the screens and into the intake manifold through a rectangular opening in the wall behind each screen. A stop gate will be provided behind each screen to prevent flow passing through to the intake manifold when a screen is not in use. When not in operation, each cylinder can also be retracted up into a maintenance position at the level of the intake wall.

Sediment Sluicing: Sediment sluicing will be incorporated at three different locations in the Project, with each location designed to discharge to the same 54-inch-diameter, 550-foot-long concrete sluiceway that discharges the consolidated sediment slurries to Mormon Slough, downstream of the roughened channel. The sediment sluicing locations are the forebay (Bellota pool immediately upstream of weir), the screenings channel, and the distribution structure. Sluicing of the forebay will

be initiated by opening a 60-inch by 60-inch sluice gate at the west end of the intake structure that discharges to the screen channel exit as well as opening a 54-inch-diameter gate within the intake structure that discharges directly to the sluice pipeline. The screen channel will be sloped to a low point at the west end of the intake structure (in-line with the 54-inch-diameter sluicegate) for sediment accumulation and discharge. Finally, the distribution structure will be equipped with a 12-inch-diameter gate for discharge to a 12-inch sluice pipeline that will be connected to the 54-inch concrete sluice pipeline.

Fish Ladder: The fish ladder, together with the roughened channel, will be used for fish passage during the irrigation season generally April to October, when the water surface level of the Bellota Pool has been raised to an elevation of 121.44 feet. The ladder consists of seven vertical slot pools and two pools and weirs. Each pool will be 8 feet wide and 10 feet long. An auxiliary water supply will not be needed because the ladder flow already meets the target attraction flow criteria defined by the National Marine Fisheries Service. Lamprey ramps are incorporated at each of the two pools and weirs, and each of the vertical slots are rounded to facilitate lamprey passage.

Anticipated Project Costs

The total construction cost is anticipated to be $68 million (M) which includes a 10 percent contingency, 5 percent escalation per year for 5 years, 4 percent markup for construction phasing complexity, and 3 percent adjustment factor for market volatility. The anticipated base cost for Project construction is estimated to be $43.9M and could vary from as low as -10 percent to as high as +20 percent based on the current level of detail and cost certainty. Major cost components include the intake/fishway ($8.9M), weir structure ($4.4M), roughened-channel ($5.7M), and conveyance structures ($3.0M). The larger cost items for the intake/fishway include the sheet pile walls ($1.7M) and the tee screens ($3.0M). The larger cost items for the roughened-channel are the foundation rock ($2.0M) and riprap for bank stabilization ($1M).

Revisions to this Document

This report iteration was developed commensurate with the 90 percent level complete design drawings, specifications, and opinions of probable construction cost. The information provided within is subject to change as updated drafts of this report will be provided at major design milestones.

1 Introduction

The Stockton East Water District (SEWD or District) retained the professional services of KSN, Inc. and HDR Engineering, Inc. (HDR) to advance the Bellota Weir Modifications Project (Project). The Bellota Weir is located immediately downstream of the Calaveras River bifurcation between the Mormon Slough and Old Calaveras River, about 17 miles downstream of the New Hogan Dam.

The Project includes weir and intake improvements that will provide more reliable water delivery systems for SEWD customers, improve migration of anadromous fish, and reduce fish entrainment at the facilities, thereby enhancing anadromous salmonid populations present in the Calaveras River. Initial efforts for this Project include alternative refinement and conceptual design activities in collaboration with SEWD, resource agencies, regulatory authorities, and other Project partners. This document summarizes the key background information and process used to identify potential viable options and select a conceptual design that balances the opportunities, constraints, and objectives of this Project.

1.1 Purpose of Design Development Report

This Design Documentation Report (DDR) documents the design basis for each Project element, including record of design decisions, assumptions, and methods related to the development of the design of the Project. The DDR also presents the technical details of the main features of the proposed Project elements.

1.2 Scope of Document

This DDR summarizes important aspects of the design process including:

- Background data and assumptions used in the design process
- Design criteria
- Team coordination, collaboration, and decision-making processes
- Geotechnical investigation
- Hydrologic and hydraulic analyses
- Functional descriptions of each proposed Project element
- Anticipated operational and construction strategy
- Development of the opinion of probable construction cost (OPCC)
- Pertinent design calculations

This report will be developed to a point of completion alongside final design drawings (drawings), specifications, and OPCC. Updated drafts of this report will be provided at the major design milestones beginning with the 65 percent design submittal. A summary of the document contents following this section is provided below.

- **Section 2** provides an overview of the Project, including detailed descriptions of the major proposed Project elements.

- **Section 3** presents the critical standards and criteria used to develop the design. This section does not go into detail on the design criteria.
- **Sections 4 through 9** provide descriptions of design components and processes considered in the 90 percent design.
- **Section 0** describes the Project's operation and maintenance (O&M) strategy.
- **Section 11** describes the proposed construction schedule and sequencing for the Project.
- **Section 12** outlines the anticipated O&M and construction costs of the Project.
- **Section 13** provides references relied upon for the information and analysis presented in this report.

1.3 General Project Description

1.3.1 Background

The existing Bellota Weir facility on the Mormon Slough/Calaveras River is owned and operated by SEWD and provides water to urban and agricultural users. The Bellota Intake provides municipal and industrial flow year-round for the City of Stockton, and also supplies irrigation water during the irrigation season (generally April to October). The Old Calaveras Headworks (Calaveras Headworks) provides flow control and prevents flooding to downstream landowners on the Old Calaveras River by routing flood waters down the Mormon Slough. During most flow regimes, both the Bellota Weir and Calaveras Headworks are complete fish barriers to upstream migrating adult salmonids.

Continued operation of the Bellota Weir and diversion will be guided by the final Calaveras River Habitat Conservation Plan (CHCP; National Oceanic and Atmospheric Administration [NOAA] 2020). The CHCP provides operational criteria to support the biological goals of maintaining a viable population of threatened California Central Valley Steelhead, *Oncorhynchus mykiss,* within the CHCP boundaries, and maintains adequate habitat conditions upstream of Bellota for fall-, late fall-, spring- or winter-run Chinook Salmon, *Oncorhynchus tshawytscha*, that may opportunistically migrate into the conservation area. While the CHCP intends to provide conditions that support Chinook Salmon should they migrate into the conservation area, these salmon are not expected to maintain a viable population based both on pre-dam and current conditions. The CHCP enables SEWD to comply with the Endangered Species Act (ESA), protecting and managing fishery resources and habitat while maintaining reliable water delivery to its constituents. Following National Marine Fisheries Service (NMFS) approval on August 11, 2020, the District is authorized for a 50-year Incidental Take Permit (ITP #23264), for ESA-listed species under NMFS authority.

Upgrade or replacement of the Bellota Diversion, Weir, and Calaveras Headworks is a required compliance measure specified in the CHCP, as part of a seven-target fish passage objective. Specifically, the CHCP's target states:

> FP1 and AE1. Avoid migration delays and blockage, and entrainment within the Old Calaveras River Channel by constructing a non-entraining barrier at the Old Calaveras River Headworks Facility and at the downstream end of the channel

> near the confluence with the [Stockton Diverting Canal] within the first ten years of the ITP.
>
> FP2/AE3. Construct and implement a combined crest gate/fishway/fish screen at the Bellota Diversion Facility to improve [salmonid] passage into/out of the 18-mile spawning and rearing reach between Bellota and New Hogan Dam and to prevent fish entrainment; target completion within first five years, but no later than 10 years of [issuance of] the ITP.

Based on the CHCP targets summarized above and SEWD's water supply needs, the three primary Project objectives are to:

- **Improve fish passage** by designing and constructing a new crest gate dam, fishway, and fish screens that include passage for California Central Valley Steelhead and opportunistic migration for fall-, late fall-, spring-, or winter-run Chinook Salmon.
- **Reduce fish entrainment** by constructing a new non-entraining fish barrier at the Old Calaveras River.
- **Provide more reliable water delivery** through weir and intake improvements.

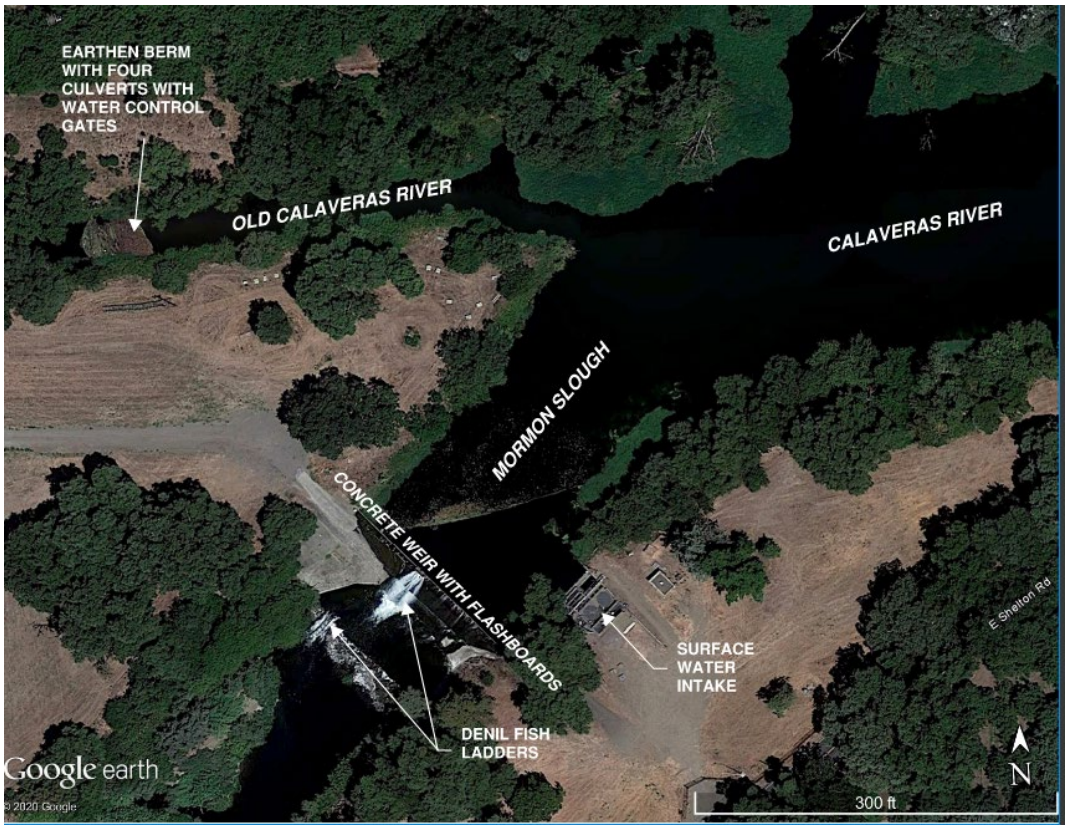
1.3.2 Existing Facility

The following sections provide descriptions of existing Project elements, including the existing Bellota Weir, Bellota Intake, and Calaveras Headworks.

Bellota Weir

Bellota Weir is the largest check dam on the Calaveras River system. The weir is located on the Mormon Slough approximately 350 feet downstream of its confluence with the Old Calaveras River. The weir is an in-water concrete structure designed to accommodate installation of a flashboard dam; during irrigation season, the check dam crest with the flashboards installed is 8 feet above the channel invert (invert elevation of 121.44 feet North American Vertical Datum of 1988 [NAVD88]). During non-irrigation seasonal (flood season) periods of the year, shorter flashboards are installed 2 feet above the channel invert. Existing features of the Bellota Weir Modifications Project are identified in Figure 1-1.

Figure 1-1. Prominent Features of the Bellota Weir Modifications Project Site



Bellota Weir includes two Denil fish ladders, installed in 1999 and 2001. During the irrigation season, only the downstream Denil fish ladder is in place, while during non-irrigation times, both Denil fish ladder sections are in place. In 2007, employees from the California Department of Fish and Wildlife (CDFW) and Fishery Foundation evaluated conditions at the ladders. The upper ladder is longer than the maximum recommended ladder length of 30 feet. The lower ladder has a 1 horizontal to 3 vertical (1H:3V) slope ratio, double the maximum recommended slope of 1H:6V. Finally, water does not flow through the lower ladder as intended but instead around the sides resulting in an inadequate water supply to the ladder that confuses fish looking for the ladder entrance (California Department of Water Resources [CDWR] 2007).

Bellota Intake

The Bellota Intake is a gravity diversion constructed in 1978, on the south riverbank about 50 feet upstream of the Bellota Weir. Water diverted through the intake is conveyed to the City of Stockton and the Dr. Joe Waidhofer Water Treatment Plant (WTP; SEWD, HDR, and KSN 2020). The structure possesses bulkhead slots for installation of stoplogs, a trashrack facility, two 14-foot-wide rectangular intake channels perpendicular to the river, two intakes with fish screens, a bypass pipeline, a sediment trap, and two slide gates to regulate flow.

Calaveras Headworks

The Calaveras Headworks is located approximately 600 feet downstream from the Old Calaveras River, Mormon Slough confluence (Figure 1-1). The existing structure includes an earthen berm, four culverts with control gates, a trashrack, and a concrete slope lining.

The earthen berm crest is 68 feet long and 10 feet wide. The 4-foot-wide, concrete culverts penetrate the berm and control flow to the Old Calaveras River. There is a 21-foot-wide trashrack on the upstream side of the berm protecting the culvert inlets, their concrete headwall, and wing walls. Flow is controlled with slide gates on the upstream side – two are operated manually and two are electrically actuated. The slide gates are operated between April and November to deliver water to downstream agricultural users, intermittently between November and April for groundwater recharge on the Old Calaveras River, and open during irrigation season to provide flow to downstream diverters (SEWD and FISHBIO 2019).

The Calaveras Headworks is a barrier to upstream adult fish migration and does not comply with juvenile downstream passage or fish protection guidelines provided by CDFW and NMFS.

In 2003, SEWD placed a small mesh net across the Old Calaveras River upstream of the Calaveras Headworks structure (CH2M 2003) to encourage juvenile fish to reside in the Bellota Pool and migrate upstream to suitable habitat or downstream to the Mormon Slough.

1.3.3 Geotechnical Conditions

Existing site information includes U.S. Geological Survey (USGS) – Geology and previously conducted drilling at the site by CH2M conducted between October 2004 and January 2005. Five borings were visually observed and logged during the previous exploration program along the southeast bank of the Mormon Slough at the previous proposed fish ladder location. The investigation indicates that the Project site is generally underlain by riverine alluvial deposits to the depths explored (CH2M 2005). The geologic map indicates presence of Holocene alluvium channel and levee deposits and Upper Modesto formation (Marchand and Bartow 1979). The geotechnical conditions identified during this study show a risk of soil liquefaction at the site.

Further geotechnical investigations have been initiated as part of the Bellota Weir Modifications Project including advancing five mud-rotary borings along the northwest bank to characterize subsurface conditions in the revised location of the new fish ladder. Geotechnical laboratory testing for soil classification, plasticity, and strength will be performed. A geotechnical report documenting the investigations performed will be prepared for use during design. See further explanation of geotechnical investigation in Section 6.

Review of available aerial photographs and topographic maps as part of the current study indicate that the Bellota Diversion was first created in the late 1940s by deepening the Mormon Slough and using the material to create a gravel weir at its current location, as indicated on both the 1953 and 1972 USGS 7.5-minute quadrangle topographic maps. Review of available as-built drawings indicate 1) the current weir was constructed

in about 1968, 2) the current weir was constructed along the original weir alignment, and 3) the original weir had been severely eroded (scoured) by stream flows. The amount of instream grading performed as part of the new weir construction is not documented in the drawings. The current weir is likely underlain by 3 to 5 feet of mounded river granular sediments. Preliminary review of the borings indicates the base of the coarse gravels that underlie the site is between elevation 95 to 100 feet NAVD88.

1.3.4 Debris Management and Sediment Characterization

After periods of high flows, sediment and debris removal efforts are occasionally necessary at the Bellota Intake. Given its orientation along the channel bank and presence of the existing profile control structure, slow water velocities cause fallout of fine sediments and accumulation of buoyant and semi-buoyant debris at the intake debris racks. Debris and sediment observed at the weir and intake are characterized as sands, silts, leaves, brush, trees, and floating trash. SEWD possesses a CDFW-issued permit for gravel removal in front of the intake structure (CH2M 2003).

There are often leaves and sticks around the existing fish screens. Larger debris travels toward the weir, unless water is being pulled to the intake. Existing intake screen cleaning is inadequate – brushes push off debris but leaves and sediment return. Sediment is cleaned from the intake approximately every two years with a routine maintenance agreement (SEWD, HDR, and KSN 2020).

1.3.5 Proposed Facility

The proposed Project facility combines several operational and engineered elements to achieve its intended objectives: intake and weir replacement and improvements for effective fish passage, reduced fish entrainment, and water supply reliability. The proposed Project elements are briefly summarized below and illustrated in Figure 1-2:

- Construct a concrete weir with a series of weir gates designed to modulate forebay pool elevations based on operational needs.
- Improve sediment conveyance by including sluicing functions in the forebay, intake manifold, and screen channel.
- Provide reliable fish passage using a roughened channel combined with a fishway utilizing both vertical slot, pool, and weir techniques to accommodate the full range of flows and river stages at the site.
- Provide reliable fish screening, both for the Bellota Intake and the Old Calaveras River, to prevent fish entrainment.
- Provide a non-entraining fish barrier to the Old Calaveras River using an earthen embankment.

Figure 1-2. Overview of Proposed Project Elements

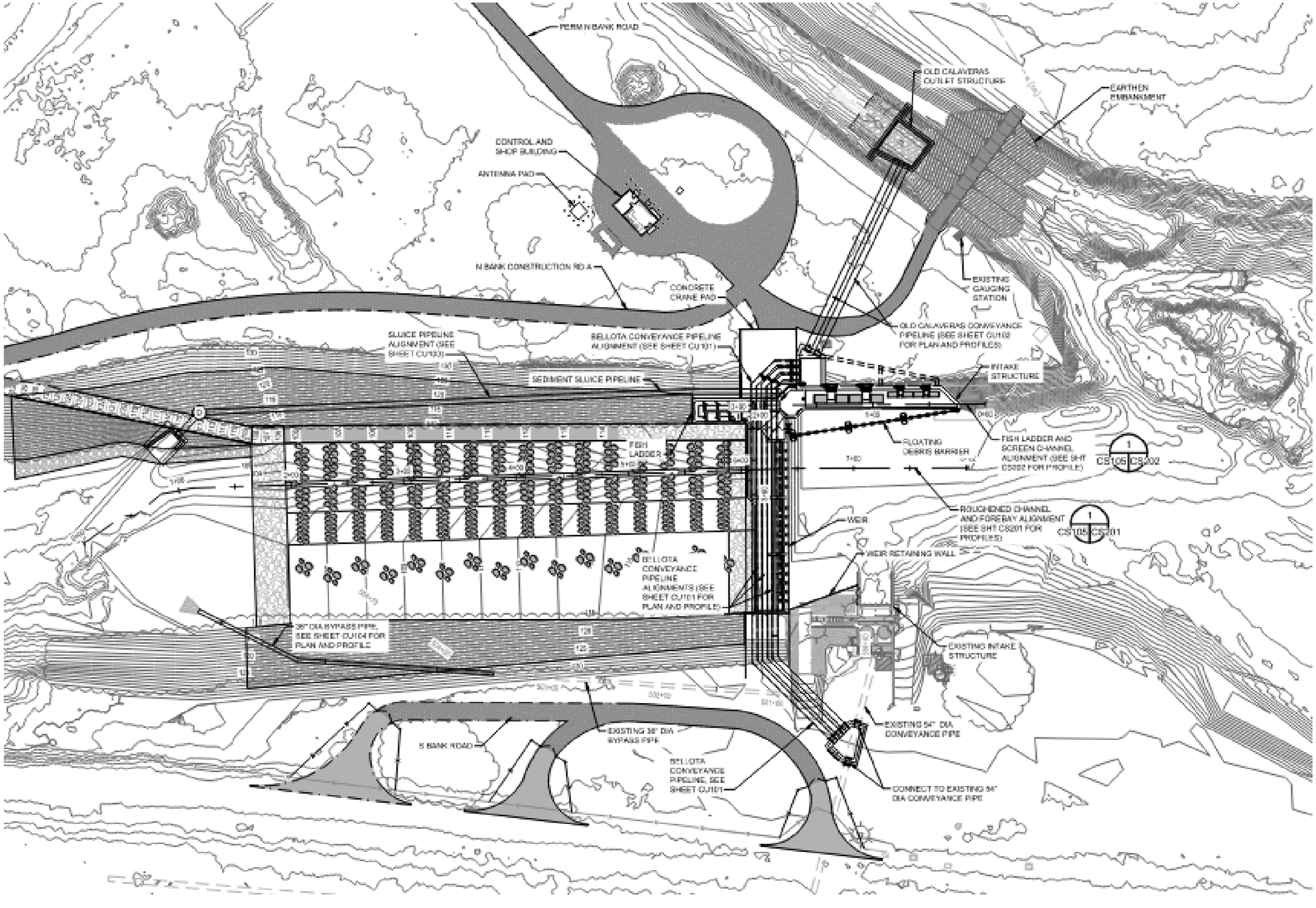
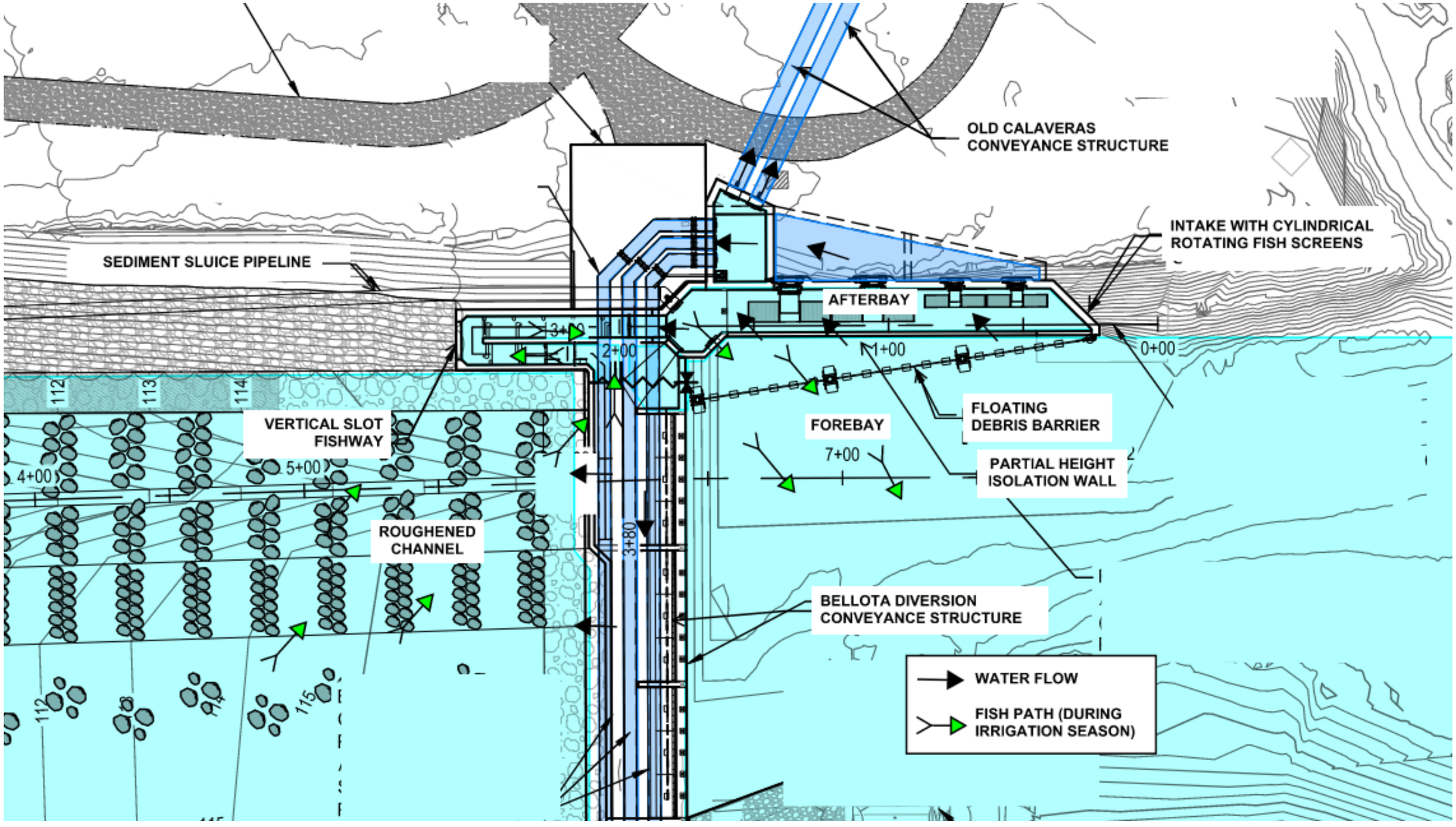


Figure 1-3. Overview of Surface Water Intake and Fish Ladder



1.4 Stakeholder Coordination

Project stakeholders include:

- SEWD and Calaveras County Water District and their customers.
- Agencies
  - NMFS (NOAA Fisheries)
  - U.S. Fish and Wildlife Service
  - CDFW
  - U.S. Army Corps of Engineers (USACE)
  - CDWR
- Individual Diverters

Design review meetings with the agencies are occurring after each phase of design to obtain stakeholder buy-in. Similarly, SEWD's ad-hoc board meetings are also occurring at each phase of the project to obtain feedback from diverters and maintain board approval for the Project.

1.5 Environmental Compliance

Environmental planning and permitting includes compliance with the following laws and regulations:

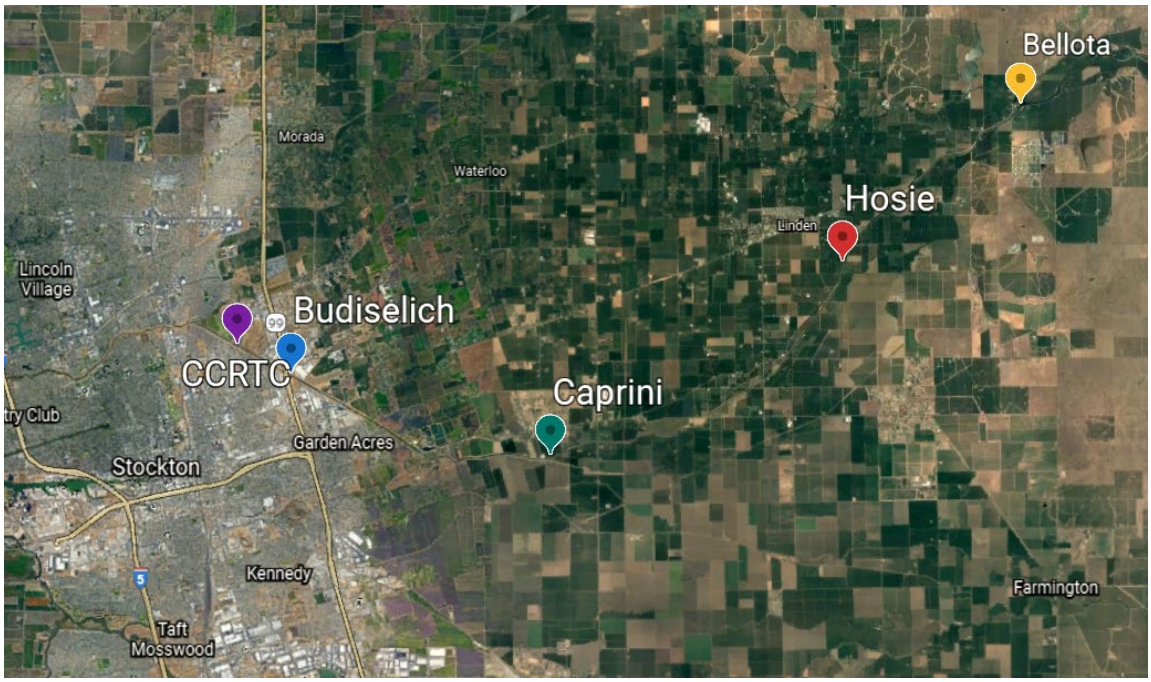
- California Environmental Quality Act
- Federal Endangered Species Act (FESA)
- California Endangered Species Act (CESA)
- California Fish and Game Code 1602, Lake and Streambed Alteration Agreement Package
- Clean Water Act Sections 401, 402, and 404
- Clean Air Act
- National Historic Preservation Act – Section 106
- NMFS – 50-year ITP
- Central Valley Flood Protection Board Permit
- State of California General National Pollutant Discharge Elimination System Permit for Construction Activities
- Applicable County building and development permits (e.g., building and grading)

Permitting to date is in the preliminary phases. This section will be expanded after permitting submittals begin, and will include a table to track the status of the various Federal, State, and Local Permit submittals.

1.6 Companion Projects Influencing Project Needs and Benefits

A number of projects are in process along the Calaveras River as a result of the recommendations identified within the CHCP, such as the Budiselich Flashboard Dam, Caprini Low Flow Road Crossing, Hosie Low Flow Road Crossing (currently in design), and Central California Railroad Traction Crossing (CCRTC). These projects have been constructed or are in planning phases to pass fish from the confluence with the San Joaquin River down to New Hogan Dam. As these projects are implemented, the range of habitat accessible to migrating fish will expand up to the Bellota Weir Project area. The Bellota Weir will be the last major fish impediment modified to connect migrating fish with spawning habitat between the weir and New Hogan Dam. These fish passage project locations on the Calaveras River are shown in Figure 1-4.

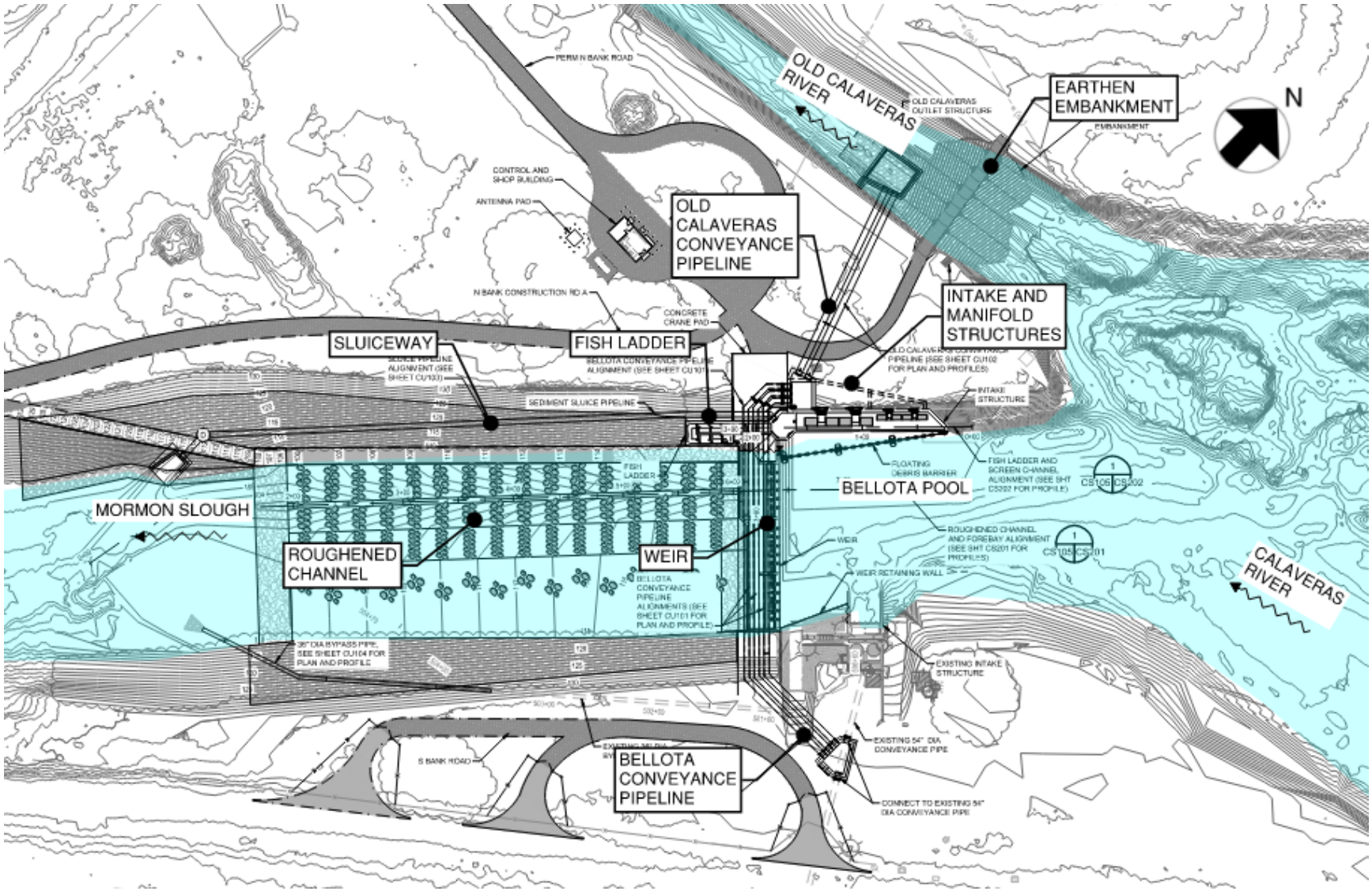
Figure 1-4. Related Fish Passage Projects



2 Overview of Project Elements

The Project will be designed to provide a reliable water supply, improve fish passage, and improve river processes, such as sediment continuity and flood conveyance through the Project reach. This section details the major proposed Project elements (Figure 2-1) and their functions. The general purpose and configuration of each element is provided in subsequent subsections. Facility design and operation details are provided Sections 3 through 0.

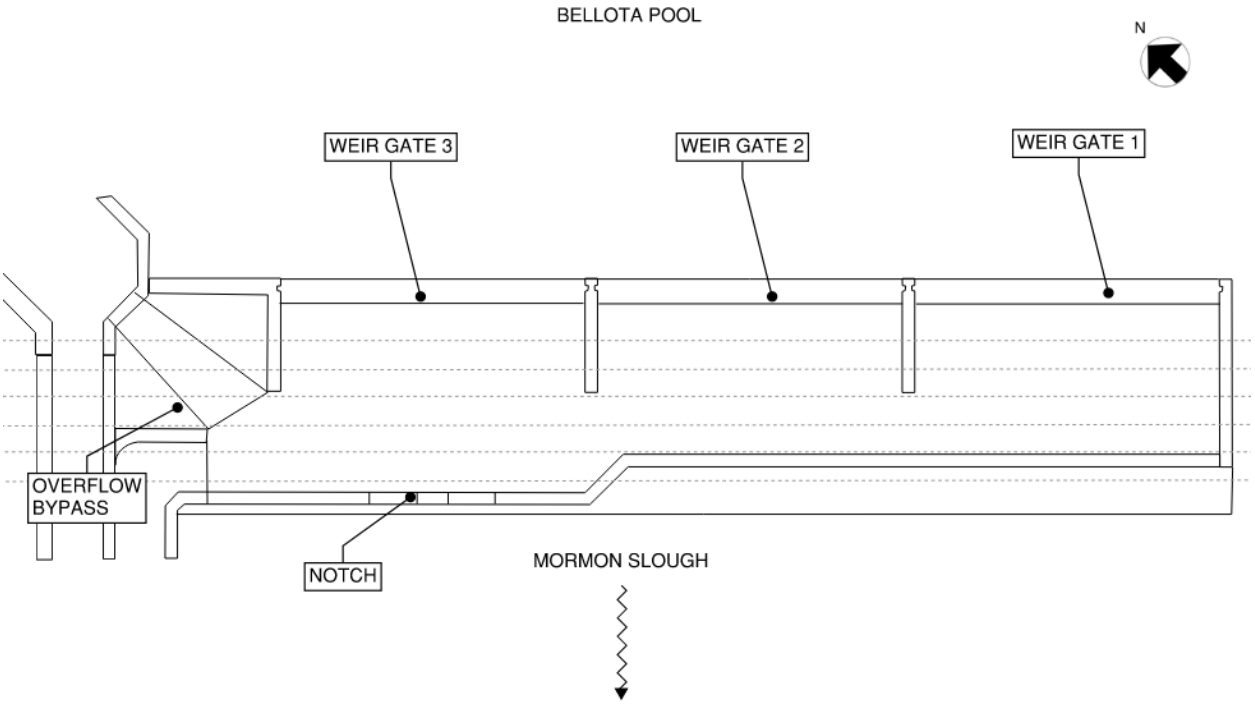
Figure 2-1. Overview of Proposed Project Elements



2.1 Weir

The Bellota Weir will include adjustable weir gates in each of the three bays across the Mormon Slough. Each bay will be separated by a pier. The North gate bay will be set 1 foot lower in elevation than the center and south gate bays. The lower bay elevation will allow fish passage when the fish ladder is not in operation. Additionally, weir gates will be used to maintain the forebay water surface elevation (WSE) when in the fully raised or partially raised position. When lowered, the weir gates allow for the passage of debris downstream and fish passage upstream of the lower-set gate at the North gate bay. Accumulated bed load could also be moved when the weir gates are fully lowered; however, it will not be the most effective method of passing accumulated bed load when it is not agitated and/or suspended in the water column. During operational periods, the minimum and maximum maintained WSE will be 115.44 feet NAVD88 and 121.44 feet NAVD88, respectively.

Figure 2-2. Weir



The adjustable weir gates will be actuated by pressurized air bladders that sit underneath the gate leaf on the downstream end which rotate about the hinge located at the upstream end. Compressed air will be piped from the control building to inflate the bladders and lift the gate to the desired position. The maximum height of the gate leaf, or the gate leaf in the fully raised position, will be set by restraining straps anchored to the concrete and connected to the gate. To lower each gate leaf, air will be released from the bladder.

The air bladder will be made from durable rubber; however, it could be susceptible to being punctured from debris. This Project will include a plate to protect the bladder. The protection plate will be attached to the gate leaf's downstream tip with a flexible attachment and the other end has a wheel allowing it to move up and down the concrete apron accommodating the full range of gate levels.

There will be a water level sensor upstream of the adjustable weir gates that will feed into the programmable logic controller (PLC) at the control building. A user will be able to set the desired WSE of the forebay and the adjustable weir gates will move and modulate to reach and maintain the desired WSE.

In the event part of the adjustable weir gate(s) needs to be maintained, there will be an attachment point on the gate and pier walls. The gate leaf could be held in the raised position by chain or wire rope to the pier without the air bladder being inflated.

Removable stoplogs will also be designed for installation at the upstream side of weir. There will be an anchoring location for mounting and dismounting the temporary stoplog. The whole weir slab will be built on deep foundation piles to mitigate the potential of settlement during seismic events and conveyance pipes will pass under the weir between foundations piles. The conveyance piping will be supported through channel elements crossing between foundation piles.

2.2 Intake Structure and Manifold

The existing intake structure along the south bank of the Calaveras River includes two fish drums for conveyance to the Dr. Joe Waidhofer WTP, which provides irrigation, municipal, and industrial flows to the District.

Under the proposed Project, the existing intake structure will remain as a backup facility but be taken offline. A new intake structure will be constructed on the north bank with four different fish screens that will serve the WTP and provide screened water flow to the Old Calaveras River for recharge and irrigation purposes of downstream diverters. The resulting maximum demand for diverting flow at this new consolidated intake location is summarized in Table 2-1.

Table 2-1. Maximum Irrigation Diversion Flows

Diversion	Present Flow (cfs)	Future Flow (cfs)
WTP	70	200
Old Calaveras River	150	150
Total	**220**	**350**

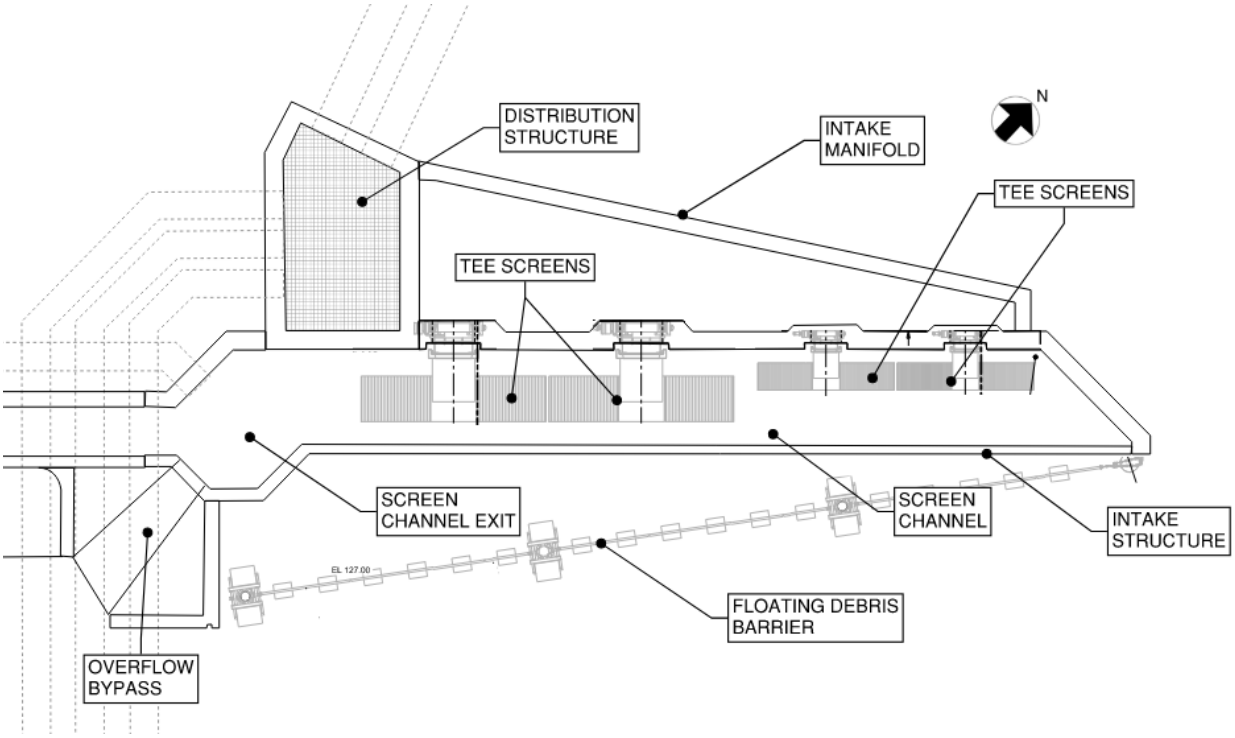
The surface water intake structure will be screened to meet NMFS criteria to prevent fish entrainment or impingement, including juvenile salmonids (Figure 2-3). The intake will incorporate four cylindrical rotating screens in a T-configuration mounted on retractable guide rails. Two will be larger, and two smaller in order to provide uniform velocities

through the large range of flows expected through the intake. Sweeping velocities will be maintained using the fish ladder and overflow bypass.

The collection manifold will be designed to promote uniform approach velocities through each of the screens. As flows increase through the screens, the width of the manifold will increase. The manifold will be underground and increase in height from upstream to downstream to accommodate increased flow. The downstream end of the manifold will end at the distribution structure, which diverts flow to either the WTP or Old Calaveras River and extends to the surface.

Sections 4, 5, 7, and 8 contain information pertaining to the hydraulic, civil, mechanical, and structural design of the intake.

Figure 2-3. Intake Structure and Manifold



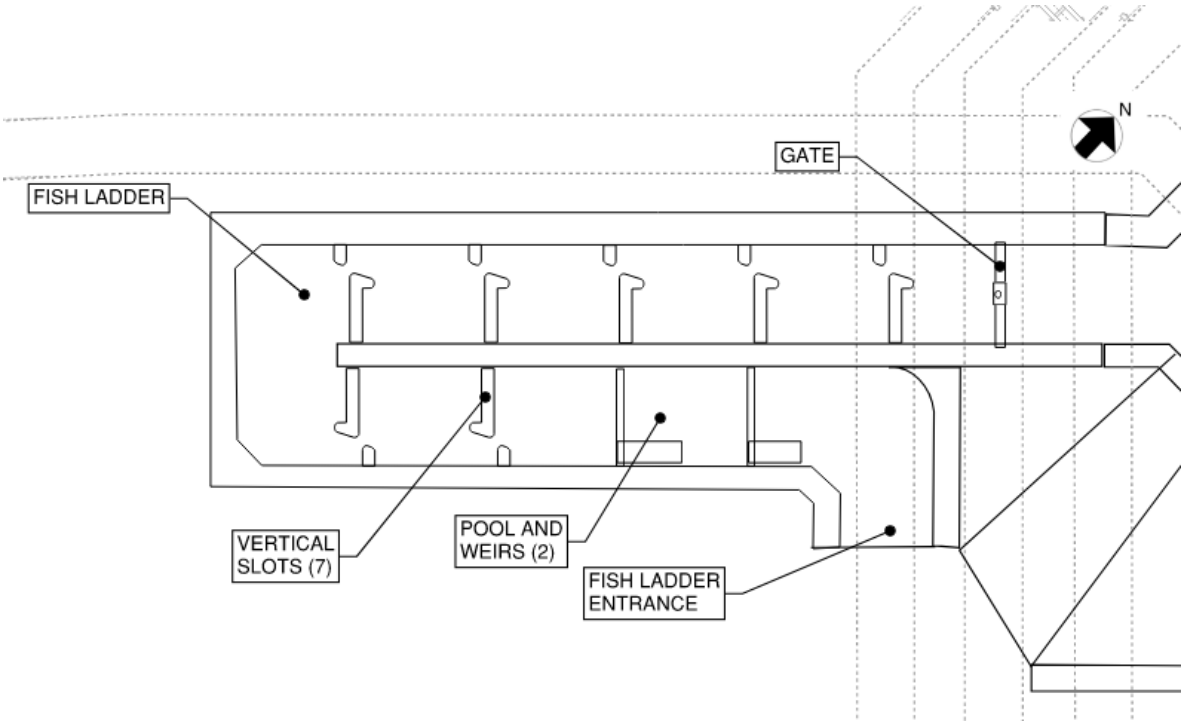
2.3 Fish Ladder

The fish ladder exit will be located downstream of the screen channel. The fish ladder will only be used during the irrigation season, when the water surface level is raised to 121.44 feet NAVD88 in the forebay. The ladder will consist of seven vertical slots and two pools and weirs (Figure 2-4). Each pool will be 8 feet wide and 10 feet long. An auxiliary water supply will not be needed because the flow in the ladder will fit the target attraction flow criteria defined by NMFS. Lamprey ramps will be incorporated at each of the two pools and weirs with vertical slots rounded to facilitate lamprey passage.

Fish monitoring will be provided at the fish ladder. The design of the fish monitoring system is not in HDR's contract and will be designed by others.

Sections 4 and 8 contain information pertaining to the hydraulic and structural design of the fish ladder.

Figure 2-4. Fish Ladder



2.4 Roughened-Channel Fishway

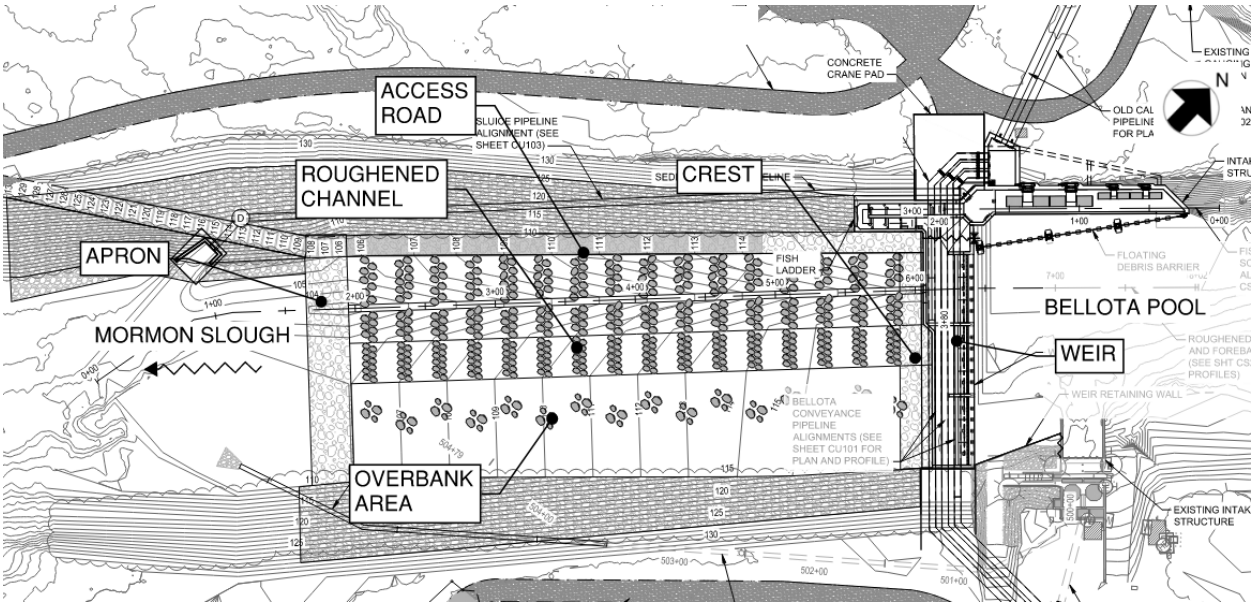
Downstream of the fish ladder entrance, there is a notch in the sill of the weir that is designed to provide fish passage at low flows. This notch is the same design as the roughened-channel fishway geometry. The roughened-channel fishway will consist of a channel spanning crest with one main channel and an overbank area (Figure 2-5). No grout or concrete will be used in the crest or fishways. The roughened-channel fishway will provide fish passage up to the high fish passage design flow and be utilized during all times of the year.

The primary roughened-channel fishway will slope downstream at 3 percent for about 400 feet until it catches the existing channel grade downstream. Additional scour and structural countermeasures such as the apron will be incorporated immediately below the roughened channel and crest to accommodate general channel scour and anticipated structure scour that may occur at both transitions. There will also be an access road located on the right bank of the channel to ease maintenance following large flow events.

Fish monitoring will be provided at the roughened-channel fishway. The design of the fish monitoring system is not in HDR's contract and will be designed by others.

Section 4 contains more information pertaining to the hydraulic design of the roughened channel.

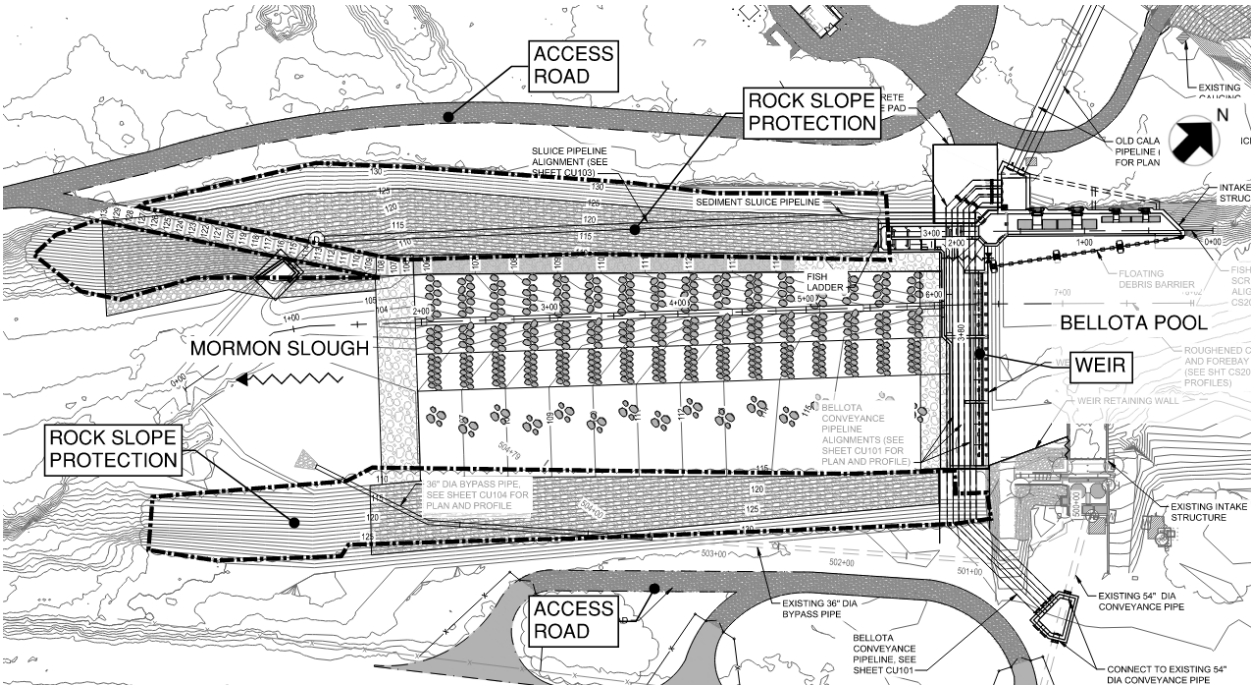
Figure 2-5. Roughened Channel Plan



2.5 Rock Slope Protection

Integration of the new Project elements into the Calaveras River and Mormon Slough will influence the velocity and shear characteristics present. Through the hydraulic design process, several locations were identified as at risk of future exposure to scour and erosion. Rock slope protection will be proposed along the banks of the roughened channel to counteract erosive processes (Figure 2-6). In addition, the rock slope protection will protect the access roads currently proposed on the site on the north and south banks of Mormon Slough.

Figure 2-6. Rock Slope Protection Areas



2.6 Sluiceways

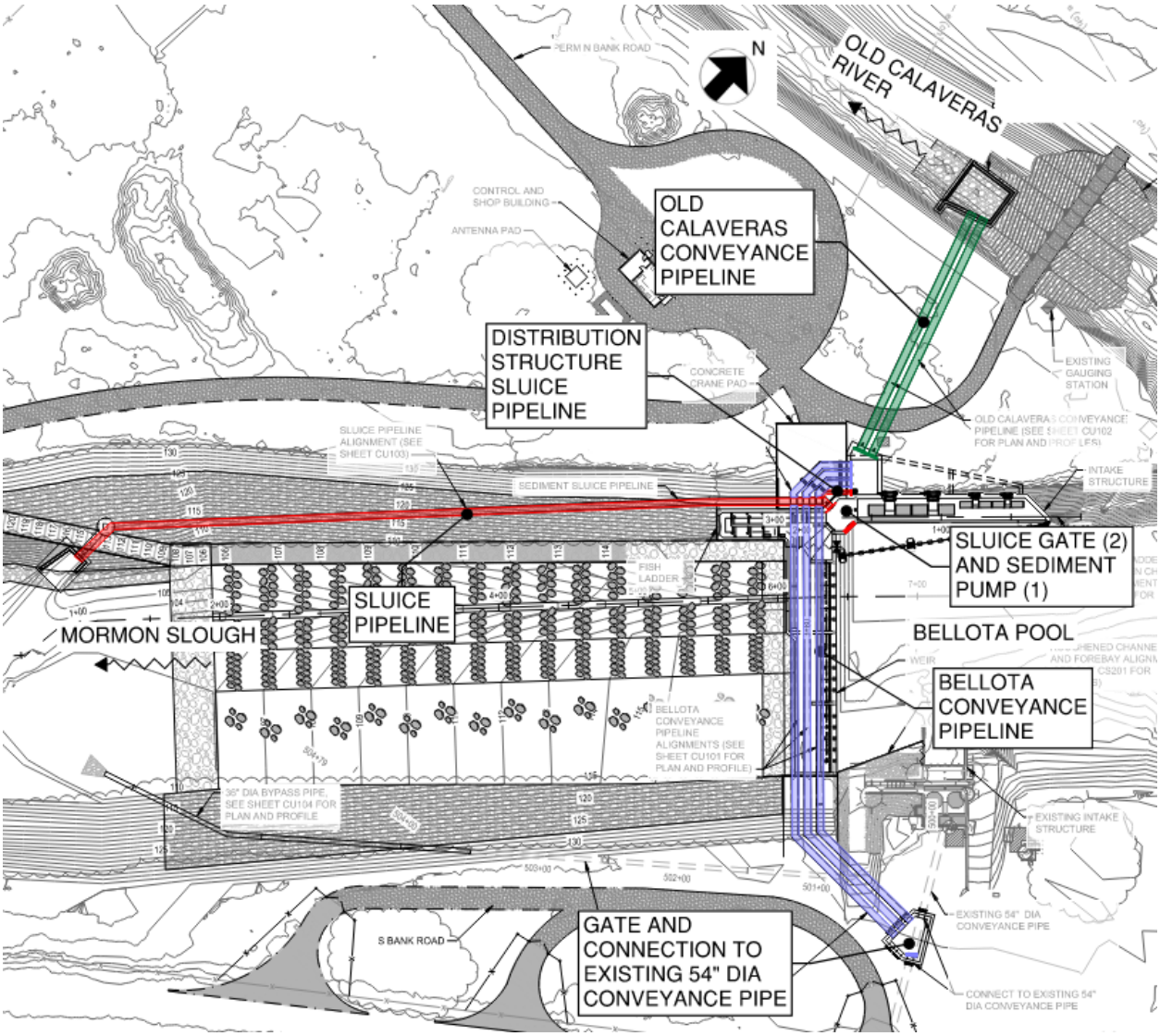
Sluicing will be incorporated at three different Project locations. There will be a sediment sluice gate for sluicing of the forebay. Whenever the sediment sluice gate is opened, any sediment that has accumulated along the debris wall will be flushed through the gate.

The screen channel will have sluicing capabilities. The screen channel will be sloped and reach a low point to accumulate sediment. When the 54-inch-diameter sluice pipeline gate is open, screen channel sediment also will be washed to the same point downstream of the roughened channel.

Finally, sediment sluicing provisions will be added to facilitate sluicing of accumulated sediment present in the distribution structure. A solids-handling submersible pump will be placed into a pit inside the distribution structure and connected to an 8-inch-diameter sluice pipe. There will be an isolation valve to isolate the pump and a check valve to prevent backflow of sediment into the pump. The pump will be operated after the eductor system in the distribution structure is activated to remove sediment from the region. When the pump is operating and the 8-inch-diameter sluice pipeline valve is open, accumulated sediment will be conveyed to the 54-inch-diameter sluice pipeline, sluicing the sediment to the same point downstream of the roughened channel.

Figure 2-7 illustrates sluicing features (red), the Bellota conveyance pipeline (purple), and the Old Calaveras conveyance pipeline (green).

Figure 2-7. Sluice Pipeline, Bellota Conveyance Pipeline, and Old Calaveras Conveyance Pipeline



2.7 Bellota Conveyance Pipeline

The main conveyance system will be the Bellota conveyance pipelines. These three 54-inch-diameter steel pipes will be formed into the concrete dam sill and capable of passing 200 cubic feet per second (cfs) of diversion flow to the District's main conveyance pipeline(s) (Figure 2-7). The three 54-inch-diameter steel pipes will extend to the south wet well, which connects with the current District conveyance system. These pipes will provide the current District demands until final buildout of their main conveyance system is capable of delivering the 200 cfs flow. There will be three methods of controlling flow: 1) calibrating the three seating head slide gates upstream of each 54-inch-diameter pipe for downstream flow control; 2) adjusting the hydraulic conditions feeding the pipeline(s) based on plant demand; and 3) manually adjusting the gates using the handwheel actuator to obtain the desired flow.

2.7.1 Connection to Existing Pipeline

There will be an initial tie-in to the existing 54-inch-diameter RCP main pipeline using a concrete vault with one new 54-inch-diameter RCP tie-in, and wall space with two knockouts for the additional buildout of the two new future 54-inch pipes or similar flow capacity conveyance lines (reference Sheet SS105). Concrete wall penetrations for both the existing RCPs and new WSPs will utilize rubber boot fittings in accordance with ASTM C923 criteria.

2.8 Fish Exclusion Improvements

2.8.1 Earthen Embankment

The earthen embankment will be located in the Old Calaveras River channel as close to its divergence from the Calaveras River/Mormon Slough as feasible to provide fish exclusion for all flows to the Old Calaveras River. The earthen embankment will consist of a low-permeability core cutoff wall, such as a clay or bentonite soil mix, an earthen embankment across the Old Calaveras River to a minimum of 3 feet above the 100-year base flood elevation, and rock slope protection on both sides of the earthen embankment to protect against erosion. The earthen embankment will have an effective crown at EL 131.6 feet, and approximate toe at EL 112 feet.

2.8.2 Old Calaveras Conveyance Pipeline

The Calaveras Conveyance pipelines will consist of two 54-inch-diameter RCPs connecting the distribution structure to the existing Old Calaveras Creek channel (Figure 2-7). During construction, one of the 54-inch lines will extend to the Bellota Pool to provide Calaveras Creek with current demands. Once the intake structure is complete, the temporary connection to the Bellota Pool will be abandoned and water supplied from the distribution structure. The Calaveras Creek outlet structure will be designed to accommodate energy dissipation required for flow entering the creek. A riprap apron will also be designed to further dissipate energy in the flow leaving the structure. Flow will be measured by two ultrasonic inline flow sensors and controlled by two modulating slide gates at the distribution structure. The gates will have three controls modes: 1) maintain a desired flow in Calaveras Creek; 2) control a targeted water surface in the distribution structure; and 3) manually adjust the gate openings to the desired flow.

2.9 Control Building

A small, two-room utility building will be located near the river intake structure. The building will consist of a mechanical/electrical room and an equipment room. The mechanical/electrical room will house the adjustable weir controls such as the compressor(s), receiver tank, and control cabinets. The mechanical/electrical room will also house all control cabinets needed for site features. No utilities will be provided to or from this building aside from electricity.

Each room will have venting louvers and intake fans for air flow. Neither room will be continuously occupied, and ventilation will be sized to have a minimum air exchange of

five air exchanges per hour per American Society of Heating, Refrigerating and Air-Conditioning Engineers 62.1 guidance. The mechanical/electrical room will have intake louvers sized to accommodate the air compressors supplying the weir gates to create negative pressure.

Air conditioning will be provided to both rooms by means of a mini split ductless heat pump with the main unit sitting outside the building on an equipment pad. Also, outside of the building will be the main switchboard. While a weatherproof enclosure will be specified, the switchboard should sit on the north side of the building to minimize sun exposure.

In the event of a power outage at the site, there will be an emergency generator on standby to provide the required site power. It will be located outside, near the control building, and by the roadside for ease of re-fueling activities. The generator should be oriented with the exhaust ports away from the building to prevent exhaust from inundating the building. Ventilation intakes for the building will be situated to prevent this from happening.

The building will be constructed of reinforced concrete masonry unit blocks and founded on reinforced concrete wall footings. The roof system will be prefabricated metal plate connected to wood trusses. A concrete pad will also be provided near the building for an antenna, switchboard and AC equipment foundation. According to geotechnical recommendations, the top 6-foot layer of soil will be removed and replaced by compacted engineered material before placing the concrete foundations.

3 Design Criteria

The primary drivers and criteria that govern the Project design are presented in this section. For additional detail on all design criteria and standards and references used in the design process, refer to Sections 4 through 0 and the Design Criteria Report in Appendix A.

3.1 Biological Criteria

This section presents a brief summary of biological criteria used in the facility design, including target species, fish migration timing, and fish passage performance. A detailed description of biological criteria for the Project can be found in Section 2 of the Design Criteria Report in Appendix A.

Table 3-1 provides a list of target fish species selected to establish fish passage design criteria for the Project based on their known occurrence at the Project site and importance to the recovery of fisheries resources within the Calaveras River system. They are selected based on feedback obtained from NMFS and CDFW and have been identified as most critical for this reach.

Table 3-1. Target Fish Species and Life Stages

Species	Life Stages	
	Upstream Passage	**Downstream Passage**
Chinook Salmon (spring, winter, late-fall, and fall run*)	Adult/Yearling	Juvenile
Steelhead	Adult/Yearling	Adult/Juvenile
Lamprey	Adult	Juvenile (ammocoetes)

Upstream and downstream movement of adult, sub-adult, and/or juvenile life stages of target fish species may be observed throughout various times of the year.

In summary, any fish passage facility located at the Bellota Weir will need to accommodate the following:

- Adult salmonids upstream migration from November through May
- Juvenile salmonid downstream migration from November through June
- Adult lamprey upstream migration from April through June

The facility will not require design for anything other than upstream migrating adult salmonids.

3.2 Facility Criteria

Design criteria related to operational strategy, irrigation demand, and hydrology are presented in this section with detailed descriptions located in Section 3 of the Design Criteria Report in Appendix A.

3.2.1 Operational Strategy

The diversion season begins mid-April and ends mid-October each year. During that period, the new intake and pipe conveyance infrastructure will be operated to meet irrigation demands up to a maximum of 350 cfs throughout a wide range of river and environmental conditions. The roughened-channel fishway will operate year-round to provide fish passage throughout the full range of anticipated river flows where fish migration is anticipated. During non-operational periods, regular maintenance activities will be dutifully accomplished to keep the facility and its elements in working order and ready to perform during the next irrigation season. Additional detail associated with the intake and roughened channel operational strategy is provided in Section 0.

3.2.2 Design Life

The Project components will be designed to accommodate a typical planning period ranging from 15 to 50 years. The generally accepted life of equipment, electrical systems, structures, and buildings is as follows (HDR 2019):

- Equipment: 15 to 25 years; maybe longer with proper operation and maintenance
- Electrical systems: 25 to 30 years
- Concrete structures: 50 years
- Major pipelines: 50 years
- Buildings: 30 to 50 years

The roughened channel will be designed to survive flows up to and including the 100-year flow (12,690 cfs) with some anticipated damage.

Project failure at this site will be defined as the inability to: 1) effectively pass fish through the reach, 2) effectively screen fish from water being diverted for water supply and irrigation demands, 3) allow flood flows to pass the improvements, or 4) provide a reliable water supply to meet irrigation demands. These occurrences will be minimized to the extent possible. The structural integrity of the weir crest will not be compromised during or following any given flow event up to the 100-year flow.

Flows greater than the 100-year event may occur during the useful lifespan of the proposed Project improvements. The level-of-service interruption or damage that may occur during an event of this nature may be evaluated further using an uncertainties analysis during a later stage of design.

3.2.3 Irrigation Demand and Seasonality

Irrigation water demand can vary from year to year, but the New Hogan reservoir releases have historically averaged about 150 cfs for irrigation demands for the Bellota and Old Calaveras River diversions (CHCP 2019). The maximum irrigation diversion flow planned for the Project is 350 cfs, which includes 200 cfs for the Bellota diversion and 150 cfs for the Old Calaveras River. Irrigation demand and the seasonality of irrigation demand are discussed in greater detail in Section 0.

3.2.4 Minimum Required Operating Water Surface Elevation

The maximum and minimum required operating WSEs during irrigation and non-irrigation seasons are 121.44 feet and 115.44 feet NAVD88, respectively. The non-irrigation operating WSE will be used as the minimum design operating WSE in the Mormon Slough and the limiting condition for design of gravity flow hydraulics through the intake and proposed conveyance systems.

3.3 Agency-Specific Criteria

Numerous agency-specific criteria and design guidance will be used to inform the design of various Project components. Table 3-2 provides a summary of the agency-specific criteria used in this design. A detailed description of each criterion and how it will be applied can be found in Section 4 of the Design Criteria Report in Appendix A.

Table 3-2. Agency Specific Criteria

Agency	Criteria/Description
Federal Emergency Management Agency (FEMA)	• Governs work in regulatory floodways and floodplains. This Project will comply with a no-rise certification strategy in collaboration with the San Joaquin County Flood Control Agency.
California State Department of Transportation	• Coordination relative to access from State Route 26 for the north bank site construction.
San Joaquin County Public Works	• County roadways, repairs, and traffic control will comply with San Joaquin County Standard Plans and Specifications. Traffic control through construction work zones will comply with both the County work zone safety and the Federal Highway Administration Work Zone Management Program (https://www.fhwa.dot.gov/workzones).
CDFW	• Fish protection and passage requirements for numerous Project elements including fish screens, fish bypass, and debris racks.
NMFS	

3.4 Hydraulic Design Criteria

Hydraulic design criteria and guidelines are uniquely formulated for the following regulatory compliance requirements and Project elements:

- Compliance with FEMA and County floodplain regulations

- Surface water intake
- Sluiceway design
- Roughened-channel, nature-like fishway
- Fish ladder
- Scour evaluation and assessment of potential impacts to existing infrastructure
- Scour and structure settlement countermeasure design

The overall design intent, requirements, and applicable guidelines are detailed in Section 5.1 of the Design Criteria Report in Appendix A and summarized at the beginning of each individual element documented in Section 4 of this document.

3.5 Technical Design Criteria

Technical criteria that govern the civil, geotechnical, mechanical, structural, and electrical design-related Project elements are detailed in Section 5 of the Design Criteria Report in Appendix A and summarized as applicable in the following sections of this document.

- Section 5 – Civil Design provides numerous applicable civil criteria that govern a number of design elements. The civil design aspects of the Project will meet State, County, and SEWD requirements and include: survey control, site safety, traffic control, road maintenance, access, civil site grading, stormwater and drainage, erosion and sediment control, security, and FEMA-regulated floodplain work.
- Section 6 – Geotechnical Design provides specific geotechnical design criteria related to testing methods, materials, permeability, seepage, foundations, slope stability, earth fill, and rock filters.
- Section 7 – Mechanical Design provides mechanical and pipeline conveyance criteria related to the mechanical design of the intake including the fish protection screens, water control gates, sluiceway weir gate, and compressed air supply system.
- Section 8 – Structural Design provides governing criteria related to concrete foundations and walls for both the sluiceway and intake structures, overhead lifting support structures, building foundations, and miscellaneous metal fabrications.
- Section 9 – Electrical Design provides electrical and instrumentation and controls criteria related to service power distribution, electrical supply, lighting, control equipment, and communication equipment.

4 Hydraulic Design

This section describes the design intent, process, and results of calculations that will be used in the hydraulic design of numerous Project elements. Specific hydraulic design calculations can be found in Appendix E. The Hydraulic Modeling Summary Report can be found in Appendix B.

Hydraulics will be evaluated to inform the design process and confirm design compliance with regulatory requirements. Specifically, the hydraulic design objectives will:

- Support regulatory permitting efforts associated with Project improvements
- Confirm that the range of hydraulic conditions anticipated after Project implementation will meet the fish passage, sediment passage, and water supply and flood resilience/stability criteria established during preliminary and final design
- Refine sediment continuity and particle mobility thresholds to inform sediment transport and sluicing targets
- Provide hydraulic conditions such as shear stress to verify the following:
    - Sizing and stability of channels
    - Boulder matrices of the roughened channel
    - Supporting rock filters for boulder structure and low flow channel
    - Potential for scour holes downstream of structure and forces on existing structures

4.1 Compliance with FEMA and County Floodplain Regulations

The Project is currently within a regulatory floodway designated by the FEMA National Flood Hazard Layer and Flood Insurance Rate Map (FIRM Panel 06077C0370F effective October 20, 2016). For new construction, the Project design team must demonstrate, through appropriate hydraulic analyses, that the proposed improvements will not result in any increase in base flood elevation for a 100-year event (1:100 chance of occurrence in a given year for the 100-year event). A one-dimensional (1D) HEC-RAS model will be used to evaluate FEMA compliance. For details on the FEMA mapping and the no-rise approach and hydraulic modeling methods see the Hydraulic Modeling Summary Report in Appendix B. The no-rise results are summarized below.

4.1.1 No-Rise Discussion

Results from the 1D existing and proposed conditions analyses indicate that the water surface for the FEMA 100-year flood event are both lower than the FEMA effective base flood elevations (Figure 4-1). A No-Rise condition will be generally achieved along the entire study reach, except for a few localized increases in WSE at and downstream of the proposed weir structure. The 100-year flow at locations that experienced an increase in WSE under proposed conditions will be contained within the channel and will not

exceed the extents of the riprap lining. Furthermore, there are no insurable structures at these locations and localized increases in WSE end at the inlet to the weir gates, which serves as the proposed hydraulic control. The local floodplain administrator (San Joaquin County) will need to be engaged to assess compliance requirements. A conditional letter of map revision may be required. The larger increases through the weir will likely be from the proposed weir extending approximately 20 feet downstream of the end of the existing weir, resulting in overlap between the dammed water surface upstream of the proposed weir sill and the existing weir tailwater. Localized increases downstream of the weir will result from the increased tailwater elevation caused by the proposed roughened channel. WSEs from cross-sections at and surrounding the existing and proposed structures are depicted in Figure 4-2.

Figure 4-1. Existing and Proposed FEMA 100-year Water Surface Profiles and FEMA Effective WSE at FEMA Cross-Sections within the Study Reach

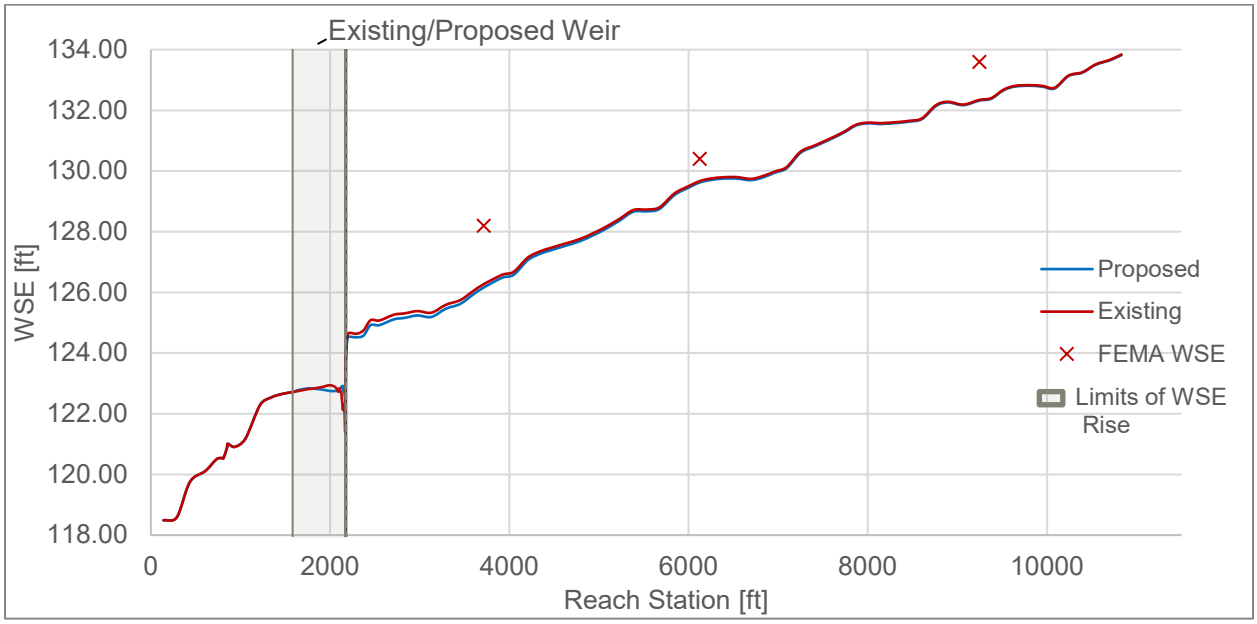
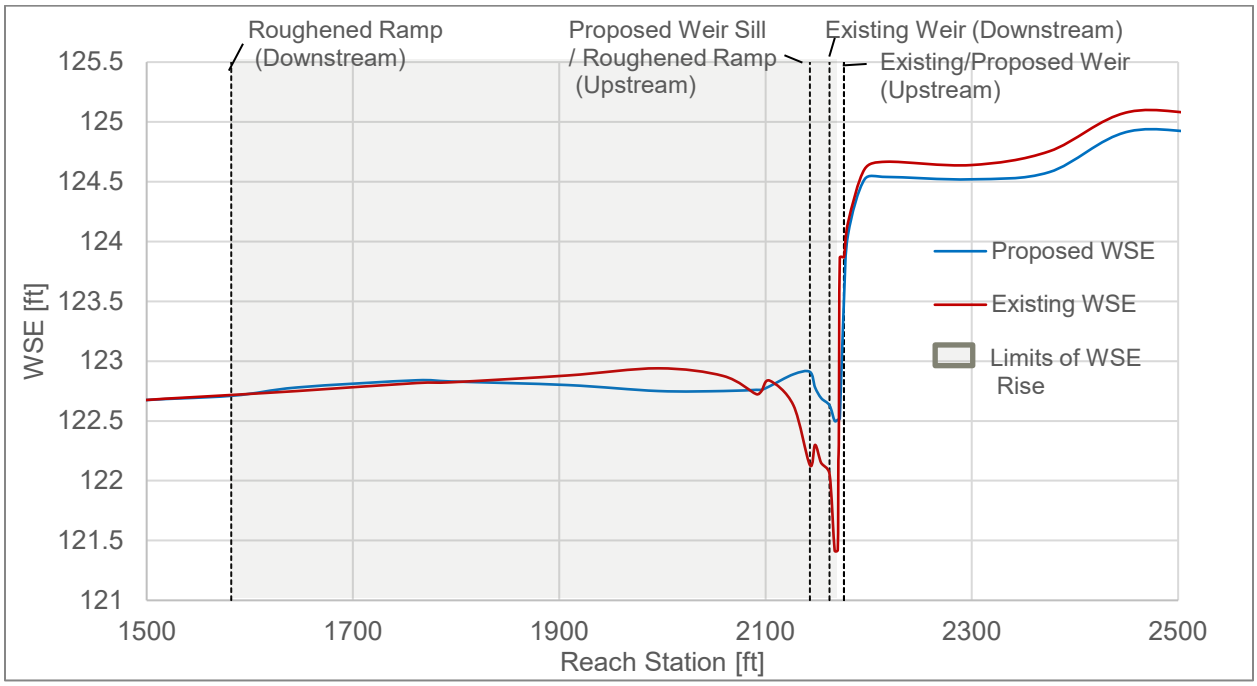


Figure 4-2. Existing and Proposed FEMA 100-year Water Surface Profiles near the Weir



4.2 Weir Design

The adjustable weir design will span approximately 150 feet across the Mormon Slough channel. There will be three bays, each 48 feet and 8 inches wide, with an adjustable weir gate spanning each bay. Each bay will be separated by a pier wall. The center and south weir gates have an operating range of elevation (EL) 115.44 feet NAVD88 when fully lowered to EL 122.5 feet NAVD88 when fully raised. The north weir gate will have an operating range of 114.5 feet to 122.5 feet.

The maximum forebay operating pool will be at EL 121.44 feet NAVD88. The weir gates will provide a 1-foot clear freeboard from the top of gate to the maximum forebay operating pool when in the raised position. In flood conditions, the 100-year flood elevation of the forebay pool will be EL 127.0 feet NAVD88.

The weir gates will be capable of passing a peak flow corresponding to the 100-year flood event, which is estimated to be 12,690 cfs. A summary of design flows is provided in Table 4-2.

The weir design is further summarized in Sections 2.1, 7.1 and Appendix F.

4.3 Intake and Forebay Design

The following section provides the design intent and results for the following Project elements.

- Intake design
- Forebay design

- Trashrack design for the fish ladder only

Fish screen design is summarized here and further described in Section 7.2.

4.3.1 Intake Design

The intake design will accomplish the following objectives:

- Design the trashrack in compliance with the effective flow-through velocity requirement of 1.5 feet per second (fps) as governed by NMFS 2011.
- Design the forebay with an equal or gradually increasing sweeping velocity across each fish screen with a target velocity ranging from 0.8 fps to 2 fps when practicable.

Intake Design Summary

Because irrigation flows are required on each side of the river at the Bellota Weir, the two intakes will be consolidated into one larger intake. The new combined fish screening system will be designed for a maximum diversion flow of 350 cfs (150 cfs for the Old Calaveras River [right bank] and 200 cfs for the SEWD Diversion to the WTP [left bank] during irrigation season). The right bank was selected as the location of the combined intake because the larger of the two diversions is located on the right bank and the existing intake is intended to remain on the left bank for redundancy purposes.

Retractable cylindrical tee screens were selected due to their efficient cleaning and maintenance history as well as their ability to optimize diverting water per linear foot of riverbank when compared to the traditional flat panel fish screens. Additionally, two different size retractable tee screens were selected to provide water diversion in both low pool and high pool conditions. Two 6.92-foot-diameter by 27.58-foot-long (143.4-cfs capacity/tee) and two 4-foot-diameter by 18.25-foot-long (48.8-cfs capacity/tee) retractable tees were chosen to best meet criteria under both irrigation and non-irrigation periods (Figure 4-3).

During non-irrigation season when the minimum low pool elevation is 115.5 feet NAVD88, the dam weirs will be in the lowered position and fish will be able to pass across the dam in the river thalweg. The maximum allowable diversion during this time will be 75 cfs. The two smaller screens will withdraw the 75 cfs with a maximum approach velocity of 0.30 fps.

During the irrigation season when the high pool elevation is 121.5 feet NAVD88, the dam weirs will be fully raised and fish will be able to pass through the fish ladder. The maximum allowable diversion during this time is 350 cfs. All four screens will withdraw the 350 cfs with a maximum approach velocity of 0.30 fps.

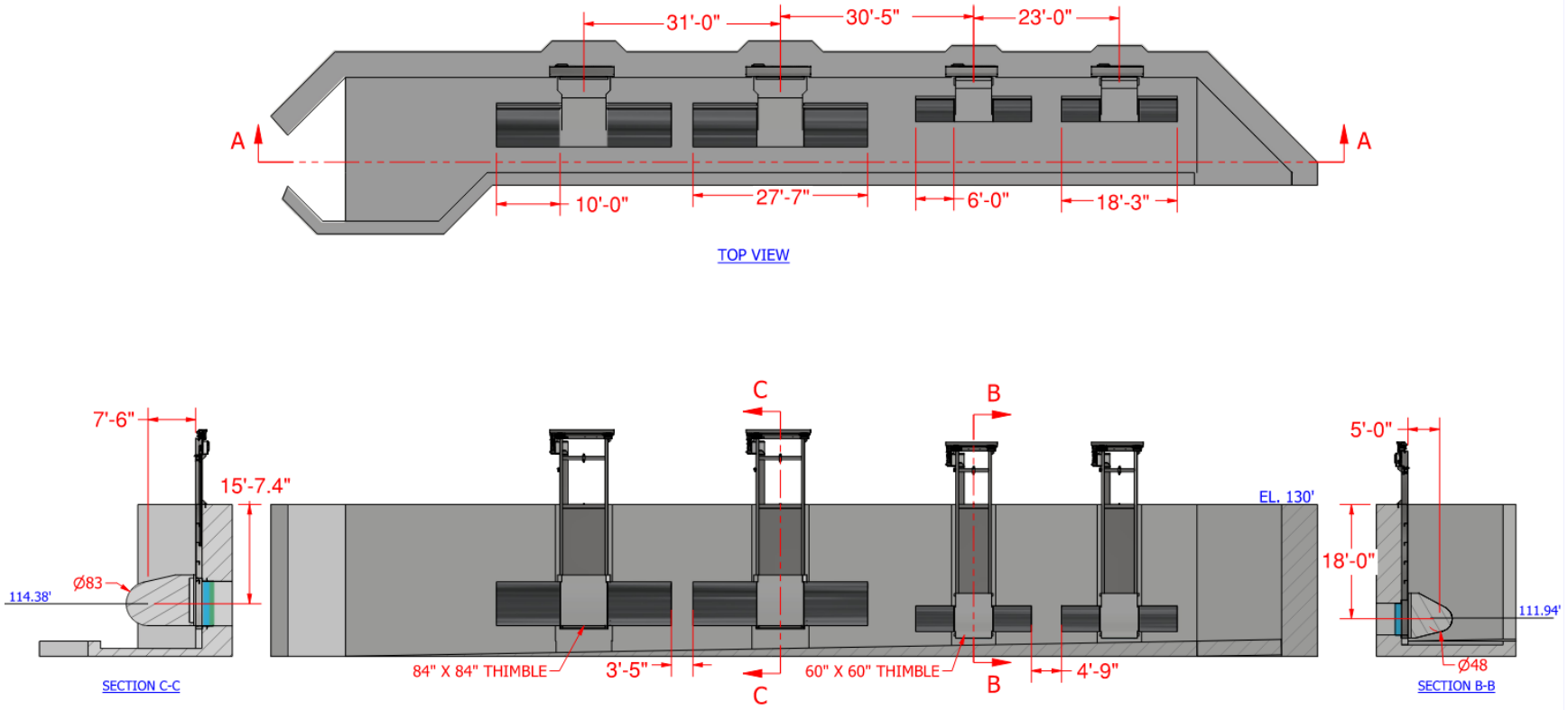
This intake will divert river water while trying to meet regulatory fish screening criteria. It should be noted that both submergence depth and clearance off the floor do not meet agency criteria in all cases. The submergence depth for the 48-inch-diameter tee screens is 1.5 feet (not 2 feet) and for the 83-inch diameter tee screens, the submergence depth is 3.6 feet (not 4 feet). The furthest upstream 48-inch tee screen basket has a clearance off the intake floor of 1.75 feet (not 2 feet) and the upstream most

83-inch tee screen has a clearance of 3.71 feet (not 4 feet). Table 4-1 details clearance for each screen during the range of water surface conditions that will occur.

Table 4-1. Screen Clearances

Screen #	Diameter (ft)	Floor Elevation (ft)	Bottom Clearance (ft)	Bottom of Screen Elevation (ft)	Top of Screen Elevation (ft)	Water Surface (ft)	Submergence (ft)
4	4.0	108.14	1.75	109.44	13.94	115.44	1.5
	4.0	107.83	2.11	109.44	113.94	115.44	1.5
3	4.0	107.73	2.21	109.44	113.94	115.44	1.5
	4.0	107.37	2.57	109.44	113.94	115.44	1.5
2	6.92	107.22	3.71	110.92	117.84	121.44	3.60
	6.92	106.66	4.26	110.92	117.84	121.44	3.60
1	6.92	106.60	4.33	110.92	117.84	121.44	3.60
	6.92	106.04	4.88	110.92	117.84	121.44	3.60

Figure 4-3. Screen Layout



Screen Channel Design

The screen channel will be designed to divert water while keeping the main thalweg out in the river for both upstream and downstream fish passage during low pool operating conditions. The screen channel has a sheet pile wall with a concrete cap that partially isolates the thalweg from the screen channel. This wall was set at EL 114.5 feet NAVD88 with a short 20-foot-long section at the upstream end of the wall for flow to the fish screens under low pool elevation conditions. Sweeping velocity will be maintained using flow through the fish ladder, overflow bypass, and adjustable weir gate 3. Weir gates will be operated automatically to maintain sweeping velocity for the fish screens and diversion.

4.3.2 Forebay Design

The forebay will be designed to allow sluicing in the dam forebay, on the right side near the intake chamber. This will be accomplished using a series of sluice gates that can be operated to sluice material from the forebay or the intake chamber. Forebay elevation will be leveled off so that periodically, when sediment builds upstream of the weir or on the river side of the intake, sluice gates can be opened to flush the sediment downstream. Sluicing strategies are further explained in Section 4.4.

4.4 Sluiceway Design

The following section describes the intent, general description, and resulting hydraulics anticipated for the piped sluiceway.

4.4.1 Sluiceway Design

The piped sluiceway will accomplish the following objectives:

- Actively promote hydraulic preference of the weir bay closest to the north bank and maintain a clear hydraulic pathway to the surface water intake.
- Promote simple maintenance of the intake and forebay by providing a means to sluice sediment and debris downstream and back into the river, which will limit the need for regular intervention of heavy equipment or maintenance crews entering the river.
- Frequently mobilize and convey sediment from upstream of the roughened channel crest to downstream of the Project site thereby improving sediment continuity through the Project reach.

4.4.2 Sluiceway Design Summary

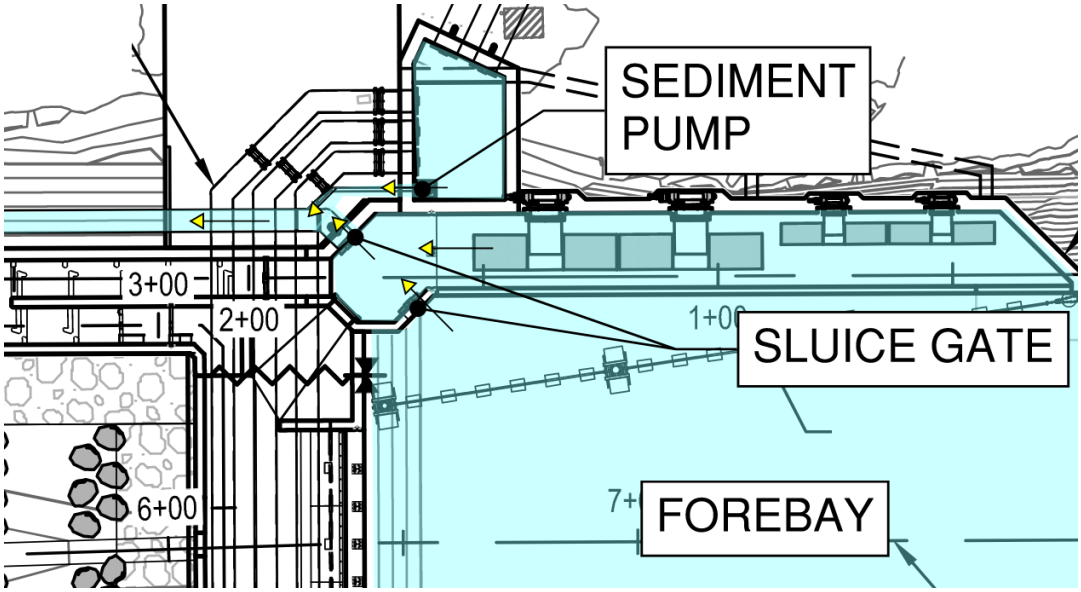
The main sluiceway will be composed of a 54-inch-diameter concrete pipeline beginning in the intake structure and will terminate at a headwall structure downstream of the roughened channel. It will be approximately 550 feet in length. There will be two different sluice gates that control flow into this pipeline (Figure 4-4).

One sluice gate will be located in the forebay at EL 110.0 feet NAVD88 and one in the intake at EL 106.5 feet NAVD88. The gates will be in the fully closed position nearly all of the time, except when sediment accumulation requires sluicing. The gates will be raised and flows in both the forebay and screen channel will sluice sediment through the 54-inch-diameter sluice pipeline. A wall at height 114.5 feet will separate the forebay from the screen channel, which is why the two will be sluiced separately.

Within the distribution structure, there will be a submersible pump that allows sediment to be pumped from within the distribution structure to the 54-inch sluice pipeline.

The 54-inch-diameter sluice pipeline will daylight at EL 104.0 feet NAVD88. A sluice pipeline outlet structure is required because the banks of the roughened channel have been supported with rock slope protection (RSP), and the headwall is required to support the pipeline. A small apron of RSP will also be located at the outlet to combat any erosive forces generated by the pipeline. The pipeline will daylight at the downstream end of the roughened channel to avoid damaging the weir boulder bands within the constructed channel, minimizing the need for maintenance of the channel.

Figure 4-4. Sluice Design



4.4.3 Preliminary Sluiceway Operations Strategy

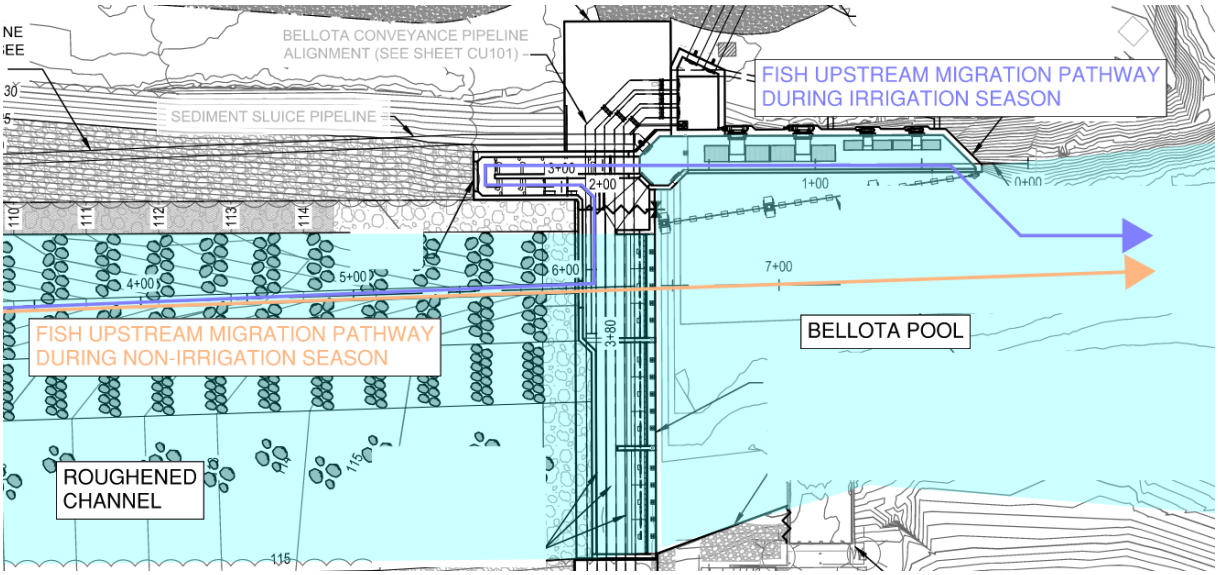
Sluicing events will be initiated to hydraulically transport sediment and debris downstream of the facility to reduce potential sediment accumulation impacts on the facility's ability to reliably divert water through the intake. The frequency of sluicing operations should be commensurate with the natural ability of the Calaveras River to mobilize and convey sediment downstream, but sluicing could happen more or less often as visual thresholds to trigger sediment sluicing are approached and event-driven opportunities to remove those sediments emerge. Sluicing will therefore be opportunistic and adaptive as the new facility is commissioned and lessons learned are gained over the first three to five years of operation.

Sluicing operations will be triggered by lifting the sluice gate to its fully raised position and initiating rapid flow through the sluiceway based on the overall differential from upstream to downstream of the Project extent. Sluicing periods will be terminated by lowering the weir gate to its normal (closed) operating position.

4.5 Fish Passage

Design of the fish ladder and roughened channel are described in the following subsections. As previously discussed, during irrigation season passage will be provided through dual use of the roughened channel and fish ladder, while during non-irrigation season passage will be provided solely through the roughened channel (Figure 4-5). Design of fish passage facilities will use the hydrologic information outlined in Section 3.2 of the Design Criteria Report (Appendix A), but does not take into account climate change at this time.

Figure 4-5. Fish Upstream Migration Pathways



The roughened channel and fish ladder components will be designed to different flow ranges as outlined in Table 4-2 and further discussed in following sections.

Table 4-2. Fish Ladder and Roughened Channel Design Flows

Time of Year	Fish Ladder Range of Design Flows (cfs)	Roughened Channel Range of Design Flows (cfs)	Resulting Overall Passage Design Flows (cfs)
Irrigation Season	20 – 202.3	20 – 1,735	20 – 202.3
Non-Irrigation Season	N/A	20 – 1,735	20 – 1,735

4.5.1 Fish Ladder Design

Hydraulic design of the fish ladder and fish screens will be performed in accordance with the design criteria outlined in the Design Criteria Report (Appendix A; HDR 2021). A summary of the hydraulic basis of design for these components is described in the following paragraphs.

Fish Ladder Design Flows

The basis of hydraulic design for the fish ladder will be defined by the anticipated fish passage flows selected for this Project. Analysis of gage data showed that the low and high fish passage design flows are 3 cfs and 1,735 cfs, respectively; see the Design Criteria Report in Appendix A for more detail.

The fish ladder will only be used during irrigation season, when the forebay WSE is 121.44 feet NAVD88. During this time, the 5 percent exceedance is 150.9 cfs (Table 4-3). The fish ladder will be conservatively designed for fish passage up to the 2 percent exceedance flow of 202.3 cfs during irrigation season (Table 4-3).

Table 4-3. Fish Passage Design Flows

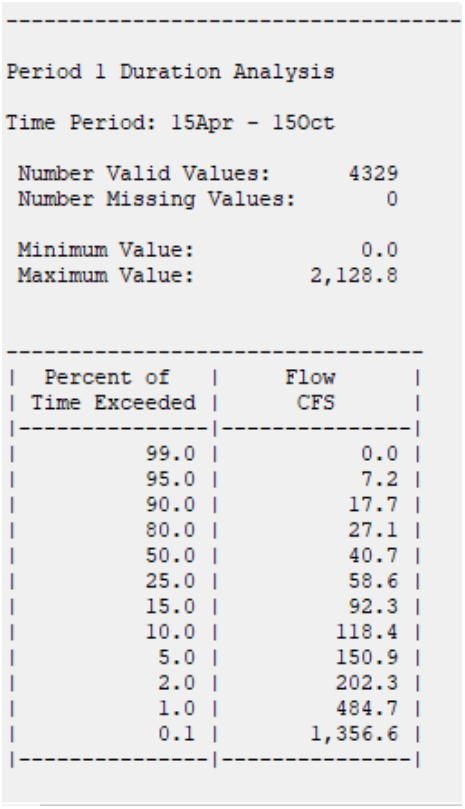
Scenario	Time period	Low Design Flow (cfs)	High Design Flow (Cfs)
		NMFS Adults (95% Exceedance)	**NMFS Adults (5% Exceedance)**
Annual	All year	**3***	545.3
Irrigation Season	April to Oct	7.2	**150.9**
Fall Chinook	Nov 1 to Dec 31	3*	420.6
Steelhead	Nov 1 to March 31	3*	1,728.6
Late Fall Chinook	Nov 1 to Feb 28	3*	**1,735.2**
Spring Chinook	March 1 to May 31	3*	908.1
Winter Chinook	Jan 1 to May 31	3*	1,710

*CDFG guidelines recommend that 3 cfs be selected for an alternative minimum design flow if the low flow is calculated to be less than 3 cfs.

As further described in Section 4.5.2, the roughened channel will be designed to pass fish beginning at a low flow of 20 cfs to meet minimum depth requirements. During irrigation season, the roughened channel and fish ladder will be used in combination (Figure 4-5), and an adult fish that unable to migrate past the roughened channel also will be unable to ascend to the fish ladder entrance. As a result, although the 95 percent exceedance flow is 3 cfs (Table 4-3), the fish ladder's low design flow will be 20 cfs (Table 4-2).

Figure 4-6 provides a summary of the design flows; the roughened channel design flows are further described in Section 4.5.2 and will accommodate all fish passage flow during non-irrigation season.

Figure 4-6. Irrigation Season Exceedance Flows



Period 1 Duration Analysis

Time Period: 15Apr - 15Oct

Number Valid Values:	4329
Number Missing Values:	0
Minimum Value:	0.0
Maximum Value:	2,128.8

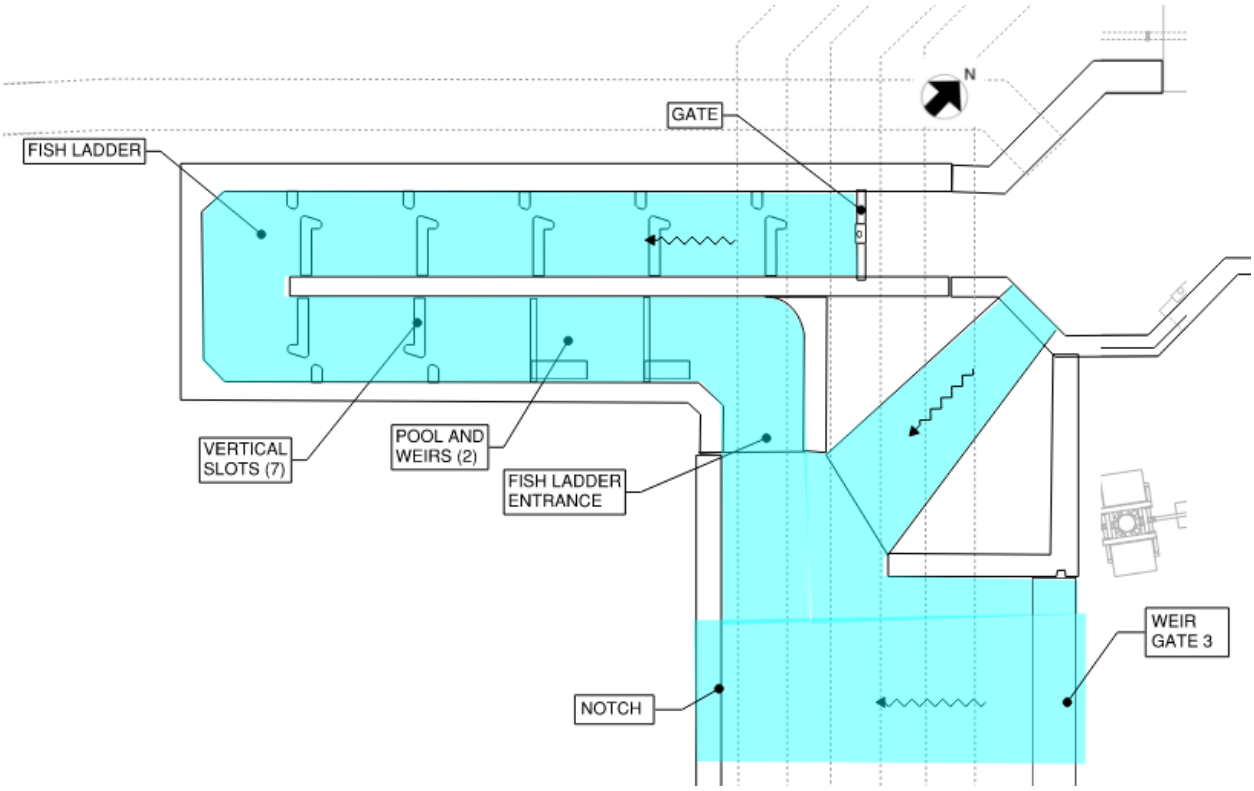
Percent of Time Exceeded	Flow CFS
99.0	0.0
95.0	7.2
90.0	17.7
80.0	27.1
50.0	40.7
25.0	58.6
15.0	92.3
10.0	118.4
5.0	150.9
2.0	202.3
1.0	484.7
0.1	1,356.6

The ladder will be designed for 20 cfs of flow at any given river flow event due to the reservoir condition held by the adjustable weir gates upstream during irrigation season. During irrigation season, the adjustable weir gates will remain at EL 121.44 feet NAVD88, and the pool elevation held with the fish ladder closed until maximum diversion demand is reached. Once the diversion demand has been met, the fish ladder will open. The ladder's normal operation will be 20 cfs of flow at any given river flow event. If river flow increases, it will be conveyed either through the overflow bypass or over the northernmost adjustable weir gate, Weir Gate 3 (Table 4-4). Table 4-4 also provides a summary of these conditions, while operating criteria is fully described in Section 3.

Table 4-4. Summary of Irrigation Season Flows to Mormon Slough

	Fish Ladder	Overflow Bypass	Weir Gate 3	Total Flow
Flow after Maximum Diversion Satisfied (cfs)	0-20	0	0	0-20
	20	0-50	0	20-70
	20	50	0+	70+

Figure 4-7. Fish Ladder Flow Pathways



The targeted attraction flow for the fish ladder will be a minimum of 10 percent of the high fish passage design flow for the channel of 202.3 cfs, equal to the ladder's design flow of 20 cfs (Table 4-2). As a result, no additional attraction flow for the ladder will be needed. Under operational conditions, flow from the overflow bypass may aid in providing attraction flow for the fish ladder entrance.

The total length of the fish ladder will be dictated by the maximum water surface differential from tailwater to forebay of 6 feet and a maximum pool-to-pool hydraulic differential criterion of 1 foot. Vertical slot hydraulic controls will be designed along a 50-foot length of the fish ladder near the upstream hydraulic inlet (fishway exit). Pool and weir controls will be proposed along the remaining 20-foot length of fishway to decrease velocity through the fish ladder. In general, as the Bellota forebay elevation and corresponding river flows fluctuate, the vertical slot portion of the fishway will distribute the total head differential in the upstream section of the fish ladder across seven vertical slot baffles into seven smaller head losses that provide navigable conditions for upstream migrating fish. Downstream of the final vertical slot baffle, water will be conveyed into a pool controlled by a pool and weir system. At the tailwater (fishway entrance), flow from the ladder will be conveyed to the roughened channel and ultimately to Mormon Slough.

Entrance pool hydraulics as well as the head loss between the upstream and entrance pools will be controlled by the stage versus discharge relationship that represents the tailwater conditions within Mormon Slough and the Calaveras River. A rating curve will be generated from the hydraulic modeling to calculate the tailwater conditions and

subsequent fish ladder conditions from a range of 20 cfs to 202.3 cfs. Appendix E provides detailed hydraulic calculations.

Pool and Weir Fish Ladder

The pool and weir portion of the fish ladder will be composed of two, 8-inch-thick weirs and corresponding pools that are distributed throughout the lower 20 feet of the fish ladder. The pool and weir portion will be included in the fish ladder to decrease velocity.

A trapezoidal notch present in the center of each weir crest will facilitate a minimum water depth of approximately 0.9 foot at a flow of 3 cfs. This will consolidate the attraction flow within the 1-foot notch to allow fish to easily find the weir entrance and provide future operational flexibility in case the fish ladder is ever required to operate at lower flows. At flows above 3 cfs, the remaining weir crest will be wetted. At the high fish ladder design flow of 202.3 cfs, the total water depth over the center of the low-flow notch is 2.0 feet and the depth over the remainder of the weir crest will be expected to be 1.1 feet. An approximately 2-foot-wide metal lamprey ramp will extend from the floor of each pool and embed into the top 4 inches of a corresponding 2-foot-wide portion on one side of the weir crest. The lamprey ramp for the entrance weir will be adjustable in height to accommodate the stoplogs.

The concrete upstream weir in the pool and weir portion of the fish ladder will act as the downstream hydraulic control for the vertical-slot portion of the fish ladder. The upstream weir will be designed to accommodate a vertical hydraulic drop of 1.0 foot or less. The crest of this weir will also be designed to provide a minimum water depth of approximately 3 feet throughout the vertical slot portion of the fish ladder and prevent the hydraulic head differential through the vertical slot ladder from exceeding 1.0 foot. The entrance weir will be designed to accommodate a vertical hydraulic drop between 1.0 and 1.5 feet through the range of target fish ladder bypass flows (20 cfs to 202.3 cfs; NMFS 2011). The entrance weir will also be adjustable by using stoplogs to accommodate the range of conditions and provide optimal entrance conditions.

In the bottom corner of the upstream weir, there will be a 12-inch-wide by 15-inch-tall opening controlled by a stainless steel fabricated stop gate. The opening and gate will provide a means for draining the fish ladder if or when necessary. The opening will not be present in the downstream weir as it will be unnecessary because stoplogs will allow the weir to drain. The opening and stoplogs could also be used for sluicing any small debris or sediment accumulations back out of the ladder during periodic maintenance activities.

Vertical Slot Fish Ladder

The vertical slot portion of the fish ladder will be composed of seven, 1-foot-thick vertical slot baffles that are distributed throughout the upper 50 feet of the fish ladder. The interior width of the vertical slot portion of the fish ladder will be 8 feet with a 10-foot length between baffles. A 20-foot-long turning pool will be provided. As a system of hydraulic control features, the vertical slot baffles will distribute the total hydraulic differential between the pool and weir portion of the fish ladder and forebay WSE. As the river flow ranges between 20 cfs and 202.3 cfs, the drop across each vertical slot baffle

will be between 0.3 and 0.8 foot at each slot. The fish ladder hydraulic inlet elevation will be designed to facilitate entry of 20 cfs into the fish ladder at a forebay WSE of 121.44 feet NAVD88, which is the target operational forebay elevation for this facility during irrigation season.

The ladder will be designed for slot widths of 1 foot (NMFS 2011). Lamprey passage will be accommodated using rounded corners at each baffle slot opening.

Flow will be measured across the vertical slot fish ladder using two ultrasonic level sensors located in pools directly adjacent to each other paired with a rating curve to be determined at a future point in the design.

4.5.2 Roughened Channel Design

Design Overview

Two-dimensional hydraulic modeling results were used to inform design of the proposed roughened channel. Through an iterative process, the slope, shape, and configuration of the roughened channel were developed to meet the Project fish passage hydraulic objectives. The roughened channel material, substrata, thickness, and rock elements were also developed, primarily with hydraulic modeling results.

The roughened-channel fishway and channel spanning weir crest will be designed to conform to several performance and engineering criteria. Engineering criteria will target the overall stability and resilience of the structure. Performance criteria will target the biological performance and periodic costs associated with maximum acceptable damages that require modification or repairs over time. The various criteria and performance targets used to establish the overall design of this feature are detailed in the Design Criteria Report (Appendix A). Below are some of the key criteria:

- The structures (weir, channels, and high flow sections) will be designed to convey and withstand flood events up to the 100-year flood event (12,690 cfs).
- Limited damage that may occur during low frequency events (high flows) will not impede the structures' ability to achieve fish passage or surface water withdrawal performance criteria.
- Damage will be associated with surface scour of rock matrix, including loss of fine materials and minor degradation of tailwater control (i.e., facility will be designed such that periodic adjustments in surface composition do not inhibit irrigation supply or fish passage, or negatively affect the Escalon Bellota Road bridge).
- Dislodging of large structural rocks will be limited to small areas where storm events create conditions that cause isolated areas of instability.
- Accumulation and removal of finer sediments and cobbles on the surface layer above the armor layer will be transitional and vary over time.
- The structure configuration and composition will be designed to meet the Project's fish passage performance objectives.

- Fish passage performance objectives of the roughened channel will be continuous and met throughout the range of anticipated migration flows identified for the various target species and life stages.

Design Flow

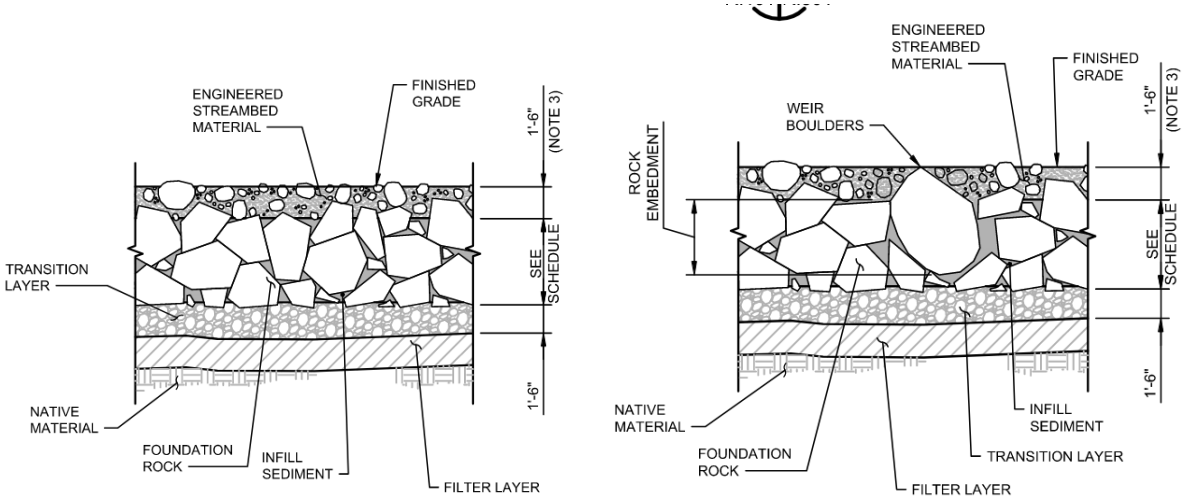
As previously mentioned, the slope, shape, and configuration of the roughened channel will be developed to meet the Project fish passage hydraulic objectives. These hydraulic objectives will be defined by depth and velocity criteria for a range of defined fish passage design discharges. Given the range of fish passage design discharges and the magnitude of flood discharges, a roughened ramp will not be suitable to meet fish passage hydraulic criteria at flows as low as 3 cfs. As previously discussed, the low flow section will be sized to meet fish passage criteria for 20 cfs. The roughened ramp section and weir sill will be configured to meet the high fish passage design discharge of 1,735 cfs. Additionally, the structural rocks and embankment protection will be designed for the 100-year discharge of 12,690 cfs.

Roughened Channel Stability

The roughened channel slope, shape, and configuration must remain intact to function as intended. If the material comprising the roughened ramp is displaced or mobilized in an unintended manner, the hydraulic performance may be degraded and could require excessive maintenance. There are three primary mechanisms for degradation of the roughened ramp. First, the foundation rock, which constitutes the primary material of the channel, is displaced from the surface due to high flow hydraulics. This could affect the shape, configuration, and grade. Second, the underlaying layer, consisting of the natural streambed or placed fill, can be degraded by the winnowing or piping of fines up through the structural rock. This loss of substrate can lead to settling of the roughened channel, affecting the shape and grade. Finally, degradation at the downstream end, due to a variety of potential scour mechanisms, could cause mobilization of the roughened channel material and/or result in a perched condition not conducive to swim-through migration.

To guard against these three degradation mechanisms and support desired passage hydraulics, the typical roughened channel section will be designed to include four individual layers installed above existing riverbed material: filter layer, transition layer, structural foundation rock layer, and engineered streambed material layer composed of sediment and rounded river cobbles (Figure 4-8).

Figure 4-8. Typical Rock Layering Details for the Roughened Channel



The left detail shows the typical layering scheme, while the right illustrates embedment of boulders used in boulder weirs and boulder clusters to promote hydraulic roughness in the primary and secondary fish passage channels.

The filter and transition layers will consist of two layers of fines, sands, and gravels to prevent upward migration of the underlying native or fill material. These layers will bridge the gap between the size of the underlying material and structural rock.

The foundation rock will constitute the primary material of the roughened channel and be sized large enough to remain stable up to the 100-year flow event. Infill material will fill the interstitial spaces between the large rock elements of the foundation rock to encourage stability of the foundation rock and discourage material loss below this layer. The foundation rock will be angular, not rounded, to provide better interlocking of the rock elements.

An engineered streambed material layer composed of sediment and cobbles will be added above the foundation rock layer to provide material roughness and aesthetics more consistent with the natural river material. The engineered streambed material layer is anticipated to actively aggrade or degrade non-uniformly across the structure based on the flow magnitude, duration, and variability of the natural hydrology within the Calaveras River.

In addition to these primary layers, boulder weirs and boulder clusters will be embedded into the foundation rock to increase roughness and help define the channel configuration as needed to support desirable hydraulic conditions. These boulders will be rounded and at least 50 percent of their nominal diameter must be embedded to remain stable. These boulders also will need to be large enough to protrude sufficiently above the engineered streambed material layer to achieve the desired effect. This will result in a size requirement larger than the foundation rock, which helps contribute to the stability of the channel.

Rock and Layer Sizing

Table 4-5 provides the recommended boulder weir and rock cluster sizing, as well as general details about the filter layer, transition layer, infill sediment, and engineered streambed material. A description of each layer's material and size is included below.

Table 4-5. Summary of Boulder Sizes

Project Area	Description/Recommendation
Weir Boulder Bands B and C per the drawings	• Maximum 2.5 feet of protrusion from the bed • Between 9 and 15 tons • Ranging from 60 to 72 inches • Falls within D_{100} range specified for NCHRP Class VIII and Class IX
Weir Boulder Band A per the drawings	• Between 4 and 6 tons • Ranging from 48 to 54 inches • Falls within D_{100} range specified for NCHRP Class VII and Class VIII
Rock Clusters	• Large enough to protrude from the bed by 1 to 2 feet • Between 4 and 6 tons • Ranging from 48 to 54 inches • Falls within D_{100} range specified for NCHRP Class VII and Class VIII

NCHRP = National Cooperative Highway Research Program

Weir Boulders and Rock Clusters

Boulders will be used to construct the 16 weir structures within the main channel and the 16 rock cluster features along the overbank. The boulder weirs will help to create passable depths and velocities throughout the main channel, and the rock clusters will provide regions of velocity refugia within the overbank. Boulder sizes within the weir structures and rock clusters will vary based on the protrusion height (maximum of 2.5 feet) needed to create the desired hydraulic effect and desired depth of embedment (one half of the long axis or greater). Boulder sizes are summarized in Table 4-5.

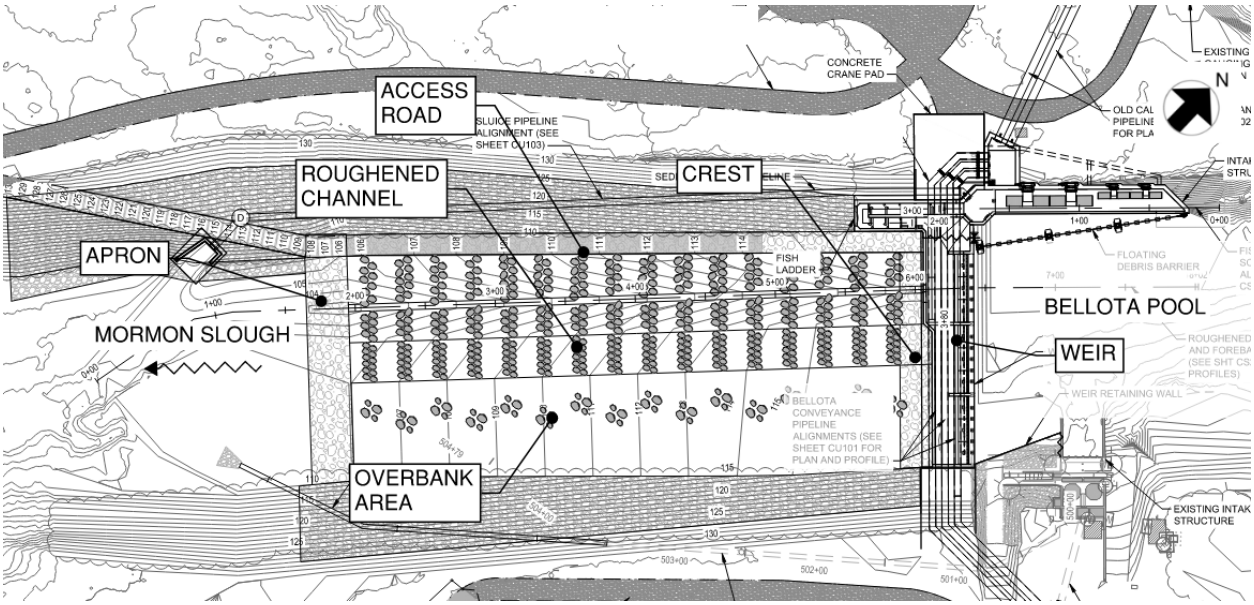
Foundation Rock Sizing

Foundation rock sizing will be based on approaches described in technical literature and detailed in the Hydraulic Modeling Summary Report (Appendix B). Extensive analysis of different flow events (10-, 25-, 50-, and 100-year) were evaluated and compared. Hydraulic characteristics of unit discharge, hydraulic gradient, energy grade slope, and velocity were reviewed to determine the need for location-specific rock sizing. It was concluded that the hydraulic characteristics were uniform enough under the controlling event (100-year) that foundational rock will be comprised of a single riprap gradation for the entire channel. This includes the access road, crest, apron, and entire roughened channel areas. Infill sediment will be used to fill voids within the foundation rock layer; the gradation for this material is the same as the gradation for the sediment portion of the engineered streambed material (Table 4-6).

Table 4-6. Proposed Foundation Rock Gradation and Layer Thickness

Sub-Region	Nom. D_{50} (in)	D_{15} (in)		D_{50} (in)		D_{85} (in)		D_{100} (in)	Blanket Thickness (in)	Weight Class	Description
		Min	**Max**	**Min**	**Max**	**Min**	**Max**				
Access Road, Crest, Channel, Overbank	21	13	18.5	20	24	27.5	32.5	42	60	3/8-ton	NCHRP Class VI
Apron	21	13	18.5	20	24	27.5	32.5	42	108	3/8-ton	NCHRP Class VI

Figure 4-9. Roughened Channel Foundation Rock Layer Zones



Engineered Streambed Material

Design of the engineered streambed material was based on approaches described in technical literature and is detailed in the Hydraulic Modeling Summary Report (Appendix B). Hydraulic characteristics of unit discharge, hydraulic gradient, and energy grade slope were reviewed to determine the appropriate gradation. Of the different flow events analyzed (10-, 25-, 50-, and 100-year), the 100-year discharge produced the most conservative hydraulic characteristics and was used to develop the proposed gradation. The resultant gradation was broken into a 1:1 mixture of two components: streambed sediment and streambed cobbles. Gradations for these components were based on the Washington State Department of Transportation standard gradations for streambed aggregates (Table 4-7 and Table 4-8).

Table 4-7. Engineered Streambed Material – Streambed Sediment Gradation

Size Diameter	Percent Passing
2.5-inch	100
2.0-inch	80
1.5-inch	73
1.0-inch	65
0.75-inch	50
0.187-inch	35
No. 40	16
No. 200	7

Table 4-8. Engineered Streambed Material – Streambed Cobbles Gradation

Size Diameter	Percent Passing
18-inch	100
15-inch	75
12-inch	50
10-inch	40
8-inch	34
6-inch	28
5-inch	23
4-inch	20
3-inch	17
2.5-inch	14
2-inch	11
1-inch	5.4

Filter and Transition Layer Material

A bedding (transition) layer material for the roughened channel foundation rock and a filter layer between the transition material and the native sands and gravels forming the roughened channel fill were designed to prevent internal erosion and piping of the channel fill. Design of the filter layer and transition layer materials are described in Section 6.3. A summary of the selected gradations are provided in Table 4-9.

Table 4-9. Transition and Filter Layer Material Gradations

Size Diameter	% Passing (Transition Material)	Size Diameter	% Passing (Filter Material)
2 ½-inch	100	2-inch	100
2-inch	80	1-inch	97 - 100
1 ¼-inch	65	½-inch	55 - 76
¾ -inch	50	#4	34 - 46

Table 4-9. Transition and Filter Layer Material Gradations

Size Diameter	% Passing (Transition Material)	Size Diameter	% Passing (Filter Material)
#4	16	#8	4 - 19
#8	10	#16	0 - 4
#18	5	#50	0 - 2
#200	3	#200	0

Roughened Channel Fish Passage Performance

Fish passage performance at the Project site is evaluated using a biometric comparison of the velocity predicted from two-dimensional (2D) modeling results and fish swimming capability using the flow velocity versus fish swimming distance relationships discussed and established in the Design Criteria Report (Appendix A). Refer to Appendix B for 2D modeling methods and results. Table 4-10 summarizes the swimming metrics used to establish fish passage performance for the Project. Metrics are modified to reflect example sub-adult fish near 250 mm (10 inches) and adult fish near 710 mm (28 inches) for comparison.

Table 4-10. Swimming Speeds for 250-mm, 1,000-mm, and 710-mm Salmonids for 75th Percentile

Fish Length	Time	Water Velocity (fps)	Swim Distance (ft)	Correlated Swim Speed (fps)
250mm[a] (10 inches)	5 seconds	9.5	16.4	12.8
	20 seconds	6.9	45.9	9.2
	3 minutes	4.3	229.6	5.5
	30 minutes	2.1	1,312.0	2.9
710mm[b] (28 inches)	5 seconds	18.0	30.5	24.1
	20 seconds	12.7	86.2	17.0
	3 minutes	7.3	450.9	9.8
	30 minutes	4.0	2,418.5	5.4

[a] Obtained from Katopodis and Gervais (2016), Appendix E, Charts J-5b and J-5c.
[b] Interpolated from Katopodis and Gervais (2016) data using dimensionless length ratio.

Depth and velocity results from the 2D modeling efforts were used to compare proposed depths and velocities with fish passage performance targets established for the Project. Comparisons of existing and proposed velocity results for juvenile salmonid passage

(flow depth equal to 0.4 feet or greater) are presented in Figure 4-10 through Figure 4-13. These results are grouped into four colored velocity zones:

Juvenile Passage Zone 1 (Green; 0 – 2.1 fps): Zone 1 includes corridors in which fish can travel unimpeded. Results indicate that juvenile passage through these zones could last up to 30 minutes for up to approximately 1,300 feet. This low velocity zone represents resting areas and areas that require lower energy expenditure over time.

Juvenile Passage Zone 2 (Light green; 2.1 – 4.3 fps): Zone 2 is characterized by corridors in which fish can travel approximately 3 minutes or 230 feet. This velocity zone fosters adequate conditions for sub-adult movement upstream for moderate distances but requires the need for low velocity areas (created by channel shape, structures, boulders, etc.) periodically within the Project footprint.

Juvenile Passage Zone 3 (Light blue; 4.3 – 6.9 fps): Zone 3 is characterized by velocity corridors where juvenile salmonids could swim for approximately 20 seconds and 45 feet before requiring areas of low velocity for rest. This velocity zone does not promote a consistent navigational corridor without the frequent (within 45 feet of one another) presence of velocity refugia created by channel shape, structures, boulders, etc.

Juvenile Passage Zone 4 (Dark blue; 6.9 – 9.5 fps): Zone 4 is made up of areas with higher velocities, where a greater number of velocity refugia are required to navigate for larger distances within this zone. Juvenile salmonids are only expected to be able to traverse approximately 16 feet, or 5 seconds. This velocity zone does not promote a consistent navigational corridor without the frequent (within 16 feet of one another) presence of velocity refugia created by channel shape, structures, boulders, etc.

Figure 4-10. Proposed Juvenile Salmonid Velocity Results at 20 cfs River Flow

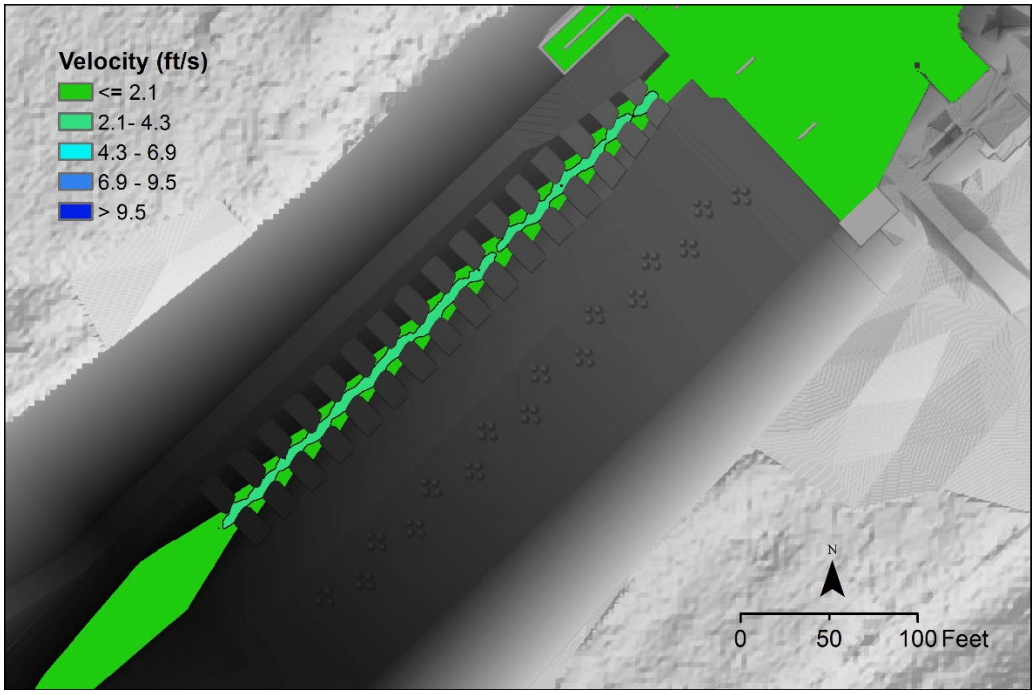


Figure 4-11. Proposed Juvenile Salmonid Velocity Results at 150.9 cfs River Flow

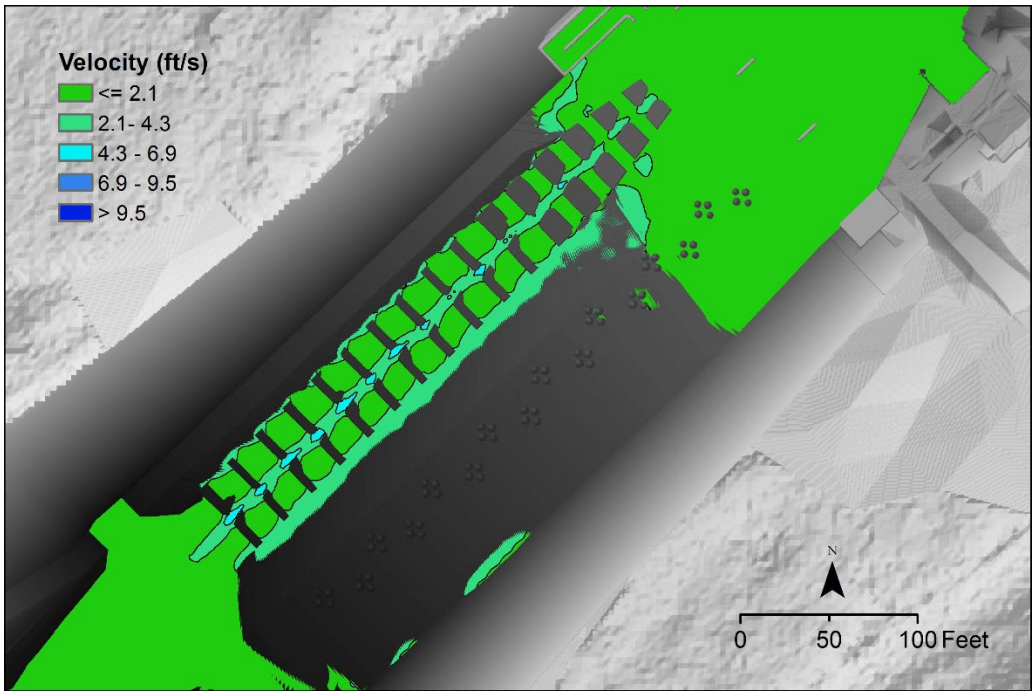


Figure 4-12. Proposed Juvenile Salmonid Velocity Results at 534 cfs River Flow

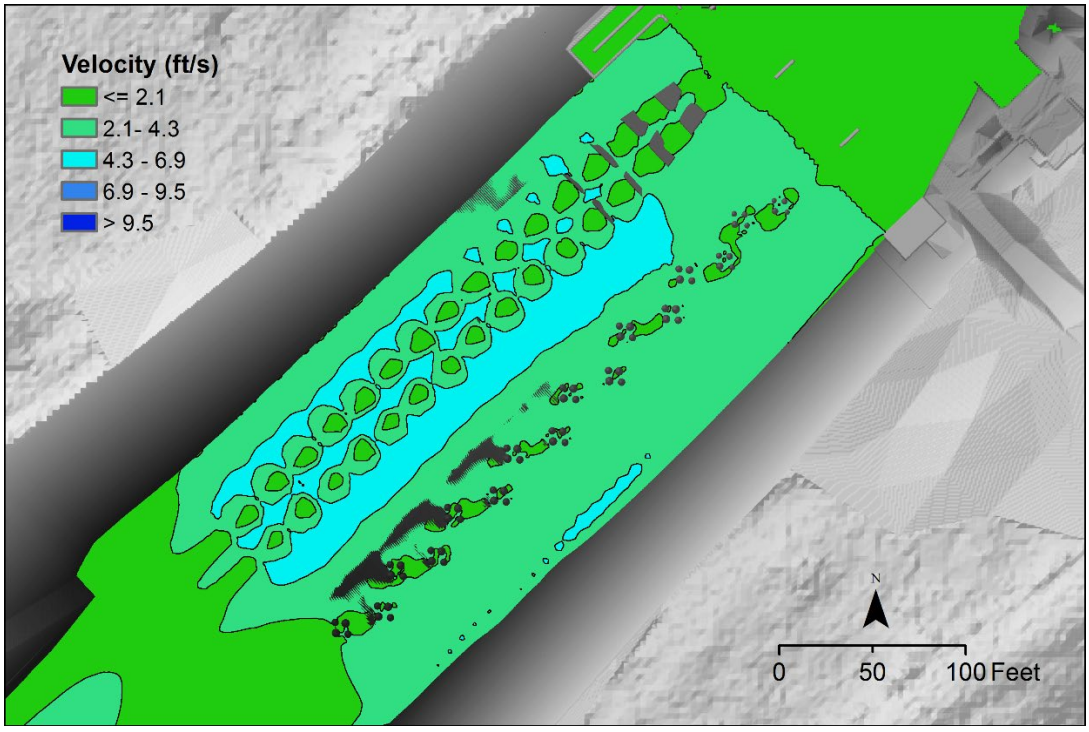
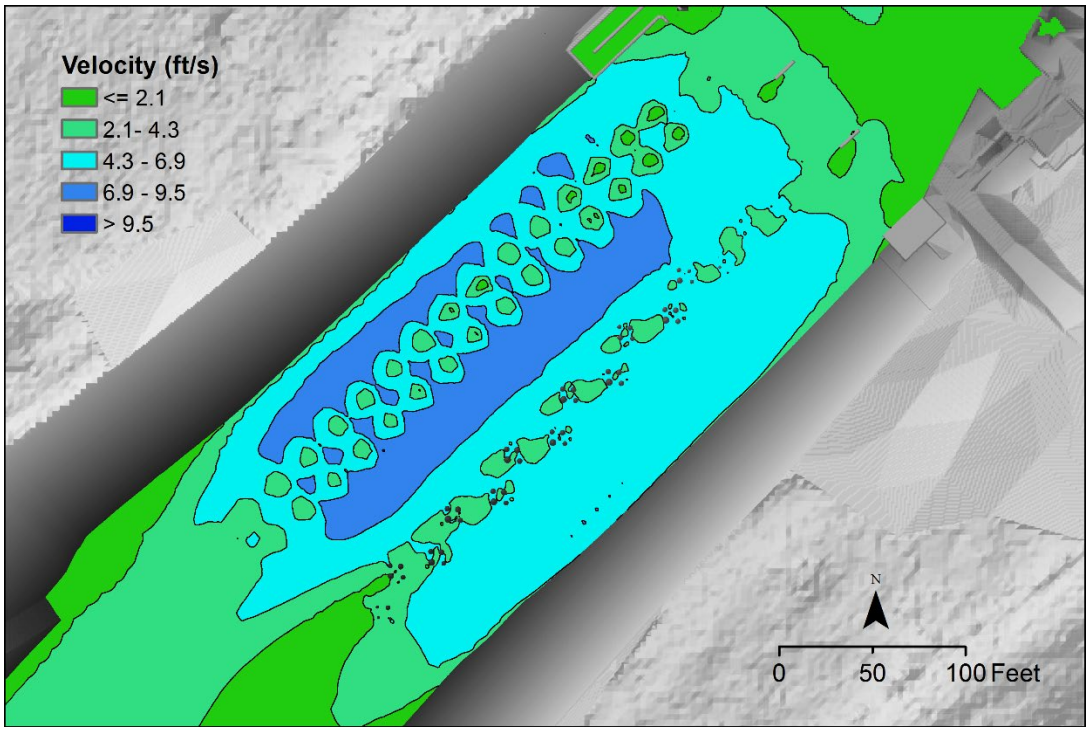


Figure 4-13. Proposed Juvenile Salmonid Velocity Results at 1735.2 cfs River Flow



Results indicate passage conditions through the roughened channel footprint and over the proposed weir sill. These hydraulic pathways present opportunities for juvenile and sub-adult fish to navigate upstream through the Project reach. The different navigational pathways and migration opportunities vary across the range of anticipated fish passage flows and are summarized in the following paragraphs. Table 4-11 describes a range of four flow conditions used to assess fish passage performance. Colored areas on the figures indicate velocity conditions at areas where the depth criterion of 0.4 feet is met. Shallower, lower velocity areas are not shown and extend beyond the colored velocity zones presented.

Table 4-11. Roughened Channel Fish Passage Performance Discharges

Description	Discharge (cfs)	Season	Percent Exceedance
Low Fish Passage Design Discharge[a]	20	N/A[b]	N/A[b]
5 percent Irrigation Season Exceedance	150.9	Irrigation mid-April to mid-Oct	5 percent

Table 4-11. Roughened Channel Fish Passage Performance Discharges

Description	Discharge (cfs)	Season	Percent Exceedance
5 percent Annual Exceedance	534	Annual	5 percent
High Fish Passage Design Discharge[a]	1,735.2	Late fall Chinook Migration Nov 1 to Feb 28	2.9

Notes:
[a] Low flow and high flow fish passage design flows are defined as the 95 percent and 5 percent exceedance value for the specified season, respectively.
[b] Low flow passage design discharge is limited to a minimum of 20 cfs due to the constraints of a roughened channel concept.

At 20 cfs (Figure 4-10), approach conditions on the downstream side of the roughened channel reflect flow velocities between 0.0 to 2.1 fps (Zone 1) where passage is unimpeded. Through the main low flow channel, velocities on the fringe of the rock bands provide flow velocities ranging from 2.1 to 4.3 fps (Zone 2) with interspersed pockets in the lower range of 0.0 to 2.1 fps. Flow velocities return to 0.0 to 2.1 fps (Zone 1) upstream of the roughened channel.

At 150.9 cfs (Figure 4-11), flow velocities ranging from 2.1 to 6.9 fps (Zones 2 and 3) are shown through the main low flow channel. In both reaches downstream and upstream of the main channel improvements, flow velocities are between 0.0 to 2.1 fps (Zone 1).

At 534 cfs (Figure 4-12), flow velocities ranging from 2.1 to 6.9 fps (Zones 2 and 3) are shown through the main low flow channel. The access road and overbank regions of the roughened channel are both inundated and meet the depth threshold for juvenile fish. The access road has flow velocities between 2.1 and 6.9 fps (Zones 2 and 3). The overbank area has relatively uniform velocities between 2.1 and 4.3 fps (Zone 2), with isolated pockets of velocities between 4.3 and 6.9 fps (Zone 3) along the south bank. In both reaches downstream and upstream of the main channel improvements, flow velocities are between 0.0 to 2.1 fps (Zone 1), where some velocities between 2.1 and 4.3 fps (Zone 2) exist downstream.

At 1,735.2 cfs (Figure 4-13), the roughened channel is fully inundated, except for the upper most portions of the rock weirs and clusters. The access road and overbank areas generally exhibit velocities from 4.3 to 6.9 fps (Zones 3 and 4), with pockets of slower velocities, ranging from 2.1 to 4.3 fps (Zone 2) between rock clusters. In the main passage channel, velocities range from 4.3 to 9.5 fps (Zones 2, 3, and 4). Velocities along the fringe of the main low flow channel generally range from 6.9 to 9.5 fps (Zone 3); velocities within this range are also observed intermittently along center of the main channel. Slower velocities, less than 4.3 fps (Zones 1 and 2), are present between boulder weirs. Rest zones where velocities are less than or equal to 2.1 fps are limited to regions between boulder weirs in the upper 200 feet of the ramp. Passage approach velocities downstream of the roughened channel are less than 4.3 fps (Zones 1 and 2). The reaches upstream and downstream of the main channel improvements exhibit similar flow velocities of 2.1 to 4.3 fps (Zones 1 and 2).

Similarly, velocity results for adult salmonid passage (flow depth equal to 0.9 foot or less) are presented in Figure 4-14 through Figure 4-17. These results are grouped into four velocity zones:

Adult Passage Zone 1 (Green; 0 – 4.0 fps): Zone 1 includes corridors in which fish can travel unimpeded. Adult passage through these zones could last for up to 30 minutes and approximately 2,400 feet.

Adult Passage Zone 2 (Light green; 4.0 – 7.3 fps): Zone 2 is characterized by corridors in which fish can travel approximately 3 minutes or 450 feet before requiring a low velocity area (created by channel shape, structures, boulders, etc.)

Adult Passage Zone 3 (Light blue; 7.3 – 12.7 fps): Zone 3 is characterized by velocity corridors where adult salmonids could swim for approximately 20 seconds and 85 feet before requiring a low velocity area to rest.

Adult Passage Zone 4 (Dark blue; 12.7 – 18.0 fps): Zone 4 is made up of areas with higher velocities, where more velocity refugia is required for larger distances within this zone. Adult salmonids are only expected to be able to traverse for approximately 5 seconds and 30 feet.

Figure 4-14. Proposed Adult Salmonid Velocity Results at 20 cfs River Flow

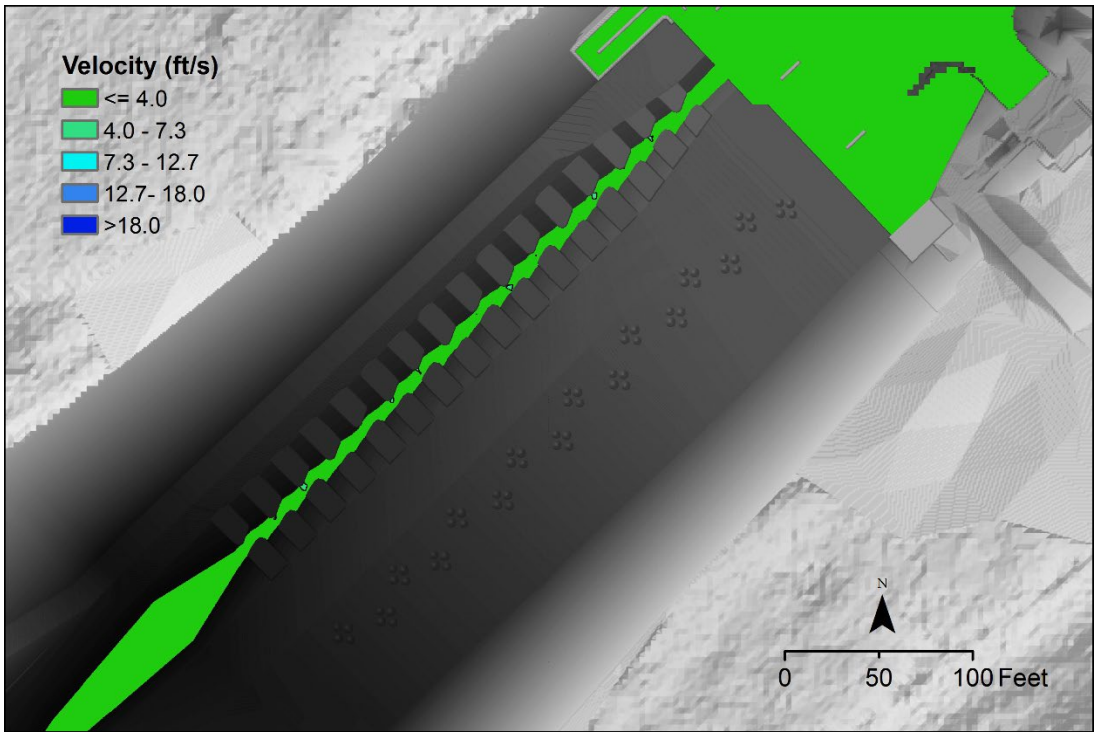


Figure 4-15. Proposed Adult Salmonid Velocity Results at 150.9 cfs River Flow

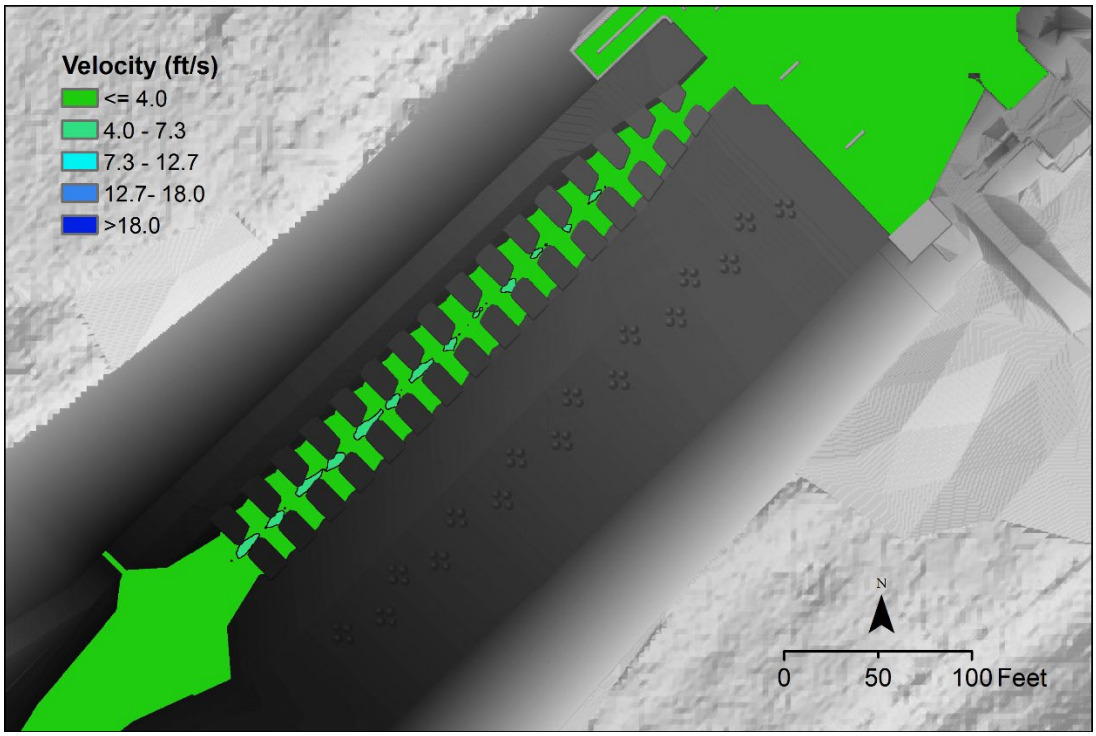


Figure 4-16. Proposed Adult Salmonid Velocity Results at 534 cfs River Flow

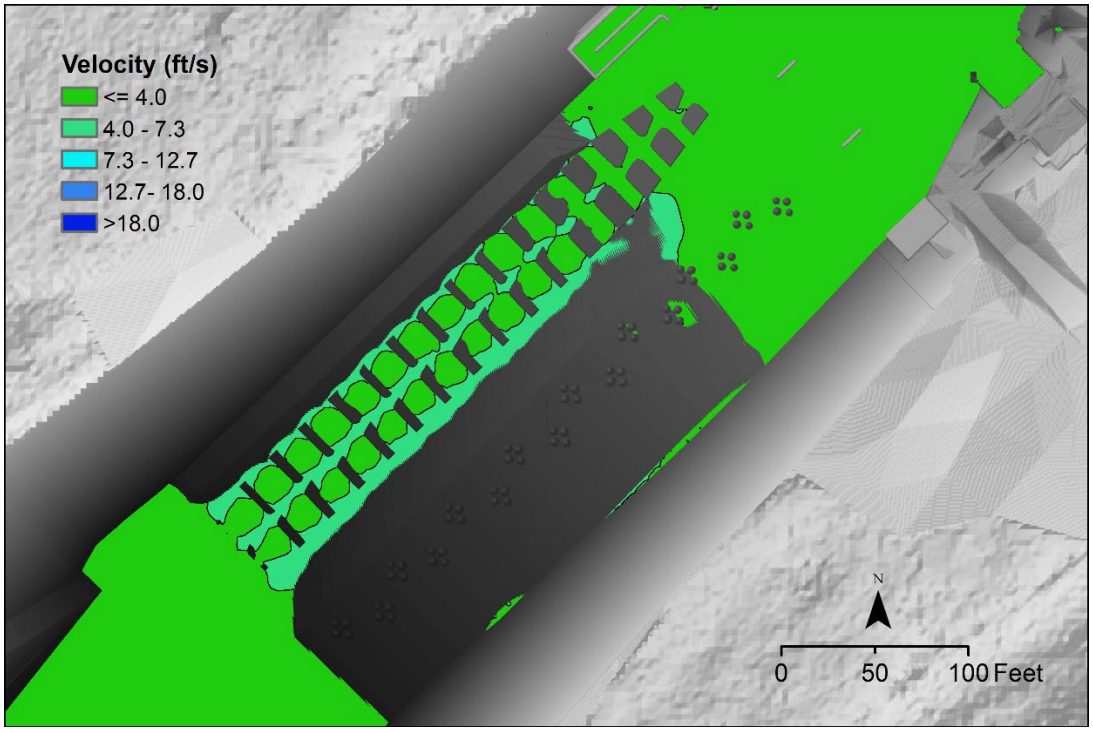
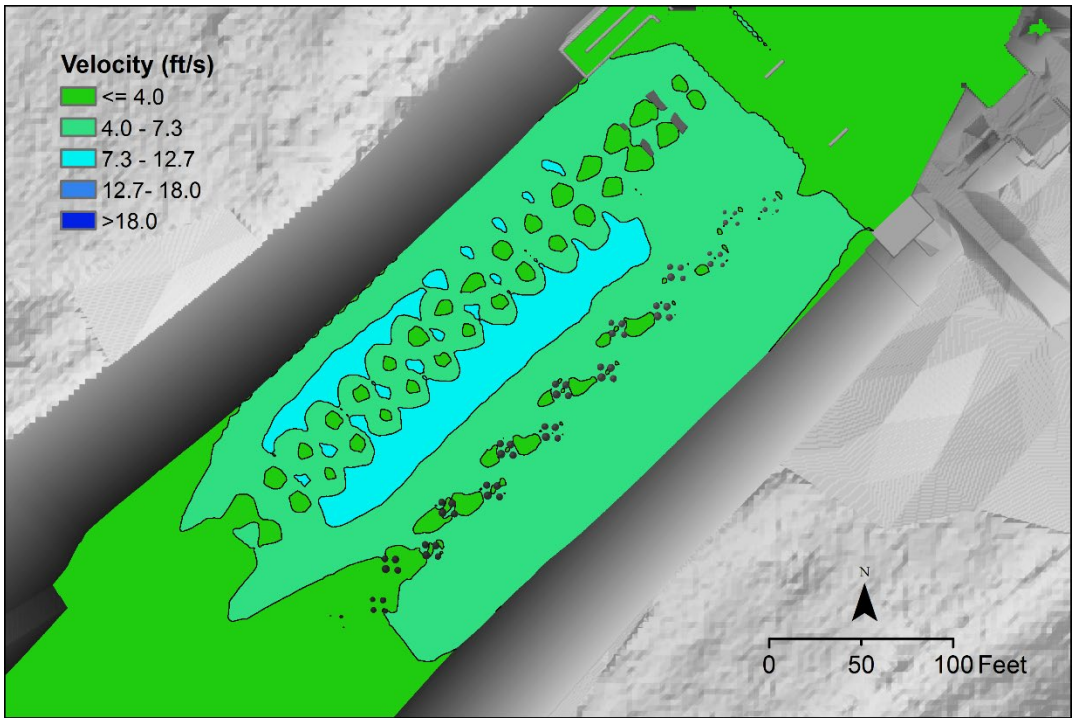


Figure 4-17. Proposed Adult Salmonid Velocity Results at 1,735.2 cfs River Flow



Similar to the results presented for sub-adult fish passage, hydraulic pathways for adult fish to navigate upstream through the Project reach exist for the full range of fish passage flows. The different navigational pathways and migration opportunities vary across the range of anticipated fish passage flows and are summarized in the following paragraphs. Colored areas on the figures indicate velocity conditions at areas where the depth criterion (0.9 foot) is met. Shallower, lower velocity areas are not shown and extend beyond the colored velocity zones presented.

At 20 cfs (Figure 4-14), continuous passage is available through the main low flow channel. Velocities through the main channel generally range from 0 to 4 fps (Zone 1), with isolated pockets of velocities ranging from 4 to 7.3 fps (Zone 2). Approach conditions on the downstream side of the roughened channel reflect flow velocities between 0 to 4 fps (Zone 1) where passage is unimpeded. Flow velocities also range from 0.0 to 4.0 fps (Zone 1) upstream of the roughened channel.

At 150.9 cfs (Figure 4-15), continuous passage is available via the main low flow channel where flow velocities range from less than 4 to 7.3 fps (Zones 1 and 2); velocities between boulder weirs are less than or equal to 4 fps (Zone 1). The overbank region of the roughened channel is wetted; however, flow depths are not sufficient to provide a continuous migration path. Similarly, portions of the access road are inundated, but flow depths do not yet support a viable swim path along its entire length. In both reaches downstream and upstream of the main channel improvements, flow velocities are between 0 to 4 fps (Zone 1).

At 534 cfs (Figure 4-16), continuous passage is available via the main low flow channel. Flow velocities down the center and along the fringes of the main channel generally

range from 4 to 7.3 fps (Zones 2), with velocities less than or equal to 4 fps (Zone 1) at the upstream end of the channel and between boulder weirs. The overbank region and access road of the roughened channel are wetted; however, flow depths are not sufficient to provide a continuous migration path. Passage approach velocities downstream of the roughened channel are less than or equal to 7.3 fps (Zones 1 and 2). The reach upstream of the main channel improvements exhibits flow velocities in the range of 0 to 4 fps (Zone 1).

At 1,735.2 cfs (Figure 4-17), the roughened channel is fully inundated, except for the upper most portions of the rock weirs and clusters. The access road and overbank areas exhibit velocities from 4 to 7.3 fps (Zones 2 and 3). Adult fish could pass upstream through these areas without rest, however rest is available between the rock weirs, where velocities range from less than 4 to 7.3 fps (Zones 1 and 2). Continuous stretches of velocities ranging from 7.3 to 12.7 fps (Zone 3) exist along the fringe of the main passage channel, as well as in isolated pockets down the center of the channel. Velocities in the overbank generally range from 4 to 7.3 fps (Zone 2), with areas of velocities less than or equal to 4 fps (Zone 1) between rock clusters. Passage approach velocities downstream of the roughened channel are less than or equal to 7.3 fps (Zones 1 and 2). The reach immediately upstream of the roughened channel exhibits flow velocities less than 4 fps (Zone 1).

4.5.3 Rock Slope Protection Design

Rock riprap will be proposed to protect the roughened channel slopes, upstream limit of intake structure, and Old Calaveras outlet structure from erosion. Riprap design will be based on approaches described in technical literature and is detailed in the Hydraulic Modeling Summary Report (Appendix B). Hydraulic characteristics of velocity and depth will be reviewed to determine the appropriate gradation. Of the different flow events analyzed (10-, 25-, 50-, and 100-year), the 100-year discharge produced the most conservative hydraulic characteristics and will be used to develop the proposed gradation. A summary of the selected riprap gradations is provided in Table 4-12.

Table 4-12. Proposed Riprap Protection Gradation and Layer Thickness

Location	Nom. D_{50} (in)	D_{15} (in)		D_{50} (in)		D_{85} (in)		D_{100} (in)	Blanket Thickness (in)	Weight Class	Description
		Min	**Max**	**Min**	**Max**	**Min**	**Max**				
Roughened Channel Slopes	18	11	15.5	17	20.5	23.5	27.5	36	36	1/4-ton	NCHRP Class V
Fish Screen Intake Wall, Old Calaveras River Outlet Structure	12	7.3	10.5	11.5	14.0	15.5	18.5	24.0	24.0	150 lb	NCHRP Class III

Filter and transition layers will also be designed for the rock slope protection. The filter layer described in Table 4-9 was found to be adequate for the rock slope protection. The transition layer will be a geotextile fabric with an apparent opening size of 0.2 mm (#70 sieve), placed over the filter layer and under the riprap (see Appendix C for calculations).

4.5.4 Scour Prediction and Impact Assessment

A range of scour mechanisms was evaluated at select locations of the proposed project to appropriately manage scour risk. These mechanisms included a range of local, general, and long-term degradation mechanisms.

Contraction, impingement, and bendway scour were evaluated for the fish screen intake wall, upstream of the intake, as it extends into the river channel. A total anticipated 100-year scour depth of about 12 feet was computed. Instead of accommodating this scour depth in the structural design, a riprap apron countermeasure was proposed to provide scour risk protection.

The downstream end of the roughened channel was studied in depth, as degradation at the downstream end of any channel spanning structure is a likely mode of failure. Downstream degradation and the localized effects of slope or channel material transitions can result in a scour hole capable of undermining the structure. Therefore, the downstream end will need to be designed to accommodate the depth of this potential scour. With an in-stream, rock structure, this is typically done with a toe-down, where rock is extended to the depth of anticipated scour to prevent undermining. Two scour mechanisms were evaluated: long-term degradation and plunging effects of an exposed toe. These were evaluated using a 1D sediment transport model based on the existing conditions 1D HEC-RAS model. Degradation of 5 feet was assessed at the toe. The modeling also indicated there is potential for headcuts to form downstream, which could migrate upstream. Therefore, a toe-down depth of 10 feet, will be determined and a thickened section of foundation rock at the downstream end of the roughened channel included in the design.

5 Civil Design

The Project's civil design features will focus on improving public works and altering the river channel for public safety, providing reliable water supply, and reducing environmental impacts to the aquatic ecosystem. Civil design elements that will be performed for the Project include:

- Establishing a Project survey control system
- Establishing demolition procedures and sequencing activities
- Developing a care of water plan to provide safe work areas and safe fish passage during in-water work activities
- Site grading, slope protection
- Providing an earthen embankment to promote fish exclusion to the Old Calaveras River
- Developing technical specifications associated with dewatering, earthwork, site restoration, and paving
- Other project elements such as safety and security as needed

5.1 Surveying

Project surveying uses horizontal coordinates referenced to the North American Datum of 1983 (2007.00 epoch), converted to the California Coordinate System of 1983, Zone 3, Survey Feet Units. Elevations are based on the California State Reference Network NAVD88 vertical datum. As-built records that used National Geodetic Vertical Datum of 1929 were converted to NAVD88 by adding 2.44 feet.

5.2 Demolition

The Project includes demolition and removal of several existing features prior to implementing the proposed elements. The demolition work will remove existing features that impair the construction or intended function of the intake or in-water work at the Project site. Overhead power and communication utility relocations will also be performed during these demolition activities. Existing facilities to be decommissioned and removed include the following, and are shown in Figure 5-1:

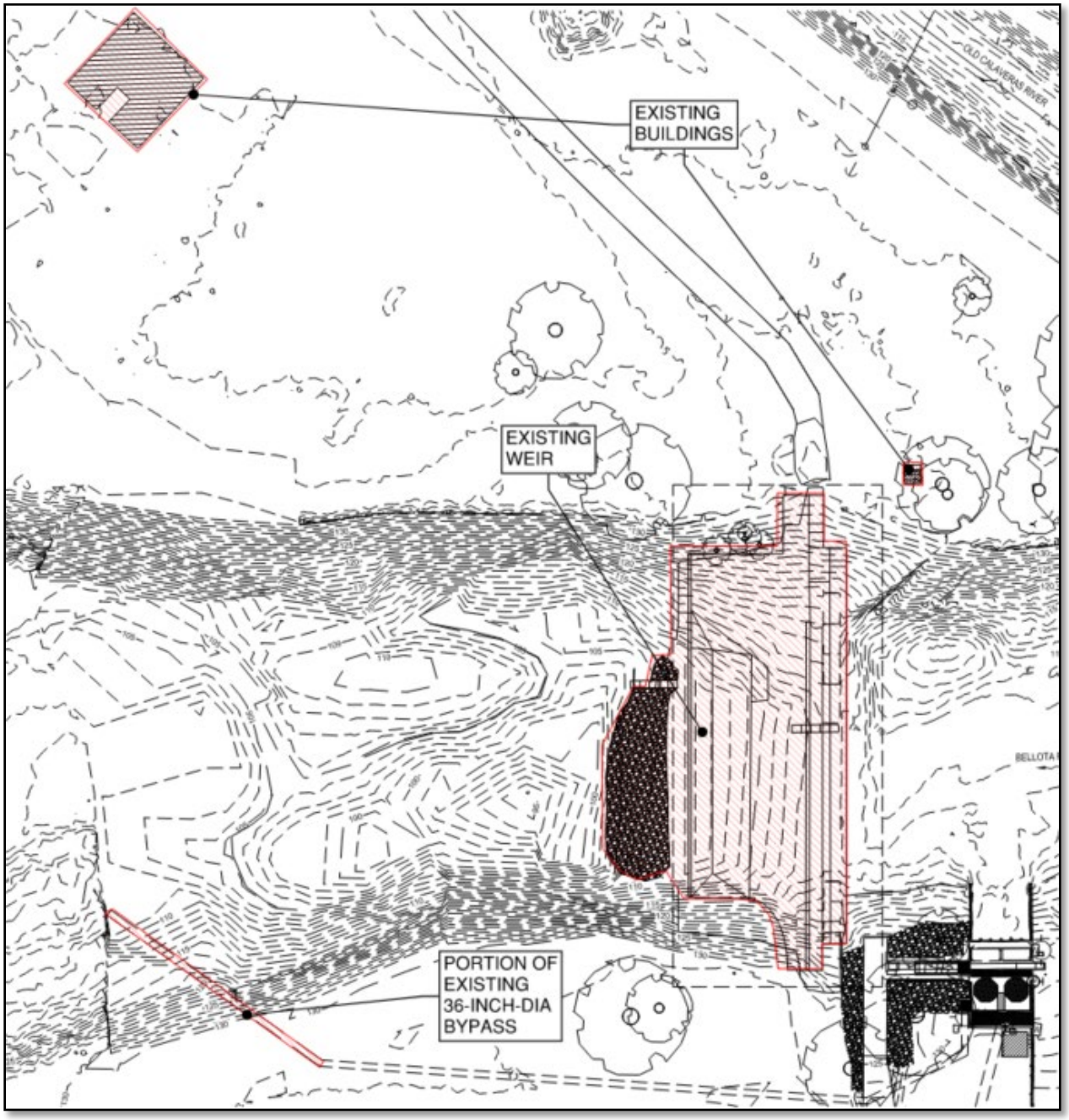
- Weir including existing sheet piling
- Existing buildings
- A portion of the existing 36-inch-diameter bypass pipe

The design currently assumes that all materials included in the demolition effort will be removed from the site and disposed at an appropriate facility that meets all local and environmental requirements. To minimize the amount of material shipped off-site, the reuse of concrete rubble as common fill behind the new right bank concrete wall and other areas where fill material will not interact with river water will be evaluated as the

design progresses. Recycling cleaner portions of the concrete rubble may be cost efficient for the Project, should it be allowed by the resource agencies. Existing rock at the Project site such as that used for the existing weir may be reused provided it meets Project specifications. The Denil ladders will be salvaged for potential future use. All construction debris from the existing buildings is anticipated to be disposed of off-site in compliance with the Project specifications. All other existing elements will be protected in place.

All facilities will be decommissioned during the first sequence. Any required utilities such as electricity will be relocated so that the existing intake can remain online throughout construction.

Figure 5-1. Existing Elements to be Demolished



Given the broad area of demolished features, sequencing of demolition activities will likely be required for consistency with overall Project construction strategy. Sequencing, in order of completion, could be as follows but may ultimately be revised as required by the construction contractor:

1. Perform temporary relocation of appropriate power and communication utilities
2. Remove existing roads and gravel surfacing

3. Remove existing buildings on the site
4. Remove a portion of the existing 36-inch-diameter bypass pipe
5. Remove existing Bellota Weir
6. Complete power and communication relocations as construction allows

For those features below the ordinary high water mark, Section 11 outlines demolition sequencing in relation to dewatering procedures.

5.3 Care-of-Water Planning

The care-of-water strategy for dewatering open excavations and implementing river flow bypass strategies to isolate construction work areas will be one of the greatest sources of construction risk and uncertainty. Sufficient care-of-water planning will be critical to protect in-water work activities during prescribed work windows (mid-June and early October), and effectively manage the treatment and discharge of nuisance water extracted from excavations and work areas. The care-of-water strategy is described in conjunction with the conceptual construction strategy in Section 11 to a level commensurate with the 90 percent design. The construction contractor will be responsible for the final care-of-water plan and implementation measures.

5.4 Access

5.4.1 North Bank Access

Permanent site access for the project will be from the north entrance on State Route 26, which is existing but will be improved as part of the Project. The gate will be equipped to open automatically, and the gate and associated fencing will be relocated so that a delivery vehicle can pull completely off of State Route 26 while waiting for the gate to open. The access road to the Project improvements will be an aggregate base road with a turnaround constructed to allow large trucks such as a 65-foot tractor-trailer or a 40-foot crane truck to access the site through forward movement.

5.4.2 Intake and Fishway Access

A new temporary construction entrance will be constructed south of the permanent entrance on State Route 26. The gate will be opened manually, and the gate and associated fencing will be located so that construction vehicles can pull completely off of State Route 26 to open the gate. The temporary access road for Project construction will be an aggregate base road, which will be left in place after construction is completed. The temporary access road will be aligned with the turnaround installed as part of the permanent improvements, so that construction vehicles are able to turn unimpeded and leave the construction site through the north entrance on State Route 26, to allow single-direction traffic during construction.

5.4.3 River and Roughened Channel Access and Rock Slope Protection

The main channel of Mormon Slough will be accessed via a pathway that begins on the north bank of Mormon Slough, connecting to the permanent road on site. The access road ramp slopes down to the riverbed from EL 131 to 108 feet NAVD88 and follows the waterward face of the north bank upstream to the proposed weir at EL 115 feet NAVD88. It will be composed of foundation rock, same as found in the roughened channel. Heavy equipment traveling to the bottom of the ramp will travel across the downstream exit of the sluice pipeline and then upstream along the access berm. The access berm will be usable for maintenance efforts such as removal of large woody debris or constructing temporary cofferdams to inspect or repair any features.

In addition, the banks of the roughened channel will be designed at a 3H:1V slope for ease of maintenance and access. To combat erosive processes, the banks will be composed of rock slope protection with infill sediment.

The rock described in this section was sized by calculations provided in Appendix E.

5.4.4 South Bank Access

Permanent site access for the south bank of Mormon Slough will be from the east entrance on East Shelton Road, which is existing but will be improved as part of the Project. The gate will be equipped to open automatically, and the gate and associated fencing will be relocated so that a delivery vehicle can pull completely off of East Shelton Road while waiting for the gate to open. The access road to the Project improvements will be an aggregate base road. A west entrance on East Shelton Road will be constructed to allow large trucks such as a 65-foot tractor-trailer or a 40-foot crane truck to access the site through forward movement. This west entrance gate will be opened manually, and the gate and associated fencing will be located so that large vehicles are not forced to park on East Shelton Road while closing the gate to leave the site.

A new temporary construction entrance will be constructed west of the permanent west entrance on East Shelton Road. The gate will be opened manually, and the gate and associated fencing will be located so that construction vehicles are not forced to park on East Shelton Road while closing the gate to leave the site. The temporary access road for Project construction will be an aggregate base road, which will be left in place after construction is completed. The temporary access road will allow single-direction traffic during construction, and provides access to the full length of the roughened channel from the south bank of Mormon Slough.

5.5 Earthen Embankment

The earthen embankment will be located in the Old Calaveras River channel as close to its divergence from the Calaveras River/Mormon Slough as feasible to provide fish exclusion for all flows to the Old Calaveras River. Screened flows to the Old Calaveras River will come from the Intake and be released and metered based on downstream irrigation demands. The earthen embankment will consist of a low-permeability core cutoff wall, an earthen embankment, and rock slope protection on both sides of the earthen embankment.

The low permeability core cutoff wall will be a minimum of 8 feet wide and constructed from materials with low water permeability such as a clay or bentonite soil mix. The earthen embankment outside of the cutoff wall will be constructed from suitable on-site engineered fill to a minimum of 3 feet above the 100-year base flood elevation, and span the Old Calaveras River channel. Both sides of the earthen embankment will be armored with a layer of rock slope protection to protect the embankment against erosion from wave wash.

5.6 Storm Drainage

The proposed Project is anticipated to introduce limited impervious surface. Drainage improvements required as part of site development are anticipated to be minimal. The roads added to the Project will be surfaced with aggregate. The tops of exposed concrete walls provide minimal contributing run-off. The utility building and crane pad will contribute 440 square feet and 2,500 square feet of impervious area, respectively. The structure on the north bank including the fish ladder and intake will contribute about 7,000 square feet of impervious area.

The design team is working with the District to determine permanent storm drainage requirements for the Project. Further details will be added in future iterations of this document.

6 Geotechnical Design

The Project area is in San Joaquin County, which lies in the region of the confluence of the San Joaquin and Sacramento rivers. The San Joaquin Valley basin has been filled over time with up to a 6-mile-thick sequence of interbedded clay, silt, sand, and gravel deposits. Based on geotechnical explorations in the area, the materials within the Project area can be divided into three layers: upper overbank deposits, a middle alluvial layer, and older, stiffer overbank deposits.

6.1 Materials

The project site consists of interbedded silts, silty sands, sands with variable percentages of gravel, silty sand, sandy gravels and clay typical for the riverine environment, The subsurface conditions within the depths of interest can be divided into three general layers: upper overbank deposits overlying a middle alluvial layer which then overlies older, stiffer overbank deposits as shown on Figure 6-1 through Figure 6-3. Brief discussion of these layers are provided below, detailed discussion with subsurface profiles are included in Figure 6-1 through Figure 6-4.

6.1.1 Upper Overbank Deposit

The Upper Overbank deposits generally consist of loose brown silty sand and non-plastic sandy silt to a depth of 12 to 15 feet with 30 percent or greater fines contents in courser soils. Gravels up 1.5 inch wide were also encountered in these soils. These deposits are located on both banks of the Mormon Slough and extend to the Old Calaveras River. A gravel deposit is located along the south bank of the Mormon Slough and may extend beneath the existing weir structure.

6.1.2 Alluvium

A 20- to 35-foot-thick alluvium layer underlies the upper overbank deposits and the entire site. The alluvium is comprised of a 15-foot-thick upper fine-grained layer consisting of interlayered loose silts, silty sands, silty clayey sands, and soft clays with discontinuous loose gravel deposits within the meandering river channel. The finer grained soils have a low plasticity with plasticity indices of less than 15. The lower layer has a coarser grained layer consisting of loose silty to clayey sands with varying gravel content and more pronounced clayey gravel zones. These discontinuous layers of clayey gravel are located beneath the existing slough and north bank, deposited in former meanders in the river. The clayey gravels contain approximately 6 to 30 percent fines and 15 to 30 percent sand.

Figure 6-1. Geotechnical Boring and Cross-Section Locations

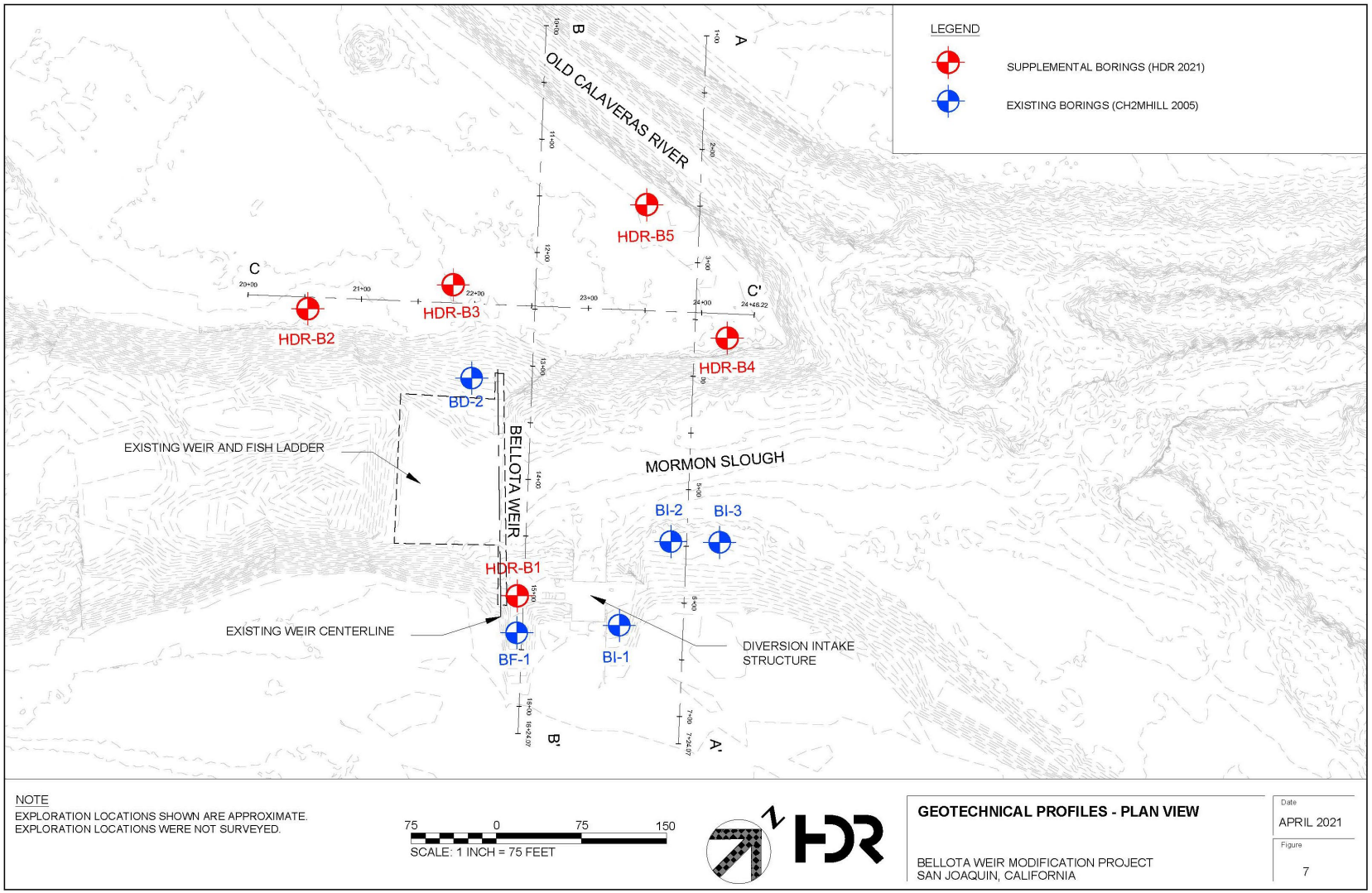


Figure 6-2. Profile A-A' Looking Upstream 170 feet upstream of Weir

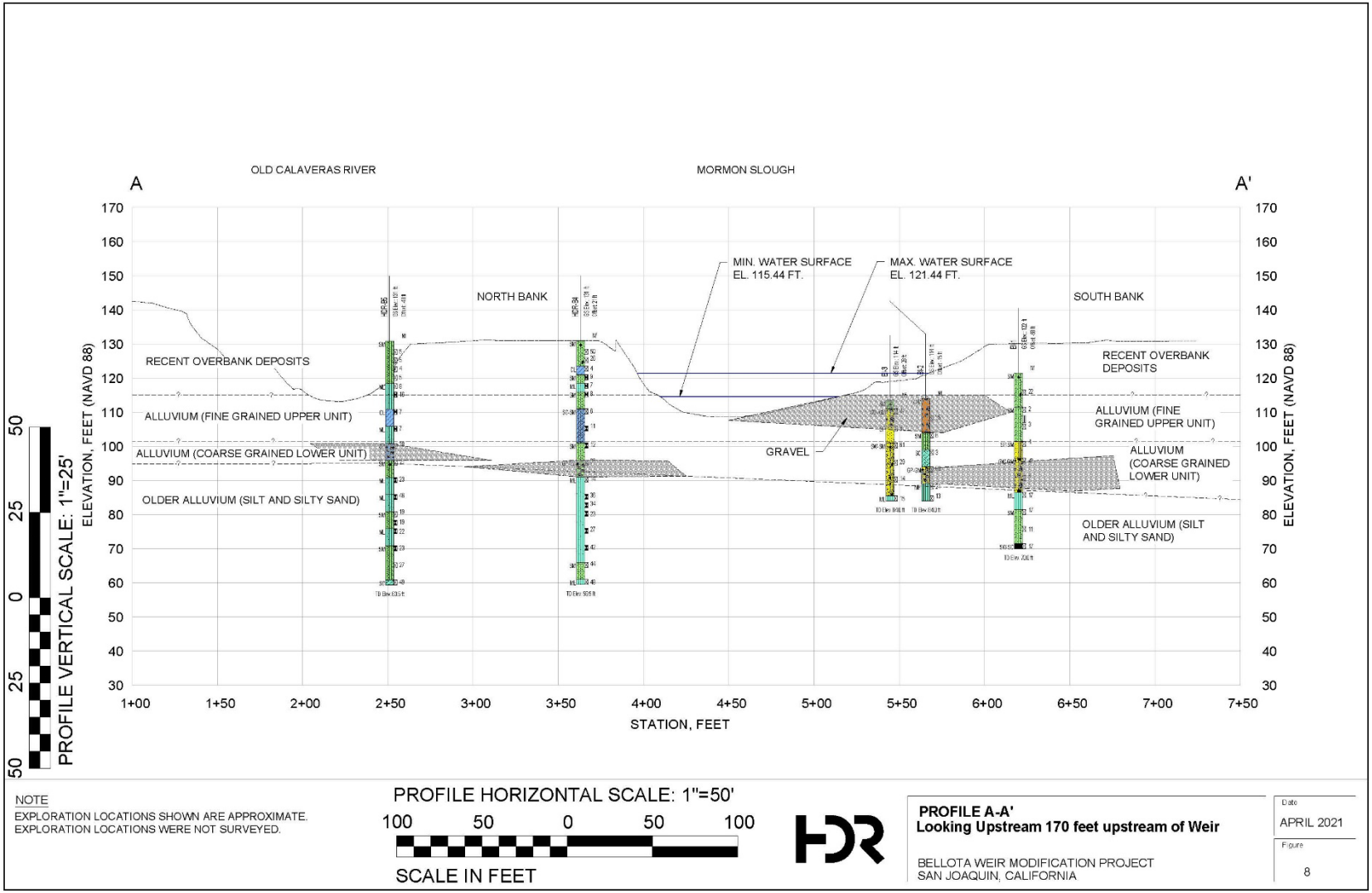


Figure 6-3. Profile B-B' Looking upstream 28 Feet from Existing Weir

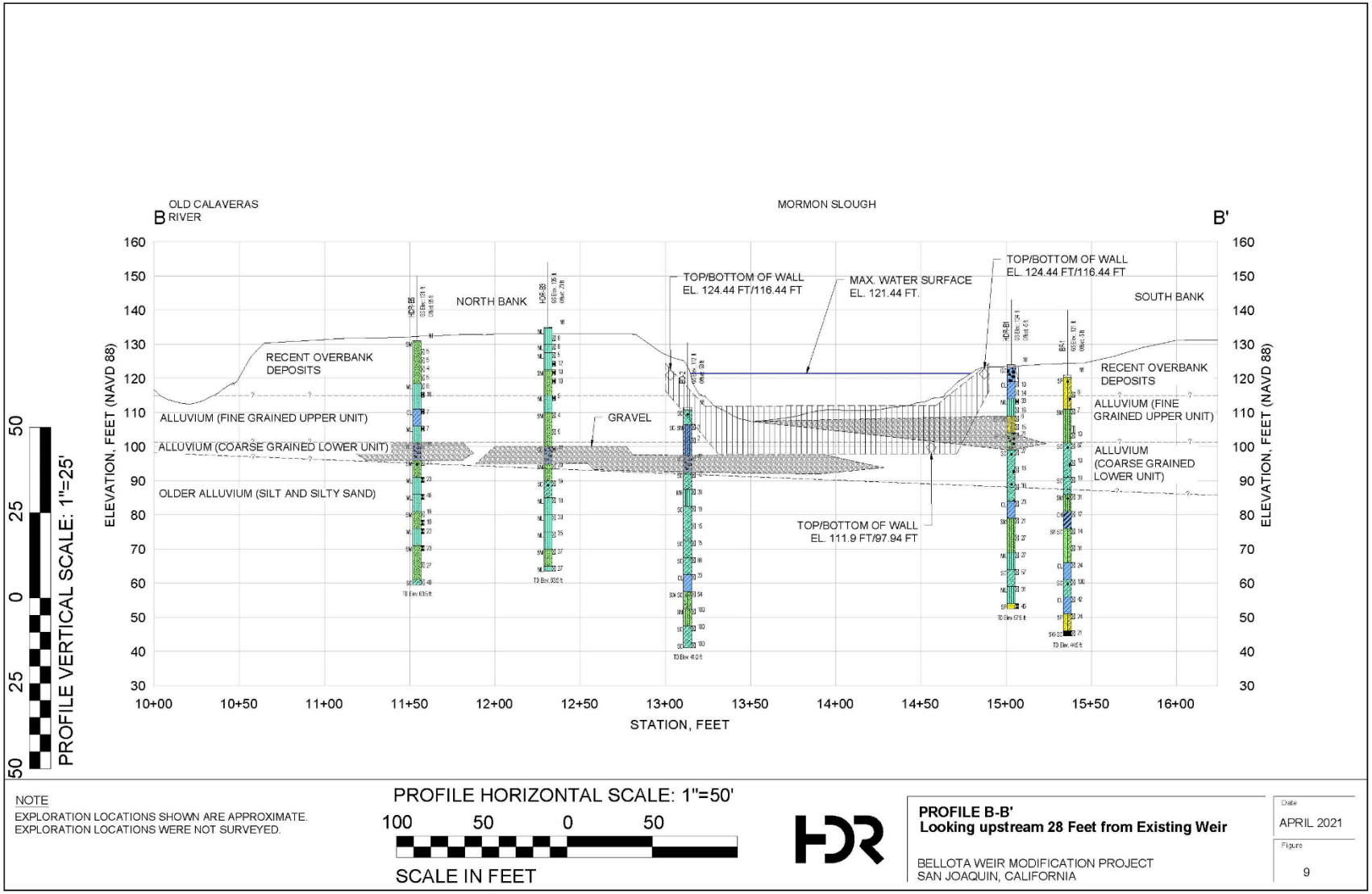
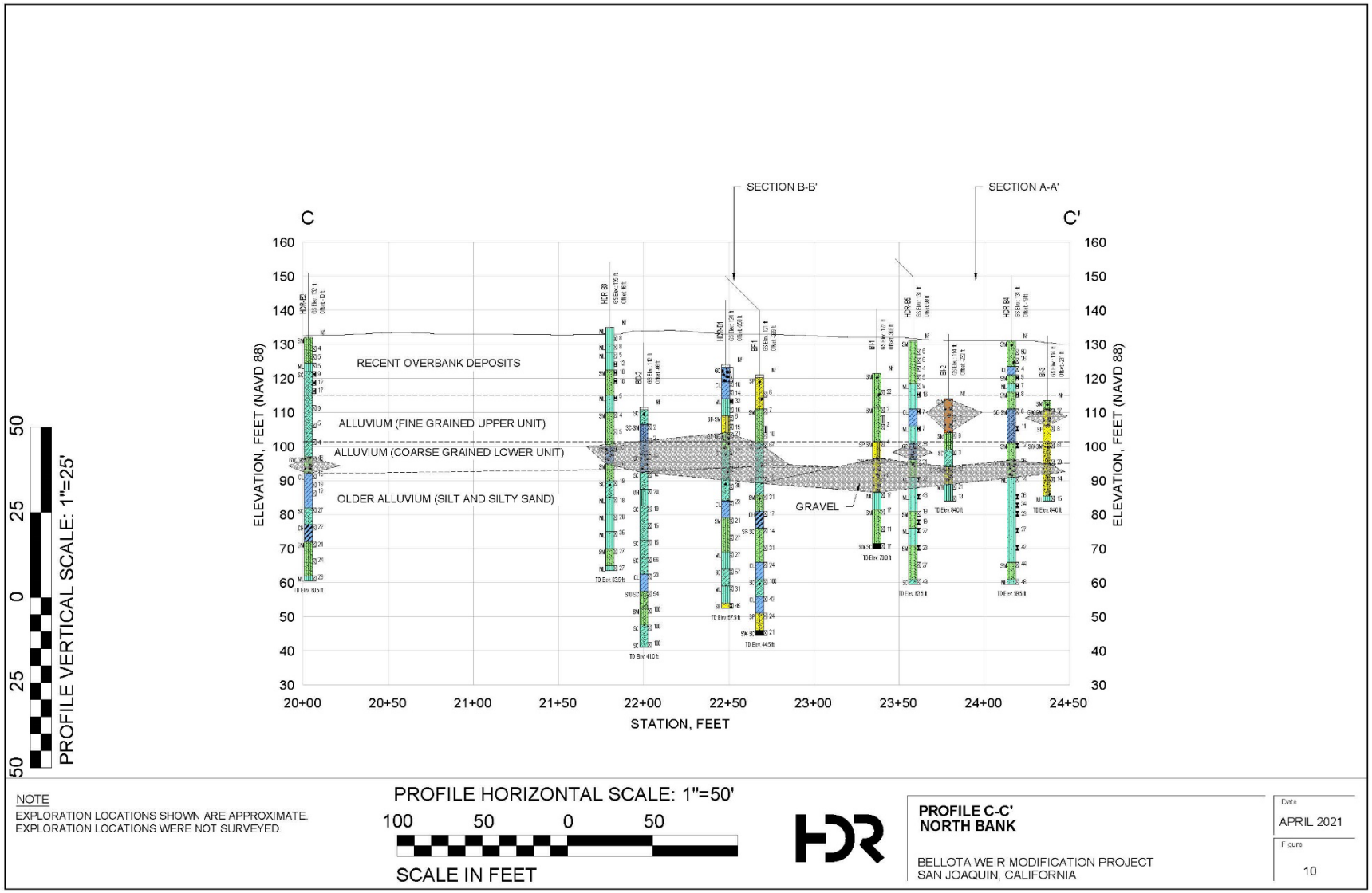


Figure 6-4. Profile C-C' North Bank



6.1.3 Older Alluvium

Older medium dense to hard alluvium is located at a depth of 40 feet and consists of alternating layers of silt and silty sand to the maximum depth explored (70 feet). Some poorly graded sand layers were encountered at depth beneath the south riverbank. The older alluvium is stiffer and denser than the overlying soils indicative of an older geologic soil unit.

6.2 Material Properties

The engineering soil parameters for the subsurface materials were developed based on the conditions depicted on the boring logs from the current and previous investigations and laboratory testing and are further discussed in Appendix C.

6.3 Roughened Channel Filter Design

The roughened channel filter and transition layer will be constructed of suitable materials (i.e., sandy gravels and gravely sands containing no more than 10 percent silt size materials) from required excavations. The material will be placed in lifts no greater than 12 inches and compacted with three complete passes of the spreading equipment. The roughened channel will be designed with a two layered zone of materials sized to prevent erosion and scour during a 100-year flood event. The layers will consist of bedding material for the roughened channel foundation rock and a filter layer between the bedding material and the native sands and gravels forming the roughened channel fill. The materials will have a gradation sized to prevent internal erosion and piping of the channel fill.

6.3.1 Filter Design

Filter layer gradation specifications were determined based on the range of particle sizes of the native riverbed soil using criteria from the USACE (1994) and U.S. Bureau of Reclamation (Reclamation 2011). A spreadsheet tool developed by the Virginia Polytechnical Institute and State University (VPI 2004) was used to the evaluate the compatibility of candidate filter and drain material gradations relative to the respective base soils/materials (i.e., filter material with respect to the backfill soils and drain materials with respect to the filter material).

Gradation for the base soil (native riverbed soil) was determined using the available grain size distribution test data for the material near EL 100 feet from the existing and supplemental explorations. Not one standard material graduation was found that would meet the filter layer requirements and a 40 to 60 percent mixture by weight of C33 -02A Size 9 sand and AASHTO M43-99 Size #57 sand provides the necessary gradation. The base soil and filter layer gradation bounds were selected based on five grain size distribution test results in Appendix C.

Bedding Material

Gradation for the transition layer was determined using the aforementioned filter zone grain size distribution and the same methodology as for the base filter above. The proposed rock infill material would meet gradation requirements as transition layer if the rock infill material had tighter controls and the percent passing the #16 and #200 sieves will provide a lower coefficient of uniformity (Cu = d60/d10) by limiting the percent finer than the # 4 sieve to 16 percent and the percent passing the # 200 sieve to 3 percent. This will provide more stable gradation for transport and placement.

6.3.2 Riprap Bedding

The Class III and Class V riprap have a D_{50} of 12.5 and 18 inches respectively per the Caltrans New Riprap Classes. Using the Cistern-Ziems Chart from NCHRP Report 568 and assuming the riprap and riprap bedding layers have a Cu = 2.4 and 2.0, the A value of the D_{50r} to D_{50b} (riprap:base layer) is 10. The D_{50b} of the Class III and Class V riprap is 1.25 and 1.8 inches. Therefore, a coarser backing layer is needed. For both Class III and Class V riprap the D_{50} of the backing layer should be a minimum of 8 inches thick with a gradation of $1.5 < D_{50} < 2$ and the backing layer to replace the upper 8 inches of the transition layer. If a geotextile would be used in lieu of a transition/backing layer, a geotextile a fabric with an Apparent Opening Size of 0.2 mm (#70 sieve) would be sufficient for placement over the filter layer based on permeability. It should be a woven geotextile with sufficient strength to not be damaged by the proposed placement method.

6.4 Sheet Pile Cutoff

The existing weir includes a sheet pile wall along the upstream edge of the foundation varying in depth from 8 feet at the abutments to 14 feet in the center of the channel. Review of the as-built drawings of the existing weir show the sheet pile wall was added after the original weir was washed out by high river flows. Due to the relatively high permeability of the soils located at this site a full seepage cutoff is not judged to be obtainable. However, a new sheet pile wall should be incorporated in the design to prevent washing out of loose channel deposits soils that could lead to undermining of the new weir structures. To construct the new weir, the existing weir should be removed, and the foundation excavated to expose the existing sheet pile wall. The existing sheet pile wall could then be exposed and either extracted and/or cut to the excavated subgrade. The new sheet pile wall should extend to a depth of 20 feet below the gated section of the weir and the tip elevation held constant as the wall progresses up the abutments terminating above the full pool water surface. A test pile program should be performed to confirm the appropriate method of pile installation to reach the design depth. The piling also will be driven through a zone of large gravels. Therefore, a driving shoe should be used to protect the pile during driving.

6.5 Foundation Design

Appendix C provides construction considerations and foundation recommendations for the design of structure foundations including pile capacity charts and lateral earth loading diagrams for the fish ladder and weir design. The piles should gain their support in the

underlying older alluvium. Downdrag load imposed by the overlying soil that could settle as a result of strong ground shaking.

The proposed structures should be designed to resist the lateral forces generated by earthquake shaking in accordance with local design practices. The seismic design parameters based on the American Society of Civil Engineers (ASCE) 7-16 as modified by Supplement #1 and the 2019 California Building Code for a Latitude of 38.052325 and a Longitude of -121.011499 are presented in Table 6-1.

Table 6-1. Seismic Design Parameters

Parameter/Coefficient Description	Parameter/Coefficient	Value
MCE_R for 0.2 sec Spectral Response Acceleration, Site Class D	S_s	0.490g
MCE_R for 1.0 sec Spectral Response Acceleration, Site Class D	S_1	0.287g
Soil Profile Type	Site Class	D
Site Coefficient	F_a	1.4
Site Coefficient	F_v	2.15
Adjusted MCE Spectral Design Parameters	S_{MS}	0.690g
	S_{M1}	0.480g
Design Spectral Acceleration Parameters	S_{DS}	0.460g
	S_{D1}	0.320g
Seismic Design Category	Occupancy Category I to IV	III

MCE = maximum considered earthquake
MCE_R = Risk targeted maximum considered earthquake

Over-excavation and replacement of near surface soils are recommended for structures to provide a suitable compacted foundation to support improvements on the north and south banks. Over-excavation should extend to a minimum depth of 5 feet and the on-site soils appear to be acceptable for re-use as engineered fill. A deeper depth of over excavation should be provided to maintain a gradual change in fill thickness to limit the potential for abrupt differential settlements, such as beneath the crane pad where deep excavations will be required to install the Bellota Conveyance pipelines.

The potential for liquefaction in the loose alluvial soils was found to be low; however, differential settlements could occur due to strong ground shaking at the site. Therefore, use of stone columns to support structures sensitive to differential settlement is recommended. The structures should bear directly on the columns and gain support in the underlying older alluvium below EL 100 feet NAVD88.

The Project will be constructed in phases to maintain stream flows. Therefore, the foundations design should be checked for various load conditions including those during construction. It is important to check that the phased construction will not adversely affect the long-term performance of the structures and their foundation. In particular, the installation of the weir pile foundations, sheetpile cutoff wall and the Bellota conveyance pipes will require the sheetpile wall to act as a cantilever retaining wall supporting

upstream material until the pipes are backfilled and the weir is slab is constructed. Construction activity will need to be sequenced so the cutoff wall is not overstressed.

6.6 Earthwork and Shallow Foundation Design Recommendations

Recommendations for performing earthwork and geotechnical design recommendations for shallow foundations are provided in Appendix C. These recommendations address:

- Soil densification
- Subgrade preparation
- Engineered fill materials
- Fill placement and compaction
- Temporary excavations
- Dewatering
- Trench backfill
- Shallow foundations
- Mat foundations
- Slab-on-grade foundations

6.7 Steel Pipe Piling

To support the weir structures, 24-inch diameter steel pipe piles have been selected. Recommendations for axial capacity of steel pipe piles assuming open end piles are driven, and a soils plug will form below EL 90 feet. Recommendations to account for lateral group effects are provided in Appendix C.

Due to the potential for liquefaction induced settlement, downdrag forces should be included assuming the soil above EL 100 ft induces downdrag forces.

The piles will be advanced through a zone of gravels and piles should be driven using a shoe with the same inside diameter as the pipe pile to ensure that the pile is in intimate contact with the soils as it is driven.

6.8 Retaining Structures

The fish passage structure and retaining walls will be designed as restrained walls that are not free to move or rotate. Therefore, at-rest pressures will be assumed for static loading conditions. To limit wall pressure, walls should be backfilled with compacted native granular soils. Based on the available geotechnical information at the overall Project area, onsite soil available for borrow and structure backfill is predominantly sandy sit and silty sand. High plasticity soil should be avoided for use as wall backfill (Appendix C). On this basis, the walls may be designed assuming the following soil parameters and earth pressure coefficients appropriate for compacted lean clay backfill:

- Backfill total unit weight of 125 pounds per cubic foot (pcf) above the phreatic surface for each design case.
- Backfill buoyant unit weight of 62.6 pcf below the phreatic surface.
- At-rest earth pressure coefficient (K_O) of 0.60 (for static loading).
- Active earth pressure coefficient (K_A) of 0.43 (for seismic loading).

Under static loading conditions, the walls will be designed to resist hydrostatic and lateral earth pressure, as well as additional loads caused by surcharging. Under seismic conditions, the active pressure will be combined with a uniform seismic pressure increment of 6H pounds per square foot (psf), where H is the height of the backfill. This seismic earth pressure was developed using the design peak ground acceleration of 0.28g. Earth pressure diagrams for design of the walls are presented in Appendix C.

Because the walls will be backfilled with granular soils, back drainage may not be required.

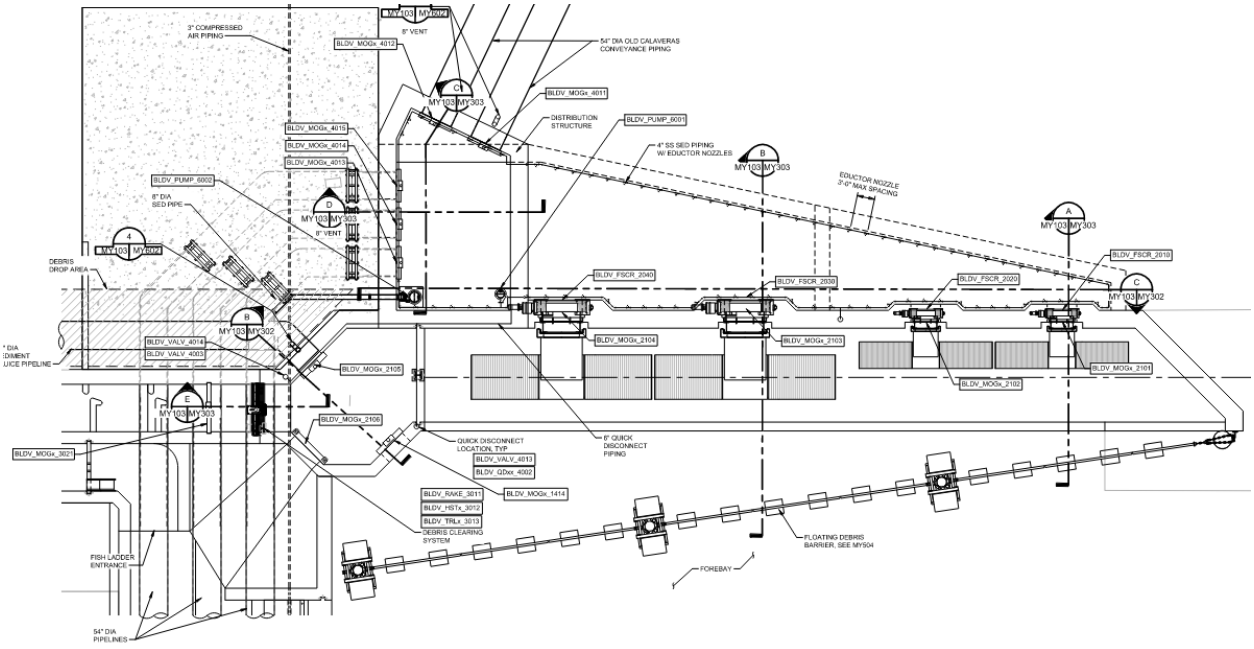
7 Mechanical Design

Mechanical design components of the Project include but are not limited to the following:

- Air actuated adjustable weir gates
- Fish screens
- Motor-operated gates and actuators
- Water jet sediment mobilization system
- Sediment pump
- Debris management system
- Screen channel and intake manifold sluice pipes
- Water conveyance pipes
- Control building heating, ventilation, and air conditioning (HVAC)

Figure 7-1 illustrates the mechanical Project components not including the control building that houses the pump for the water jet sediment suspension mobilization system and the compressor for the air actuated adjustable weir.

Figure 7-1. Mechanical Intake Plan



7.1 Air-Actuated Adjustable Weir Gates

The adjustable weir gates will serve the following three purposes:

- To maintain the upstream WSE for the intake during the intake's operational periods

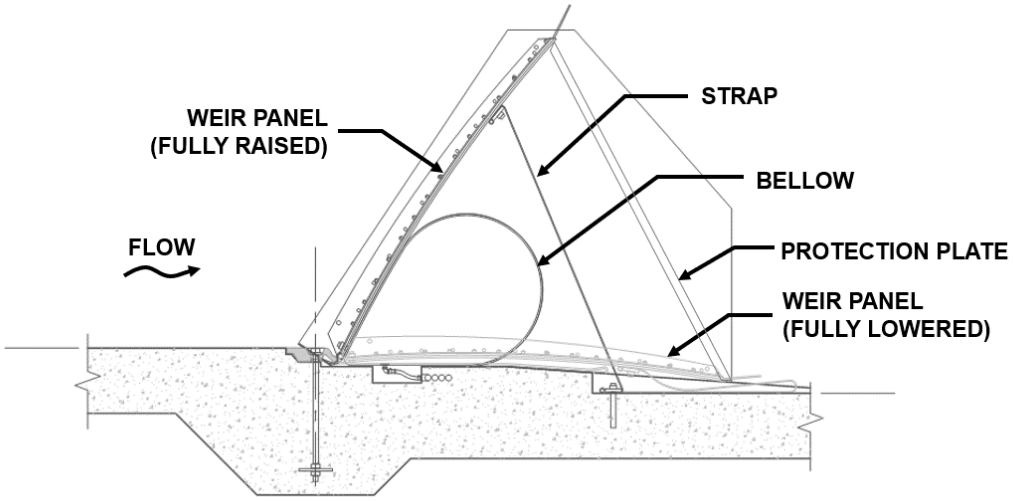
- To move accumulated bed load and debris downstream when in its fully lowered position
- To allow fish passage over the low-set gate when in its fully lowered position

The minimum WSE to be maintained during operational periods will be EL 115.44 feet NAVD88.

7.1.1 Design Description

The adjustable weir gates will be a procurement item with the full turn-key style package provided by a suitable manufacturer. A general layout is presented in Figure 7-2.

Figure 7-2. Elevation View of Adjustable Weir



The adjustable weirs will be air actuated. An air compressor, receiver tank, and other associated accessories will be located in the control building approximately 200 feet from the adjustable weir gates. Piping from the receiver tank feeds air to the bellow underneath the weir panel. As the bellow fills with air, the weir panel rises. Straps attached to the downstream edge of the weir panel control the height of the weir panel when in its fully raised position. When the adjustable weirs are in their fully raised position, the straps will result in the gate crest at EL 121.44 feet NAVD88.

When the adjustable weirs are in their fully lowered position, the positive downward angle of the weir panel will be maximized so it can best move any accumulated debris and bed load downstream and allow for fish passage upstream. Historically, these types of gates have been inefficient when in the lowered position and operated with debris or bed load present on top of the weir panel's skin plate. The concrete surface underneath the weir panel will also be sloped downward to encourage any debris or sediment to move downstream.

The governing parameters that dictate adjustable weir gate sizes are hydraulically and structurally driven. The concrete intake's width of 150 feet dictates the three weir widths, with each gate being 48 feet, 8 inches wide and having a 2-foot concrete pier separating each gate. However, each gate must be split into smaller,16 feet, 2 inches wide panels, as the air bladder cannot accommodate a single 48-foot, 8-inch-wide panel. The maximum WSE within the intake determines the three weir heights in the fully raised position. The weir gate closest to the new intake structure will have a sill elevation 1 foot lower than the other two gates to allow for fish passage while the gates are in the fully lowered position. The force imparted from the bellow to the weir panel will be dictated by the anticipated hydraulic forces and developed further by the manufacturer, along with other detailed design calculations because this is a procured item

7.1.2 Operational Strategy

During the non-irrigation period, the adjustable weir gates will be in the fully lowered position. The center and south weir gates will have a control surface EL of 115.5 feet NAVD88, while the north weir gate will have a control surface EL of 114.5 feet NAVD88. The north weir gate will be set lower for fish passage considerations at the dam. During the irrigation season, the weir gates will be raised to a set position to maintain the targeted forebay WSE. The maximum forebay WSE for normal operation will be 121.5 feet NAVD88.

During normal operating conditions for the irrigation season, the adjustable weir gates will modulate to maintain a constant upstream water surface for the intake. The adjustable weir gate and water control gates in the surface water diversion intake will be used to divide river flows between SEWD, Old Calaveras River, fish ladder and Mormon Slough demands. During extreme low flow conditions, the adjustable weir gates could be set to not pass any flow.

7.2 Fish Screen Design

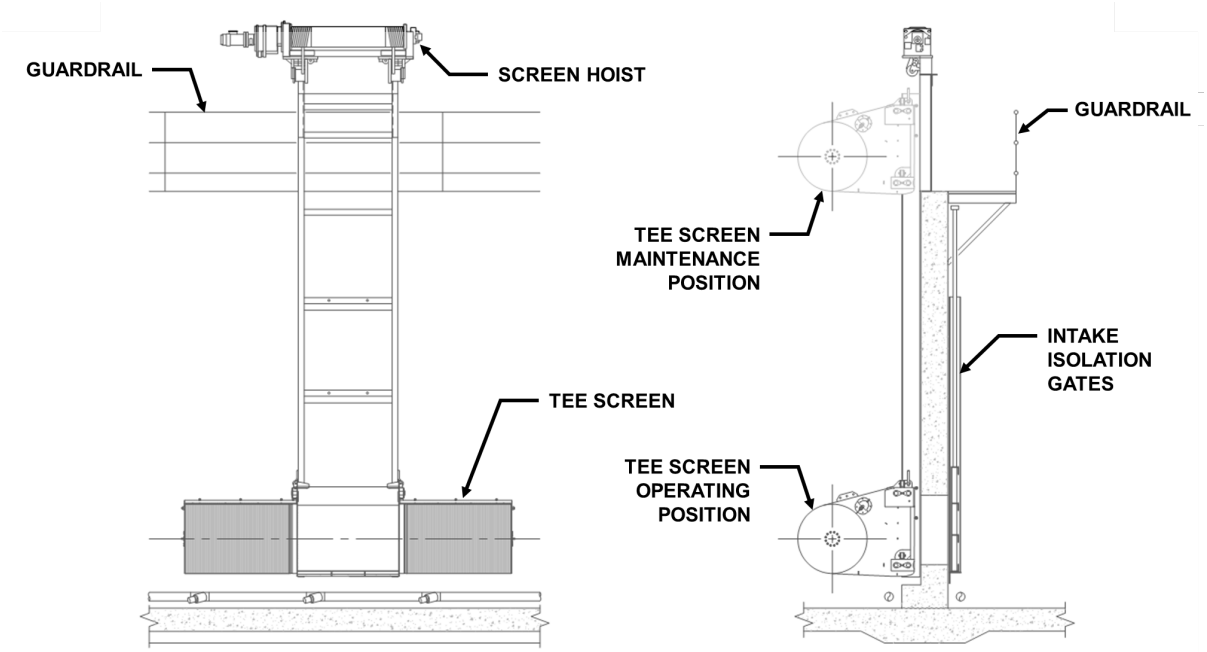
Fish screens will be provided in the screen channel on the north side of the forebay to exclude juvenile and sub-adult fish from passing to the intake manifold and exiting the river through the diversion conveyance piping. The fish screens will be designed to comply with NMFS and CDFW (formerly known as California Department of Fish and Game [CDFG]) fish protection guidance and criteria (NMFS 2011; CDFG 2002).

7.2.1 Design Description

Fish screening will be accomplished by integrating four cylindrical tee screen systems mounted on retractable rails along the southern wall of the forebay. There will be two tee screen sizes located in the intake. The smaller tee screens will be composed of two, 48-inch-diameter and 6-foot-long cylinders wrapped with type 316 stainless steel wedge wire with 0.069-inch (1.75-mm) slots. The larger tee screens will be composed of two, 83-inch-diameter and 10-foot-long cylinders with type 316 stainless steel wedge wire with 0.069-inch (1.75-mm) slots. The two cylinders will be mounted in-line on a single manifold with a total fabricated length of approximately 18 feet, 3 inches for the smaller tee screens and approximately 27 feet, 7 inches for the larger tee screens. In the down position, the

cylinders will typically be submerged below the water surface with a minimum submergence of one radius (24 inches for the smaller tee screen and 41.5 inches for the larger tee screen). The actual submergence values are shown in Table 4-1. During diversionary periods, water passes through the screens and into the intake manifold through a rectangular opening in the wall behind each screen. A stop gate will be provided behind each screen to prevent flow passing through to the intake manifold when a screen is not in use. When not in operation, each cylinder could also be retracted up into a maintenance position at the level of the intake wall. Figure 7-3 shows an elevation and section view of a single cylindrical screening system.

Figure 7-3. Elevation and Section of Single Cylindrical Tee Screen



The screening array provides enough effective screen area to maintain an approach velocity at or below 0.4 fps during maximum diversion demands of 350 cfs with an additional factor of safety of at least 1.1 to account for hydraulic non-uniformities such as uneven occlusion of debris on screen face or structural blinding that occurs as flow passes through the different baffling, manifolds, and openings present in the screens.

Each cylindrical tee screen exhibits a total effective surface area of 150.8 square feet (smaller) or 434.6 (larger) with a total effective area provided by all four screens of 1,170.8 square feet, when submerged. The required effective area to accommodate an approach velocity of 0.4 fps with a factor of safety of 1.1 is 963 square feet. Appendix F provides detailed design calculations for the fish screens.

Each cylinder is cleaned actively during periods when flow passes through the screens. Static brushes are located on the inside and outside faces of each cylinder. When operating, electric motors rotate the cylinder and the brushes scour the inside and outside screen surfaces removing debris from the slots and eliminating biological growth.

7.2.2 Operational Strategy

Screens are in their full down operating position during diversionary periods with the isolation slide gate fully open (up). The screen cleaning cycle is put on automatic and the cylinders rotate at a user-specified frequency. Settings for normal operation or periods where debris loading causes higher differentials will regulate how often and how many screens rotate at a time. During non-operational periods, the screens are retracted to their maintenance position at the full up position and the isolation slide gates are in their full down (closed) position. In this position, screens are level with the top of wall and can be easily inspected, maintained, or repaired. Table 7-1 shows the active screens during the irrigation and non-irrigation periods.

Table 7-1. Minimum Active Tee Screens

	Tee screen 1 83-inch	Tee screen 2 83-inch	Tee screen 3 48-inch	Tee Screen 4 48-inch
Irrigation (current)	Active	Retracted	Active	Retracted
Irrigation (future)	Active	Active	Active	Active
Non-irrigation (current)	Retracted	Retracted	Active	Active
Non-irrigation (future)	Retracted	Retracted	Active	Active

Note: Active screens can be interchanged for another screen of the same size.

During the non-irrigation season (flows up to 75 cfs), only the two, 48-inch screens will be in use. To achieve the current maximum flow required (220 cfs), one 48-inch screen and one 84-inch screen will be used. To achieve the future maximum flow requirement (350 cfs), all four tee screens will be used.

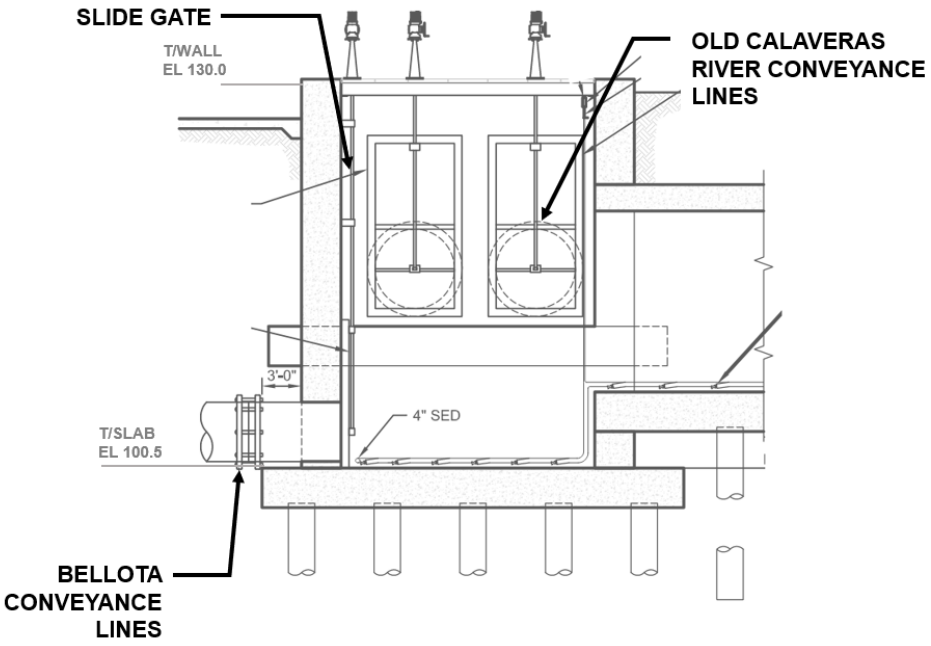
7.3 Motor-Operated Gates and Actuators.

There are 16 motor-operated gates and actuators within the intake structure to control flow through the intake. Each of the 16 gates will be equipped with a manual override handwheel option.

7.3.1 Vertical Slide Gates

Most of the slide gates will be undershot style gates, non-self-contained, face-mounted frames fabricated with stainless steel components. The slide gates located at the fish screens and in the fish ladder will be self-contained. The gates will provide either modulating or open/close service and can be operated locally at the actuator or at the touch screen in the control building. The 15 vertical slide outlet gates require design for a maximum seating head between 0 and 26 feet. Figure 7-4 shows the slide gates located in the screen channel, distribution structure, and fish ladder.

Figure 7-4. Elevation/Section View of the Old Calaveras Pipelines vertical slide gates and Bellota Conveyance Pipeline



Each vertical slide gate is equipped with motorized actuators and control systems that allow manual control locally as well as remotely from the control building. All gate movements will be decided by operations personnel.

To estimate the electrical load required for the site, the approximate power required to operate each gate was estimated (Appendix F). The actual actuator used for the Project will need to be determined by the gate manufacturer.

7.3.2 Vertical Weir Gate

There is one vertical weir gate to direct water from the screen channel to downstream of Bellota Weir. This gate will be used to remove any buoyant or semi-buoyant debris that may enter the screening channel, pass fish out of the screened channel when the fish ladder is not in operation, and serve as an auxiliary water supply as needed for the fish ladder.

The vertical weir gate is equipped with a motorized actuator and a control system that will allow manual control locally and remotely from the control building. Gate movements are decided by operations personnel. The estimated electrical load required at the site to operate the weir gate was estimated (Appendix F). The actual actuator used for the Project will need to be determined by the gate manufacturer.

7.4 Manual Gates

There will be one manual stop gate on the upstream weir of the fish ladder near the fish ladder entrance. The gate will be actuated using a handle. The gate will provide sluicing

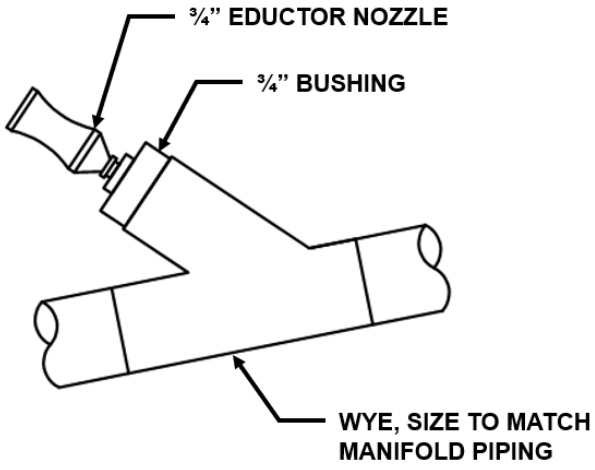
provisions as needed in the fish ladder. The maximum seating and unseating heads on the gate will be 5.7 feet and 5.5 feet, respectively.

7.5 High Pressure Water Jet Sediment Mobilization System

A water jet sediment mobilization system prevents blockage and water storage loss caused by sediment build up in the intake manifold and distribution structure. It agitates sediment so it can become entrained in the flow of water and passed downstream through the various conveyance or sluice piping. For the Project, entrained sediment in the intake manifold can be sluiced back to the river by means of the sediment sluice pipe.

The water jet system will be comprised of an intake supply pipe, rail submersible pump, and outlet pipe. Once the pipe is routed to the intake manifold and distribution structure, it leads to a piping loop along the perimeter's bottom edge. 3/4-inch-diameter eductor nozzles will be spaced evenly every 3 feet along the piping loop with the nozzles pointed either downstream or toward the sediment sluice pipe. Figure 7-5 illustrates an eductor nozzle detail example. Each high point along the piping loop for both bays will have an air vacuum valve or pipe routed below the water surface to release any trapped air from the system.

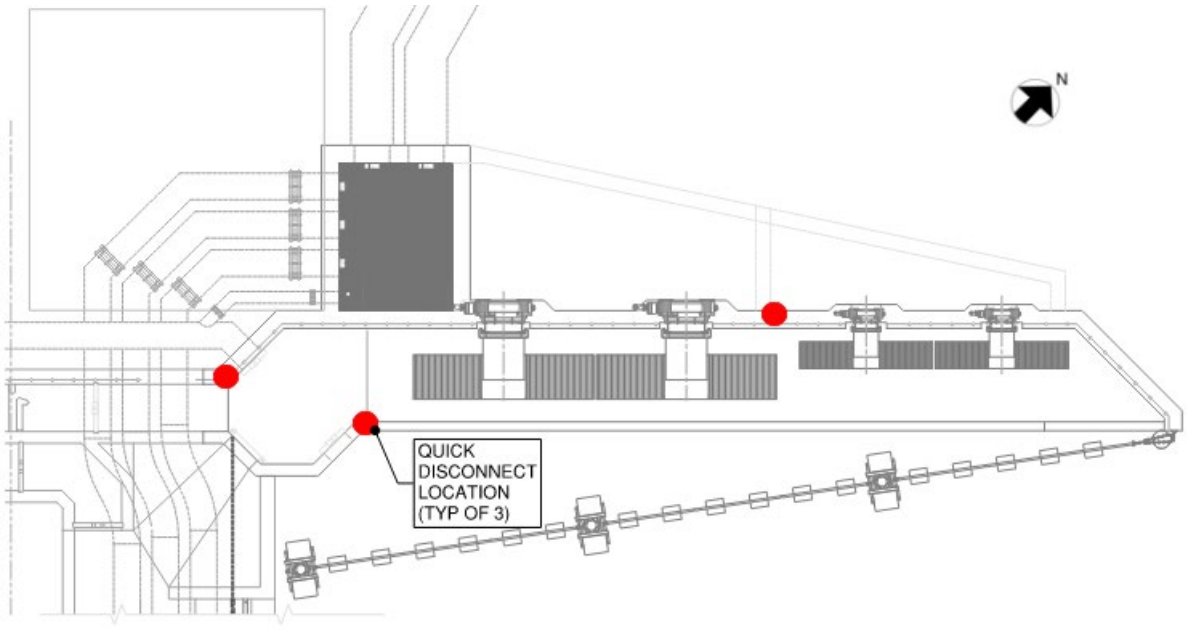
Figure 7-5. Eductor Nozzle Detail



When running at the required pressure, the system will provide a minimum of 10 lbs of force at each eductor nozzle to stir up sediment. The minimum force to entrain sediment particles 0.069 inch and smaller into the flow of water was estimated to be10 lbs (Appendix F). The system will operate as needed to evacuate bedload sediment from the intake manifold to maintain its storage capacity.

The pump will be 60 horsepower and the pump supply piping will be routed to the eductor nozzle system and three quick disconnect locations located throughout the intake. The locations of the three quick disconnect locations are shown in Figure 7-6.

Figure 7-6. Location of quick disconnect outlets to clean intake area.



7.6 Debris Management System

A floating debris boom, a trashrack/debris rake system, a sediment pump, and sediment sluice pipes will be used to prevent debris from accumulating in the intake area.

7.6.1 Floating Debris Boom

The floating debris boom will be located in the forebay directly upstream of the debris wall that leads to the screen channel. The boom will be supported by piles and will have floating sections to prevent large buoyant debris from entering the screen channel. A metal sheet will be attached to each of the floating components to prevent semi-buoyant debris from passing through. The three, 36-inch-diameter steel pipe piles will be driven into the ground such that the bottom elevation is at EL 54.0 NAVD88 and the top of the pile at EL 130.0 NAVD88. There will be a stop plate to prevent further downward movement of the debris boom at EL 114.5 NAVD88. This will prevent the boom from becoming stuck in the sediment at the forebay. There will be a wall-mounted slide rail located at the East side of the boom to prevent debris from passing in that area. Because this is a procured item, detailed design calculations of the boom and wall-mounted rail piece will be prepared by the manufacturer.

7.6.2 Fish Ladder Trashrack and Debris Rake

A trashrack and automated debris rake will be provided at the upstream end of the fish ladder. Both will be supplied by the same vendor to ensure proper fit between the rake and trashrack bars. The trashrack will be vertically oriented rather than inclined to provide sufficient space around the fish ladder exit for other components. The trashrack will consist of vertically spanning parallel steel trash bars with a clear opening between

bars that will provide optimum water flow, fish passage, and protection from debris. The vertical bar placement will also provide a means of manually cleaning the screen. Bars will be rectangular and run vertically from the bottom of the channel to the top. Each bar will individually carry loads into the structure. Horizontally spanning support beams will be uniformly spaced to support the trash bars. Loads from the support beams will be transferred into the sides of the surrounding concrete structure. The bar screen trashrack will have a minimum bar spacing of 10 inches to accommodate Chinook passage per NMFS.

The debris rake will be used to pick up loads of approximately 500 lbs or less and deposit them to the side of the fish ladder for disposal by maintenance personnel. The rake system will have an electric hoist and trolley. The debris rake cleaning procedure can be initiated at the PLC in the control building or manually at the local control panel.

7.6.3 Sediment Pump

To remove accumulated sediment from the distribution structure area of the intake, a sediment pump placed within a sump pit will be operated as needed when sediment accumulates. The pump could also be used to drain the distribution box as needed. The pump will be connected to a slide rail so that it can be easily raised and lowered from the distribution structure. Because the area is behind the fish screens, the sediment pump likely will have to handle fine solids (less than 0.069-inch-diameter). The 8-inch sediment pump piping will connect to the larger 54-inch-diameter sluice pipe downstream. To prevent backwater from reaching the pump, a check valve will be installed downstream. Additionally, an isolation valve will be installed to allow for pump maintenance.

7.6.4 Sediment Sluice Pipe

There will be a 54-inch sediment sluice pipe leading from the screening channel to downstream of the roughened channel to transport sediment intermittently. The sluice system will be operated by opening the slide gate directly upstream of the 54-inch pipe.

7.7 HVAC

Enclosed spaces must be properly ventilated at a minimum and possibly require heating and cooling to meet the needs of the space. The control building will house the air actuated adjustable weir air compressor equipment, and most of the electrical equipment. The HVAC system for the building will include forced air ventilation and a small air conditioning unit to provide cooling for the summer months. A fan and combination louver will make up the forced ventilation inside the building.

7.7.1 Design Description

Per the American Society of Heating, Refrigerating and Air-Conditioning Engineers, the forced ventilation system for the control building will be designed to have five air changes per hour. The fan will be located 1.5 feet below the ceiling, and the louver on the opposite corner of the building with the invert 1 foot off the ground. This strategic placement of the fan and louver will create a cross draft to ventilate the entire room.

7.8 Water Conveyance and Supply

There are two conveyance lines that begin at the distribution structure—the Bellota conveyance pipeline and the Old Calaveras conveyance pipeline. The water conveyance design objective is to reliably provide conveyance to each of the diversions for the full range of flows at both the minimum and maximum Bellota pool WSEs.

7.8.1 Bellota Conveyance Pipe Alignment and Sizing

Three 54-inch-diameter steel pipelines will lead from the distribution structure, under the dam, and out to the south bank vault that connects to the existing 54-inch-diameter Bellota pipeline. Current flow demands to the existing 54-inch-diameter Bellota pipeline will require only one 54-inch-diameter steel pipeline to be operating. The two additional pipelines could be operated as needed or shut off using slide gates upstream. This will allow SEWD to build out the system as desired for future flow demands. Each pipe will be outfitted with a flexible pipe coupling near the upstream end of the pipe to account for any settling after construction. The flow through the pipes will be controlled by three motor-operated slide gates (Section 7.3.1). There will be a flow meter located in the existing 54-inch-diameter pipe leading out of the vault. There will be an additional motor-operated modulating slide gate at the existing 54-inch-diameter pipe leading out of the vault to control flow through the Bellota pipeline. Flow will be measured using an inline ultrasonic flow sensor downstream of the modulating slide gate.

A hydraulic profile was created for the pipelines to determine the expected WSE at the connection to the existing SEWD water supply pipeline, taking into account losses through the facility. Beginning with a forebay WSE of 121.44 feet, and using one pipe to convey 70 cfs through the facility, the hydraulic grade line elevation at the connection to the existing SEWD water supply pipeline was estimated to be approximately 118.6 feet. Using three pipes to convey the same flow, the hydraulic grade line was estimated at approximately 120.4 feet.

Under future conditions when using three pipes to convey 200 cfs, the hydraulic grade line elevation at the connection will be approximately 118.7 feet.

7.8.2 Old Calaveras Conveyance Pipe Alignment and Sizing

There will be two, 54-inch-diameter concrete pipes conveying water from the distribution structure to the Old Calaveras River. Each pipe will be outfitted with a flexible pipe coupling near the upstream end of the pipe to account for any settling after construction. The flow through the pipes will be controlled by two motor-operated slide gates (also see Section 7.3.1). There will be an inline ultrasonic flow meter located in each of the 54-inch-diameter pipes.

The pipes will reach the river at a nearly 90-degree angle. To prevent erosion of the river channel, a Calaveras outlet structure was designed. This structure was sized to be longer than the length of bend scour according to scour guidelines, in order to dissipate energy (NRCS 2007). A riprap apron was designed at the outlet of the structure to further dissipate energy (FHWA 2006). This gradation is summarized in Table 7-2.

Table 7-2. Proposed Riprap Apron Gradation and Layer Thickness

Location	Nom. D_{50} (in)	D_{15} (in)		D_{50} (in)		D_{85} (in)		D_{100} (in)	Blanket Thickness (in)	Weight Class	Description
		Min	**Max**	**Min**	**Max**	**Min**	**Max**				
Old Calaveras River Outlet Structure Apron	12	7.3	10.5	11.5	14	15.5	18.5	24	12	150-lb	NCHRP Class III

Filter and transition layers will also be designed for the rock slope protection. The filter layer described in Table 4-9 was found to be adequate for the rock slope protection. The transition layer will be a geotextile fabric with an apparent opening size of 0.2 mm (#70 sieve), placed over the filter layer and under the riprap (see Appendix C for calculations).

7.8.3 Operational Strategy

A summary of the target design flows for the Bellota and Old Calaveras River diversion structure is below in Table 7-3.

Table 7-3. Summary of Target Design Flows to the Bellota Pipeline and Old Calaveras River

Demand/Supply	Irrigation Seasonal Flows (cfs)	Non-Irrigation Seasonal Flows (cfs)
	April to October	**October 30 to March 31**
Bellota (current)	46-70	0-46
Bellota (future)	60-200	0-60
Old Calaveras River (current)	0-150	0-15
Old Calaveras River (future)	0-150	0-15
Total (current)	**46-220**	**0-61**
Total (future)	**60-350**	**0-75**

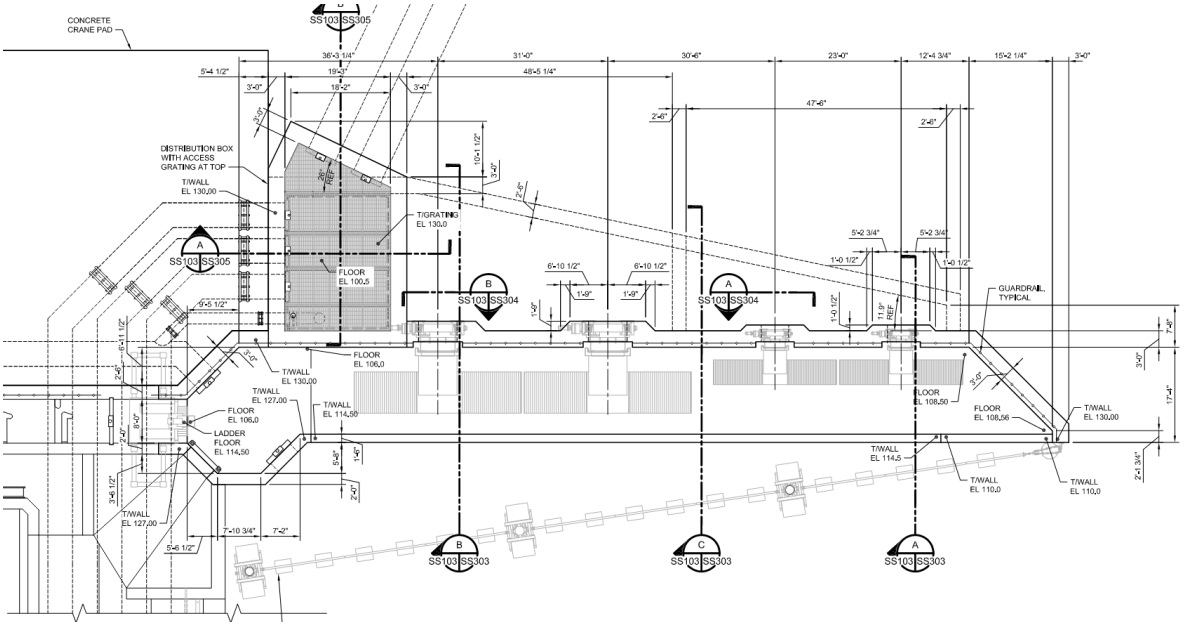
8 Structural Design

This section presents the basis of structural design for the intake structure including manifold, fish ladder, weir slab, pile foundation, soil improvement, stone column design, concrete masonry unit utility building, miscellaneous metals, grating, and guardrails. Detailed structural calculations are provided in Appendix G.

8.1 Intake Structure and Manifold

The intake structure will be constructed over pile supported concrete slabs with perimeter walls (Figure 8-1). The top of the walls will be at EL 130 feet NAVD88. The manifold structure will be similar to a concrete box culvert attached to the west wall of intake structure. The invert elevation of intake and manifold will change from EL 108.56 feet at the north side of the intake to EL 106.00 feet NAVD88 at the south side. The manifold height is changing from EL 118.5 feet to EL 122.00 feet NAVD88 at the middle. The manifold will be connected to the distribution structure on the south side. The top of the walls of distribution structures will be at EL 130.00 feet with the invert EL of 100.5 feet NAVD88. In front of the tee screens, the east intake top of wall will be at EL 110.00 feet for the upstream 20 feet and 114.5 feet NAVD88 for the remainder. The west wall, manifold, and distribution structures will support backfills and be designed for earth pressure in addition to all other loads such as hydrostatic and seismic loads. Valve flappers will be provided at the north wall of manifold at EL 115.00 feet NAVD88 to relieve the water pressure behind the wall. The load cases are discussed in Section 8.9).

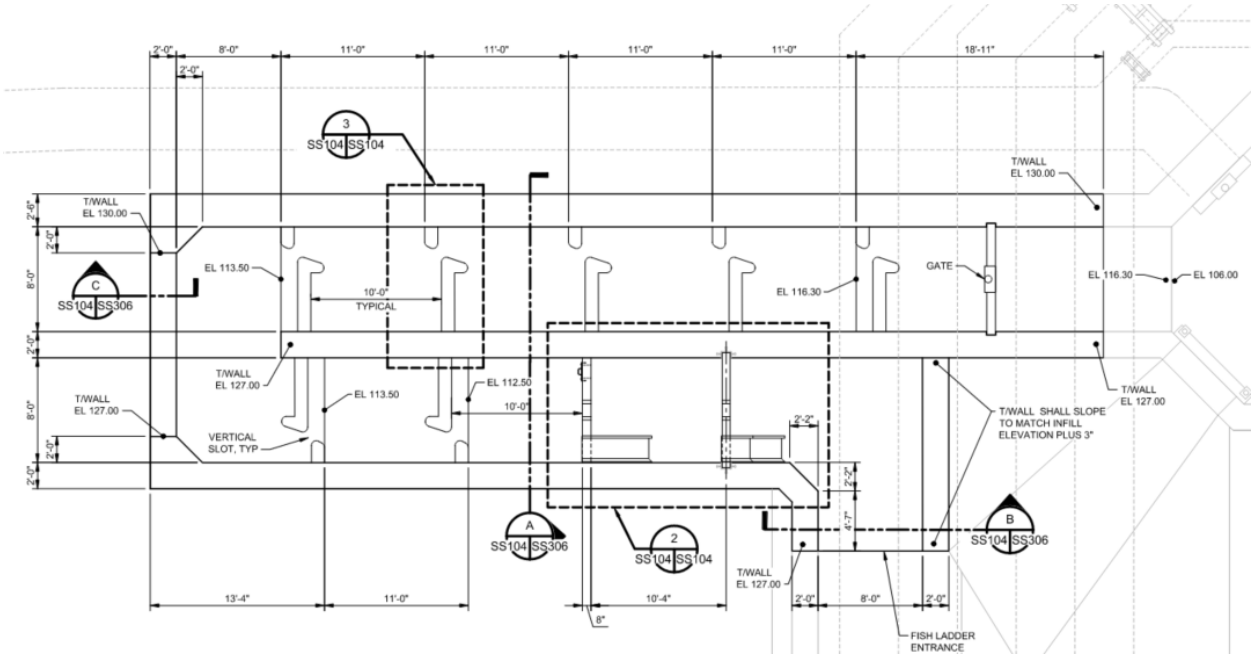
Figure 8-1. Intake Structure and Culvert Box



8.2 Fish Ladder Structure

Fish ladder structure will consist of slab and walls supported on a pipe pile foundation. The invert elevation of slab will start at EL 116.3 feet upstream and end at EL 112.5 feet NAVD88 downstream. The top of the north fish ladder wall will be at EL 130.00 feet and the top of all other walls at EL 127.00 feet NAVD88. The north wall is supporting backfill behind the wall at EL 130.00 feet NAVD88 and will be designed for lateral earth pressure in addition to other load cases discussed in Section 8.9. Figure 8-2 illustrates the layout of the fish ladder structure. Conservatively 8 feet head differential will be assumed on baffle wall for design during normal operation to take into account the debris blockage that caused the head differential on walls.

Figure 8-2. Fish Ladder Structure



8.3 Weir Slab

The concrete weir slab will be supported on pipe pile foundation. The slab width in cross canyon direction will be about 189 feet long to fit three 48.67-foot Obermeyer gates. The gate elevation on slab at the upstream side will change from EL 113.63 feet at right side to EL 114.63 feet NAVD88 at left side looking downstream. Figure 8-3 illustrates the weir slab plan view. As shown in section view, three 54-inch-diameter conveyance pipes will run underneath the entire length of the weir (Figure 8-4). The conveyance pipes will be supported by channel profiles crossing between pile foundations.

Figure 8-3. Weir Slab

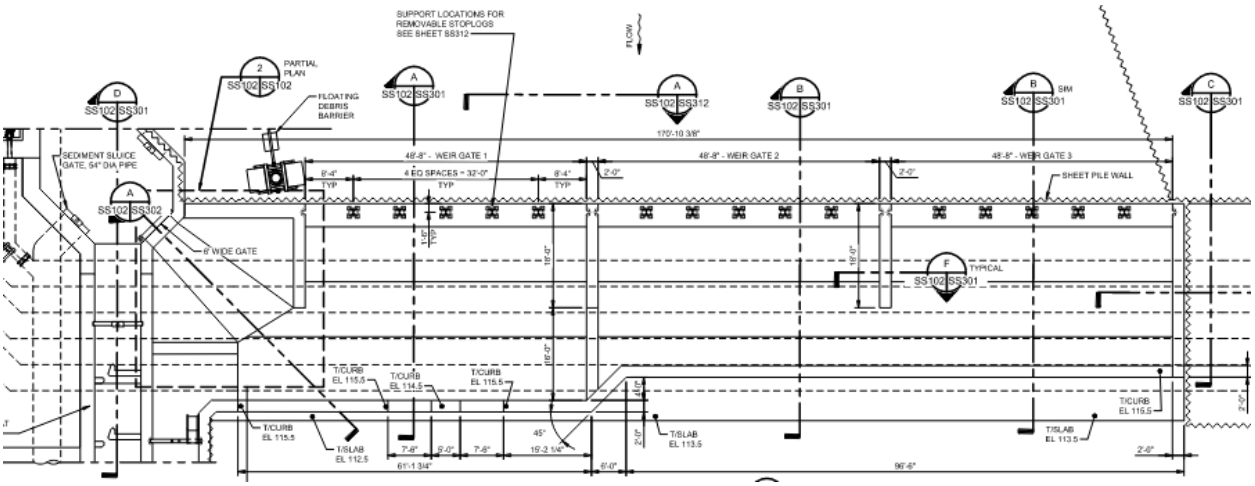
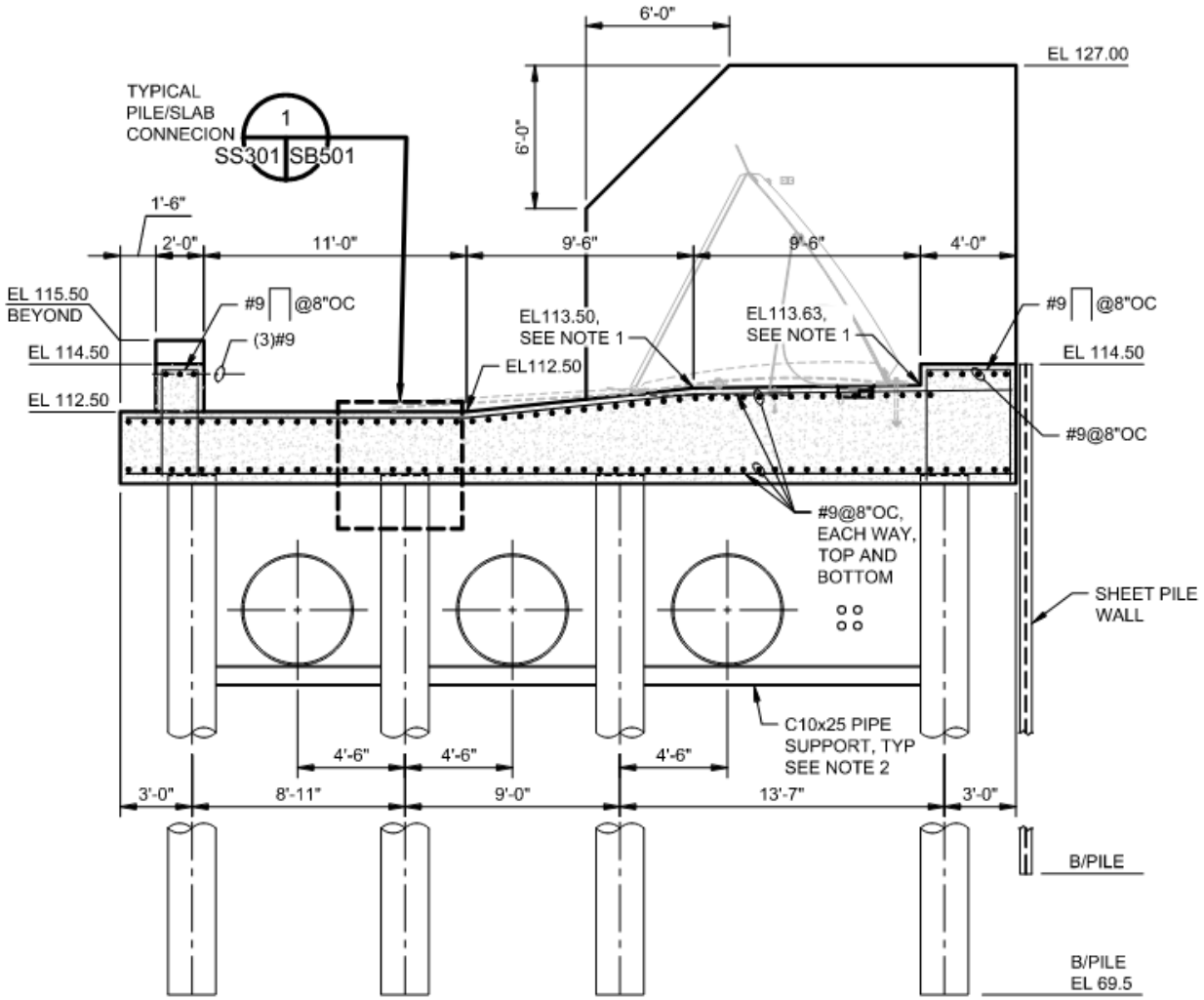


Figure 8-4. Weir Slab Section View



8.4 Structural Analysis

For a practical design, each structure will be individually considered for modeling and analysis as each cluster of piling system for intake, fish ladder, and weir slab behaves independently and therefore designed separately.

The structural walls and slabs will be modeled in SAP2000 program using shell elements. Spring elements will be defined at the pile-slab interfaces in horizontal and vertical directions. The horizontal spring stiffnesses will represent the flexibility of piles for transversal deformations and the vertical spring represents the axial stiffness of steel pipe pile. The earth pressure, hydrostatic and surcharge loads will be applied as static loads and the seismic inertia and added mass due to the hydrodynamic effect of water will be considered by running a response spectrum analysis. GROUP program by EnSoft Inc. will be used and developed to analyze the piles as group of elements to take into account the rigidity and group effects of all piles. Iterative process between GROUP program and SAP2000 program will be performed to find the actual seismic performance of structure and demand on piles considering the soil spring softening during the seismic lateral deformations of piles .

8.5 Applicable Codes, Standards, and Regulations

The different structural components of the intake, fish ladder, and weir slab structures will be designed in accordance with current USACE and industry standards, including:

- American Concrete Institute (ACI) 318-19, Building Code Requirements for Structural Concrete
- ACI 350.3-11, Code for Environmental Engineering Concrete Structures and Commentary
- ASCE 7-16, Minimum Design Loads for Buildings and Other Structures
- American Institute of Steel Construction Steel Construction Manual
- USACE standards:
    - EM 1110-2-2100, Stability Analysis of Concrete Structures
    - EM 1110-2-2104, Strength Design for Reinforced Concrete Hydraulic Structures
    - EM 1110-2-2502, Retaining and Flood Walls
    - EM 1110-2-2906, Design of Pile Foundations
    - EM 1110-2-6050, Response Spectra and Seismic Analysis for Concrete Hydraulic Structures
    - EM 1110-2-6053, Earthquake Design and Evaluation of Concrete Hydraulic Structures
    - Engineering Technical Letter 1110-2-584, Design of Hydraulic Steel Structures

- Engineering Regulation (ER) 1110-2-1806, Earthquake Design and Evaluation for Civil Works Projects
- ER 1110-2-8157, Responsibility for Hydraulic Steel Structures

8.6 Engineering Properties of Construction Material and Unit Weights

8.6.1 Concrete and Reinforcement Properties

The concrete used in the intake, fish passage, and slab weir structures will have a 28-day compressive strength of 5,000 pounds per square inch (psi). For design purposes, a concrete density of 150 pcf will be used. Standard American Society for Testing and Materials (ASTM) A615 Grade 60 reinforcing bars with a minimum yield strength of 60,000 psi and a minimum tensile strength of 90,000 psi will be used for all reinforcement. The modulus of elasticity for the steel will be 29,000 kips per square inch (ksi). All reinforcement will be provided with 4 inches of clear cover.

8.6.2 Unit Weights

Below is the list of unit weight and material properties that will be used in design:

- Water: 62.4 pcf
- Concrete: 150 pcf
- Structural Steel: 490 pcf

8.6.3 Geotechnical Design Inputs and Foundation properties

Geotechnical design inputs and foundation properties used in the structural design of the fish passage and ancillary structures are presented in Section 6.

8.7 Design Loads and Loading Summary

Design loads will be determined based on geographic settings and surroundings, material weight, code compliance, geotechnical data, and other operational requirements of the intake, fish ladder, and weir slab structures. Because the intake, fish ladder, and weir slab structures will not be considered a high hazard structure whose failure would result in loss of life in accordance with EM 1110-2-2100 and ER 1110-2-1806, they will be classified as a **Normal Structure**.

The design loads will include the self-weight of the structure; weight of the water on top of it, which would correspond to the flood event WSE for the load case chosen, if applicable; hydrostatic forces, including uplift; and lateral earth pressures, construction surcharge loads, and seismic loading, including both inertial and hydrodynamic effects.

- Dead Loads (D): are actual weights of material, self-weight of the structures and fixed service equipment loads.

- Live Load (L): are loads produced by the use or occupancy of the subject structure that do not include environmental or construction loads.
- Hydrostatic (F): Water pressures are included at various WSEs depending on the load case as described below.
- Lateral Earth Pressures (H) – lateral loads imposed on structures from the horizontal pressure exerted by soil. Loads will be applied in accordance with geotechnical recommendations for at-rest, active, and passive pressures as required.
- Hydrodynamic pressures (Hd): are calculated using the Westergaard method.
- Wave loading: is not considered for these structures
- Seismic and static lateral earth pressures were used per the earth pressure criteria provided in Section 6.
- Vehicle surcharge (construction surcharge) loading is calculated at 250 psf.
- Earthquake Loading (E) – The code-prescribed vertical and lateral forces induced on a structure from earthquakes. For this Project, the mapped Spectral Response acceleration parameter, short period (SS) is 0.490. The mapped spectral response acceleration parameter, 1 second period (S1) is 0.287. A soil Site Class of D has been assumed.
- Debris load was considered to design debris boom at flood surface elevation for the design of debris pile and connections. The load magnitude was obtained from mechanical.
- Wind (W) – The ASCE 7-16 code prescribes forces on a building or structure due to wind. Loads may be horizontal, vertical, or a combination of both. For this Project, a basic wind speed of 88 miles per hour will be used. A Risk Category of I will be used for all buildings and structures.

8.8 Load Condition Categories

Loads that the structure may encounter during its service life are grouped into three categories: Usual, Unusual, and Extreme, in accordance with EM 1110-2-2100. Each load category has a different expected performance requirement based on duration and frequency. Each category is associated with a return period based on the frequency of loading (Figure 8-5). All structures will be classified as Normal Structures as stated above.

Figure 8-5. Load Category versus Return Period

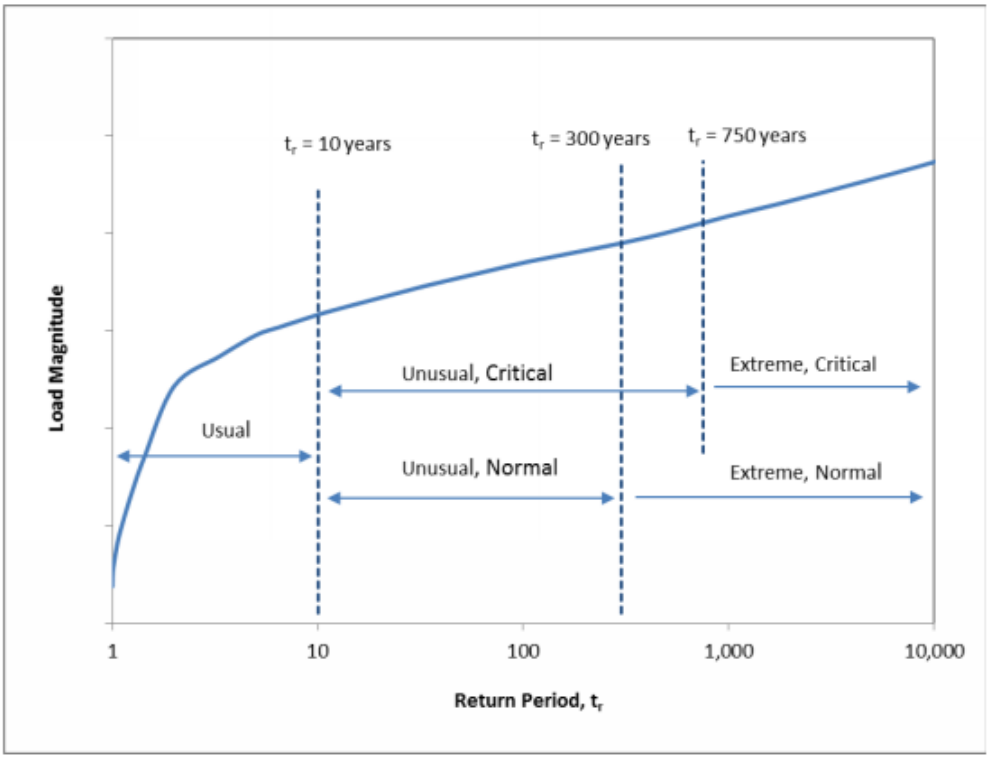


Figure 3-1. Load Category versus Return Period.

Source: EM 1110-2-2100 (USACE)

8.9 Load Cases

The following load cases will be considered for fish passage and ancillary structures for the 90 percent design phase.

8.9.1 Load Case 1 – Usual Load Condition – Less than 10-Year Flood Event

- Structure completed
- Maximum Normal WSE at 121.4 feet NAVD88 in the Bellota Pool, behind the gates, inside intake and manifold
- Groundwater elevation behind the manifold and fish ladder is assumed at 115.00 feet NAVD88
- Fish ladder channel is in normal operation and WSE starts from 121.4 feet NAVD88 at upstream to 118.3 feet NAVD88 at downstream
- Lateral soil pressure and hydrostatic pressures applied on walls by considering the water elevations
- Uplift pressure applied on the slabs of intake, fish ladder, and weir

8.9.2 Load Case 2 – Unusual Load Condition – 100-Year Flood Event

- Structure completed
- WSE at 124.2 feet NAVD88 in the Bellota Pool, inside intake
- Gates are in open position
- Fish screens are up, and manifold and distribution structures empty
- Groundwater elevation behind the manifold and fish ladder is assumed at 124.2 feet
- Fish ladder gate is closed and the channel empty
- WSE behind all fish ladder walls are at 124.2 feet NAVD88
- Lateral soil pressure and hydrostatic pressures applied on walls by considering the WSEs
- Uplift pressure applied on the slabs of intake, fish ladder, and weir

8.9.3 Load Case 3 – Extreme Load Condition – 500-Year Flood Event

- Structure completed
- WSE at 127.0 feet NAVD88 in the Bellota Pool, inside intake
- Gates are in open position
- Fish screens are up, and manifold and distribution structures empty
- Groundwater elevation behind the manifold and fish ladder is assumed at 127.00 feet NAVD88
- Fish ladder gate is closed, and the channel empty
- WSE behind all fish ladder walls are at 127.00 feet NAVD88
- Lateral soil pressure and hydrostatic pressures applied on walls by considering the water elevations
- Uplift pressure applied on the slabs of intake, fish ladder, and weir

8.9.4 Load Case 4 – Extreme Load Condition – Maximum Design Earthquake

- Structure completed
- Maximum Normal WSE at 121.4 feet NAVD88 in the Bellota Pool, behind the gates, inside intake and manifold
- Groundwater elevation behind the manifold and fish ladder is assumed at 115.00 feet NAVD88
- Fish ladder channel is in normal operation and water elevation starts from 121.4 feet NAVD88 at upstream to 118.3 feet NAVD88 at downstream

- Lateral soil pressure and hydrostatic pressures applied on walls by considering the normal water elevations
- Uplift pressure applied on the slabs of intake, fish ladder, and weir
- Maximum design earthquake seismic inertial load in horizontal direction (Code defined response spectrum)
- Maximum design earthquake seismic in vertical direction use 0.84 conversion factor for vertical response spectrum (based on EM 1110-2-6053). This will be considered in 90 percent design submittal
- Hydrodynamic added mass of water on north wall intake structure, upstream walls, and weir gates
- Seismic increment soil pressure on walls including manifold structure

Construction load cases will not be considered for these structures as the structures will be designed for higher load factors for normal loading and enough shoring will be assumed provided by contractor during construction for corresponding vehicular loadings.

8.10 Load Combinations

Load combinations will follow factored loads for strength design in accordance with Table 3-1 in EM 1110-2-2104. The summary of load combinations used for this strength design of concrete components is presented in Table 8-1.

Table 8-1. Strength Design Load Combinations

Load Case	Load Description	Load Category	Factored Loads
R1	Normal Operating	Usual	$2.2(D + EH + EV + Hs_U)$
R2A	Normal Operating + Surcharge	Unusual	$1.6(D + EH + EV + Hs_U + ES_N)$
R2B	Normal Operating + Short Duration Water Load	Unusual	$1.6(D + EH + EV + Hs_N)$
R3A	Normal Operating + OBE	Unusual	$1.5(D + EH + EV + Hs_U + EQ)$
R3B	Normal Operating + MDE	Extreme (Strength)	$(1.2 \text{ or } 0.9)D + (1.5 \text{ or } 0.5)EH + (1.35 \text{ or } 1.0)EV + 1.0\ Hs_U + 1.25EQ$
R4	Normal Operating + Maximum Water	Extreme (Strength)	$(1.2 \text{ or } 0.9)D + (1.5 \text{ or } 0.5)EH + (1.35 \text{ or } 1.0)EV + 1.3Hs_X$

Source: EM 1110-2-2104
Where: D: Dead Load; EH: Lateral Earth Pressure; EV: Vertical Earth Load; ES: Surcharge Load; HS_N: Normal Hydroststaic Load; HS_U: Hydroststaic Load during Flood; EQ: earthquake Load

8.11 Seismic Design

Based on USACE ER 1110-2-1806 and USACE EM 1110 2-6503 codes and review of the hydraulic structures classifications, the fish passage and ancillary structures will not be considered a high hazard structure whose failure would result in loss of life. Therefore, the structures will be classified as a Normal Structure.

The proposed structures should be designed to resist the lateral forces generated by earthquake shaking in accordance with local design practices. The seismic design parameters based on the ASCE 7-16 as modified by Supplement #1 and the 2019 California Building Code for project location (Latitude 38.052325 and Longitude -121.011499) are presented in Table 8-2. This earthquake will be considered as maximum design earthquake for extreme load cases.

Table 8-2. Considered Seismic Parameters

Parameter/Coefficient Description	Parameter/Coefficient	Value
MCE_R for 0.2 sec Spectral Response Acceleration, Site Class D	S_s	0.490g
MCE_R for 1.0 sec Spectral Response Acceleration, Site Class D	S_1	0.287g
Soil Profile Type	Site Class	D
Site Coefficient	F_a	1.4
Site Coefficient	F_v	2.15
Adjusted MCE Spectral Design Parameters	S_{MS}	0.690g
	S_{M1}	0.480g
Design Spectral Acceleration Parameters	S_{DS}	0.460g
	S_{D1}	0.320g
Seismic Design Category	Occupancy Category I to IV	III

Source: Office of Statewide Health Planning and Development Seismic Design Maps https://seismicmaps.org/

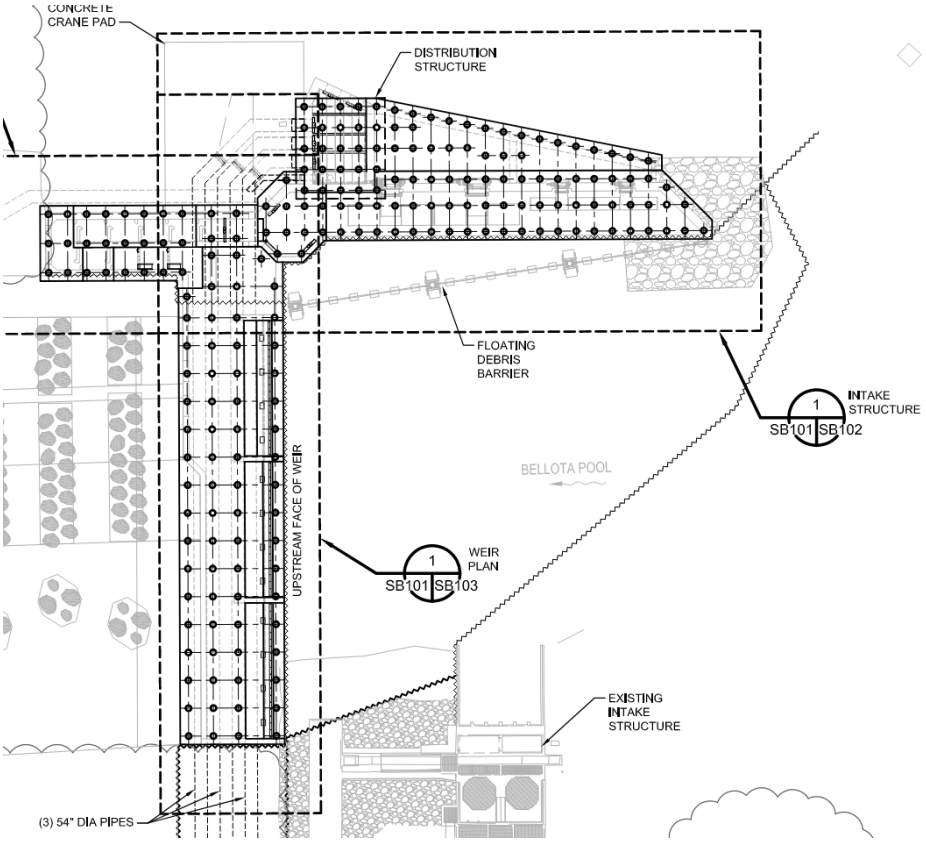
8.12 Pile Foundation Design

Based on the geotechnical report, the site is comprised predominately of loose to medium dense sands silts and gravels above EL 90 feet NAVD88. Below EL 90 feet the soils consist of alternating layers of medium dense to hard silts and silty sands. The soil composition of the upper soils meets the potentially liquefiable criteria presented in Seed et al. (2003). The liquefaction triggering analyses confirmed the 2005 investigation finding that the south bank contains soils with high potential for liquefaction and, if not mitigated, would result in seismically induced surface settlements from soil layers that have factors of safety against liquefaction between 1.0 and 1.3. These layers range in thickness from 5 to 10 feet and are located between EL 100 and 115 feet NAVD88, 10 to 20 feet below the ground surface. Analysis of the current investigation of the north bank found that the site has less potential for liquefaction, but would experience seismically induced settlements due to layers with factors of safety against liquefaction between 1.0 and 1.3. Further investigation by the geotechnical team showed that due to wide variation in calculated settlements and the proximity of the proposed structures to the river, differential settlements are anticipated to be significant. The structures should be supported on deep foundations to gain support in the stiff to hard soils of the older alluvium, below EL 90 feet NAVD88.

Based on the geotechnical findings, the intake, fish ladder, and weir slab will be designed to be supported on 24-inch-diameter plugged steel pile with 0.5-inch wall thickness.

Figure 8-6 shows the pile layout proposed for design, which will meet both bearing capacity due to vertical loads and lateral capacity due to seismic loads. The bearing capacity is a function of driven pile depths provided by the geotechnical engineer. The lateral capacity of piles will be analyzed according to the soil layer parameters provided by the geotechnical engineer and used in the GROUP program to evaluate lateral performance of piles under seismic loads. Due to lateral flexibility of piles, the dynamic response of structures will be the function of pile deformations, therefore an iterative process will be followed between pile lateral analysis and structural analysis to find the actual performance considering pile/structure interaction under seismic loads. Depending on the pile spacing, group effects of piles will be included in the geotechnical analysis and specified individually to each pile in GROUP models.

Figure 8-6. Pile Foundation



8.13 Sheet Pile Design

The sheet pile walls will be used on different locations at the site for seepage and structural retaining wall purposes (Figure 8-7). As shown in Figure 8-7, the sheet pile walls S1, S2 and S3 will be used for seepage purposes. The sheet pile walls S4 and S5 will be used for soil retaining purposes.

Figure 8-7. Sheet Pile Wall

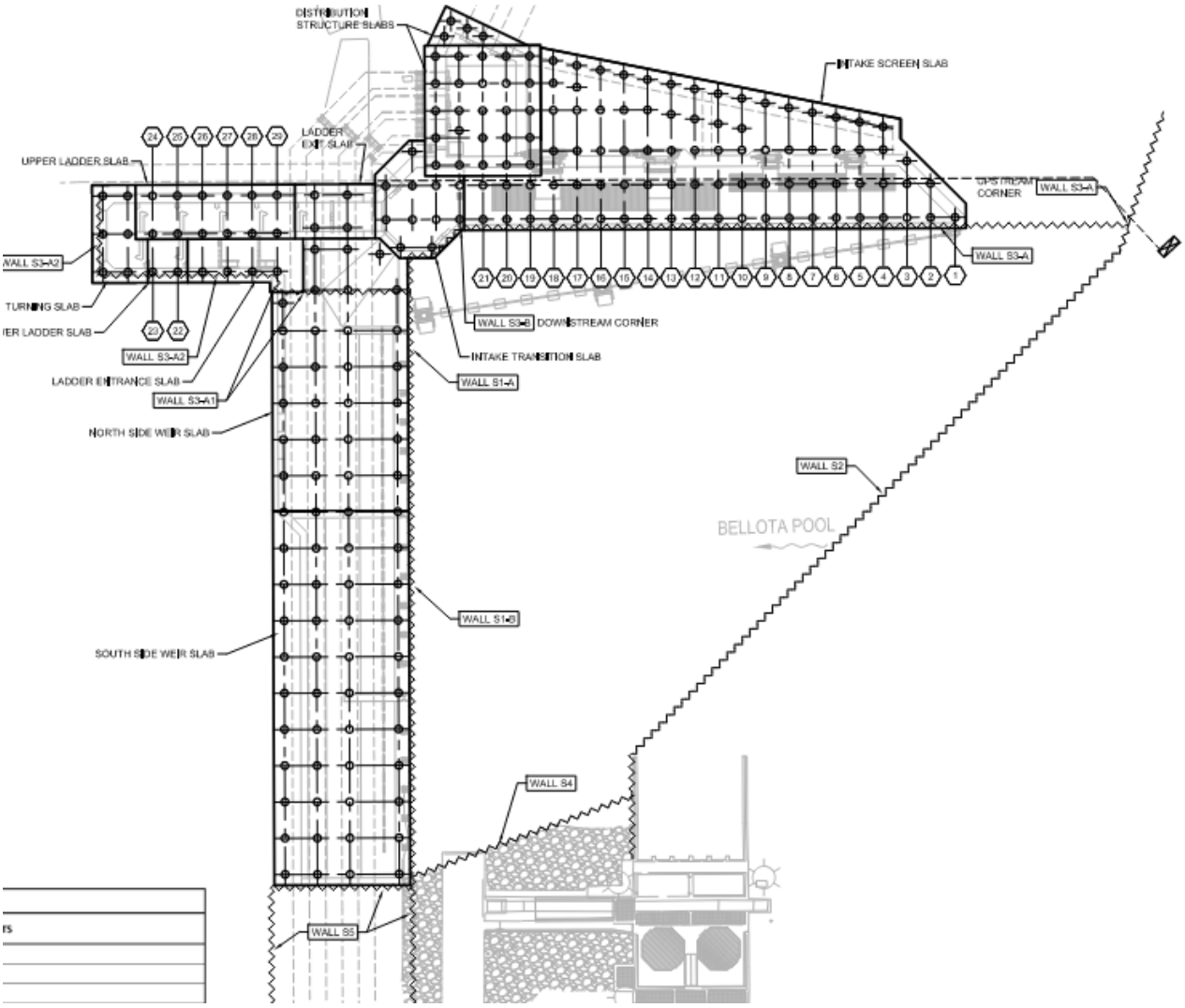
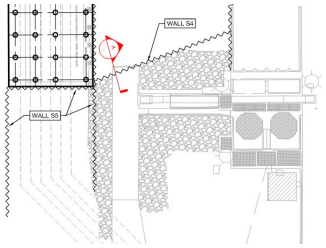


Figure 8-8 shows the cross section grading for sheet pile wall S4. The soil elevation at the left bank looking downstream at 124 ft and the soil elevation at right bank looking downstream will be 113 feet. The WSE at both sides was assumed 115 feet for normal and seismic load case and 124.5 feet for 100-year flood case. Due to high soil level behind the walls using single sheet pile alone will be too flexible ended to use too large size sheet pile; therefore, a combination of sheet pile and HZ pile will be used to provide more rigidity and strength. LPile program developed by EnSoft Inc. will be used to analyze Wall S4.

Figure 8-8. Grade Profile for Sheet Pile Wall S4



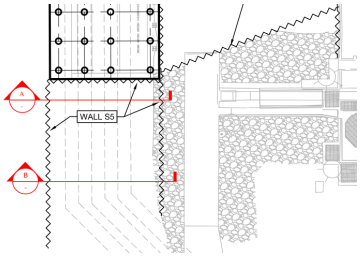
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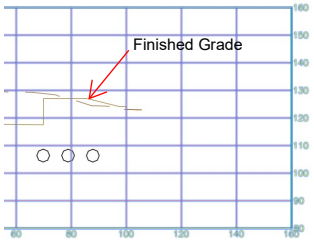
Section A-A

Figure 8-9 shows the cross section grading for sheet pile wall S5. The soil elevation at upstream face will be 127 feet and the soil elevation at downstream face will vary from 117 feet to 127 feet. To design the wall, the average soil height at the downstream face will assume EL 120 feet. The groundwater on both side of the wall will assume EL 115 feet. CWALSHT program developed by USACE will be used to analyze sheet pile Wall S5 and. According to EM-1110-2-575, safety factor of 1.5, 1.3 and 1.1 will be used for rotational stability analysis under Normal, Flood and Seismic load cases respectively for Wall S5.

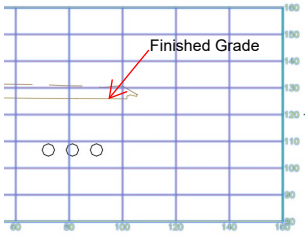
Figure 8-9. Grade Profile for Sheet Pile Wall S5



Plan



Section A-A



Section B-B

A 250 psf surcharge live load surface pressure will also be considered with normal load case for both S4 and S5 Walls. According to the soil input parameters and required safety factor, CWALSHT program computes the minimum penetration depth to achieve the acceptable rotational stability and LPile program provided the generated deformation along the pile to provide enough penetration. Extracted from EM 2502-2020 (Draft Version), the information in Table 8-3 was also considered to limit the deflection of wall at ground surface for different loading levels.

Table 8-3. Maximum Deflection of Wall at Ground Surface

Loading Category	Maximum Deflection
Usual	0.5 in
Unusual (100-Year Flood)	1.5 in
Extreme	3.0 in

Both CWALSHT and LPile programs also computed the developed bending and shear distribution along the wall to perform strength design of walls. According to EC 1110-2-

6066, a single load factor of 2.2, 1.6 and 1.0 will be assumed for strength design of wall for Normal, Flood and Seismic load respectively.

8.14 Strength Design of Intake, Fish Ladder, and Weir Slab Structures

Strength design of the structures will be performed in accordance with EM 1110-2-2104 (USACE 2016) to determine the required reinforcement and thickness of the reinforced concrete elements.

The structural design of the structures will include the finite element analysis and results, and spreadsheets for loading and structural element design.

All of the concrete structures will be included in three-dimensional models in the SAP2000 finite element analysis program. Walls and slabs will be modeled with thick shell elements able to consider the shear deformations along the thickness of each element. Piles will be modeled with nodal springs (Section 8.12). The models will then be loaded by lateral soil, hydrostatic, hydrodynamic, live surcharge, and self-weight dead loads. In addition, the code-specified seismic spectra provided in Section 8.11 is used to apply seismic inertial loads to walls and slabs. Additional seismic mass will be included on the north wall of the intake to include the hydrodynamic effect of water for seismic load condition.

SAP2000 outputs will be exported for piles (nodal springs), walls, and slabs, including out-of-plane shear forces and flexural moments for wall and slab elements. The number of piles and their spacing will be designed such that their axial and lateral factored demand is less than their capacity. Wall, roof, and slab thicknesses will be designed to resist their maximum out-of-plane shear forces without the need for shear reinforcement. Flexural reinforcement will be calculated for each face and each direction to resist the maximum obtained design forces. Both the shear and flexural capacities will be calculated per ACI 318-19 using a load and resistance factor design method, with the assumption of 4-inch concrete cover for reinforcement per EM 1110-2-2104 Section 2.7. Flexural reinforcement will be designed for each face of wall and slab, along each way, to resist flexural moments. All reinforcement and wall thicknesses will be designed to make sure that the demand/capacity ratio is less than 1.0 once all load factors and strength reduction factors have been applied. Minimum temperature and shrinkage reinforcement will be considered per EM 1110-2-2104 Section 2.9.

Figure 8-10 shows the finite element model created for the intake and manifold structures.

Figure 8-10. Finite Element Model of Intake and Manifold Structures

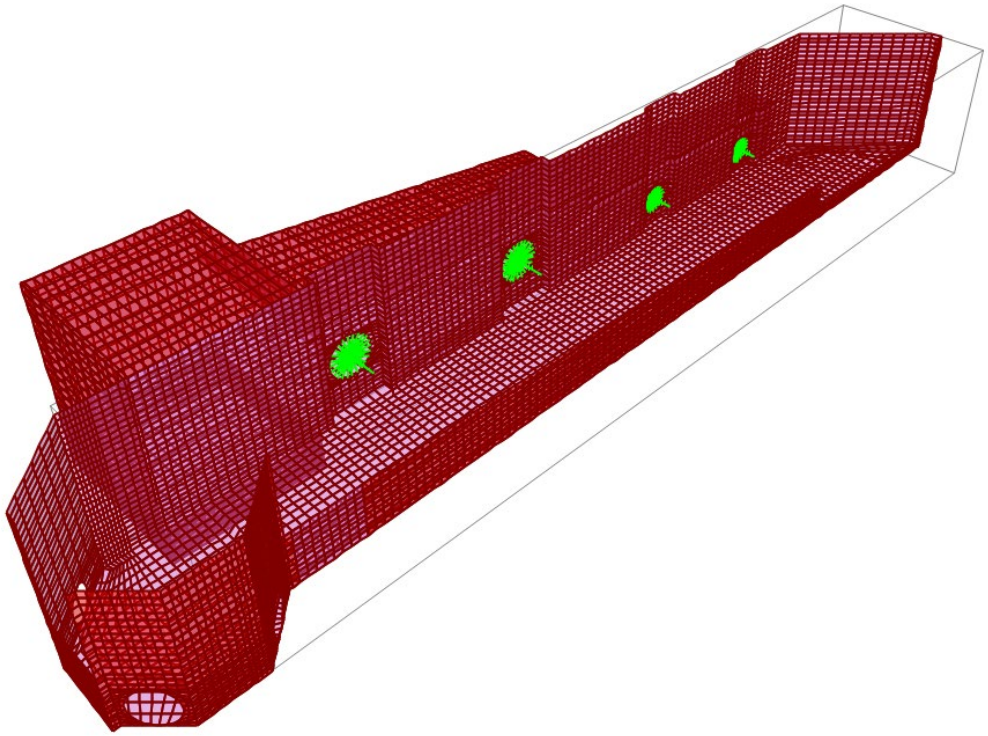


Figure 8-11 shows the soil pressure applied on the north wall of the intake structure.

Figure 8-11. Lateral Soil Pressure (psf) on Intake Structure

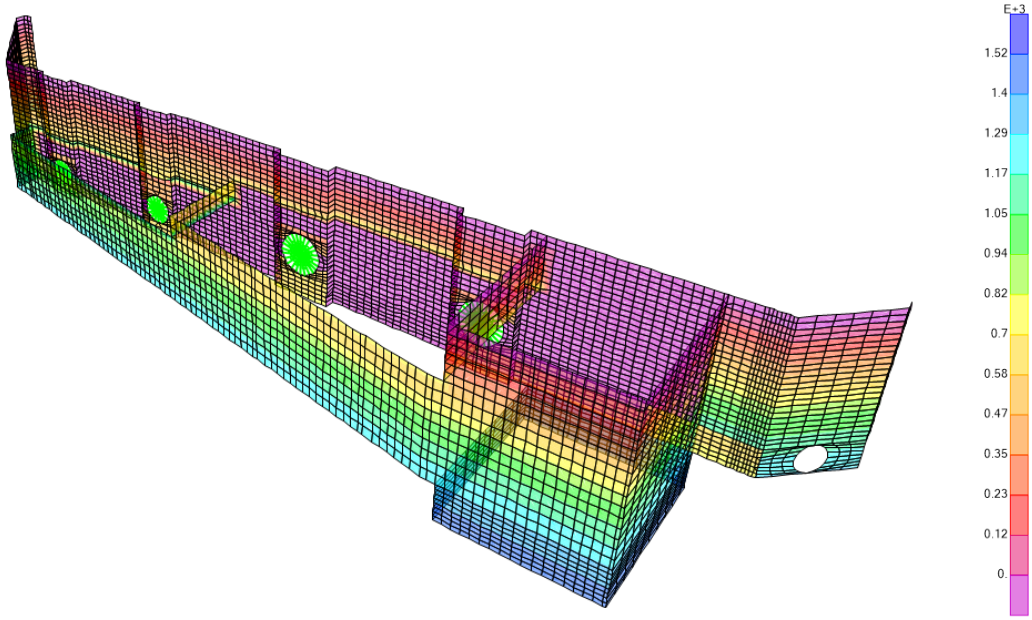


Figure 8-12 shows the finite element analysis model of fish ladder structure and Figure 8-13 shows the lateral soil pressure on the north wall of the fish ladder structure.

Figure 8-12. Finite Element Model of Fish Ladder Structure

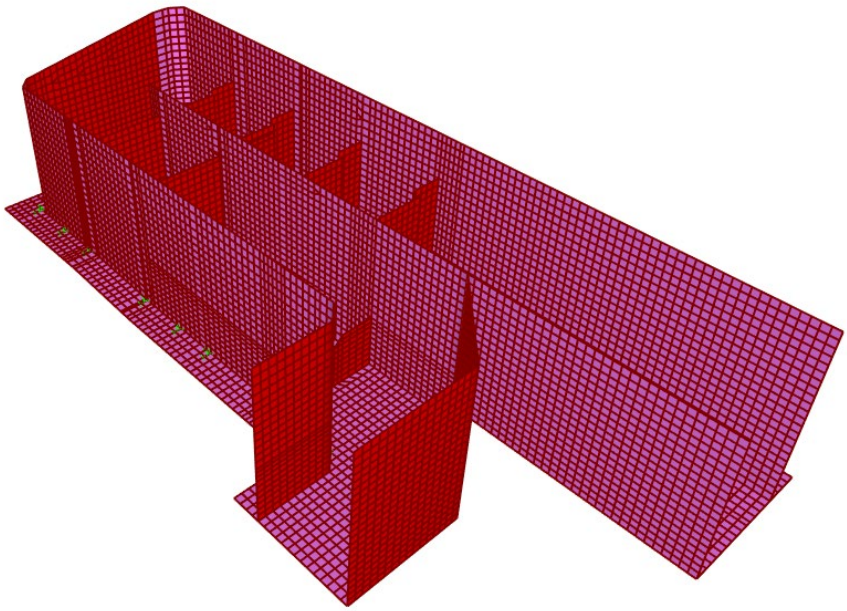


Figure 8-13. Lateral Soil Pressure (psf) on North Wall of Fish Ladder

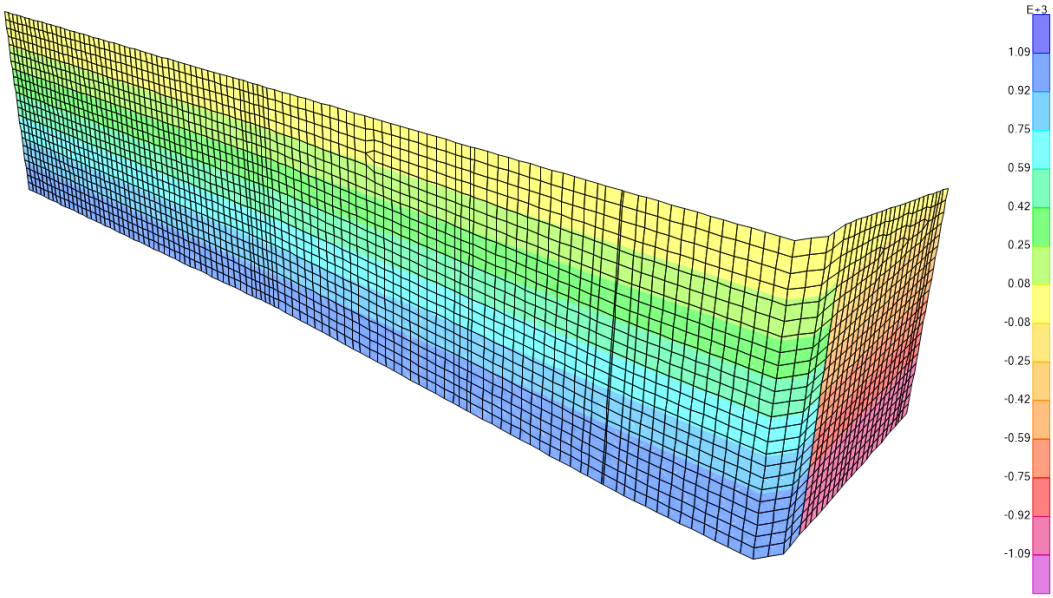


Figure 8-14 shows the finite element model of the weir slab. The effect of hydrostatic and hydrodynamic loads from gates will be applied as point loads located at the gate anchorage on upstream face of dam. Hydrodynamic added mass will also be applied on upstream walls at right and left of weir slab. Lateral soil pressure will be applied on south shore diversion structures (Figure 8-15).

Figure 8-14. Finite Element Model of Weir Slab Structure

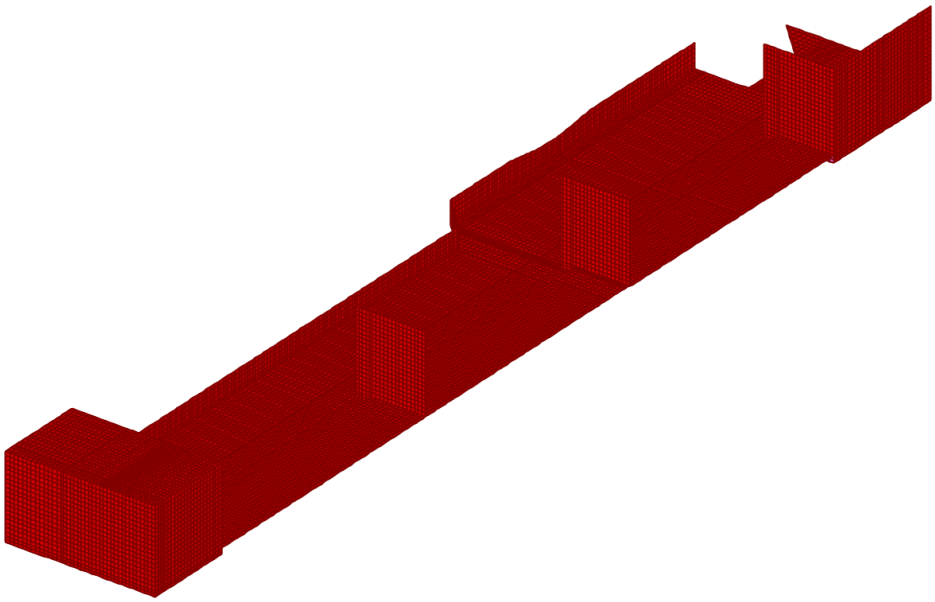
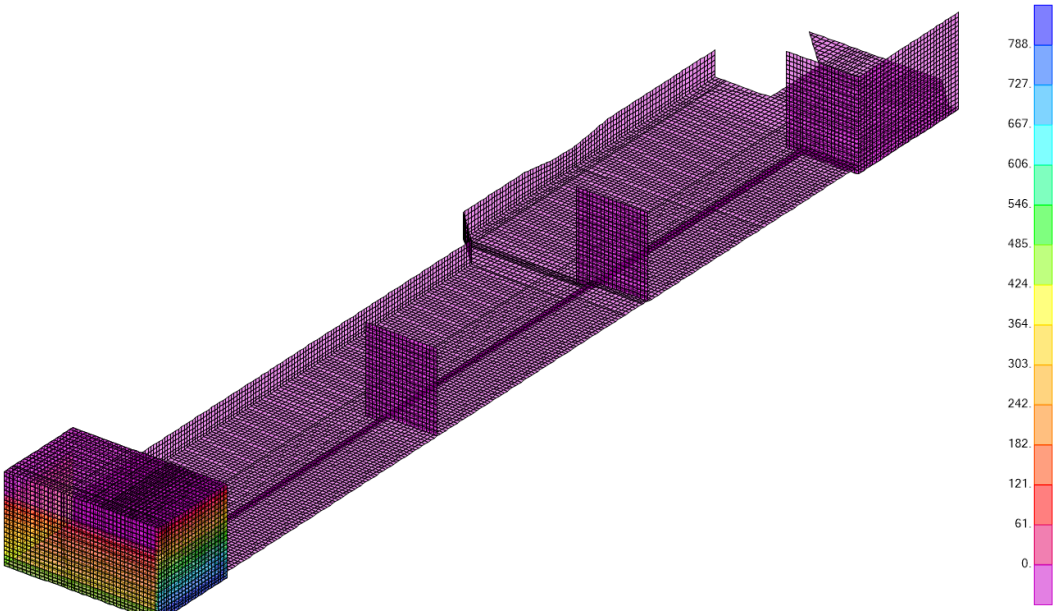


Figure 8-15. Lateral Soil Pressure (psf) on Diversion Structure



8.14.1 Gate Anchorage Design

The Obermeyer gate's anchorage to the weir slab will be designed for normal and extreme load cases mobilized from gate to anchors. The concrete break out capacity and strength of steel anchors will be checked using ACI 318-19 Chapter 17.

8.15 Control Building

A small, 28-foot by 20-foot utility building will be located near the Calaveras River. The building will be constructed of reinforced concrete masonry units and founded on reinforced concrete walls and spread footings. Footings will be a minimum of 2 feet below grade for frost protection. The roof will be constructed using pre-manufactured metal-plate-connected wood trusses with APA-rated structural sheathing. A roof slope of 12H:4V will be used to reduce the snow accumulation. A concrete pad will be provided for installation of the switchboard and AC generator. There will also be an antenna pad located at southeast corner of building. The pad will be a 16-foot by16-foot, 4-foot-thick concrete slab to support a 100-foot-tall antenna.

8.15.1 Design Description

Structural Design

The utility building will be subjected to forces imposed by gravity, live load and equipment, wind, snow, and seismic activity. Structural design of the building and its components will be performed to resist these forces as prescribed by the provisions stated in the Building Code. Masonry design will conform to The Masonry Society 402-13, ACI 530-11, ASCE 5-11 Building Code Requirements and Specification for Masonry Structures.

Design Loads and Combinations

Design loads will include loads such as dead loads, super-imposed dead loads, live loads, wind loads, seismic loads, and snow loads. Design loads will be determined based on the requirements provided in ASCE 7-16 and then combined per load combinations listed in Chapter 2 of ASCE 7-16.

8.16 Miscellaneous Metals

Miscellaneous metals will include fabricated items such as embedded guides, stairs, checkered plate, bollards, ladders, hatches, supports, lintels, and other similar items. The design and fabrication of these items will be provided in specification Section 05 50 00.

8.16.1 Design Description

The design of items that fall under this category will be per International Building Code requirements for the material selected. Design loads will be determined per ASCE 7-16 and the applicable load combinations applied. The item will then be designed to resist the prescribed loading.

Protective Coatings

Miscellaneous metal items will typically be fabricated using stainless steel, aluminum, or hot-dipped galvanized steel depending on the location used and design requirements.

8.17 Grating

Grating will be used in access walkways to the fish screens, stoplogs, trashrack, and other areas of the facility. Aluminum bar grating will be used to provide the strength, stiffness, safety, and durability prescribed by the International Building Code. Walkways will be designed and constructed with 4-inch-tall toe boards and 42-inch-tall guardrail as measured from the top of the walking surface. A serrated surface will be specified to provide a non-slip surface.

8.17.1 Design Description

Design Loads and Combinations

A uniform live load and concentrated load of 40 psf and 300 pounds, respectively, will be used for the bar grating design with the loading condition that produces the maximum force.

8.18 Guardrails

Guardrails will be used at all locations where fall protection is required. This will include all walkways and tops of retaining walls where a height difference greater than 30 inches exists. Aluminum will be used for all guardrails, handrails, and connection hardware for durability and corrosion resistance.

8.18.1 Design Description

Guardrail and handrail systems will be designed to resist a single concentrated load of 200 pounds applied in any direction at any point on the handrail or top rail with this force being transferred into the structure to provide the maximum design load effect. Rails will also be designed to resist a uniform force of 50 pounds per foot applied in any direction along the handrail or top rail.

8.19 Hoist Gantry

A hoist gantry will be installed at the intake to provide access for removal of the over-under gate located at the juvenile bypass for necessary maintenance and sediment sluicing. The hoist gantry will be a fabricated support structure with an electric-actuated hoist to lift the gates off of the bypass. The hoist gantry will be sized to lift the over-under gate within the juvenile bypass.

9 Electrical Design

9.1 Electrical Design References and Standards

The work will be performed in accordance with the latest provisions of codes and standards published by the following organizations:

- American National Standard Institute
- ASTM
- California Code of Regulations, Title 24, Part 1 - California Administrative Code
- California Code of Regulations, Title 24, Part 3 - California Electrical Code
- Electrical Testing Laboratories
- Insulated Cable Engineers Association
- Institute of Electrical and Electronic Engineers
- International Electrical Testing Association
- National Electrical Code (NEC)
- National Electrical Manufacturers Association (NEMA)
- National Electrical Safety Code
- National Fire Protection Association (NFPA) 70E
- National Institute of Standards and Technology
- Underwriters' Laboratories
- Occupational Safety and Health Administration

9.2 Electrical Service

The electrical system design for the Project will incorporate the requirements of the latest NEC - NFPA 70, applicable provisions of the California Electric Code, Pacific Gas & Electric (PG&E) Service Requirements (TD-70001) Greenbook Standards, and District Standards and Ordinances, as applicable.

9.2.1 Temporary Power Service

The existing Bellota Weir northern site is serviced by a pole-mounted transformer that provides power to two service meters mounted on the exterior wall of a block house. The service meters are 120VAC, 2W, 60HZ, 100A meters used for SEWD stream gage and fish monitoring equipment.

As part of the initial phase of construction, the existing service meters will be removed and replaced with a single 120/240VAC, 4W, 60HZ, 200A (min.) service meter and distribution panel that will provide temporary power for fish monitoring equipment and construction services.

9.2.2 Permanent Power Service

The new electrical equipment will be serviced from existing 12KV overhead power lines located northeast of the Bellota Site. Based on site observations, overhead power lines from PG&E are located adjacent to the Old Calaveras River. The primary conductors will riser down to underground and be routed underground to an on-site PG&E-furnished, outdoor-rated transformer.

The PG&E transformer will connect to the PG&E metering cabinet inside the proposed NEMA 3R Switchboard rated for the 480Y/277-VAC, three-phase, four-wire distribution system, with 600-ampere rated bussing and automatic transfer switch (ATS) for standby power provisions.

The ATS will have an emergency power source connection from a permanent 400 kilowatt, 480-volt (V), three-phase, four-wire generator located outdoors and adjacent to the Control Building.

A PG&E electric service application will be submitted to initiate service planning. The costs associated with the electric service will be arranged and paid for by SEWD as determined by the PG&E service estimation department.

9.3 Electrical Distribution

9.3.1 Low-Voltage Switchboard

A NEMA 3R rated switchboard will be installed outside of the Control Building. Cabling will be insulated and rated for 600V. Feeders powered from the switchboard will be 480V, three-phase and all equipment will be rated to handle short circuit currents equal to or greater than the available fault current.

The switchboard will feature power overcurrent protection main breakers with solid-state trip units with long-time, short-time, instantaneous, and ground-fault protection functions. Distribution breakers will be molded case type with solid-state adjustable trip units to improve coordination and downstream arc flash characteristics.

A surge protective device will be installed with a minimum surge rating of 240 kiloamperes (kA) per phase. The total calculated load for the site shall not exceed 80 percent of utility service or service panel rating.

9.3.2 Low-Voltage Motor Control Center and Starters

A NEMA 1 rated motor control center (MCC) will be installed in the Control Building. MCC power monitoring units will have ethernet network protocol to communicate with the supervisory control and data acquisition (SCADA) system. Control cabling installed within the MCC will be insulated and rated for 600V.

Feeder circuits powering the MCC originating from the switchboard feeder breaker will be 480V, three-phase, three-wire. The MCC will have a tin-plated copper phase bus and copper ground bus. The MCC will be a single bus configuration rated to handle short circuit currents equal to or more than the available fault current.

The MCC main breaker will be molded case type with solid-state trip units. The MCC main breaker will also be provided with ground-fault protection when required by the NEC. Distribution breakers on the MCC will be molded case type with solid state or thermal-magnetic trip units.

The MCC will measure 20 inches deep. The MCC shall be equipped with a minimum of two 12-inch spare spaces above and beyond that anticipated for any future loads.

Except for packaged equipment and HVAC equipment, motor starters will generally be located within the MCC. Starters will include a GREEN indicated lamp for RUNNING (ON), a RED indicating lamp for OFF, and an AMBER indicating lamp for trouble or FAILURE (where applicable). All pilot lamps will be push-to-test type.

9.3.3 Motors and Variable Frequency Drives

Motors will be specified with NEMA Premium high efficiency ratings. Motor enclosures will be suitable for the environment in which they are installed. All motors driven from variable frequency drives (VFDs) will be inverter-duty rated and rated for such applications. The heaters will be designed to operate on 120VAC power from the associated motor starter or VFD. VFDs will be pulse width modulated type and fed from a dedicated 480V, three-phase feeder. Drives for motors smaller than 75 horsepower will be 6-pulse type drive.

9.3.4 Power Transformers

Transformers that supply 208Y/120 volts or 240/120 volts for lighting requirements will be dry type with copper windings and suitable for the area located. Transformers will be industry standard sizes.

9.3.5 Panelboards

Power distribution panelboards or power centers, if required, will be 480V, three-phase, with a copper bus and main circuit breaker. Lighting panelboards will be 208Y/120V, three-phase, four-wire type with a copper bus and a main circuit breaker sized to match the lighting transformer capacity. Each panelboard will have a minimum of 20 percent of its capacity reserved for spare breakers with spaces, bus work, and terminations to complete the standard sized panelboard. Surge protective devices will be provided as integral to each panel assembly, when specified on electrical design.

9.3.6 Receptacles

Convenience receptacles for general service will be located on walls. Provisions for receptacles at all air conditioning units and air handling units will be made as required by NEC. Convenience receptacles will generally be mounted 18 inches above floors, except outdoors or in rooms where equipment could be hosed down. Receptacles in such areas will be mounted 36 inches above the floor or grade.

Weatherproof in-use receptacles will be utilized outdoors, and in wet and damp locations. Receptacles installed outdoors will be provided with ground fault circuit interrupting and self-test capability.

9.3.7 Raceways and Cable

Specific types of raceways will be chosen for locations vulnerable to damage by moisture, extreme temperature, or corrosion, and considering voltage and cost. An underground duct bank consisting of direct buried schedule 40 polyvinyl chloride (PVC) type conduits for non-traffic areas and concrete-encased schedule 40 PVC type conduits for traffic areas will be provided for most circuits that are routed outside of buildings. Duct banks will include spare conduits. The following systems will be separately grouped in duct banks:

- Power and discrete control wiring 600V and below.
- Process instrumentation analog and communication wiring, including 24V discrete signals, and 600V rated CAT6 for ethernet communication.

The following general guidelines will be used for raceway sizing, selection, and installation:

- Conduit will be sized for XHHW-2 insulation for all conductors 600V and less.
- The minimum diameter of all conduit will be 1 inch.
- Raceways in duct banks will not be smaller than 1 inch.
- Exterior Conduit Installation:
    - Exposed conduit will be PVC-coated galvanized rigid steel.
    - Underground, direct buried and concrete-encased conduit will be schedule 40 PVC.
- Interior Conduit Installation:
    - Exposed conduit in non-corrosive areas will be rigid galvanized steel.
    - Exposed conduit in corrosive areas will be Schedule PVC-coated galvanized rigid steel.
- Power and control wiring rated 600V and less will be stranded copper conductors with XHHW-2 insulation.
- Lighting and receptacle circuit wiring rated 600V and less will be single solid conductor copper with THHN/THHW insulation.
- Individual 14 American wire gauge conductors will be used for discrete control circuits, unless it is practical to use multi-conductor cables to group control circuits. Cables will have 600V insulation.
- Power cable to equipment controlled by a VFD operating 600V or less will be 1,000V VFD cable with stranded copper conductors and cross-linked polyethylene insulation.
- Twisted shielded pair control cable with 16 American wire gauge individual stranded copper conductors, PVC insulation, and an aluminum mylar tape shield around the pair will be used for analog signals. Multi pair cables will be used where grouping of circuits is practical. Cables will have 600V rating.

9.4 Standby Generator

A standby emergency-rated diesel engine-generator will be provided at the site in the event utility supplied power is not available. The engine-generator package will consist of an engine-generator, control panel, cooling system, and accessory items all installed outside in a weather-proof Level 2 enclosure on a concrete base.

In the event of a utility company power outage, the engine-generator will automatically start and the ATS will transfer power from utility to engine-generator to power the station electrical loads. Once utility power is restored, the ATS will transfer back to the utility power and then shut down the standby generator after a pre-set cooling time.

The engine-generator size is based on operating demand requirements. The demand load will be based on the number of gates, motor loads, compressors, and miscellaneous loads for the worst-case operating scenario.

The engine-generator will be housed in a sound attenuated and weather-protected enclosure to have lowest possible noise and exhaust emission per the local noise ordinance and San Joaquin Air Quality Management District air pollution requirements. The Contractor will be specified to apply and pay all costs to obtain all the required local permits including Permit to Construct and Permit to Operate the engine-generator within the area surrounding the Project.

9.5 Grounding and Bonding

The electrical system and equipment will be grounded in compliance with the NEC. Conductors will be at least 4/0 American wire gauge copper, for interconnecting ground rods and connections to transformers and switchboard. A grounding ring will be provided for each area. Electrical equipment, devices, panelboards, and metallic raceways that do not carry current will be connected to the ground ring as well as building steel and metal water piping.

Transformer neutrals of wye connected transformers will be solidly grounded through a grounding electrode conductor connected to the grounding electrode system.

9.6 Electric Features

9.6.1 Control Systems

A control panel equipped with Allen-Bradley CompactLogix class PLC and local Panel PC operator interface terminal (OIT) will be designed for automatic control and monitoring of the station processes and equipment. The PLC ladder logic and OIT graphics will need to be developed from the control strategies prepared by HDR with input from the SEWD. Close coordination with the SEWD will be required to ensure the PLC program and OIT application are written in compliance with facility standards.

Manual control devices will be provided on the control panel door for operating the process in case of PLC failure. The OIT will display the flow/level data and provide

operators with a means to adjust equipment and alarm setpoints. The station control data and alarms will display OIT for operator viewing.

The OIT will be programmed and configured with screens to provide a window to the processes. The operators will be able to set the process setpoints and view the alarms and status of the equipment. Some of the screens will enable the operators to manually control the equipment.

The PLC will collect and transfer various area controls, status, and alarms to the SEWD's new SCADA system via radio.

A Reset pushbutton will be used in the control panel to locally reset the failure alarm after investigation of the failure.

Station alarms will include:

- Flowmeter failure
- Pump failures
- Motor Hi Temperature (may not be required)
- VFD failures
- Utility company power failure
- Generator failures including low fuel level alarm
- Generator battery charger failure
- ATS failure
- UPS failure
- Low voltage power source failure
- Intrusion
- Other alarms as required during the design review.

Data and status will include:

- Discharge flow
- Water Level
- Pump running
- Control switch status
- Gate status
- Fish screen status
- Trashrack status
- ATS status for utility power on
- ATS status for generator power on
- Generator status

- Generator charger/battery status
- Other data and statuses as required during the design review.

9.6.2 Supervisory Control and Data Acquisition

The SCADA system will utilize the Inductive Automation Ignition® platform and consist of a SCADA client computer located on the control panel.

The SCADA application will be hosted from a local rack- mounted server with an uninterrupted power supply to provide continuous data collection and communication between the Bellota Weir Site and SEWD WTP.

Operators will remotely interact with equipment by interfacing with the SCADA human machine interface graphical displays. The graphical displays will represent system processes and be designed based on design standards listed herein and as shown on the piping and instrumentation diagrams from the final drawings. The SCADA system will be accessible from the District's WTP and available to operations through secure methods such as a virtual private network and individual account verification.

All process equipment will be operated in one or more of the following modes:

- Hand: The equipment is manually controlled from a local control station or from the MCC (if no local control station exists).
- Auto: The equipment is automatically controlled locally by the packaged equipment PLC or through hardwired interlocking scheme.
- SCADA Manual: The equipment is controlled manually through the PLC based on commands issued from an OIT. Such commands are received by the local PLC and converted into physical outputs to the field devices.
- SCADA Auto: The equipment is controlled automatically through the PLC based on measured process parameters, or calculated values received from field devices, or remote PLCs, and commands and setpoints issued from an OIT. Such commands, setpoints, and process values are received by the local PLC. The local PLC will adjust the equipment accordingly, through physical outputs, to meet the process setpoint. Some equipment may have more than one remote automatic mode of control.

The control mode will be selectable where applicable based on hand/off/auto switches located at the devices, MCC, and device control panels. Selector switch position feedback will be wired to the PLC, allowing an operator using an OIT to know whether a device is available for remote control from the OIT.

9.6.3 Communications

To be completed for the 100 percent design level.

9.6.4 Lattice Antenna Tower

The self-supported tower will incorporate the requirements of the latest California Building Code, ASCE, American National Standards Institute, ACI, and District Standards and Ordinances, as applicable. The communication system will require a 100-foot self-supported lattice tower for direct line of sight between the Bellota Weir Site and SEWD WTP. The tower will be equipped with safety climbing assembly and climbing ladder required to install radios, repeaters, and additional antennas.

9.6.5 Security

Magnetic type intrusion switches will be installed at all Control Building entrances, control panels, and outdoor electrical equipment. When a door is opened, contacts on the intrusion switch will open to trigger an alarm.

Additionally, all field-mounted panels will be furnished with pad-lockable hasp to prevent intrusion and tampering of equipment.

Security cameras will be installed to monitor various areas such as standby generators, control building, fish screens, and additional areas. A DVR device will be installed in the server rack and will collect data from various cameras. The DVR will be connected to the network switch to provide remote monitor from the SEWD WTP facility.

9.7 Lighting and Illumination

LED lighting systems will be used for all areas including interior, exterior building, and site lighting. Illumination levels in the Electrical Building will be provided following the recommended levels suggested in the Illuminating Engineering Society handbook and by the International Dark-Sky Association for the space and tasks being performed. Lighting fixture types will be suitable for the environments where installed and be located (serviceable and accessible) for routine maintenance.

- Indoor Locations: Fixtures will be switched. Emergency lighting will include an emergency battery pack integral with the fixture.
- Exit Signs: Provide LED type and placed inside the facilities per the latest NFPA requirements.
- Exterior Locations: Fixtures mounted to exterior building will be photocell controlled. Site lighting will be photocell controlled with bypass switch to allow facility to manually turn off each light pole.

Exterior LED light fixtures will have a color temperature of 3,000 Kelvin (K) as recommended by International Dark-Sky Association to limit exposure to blue light. Each fixture will be equipped with full cutoff and be shielded to minimize skyglow, glare, and light pollution.

Facility entrances will be classified as straight roads with low to no pedestrian conflict classifications. The roadway lighting spacing will be based on the light levels and uniformity requirements in the Illuminating Engineering Society RP-8 – Standard for Roadway Lighting.

- Light Level Range to Ground: 0.6 to 1.7 footcandles
- Uniformity Ratio Range Avg/Min: 3 to 4 footcandles

10 Operation and Maintenance

O&M details for the proposed Project are presented herein.

10.1 Operational Strategy

The Project's major operational components include: intake facility screens, weir and intake, sluiceways and gates, and fish passage infrastructure. All equipment will be operated under specific conditions to meet the primary objectives of the Project. Flow will be conveyed down the roughened channel to facilitate fish passage under all operational scenarios.

Facility equipment will operate per the following four conditions:

- Normal operation during the non-irrigation season
- Normal operation during the irrigation season
- High-flow conditions during the irrigation season
- Periodic sluicing of debris and sediment through the sluice pipelines

10.1.1 Normal Operation during the Non-Irrigation Season

The non-irrigation operating season occurs outside the typical diversion season and represents a period when the Calaveras River flows are conveyed primarily over the lowered Obermeyer weirs to Mormon Slough. During this period, river flows can vary widely from 0 cfs to as high as 10,000 cfs (USACE 1997). New Hogan Dam reservoir releases during the non-irrigation season are operated by the USACE primarily for flow and flood control to protect agricultural, urban, and suburban land along the Calaveras River, Mormon Slough, and Stockton Diverting Canal (CH2M 2006). High-flow events will be used to sluice bedload downstream of the Project from the forebay, screen channel, and intake manifold. Low-frequency, high-magnitude flows may require emergency measures or actions to protect existing infrastructure when they occur. Emergency scenarios will be developed further as the Project design advances.

Intake Screens

During the non-irrigation season at current flow requirements, the two larger 83-inch-diameter fish screens are raised to their full maintenance position above the water surface, the isolation gates behind these screens are fully closed, and the weir gates are in a fully down position. Non-irrigation flows will be obtained using the two smaller 48-inch-diameter screens capable of diverting up to 90 cfs (maximum) without exceeding the maximum 0.33 fps approach velocity criteria.

Bellota Weir and Bellota Intake

While most agricultural diversions have limited use during the flood season, SEWD and Calaveras County Water District divert flow for municipal purposes year-round for the WTP. During the non-irrigation season, the headwater elevation will be reduced to

approximately 115.44 feet NAVD88 by lowering the weir gates. Water will pass through the fish screens into the intake manifold, be equalized in the intake manifold, and then routed to the distribution structure. Water control gates at the distribution structure will modulate to the desired diversion flow rates for both the Bellota and Old Calaveras River diversions.

Under present conditions, approximately 46 cfs will be diverted to the WTP through the Bellota intake and 15 cfs to the Old Calaveras River. Remaining flow will be diverted downstream into Mormon Slough.

Under future conditions, approximately 60 cfs will be diverted to the WTP through the Bellota intake and 15 cfs to the Old Calaveras River. Remaining flow will be diverted downstream into Mormon Slough.

Fish Passage

During the non-irrigation season operation, fish passage will be conveyed only through use of the roughened channel. The fish ladder will be closed.

Operations Overview

During the non-irrigation season, the weir gates will be lowered to elevation 115.44 feet. All other gates in the system will be closed with the exception of the two fish screen gates mentioned above, and the open/close SEWD water supply gate delivering water to the WTP. The modulating SEWD water supply gate will be closed while the system is primed. Once the WSE in the forebay is at least 115.44 feet, the modulating SEWD water supply gate will be opened to divert 46 cfs to the WTP. The weir gate in Bay 1 will then be lowered to elevation 114.5 feet to allow fish passage through the roughened channel notch. If the WSE in the forebay is 115.44 feet or greater, the modulating Calaveras River water supply gate will be opened to provide 15 cfs to the Old Calaveras River. The remaining will flow down Mormon Slough.

10.1.2 Normal Operation during the Irrigation Season

Normal operating conditions for the diversion facility occur during the irrigation season. During this period, the weir gates will be operated automatically to maintain a minimum operating WSE of 121.44 feet NAVD88 in the Bellota pool to achieve the necessary driving head to deliver water to the treatment facility with up to 200 cfs as well as provide adequate depth and sweeping velocity for the fish screens and diversion.

Intake Screens

During normal future irrigation season operating conditions, the screen isolation gates will be in their fully open positions and all four of the fish screens will be in the down position in the water. Under the current maximum demand of 220 cfs, at least one 48-inch-diameter screen and one 84-inch-diameter screen will be in operation at any given time, with the other two screens on standby.

Bellota Weir and Bellota Intake

During the irrigation season, the headwater elevation will be raised to approximately 121.44 feet NAVD88 by raising the weir gates. Water will pass through the fish screens into the intake manifold, be equalized in the intake manifold, and then routed to the distribution structure. Water control gates at the distribution structure will modulate to the desired diversion flow rates for both the Bellota and the Old Calaveras River diversions.

Under present conditions, approximately 70 cfs will be diverted to the WTP through the Bellota intake. Approximately 150 cfs will be diverted to the Old Calaveras River and 60 to 100 cfs downstream into Mormon Slough, though these may be exceeded.

Under future conditions, approximately 200 cfs will be diverted to the WTP through the Bellota intake. Approximately 150 cfs will be diverted to the Old Calaveras River and 60 to 100 cfs downstream into Mormon Slough, though these may be exceeded.

Fish Passage

During the irrigation season, fish passage flows will be conveyed using both the roughened channel and fish ladder. Up to about 20 cfs of the flow diverted downstream in Mormon Slough will pass through the fish ladder. Remaining flow diverted downstream into Mormon Slough will be conveyed through either the Overflow Bypass or Obermeyer Weir Gate 3.

Operating conditions under both present and future operation are summarized in Table 10-1 and Table 10-2. The tables provide direction as to flow pathways at various flow events. Figure 10-1 depicts these flow pathways.

Operations Overview

During the irrigation season, the weir gates will be raised to elevation 121.44 feet. All other gates in the system will be closed with the exception of the two fish screen gates mentioned above, and the open/close SEWD water supply gates delivering water to the WTP. The modulating SEWD water supply gate will be closed while the system is primed. Once the WSE in the forebay is at least 121.44 feet, the modulating SEWD water supply gate will be opened to divert 70 cfs to the WTP, maintaining the forebay elevation. After this, the fish ladder gate will be opened to divert 20 cfs down the fish ladder for fish passage. If the WSE in the forebay is 121.44 or greater, the modulating Calaveras River water supply gates will be opened to provide 150 cfs to the Old Calaveras River. Maintaining the 121.44-foot forebay elevation, the bypass flow gate will then be opened to divert 50 cfs down Mormon Slough. With additional flow, the Obermeyer weir gate in Bay 1 will be lowered to EL 120.44 feet to send more flow down Mormon Slough.

Table 10-1. Present Demand Operating Conditions

Time of Year	A. Calaveras River Flow Upstream of Weir (cfs)	B. WTP Demand (cfs)	C1. Fish Ladder Flow (cfs)	C2. Overflow Bypass (cfs)	C3. Adjustable Weir Gates (cfs)	D. Total Roughened Channel/Mormon Slough Flow (C1+C2+C3) (cfs)	E. Old Calaveras River Demand (cfs)
Irrigation season Roughened channel and fish ladder in use Obermeyer Weirs are raised to WSE 121.44 feet NAVD88 One 48-inch and one 84-inch cylindrical tee screen in use	0 – 70	0 – 70	0	0	0	0	0
	70 – 240	70	0 – 20*	0	0	0 – 20	0 – 150*
	240 – 290	70	20	0 – 50	0	20 – 70	150
	290+	70	20	50	0+	70+	150
Non-irrigation season Roughened channel in use Obermeyer Weirs are lowered to for WSE 115.44 feet NAVD88 Two 48-inch cylindrical tee screens in use	0 – 46	0 – 46	N/A	N/A	0	0	0
	46 – 61	46	N/A	N/A	0	0	0 – 15
	61+	46	N/A	N/A	0+	0+	15

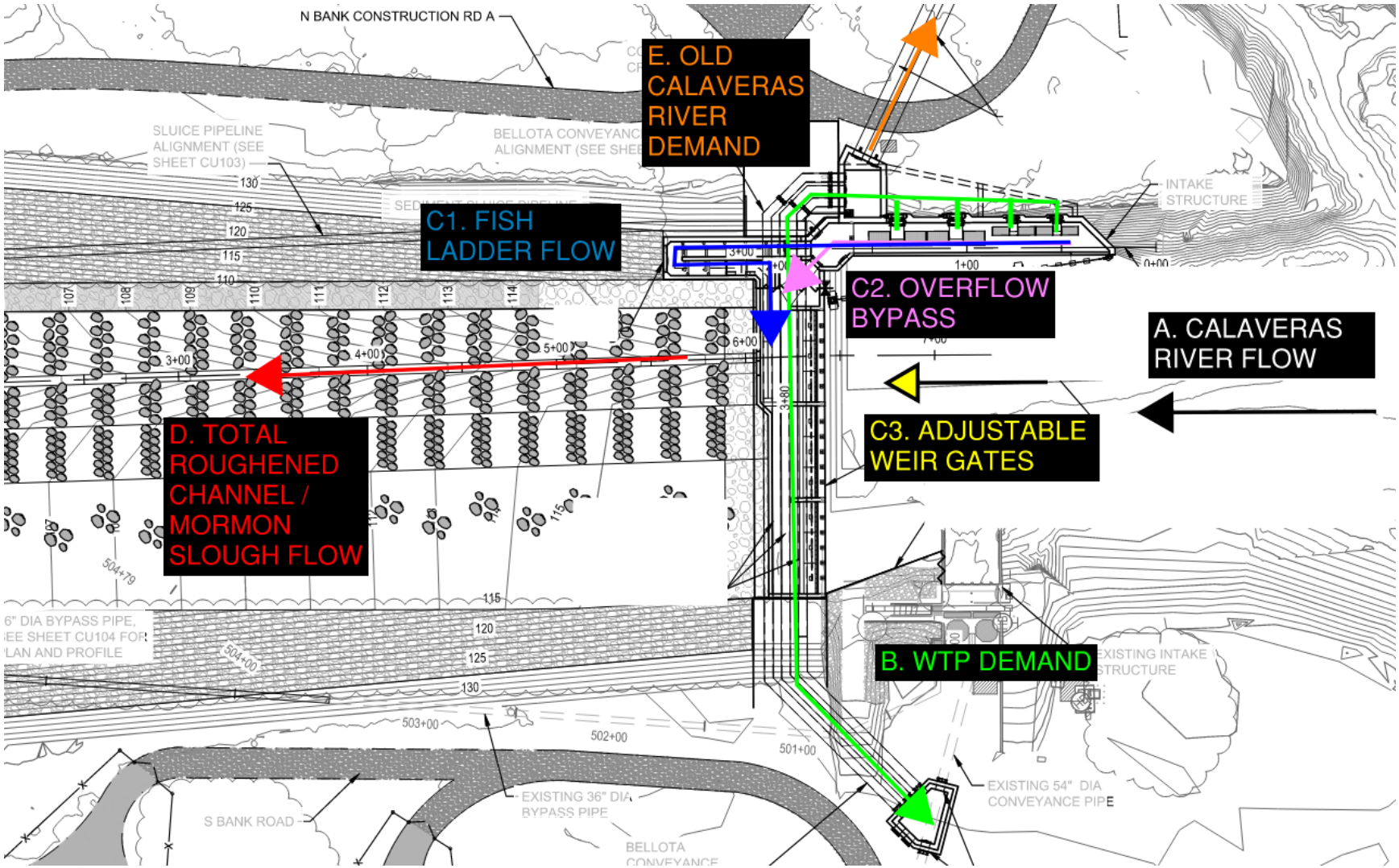
*until fish ladder operation flow rate is met

Table 10-2. Future Demand Operating Conditions

Time of Year	A. Calaveras River Flow Upstream of Weir (cfs)	B. WTP Demand (cfs)	C1. Fish Ladder Flow (cfs)	C2. Overflow Bypass (cfs)	C3. Adjustable Weir Gates (cfs)	D. Total Roughened Channel Flow/ Mormon Slough Flow (C1+C2+C3) (cfs)	E. Old Calaveras River Demand (cfs)
Irrigation season Roughened channel and fish ladder in use Obermeyer Weirs are raised to WSEL 121.44 NAVD88 All four cylindrical tee screens are in use	0 – 200	0 – 200	0	0	0	0	0
	200 – 370	200	0 – 20*	0	0	0 – 20	0 – 150*
	370 – 420	200	20	0 – 50	0	20 – 70	150
	420+	200	20	50	0+	70+	150
Non-irrigation season Roughened channel in use Obermeyer Weirs are lowered to WSE 115.44 feet NAVD88 Two 48-inch cylindrical tee screens are in use	0 – 60	0 – 60	N/A	N/A	0	0	0
	60 – 75	60	N/A	N/A	0	0	0 – 15
	75+	60	N/A	N/A	0+	0+	15

*until fish ladder operation flow rate is met

Figure 10-1. Operating Conditions



10.1.3 High-Flow Conditions

Periods of high flow, turbidity, sediment, and debris movement are anticipated to occur within the irrigation season. Conditions that may diminish the effectiveness of the surface water diversion may occur once out of every 5 to 10 years when large-magnitude river flow events mobilize large amounts of buoyant and semi-buoyant debris in the water column. The proposed design incorporates several measures to mitigate these conditions, but the actual performance and resilience of the intake facility during these periods will be relatively uncertain until several years after operation of the facility begins. The screen and intake design is configured to protect the fish screens during high debris and bedload movement. Raising or lowering the screens during storm events is not anticipated. The debris boom is intended to exclude large debris from entering the screen bay while the bypass maintains sweeping velocities across the screen to convey buoyant and semi-buoyant debris downstream. Fine sediments in the screened intake manifold will be addressed with a water jet sediment removal system to keep the fine sediment in suspension. Sediment accumulation in the intake manifold and/or distribution structure will be addressed by a 12-inch-diameter sluice pipe connected to the 54-inch-diameter sluicing pipeline.

High flow events may be taken advantage of by using the higher velocities to sluice accumulated debris or sediment.

10.1.4 Periodic Sluicing through the Sluice Pipelines

Sluicing through the piped sluiceway will occur periodically as needed to flush out material. Bedload and debris will be moved from upstream of the roughened channel to downstream of the facility.

This sluicing strategy will maintain an unimpeded hydraulic pathway from the forebay to the intake screens by clearing any bedload or debris that has accumulated upstream of the weir near the intake as well as in the screenings channel. Further, the sluiceway can be used strategically to bypass the sediment around the weir crest and roughened channel to increase the magnitude and frequency of sediment movement through the reach. The operating rule curves and operating strategy will be developed further as the design progresses.

The sluicing pipeline operational strategy will be opportunistic in nature and focus on those river flow events that provide the highest velocity and shear. This strategy will be developed to maximize the anticipated capacity of sediment transport and minimize the accumulation of sediment, bedload, and debris near and in front of the trashrack inlet to the fish ladder.

10.2 Maintenance Expectations

Anticipated maintenance efforts for the intake, water conveyance system, sluiceway, and roughened-channel fishway are described in this section.

10.2.1 Intake

Screen Maintenance

Occasional maintenance may be necessary throughout the life of the intake structure to remove large debris from screens or sluiceway, and check or perform minor repairs on screens. Cleaning of the screens will occur largely without human interaction via an automated system during normal operating periods. Occasional maintenance may be necessary for the automated debris rake system as well to ensure all systems are running properly. Higher levels of effort and attention to the fish screens may be required during periods of higher flows and high diversion demand that may periodically overlap. Sluicing of the area in front of and behind the fish screens is an operation that may be initiated manually but is carried out via push-button controls. Removal of sediment within the intake is expected to occur as frequently as required for the existing intake.

Consolidated Intake Maintenance (Instream Structures)

Most maintenance actions related to the new intake will occur within the new infrastructure and therefore isolated from the river channel. Following a large flood event, however, minor replacement of armoring adjacent to the new intake structure or removal of debris from the sluiceway exit may be necessary.

To address maintenance needs, the District anticipates most repairs will be accomplished using an excavator positioned on the access road along the north bank of the Mormon Slough, or from the bank. In-water work may be necessary. In-water maintenance activities may be covered under a maintenance permit or could require a stand-alone permit; this will be determined through later stages of design and conversations with the District.

10.2.2 Sluiceway

The 54-inch-diameter sluiceway will provide sufficient flows to remove accumulated bedload in the forebay or intake structure. During forebay sluicing, the water level should be lowered enough to induce mobilization of sediment. The same principle applies to sluicing the intake structure.

10.2.3 Roughened-Channel Fishway

The roughened-channel fishway and crest are designed to withstand flows up to and including the 100-year flow (12,690 cfs) with some anticipated limited damage. Project failure at this site is defined as the inability to effectively pass fish through the reach or provide a reliable water supply to meet irrigation demands. The structural integrity of the weir crest and functional stability of the roughened channel should not be compromised during or following any given flow event up to the 100-year flow. Despite this, annual maintenance activities in the roughened-channel fishway may be required to remove large debris if such debris is interfering with low flow conditions. Large debris removal will likely be accomplished using an excavator with an extension arm operating from the access road.

Although the design emphasizes stability, higher flow events may cause unanticipated scour and erosion altering the roughened channel crest and/or bed configuration

requiring maintenance or repair activities. Repair activities may include filling eroded areas with a designed rock matrix similar to the original design, or resetting specific large rocks at the crest or mid-fish channel to reestablish the indented hydraulic conditions. In-water activities required to maintain intended operations in the same footprint of the original facility are assumed to fall under maintenance exemptions. Larger scale repairs may require review and individual permits approved through multiple government agencies on a case-by-case basis.

Annual or Typical Maintenance

Large debris removal will be accomplished using an excavator with an extension arm operating atop the riprap wall of the sluiceway; no equipment will enter the channel except for the bucket arm. Plant removal will also likely be required during annual maintenance. Maintenance periods would occur on an opportunistic basis, when low flow periods downstream of the weir occur, typically during the irrigation season and in-water work window. These low flow periods may occur, but not necessarily within the current anticipated migration window. The annual level of effort associated with maintenance is anticipated to be zero to eight work days.

Infrequent Maintenance

Although the design emphasizes stability, higher flow events may scour and erode the streambed and alter the roughened channel crest and/or bed configuration to a level requiring maintenance or repair activities. Larger scale repairs will possibly require coordination with the CDFW, NMFS, and USACE for review and approval.

Infrequent repair activities may include filling eroded areas with a designed rock matrix similar to the original design, or resetting specific large rocks at the crest or mid-fish channel to reestablish the indented hydraulic conditions. In-water activities required to maintain intended operations in the same footprint of the original facility are assumed to fall under maintenance exemptions for several permits (e.g., Hydraulic Permit Application and Clean Water Act 404/401). If such activities cannot be completed solely from the access road along the north bank or in the dry during low flow periods, an approach using a cofferdam and dewatering, including protocols for fish salvage, will be implemented as necessary.

Maintenance periods would occur on an opportunistic basis, when low flow periods downstream of the weir occur, typically during the irrigation season. These low flow periods may occur, but not necessarily within the current anticipated migration window. The level of effort for these types of maintenance events may be on the order of 10 work days.

10.2.4 Weir

The adjustable weir gate system will need to have general maintenance on an annual basis. Main components to be inspected and maintained are the air compressors, receiver tank, connection to the air bladder, gate leaves, restraining straps, protection plate, and protection plate wheel.

Infrequent maintenance may require that the area be dewatered. Stoplogs can be placed upstream to block off the area so maintenance can be performed in the dry. If the

bladder needs to be accessed, there are attachment points on the gate leaves and pier walls so the gate can be in the raised position without the bladder holding up the gate leaf.

10.2.5 Fish Ladder

The fish ladder foundation and walls should be inspected for structural integrity, cracking, and other signs of damage. Visual, surface level inspections of concrete elements will be made to identify obvious defects, hazards or potential problems, and to monitor known problems.

Inspections and maintenance should be recorded to provide a historical account of the fish ladder's condition. Any cracking or other signs of damage to the ladder structure should be recorded. The ladder structure should be inspected in accordance with the following:

- Confirm the fish ladder ramps, slots, and weirs have not been damaged
- Inspect the weirs, slots, and walls for signs of cracking, spalling, or corrosion
- Check the extent of vertical and horizontal cracking; cracks that run the entire height or width of the ladder may indicate movement or undermining of the ladder's foundations

The fish ladder will be sluiced using higher flows to dislodge any accumulated debris or sediment. The gate on the upstream pool and weir will be opened to accommodate this sluicing. Some sediment or debris may not be removable by sluicing, due to eddies typical of vertical slot ladders. If apparent, use a high-pressure water source to remove built-up sediment, grit, biological growth (algae), and other debris that has accumulated in the fish ladder structure. The stoplogs can be removed from the entrance weir so that the entire fish ladder is drained.

The fish ladder should be inspected and cleaned regularly, and debris should be removed by properly trained personnel. Infrequently, larger debris may become jammed within the fish ladder. The trashrack is anticipated to limit all larger debris that could enter the ladder, though if any larger debris does pass, it will likely need to be dislodged manually.

All maintenance activities should occur during the non-irrigation season when the ladder is not in use.

11 Construction Schedule and Sequencing

The Project will be constructed in sequences that optimize constructability, cost effectiveness, environmental impact, and operational continuity. Primarily, sequences are organized to provide full, complete elements that can be utilized even if future sequences are delayed.

The current sequencing strategy could be completed within about 3 years, but is designed such that construction sequences could be delayed several year if needed. This sequenced approach provides a slower construction period to allow for adequate funding to be located between sequences if necessary.

Within each sequence, there are Project elements that may be constructed simultaneously. These are not identified in the text and are up to the discretion of the Contractor. Sheet pile wall installations, removals, and manipulations should remain as directed or thought through carefully, as in many cases the sheet pile walls are acting as temporary flashboards to provide irrigation water during irrigation season and must be cut following irrigation season to install remaining Project elements and provide fish passage.

11.1 Sequence 1A

Construction Sequence 1A may take place during the non-irrigation season, outside of the in-water work window, if other schedule limitations allow. Fish passage will be unchanged from the existing condition during this sequence. Construction of this sequence will require approximately 4 months and commence as early as fall 2022. Most of the features of work in this sequence will be isolated from the river with silt fencing, and by limiting the work to occur landward of the waterside hinge point to the waterways. The following will be completed during construction Sequence 1A as shown on sheet CS106:

11.1.1 Sequence 1A

- Install landside erosion and sediment controls, including silt fencing and stabilized construction entrances, and placing concrete washouts.
- Modify existing fencing, gates, and driveways and construct new entrances adjacent to State Route 26 and East Shelton Road.
- Identify existing features to be protected in place, and demarcate them to be protected during construction as appropriate.
- Demolish the existing building in the construction staging area on the north side of the Project.
- Prepare and secure the contractor staging areas on the north and south sides of the Project.
- Construct the temporary roads for construction vehicle access, including the North Bank Construction Road and the South Bank Construction Road. These temporary roads will be left in place after construction.

- Construct the permanent access roads, including the Permanent North Bank Road and the Permanent South Bank Road. These roads will be restored for permanent access prior to closeout of construction.

11.2 Sequences 1B-1D

Construction of the first sequence (1B-1D) will occur during the irrigation season (mid-April to mid-October), during the in-water work window (mid-June to early October). Stream diversion will be provided using the constructed 54-inch sluiceway and existing 36-inch bypass in the existing intake. Temporary fish passage will not be provided during this time because according to the HCP, steelhead and fall-run Chinook salmon (known to opportunistically use the river) are not present; the document shows no adult migration or juvenile outmigration occurring from June to October (CHCP 2019). This is the same case for winter, spring, and late-fall run Chinook, though each of these are considered extirpated from the area and not likely to use the Calaveras River. Therefore, construction between April and October would not affect these fish.

Construction of this sequence will require up to 8 months and commence as early as spring 2023, on issuance of applicable permits and federal, state, and local authorizations and availability of funding.

11.2.1 Sequence 1B

The following will be completed during construction sequence 1B as shown on sheet CS107.

- Extend the existing 36-inch water bypass system downstream of the roughened channel and confirm bypass is operational to accommodate the passage of flow during construction. An RSP apron will be constructed for energy dissipation at the outlet.
- Construct access road along right bank adjacent to the roughened channel including the temporary access road adjacent to the existing north ramp of the weir.
- Install sheet pile wall S3-B from the downstream end of the future new fish ladder upstream to the tie-in to right bank EL 127 feet upstream of the future new intake structure as shown on the drawings.
- Install the new 54-inch RCP sluice pipeline extending through sheet pile wall S3-B with water control gate mounted to wall for passage of flow during construction. Best management practices will be utilized at the inlet and the associated RSP apron will be constructed at the outlet, located downstream of the access road.
- Install sheet pile walls S2, S4, and S5 to the tip elevations as shown on the drawings and finish top at EL 127 feet. This should occur prior to the irrigation season so that the WSE in the Bellota Pool provides the appropriate irrigation season flows to the WTP.
- Raise Bellota Pool to EL 121.44 feet.
- Regulate bypass flows using both bypass pipelines to maintain pool EL 121.44 feet.

- Install temporary dewatering sumps to control river seepage in construction work areas.
- Remove and salvage two Denil fish ladders.
- Demolish existing weir.
- Remove existing riprap and salvage any rock meeting project specifications.

11.2.2 Sequence 1C

The following will be completed during construction sequence 1C as shown on sheet CS108.

- Grade new weir forebay to finish elevations as shown on the drawings.
- Excavate the riverbed to approximate EL 104 feet across the entire river channel within the footprint of the weir to accommodate the three 54-inch WSP Bellota WSPs.
- Install sheet pile wall S1 to the tip elevations as shown on the sheet CS108 with finish top EL 115.5 feet.
- Install the 4 debris boom dolphin piles, and 78 pipe piles for new weir footing, and 2 pipe piles for a small portion of fish ladder footing as shown on sheets CS108 and SB104.
- Install cross bracing on pipe piles to support the three 54-inch WSPs at the specified elevations across the river channel.
- Install three 54-inch pipelines with blind flange caps that extend beyond weir footprint.
- Excavate roughened channel to design elevations shown on sheet RI201.
- Stockpile excavated spoils for future use.
- Backfill new weir footprint with flowable fill around the three 54-inch pipelines to EL 109.5 feet.
- Install weir slab, columns forms, and rebar including portion of fish ladder wall and slab.
- Install necessary mechanical conveyance lines for Obermeyer weir gate from mechanical building across weir structure.
- Pour weir slab, columns, and walls.
- Strip all formworks.
- Backfill roughened channel with specified riverbed materials.
- Finish-grade roughened channel and associated weir boulder bands.

11.2.3 Sequence 1D

The following will be completed during construction sequence 1D as shown on sheet CS109.

- Install debris boom components.
- Remove temporary portion of 54-inch sluice pipe and install per final 54-inch pipe alignment. Mount temporary slide gate on sheet pile wall S-3B.
- Remove temporary access road (east ramp).
- Install sheet pile wall S-3A.
- Install riverbank armoring.
- Lower Bellota Pool to EL 115.44 feet.
- Remove in-river dewatering systems.
- Remove sheet pile wall S2.

11.3 Sequence 2

Construction Sequence 2 will occur during the non-irrigation season, outside of the in-water work window. Fish passage will be provided using the roughened channel. Construction of this sequence will require approximately 6 months and commence as early as fall 2023. Most of the work features in this sequence will be isolated from the river with sheet pile walls S-3A and S-3B. The following will be completed during the second construction sequence as shown on sheet CS110:

- Install dewatering system to control potential seepage flows from the river and high groundwater.
- Install the remainder of the Bellota conveyance 54-inch pipelines on the north bank to the water distribution structure.
- Construct the distribution structure.
- Complete construction of the concrete fish ladder structure.
- Connect the sluice pipeline into the concrete fish ladder forebay structure.
- Construct temporary fish ladder exit and temporary wall in the fish ladder structure to provide fish passage during the next irrigation season.
- Demolish existing buildings on the site.
- Complete control and shop building including all associated equipment and controls. All electrical work will be complete after this sequence.

11.4 Sequence 3

Construction of the third sequence will take place during the irrigation season, during the in-water work window. Stream diversion will be provided primarily through the existing 36-inch bypass and if necessary, the newly constructed 54-inch-diameter sluiceway. Fish passage will be provided using the roughened channel and fish ladder, with a temporary exit. Construction of this sequence will require approximately 6 months and commence as early as spring 2024. The following will be completed during the third construction sequence as shown on sheet CS111:

- Erect removable Bellota bulkhead piers and stoplogs across entire weir.
- Construct temporary coffer dam at future Calaveras outlet structure to isolate the Old Calaveras conveyance pipeline installation.
- Raise the Bellota Pool to EL 121.44 feet prior to the beginning of irrigation season so that the WSE in the Bellota Pool provides the appropriate irrigation season flows to the WTP.
- Use the 36-inch existing bypass pipe and fish ladder to provide bypass flows to the Mormon Slough.
- Install new Obermeyer weirs and associated equipment.
- Close 36-inch bypass flow and protect inlet/outlet for mothballing system.
- Close fish ladder operations.
- Remove removable bulkhead piers and stoplogs coincident with the end of the irrigation season.

11.5 Sequence 4

Construction Sequence 4 will occur during the non-irrigation season, outside of the in-water work window. Fish passage will be provided using the roughened channel. Construction of this sequence will require approximately 6 months and commence as early as fall 2024. The following will be completed during the fourth construction sequence as shown on sheet CS112:

- Construct the necessary dewatering system to control both river and groundwater seepage.
- Install the intake structure, connecting it to the previously constructed fish ladder and distribution structure.
- Install dual RCP 54-inch Old Calaveras conveyance pipelines from new intake structure to the Old Calaveras River.
- Install concrete crane pad.
- Install pumping systems and complete startup and testing of various equipment.
- Remove/abandon portion of the 36-inch bypass where in conflict with the Bellota 54-inch WSP conveyance pipelines.
- Install the three 54-inch WSPs from the weir to the south side diversion structure.
- Complete 54-inch RCP main line connection and extension to the new 54-inch steel pipe.
- Cut sheet pile wall S3 down to final elevation as shown on sheet SB502, Detail 1.
- Finish site grading and final features above ordinary high water.

11.6 Sequence 5

Construction Sequence 5 will occur during the irrigation season, during the in-water work window. No stream diversion will be needed. The work will require approximately 4 months and commence as early as spring 2025. The following will be completed during the fifth construction sequence as shown on sheet CS113:

- Install temporary coffer dams on either side of the earthen fill and in front of bank armoring adjacent to the new intake.
- Construct the necessary dewatering system to control both river and groundwater seepage.
- Construct temporary water supply system utilizing the new Calaveras 54-inch conveyance pipelines to Old Calaveras River.
- Construct the new concrete Calaveras outlet structure.
- Construct earthen fill levee with appropriate seepage controls.
- Install Bellota Pool right bank armoring.
- Complete all the necessary startup operational controls to operate the intake facility as intended.

12 Project Costs

Project costs generated alongside the 90 percent design include O&M, construction, implementation, and anticipated life cycle. Details of each are included in this section and Appendix D.

12.1 Operation and Maintenance Costs

Annual O&M costs, estimated at the 90 percent phase of design, are $169,000 per year for the entire facility after completion of the final construction sequence. This estimated cost is higher than previously provided due to employee rate increases to be more commiserate with SEWD current rates. Anticipated O&M costs for each construction sequence will be provided at the 100% phase of design.

12.2 Construction Costs

Cost data generated as part of this OPCC is based on bids received from other similar projects, available vendor cost data, details from cost estimates prepared for other projects of similar scope, RSMeans cost data, and professional judgment.

The total construction cost is anticipated to be $68 million (M) which includes a 10 percent contingency, 5 percent escalation per year for 5 years, 4 percent markup for construction phasing complexity, and 3 percent adjustment factor for market volatility. The anticipated base cost for Project construction is estimated to be $43.9M and could vary from as low as -10 percent to as high as +20 percent based on the current level of detail and cost certainty. Major cost components include the intake/fishway ($8.9M), weir structure ($4.4M), roughened-channel ($5.7M), and conveyance structures ($3.0M). The larger cost items for the intake/fishway include the sheet pile walls ($1.7M) and the tee screens ($3.0M). The larger cost items for the roughened-channel are the foundation rock ($2.0M) and riprap for bank stabilization ($1M).

12.3 Soft Costs

Soft costs for the project were evaluated estimated as the following percentage of the total direct construction costs:

- 1% for Testing (Materials and General)
- 5% for Construction Management Services
- 5% of Construction Inspection Services
- 5% for Engineering Services during Construction

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Appendix A. Design Criteria Report

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Draft Design Criteria Report

Bellota Weir Modifications Project

Mormon Slough, Calaveras River, CA

March 29, 2021

Prepared for

STOCKTON EAST WATER DISTRICT

PROVIDING SERVICE SINCE 1948

Prepared by HDR Inc & KSN, Inc.

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Abbreviations

°F	degree(s) Fahrenheit
1D	one-dimensional
2D	two-dimensional
AA	Aluminum Association
ACI	American Concrete Institute
AISC	American Institute of Steel Construction
ANSI	American National Standards Association
ASCE	American Society of Civil Engineers
ASME	American Society of Mechanical Engineers
ASTM	American Society for Testing and Materials
AWS	American Welding Society
AWWA	American Water Works Association
Caltrans	California State Department of Transportation
CCWD	Calaveras County Water District
CDFG	California Department of Fish and Game
CDFW	California Department of Fish and Wildlife
CDWR	California Department of Water Resources
CFR	Code of Federal Regulations
cfs	cubic foot/feet per second
CHCP	Calaveras River Habitat Conservation Plan
CPE	Control Point Engineering
District	Stockton East Water District
EM	Engineer Manual
ESA	Endangered Species Act
FEMA	Federal Emergency Management Agency
FHWA	Federal Highway Administration
FIRM	Flood Insurance Rate Map
fps	foot/feet per second
ft	foot/feet
ft-lb	foot-pound(s)
HDPE	high-density polyethylene
HDR	HDR Engineering, Inc.
HEC	Hydraulic Engineering Circular
ITP	Incidental Take Permit
H:V	horizontal:vertical
lb	pound(s)
mm	millimeter(s)
NAAMM	National Association of Architectural Metal Manufacturers
NAVD88	North American Vertical Datum of 1988

NEMA	National Electrical Manufacturers Association
NMFS	National Marine Fisheries Service
Obermeyer	Obermeyer weir gate
OSHA	Occupational Safety and Health Administration
pcf	pound(s) per cubic foot
Project	Bellota Weir Modifications Project
psi	pound(s) per square inch
PVC	polyvinyl chloride
Reclamation	U.S. Bureau of Reclamation
SCADA	supervisory control and data acquisition
SEWD	Stockton East Water District
SPT	Standard Penetration Test
USACE	U.S. Army Corps of Engineers
USGS	U.S. Geological Survey

1 Introduction

The Stockton East Water District (SEWD or District) retained the professional services of KSN, Inc. and HDR Engineering, Inc. (HDR) to advance the Bellota Weir Modifications Project (Project). The Bellota Weir is located immediately downstream of the Calaveras River bifurcation between the Mormon Slough and Old Calaveras River, about 17 miles downstream of the New Hogan Dam.

The Project includes weir and intake improvements that will provide more reliable water delivery systems for SEWD customers, improve migration of anadromous fish, and reduce fish entrainment at the facilities, thereby enhancing anadromous salmonid populations present in the Calaveras River. Initial efforts for this Project include alternative refinement and conceptual design activities in collaboration with SEWD, resource agencies, regulatory authorities, and other Project partners. This document summarizes the key background information and process used to identify potential viable options and select a conceptual design that balances the opportunities, constraints, and objectives of this Project.

1.1 Purpose of Design Criteria Report

This Design Criteria Report documents and summarizes key design considerations and criteria used to develop the various Project elements, which will serve as the foundation for the Project's design and development through preparation of final construction documents.

1.2 Scope of Document

This document presents the fundamental considerations and criteria that influence and govern the design for Project elements. Following this introductory section, a summary of the document contents is provided below:

- **Section 1** provides the project background and an overview of the existing facility and existing conditions, as well as the proposed facility modifications.
- **Section 2** provides the biological criteria used as the basis of design. The biological criteria include information relative to target fish species as well as key information regarding their migration timing, life history, occurrence, and swimming capability.
- **Section 3** provides a summary of the facility-specific criteria that define the operational strategy and range of conditions for which each Project element is designed. These criteria include the diversion period, water demand information, design life, minimum operating conditions, and range and seasonality of Calaveras River hydrology.
- **Section 4** outlines agency-specific criteria put forth by the Federal Emergency Management Agency (FEMA), National Marine Fisheries Service (NMFS), California Department of Fish and Wildlife (CDFW), and San Joaquin County Public Works that will influence many aspects of Project design.

- **Section 5** presents the technical design criteria: hydraulic, geotechnical, civil, mechanical, structural, and electrical.
- **Section 6** provides additional criteria that do not fall under biological, facility, agency-specific, or technical design criteria. This includes criteria such as fire protection, lightning protection, and public safety.
- **Section 7** provides references relied upon for the information and analysis presented in this report.

1.3 Background

The existing Bellota Weir facility on the Mormon Slough/Calaveras River is owned and operated by SEWD and provides water to urban and agricultural users. The Bellota Intake provides municipal and industrial flow year-round for the City of Stockton, and also supplies irrigation water during the irrigation season (generally between mid-April and mid-October). The Old Calaveras Headworks (Calaveras Headworks), provides flow control and prevents flooding to downstream landowners on the Old Calaveras River by routing flood waters down the Mormon Slough. During most flow regimes, both the Bellota Weir and Calaveras Headworks are complete fish barriers to upstream migrating adult salmonids.

Continued operation of the Bellota Weir and diversion will be guided by the final Calaveras River Habitat Conservation Plan (CHCP; National Oceanic and Atmospheric Administration 2020). The CHCP provides operational criteria to support the biological goals of maintaining a viable population of threatened California Central Valley steelhead, *Oncorhynchus mykiss,* within the CHCP boundaries, and maintains adequate habitat conditions upstream of Bellota for fall-, late fall-, spring- or winter-run Chinook Salmon, *Oncorhynchus tshawytscha*, that may opportunistically migrate into the conservation area. While the CHCP intends to provide conditions that support Chinook Salmon should they migrate into the conservation area, these salmon are not expected to maintain a viable population based both on pre-dam and current conditions. The CHCP enables SEWD to comply with the Endangered Species Act (ESA), protecting and managing fishery resources and habitat while maintaining reliable water delivery to its constituents. Following NMFS approval on August 11, 2020, the District is authorized for a 50-year Incidental Take Permit (ITP #23264), for ESA-listed species under NMFS authority.

Upgrade or replacement of the Bellota Diversion, Weir, and Calaveras Headworks is a required compliance measure specified in the CHCP, as part of a seven target fish passage objective. Specifically, the CHCP's target states:

> FP1 and AE1. Avoid migration delays and blockage, and entrainment within the Old Calaveras River Channel by constructing a non-entraining barrier at the Old Calaveras River Headworks Facility and at the downstream end of the channel near the confluence with the [Stockton Diverting Canal] within the first ten years of the ITP.
>
> FP2/AE3. Construct and implement a combined crest gate/fishway/fish screen at the Bellota Diversion Facility to improve [salmonid] passage into/out of the 18-mile spawning and rearing reach between Bellota and New Hogan Dam and to

> prevent fish entrainment; target completion within first five years, but no later than 10 years of [issuance of] the ITP.

Based on the CHCP targets summarized above and SEWD's water supply needs, the three primary Project objectives are to:

- **Improve fish passage** by designing and constructing a new crest gate dam, fishway, and fish screens that includes passage for *Oncorhynchus mykiss* and opportunistic migration for fall-, late fall-, spring-, or winter-run Chinook Salmon.
- **Reduce fish entrainment** by constructing a new non-entraining fish barrier at the Old Calaveras River.
- **Provide more reliable water delivery** through weir and intake improvements.

1.4 Existing Facility

1.4.1 Bellota Weir

Bellota Weir is the largest check dam on the Calaveras River system. The weir is located on the Mormon Slough approximately 350 feet downstream of its confluence with the Old Calaveras River. The weir is an in-river concrete structure designed to accommodate installation of a flashboard dam; during irrigation season, the check dam crest with the flashboards installed is 8 feet above the channel invert (invert elevation of 121.44 feet [ft] North American Vertical Datum of 1988 [NAVD88]). During non-irrigation seasonal (flood season) periods of the year, shorter flashboards are installed 2 feet above the channel invert. Existing features of the Bellota Weir Modifications Project are identified in the Figure 1-1.

Figure 1-1. Prominent Features of the Bellota Weir Modifications Project Site



Bellota Weir includes two Denil fish ladders, installed in 1999 and 2001. During the irrigation season, only the downstream Denil fish ladder is in place, while during non-irrigation times, both Denil fish ladder sections are in place. Employees from CDFW and Fishery Foundation evaluated conditions at the current ladders in 2007. The upper ladder is longer than the maximum recommended ladder length of 30 feet. The lower ladder has a 1 horizontal to 3 vertical (1H:3V) slope ratio, double the maximum recommended slope of 1H:6V. Finally, water does not flow through the lower ladder as intended but instead around the sides resulting in an inadequate water supply to the ladder that confuses fish looking for the ladder entrance (California Department of Water Resources [CDWR] 2007).

1.4.2 Bellota Intake

The Bellota Intake is a gravity diversion constructed in 1978, on the south riverbank about 50 feet upstream of the Bellota Weir. Water diverted through the intake is conveyed to the City of Stockton and the Dr. Joe Waidhofer Water Treatment Plant (SEWD, HDR, and KSN 2020). The structure possesses bulkhead slots for installation of stop logs, a trashrack facility, two 14-foot-wide rectangular intake channels perpendicular to the river, two intakes with fish screens, a bypass pipeline, a sediment trap, and two slide gates to regulate flow.

1.4.3 Calaveras Headworks

The Calaveras Headworks is located approximately 600 feet downstream from the Old Calaveras River, Mormon Slough confluence (Figure 1-1). The existing structure includes an earthen berm, four culverts with control gates, a trashrack, and concrete slope lining.

The earthen berm crest is 68 feet long and 10 feet wide. The 4-foot-wide, concrete culverts penetrate the berm and control flow to the Old Calaveras River. There is a 21-foot-wide trashrack on the upstream side of the berm protecting the culvert inlets, their concrete headwall, and wing walls. Flow is controlled with slide gates on the upstream side – two are operated manually and two are electrically actuated. The slide gates are operated between April and November to deliver water to downstream agricultural users, and intermittently between November and April for groundwater recharge on the Old Calaveras River and open during irrigation season to provide flow to downstream diverters (SEWD and FISHBIO 2019).

The headworks is a barrier to upstream adult fish migration and does not comply with juvenile downstream passage or fish protection guidelines provided by CDFW and NMFS.

In 2003, SEWD placed a small mesh net across the Old Calaveras River upstream of the Old Calaveras Headworks structure (CH2M 2003) to encourage juvenile fish to reside in the Bellota Pool and migrate upstream to suitable habitat or downstream to the Mormon Slough.

1.4.4 Geotechnical Conditions

Existing site information includes U.S. Geological Survey (USGS) – Geology and previously conducted drilling at the site by CH2M conducted between October 2004 and January 2005. Five borings were visually observed and logged during the previous exploration program along the southeast bank of the Mormon Slough at the previous proposed fish ladder location. The investigation indicates that the Project site is generally underlain by riverine alluvial deposits to the depths explored (CH2M 2005). The geologic map indicates presence of Holocene alluvium channel and levee deposits and Upper Modesto formation (Marchand and Bartow 1979). The geotechnical conditions identified during this study show there is a risk of soil liquefaction at the site.

Further geotechnical investigations have been initiated as part of the Bellota Weir Modifications Project including advancing five mud-rotary borings along the northwest bank to characterize subsurface condition in the revised location of the new fish ladder. Geotechnical laboratory testing including tests for soil classification, plasticity, and strength will be performed. A geotechnical report documenting the investigations performed will be prepared for use during design.

Review of available aerial photographs and topographic maps reviewed as part of the current study indicate that the Bellota Diversion was first created in the late 1940s by deepening the Mormon Slough and using the material to create a gravel weir at its current location, as indicated on both the 1953 and 1972 USGS 7.5-minute quadrangle topographic maps. Review of available as-built drawings indicate 1) the current weir was constructed in about 1968, 2) the current weir was constructed along the original weir alignment, and 3) the original weir had been severely eroded (scoured) by stream flows.

The amount of instream grading performed as part of the new weir construction is not documented in the plans. The current weir is likely underlain by 3 to 5 feet of mounded river granular sediments. Preliminary review of the borings indicates the base of the coarse gravels that underlie the site is between elevation 95 to 100 ft (NAVD88).

1.4.5 Debris Management and Sediment Characterization

After periods of high flows, sediment and debris removal efforts are occasionally necessary at the Bellota Intake. Given its orientation along the channel bank and presence of the existing profile control structure, slow water velocities cause fall out of fine sediments and accumulation of buoyant and semi-buoyant debris at the intake debris racks. Debris and sediment observed at the weir and intake are characterized as sands, silts, leaves, brush, trees, and floating trash. SEWD possesses a CDFW-issued permit for gravel removal in front of the intake structure (CH2M 2003).

There are often leaves and sticks around the existing fish screens. Larger debris travels toward the weir, unless water is being pulled to the intake. Existing intake screen cleaning is inadequate – brushes push off debris but leaves and sediment return. Sediment is cleaned from the intake approximately every two years with a routine maintenance agreement (SEWD, HDR, and KSN 2020).

1.5 Proposed Facility

The proposed Project facility combines several operational and engineered elements to achieve its intended objectives: intake and weir replacement and improvements for effective fish passage, reduced fish entrainment, and water supply reliability. The proposed Project elements are briefly summarized below and illustrated in Figure 1-2:

- Construct a concrete dam with a series of weir gates designed to modulate forebay pool elevations based on operational needs.
- Improve sediment conveyance with the inclusion of a vertical slide gate at the dam, operated to facilitate bypass and sluicing functions.
- Provide reliable fish passage using a roughened channel in combination with a vertical slot fishway to accommodate the full range of flows and river stages at the site.
- Provide reliable fish screening, both for the Bellota Intake and the Old Calaveras River, to prevent fish entrainment.
- Provide a non-entraining fish barrier to the Old Calaveras River.

Figure 1-2. Overview of Proposed Project Elements

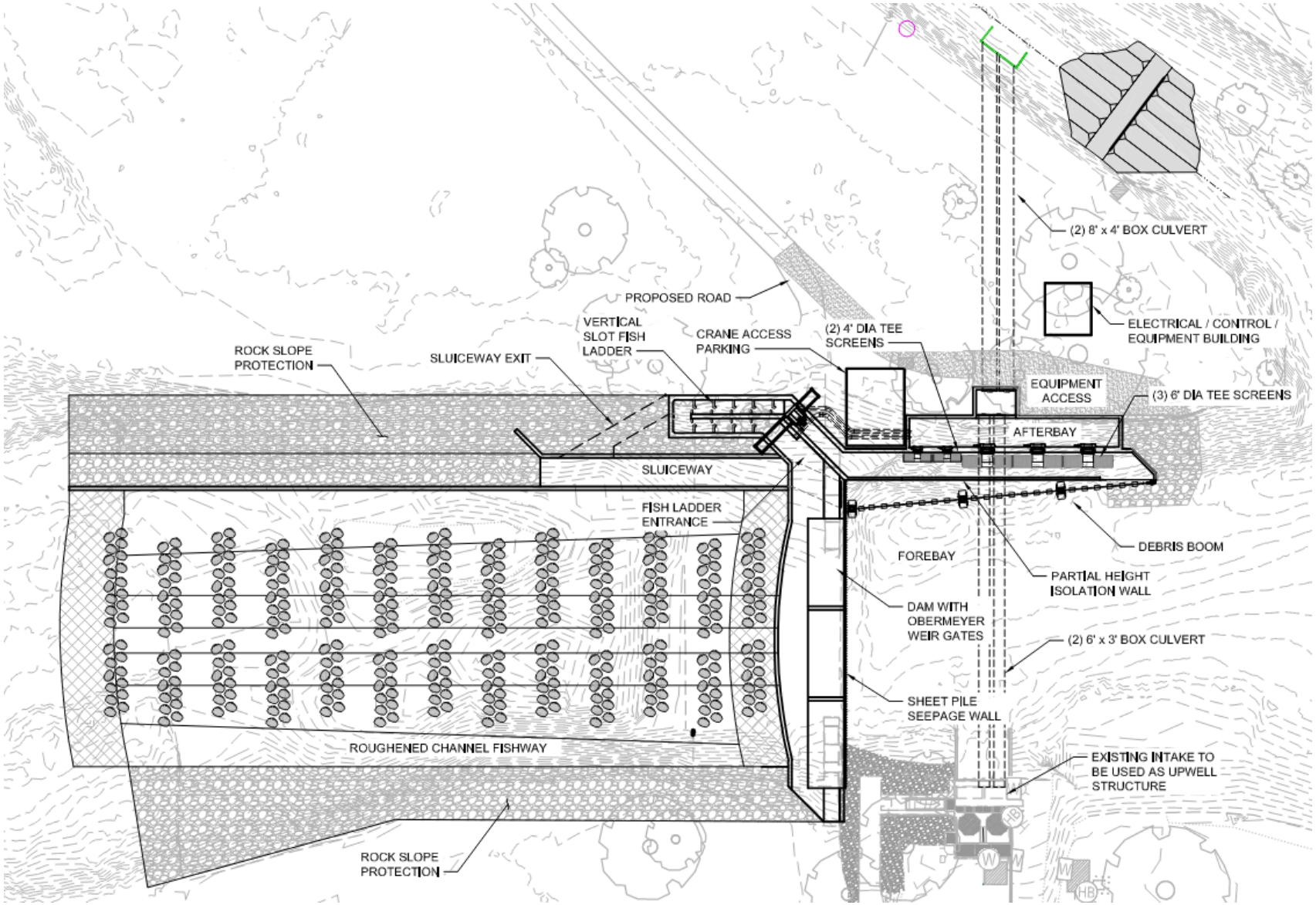
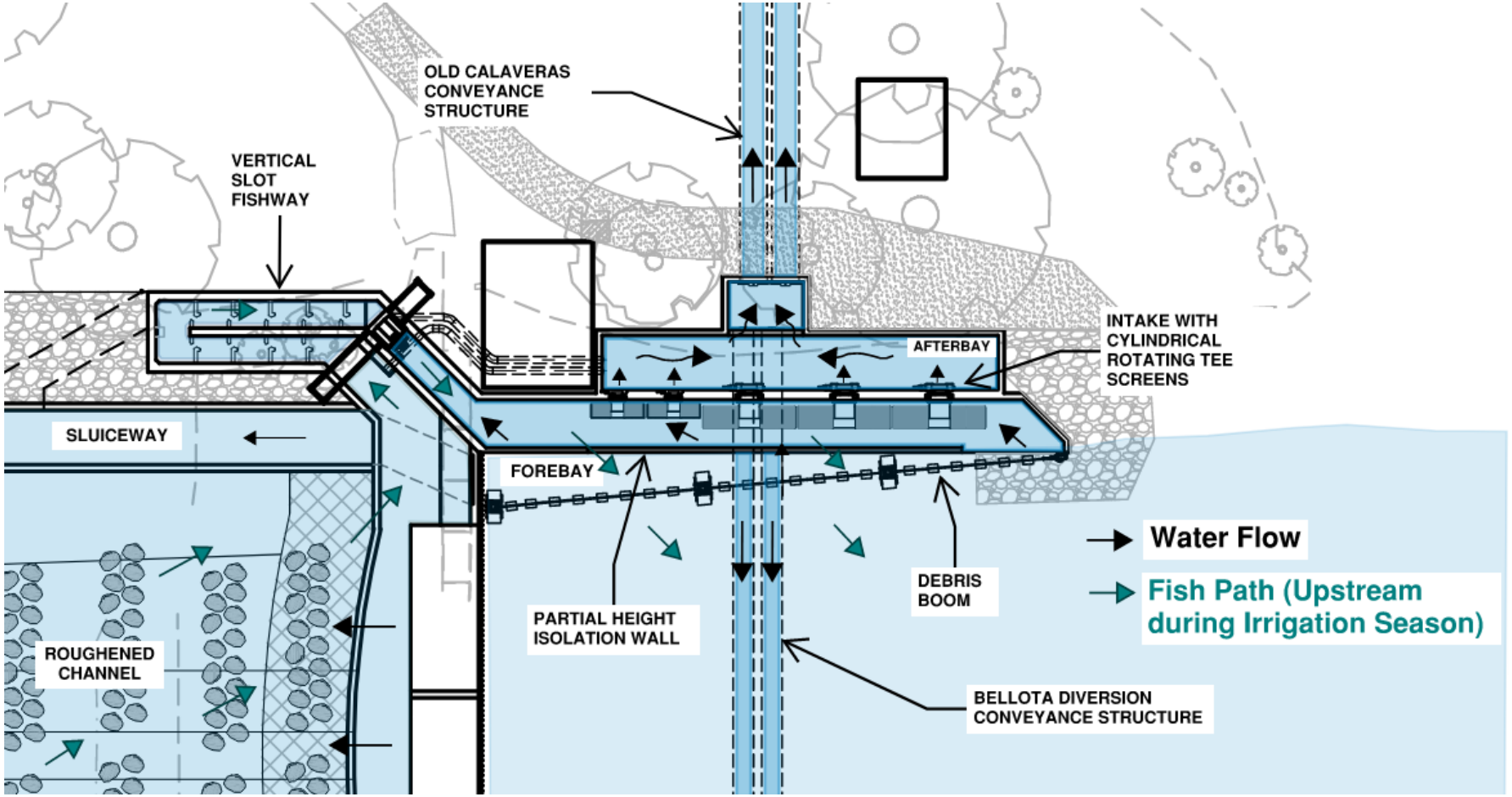


Figure 1-3. Overview of Surface Water Intake and Fish Ladder



2 Biological Criteria

This section presents biological criteria used in the design of the facility, including target species, fish migration timing, and fish passage performance.

2.1 Habitat Conditions and Related Projects

A CDWR evaluation of spawning gravels and riparian canopy upstream of the Bellota Weir described the existing Calaveras River as adequate (CDWR 2007). For the 2007 CDWR report, temperatures were noted as cold enough to sustain cold-water fish, vegetation in the reach ranged from orchards to riparian forest and grassland, and runoff to the stream was thought to contain nutrients, bacteria, and sediment. In 2004, Stillwater Sciences opined that habitat in the reach is capable of supporting self-sustaining populations of steelhead and fall and spring runs of Chinook Salmon (CDWR 2007)[1].

NMFS in 2014 reported that the Calaveras River has the potential to support anadromous fish based on habitat characteristics such as a 22-feet-per-mile gradient, numerous riffles and pools, adequate spawning gravels, and a dense riparian canopy. Downstream of Bellota Weir, spawning gravels are limited yet known to produce fry salmon (NMFS 2014).

Based on these documents and information in the CHCP, the stream reach between the Bellota Weir and New Hogan Dam is desirable habitat for coldwater salmonids. Related projects in the area such as the Budiselich Flashboard Dam, Caprini Low Flow Road Crossing, Hosie Low Flow Road Crossing (currently in design), and Central California Railroad Traction Crossing have also been constructed or are in planning phases to pass fish from the confluence with the San Joaquin River down to New Hogan Dam. As these projects are implemented, the range of habitat accessible to migrating fish will expand up to the Bellota Weir Project area. The Project will correct the last major impediment for migrating fish identified in the CHCP, and enable migrating fish to reach spawning habitat between the weir and New Hogan Dam.

2.2 Target Species

The CHCP identifies fish species that inhabit or may inhabit the Project reach and include the following (SEWD and FISHBIO 2019):

- Steelhead
    - Listed as threatened under the ESA by NMFS

[1] Spring-run Chinook are notably extirpated from the San Joaquin River and its tributaries. Only an experimental population exists on the San Joaquin River that is not recognized for Section 7 protection under the ESA. In addition, the rain-fed seasonal hydrology (naturally occurring periodic discontinuous flow between the Calaveras River and its confluence to the San Joaquin River) results in opportunistic access for Chinook Salmon based on water year type and precipitation-driven flow events.

- Fall-, late fall-, spring-, or winter-run Chinook Salmon
    - Fall-run Chinook Salmon is the evolutionarily significant unit most frequently reported in the San Joaquin basin and considered a Species of Concern by NMFS and a Species of Special Concern by CDFW

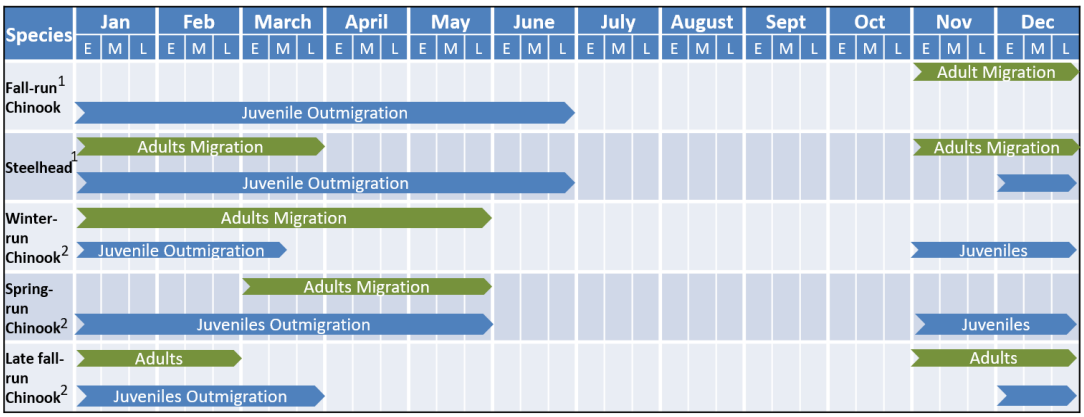
Steelhead and fall-run Chinook Salmon are considered the species currently using the Calaveras River; the spring- and winter-run Chinook Salmon are not likely to use the Calaveras River. The Bellota Weir modifications will provide appropriate adult passage conditions for each of the salmonids identified above. SEWD will target flow management for steelhead, but also provide opportunistic passage for Chinook Salmon as natural hydrology (i.e., rain event) allows.

Although not listed in the CHCP, lamprey passage will be considered during design development as discussed with SEWD and the fish agencies.

2.3 Fish Migration Timing

Outmigration monitoring and snorkel surveys on the Calaveras River by FISHBIO provide the basis for periodicity in the Calaveras River. SEWD, in conjunction with FISHBIO, created tables summarizing existing data regarding the life stage timing for the target species of the CHCP. Figure 2-1 shows November through March will be the busiest fish passage months for species likely to use the Calaveras River. For potential species not currently observed in the Calaveras River, the adult migration months will extend from November through May.

Figure 2-1. Life Stage Timing for Covered CHCP Species that May Utilize the Calaveras River



[1] Species likely to utilize the Calaveras River

[2] Potential species not currently observed in the Calaveras River

Source: SEWD and FISHBIO (2019)

In summary, any fish passage facility located at the Bellota Weir will need to accommodate the following:

- Adult salmonids upstream migration from November through May
- Juvenile salmonid downstream migration from November through June

It is not currently determined whether or not this facility will require design for the upstream migration of juvenile and resident fish.

2.4 Fish Passage Performance

There is limited guidance that provides quantitative and measurable performance criteria specific to the design of passable, nature-like, roughened-channel fishways. Guidance varies by region, river composition, and the life history of the various target species present. In addition to stability throughout the full range of hydrologic conditions, roughened channels must be designed to accommodate the specific swimming requirements of the target fish species and life stages selected for design, which requires understanding the anticipated swimming speeds, endurance, and distances fish are able to navigate given the flow velocities and energy expended as they attempt to ascend the roughened channel feature, or a portion thereof. Available literature provides a wide range of swimming and energy expenditure characteristics, models, and approaches that can be used for a general understanding of relative swimming capability. However, there is tremendous variability in the available data and not one resource is typically relied upon for use in design development. Individual data sets offer significant variation in test conditions, methods, metrics, and thus conclusions.

In general, fish swimming speed through water is dependent on fish species, length, condition, time to exhaustion (Hunter and Mayor 1986) as well as other physical conditions such as temperature, oxygen saturation, and turbidity. Fish may travel expending low levels of energy for a longer period of time (sustained speed) and at maximum effort (high level of energy expenditure) for a short period of time (burst speed). Moderate levels may be exerted for time durations in-between (prolonged speed).

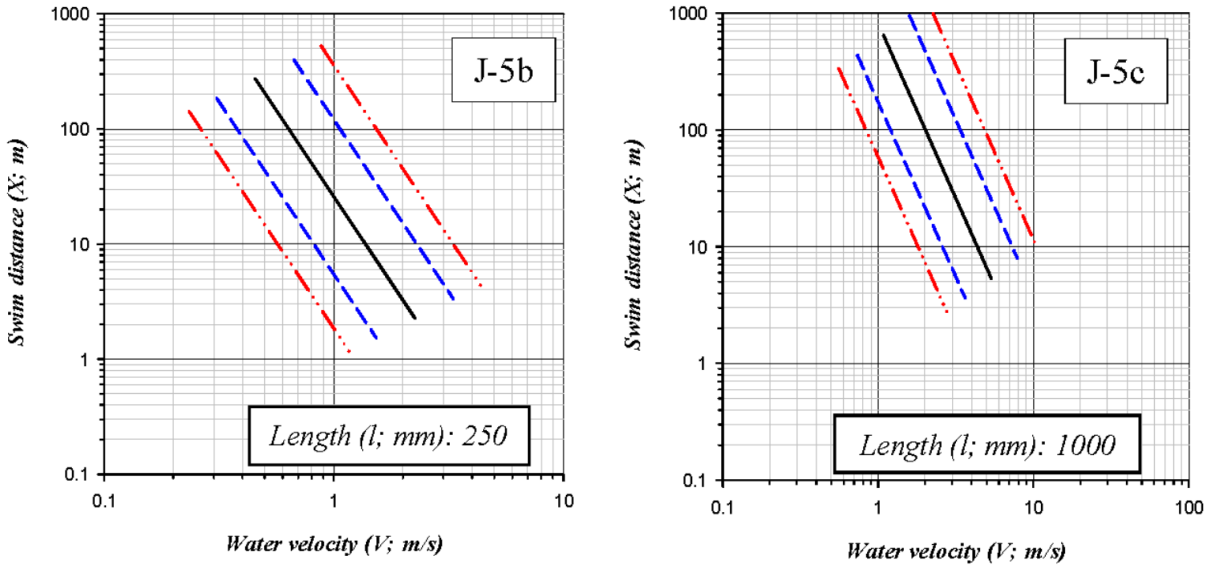
The Fish Swimming Performance Database and Analyses report presents and summarizes an extensive database of fish swimming performance generated from literature and data sets obtained after 1990 (Katopodis and Gervais 2016). These data are used to generate swim speed, fatigue, and swim distance curves that can be used in the design formulation process. For derived swim distance, water velocity curves are presented for a number of fish groups such as salmon over a range of size classes.

Fish passage performance for the roughened channel and pilot channels will be examined based on a biometric comparison of the velocity predicted from two-dimensional (2D) modeling results and fish swimming capability using the flow velocity versus fish swimming distance relationships developed in Katopodis and Gervais (2016). These nomographs use dimensionless time and endurance statistics from lab results to model the distance a fish can travel given a specific river flow velocity. The nomographs lump salmonids into one general category but provide specific curves for 250-millimeter (mm) sub-adult fish and 1,000-mm adult fish. The nomographs also provide separate lines that represent upper and lower estimates of swim time and distance passed on prediction intervals of 75 and 95 percent. For this assessment, the 75 percent prediction intervals will be used to approximate a condition that accommodates fish passage for 75 percent of the fish population. Nomograph data for 1,000-mm adult fish also are reduced to 710 mm adult fish (approximately 28 inches) using a dimensionless size reduction ratio to better reflect the size class of adult

salmonids present within the Calaveras River. Each swim distance curve corresponds to endurance intervals ranging from 3 seconds (burst swim speed) to 30 minutes (prolonged swim speed).

The curves will be used to evaluate the maximum distance a fish can swim given a hydraulic pathway exhibiting an anticipated flow velocity. Swimming pathways will be evaluated based on the total length and proximity to low velocity areas that represent resting areas. Higher velocities predicted by the 2D model will require design features be added to increase the frequency of resting areas. Lower velocities predicted from 2D modeling will require fewer resting areas. The curves are presented in Figure 2-2.

Figure 2-2. Swim Distance vs. Flow Velocity for 250 mm and 1,000 mm Salmonids



The nomographs J-5b and J-5c were used to tabulate correlated metrics that will be used to evaluate fish passage performance of the proposed roughened channel and pilot channels. The time to exhaustion, water velocity, swim distance, and correlating fish swim speed for the three size classes of salmonids are presented in Table 2-1. Only values representing the 75th percentile are reported for comparative purposes.

Table 2-1. Swimming Speeds for 250-mm, 1,000-mm, and 710-mm Salmonids for 75th Percentile

Fish Length	Time	Water Velocity (fps)	Swim Distance (ft)	Correlated Swim Speed (fps)
250mm[a] (10 in)	5 seconds	9.5	16.4	12.8
	20 seconds	6.9	45.9	9.2
	3 minutes	4.3	229.6	5.5
	30 minutes	2.1	1312.0	2.9
1,000mm[a] (39 in)	5 seconds	23.3	39.4	31.2
	20 seconds	16.4	111.5	22.0

Table 2-1. Swimming Speeds for 250-mm, 1,000-mm, and 710-mm Salmonids for 75th Percentile

Fish Length	Time	Water Velocity (fps)	Swim Distance (ft)	Correlated Swim Speed (fps)
	3 minutes	9.2	590.4	12.5
	30 minutes	5.3	3116.0	7.0
710mm[b] (28 in)	5 seconds	18.0	30.5	24.1
	20 seconds	12.7	86.2	17.0
	3 minutes	7.3	450.9	9.8
	30 minutes	4.0	2418.5	5.4

fps = feet per second

a. Obtained from Katopodis and Gervais (2016), Appendix E, Charts J-5b and J-5c.
b. Interpolated from Katopodis and Gervais (2016) data using dimensionless length ratio.

Performance criteria also will include depth. For adult salmon and Steelhead Trout, depths of 0.9 foot or greater will be used. Juvenile and resident upstream migrants will require 0.4 foot or greater passage corridors depths (CDFW 2017). Hydraulic pathways not meeting the above criteria will be identified as sub-optimal and a potential impediment to upstream migration.

3 Facility Criteria

Design criteria related to operational strategy, irrigation demand, and hydrology are presented in this section.

3.1 Operational Strategy

The Project has four major operational components: intake facility, sluiceway, roughened channel fishway, and conveyance pipelines. Of these, equipment present in the intake and sluiceway will be operated under specific conditions to meet the primary objectives of the Project. Flow will be conveyed down the roughened channel to facilitate fish passage under all operational scenarios.

Facility equipment will operate per the following four conditions:

- Non-operation
- Normal operation during the diversion season
- High-flow conditions during the diversion season
- Periodic sluicing of debris and sediment through the concrete sluiceway

3.1.1 Existing Operation

Existing operational considerations of the Project facilities change seasonally. Flood control season generally occurs from October to April, while irrigation season occurs from April to October. Installation and takedown time of the flashboards may vary slightly from year to year depending on irrigation needs or the arrival or termination of the flood control season.

Non-Irrigation Season

The New Hogan Dam is operated to meet a maximum 12,500-cubic foot per second (cfs) objective release downstream in the Calaveras River/Mormon Slough, with consideration of the following (U.S. Army Corps of Engineers [USACE] 1999):

- Releases are not to be increased or decreased more than 2,000 cfs in any 2-hour period.
- After a storm event, any major release to evacuate flood management storage can potentially affect downstream property owners, as well as the river channel. Releases must balance the potential damage and the need to evacuate flood management storage.

Reservoir releases in the non-irrigation season from mid-October to mid-April are operated by the USACE primarily for flow and flood control purposes to protect agricultural, urban, and suburban land along the Calaveras River, Mormon Slough, and Stockton Diverting Canal (CH2M 2006). During non-irrigation season, New Hogan Dam water conservation storage includes a reservation for flood control storage. Use of in-river dams and flashboard structures is limited to provide maximum flood-flow capacity in the river system.

Bellota Weir and Bellota Intake

While most agricultural diversions have limited use during the flood season, SEWD and Calaveras County Water District (CCWD) divert flow for municipal purposes year-round. During the non-irrigation season (generally mid-October through mid-April), 2-foot flashboards are installed on the Bellota Weir to keep the headwater elevation at approximately 115.44 feet NAVD88, which allows the Bellota Intake to divert a maximum 46 cfs for municipal and industrial purposes. When the headwater elevations exceed 115.44 feet NAVD88, flows can be released to the Old Calaveras River (CH2M 2006). Table 3-1 provides a summary of the existing operational diversion demands of the Bellota Weir and Bellota Intake.

Table 3-1. Summary of Target Design Flows and Seasonality for the Bellota and Old Calaveras River Diversion Structure

Demand/ Flow Description	Diversion Demands (cfs)		Calaveras River Flows (cfs)
	Irrigation Season	Non-Irrigation Season	Flood Control
	April 10 to October 15	October 16 to April 9	All Year
Current Demands/Flows			
Bellota Diversion	46-75	46	N/A
Old Calaveras River Diversion	0-160	0-30	N/A
Subtotal, Diversion Flows	46-235	46-76	N/A
Mormon Slough, downstream of Bellota	20-100	N/A	0 to 12,644
Total River Flows	**0 to 12,690 [a]**		
Future Demands/Flows[b]			
Bellota Diversion[b]	60-173	60	N/A
Old Calaveras River Diversion	0-300	0-30	N/A
Subtotal, Diversion Flows	60-473	60-90	N/A
Mormon Slough, downstream of Bellota	20-100	N/A	0 to 12,630
Total River Flows	**0 to 12,690 [a]**		

[a] 100-yr flood flow of 12,690 cfs per FEMA Flood Insurance Study (2016); 500-yr flood flow per FEMA is 16,000 cfs

[b] Future Irrigation season flows were calculated as follows:

SEWD minimum diversion flow: 46 cfs + 30% redundancy = 60 cfs

SEWD maximum diversion flow: 75 cfs existing + 30% redundancy + 75 cfs future = 173 cfs

N/A – Not Applicable

Calaveras Headworks

Flow to the Old Calaveras River is mostly for downstream irrigation and groundwater recharge needs. When water is available from the New Hogan Dam, surface water is available to the approximately 80 diversions downstream of the Calaveras Headworks. However, in low flow years water may only be diverted to the Mormon Slough, and diverters may need to pump groundwater (CDWR 2007).

Table 3-1 provides a summary of the existing operational diversion demands of the Old Calaveras River.

Irrigation Season

Under non-flood-control periods and during the irrigation season, SEWD's and CCWD's needs dictate releases from the New Hogan Dam. Dam releases tend to increase in April when irrigation season begins.

Bellota Weir and Bellota Intake

During the irrigation season, an 8-foot flashboard check dam is installed on the Bellota Weir to maintain the headwater elevation at approximately 121.44 feet NAVD88 (Figure 3-1). SEWD identified that the 8-foot flashboard installation and uninstallation may occur at different times each year (as early as March 20 and after October 15) depending on irrigation needs and flood arrival (SEWD, HDR, and KSN 2020). The flashboard dam maintains the headwater elevation of the weir at approximately 121.44 feet NAVD88, which allows the Bellota Intake to divert 46 to 75 cfs maximum for municipal and industrial purposes as well as irrigation. Table 3-1 provides a summary of the existing operational diversion demands of the Bellota Weir and Bellota Intake.

Figure 3-1. Bellota Weir with 8-Foot Flashboards Installed



The flashboard dam causes a pool at Bellota that extends approximately 1.5 miles upstream of the structure, which increases the pressure head of the Bellota Intake and benefits upstream water diverters. The seasonal flashboard check dam also helps concentrate flows into a seasonal fish ladder installed at Bellota Weir during spring.

SEWD identified the flow released downstream of Bellota Weir ranges from 20 to 100 cfs during the irrigation season; there is a call-in service for diverters to identify when they need more water (SEWD, HDR, and KSN 2020).

Calaveras Headworks

During the irrigation season, the Mormon Slough and Calaveras River are watered to supply surface water to irrigators (CDWR 2007). A maximum of 160 cfs may be diverted down the Old Calaveras River. Table 3-1 provides a summary of the existing operational diversion demands of the Old Calaveras River.

3.1.2 Future Operation

In the future, general operational characteristics of the facilities will remain the same; however future system demands may change.

Non-Irrigation Season Demand

- The existing minimum required flow at the Bellota Intake will remain 46 cfs, but include a 30 percent redundancy factor. As a result, the minimum demand during non-irrigation season will be 60 cfs, corresponding to a minimum headwater elevation of 115.44 feet NAVD88.
- The demand for the Old Calaveras River will remain 30 cfs.

Irrigation Season Demand

- The existing minimum required flow at the Bellota Intake will remain 75 cfs but include a 30 percent redundancy factor. As a result, the minimum demand during irrigation season will be 98 cfs, corresponding to a minimum headwater elevation of 121.44 feet NAVD88.
    - SEWD is expected to increase the Bellota Intake capacity by an additional 75 cfs to meet future needs. Therefore, the minimum future demand during the irrigation season will be 173 cfs, corresponding to a minimum headwater elevation of 121.44 feet NAVD88.
- The demand for the Old Calaveras River will increase to 300 cfs.

Design flows for both the existing and future operation of the Bellota Weir and Calaveras Headworks are summarized in Table 3-1.

Design Life

The Project components will be designed to accommodate a typical planning period ranging from 15 to 50 years. The generally accepted life of equipment, electrical systems, structures, and buildings is as follows:

- **Equipment:** 15 to 25 years; maybe longer if the equipment is properly operated and maintained
- **Electrical systems:** 25 to 30 years
- **Concrete structures:** 50 years

- **Major pipelines:** 50 years
- **Buildings:** 30 to 50 years

Project failure at this site is defined as the inability to effectively pass fish through the reach, pass flood flows past the improvements, or provide a reliable water supply to meet irrigation demands. These occurrences will be minimized to the extent possible. The structural integrity of the weir crest will not be compromised during or following any given flow event up to the 100-year flow.

It is recognized that flows greater than the 100-year event may occur during the useful lifespan of the proposed Project improvements. The level-of-service interruption or damage that may occur during an event of this nature may be evaluated further using an uncertainties analysis during a later stage of design.

Minimum Required Operating Water Surface Elevation

The minimum required operating water surfaces in the irrigation and non-irrigation seasons is 121.44 ft and 115.44 ft, respectively. The non-irrigation operating water surface will be used as the minimum design operating water surface in the Mormon Slough and the limiting condition for design of gravity flow hydraulics through the intake and proposed conveyance systems.

3.2 Calaveras River Hydrology

This section describes the Calaveras River hydrology, including historical and ongoing flow regulation, Project-specific flow uncertainties, monthly and seasonal flow characteristics, flow duration, flood recurrence, and fish passage design flows.

Hydrology in the region was studied and analyzed using desktop research, a USACE-installed gage downstream of the weir, and data provided by FEMA.

3.2.1 Historic and Ongoing Flow Regulation

Calaveras River/Mormon Slough flows are regulated by operation of New Hogan Dam about 17 miles upstream of the bifurcation of the Mormon Slough and Old Calaveras River. Discharge from the dam is altered throughout the year. Additionally, diversions throughout the reach between Bellota Weir and New Hogan Dam contribute to water withdrawals that must be considered when evaluating low flow conditions.

Flow release from New Hogan Dam is regulated by the USACE; rights to storage releases are held by the U.S. Bureau of Reclamation (Reclamation) and contracted to SEWD and CCWD. Streamflow records are available using a USACE gage for both pre- and post-construction of the dam according to the CH2M reports, but gage information prior to 2002 was not found online and multiple information requests to USACE yielded no results. There is no USGS gage nearby.

Releases to the river are lowest between October and December. The dam occasionally releases flood flows between January and March but become more consistent during the irrigation season. These flows supply most of the irrigation demand for diversions between the dam and weir (CH2M 2003). Because much of the upper watershed is

below elevation 4,000 feet (typical winter snowline), flow in the Calaveras River is heavily reliant on precipitation instead of snow melt unlike many other rivers nearby.

3.2.2 Project-Specific Flow Uncertainties

Numerous hydrologic inputs and withdrawals in the Project reach of the Calaveras River system create high flow variability during seasonal low flow conditions. The Fish Passage Barriers Assessment report shows there are 29 diversions between the weir and New Hogan Dam on the Calaveras River (CDWR 2007).

In addition, New Hogan Lake influences river flow through reservoir releases, which may devalue the exceedance analysis performed. High or low flows may result from reservoir releases rather than natural hydrology.

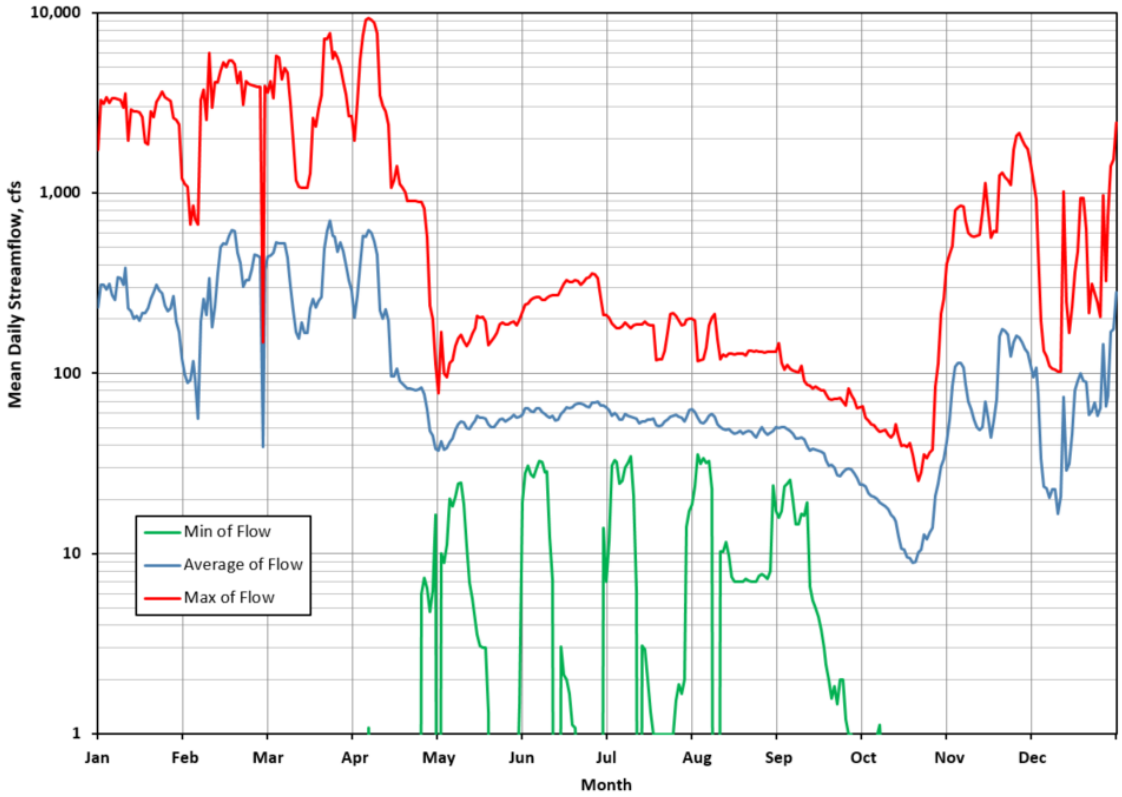
The following assumptions have been adopted as part of the design process:

- Mean daily flow statistics will be based on data available after 2001 (USACE 2002). Use of data recorded between January 1, 2002 and February 13, 2020 provides an 18-year period of record that gives an adequate representation of current and future operational conditions in the Project reach. This information will be used herein to make reasonable assumptions for development of design criteria used for alternative assessment. The CH2M- and USACE-referenced additional period of record at this gage was not located but has been requested.
- The exceedance analysis is valid considering generalities and seasonalities; it is influenced by New Hogan Dam releases and diversion of flow. Any adjustment in dam releases or diversion intakes will influence flow magnitudes at the Project site.

3.2.3 Monthly and Seasonal Flow Characteristics

Mean daily streamflow data obtained from USACE Gage ID MRS were examined over the 2002 to 2020 period of record to study monthly and seasonal flow patterns of the Calaveras River at Bellota (USACE 2002). When CH2M performed a mean daily streamflow analysis of the gage data, 75 cfs was added to each diversion flow value to account for flows entering the Bellota Intake or those directed down the Old Calaveras River. The 75 cfs is not included in the flow for the proposed structural weir modifications, as the intake is located upstream of the weir. Figure 3-2 illustrates the mean daily minimum, average, and maximum flows over the 18-year period of record.

Figure 3-2. Mean Daily Minimum, Average, and Maximum Flows at the Bellota at Mormon Slough Gage ID MRS, Period of Record 2002 to 2020



Examination of mean daily flows over the period of record at Gage ID MRS illustrates the variability of annual flows, as well as typical flows observed during different seasons. These results show that typical high flow periods occur in the late fall, winter, and early spring seasons with maximum mean daily flows between 6,000 and 10,000 cfs. During these seasons, gage flows are variable with observed mean daily minimums as low as 0 cfs, because there are several diversions between Bellota Weir and the New Hogan Dam that divert water from the river. Annual average mean daily flows of 0 cfs were recorded for 208 days throughout the period of record. Installation of an 8-foot flashboard check dam on the Bellota Weir during the irrigation season is clearly visible in the flow record from mid-April until mid-October; between these time periods, backwater caused by the check dam allows the nine diversions in the Bellota Pool area to pump water from the river reducing flow in the Mormon Slough, downstream of Bellota.

In a February 27, 2020 workshop, SEWD stated 20 to 100 cfs of flow is released downstream of Bellota Weir during the irrigation season (SEWD, KSN, and HDR 2020). Based on this information, the minimum flow shown in Table 3-1 may not reflect current downstream flow conditions.

3.2.4 Flow Duration

A flow duration analysis was performed using mean daily flow data obtained at the USACE Gage ID MRS for the period of record from 2002 to 2020 resulting in a range of values for percent chance exceedance. The resulting percent chance exceedance and corresponding flows statistically represent the flow equaled or exceeded during certain

percentages of time. The analysis described herein was performed for two durations: 1) annual and 2) during irrigation diversion season. The results of each analysis are provided in Table 3-2.

Table 3-2. Percent Chance of Exceedance of Mean Daily Flows for Period of Record 2002 to 2020

Percent Chance of Exceedance (%)	Annual Duration (cfs)	Irrigation Season Mid-April to Mid-October (cfs)
99	0	0
95	0	7
90	1	17
80	7	25
75	11	28
50	33	40
25	53	51
20	63	55
15	92	63
10	141	84
5	394	127
1	3,258	310

Values presented in Table 3-2 suggest that the mean daily flow exceeded 50 percent of the time (P=50%) is 33 cfs when considering flows occurring all year and increases to 40 cfs during the irrigation season. The mean daily flow exceeded 95 percent of the time is 0 cfs on an annual basis and 7 cfs during the irrigation season. The mean daily flow exceeded 5 percent of the time is 394 cfs and 127 cfs for the annual and irrigation season periods, respectively. During the February 27, 2020 workshop, SEWD stated that the 1 percent exceedance flows during the irrigation season are sometimes above 310 cfs and have occurred in the range of 400 to 600 cfs in Mormon Slough (SEWD, HDR, and KSN 2020).

3.2.5 Flood Recurrence

Flood recurrence in the Calaveras River and Mormon Slough at Bellota was studied by FEMA (2016). On February 20, 2020, HDR performed a Log Pearson Type III (Bulletin 17B) analysis on the available data for the period of record January 1, 2002 to February 13, 2020. The resulting values calculated for various recurrence intervals correlate relatively well with the FEMA flows (Table 3-3). Key values such as the FEMA 100-year storm event (12,690 cfs) are used in this document to establish water surface profiles at the Project location for alternative development, comparison, and measurement of performance. The FEMA values will be used for design, while the Log Pearson Type III values were used to confirm the accuracy of the FEMA study.

Table 3-3. Flood Frequency and Magnitude of Calaveras River Flow at Mormon Slough at Bellota USACE Gage ID MRS

Recurrence Interval	Annual Exceedance Probability (%)	FEMA (2016) (cfs)	Log Pearson Type III Analysis (cfs)
1-year	99.9	-	47.7
2-year	50	-	2,221
5-year	20	-	5,566
10-year	10	12,500	8,244
20-year	5	-	10,934
50-year	2	12,500	14,402
100-year	1	12,690	16,908
500-year	0.2	27,100	22,204

As provided in the table, the FEMA 50-year and 100-year flows are similar due to the annual objective release flow target from New Hogan Dam, which is 12,500 cfs.

3.2.6 Fish Passage Design Flows

Selection of site-specific fish passage design flow criteria influences fish passage facility size, complexity, and anticipated performance. Guidelines presented by NMFS and CDFW are based on exceedance calculations of mean daily flows but can be modified to suit site-specific requirements. The exceedance flows statistically represent the flow equaled or exceeded during certain percentages of the time when migrating fish may be present or collected at a facility. The established guidelines are used to set instream flow depths, flow velocities, debris and bedload conditions, fish attraction requirements, tailwater fluctuations, and numerous other factors a facility might experience while target fish species are migrating.

NMFS (2011) requires the high flow fish passage design flow to be the mean daily streamflow that is exceeded 5 percent of the time during periods when target fish species are migrating. NMFS (2011) requires a low flow fish passage design flow equal to the mean daily streamflow that is exceeded 95 percent of the time during periods when migrating fish are typically present. The developed concepts in this report are for adult life stages only; fish passage design flows typically considered for adult upstream migration are provided in Table 3-4.

Mean daily flows for water years 2002 through 2020 from USACE Gage ID MRS on the Mormon Slough at Bellota were used to estimate flows at the Bellota Weir Project site. California Department of Fish and Game (CDFG) guidelines recommend that 3 cfs be selected for an alternative minimum design low flow for adult upstream migration if the low flow is calculated to be 0 cfs.

Table 3-4. Calculated Fish Passage Design Flows for each Target Fish Species, Period of Record 2002 to 2019

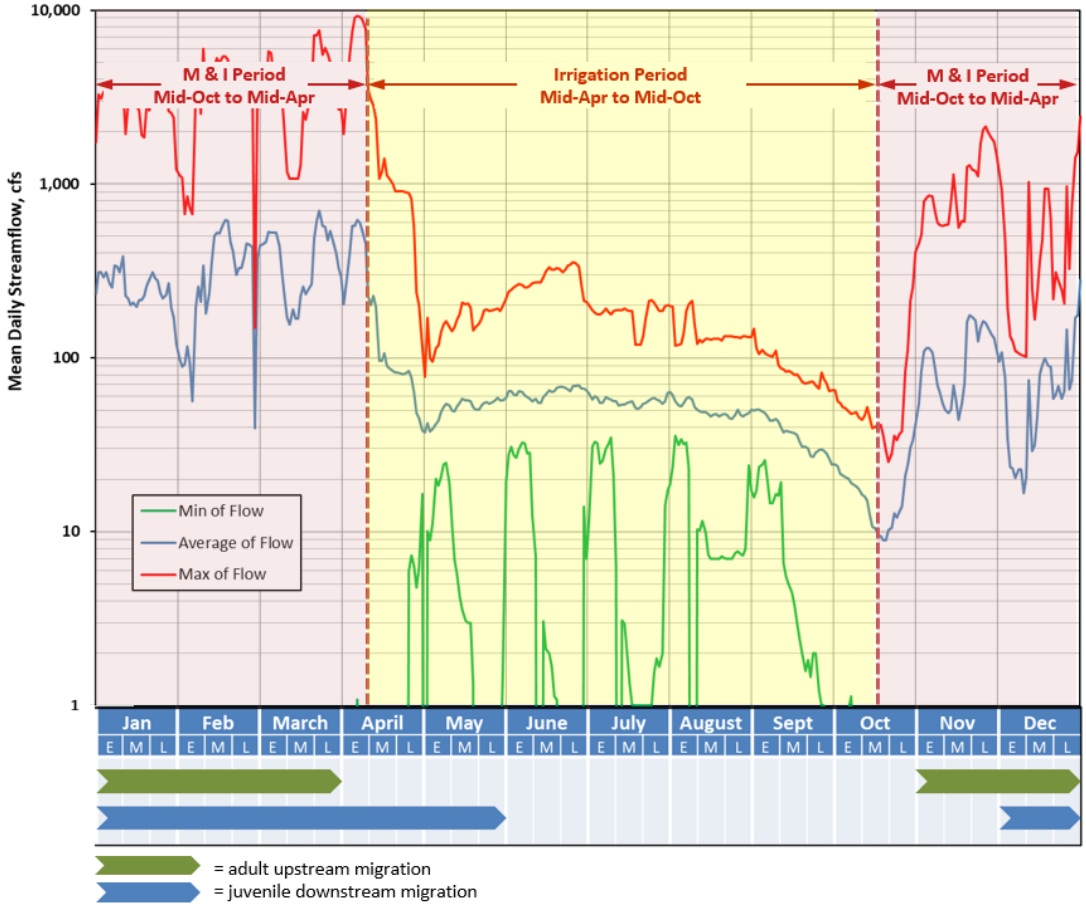
Species	Low Design Flow (cfs)	High Design Flow (cfs)
	NMFS Adults (95% exceedance)[1]	NMFS Adults (5% exceedance)
Likely to Use the Calaveras River		
Fall Chinook	3	507
Steelhead	3	1,286

[1] Calculated value is 0 cfs. Alternate low design flow selected per Table 4, CDFG 2002a and Table IX-5, CDFG 2004

An overlay of the fish migration period for the species likely to use the Calaveras River over mean daily stream flows shows adult fish are not migrating through the irrigation season (Figure 3-3). Fall Chinook Salmon adults migrate in November and December, and steelhead adults migrate November through March. Some juvenile outmigration for these species occurs as late as April and May. Figure 3-3 shows flow through the weir is minimal at this time, perhaps skewing the fish passage flow results for these species. During mid-October through mid-April, the main fish migration period, flow is only diverted for municipal and industrial purposes.

For the species unlikely to use the Calaveras River (winter-, spring-, and late-fall-run Chinook Salmon), the adult migration begins in November with late-fall-run Chinook Salmon, and ends in May with winter- and spring-run Chinook Salmon.

Figure 3-3. Overlay of Migration Periods, Seasonal River Flow Conditions, and Irrigation Period for Species Currently Using the Calaveras River



The range of flows is typically bounded by the lowest of the low fish passage design flows, through the highest of the high fish passage design flows. Using the results of this analysis, fish passage is to be provided through the proposed Project improvements from a low fish passage flow of 3 cfs to a high fish passage flow of 1,286 cfs (Table 3-5).

Table 3-5. Fish Passage Design Flows for Target Species Likely to Use the Calaveras River

Design Flow	Flow (cfs)
High (1% exceedance)	1,286
Low (95% exceedance)	3

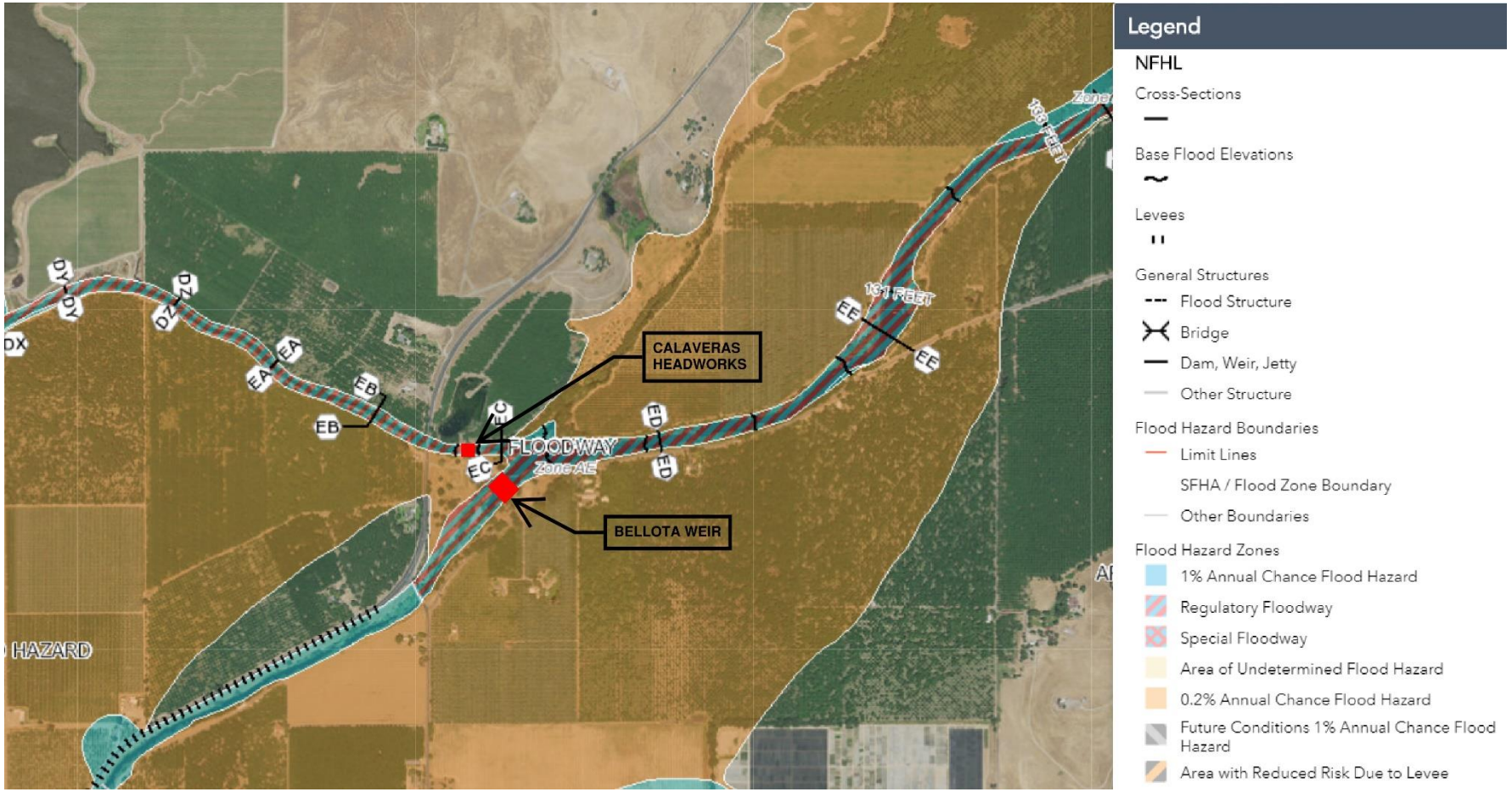
3.2.7 Floodway Regulatory Setting

The Calaveras River and Mormon Slough were subject to detailed study for FEMA flood mapping purposes in 1994 (FEMA 2002). A one-dimensional (1D) USACE HEC-RAS model was likely developed to support the original mapping effort, and would be classified as the Effective model. As a matter of general practice, the Effective model should be used as the basis for determining compliance with FEMA requirements and is

typically obtained from FEMA or the local community floodplain administrator. However, through extensive effort and communication with FEMA and representatives from local flood control authorities, the Effective modeling has not been located. The FEMA Engineering Library has no record of this model, nor does the community floodplain administrator, San Joaquin County.

The Project is currently within a regulatory floodway designated by the FEMA National Flood Hazard Layer and Flood Insurance Rate Map (FIRM) (FIRM Panel 06077C0370F effective 10/20/2016). A portion of the FIRM map from October 20, 2016 is included in Figure 3-4.

Figure 3-4. FIRM of Bellota Weir



Source: FEMA (2016)

For new construction to occur, the Project design team must demonstrate, through appropriate hydraulic analyses that the proposed encroachment will not result in any increase in base flood elevation for a 100-year event (1:100 chance of occurrence in a given year for the 100-year event). In the absence of the Effective model, FEMA guidance, as confirmed in correspondence with San Joaquin County (Shayan Rehman, Senior Engineer, personal communications, 2021), new existing conditions modeling will serve as the basis to evaluate a No-Rise condition, provided that the proposed flood profiles are equal to or less than the current Effective base flood elevations.

The FEMA Flood Insurance Study report details base flood elevations along the Calaveras River and Old Calaveras River (FEMA 2016). Using the published flood profiles, base flood elevations were determined at the confluence between the Old Calaveras River and Calaveras River/Mormon Slough, which is about 350 feet upstream of Bellota Weir and provides a good representative value until model inputs can be evaluated (Table 3-6).

Table 3-6. Base Flood Elevations from FEMA Hydraulic Model at Confluence of the Mormon Slough and Old Calaveras River

	Flood Event		
	500-Year	**100-Year**	**10-Year**
Base Flood Elevation (feet, NAVD88)	134.8	127.0	126.6

Source: FEMA (2016)

The 100-year base flood elevation at Bellota Weir is 127 ft. Effective flows were also calculated from the Flood Insurance Study report and recorded in Table 3-3.

The Project may require authorization from USACE under 33 United States Code Section 408. A Section 408 authorization is mandated for any use or potential alteration of a USACE project. The Mormon Slough system is a federally authorized flood control facility and falls under USACE jurisdiction. USACE's Operation & Maintenance manual for the Mormon Slough flood control system is unclear as to whether USACE's jurisdiction extends to the Bellota Weir, therefore, an early determination of a potential Section 408 nexus is recommended.

4 Agency-Specific Criteria

Several agency-specific criteria were used to inform the design of various components of the Project. These criteria are described and listed in the following sections.

4.1 Federal Emergency Management Agency

The proposed Project occurs in a regulatory floodway governed by FEMA and therefore must conform to FEMA Code of Federal Regulations (CFR) Title 44 Section 60.3 relative to potential impacts to the regulatory floodplain and floodway. While FEMA sets minimum standards, flood hazard permits and associated code are administered by the local jurisdiction, in this case, San Joaquin County. The Project's strategy is to achieve a no-rise condition and submit a No-Rise Certification to San Joaquin County for consideration. The potential for no-rise will be evaluated using the effective 100-year flow published by FEMA, which is 12,690 cfs.

4.2 California State Department of Transportation

The Project will require coordination with the California State Department of Transportation (Caltrans) relative to access off Highway 26 for the north bank construction at the site.

4.3 San Joaquin County Public Works

County roadways, repairs, and traffic control will comply with San Joaquin County Standard Plans and Specifications. Traffic control through construction work zones will comply with both the County work zone safety and the Federal Highway Administration (FHWA) Work Zone Management Program (https://www.fhwa.dot.gov/workzones).

4.4 Fish Screening

Fish screening facilities will be designed in accordance with the following guidance documents:

- Anadromous Salmonid Passage Facility Design. NMFS (2011).
- Fish Screening Criteria. CDFG (2000).
- Fish Protection at Water Diversions. Reclamation (2006).

Table 4-1 provides a summary of the fish screen criteria for designing a screening system that meets Project requirements.

Table 4-1. Fish Screen Criteria

Component	Applicable Code and/or Standard
Structure orientation	In a river, the screen will be oriented parallel to river flow. Upstream and downstream transitions must minimize eddies. The design must accommodate the entire range of forebay conditions (NMFS 2011).
Maximum diversion rate	The maximum diversion rate is 473 cfs.
Uniform approach velocity	The maximum approach velocity is 0.33 fps. Provisions will be provided to adjust flow patterns across the face of the screen to achieve flow uniformity (CDFG 2002b).
Screen size	The minimum effective screen area required is 1,433.3 square feet as determined by dividing the maximum screened flow by the allowable approach velocity (NMFS 2011). A factor of safety of 1.2 will be applied to accommodate unforeseen conditions that may inhibit uniform conditions across the screen face, resulting with an actual screen area of 1,720 square feet.
Minimum screen open area	The minimum screen open area will be at least 27% of the required screen area (NMFS 2011)
Sweeping velocity	The maximum sweeping velocity will be a minimum of 0.8 fps with the target sweeping velocities ranging from 1.0 to 2.0 fps.
Screen slot size	The maximum screen slot size will be 1.75 mm (NMFS 2011).
Screen materials	The screens will be constructed of Type 304L stainless steel.
Screen cleaning	Automated screen cleaning will be accomplished by mechanical cleaning using brushes on both the outside and inside of the screen surface. The cleaning system will provide complete screen cleaning within 5 minutes and be operated as required to prevent debris accumulation on the screen surface. The cleaning system will be automatically triggered if the head differential across the screen exceeds 0.1 foot beyond the clean condition or as agreed to by NMFS (2011). This head differential will be calibrated when the screen is completely clean and passing the maximum flow through the screen.

4.5 Fish Passage Design Criteria

Fish passage facilities will be designed in accordance with the engineering principles and guidance presented within numerous documents in additional to professional experience designing like facilities at other locations. This section identifies specific technical design criteria and references specific sources of design criteria and guidance relevant to the development of technical and nature-like fishway designs. Design guidance is provided in the form of criteria and guidelines. Criteria are specific standards for fish passage design that require an approved variance from the governing state or federal agency (i.e., NMFS or CDFW) to deviate from the established criteria. Guidelines provide a range of values or, in some instances, specific values the designer should seek to achieve but that can be adjusted without agency approval in light of project-specific conditions. Site-specific biological and physical rationale for deviating from an agency-established criterion is required. Values different from established guidelines should support better performance or solve site-specific issues. The reasons for using values

different from established guidelines may be requested by governing agencies during development of the design.

Note: the terms entrance and exit are used for notation of fish passage features in reference to the direction that a fish is travelling. Because fish ladders are typically designed for adult upstream passage, the fish ladder entrance refers to the downstream end or hydraulic outlet, and the fish ladder exit refers to the upstream end or hydraulic inlet. Because fish bypass features are typically designed for juvenile (and sometimes post-spawn adults) downstream passage, the bypass entrance refers to the upstream end or hydraulic inlet, and the bypass exit refers to the downstream end or hydraulic outlet. Terminology in the following criteria subsections follows standard terminology for fish ladders (upstream passage) and fish bypasses (downstream passage), depending on the type of passage facility is being addressed.

The following list of criteria is not intended to be an all-inclusive list of criteria documents for design but provides a sample of the more prominent guidelines that will be used to guide overall project development.

- Anadromous Salmonid Passage Facility Design. NMFS 2011.
- California Salmonid Stream Habitat Restoration Manual Park XII – Fish Passage Design and Implementation. CDFG 2009.
- Fisheries Handbook of Engineering Requirements and Biological Criteria. USACE (Milo Bell) 1991.
- Fishway Design Guidelines for Pacific Salmon. Bates 1992.
- New Designs for Vertical Slot Fishways. Canadian Journal of Civil Engineering, volume 19, pages 402-414. Rajaratnam, N., C. Katopodis, and S. Solank 1992.
- Design of Fishways and other Fish Facilities. Department of Fisheries, Canada. FS 31-1961/1. Clay, C.H. 1961.

4.5.1 Technical Fishway Criteria

Upstream fish passage designs at dams use widely recognized fishway design guidelines and references and are traditionally designed for the adult fish life stage. There are three major components to a fishway: the fishway entrance, fish ladder, and fishway exit. The fishway entrance's primary objective is to maximize fish attraction. The fish ladder's primary objective is to provide hydraulic conditions that promote fish passage up and around a passage barrier like a dam. The fishway exit's primary function is to maintain hydraulic conditions suitable for fish passage for the range of forebay or reservoir water surface elevations.

Fishway criteria were obtained from the Anadromous Salmonid Passage Facility Design document (NMFS 2011) and the California Stream Habitat Restoration Manual (CDFG 2009), and are summarized in Table 4-2 through Table 4-4.The design criteria specific to each component are presented below.

Table 4-2. Fishway Entrance Criteria

Component	Applicable code and/or standard
Location	Easily located by fish (NMFS 2011)
Opening Width	4 feet, minimum (NMFS 2011)
Depth	3 feet, minimum (site specific selection)
Head Differential, adults	1 – 1.5 feet (NMFS 2011)
Attraction Flow	5% – 10% of the maximum of the 5% exceedance flows for the migration period of each species (NMFS 2011 and CDFG 2009)
Auxiliary Water System Energy Dissipation Factor	16 ft-lbs/sec/ft^3 (NMFS 2011)
Auxiliary Water System Diffuser Velocity, vertical	1 fps, maximum (NMFS 2011)
Auxiliary Water System Diffuser Velocity, horizontal	0.5 ft/s, maximum (NMFS 2011)
Auxiliary Water System Diffuser Bar Spacing	1.75 mm, maximum (juvenile criteria; NMFS 2011)

Table 4-3. Fish Ladder Criteria

Component	Applicable code and/or standard
Fish ladder type	Vertical Slot with Pool and Weir Downstream Control
Head differential, adults	1.0 foot maximum feet (NMFS 2011)
Energy Dissipation Factor (pool and weir sections only if applicable)	4 ft-lbs/sec/ft^3 (adult criterion; NMFS 2011)
Turning Pool	Radius corners (NMFS 2011)
Pool Width	8 feet, minimum (NMFS 2011, Rajaratnam et al. 1992)
Pool Length	10 feet, minimum (NMFS 2011, Rajaratnam et al. 1992)
Pool Depth	3 feet, minimum (site specific selection)
Freeboard	3 feet, minimum (NMFS 2011)
Vertical Slot Width	12 inches
Maximum Slot Velocity	6 fps (Bell 1991)
Surface Irregularities	No finished surface and projections inside the fishway can have a sharp edge, points, or projections that can injure fish (NMFS 2011)
Concrete Work	All submerged concrete surfaces must have a smooth finish and corners shall possess a 25 mm, 45-degree chamfer (NMFS 2011)
Protrusions	Protrusions (i.e., valve stems, bolts, gate operators, pipe flanges) must not extend into the flow path of the fishway (NMFS 2011)

Table 4-3. Fish Ladder Criteria

Component	Applicable code and/or standard
Natural Light Penetration	The top surface of the fishway must be a grated surface to enable natural light penetration. Ambient lighting is preferred throughout the fishway, and in all cases abrupt lighting changes must be avoided (NMFS 2011)

Table 4-4. Fish Exit Criteria

Component	Applicable Code and/or Standard
Length	Minimum 2x the calculated hydraulic fish ladder pool length Fishway exit pool length is measured from the downstream end of the trashrack to the most upstream baffle.
Location	Along shoreline

4.5.2 Pool, Baffle, and Slot Criteria for Vertical Slot Fishways

Pool, baffle, and slot dimensions will be sized based on relationships presented by Rajaratnam et al. (1992), originally generated from laboratory testing and analysis. Pool dimensions will be designed to accommodate total conveyance, limit turbulence, and provide adequate holding areas for fish as they ascend each pool. For a slot width of b_0, vertical slot fishways operate satisfactorily when the length and width of the pool is at a proportion of $10b_0$ and $8b_0$, respectively. Figure 4-1 and Figure 4-1 illustrate the optimal dimension relationships of pools, baffles, and slots within a vertical slot ladder.

Figure 4-1. Vertical Slot Fishway Pool Design

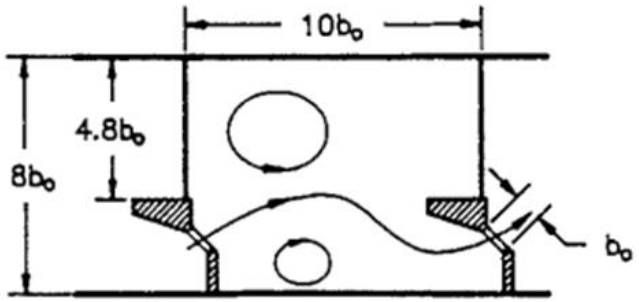
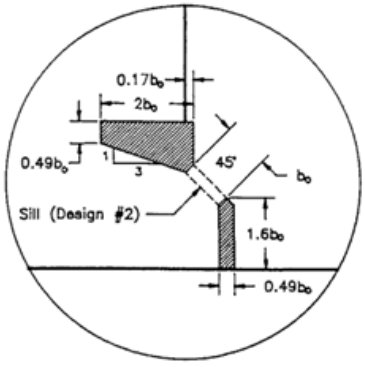


Figure 4-2. Vertical Slot Fishway Baffle Design



4.6 Debris Rack Criteria as Applied to Fishway Exits

For any coarse debris racks occurring within fish passage navigational corridors, the clear space openings should be a minimum of 12 inches given that adult Chinook

Salmon are present. NMFS criteria state that approach velocity should be uniformly distributed to the extent possible with magnitudes not exceeding 1.5 fps. Debris racks should be sloped at 1H:5V or flatter to assist with manual cleaning. Slopes may vary dependent on the type of automated cleaning system incorporated into design and may be subject to manufacturer requirements. In systems with coarse floating debris, debris booms or other provisions must be incorporated into the debris rack design (NMFS 2011).

5 Technical Design Criteria

The following sections present the technical design criteria for the Project. For this document, technical design criteria refer to the hydraulic, geotechnical, civil, mechanical, structural, and electrical criteria that govern the overall Project design.

5.1 Hydraulic Design Criteria

The following sections provide a brief discussion of the Project's hydraulic design criteria along with a short description of the anticipated modeling strategy.

5.1.1 Modeling Strategy

A series of 1D and 2D models will be used to inform the design process, confirm conformance with physical model testing, and confirm design compliance with regulatory requirements. Specifically, the objectives of the combined modeling efforts are to:

- Confirm that the range of hydraulic conditions anticipated after Project implementation will meet the fish passage, sediment passage, and water supply and flood resilience/stability criteria established during preliminary and final design
- Refine sediment continuity and particle mobility thresholds to inform sediment transport and sluicing targets
- Provide hydraulic conditions such as shear stress to verify the following:
    - Sizing and stability of channels
    - Boulder matrices of the roughened channel
    - Supporting rock filters for boulder structure and low flow channel
    - Potential for scour holes downstream of structure and forces on existing structures

Table 5-1 shows the specific modeling to be performed at each design phase, with targeted objectives identified for each model at each phase.

Table 5-1. Hydrologic and Hydraulic Modeling Tasks

Design Phase	Model Type	Objectives
30% Design	1D HEC-RAS	1. Refine the existing 1D HEC-RAS model to examine reach-based hydraulics 2. Initial assessment of Project compliance with FEMA and San Joaquin County Flood Control Zone District regulations a. FEMA Title 44 CFR 60.3: zero rise of the calculated 100-year water surface profile

Table 5-1. Hydrologic and Hydraulic Modeling Tasks

Design Phase	Model Type	Objectives
	2D HEC-RAS	1. Inform development of the existing 1D HEC-RAS model. 2. Inform the design process and confirm that performance and hydraulic design criteria are being met relative to: a. Scour assessment and scour countermeasure design b. Stability of roughened channel elements c. Potential impact to adjacent structures
60%–90% Design	1D HEC-RAS	1. Final confirmation that the Project complies with FEMA 44 CFR 60.3 and provides justification for No-Rise Certification
	2D HEC-RAS	1. Inform the design process and confirm that performance and hydraulic design criteria are being met relative to: a. Fish passage b. Sediment continuity/mobility c. Potential impact to adjacent structures 2. Further evaluate and confirm anticipated hydrodynamic conditions in and around proposed Project elements (velocity, depth, shear, etc.)

5.1.2 Hydraulic Structures

The fishway and dam with weir crest gates will be designed to conform to several performance and engineering criteria. Engineering criteria will target the overall stability and resilience of the structure. Performance criteria will target the biological performance and periodic costs associated with maximum acceptable damages that require modification or repairs over time. The various criteria and performance targets used to establish the overall design of this feature are summarized in the following paragraphs.

Stability and Longevity

- The structures (weir, channels, and high flow sections) will be designed to withstand flood events up to the 100-year flood event (12,690 cfs).
- Limited damage will not impede the structures ability to achieve fish passage or surface water withdrawal performance criteria.
- Erosion created by this structure will not compromise adjacent infrastructures.
- Material or debris deposited onto the channel spanning weir naturally by the river (cobbles, gravels, trees, wood) will not affect the structures ability to meet stability and longevity criteria.

Biological Performance Criteria

- The structure configuration and composition will be designed to meet the Project's fish passage performance objectives.

- Fish passage performance objectives will be continuous and met throughout the range of flows identified to allow reasonable usage of second low flow channel for left floodplain habitat continuity during a normal year and inspections/repairs of low flow channel.
- Fall back, false attraction, delays, or non-passage periods will be minimized to the extent practicable given the uncertainty inherent with structures of this nature.

Design References

Design of the fishway and dam will use and comply with the following technical guidelines:

- USACE Engineer Manual (EM) 1110-2-1601, Hydraulic Design of Flood Control Channels, 1994.
- USACE EM 1110-2-1418, Channel Stability Assessment for Flood Control Projects, 1994.
- California Salmonid Stream Habitat Restoration Manual Park XII – Fish Passage Design and Implementation. CDFG 2009.

5.1.3 Roughened-Channel Fishway

The roughened channel fishway and channel spanning weir crest will be designed to conform to several performance and engineering criteria. Engineering criteria will target the overall stability and resilience of the structure. Performance criteria will target the biological performance and periodic costs associated with maximum acceptable damages that require modification or repairs over time. The various criteria and performance targets used to establish the overall design of this feature are summarized in the following sections.

Targets for Stability and Longevity

- The structures (weir, channels, and high flow sections) will be designed to withstand flood events up to the 100-year flood event (12,690 cfs).
- Limited damage that may occur during low frequency events will not impede the structures ability to achieve fish passage or surface water withdrawal performance criteria.
- Damage will be associated with surface scour of rock matrix, including fine materials creating volume loss, and minor degradation of tailwater control (i.e., facility will be designed such that periodic adjustments in surface composition do not inhibit irrigation supply, fish passage, or negatively affect the Esacalon Bellota Road bridge).
- Plucking of large structural rocks will be limited to small areas where storm events create conditions that cause isolated areas of instability.
- There will be no damage to the seepage wall or the structural matrix just downstream of the weir at the structure terminus that composes the base of the overall structure.

- The 5-year average maintenance and repair cost of the roughened channel will be targeted to remain within $25,000 ($U.S. 2019).
- Erosion created by this structure will not compromise adjacent infrastructure.
- Material or debris deposited onto the roughened-channel fishway and/or channel spanning weir naturally by the river (cobbles, gravels, trees, wood) will not affect the structures ability to meet stability and longevity criteria.
- Accumulation and removal of finer sediments and cobbles on the surface layer above the armor layer will be transitional and vary over time.

Ecohydraulic Performance Criteria

- The structure configuration and composition will be designed to meet the Project's fish passage performance objectives.
- Fish passage performance objectives of the roughened channel will be continuous and met throughout the range of anticipated migration flows identified for various target species and life stages.
- Fall back, false attraction, delays, or non-passage periods will be minimized to the extent practicable given the uncertainty inherent with structures of this nature.

Design References

Design of the roughened-channel fishway will use and comply with the following technical guidelines:

- Reclamation Rock Ramp Design Guidelines, 2007.
- USACE Engineer Manual (EM) 1110-2-1601, Hydraulic Design of Flood Control Channels, 1994.
- USACE EM 1110-2-1418, Channel Stability Assessment for Flood Control Projects, 1994.
- California Salmonid Stream Habitat Restoration Manual Park XII – Fish Passage Design and Implementation. CDFG 2009.

5.1.4 Scour

Contraction, bend, and local scour will be evaluated at appropriate locations, along with risk of bank erosion and long-term degradation.

Scour and general incipient motion of particles, including fines, will be evaluated by the following methods to establish design issues:

- U.S. Department of Agriculture Technical Supplement 14B, Scour Calculations, 2007.
- FHWA Hydraulic Engineering Circular 18, Evaluating Scour at Bridges, 2012.

5.1.5 Scour and Structure Settlement Countermeasure Design

Scour countermeasures in the channel and on the structure including attention to structure design matrices to prevent winnowing and settlement will be implemented where the analysis indicates that the Project will increase risk, complying with the following technical guidelines:

- USACE EM 1110-2-1601, Hydraulic Design of Flood Control Channels, 1994.
- FHWA Hydraulic Engineering Circular 23, Bridge Scour and Stream Instability Countermeasures, 2009.
- Reclamation Rock Ramp Design Guidelines, 2007.
- Reclamation Design of Small Dams, 1987.
- Chapter 7: Sediment Transport Technology, Simons and Senturk, 1977.
- FEMA Filters for Embankment Dams – Best Practices for Design and Construction, 2011.

5.2 Geotechnical Criteria

The following sections provide a brief discussion of the Project's geotechnical design criteria, along with applicable codes, standards, and design issues to be considered.

5.2.1 General Design Description

Components of the Project that use the geotechnical design criteria described in this section include but are not limited to the following:

- The channel-spanning roughened channel
- The intake and technical fishway
- The dam with weir crest gates
- Sheet pile cutoff wall on the upstream face of the weir to control seepage through the roughened channel structure
- Temporary retaining structures
- Dewatering

5.2.2 Applicable Codes and Standards

Geotechnical design components will adhere to the following references and design standards:

- American Society of Civil Engineers (ASCE) 7-10 Minimum Design Loads for Buildings and Other Structures, 2009 and USGS Earthquake Hazards Program.
- ASCE 7-16 Chapter 15 Seismic design of Nonbuilding structures and Nonstructural Components, 2017.
- California Building Code 2019.

- Standard Penetration Test (SPT)-based probabilistic and deterministic assessment of seismic soil liquefaction potential. Cetin et al, 2018.
- Ecology Dam Safety Guidelines, Part 4, Dam Design and Construction, July 1993.
- Filters for Embankment Designs – Best Practices for Design and Construction. FEMA 2011.
- Design and Construction of Pile Foundation FWHA GEC 12 July 2016.
- International Building Code 2018.
- Reclamation Design Standard 13, Embankment Dams.
- Boulanger and Idriss (2014 CPT and SPT based Liquefaction triggering Procedure).
- United Research Services Corporation 2012 DWR Urban Levee Evaluation Relationships.
- USACE EM 1110-2-2504 Design of Sheet Pile Walls, 1994.

5.2.3 Specific Criteria

Specific geotechnical criteria and codes/standards are presented in Table 5-2.

Table 5-2. Geotechnical Design Criteria

Parameter	Applicable Code and/or Standard
Geotechnical test methods	SPT, in accordance with ASTM D-1586.
	Laboratory Testing per ASTM D Particle Size Analysis (Gradation, percent passing #200 Sieve), Moisture/Density, Triaxial Strength,
Materials	Soil Classification, in accordance with ASTM D-2487, Standard Practice for Classification of Soils for Engineering Purposes (Unified Soil Classification System).
Permeability	Permeability of the materials will be determined based on the gradations determined by the laboratory testing and using the guidance in Reclamation Design Standard 13, Embankment Dams, Chapter 8, Seepage and DWR Urban Levee Evaluation material property relationships.
Seepage analysis	Seepage will be evaluated based on the requirements of the structure and in accordance with Reclamation Design Standard 13, Embankment Dams, Chapter 8, Seepage. Analysis will be performed using the seepage software program Seep/W, in the GeoStudio suite of programs.
Liquefaction triggering analysis	Boulanger and Idriss 2014 CPT and SPT Liquefaction Triggering Analysis. SPT-based probabilistic and deterministic assessment of seismic soil liquefaction triggering hazard (Cetin et al 2018).
Slope stability	Slope stability of the roughened channel will be evaluated based on the requirements of the structure and in accordance with Reclamation Design Standard 13, Embankment Dams, Chapter 4, Static Stability Analysis, and Chapter 13, Seismic Analysis & Design. Analysis will be performed using the slope stability software program GeoSlope/W, in the GeoStudio suite of programs.

Table 5-2. Geotechnical Design Criteria

Parameter	Applicable Code and/or Standard
Filters	The roughened-channel filter design will be in accordance with Reclamation Design Standard 13, Embankment Dams, Chapter 5, Protective Filters November 2011 and Filters for Embankment Designs – Best Practices for Design and Construction (FEMA 2011)
Seismic loading	Seismic analysis will be in general accordance with Reclamation Design Standard 13, Embankment Dams, Chapter 13, Seismic Analysis & Design, with input from ASCE 7-10 and USGS Earthquake Hazards Program. Seismic design acceleration factors will be in accordance with 2019 California Building Code and ASCE 7-10 addendum.
Sheetpile Cutoff	USACE EM 1110-2-2504; Design of Sheet Pile Walls
Pile Foundations	FHWA Design and Construction of Pile Foundation FWHA GEC 12 July 2016 and Ensoft Suite of analysis programs as appropriate

ASTM = American Society of Testing and Materials International

5.3 Civil Criteria

Civil criteria and applicable codes, standards, and design issues used for final design are included herein.

5.3.1 General Design Description

In general, the civil design aspects of the Project include the following work elements:

- Survey control
- Site safety
- Traffic control
- Road maintenance
- Site civil grading
- Stormwater and drainage
- Erosion and sediment control
- Earthwork
- Working within a floodplain regulated by FEMA

These features will be designed to meet the requirements of SEWD, San Joaquin County, and the State of California including the Environmental Protection Agency.

5.3.2 Applicable Codes and Standards

Civil design components will adhere to the following references and design standards:

- Americans with Disabilities Act design standards in ANSI 117.1-2017, 2017.
- American Water Works Association (AWWA)

- USACE EM 1110-2-1601 Hydraulic Design of Flood Control Channels, 1994.
- California Salmonid Stream Habitat Restoration Manual, 2009.
- California Bank and Shore Rock Slope Protection Design, 2000.
- California Department of Transportation (Caltrans) Highway Design Manual, 2018.
- California Division of Occupational Safety and Health Administration Standards.
- California Environmental Protection Agency.
- California State Water Resources Control Board.
- California State Air Resources Board.
- American Association of State Highway and Transportation Officials.
- ASTM.
- U.S. General Services Administration.
- Occupational Safety and Health Administrations (OSHA).
- FEMA 44 CFR 60.3 (see Section 4.1).
- San Joaquin County Improvement Standards, 2014
- SEWD Standard Specifications
- SEWD Standard Details

5.3.3 Project Survey Control and Datum

Horizontal datum: North American Datum of 1983 (2007.00 epoch), converted to the California Coordinate System of 1983, Zone 3 (CCS83-Z3), Survey Feet Units.

Vertical datum: NAVD88

The contractor will verify the existing Project survey control, to be shown on the Project contract documents, prior to constructing the Project. The contractor is responsible for establishing any supplemental survey control required to build the Project and protect existing Project survey control shown on the contract documents. Supplemental survey control will be placed on site or adjacent to the Project site after review by SEWD. At least two fixed locations for daily progress construction photographs will be established at the direction of SEWD.

5.3.4 Safety

The design will incorporate both site safety and public safety into the design elements because of the size and complexity of the Project. A preliminary site safety plan will be prepared following both federal and state OSHA guidelines as well as county-specific codes.

5.3.5 Security

To ensure public safety, the entire site will be fenced on both sides of the river. Signs warning of construction area will also be located both upstream and downstream of Project boundary.

- 7-foot-high chain-link security fence around diversion structure.
- Locked devices on valve and gate operators.
- Construction Work Area – Keep Out signage, located both upstream and downstream of work area.

5.3.6 Traffic Control

Because of the anticipated heavy truck traffic during construction, the design will incorporate traffic control and staging areas. It is anticipated that traffic control will be coordinated with adjacent landowners, San Joaquin County, and state agencies; any permits necessary for implementation of traffic control will be acquired by the contractor.

5.3.7 Air Quality

Because of the high truck traffic and on-site equipment, carbon emissions will need to be estimated for air quality for the supplemental environmental permitting process for the Project, to comply with the requirements of the California Air Resources Board. Air quality monitoring stations will be set up at recommended locations within the work area, including one likely upstream of the site.

5.3.8 Road Maintenance

With high truck traffic comes excessive wear on public roads. Pre-construction condition assessments of both State Highway 26 and Shelton Road will be performed. Limits of the assessment will be 500 feet on either of the access roads to the Project or any other locations where roads may be affected. Road repairs will be performed in compliance with county-applicable and state criteria.

5.3.9 Site Restoration

The design of the finished site condition will strive to protect the natural and historic resources of the area. As much as possible, areas disturbed during construction, including the Project site, and temporary construction staging areas and access routes, will be restored to pre-construction condition. Specific requirements for site restoration will be developed in the supplemental environmental permitting process for the Project. Specific criteria may include:

- Utilize existing access roads to the greatest extent possible.
- No landscaping will be proposed other than revegetation of disturbed areas with appropriate native species.
- Avoid and/or minimize disturbance of large stands of mature, healthy trees.
- Minimize the alteration of natural topography and vegetation.

5.3.10 Erosion and Sediment Control

During construction activities, it will be necessary to control disturbed soils from eroding into any waterways and air born sediment (dust) from leaving the work areas.

Sediment Control

Sediment control will be required during the entire construction period. Sediment control will be accomplished using a combination of best management practices and sediment control devices, which will be placed around any areas with disturbed, stockpiled or exposed soil. A Stormwater Pollution Prevention Plan prepared by the Contractor will be required to address the sediment control plan in detail and may include best management practices such as:

- Earthen berms surrounding staging or material storage areas.
- Sediment control barriers (i.e., silt fences, fiber rolls, and/or staggered rows of proprietary solutions such as Terra Tubes) on slopes to prevent runoff or dewatered material from entering Mormon Slough or Old Calaveras River.
- Trucking control devices (i.e., stabilized construction entrance and entrance/outlet tire wash area) to reduce tracking of sediment onto public or private roads by construction vehicles.
- Identified concrete truck wash down area and containment controls.

Stockpile Control

Stockpiling will be performed according to the following design criteria:

- Locate stockpile at a practical distance from the work area that also minimizes haul distances.
- Locate stockpile on relatively flat areas as identified from the topographic mapping.
- Limit stockpile heights to 6 to 8 feet with no side slope greater than 2H:1V.
- Cover stockpiles during periods of inactivity.
- Ensure that areas slope away from or do not drain directly into the Mormon Slough or Old Calaveras River.

5.3.11 Conveyance and General Piping design

The primary conveyance installation for this Project consists of large diameter or box conduits conveying water from the new intake under the river to the District's main water supply pipeline and the Old Calaveras River. Other general piping will be associated with mechanical systems necessary for the Project. This section presents conveyance system design criteria.

The following items were considered in the development of the conduit design criteria:

1. Excavation
2. Bedding

3. Backfill
4. Conduit materials
5. Operating pressure
6. External loading and other design considerations
7. Flotation/depth of cover
8. Pipe pressure classes

The conduit design criteria within each of these categories is defined below.

Excavation

Excavation must be conducted in accordance with the requirements established by the OSHA. Shoring may be required at certain locations due to deep excavation and space constraints. Shoring design and trench safety will be the responsibility of the contractor. The selection of a protective system (shoring), and/or angle of excavated slopes will be determined after considering applicable local, state and federal (OSHA) safety standards and regulations and the geotechnical engineer's recommendation.

Bedding

Pipe-bedding and pipe-zone materials consist of materials surrounding the conduits in the trench zone from typically 6 inches below to 12 inches above the pipe. This material must be compacted so that pipe walls are well supported. The soil structure created in this zone is intended to keep the pipe wall flexure within specified limits.

For piping, the maximum deflection, as a percentage of the pipe diameter, will be determined by the pipe material selected. The allowable deflection will be based on the pipe inside diameter.

Pipe zone material is often imported as an approved classified granular material. To protect the exterior pipe coating, imported pipe zone material is typically specified to be sand or a mixture of small aggregate and sand particles. Pipe zone backfill for flexible coated pipe (tape-coated steel pipe or polyolefin-coated) will consist of clean sand, 100 percent passing 3/8 mesh screen and at least 90 percent passing or No. 4 sieve, with a minimum sand equivalent of 30 percent unless otherwise required or recommended by the manufacturer to maintain warranty.

Backfill

Trench and structure backfill will be compacted to a minimum of 95 percent maximum density (ASTM D698 Standard Proctor). If recommended by the geotechnical engineer, native excavated materials may be used for backfill above conduits/pipe zone provided they are free of organics and other unsuitable material and screened to remove particles greater than 2 inches in diameter. Pipe backfill under the river will vary from buried pipe trench backfill outside of the river, and larger river rock may be determined suitable.

Conduit Material Selection

Appropriate conduit material alternatives for the various Project sections and appurtenances include: Concrete box culverts, high-density polyethylene (HDPE), polyvinyl chloride (PVC), and steel pipe. Actual pipe materials and pipe strengths will be determined during the design process. However, the following sections briefly define expected field conditions.

Operating Pressure

Because the piping for this project will be under minimal internal pressure (approximately 15 feet of head) the minimum wall thickness will be dictated by the handling, flotation, and external loading requirements rather than the internal pressure.

The maximum internal pressure will occur during field hydrostatic testing of the pipe. The hydrostatic test pressure will be equal to the maximum surge pressure, or 1.5 times the normal operating pressure, whichever is greater.

External Loading and other design considerations

Concrete conduits will be designed for HS20 loading and a maximum burial depth of 25 ft. Pipes will be designed to resist buckling due to external pressure, arising from external loads, in accordance with AWWA M11 for steel pipe, M23 for PVC pipe, and M55 for HDPE pipe. The external loading will include both live and dead loads (including wheel, soil, and hydrostatic loading) and deflection will be limited based on the specific pipe materials.

Flexible type couplings will be used to account for pipe deflections or movement due to differential settling and piping will be designed to prevent potential pipe buckling or collapsing.

Floatation/Depth of Cover

All buried conduits, pipes, and structures will be designed to resist flotation when dewatered, under the most extreme flood conditions, and under earthquake conditions. The water table will be assumed to be at the ground surface or the 500-year flood level, whichever is higher for pipe buried in trenches that are normally above the Bellota forebay level. For piping installed under the Bellota forebay, the design water level will be the 500-year flood event.

The factors of safety to resist uplift will be:

- 1.1, if only the pipe or structure weight is included in uplift resistance, or
- 1.5, if contribution of soil weight to uplift resistance is included.

5.3.12 Civil Materials

Civil materials to be used are summarized in Table 5-3.

Table 5-3. Civil Materials

Material	Applicable code and/or standard
Local and Imported borrow	Caltrans Standard Specifications Section 19-7
Pipe bedding	Caltrans Standard Specifications Section 19-3.02F
Structure backfill	Caltrans Standard Specifications Section 19-3.02C
Streambed material	To be specifically designed by river engineers
Streambed cobbles	Caltrans Standard Specifications Section 72-2.02B
Streambed boulders	Caltrans Standard Specifications Section 72-2.02B
Habitat boulders	Caltrans Standard Specifications Section 72-2.02B

5.4 Mechanical Criteria: Intake

This section presents mechanical design criteria for the intake of the facility, including a general design description, applicable codes and standards, and specific criteria.

5.4.1 General Design Description

Components of the Project that use the mechanical design criteria described in this section include but are not limited to the following:

- Fish protection screens

  Five tee screens will be provided to screen water at the intake. Two will be smaller for when lower flows are sufficient to meet system demands. The tee screens will have a local structure supporting them with the ability to lift the tee screen out of the water for maintenance or other purposes. When in the lifted position, a stop gate integrated with the frame will lower to shut off flow through the opening.

- Slide-type water control gates

  Twelve slide-type water control gates will be provided on the site. Five will be dedicated to shutting off the intake screen penetrations; two to the SEWD water supply culverts; two to the Calaveras River supply culverts; one to the sluicing of material in the screened intake wet well; one to the fishway's auxiliary water supply; and one to shutting of the sluiceway. All the slide type water control gates will have electric actuators and follow the control scheme outlined in Table 5-5.

- Weir gate system

  At the dam, there will be three separate adjustable weir gates that will control the forebay water surface elevation. Each of these weir gates will be actuated with air bellows under the gate leaf on the downstream side. Compressed air will be supplied by means of a compressor and receiver tank located nearby. Inputs to the system will be level transmitters upstream and downstream of the gate.

- Flow meters

  The amount of flow being diverted to the SEWD water supply system, as well as to the Calaveras River, will be measured using flow meters.

- Debris cleaning system

  A debris cleaning system will be added at the intake/fish ladder exit to prevent any blockages in water flow to the system. The system will be on a timer system to keep the system clear unless site personnel override the logic and either run an intermediate cleaning cycle or shut down for maintenance. This will follow the control scheme outlined in Table 5-5.

5.4.2 Applicable Codes and Standards

Mechanical design components will adhere to the following references and design standards:

- Anadromous Salmonid Passage Facility Design, NMFS, 2011.
- ASTM.
- American National Standards Institute (ANSI)/Hydraulic Institute 9.8-2012, American National Standard for Pump Intake Design.
- American Society of Mechanical Engineers (ASME).
- American Welding Society (AWS) Structural Welding Code.
- National Fluid Power Association International Organization for Standardization 4406:2017: Hydraulic fluid power – Fluids – Method for coding the level of contamination by solid particles.
- AWWA Standards.
- Compressed Air and Gas Association (CAGI).
- National Electrical Manufacturers Association (NEMA).
- Underwriters Laboratories.

5.4.3 Specific Criteria

Mechanical materials to be used are summarized in Table 5-4.

Table 5-4. Mechanical Materials

Material/Component	Applicable Code and/or Standard
Fish Screens	
Carbon steel bars, plates, and shapes	ASTM A36
Stainless steel plates	ASTM A167 or A240, Type 304L
Stainless steel bars and shapes	ASTM A276, Type 304L

Table 5-4. Mechanical Materials

Material/Component	Applicable Code and/or Standard
Bolts and nuts	ASTM A193, B8; ASTM A194
Stainless steel pipe	ASTM A500, Grade B
Welding electrodes	E70XX or Type 308-15, as required for base metal
Screening materials	Type 304L stainless steel; profile bar
Water Control Gates	
Gate construction	AWWA C560
Allowable leakage	Maximum allowable leakage rates will not exceed 0.1 gallon per minute per foot of seat perimeter with water at top of gate slide and operating seat position.
Gate seals	Gate seals will use J-seals and resilient seats of synthetic rubber conforming to AWWA C560.
Allowable deflection	Slide cover will have a maximum allowable deflection: 1/360 of the span of the gate under maximum head condition.
Gate yoke design	The pocket and yoke of gate will be capable of taking at least twice the rated thrust output of the operator at 40-pound (lb) pull. Slide material will be the same as the frame and guides.
Gate stem	Stem diameter capable of withstanding twice the rated output of the operator at 40-lb pull, and support such that L/r ratio for unsupported part of the stem will not exceed 200.
Obermeyer Water Control Gate	
Steel panels, splitters, bars, and restrain strap clamps	ASTM A572 Grade 50 steel plate.
Bolts and nuts	ASTM A193, B8; ASTM A194.
Air bladder material	The material for the air bladder will be reinforced with not less than four layers of 1,500/2/3 polyester bias ply fabric with a minimum tensile strength of 2,320 lb/linear inch.
Outer rubber cover	The outer rubber cover will meet or exceed ASTM D2000.
Minimum material thickness	The air bladder material will have a minimum thickness of 0.4375 inch as measured in accordance with ASTM D751.
Bladder abrasion resistance	An abrasion test using an H18 stone with a 2.2 lb load for 1,000 repetitions as modified from ASTM C501 will show less than 0.03 cubic inches of abrasion.
Control panel	Control panels will be built under the provisions of Underwriters Laboratories 508 and 508A.
Control panel enclosures	Heavy-duty NEMA 4/13 watertight/oil-tight. NEMA 250, Enclosures for Electrical Equipment (1,000 volts maximum).
Compressed Air System: Obermeyer	
Stainless steel pipe	Stainless steel pipe will be Schedule 40 wall thickness available for the size noted on the drawings. Pipe shall be Type 304 or 316 stainless steel and conform to ASTM A312 or ASTM A376.

Table 5-4. Mechanical Materials

Material/Component	Applicable Code and/or Standard
Copper pipe	Buried copper piping will conform to ASTM B88, Type K; above-grade copper piping shall conform to ASTM B88, Type L.
Copper pipe fittings	Fittings for copper pipe will be solder joint fittings conforming to ANSI B16.22 or ANSI B16.18.
Flanges, threads, and unions	Flanges on equipment and appurtenances will conform in dimensions and drilling to ANSI/ASME B16.1, Class 150. All pipe threads will conform in dimension and limits of size to ANSI/ASME B1.20.1, Taper Pipe Thread.
Welded fittings	Fittings will be carbon steel to meet criteria of ANSI/ASME B16.11 and ASTM A105. Fittings will be of the size to match Schedule 40 or Schedule 80 black steel pipe. Strength shall be equal to 2,000 pounds per square inch (psi) minimum. Connections will be butt-weld, threaded, or socket weld type.
Steel pipe	ANSI/AWWA M11.
Air receivers	ASME Boiler and Pressure Vessel Code Section VIII: Rules for Construction of Pressure Vessels, 1995.
Ball valves	Valves will be rated for 275 psi minimum, for air service at temperatures ranging from - 20°F to 400°F.
Safety relief valves	Safety relief valves will be designed for air service and sized as shown on the drawings. Valves will be rated for a maximum inlet pressure of 300 psi at 300°Fahrenheit (°F).
Pressure-regulating valve	Pressure-regulating valves will be rated for operation with air and a maximum operating pressure of 200 psi or greater, with an adjustment range of 5 to 100 psi.

The mechanical equipment on site will have one or more of the following control schemes:

- **Supervisory Control and Data Acquisition (SCADA) Remote** control is where the operator can control the equipment from anywhere by means of radio or satellite and from the control room. With this, the operator can override the command logic controlling the equipment or restore remote automatic control for the equipment.
- **Remote Automatic** control is where the system is controlled means of command logic. That logic can be controlled by input such as water levels, or pressure differentials.
- **Remote Manual** control is where the operator can control the equipment from the control room and does not have to be next to the equipment. The type of control allows the operator to override any other inputs or command logic that controls the equipment.
- **Local Manual** control consists of one of two scenarios: 1) The electric actuator is switched into local mode and an operator must physically push a button to operate the equipment, and 2) The equipment is actuated by manual means such as a handwheel or t-stem.

Table 5-5 shows the type of control anticipated for the mechanical equipment at the Project site.

Table 5-5. Mechanical Equipment and Associated Control Schemes

Equipment	Physical Location	Control			
		SCADA Remote	Remote Automatic	Remote Manual	Local Manual
Intake Screens	Intake	x	x	x	x
Isolation Shutoff Gate for Intake Screens (5x)	Intake				x
Slide Gates for SEWD (2x)	Wet well at intake	x	x	x	x
Slide Gates for Calaveras River (2x)	Wet well at intake	x	x	x	x
Auxiliary Water Supply Shut Off Gate	Intake	x	x	x	x
Sluice Gate for intake	Wet well at intake	x	x	x	x
Sluice Gate for Sluiceway	Dam			x	x
Weir Gates	Dam	x	x	x	x
Debris Cleaning System	Intake/Fish Ladder Exit	x	x	x	x

5.5 Structural Criteria

This section includes the structural design features, concepts, and criteria for the facility including applicable codes, standards, and regulations; engineering properties of construction materials; design of proposed structures; and analysis.

All design and analysis, regardless of material or structural element, would utilize the Strength Design method, or Load and Resistance Factor Design method, unless otherwise noted. Structural calculations may be completed by hand or with the aid of a finite element analysis software such as SAP 2000 structural software. Other software may also be used such as: Excel, and/or Mathcad.

The overturning, sliding and flotation stability of hydraulic gravity structures such as spillway and walls will be checked according to the relevant codes and guidelines mentioned in this section.

5.5.1 General Design Description

Components of the Project that use the structural design criteria described in this section include but are not limited to the following:

- Concrete dam and spillway
- Vertical lift weir steel gate
- Concrete foundations and walls for the intake structure
- Pile foundation
- Gantry and overhead lifting support structures and gate hoist platform

- Building foundations
- Miscellaneous metal fabrications including grating/walkway support, embeds, ladders, stairs, structural supports, guardrails, and trashracks

5.5.2 Applicable Codes and Standards

Structural design components will adhere to the following references and design standards:

- Aluminum Association (AA) Aluminum Design Manual, 2020
- ANSI/American Institute of Steel Construction (AISC) 360-616 Specification for Structural Steel Buildings, AISC, ly76July 7, 2016
- ANSI/AISC 341-818 AISC Seismic Provisions for Structural Steel Buildings, AISC 2018
- Building Code Requirements for Structural Concrete, American Concrete Institute (ACI) 318-919, ACI 2019
- Code Requirements for Environmental Engineering Concrete Structures, ACI 350-06, ACI 2006
- AISC Steel Construction Manual, 515th Edition, AISC 2016
- AISC Seismic Design Manual, 3th3th Edition, AISC 2016
- Minimum Design Loads for Buildings and Other Structures, ASCE 7-616, ASCE 2018
- Structural Welding Code Steel, AWS D1.1/D1.1M:2010, AWS 2010
- Structural Welding Code Aluminum, AWS D1.2/D1.2M:2008, AWS 2008
- Structural Welding Code Stainless Steel, AWS D1.6/D1.6M:2007, AWS 2007
- USACE EM 1110-2-2100, Stability Analysis of Concrete Structures, 2005
- USACE EM 1110-2-2104, Strength Design for Reinforced Concrete Hydraulic Structures, 2016
- USACE EM 1110-2-2502, Retaining and Flood Walls, 1989
- USACE EM 1110-2-2906, Design of Pile Foundations, 1991
- USACE EM 1110-2-6050, Response Spectra and Seismic Analysis for Concrete Hydraulic Structures, 1999
- USACE EM 1110-2-6053, Earthquake Design and Evaluation of Concrete Hydraulic Structures, 2007
- USACE Engineering Technical Letter 1110-2-584, Design of Hydraulic Steel Structures, 2014
- USACE Engineering Regulation 1110-2-1806, Earthquake Design and Evaluation for Civil Works Projects, 2016
- USACE Engineering Regulation 1110-2-8157, Responsibility for Hydraulic Steel Structures, 2009

- International Building Code 2018, International Code Council

5.5.3 Specific Criteria

The material properties applicable for final engineering are summarized in Table 5-6.

Table 5-6. Structural materials

Material	Applicable Code and/or Standard
Stainless Steel	
Stainless steel sheet	ASTM A240 Type 304
Stainless steel hollow sections	ASTM A312 Type 304
Stainless steel bars and shapes	ASTM A276 Type 304
Stainless steel anchor rods	ASTM A193 Type 316
Stainless steel bolts	ASTM A193 Grade B8M, Class 1
Stainless steel nuts	ASTM A194 Grade 8M
Stainless steel washers	Stainless steel Type 316
Mild Steel	
Steel plates and bars	ASTM A36 Grade 36, ASTM A572 Grade 50
Steel hollow structural sections	ASTM A500 Grade C (Fy = 50 kilopound per square inch)
Steel shapes	ASTM A992, ASTM A36
Steel anchor rods	ASTM F1554
Steel bolts	ASTM F3125
Steel nuts	ASTM A563
Miscellaneous Metals	
Grating	NAAMM MBG 531
Embeds and guide slots	AA, AISC
Gantry	Mild steel: hot dipped galvanized
Ladders	OSHA, ANSI 14.3, ASCE 7
Stairs	NAAMM AMP 510, International Code Council A117.1
Handrails	AA, AISC, OSHA, International Code Council
Epoxy anchors	Two component epoxy, ASTM C-881
Reinforced Concrete	
f'c for structural concrete	4,500 psi
F'c for concrete fill	2,500 psi
Reinforcing steel (typical)	ASTM A615 Grade 60

Table 5-6. Structural materials

Material	Applicable Code and/or Standard
Reinforcing steel (welded)	ASTM A706 Grade 60
Minimum reinforcing steel quantity: water-bearing with expansion and/or control joint spacing less than 40 ft	ACI 350-06 Table 7.12.2.1 Grade 60 Values
Maximum water to cement + fly ash ratios	0.42 (hydraulic structures) 0.45 (non-hydraulic structures)
Concrete Cover	3 inches against earth. 2 inches all other locations.

The use of the following materials or their alloys in contact with fish facility water (including locations where natural drainage or wash water and windblown materials can readily enter pool water) is prohibited:

- Zinc
- Copper
- Brass
- Mercury
- Cadmium
- Lead
- Organohalogens (not polymerized)
- Asbestos
- Asphaltic compounds
- Petroleum products
- Coal tar
- Known carcinogens, mutagens, or teratogens
- Pressure-treated wood products

Load criteria for the following loads are summarized in Table 5-7. The general loads considered in the design of the intake facility are summarized in the following sections. These loads plus site-specific loads will be grouped into various loading conditions to assess factors of safety:

- Dead loads
- Hydrostatic loads
- Uplift
- Live loads
- Wind loads

- Snow loads
- Seismic loads
- Impact, debris, and trash loads
- Temperature loads

Table 5-7. Load Criteria

Material	Applicable Code and/or Standard	
Dead Loads		
Concrete: unit weight	150 pounds per cubic foot (pcf)	
Steel: unit weight	490 pcf	
Aluminum: unit weight	169 pcf	
Hydrostatic Loads		
Water: unit weight	62.4 pcf	
Uplift		
At base of hydraulic structures	100% adjacent river pressure over 100% of the base area	
At internal planes	Vary linearly from hydrostatic head at the external surface of a hydraulic structure to the hydrostatic head at any internal surface	
During an earthquake	Remain unchanged	
Live Loads	**Uniform Load (lb per square foot)**	**Concentrated Load (lb)**
Floor live loads (includes mechanical/electrical allowance)	125	2,000
Concrete slabs on grade, including yards subject to trucking	250, HS20	
Elevated concrete floors with light mechanical equipment storage	125	2,000
Corridors, exits, stairs	100	
Vehicle access	HS20	
Wind Loads[a]		
Three-second gust basic wind speed	8,888 miles per hour (ASCE 7-616 Fig. 26.5-B1B)	
Exposure category	C (ASCE 7-616 Section 26.7)	
Risk category	I (ASCE 7-616 Table 1.5-11)	
Impact, Debris, and Trash Loads		
Impact loads	Approximated based on the methods described in ASCE 7-616, Chapter C5, Section C5.4.5	

Table 5-7. Load Criteria

Material	Applicable Code and/or Standard
Temperature Loads[c]	
Maximum high	9494°F
Maximum low	616°F (Winter design temperature)
Seismic Loads	
S_S	0.490
S_1	0.23223
Site C[d] Class[d]	D
S_{MS}	0.690690
S_{M1}	0.480480
S_{DS}	0.460460
S_{D1}	0.320320
SDC	D

a. Wind loads will be computed in accordance with ASCE 7-616 Chapter 26, based on the parameters in this table.
b. Snow load analysis will be computed in accordance with ASCE 7-616 Chapter 7, based on the parameters in this table.
c. Average temperatures were taken from the following website: https://www.weatherbase.com/weather/weather.php3?s=29427&cityname=Stockton-California-United-States-of-America&units=us.
d. The site classification was selected based on the N-value of the soil of 24 blows/ft suggesting stiff soil.

Design Basis

Required Strength

Reinforced concrete, steel, and aluminum members will be designed to have strength and capacity at all sections at least equal to the effects of factored loads in accordance with the following equation:

$$U = \sum \gamma_i L_{ni} \leq \phi R_n$$

where:

$U =$ required strength

$\gamma_i =$ load factors per ASCE7, ACI350, or USACE as applies

$L_{ni} =$ loads

$\phi =$ resistance factor per ACI, AISC, or AA as applies

$R_n =$ nominal resistance from ACI, AISC, or AA as applies

Stability Criteria

The reinforced concrete sill will be designed to achieve the minimum factors of safety against overturning, sliding, and flotation prescribed in USACE EM 1110-2-2100, Stability Analysis of Concrete Structures.

5.6 Electrical and Instrumentation and Controls Criteria

The electrical and instrumentation & control design for the Project will be completed by Control Point Engineering (CPE). The following section is limited to a general description of anticipated power and control needs at the site. More detailed electrical criteria will be supplied by CPE, upon their incorporation into the Project team at the 65 percent design phase.

5.6.1 General Design Description

Components of the Project that will require electrical design include but are not limited to the following:

- Power and control to the compressed-air supply for the Obermeyer weir
- Power and control to the water control gate actuators
- Power to the local control panel for the fish protection screens
- Power to the fish protection screen hoists
- General power supply to utility building equipment
- Indoor and outdoor lighting
- Programmable logic controller and telemetry system
- Transmitters and primary elements

Three phase, 480-volt power is available at the site. HDR will advise CPE on the required power loads for the proposed design. CPE will perform an analysis of the available power with respect to new power loads in order to evaluate the adequacy of the existing service transformer. A new distribution panel will be provided to power all loads provided for the Project. A power interlock, disconnect, and socket will be provided for a trailered stand-by generator. New electrical equipment will be sized adequately for this conceptual level of design. All equipment will be new as part of the Project design.

Obermeyer water control gates are provided with a manufacturer-supplied automatic water level control system and powered by air from the Obermeyer compressed-air system. Although the Obermeyer may be lowered during power outages, it will not be able to be raised without a source of power. The control system is used to maintain upstream operational pool elevations and automatically lowers as desired to conduct sluicing operations.

The fish protection screens are typically furnished with manufacturer-supplied control panels to provide automatic cleaning and the ability to raise/lower the screens. The control panel will be outdoor rated and placed in line-of-site with the fish screen units.

In addition to the controls described above, a control panel will likely be furnished at the diversion structure and located within the utility building. The control panel will include a small, industrial programmable logic controller intended to provide remote control and monitoring of the equipment. A front panel-mounted operator interface terminal can be used to provide local display of equipment status, surface water elevations, flow readings, and ability for operator inputs. The control panel will include a radio modem to allow the system to be monitored using SEWD's existing SCADA system. SCADA will be compatible with SEWD's existing system.

Pool and discharge surface water elevations will be measured using a combination of hydrostatic submerged level transmitters installed in stilling wells as well as overhead non-contact radar sensors. The surface water level readings will be transmitted to the control panel at the diversion structure for display and monitored remotely by SCADA.

The flow for each diversion pipeline will need to be metered to measure and totalize flows. The type of flowmeter used will be determined during the design process; however, at this time inline, electromagnetic type flowmeters are proposed for use because of their high level of accuracy and reliable operation. The location of the flowmeter sites will also be determined during design. Flow readings will be transmitted to the control panel at the diversion structure for display and monitored remotely by SCADA.

6 Other Criteria

Criteria that do not specifically fall under agency-specific or technical design criteria are included in this section.

6.1 Fire Protection

No fire protection requirement associated with this Project is anticipated at this time.

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Appendix B. Hydraulic Modeling Summary Report

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DRAFT 90% Hydraulic Modeling Summary Report

Bellota Weir Modifications Project

Mormon Slough, Calaveras River, CA

February 4, 2022

Prepared for the Stockton East Water District

Prepared by HDR with technical assistance from KSN.

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Appendix B. Hydraulic Support Exhibits

Appendix C. 1D HEC-RAS Results

Appendix D. 2D HEC-RAS Results

Appendix E. Rock and ESM Sizing Calculations

Acronyms and Abbreviations

1D	one-dimensional
2D	two-dimensional
CCWD	Calaveras County Water District
CDFW	California Department of Fish and Wildlife
CDWR	California Department of Water Resources
cfs	cubic foot/feet per second
CHCP	Calaveras River Habitat Conservation Plan
CVFED	Central Valley Floodplain Evaluation and Delineation Program (
District	Stockton East Water District
FEMA	Federal Emergency Management Agency
FHWA	Federal Highway Administration
FIRM	Flood Insurance Rate Map
FIS	Flood Insurance Study
ITP	Incidental Take Permit
H:V	horizontal:vertical
HEC-RAS	Hydrologic Engineering Center - River Analysis System
LiDAR	light detection and ranging
NAVD88	North American Vertical Datum of 1988
NCHRP	National Cooperative Highway Research Program
NMFS	National Marine Fisheries Service
NSSDA	National Standards for Spatial Data Accuracy
Project	Bellota Weir Modifications Project
Reclamation	U.S. Bureau of Reclamation
SEWD	Stockton East Water District
USACE	U.S. Army Corps of Engineers
USGS	U.S. Geological Survey
WSE	water surface elevation

1 Introduction

The Stockton East Water District (SEWD or District) is developing designs for the Bellota Weir Modifications Project (Project) located at the fork of the Mormon Slough and the Old Calaveras River, about 17 miles downstream of the New Hogan Dam. It is a continuation of the Calaveras River Anadromous Fish Protection Project. The Project includes removal of the existing weir and construction of several new elements intended to provide more reliable water delivery systems for SEWD and Calaveras County Water District (CCWD) customers, improve migration of both juvenile and adult fish, and reduce fish entrainment at the facilities, thereby enhancing populations of anadromous salmonids using the Calaveras River (CH2MHILL 2003). The following Project actions will be taken to achieve these goals:

- Replace the existing profile control infrastructure
- Provide fish passage facility for fish species throughout various life stages at Project site
- Provide fish exclusion and protection measures for SEWD intake at Bellota Weir
- Exclude fish from migrating downstream into the Old Calaveras River
- Maintain existing flood conveyance

1.1 Purpose

This report documents and summarizes the hydrologic, hydraulic, and scour analyses conducted for the Project. It also summarizes key background conditions, characteristics, analysis input, and criteria that shaped the hydraulic approach, as well as results and recommendations to meet the following hydraulic objectives:

- Support regulatory permitting efforts associated with Project improvements at Bellota Weir
- Inform design by analysis of proposed hydraulics and scour potential
- Determine Project impacts on flood conveyance

1.2 Approach

To address the hydraulic objectives, the following were conducted:

- **Define Study Area:** Defined the study area extent and described the major features, facilities, and regulatory conditions as needed to provide context to study efforts.
- **Review Existing Data and Past Studies:** Summarized the available and pertinent existing data and studies as data sources for this study and identified data gaps.
- **Establish Hydrology:** Summarized the Calaveras River and Mormon Slough flow conditions used in the hydraulic analyses for this study.
- **Establish Existing Hydraulic Conditions:** Described the hydraulic analysis approach, input, and results of current conditions within the study area.

- **Analyze Proposed Hydraulic Conditions:** Described the input, modeling, and results of proposed hydraulic conditions within the study area. Reported hydraulic results as they relate to established design and regulatory criteria and compared to existing conditions.
- **Perform Scour Analysis of the Proposed Conditions:** Described the scour analysis approach, input, calculations, and results under proposed conditions.
- **Recommendations:** Provided recommendations to proposed facility design, detailed recommendations and inputs into scour countermeasure design, and demonstrated compliance with established design and regulatory criteria.

1.3 Background

The existing Bellota Weir facility on the Mormon Slough/Calaveras River is owned and operated by SEWD and provides water to urban and agricultural users. The Bellota Intake provides municipal and industrial flow year-round for the City of Stockton, and also supplies irrigation water to local farmers during the irrigation season (generally between mid-April and mid-October). The Old Calaveras Headworks (Calaveras Headworks), provides flow control and prevents flooding to downstream landowners on the Old Calaveras River by routing flood waters down the Mormon Slough. During most flow regimes, both the Bellota Weir and Calaveras Headworks are complete fish barriers to upstream migrating adult salmonids.

Continued operation of the Bellota Weir and diversion will be guided by the final Calaveras River Habitat Conservation Plan (CHCP; National Oceanic and Atmospheric Administration 2020). The CHCP provides operational criteria to support the biological goals of maintaining a viable population of threatened California Central Valley steelhead, *Oncorhynchus mykiss,* within the CHCP boundaries, and maintains adequate habitat conditions upstream of the Bellota Weir for fall-, late fall-, spring-, or winter-run Chinook Salmon, *Oncorhynchus tshawytscha*, that may opportunistically migrate into the conservation area. While the CHCP intends to provide conditions that support Chinook Salmon should they migrate into the conservation area, these salmon are not expected to maintain a viable population based both on pre-dam and current conditions. The CHCP enables SEWD to comply with the Endangered Species Act, protecting and managing fishery resources and habitat while maintaining reliable water delivery to its constituents. Following National Marine Fisheries Service (NMFS) approval on August 11, 2020, the District is authorized for a 50-year Incidental Take Permit (ITP #23264), for Endangered Species Act-listed species under NMFS authority.

Upgrade or replacement of the Bellota Diversion, Weir, and Calaveras Headworks is a required compliance measure specified in the CHCP, as part of a seven target fish passage objective. Specifically, the CHCP's target states:

> FP1 and AE1. Avoid migration delays and blockage, and entrainment within the Old Calaveras River Channel by constructing a non-entraining barrier at the Old Calaveras River Headworks Facility and at the downstream end of the channel near the confluence with the [Stockton Diverting Canal] within the first ten years of the ITP.

> FP2/AE3. Construct and implement a combined crest gate/fishway/fish screen at the Bellota Diversion Facility to improve [salmonid] passage into/out of the 18-mile spawning and rearing reach between Bellota and New Hogan Dam and to prevent fish entrainment; target completion within first five years, but no later than 10 years of [issuance of] the ITP.

Based on the CHCP targets summarized above and SEWD's water supply needs, the three primary Project objectives are to:

- **Improve fish passage** by designing and constructing a new crest gate dam, fishway, and fish screens that include passage for *Oncorhynchus mykiss* and opportunistic migration for fall-, late fall-, spring-, and winter-run Chinook Salmon.
- **Reduce fish entrainment** by constructing a new non-entraining fish barrier at the Old Calaveras River.
- **Provide more reliable water delivery** through weir and intake improvements.

2 Site Conditions

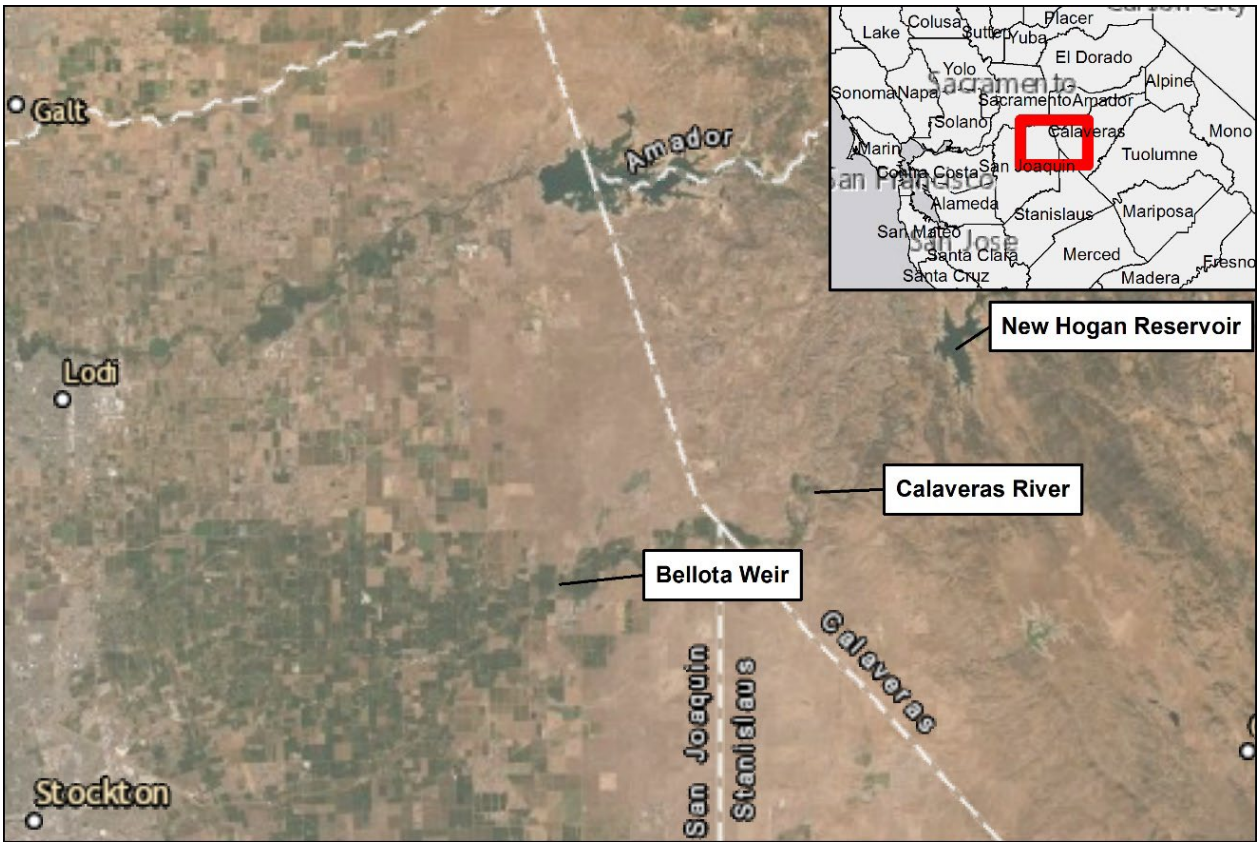
Physical considerations related to the existing facilities' hydrologic, hydraulic, and geotechnical conditions are presented in this section.

2.1 Calaveras River

The Project is located within the Calaveras watershed at the divergence of the Calaveras River Channel and Mormon Slough. Upstream of the Project site, the Calaveras River has one reach upstream of the New Hogan Reservoir and Dam and the other between the dam and Mormon Slough. The hydrology is dominated by rain events due to the low elevation of the upper watershed area. The basin tributary to the New Hogan Reservoir is 363 square miles. The Calaveras watershed between New Hogan Reservoir and the Project is an additional 107 square miles. The Calaveras River flow downstream of New Hogan Dam are metered by reservoir operation and influenced by diversion operations during irrigation season. Figure 2-1 shows the general Project location and surrounding area.

At the Project Site, the Calaveras River channel below Mormon Slough is controlled by the Calaveras Headworks. Compared to the channel upstream of Mormon Slough, the channel capacity is significantly reduced due to excessive vegetation caused by irregular flows.

Figure 2-1. Project Location Map



2.2 Existing Facilities

Artificial structures located in the watershed can become impediments to salmonids at various times due to flow conditions. Fish entering the Stockton Diverting Canal or Mormon Slough can become stranded in pools to die or become prey. This report is most concerned with:

- Bellota Weir, built in the late 1940s
- Bellota Intake for the Dr. Joe Waidhofer Water Treatment Plant built in 1978
- Old Calaveras Headworks, built in 1933
- Mormon Slough, built in 1971

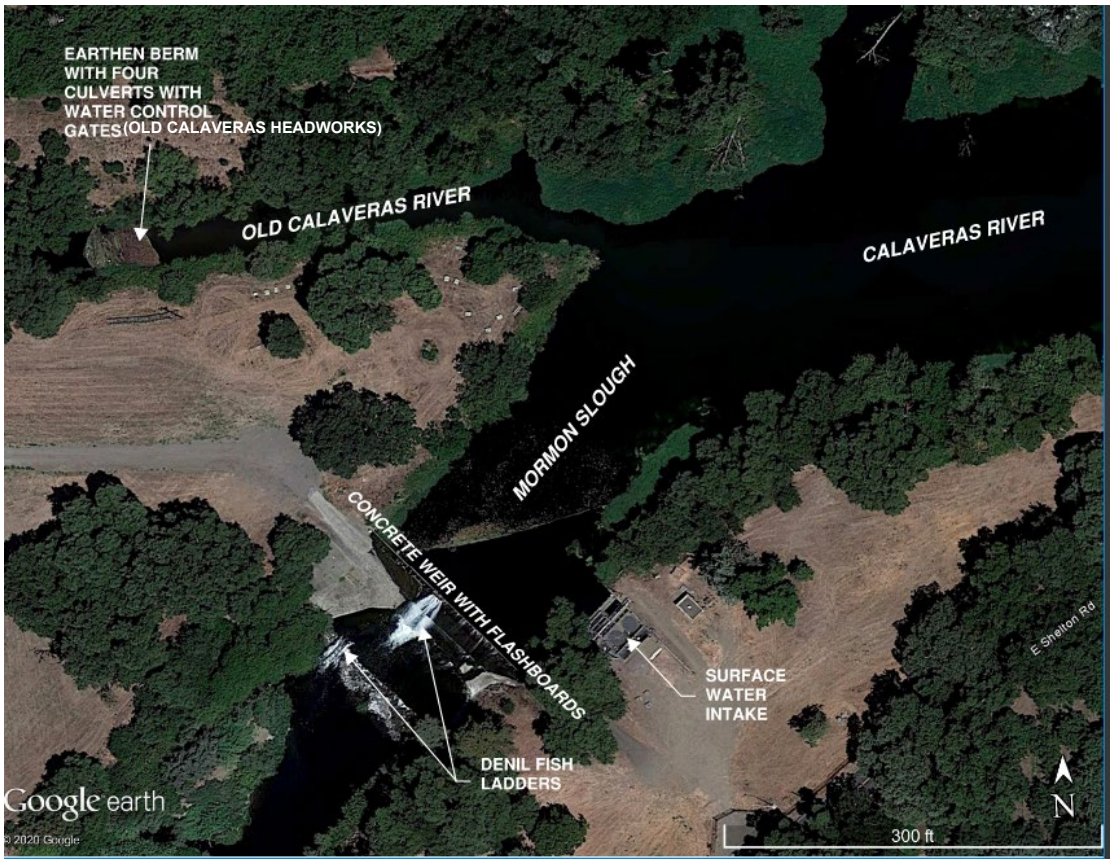
2.2.1 Mormon Slough

The Mormon Slough is a flood control facility completed in 1971 to divert flood flows from the Upper Calaveras River and re-introduce them closer to the City of Stockton. The Project included widening the Mormon Slough, levee construction, and bank protection. The design discharge for the Mormon Slough is 12,500 cubic feet per second (cfs) and is intended to accept the full flow of the Calaveras River for the 100-year event.

2.2.2 Bellota Weir

The Bellota Weir is the largest check dam on the Calaveras River system. The weir is located on the Mormon Slough approximately 350 feet downstream of its confluence with the Old Calaveras River. The weir is an in-river concrete structure designed to accommodate installation of a flashboard dam; during irrigation season, the check dam crest with the flashboards installed is 8 feet above the channel invert (invert elevation of 121.44 feet North American Vertical Datum of 1988 [NAVD88]). During non-irrigation seasonal (flood season) periods of the year, shorter flashboards are installed 2 feet above the channel invert. Existing features of the Project are identified in the Figure 2-2.

Figure 2-2. Prominent Features of the Bellota Weir Modifications Project Site



The Bellota Weir includes two Denil fish ladders, installed in 1999 and 2001. During the irrigation season, only the downstream Denil fish ladder is in place, while during non-irrigation times, both Denil fish ladder sections are in place. Employees from California Department of Fish and Wildlife (CDFW) and Fishery Foundation evaluated conditions at the ladders in 2007. The upper ladder is longer than the maximum recommended ladder length of 30 feet. The lower ladder has a 3 horizontal to 1 vertical (3H:1V) slope ratio, double the maximum recommended slope of 6H:1V. Finally, water does not flow through the lower ladder as intended but instead flows around the sides resulting in an inadequate water supply to the ladder that confuses fish looking for the ladder entrance (California Department of Water Resources [CDWR] 2007).

2.2.3 Bellota Intake

The Bellota Intake is a gravity diversion constructed in 1978 on the south riverbank about 50 feet upstream of the Bellota Weir. Water diverted through the intake is conveyed to the City of Stockton and the Dr. Joe Waidhofer Water Treatment Plant (SEWD, HDR, and KSN 2020). The structure possesses bulkhead slots for installation of stop logs, a trashrack facility, two 14-foot-wide rectangular intake channels perpendicular to the river, two intakes with fish screens, a bypass pipeline, a sediment trap, and two slide gates to regulate flow.

2.2.4 Old Calaveras Headworks

The Old Calaveras Headworks is located approximately 600 feet downstream from the Old Calaveras River, Mormon Slough confluence (Figure 2-2). The existing structure includes an earthen berm, four culverts with control gates, a trashrack, and concrete slope lining.

The earthen berm crest is 68 feet long and 10 feet wide. The 4-foot-wide, concrete culverts penetrate the berm and control flow to the Old Calaveras River. There is a 21-foot-wide trashrack on the upstream side of the berm protecting the culvert inlets, their concrete headwall, and wing walls. Flow is controlled with slide gates on the upstream side – two are operated manually and two are electrically actuated. The slide gates are operated between April and November to deliver water to downstream agricultural users, intermittently operated between November and April for groundwater recharge on the Old Calaveras River, and open during irrigation season to provide flow to downstream diverters (SEWD and FISHBIO 2019).

The headworks is a barrier to upstream adult fish migration and does not comply with juvenile downstream passage or fish protection guidelines provided by CDFW and NMFS.

In 2003, SEWD placed a small mesh net across the Old Calaveras River upstream of the Old Calaveras Headworks structure (CH2MHILL 2003) to encourage juvenile fish to reside in the Bellota Pool and migrate upstream to suitable habitat or downstream to the Mormon Slough.

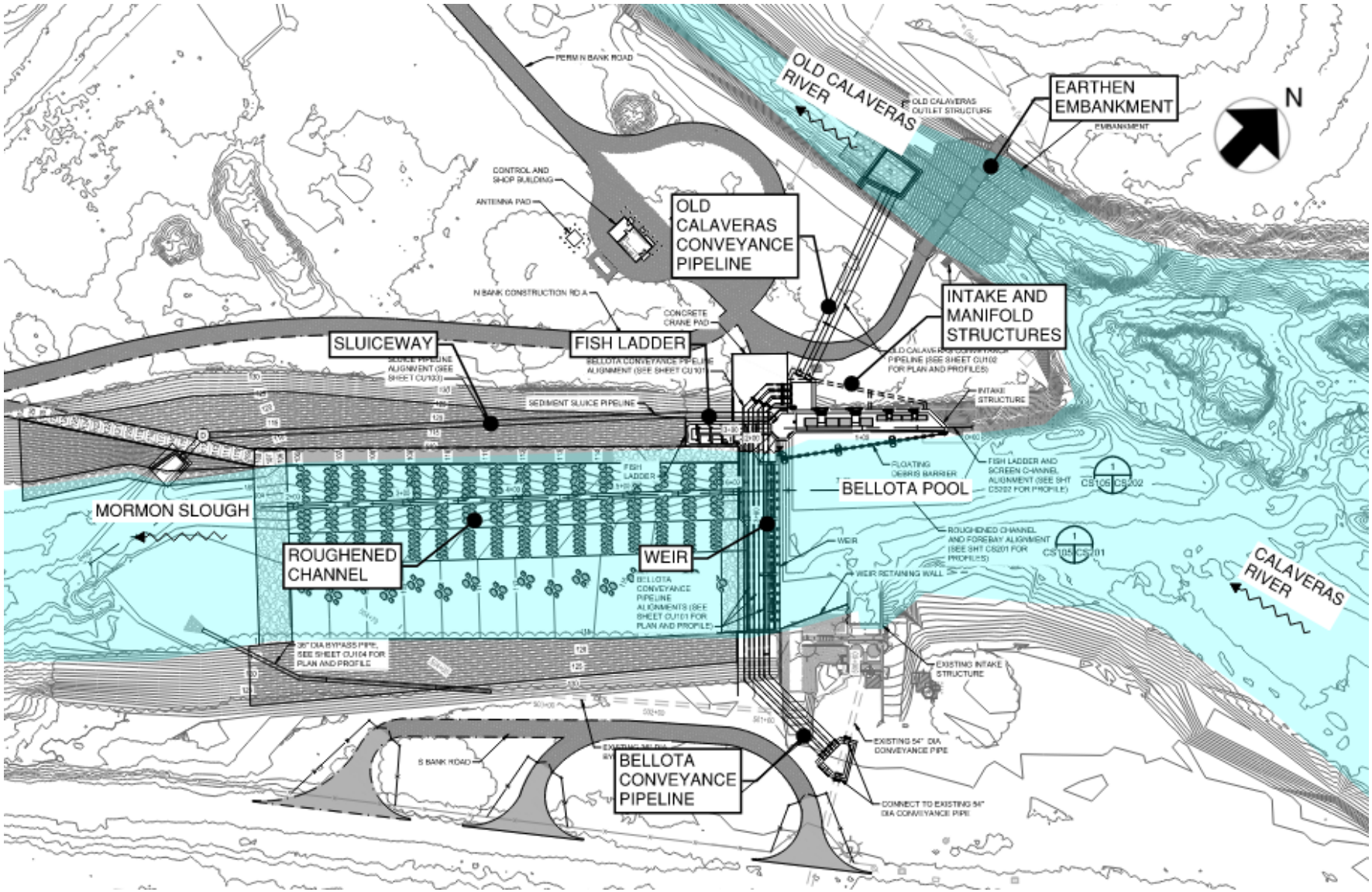
2.3 Proposed Facility

The proposed Project facility combines several operational and engineered elements to achieve its intended objectives: intake and weir replacement and improvements for effective fish passage, reduced fish entrainment, and water supply reliability. The proposed Project elements are briefly summarized below and illustrated in Figure 2-3:

- Construct a concrete weir with a series of weir gates designed to modulate forebay pool elevations based on operational needs. The proposed weir consists of a 150-foot-wide gated concrete weir with three gate openings. A concrete sill (elevation 115.5 feet) just downstream of the weir gates regulates the upstream water surface during normal operation. A low flow notch (invert elevation 114.3 feet) in the weir sill allows passage of low flows (Figure 4-5.).

- Improve sediment conveyance by adding a vertical slide gate at the dam, operated to facilitate bypass and sluicing functions.
- Provide reliable fish passage using a roughened channel in combination with a vertical slot fishway to accommodate the full range of flows and river stages at the site. The fish passage ramp with a low flow channel extends from weir sill approximately 360 feet downstream to where it terminates at an approximately 2.5-foot-deep pool that regulates the water surface during low flows, and then ties in with the existing channel invert. Roughness features (boulder weirs and rock clusters) along the fish passage ramp will provide sufficient depth during low flows and velocity refugia during high flows. Boulder weirs will be located at approximately 8-foot intervals along the low flow channel; the boulders will protrude from 1.5 to 2.5 feet from the low flow channel invert. Rock clusters will be placed at a similar interval along the southern portion of the fish passage ramp and protrude approximately 1.5 feet from the ramp invert.
- Provide reliable fish screening, both for the Bellota Intake and the Old Calaveras River, to prevent fish entrainment.
- Provide a non-entraining fish barrier to the Old Calaveras River.

Figure 2-3. Overview of Proposed Project Elements



3 River Hydrology

3.1 Calaveras River Hydrology

This section describes the Calaveras River hydrology, including historic and ongoing flow regulation, flood recurrence, fish passage design flows, and Project-specific flow uncertainties.

Hydrology in the region was analyzed using desktop research, a U.S. Army Corps of Engineers (USACE)-installed gage downstream of the weir, and data provided by the Federal Emergency Management Agency (FEMA).

3.1.1 Historic and Ongoing Flow Regulation

Calaveras River/Mormon Slough flows are regulated by operation of New Hogan Dam about 17 miles upstream of the Mormon Slough and Old Calaveras River bifurcation. Discharge from the dam is altered throughout the year. Additionally, diversions throughout the reach between Bellota Weir and New Hogan Dam contribute to water withdrawals that must be considered when evaluating low flow conditions.

Flow release from New Hogan Dam is regulated by the USACE; rights to storage releases are held by the U.S. Bureau of Reclamation (Reclamation) and contracted to SEWD and CCWD. Streamflow records are available using a USACE gage for both pre- and post-construction of the dam according to the CH2MHILL reports, but gage information prior to 1997 was not found online and multiple information requests to USACE yielded no results. There is no U.S. Geological Survey (USGS) gage nearby.

Releases to the river are lowest between October and December. Because much of the upper watershed is below elevation 4,000 feet (typical winter snowline), flow in the Calaveras River is heavily reliant on precipitation instead of snow melt unlike many other rivers nearby. The dam occasionally releases flood flows between January and March, but releases become more consistent during the irrigation season (Mid-April to Mid-October). These flows supply most of the irrigation demand for diversions between the dam and weir (CH2MHILL 2003). In a February 27, 2020 workshop, SEWD stated 20 to 100 cfs of flow is released downstream of Bellota Weir during the irrigation season (SEWD, KSN, and HDR 2020).

3.1.2 Flood Recurrence

Previously established flood events were used to determine water surface profiles at the Project location for alternative development, comparison, and measuring performance. Flood recurrence information for the Calaveras River and Mormon Slough at Bellota was obtained from two sources: the Flood Insurance Study for San Joaquin County conducted by the FEMA (2016), and the Lower San Joaquin River Feasibility Study conducted by the USACE (2017). FEMA flow values were used to inform the design of the proposed facility as it relates to potential impacts on the regulatory floodplain. The USACE study includes flood recurrence values that account for flow regulation by New Hogan Dam (regulated values) and have been used in flood risk management planning for the Stockton area. A combination of FEMA and USACE regulated values informed the proposed design as it relates to channel and structural stability. FEMA and USACE

flow values for selected recurrence intervals are listed in Table 3-1. The FEMA 50-year and 100-year flows are similar due to the 12,500-cfs annual objective release flow target from New Hogan Dam (Table 3-1).

Table 3-1. Flood Frequency and Magnitude of Calaveras River Flow at Mormon Slough at Bellota USACE Gage ID MRS

Recurrence Interval	Annual Exceedance Probability (%)	FEMA (2016) (cfs)	USACE, Regulated (2017) (cfs)
1-year	99.9	-	-
2-year	50	-	3,520
5-year	20	-	9,520
10-year	10	12,500	9,530
25-year	4	-	10,640
50-year	2	12,500	12,500
100-year	1	12,690	12,500
500-year	0.2	27,100	16,000

3.1.3 Fish Passage Design Flows

Selection of site-specific fish passage design flow criteria influences fish passage facility size, complexity, and anticipated performance. Guidelines presented by the NMFS and CDFW are based on exceedance analysis of mean daily flows during salmonid migration season, but can be modified to suit site-specific requirements.

An exceedance analysis was conducted to determine annual and seasonal low and high fish passage design flows (HDR 2021). The analysis included mean daily streamflow data obtained from USACE Gage ID MRS (USACE 1997) for the period of 1997 to 2021. Seasonal periods analyzed included irrigation season and migration seasons for salmonid species covered by the CHCP. Per NMFS (2011) guidelines, low and high fish passage design flows were defined as the 95 percent and 5 percent exceedance values, respectively.

The maximum high flow value across all of the analyzed migration seasons was selected as a high fish passage design flow. The irrigation season high flow and the annual high flow were also considered during the design process to verify adequate fish passage across a range of flows for which hydraulics in the roughened channel may vary. Due to upstream flow regulation and diversion, the Calaveras River near the Bellota Weir periodically runs dry throughout the year, resulting in a calculated low fish passage design flow equal to 0 cfs. CDFW guidelines recommend that 3 cfs be used as an alternative low fish passage design flow in instances where the calculated low flow value is 0 cfs; however in a February 27, 2020 workshop, SEWD stated 20 to 100 cfs of flow is released downstream of Bellota Weir during the irrigation season (SEWD, KSN, and HDR 2020). Additionally, 20 cfs was assessed as the likely lower threshold of discharge

for passable hydraulics within the anticipated roughened channel concept, which relies on a low flow notch and rock weirs to concentrate flows to support a swim through condition. This configuration must be formed by rock elements large enough to remain stable under the high flow condition and the size of these elements constrains the physical dimensions of the low flow section. Based on this information, 20 cfs was selected as the low fish passage design flow.

Table 3-2 provides fish passage design flows included in hydraulic analysis.

Table 3-2. Low Fish Passage and High Fish Passage Design Flows for Annual, Irrigation, and Salmonid Migration Seasons

Season	Period	Low Fish Passage Design Flow (cfs)	High Fish Passage Design Flow (cfs)
Annual	All year	3.0*	**545.3**
Irrigation	Mid-April to Mid-Oct	7.2	**150.9**
Fall Chinook Migration	Nov 1 to Dec 31	3.0*	420.6
Steelhead Migration	Nov 1 to March 31	3.0*	1,728.6
Late Fall Chinook Migration	Nov 1 to Feb 28	3.0*	**1,735.2**
Spring Chinook Migration	March 1 to May 31	3.0*	908.1
Winter Chinook Migration	Jan 1 to May 31	3.0*	1,710.0

Notes: Low and High Fish Passage Design Flow are defined as the 95 percent and 5 percent exceedance value for the specified migration season, respectively.
Bolded values indicate the selected high fish passage design flows
* Calculated value is 0 cfs. Alternate low design flow selected per California Department of Fish and Game 2002 (Table 4) and 2004 (Table IX-5)

3.1.4 Project-Specific Flow Uncertainties

Numerous hydrologic inputs and withdrawals in the Project reach of the Calaveras River system create high flow variability during seasonal low flow conditions. The Fish Passage Barriers Assessment report shows there are 29 diversions between the weir and New Hogan Dam on the Calaveras River (CDWR 2007).

In addition, New Hogan Lake influences river flow through reservoir releases, which may devalue the exceedance analysis performed. High or low flows may result from reservoir releases rather than natural hydrology.

The following assumptions have been adopted as part of the design process:

- Mean daily flow statistics will be based on data available after 1997 (USACE 1997). Use of data recorded between December 10, 1997 and July 28, 2021 provides an 18-year period of record that gives an adequate representation of current and future operational conditions in the Project reach. This information will be used herein to make reasonable assumptions for development of design criteria used for alternative assessment.

- The exceedance analysis is valid considering generalities and seasonality; it is influenced by New Hogan Dam releases and diversion of flow. Any adjustment in dam releases or diversion intakes will influence flow magnitudes at the Project site.

4 Hydraulics

A combination of one-dimensional (1D) and two-dimensional (2D) models were used to inform the design process and confirm design compliance with regulatory requirements. Specifically, the objectives of the combined modeling efforts are to:

- Support regulatory permitting efforts associated with Project improvements
- Confirm that the range of hydraulic conditions anticipated after Project implementation will meet the fish passage, sediment passage, and water supply and flood resilience/stability criteria established during preliminary and final design
- Refine sediment continuity and particle mobility thresholds to inform sediment transport and sluicing targets
- Provide hydraulic conditions such as shear stress to verify the following:
    - Sizing and stability of channels
    - Boulder matrices of the roughened channel
    - Supporting rock filters for boulder structure and low flow channel
    - Potential for scour holes downstream of structure and forces on existing structures

Two distinct modeling efforts were completed to meet these objectives. A 1D Hydrologic Engineering Center - River Analysis System (HEC-RAS) model was used to verify compliance with the FEMA No-Rise condition. A 2D HEC-RAS model was used to inform the proposed weir structure and fish passage ramp design.

4.1 One-Dimensional Model – No-Rise Analysis

The Project is currently within a regulatory floodway designated by the FEMA National Flood Hazard Layer and Flood Insurance Rate Map (FIRM) (FIRM Panel 06077C0370F effective October 20, 2016. For new construction to occur, the Project design must demonstrate, through appropriate hydraulic analyses, that the proposed improvements will not result in any increase in base flood elevation for a 100-year event (1:100 chance of occurrence in a given year for the 100-year event).

4.1.1 Effective FEMA Mapping

The Calaveras River was first subject to detailed study for FEMA flood mapping purposes in 1994 (FEMA 2002). Per the FEMA Flood Insurance Study (FIS), a restudy was completed in 1998 (San Joaquin Area Flood Control Agency 1998) for both the Calaveras River and Mormon Slough, however, the restudy limits did not extend upstream to the Project. For the Calaveras River, the restudy extended to Solari Ranch Road. For the Mormon Slough, the restudy extended to Jack Tone Road. Both locations

are miles downstream of the Project and do not serve as the basis for mapped conditions near the Project.

Near the Project, the Calaveras River includes a stream centerline, lettered cross sections, mapped base flood elevations, designated floodway, and areas of floodway fringe. Figure 4-1 provides a portion of the October 20, 2016 FIRM map. In the figure, an area of floodway is mapped in the upstream most 1,600 feet of the Mormon Slough, starting at the channel bifurcation and ending at the Bellota Road Bridge. While mapped as a floodway, there are no lettered cross sections, base flood elevations, or centerline in this reach. This designated floodway transitions to a FEMA Zone A floodplain downstream. A detailed analysis likely was not used to map the Mormon Slough reach immediately downstream.

4.1.2 FEMA Flood Insurance Study

The FEMA FIS report (2016) details information on the existence, severity, and mapping basis of flood hazards within the County of San Joaquin for use under the National Flood Insurance Program. The FIS was last revised in 2016 as FIS Number 06077CV001B. The FIS provides summaries of included flood sources, associated discharges with these flood sources, floodway data, and flood profiles, among other information. The Calaveras River is included in these summaries, but with some inconsistencies. The flood source table lists the Calaveras River, but places the upstream limit at Solari Ranch Road (approximately 13 miles downstream of the Old Calaveras Headworks), instead of perhaps New Hogan Dam or the Stanislaus County line. The discharge summary table provides the most upstream discharge location at Jack Tone Road, with a discharge of 1,450 cfs and a contributing area of 14.4 miles. This discharge matches the re-study hydraulics from 1998 (San Joaquin Area Flood Control Agency 1998).

By contrast, the floodway data table, which details floodway widths and surcharge elevations at each lettered cross section, includes data for sections to the upstream limit at the Stanislaus County line. Additionally, the FIS includes flood elevation profiles for the 10-, 100-, and 500-year events from the county line to the Calaveras Headworks (labeled Diversion Structure). The 50-year profile overlaps the 10-year due to shared discharge values. No profile overtops the Diversion Structure and each is considerably reduced in depth downstream of the structure. The 50-year profile begins at the Calaveras Headworks and extends downstream.

The following conclusions were made from the FIS study:

- The 100-year event from the Calaveras River is diverted entirely to the Mormon Slough.
- No discharge passes the Calaveras Headworks. The Calaveras River discharge at Jack Tone Road is based on tributary areas downstream of the Calaveras Headworks.
- The original detailed study extended to the Stanislaus County line, but was not revisited during the 1998 restudy.

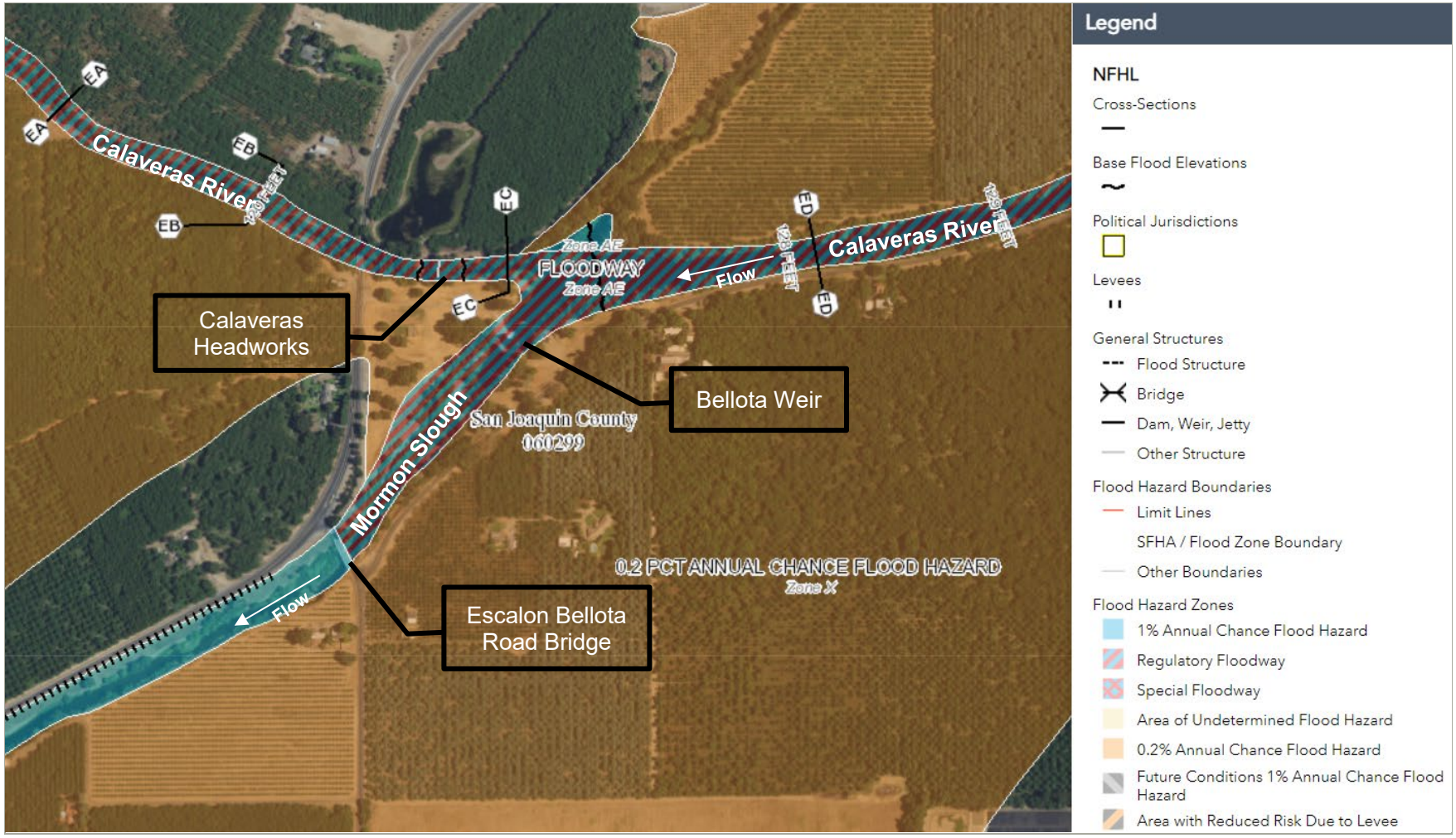
The Mormon Slough is also listed as a flood source and included in the various summary tables. The Summary Discharge table indicates that Mormon Slough starts at Bellota Weir and conveys diverted flow from the Calaveras River (FEMA 2016). The listed

drainage area is 480 square miles. However, the Floodway Data table only includes the Mormon Channel, a downstream facility, but not the Mormon Slough. No flood profiles are included for the Mormon Channel, which is typical for a FEMA Zone A.

In the absence of additional information, the following uncertainties exist:

- The basis of the designated floodway in the upper reach of the Mormon Channel. Potentially, the floodway was arbitrarily mapped. No Effective elevation data is present from which to compare a proposed condition (e.g., surcharge values).
- The influence of Mormon Channel conveyance on the flood profiles for the Calaveras River upstream of the Calaveras Headworks. In this area, the flood profiles are flat, which is anticipated for slack or ponded water. However, the backwater elevation should be based on the normal depth at the Mormon Channel entrance.

Figure 4-1. Effective Floodplain Exhibit



Source: FEMA (2016)

4.1.3 FEMA Data Request and Effective Model

A 1D USACE HEC-RAS model was likely developed to support the original mapping effort and would be classified as the Effective model. As a matter of general practice, the Effective model should be used as the basis for determining compliance with FEMA requirements and is typically obtained from FEMA or the local community floodplain administrator. However, through extensive effort and communication with FEMA and representatives from local flood control authorities, the Effective modeling has not been located. The FEMA Engineering Library has no record of this model, nor does the community floodplain administrator for San Joaquin County.

4.1.4 No-Rise Approach

For new construction to occur in the floodway, the Project must demonstrate, through appropriate hydraulic analyses, that the proposed floodway encroachment will not result in an increase in base flood elevation for a 100-year event (1:100 chance of occurrence in a given year for the 100-year event). In the absence of the Effective model, FEMA guidance, as confirmed in correspondence with San Joaquin County (Shayan Rehman, Senior Engineer, personal communications, 2021), stipulates new existing conditions modeling will serve as the basis to evaluate a No-Rise condition. A No-Rise condition will be accepted without the Effective model if the proposed flood elevation is below the existing conditions flood elevation and if the proposed flood profile is equal to or less than the current Effective base flood elevations.

4.1.5 Existing Condition Model

A 1D HEC-RAS model was developed for existing conditions to serve as a benchmark to evaluate the No-Rise condition for the proposed facility. The existing condition represented in the 1D model was the existing weir structure with flashboards implemented to raise the weir invert by 2 feet, from 113.44 feet to 115.44 feet.

Study Area

The study area covers an approximate 2-mile reach that begins on the Calaveras River approximately 1.6 miles upstream of where the Calaveras River is diverted into the Mormon Slough and ends on the Mormon Slough approximately 0.4 mile downstream of the diversion. Duck Creek confluences with the Calaveras River just upstream of the diversion. The Bellota Weir is approximately 300 feet downstream of the diversion. Apart from the Bellota Weir, Escalon Bellota Road Bridge approximately one-quarter mile downstream of the weir, is the only other structure within the study reach.

Model Geometry

One-dimensional model geometry consists of a channel centerline that defines the length of the flow path; cross-sections that define the hydraulic characteristics of the channel, including shape, roughness, and slope; and hydraulic structure elements that represent structures obstructing or affecting flow within the channel.

Terrain Development

Topographical data was aggregated within HEC-RAS RASMapper to develop modeling terrain surfaces from which cross-section geometry was extracted. The primary source of topographic or terrain data for construction of the HEC-RAS models was light detection and ranging (LiDAR) data compiled by CDWR under the Central Valley Floodplain Evaluation and Delineation Program (CVFED) and Delta Risk Management Study from March to April 2008. This served as the base layer for a composite modeling surface. The LiDAR data comprises points that densely cover the entire region, with a point spacing of 18 to 30 inches, depending on vegetation. The minimum expected horizontal accuracy was tested to meet or exceed a 3.5-foot horizontal accuracy at 95 percent confidence level using RMSE(r) x 1.7308 as defined by the National Standards for Spatial Data Accuracy (NSSDA). Final ground surface LiDAR point elevation data in areas other than open terrain meet or exceed NSSDA standards of 0.6 feet RMSE vertical (Accuracy z = 1.2 feet at the 95 percent confidence level). Accuracy was tested to meet a 0.6-foot fundamental vertical accuracy at 95 percent confidence level using RMSE(z) x 1.9600 as defined by the NSSDA. The horizontal datum of the stations is North American Datum of 1983 (2007) and the vertical datum is NAVD88.

Project survey data conducted by KSN and provided on June 04, 2020 to support site design was the best available data and used to enhance the base LiDAR data to provide correlation to proposed design elements.

The Calaveras River channel was supplemented by two survey datasets. In the area of the Bellota Weir, the Calaveras Headworks and approximately 4,600 feet upstream of the weir, bathymetric survey was completed as part of the Project survey dataset. Upstream of this bathymetric survey, channel cross-sections collected by CDWR for use in the CVFED effort and connecting three-dimensional polylines was used to develop simple bathymetry data in the upper areas of the study reach.

Cross-Section Development

The study reach was discretized into 96 cross-sections spaced along the channel centerline at approximately 150-foot intervals, with finer spacing near the weir. Each cross-section was extended across the study reach perpendicular to the direction of flow, ensuring that high points on either side of the study reach were included within the cross-section limits. Cross-section geometry was extracted from the composite terrain surface described above. Ineffective flow areas were used to reduce cross-section area available for conveyance at locations where structures or channel orientation obstructed flow. In addition to geometry, definition of bank stations and Manning's roughness values for each cross-section are required to perform flow calculations.

Bank stations partition the cross-section into three conveyance areas—main channel, left overbank, and right overbank—for which flow calculations are performed. Bankfull flow stage is typically used to estimate the extents of main channel. Grade breaks in the bank slope based on the terrain and changes in vegetation cover from aerial imagery were used for preliminary demarcation of the bankfull flow boundary. Channel banks within the study reach were generally steep and elevations at which grade breaks occurred were

inconsistent. Additionally, dense, bushy vegetation generally obscured the entire bank, making it difficult to differentiate the extents of overbank vegetation from vegetation overhanging into the main channel. Relatively frequent flood events (1- to 10-year recurrence intervals) are often used to estimate bankfull discharge in the absence of clear visual indicators (Natural Resources Conservation Service 2001). Thus, the 2-year regulated flow value (3,520 cfs) estimated by USACE (2017) was input into the 1D model to approximate bankfull flow stage and inform bank station placement. Preliminary bank stations that did not align with the 2-year water surface elevations (WSE) were adjusted accordingly.

Manning's roughness coefficients (*n*) are used to represent energy loss in the model. Manning's *n* values used in the model were associated with land cover types delineated within the study area based aerial imagery and available photos. Three land cover types were identified within the existing channel:

Alluvial channel with light to moderate vegetation land cover type was assigned to portions of the channel (primarily the channel invert) where either water was present or minimal vegetation growth was observed in the aerial imagery.

Alluvial channel/overbank with dense vegetation land cover type was assigned to the channel banks and portions of the channel where dense brush and reeds that would be submerged during high flows were observed in the aerial imagery.

Concrete land cover type was assigned to the concrete weir structure.

Each land cover type was assigned a Manning's *n* value based on descriptions presented by Chow (1959). Table 4-1 shows the land cover types and their associated Manning's *n* values and description.

Table 4-1. Manning's Roughness Values Used for the Existing Channel in the 1D HEC-RAS Model

Land Cover Type	Source	Description	Manning's *n*
Channel, earthen with light to moderate vegetation	Chow (1959)	Main channel at full stage; clean, straight, no rifts or deep pools, with some stones and weeds (normal value)	0.035
Alluvial channel/overbank with dense vegetation	Chow (1959)	Channels not maintained; weeds and brush uncut, highest stages of flow (normal value)	0.070
Concrete	Chow (1959)	Concrete; trowel finish (normal value)	0.013

Hydraulic Structure Geometry

Bellota Weir – The Bellota weir was represented within the terrain, rather than as an inline structure element. In HEC-RAS, flow over an inline structure element can be modeled using a weir flow equation. However, the depth of flow over the weir for the 100-year flow event considered in this study is large (approximately 6 feet) and flow is more likely to resemble flow through a channel than over a weir. Four cross-sections were used to represent the existing weir: two cross-sections each to represent the upstream

and downstream extents of the weir (invert elevation = 113.44 feet; Figure 4-2) and the flashboards (invert elevation = 115.44 feet; Figure 4-3), respectively. Cross-section geometry was modified as necessary based as-built drawings (R.B. Welty and Associates 1967).

Figure 4-2. HEC-RAS Cross-Section for Existing Weir (Invert Elevation 113.44 feet)

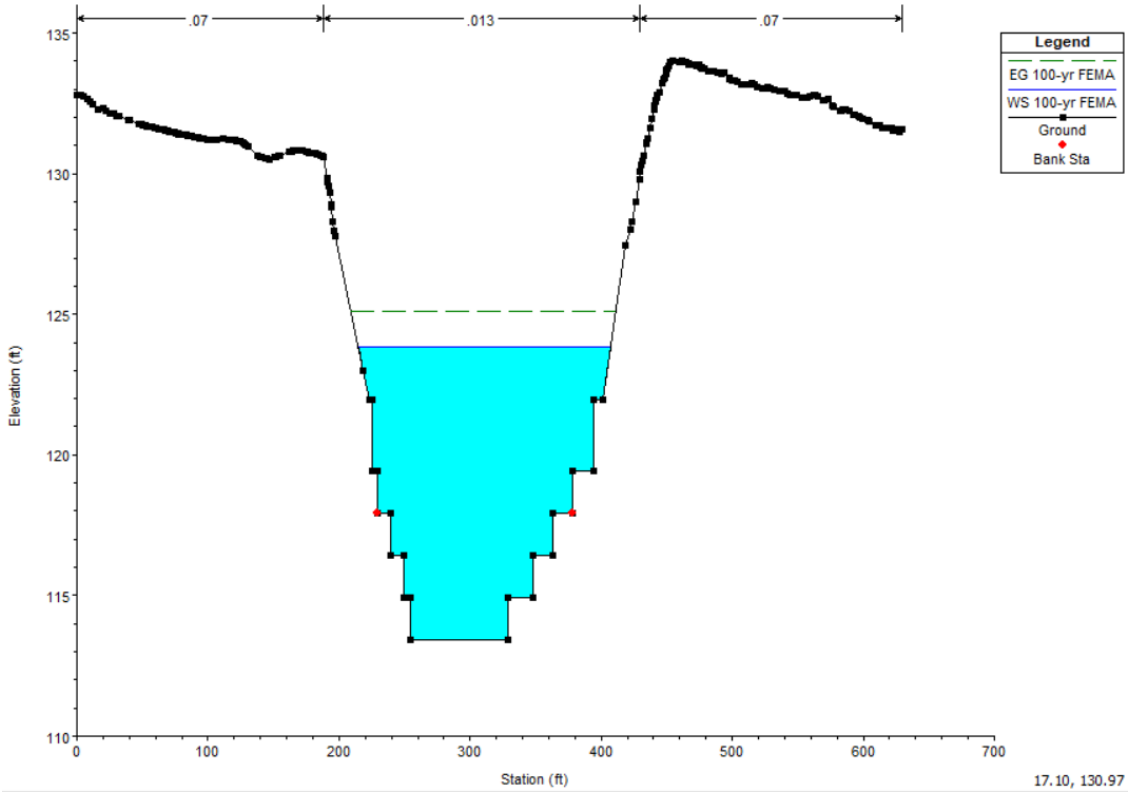
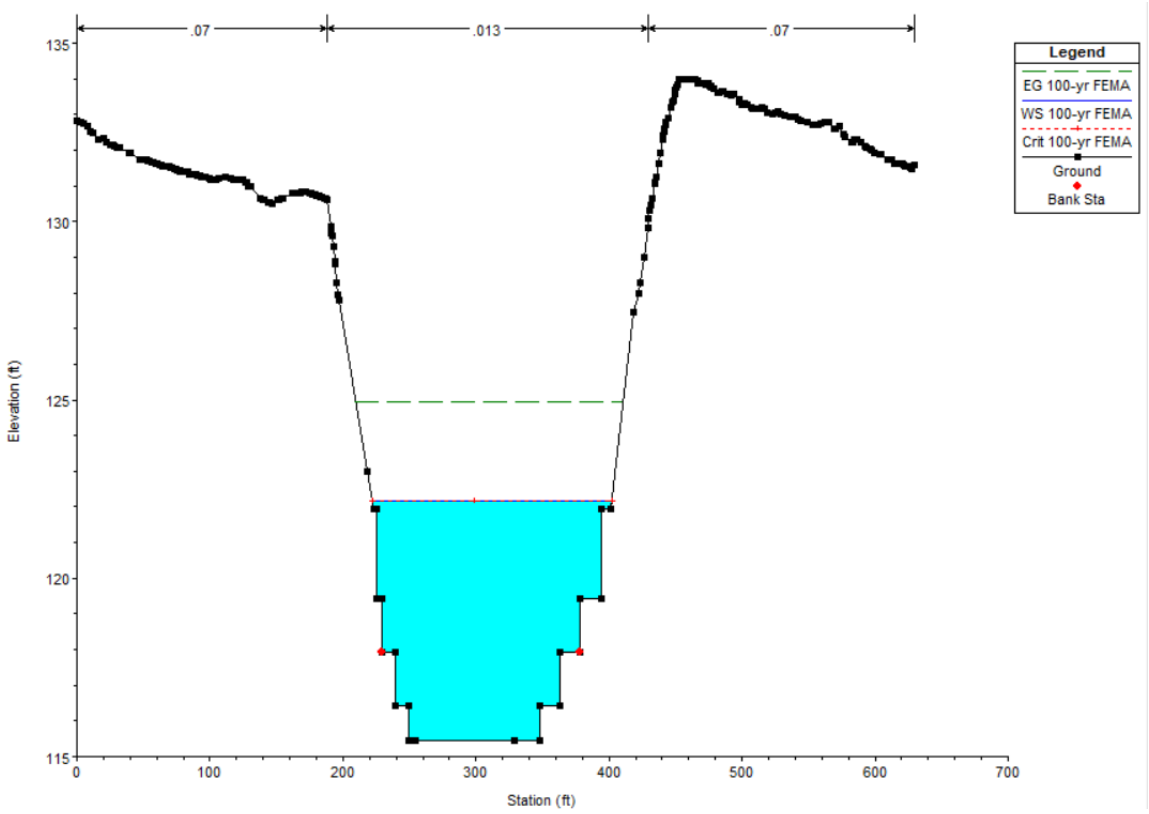


Figure 4-3. HEC-RAS Cross-Section for Existing Weir with Flashboards Implemented (Invert Elevation = 115.44 feet)



Escalon Bellota Road Bridge– Bridge geometry was obtained from the CVFED model. The source of the data used in development of the CVFED bridge geometry is unknown.

Steady Flow Data

Steady flow data is required for the model to compute a water surface profile. Data required includes specification of flow regime, boundary conditions, and channel discharge.

Flow Regime

Hydraulic calculations in the 1D model were performed under the assumption of a subcritical flow regime to provide a conservative WSE.

Boundary Conditions

Boundary conditions provide the information required to estimate the initial WSE used to initiate hydraulic profile calculations. Calculation of a subcritical profile begins at the downstream cross-section; thus, a normal depth boundary condition was set at the downstream cross-section. Normal depth boundary conditions require specification of the energy grade slope at the cross-section. The channel invert slope was used to

approximate energy grade slope; an invert slope of 0.002 feet/feet was estimated at the boundary using the terrain surface.

Channel Discharge

The 100-year FEMA flow value (12,690 cfs; Table 3-1) was the channel discharge specified for the existing conditions model.

4.1.6 Proposed Condition Model

A 1D HEC-RAS model was developed for the proposed facility, including a new weir structure with fish passage ramp and fish ladder.

The proposed condition model is the same as the existing conditions model, with modified geometry and manning's roughness coefficients in cross-sections at the weir and along the proposed roughened channel (cross-section 2375 through 1474):

- Cross-section geometry was updated to reflect the proposed weir and roughened channel dimensions. Assumes all Obermeyer weir gates are at elevation 115.5 feet NAVD88.
- Ineffective flow areas were updated to reflect areas obstructed by the new weir structure, including the fish ladder, the exit for which is likely to be closed under high flow conditions.
- Manning's *n* values were updated to represent the proposed concrete weir and fish passage ramp, riprap-lined slopes, and weir access road:
    - **Fish Passage Ramp** with boulder weirs and rock clusters. Limitations of 1D hydraulic models make it difficult to adequately capture the hydraulic impact of detailed features like boulder weirs and rock clusters, thus, the roughness features on the fish passage ramp were represented by Manning's *n* rather than as features in the terrain. The fish passage ramp was divided into two land cover regions:
        - **Engineered stream bed material (ESM) with boulder weirs** land cover type was assigned to the low flow section of the ramp. This section is comprised of ESM with boulder weirs (maximum 2.5 feet exposed above the low flow channel invert) spaced longitudinally along the fish passage ramp at approximately 8-foot intervals. The boulder weirs generally span the width of the low flow channel. Manning's *n* was calculated using the Limerinos relationship between the 84th percentile bed-material diameter (D_{84}), hydraulic radius, and Manning's *n* described in Equation 5 of USGS (1989). The D_{84} calculated for the engineered streambed material (1.6 feet; Section 5.3.4) was used to estimate of Manning's *n* throughout the boulder weir section of the fish passage ramp.
        - **Fish Passage Ramp, engineered stream bed material with rock clusters** land cover type was assigned to the overbank region of the ramp. This region is comprised of primarily engineered streambed material, with rock clusters

(maximum 1.5 feet exposed above the overbank invert) located at approximately 8-foot intervals longitudinally along the fish passage ramp**.** As with the low flow region, Manning's *n* in the overbank was calculated using the Limerinos relationship between the 84th percentile bed-material diameter (D_{84}) and Manning's *n* described in Equation 5 of USGS (1989). The D_{84} calculated for the portion of the engineered streambed material gradation not used in construction of the boulder weirs (1.2 feet; Section 5.3.4) was used to estimate Manning's *n* throughout the overbank region of the fish passage ramp.

- **Riprap** land cover type was assigned to the riprap-lined banks of the proposed fish passage ramp. Manning's *n* was calculated using the Manning-Strickler equation presented in Chang (1992), which relates D_{50} to Manning's *n*. The maximum D_{50} for 1/2-ton riprap (2.3 feet) based on NCHRP (2006) riprap gradations was used to calculate Manning's *n*.
- **Access road** land cover type was assigned to the proposed foundation rock access road that runs along the north bank of the channel and provides access to the proposed weir. The access road will be comprised of material similar to the riprap slope lining, and, thus, was assigned the same Manning's *n*.

Manning's *n* values used for the roughened ramp and their associated descriptions are provided in Table 4-1.

Table 4-2. Manning's Roughness Values Used for Proposed Condition 1D HEC-RAS Model Only

Land Cover Type	Source	Description	Manning's *n*
Riprap/Access Road	Chang (1992)	Calculated value for 1/2-ton riprap (d_{50} = 2.3 ft) using Manning-Strickler equation.	0.045
Roughened Channel, low flow region – engineered streambed material with boulder weirs (maximum 2.5-foot protrusion from low flow channel invert)	USGS (1989)	Calculated *n* using Limerinos relationship with D_{84} = 1.6 ft based on D_{100} calculated for engineered streambed material.	0.052
Roughened Channel, overbank region– engineered streambed material with rock clusters (maximum 1.5-foot protrusion from overbank invert)	USGS (1989)	Calculated *n* using Limerinos relationship with D_{84} = 1.2 ft based on D_{84} calculated for portion of the engineered streambed material gradation not used in construction of the boulder weirs.	0.048

Figure 4-4 through Figure 4-6 show representative cross-sections for the proposed weir and roughened channel. Cross-sections for existing and proposed conditions are provided in Appendix A.

Figure 4-4. HEC-RAS Cross-Section for Proposed Weir Opening

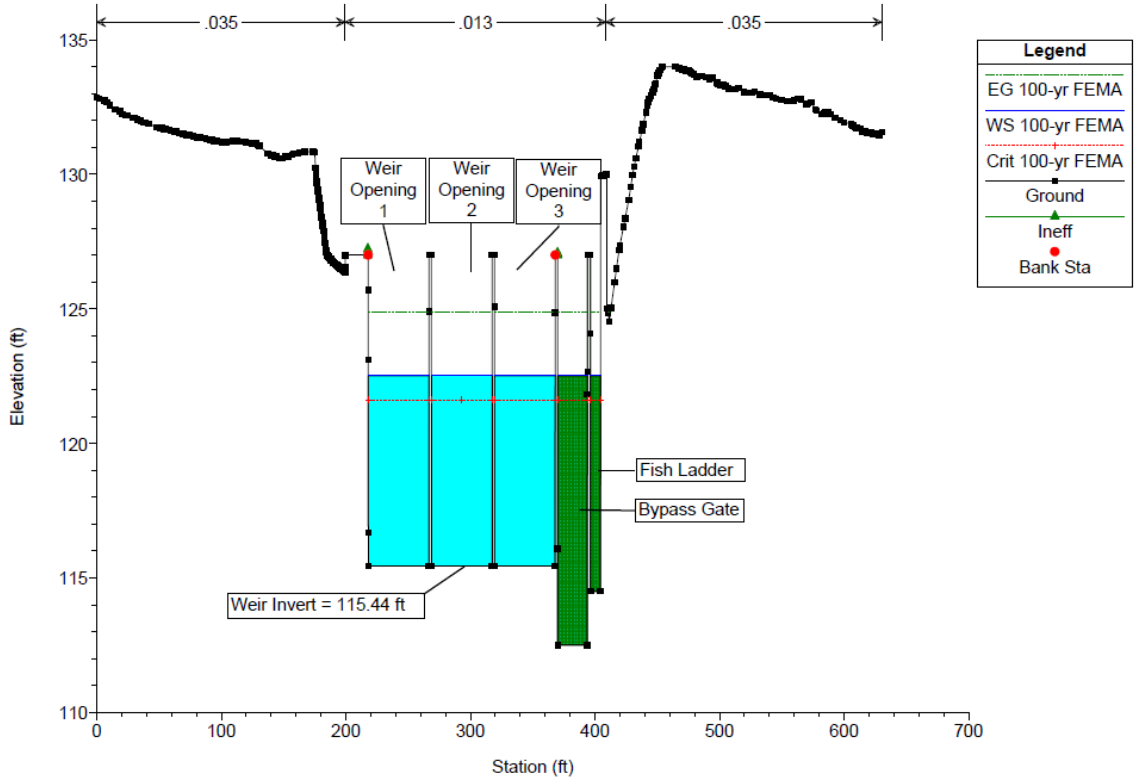


Figure 4-5. HEC-RAS Cross-Section for Proposed Weir Sill

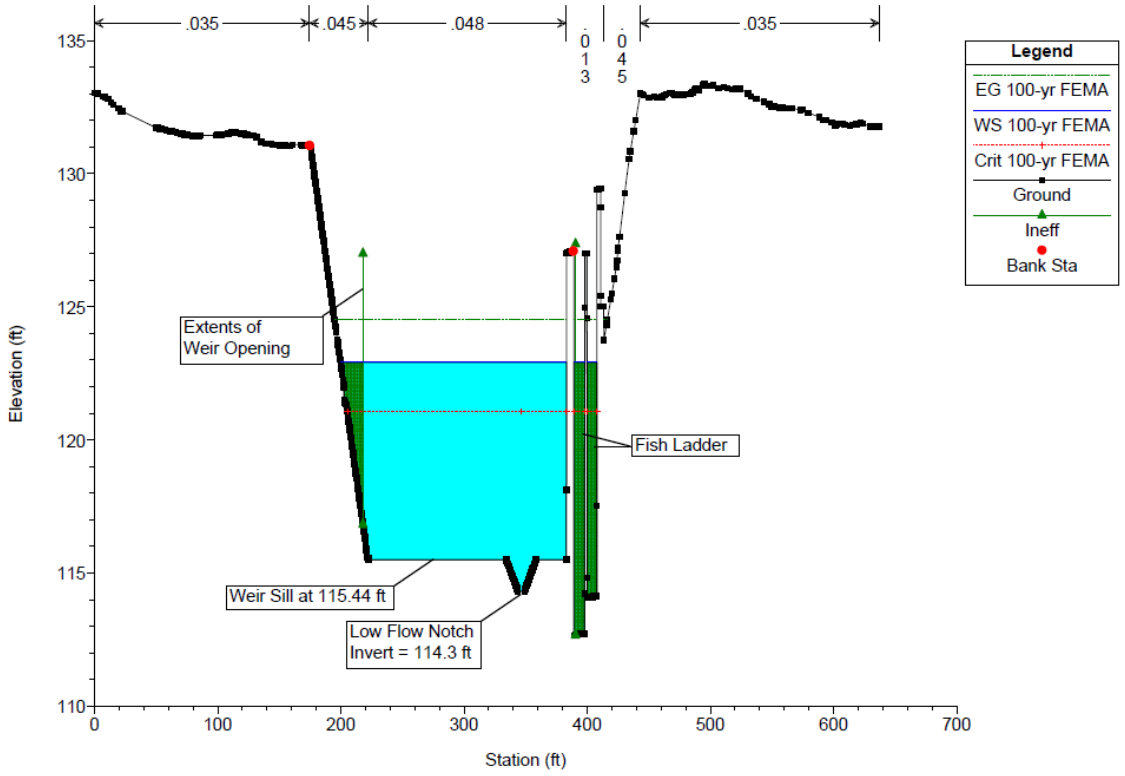
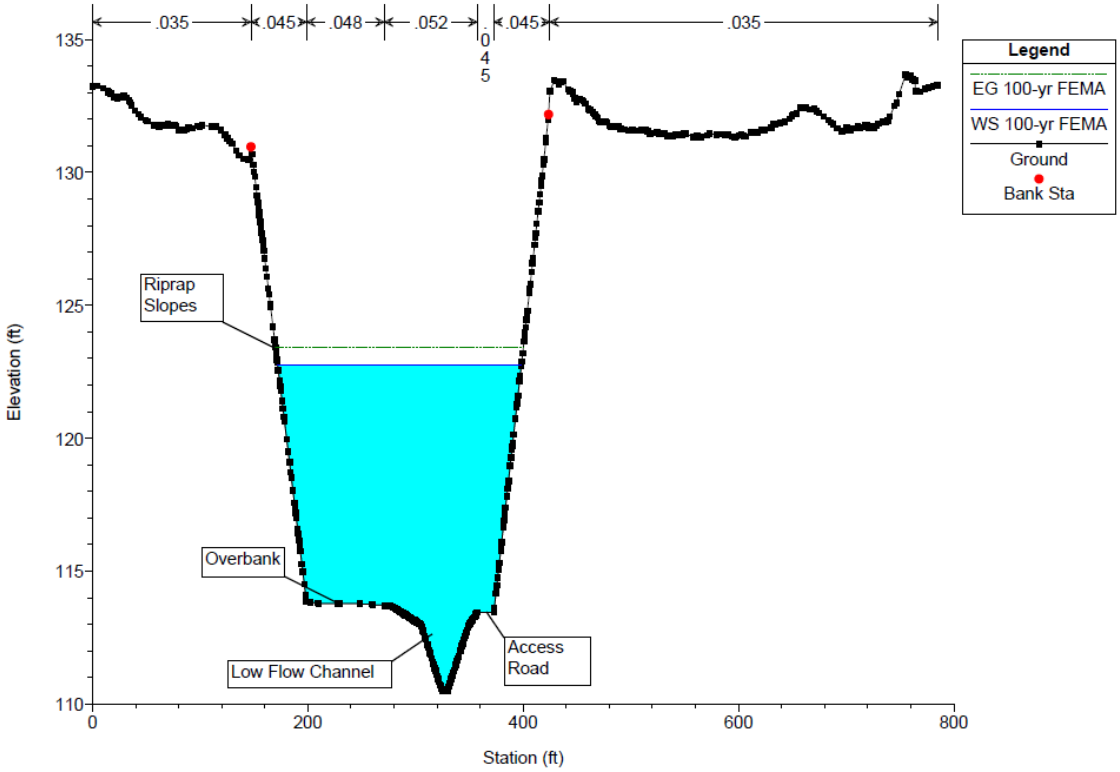


Figure 4-6. Representative HEC-RAS Cross-Section for Proposed Roughened Channel



4.1.7 No-Rise Results

A No-Rise condition is achieved along most of the study reach, except for some localized increases observed around the weir and roughened fish passage ramp. Localized increases were confined to an approximately 590-foot reach extending from the proposed weir (HEC-RAS reach station 2171) to the downstream limit of proposed fish passage ramp (HEC-RAS reach station 1582). Increases in the water surface along the roughened fish passage ramp were generally between 0.01 – 0.3 foot. Larger increases ranging from approximately 0.5 - 1.1 feet were observed through the weir structure and within the upstream 20 feet of the fish passage ramp. The larger increases through the weir are likely due to proposed weir extending approximately 20 feet downstream of the end of the existing weir, resulting in overlap between the dammed water surface upstream of the proposed weir sill and the existing weir tailwater. Localized increases downstream of the weir are a result of the increased tailwater elevation caused by the proposed roughened channel. At all other cross-sections, the proposed water surface is lower than, or equal to the existing water surface profile. Additionally, both the existing and proposed water surface profiles are below the FEMA effective water surface within the portion of the study reach for which FEMA effective base flood elevations were available (Figure 4-7). WSEs from cross-sections at and surrounding the existing and proposed structures are provided in Figure 4-8 and Table 4-3.

Figure 4-7. Existing and Proposed FEMA 100-year Water Surface Profiles and FEMA Effective Water Surface Elevation at FEMA Cross-Sections within the Study Reach

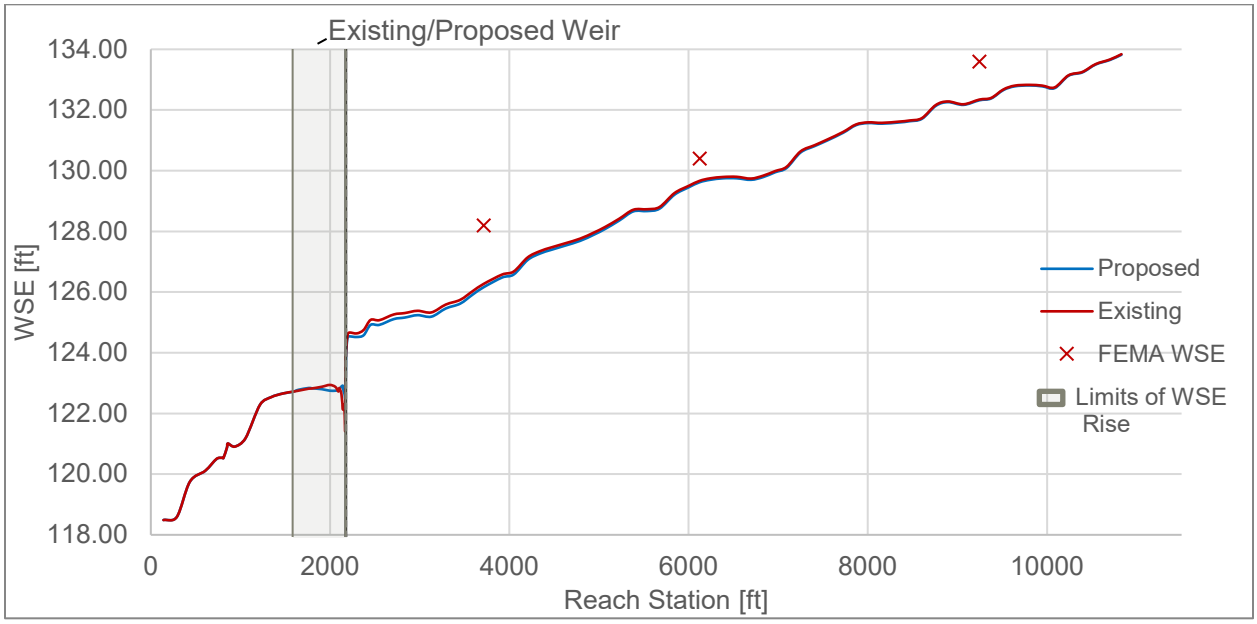


Figure 4-8. Existing and Proposed FEMA 100-year Water Surface Profiles near the Weir

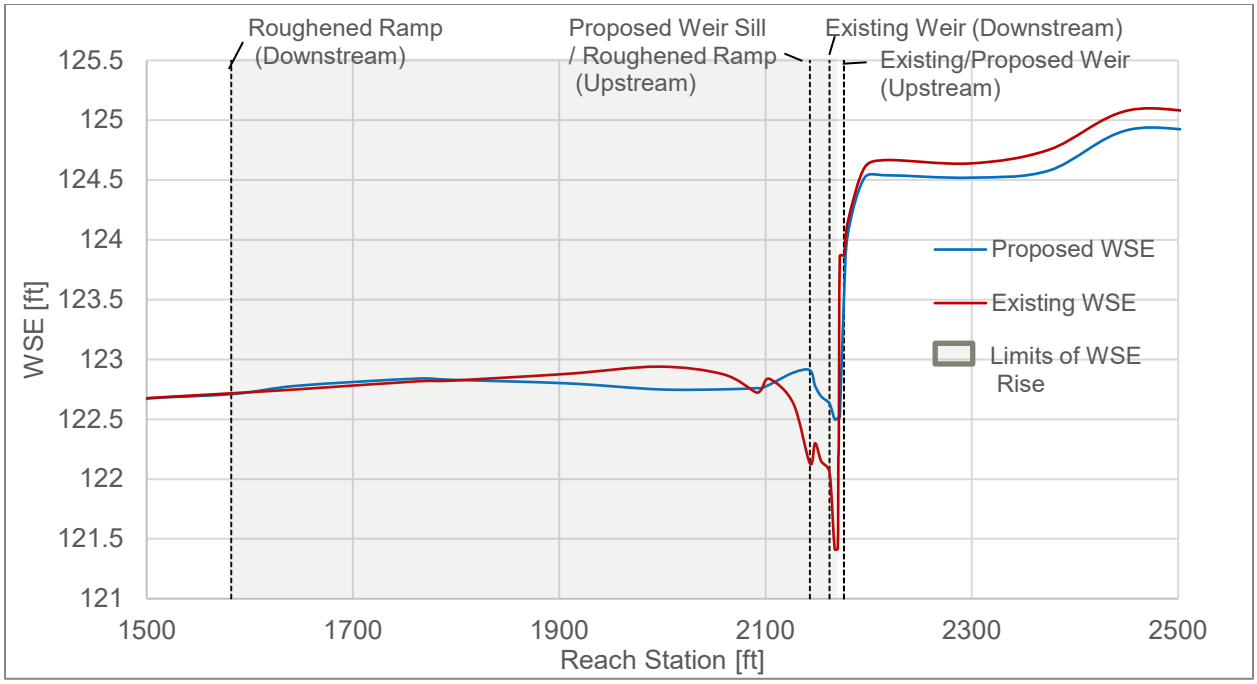


Table 4-3. Hydraulic Analysis Results, Cross-Sections 2550 - 1219

Cross-Section Location	Existing Station (ft)	Existing WSE (ft)	Proposed WSE (ft)	WSE Difference (Proposed - Existing) (ft)
	2550	125.07	124.92	-0.15
	2452	125.08	124.92	-0.16
Upstream Limit of Proposed Channel Modifications	2375	124.75	124.58	-0.17
	2299	124.64	124.52	-0.12
	2217	124.67	124.54	-0.13
	2196	124.61	124.52	-0.09
	2180	124.16	124.06	-0.10
Existing Weir - Upstream; Proposed Weir - Upstream	2176	123.87	123.51	-0.36
	2172	123.87	122.52	-1.35
	2171	122.30	122.51	0.21
	2170.6	122.17	122.51	0.34
	2170	121.42	122.51	1.09
	2167	121.42	122.50	1.08
Existing Weir - Downstream	2162	122.06	122.63	0.57
	2154	122.15	122.69	0.54
	2148	122.30	122.78	0.48
Weir Sill/Upstream limit of Roughened Fish Ramp	2143	122.13	122.91	0.78
	2127	122.63	122.89	0.26
	2104	122.84	122.79	-0.05
	2096	122.75	122.76	0.01
	2091	122.72	122.76	0.04
	2059	122.88	122.75	-0.13
	1996	122.94	122.75	-0.19
	1908	122.88	122.80	-0.08
	1795	122.82	122.83	0.01
	1762	122.82	122.84	0.02
	1646	122.75	122.78	0.03
Downstream Limit of Roughened Fish Ramp	1582	122.72	122.71	-0.01
Downstream Limit of Proposed Channel Modification	1474	122.66	122.66	0.00
	1343	122.54	122.54	0.00
	1219	122.29	122.29	0.00

Notes: Light gray shading indicates a WSE increase from existing condition.

4.1.8 No-Rise Discussion

Results from the 1D analyses indicate that the water surface for the FEMA 100-year flood event are lower than the FEMA effective base flood elevations (Appendix C). A No-Rise condition is generally achieved along the entire study reach, except for a few localized increases in WSE at the proposed weir and along the proposed fish passage ramp. The water surface was not affected downstream of the proposed channel modifications, and the water surface upstream of the proposed weir was lower under proposed conditions than under existing conditions. Within the reach that experienced an increase in the 100-year water surface under proposed conditions, the proposed WSE was contained within the channel and did not exceed the vertical extents of the riprap lining. Furthermore, there are no insurable structures at these locations and localized increases in water surface are unlikely to pose increased flooding risk along the study reach. A Conditional Letter of Map Revision may be required if the San Joaquin County Floodplain Administrator considers the localized increases in WSE within the Project area to be unacceptable. Hydraulic results will be refined as the design progresses.

4.2 Two-Dimensional Hydraulic Analysis

A 2D hydraulic analysis was performed for the proposed condition using the USACE's HEC-RAS version 6.1. Results from the 2D analysis were used to inform design, including evaluation of fish passage performance, and design of the rock material that will line the slopes of and serve as the foundation for the fish passage ramp. The 2D hydraulic model consisted of several components:

- A model domain to define the area for which hydraulic calculations are performed
- A computational mesh to define the hydraulic properties and spatial resolution of calculations within the model domain
- Boundary conditions to provide the necessary information to initiate hydraulic calculations across the computational mesh

4.2.1 Model Domain

The model domain extends from approximately 0.4 mile upstream of the Bellota Weir to approximately 360 feet downstream of the Escalon Bellota Road Bridge (0.3 mile downstream of the Bellota Weir). The surrounding area is comprised primarily of agricultural land use. Exhibit B5, in Appendix B, shows the boundary of the model domain.

All elements within the HEC-RAS 2D model domain were georeferenced to the North American Datum of 1983, California State Plane coordinate system, Zone III. Vertical data is referenced to NAVD88. Horizontal and vertical units are in U.S. survey feet.

4.2.2 Computational Mesh

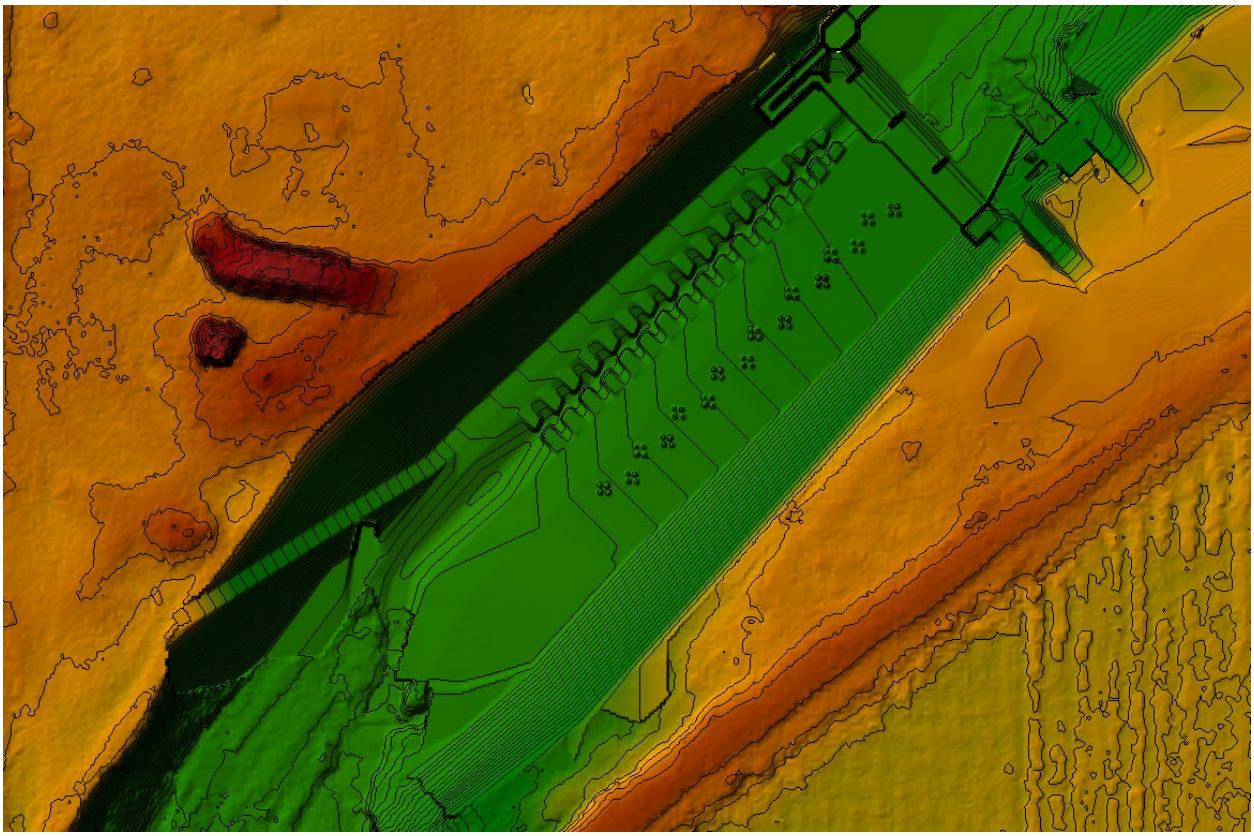
HEC-RAS 6.1 uses a finite-volume scheme to numerically solve the Full Momentum Saint-Venant Equations for flow depths and velocities across an irregular computational

grid (mesh). The mesh divides the model domain into discrete areas (cells) for which hydraulic properties based on terrain and Manning's roughness data are calculated and hydraulic computations performed. A flow depth is calculated at the center of each cell, while flow velocities are calculated at each cell face. The grid size (resolution) of the mesh defines the spatial (horizontal) interval at which calculations are performed. A 20-foot mesh resolution was generally used within the channel; resolution was reduced to 5 feet near the weir (approximately 50 feet upstream). A mesh resolution of 3 to 5 feet was used to represent features at the weir and along the fish passage ramp. Mesh cells were oriented to align with the general direction of flow and define significant features (e.g., channel banks, weir gates, weir sill, fish ladder, boulder weirs, rock clusters). Computations were performed at a variable time-step (maximum time-step = 4 seconds; minimum time-step = 0.063 second) governed by the Courant number, which is the ratio of the computational time-step to the flow residence time for a given computational grid-cell. Exhibit B6, in Appendix B, shows the computation mesh within the Project area.

Terrain

Terrain data defines the elevation of each mesh cell and cross-sectional geometry at each cell face. This information is used to calculate relationships between depth and the area and volume at each cell face. The existing conditions terrain developed for the 1D HEC-RAS model (see Section 4.1.5) was used as the base terrain for the 2D HEC-RAS model. A separate terrain was developed for the proposed weir and fish passage features and layered on top of the existing terrain to create a composite proposed terrain for the 2D model domain. Unlike with the 1D HEC-RAS model, the weir boulders and rock clusters were represented as terrain features. Boulder weirs were represented as simple blocks of raised terrain rather than as individual boulders to produce the desired hydraulic impact of the weirs without needing to determine currently unknown details such as number of boulders per weir, lateral boulder spacing, gradation of materials between boulder spaces. Figure 4-9 shows the terrain developed for the proposed weir and fish passage features used in the 2D hydraulic analysis.

Figure 4-9. Proposed Terrain Developed for the Project Area



Manning's *n* values

Manning's *n* values associated with land cover are used to calculate friction losses throughout the model domain. The same land cover types used for the 1D proposed HEC-RAS model were used for the 2D model; however the Manning's *n* values associated with each land cover type were adjusted to accommodate for energy loss caused by turbulence that is accounted for in the 2D hydraulic calculations, and to produce sufficiently conservative velocities and depths for riprap sizing and evaluation of fish passage performance. The following adjustments were made to Manning's *n* values in the 2D model:

- **Existing channel and overbanks** were assigned Manning's *n* values from the 1D model; however, Manning's *n* values were lowered to the minimum, and average of minimum and normal values for the same channel type from Chow (1959).
- **Concrete Weir** was assigned the lowest Manning's *n* value listed for Lined or Constructed Channels in Chow (1959).
- **Fish Passage Ramp, engineered stream bed material with boulder weirs/rock clusters** were assigned Manning's *n* values based on a Depth-Manning's *n* rating curve developed by NHC (2020) for a similar structure designed as part of the Nelson Dam Removal Project. NHC (2020) calibrated the rating curve based on the results of a physical model and 2D numerical hydraulic model; it is applicable for the

engineered streambed material (without boulder weirs or rock clusters) at depths from 0 to 10 feet. Roughness along the fish passage ramp was divided into the same two regions used for the proposed 1D HEC-RAS model. Due to significant longitudinal variation in depth along the fish passage ramp, the depth halfway down the fish passage ramp (approximately 200 feet downstream of the weir sill) was selected as the representative depth to determine Manning's *n* for each roughness region. The lowest Manning's *n* provided by the rating curve (*n* = 0.035) was assigned across the entire fish passage ramp for scenarios used in design of foundation rock and riprap (100-year, 50-year, 25-year, and 10-year discharges) to produce conservative velocities. The boulder weirs and rock clusters impact on energy loss was assumed to be accounted for in the 2D hydraulic calculations.

A summary of the Manning's roughness coefficients used for this analysis is provided in Table 4-4. Table 4-5 provides the rating curve developed by NHC (2020) for the Nelson Dam Removal Project.

Table 4-4. Manning's Roughness Values Used for 2D Proposed Condition Model

Land Cover Type	Source	Description	Manning's *n*
Channel, earthen with minor/moderate vegetation	Chow (1959)	Main channel at full stage; clean, straight, no rifts or deep pools, with some stones and weeds (minimum value)	0.025
Channel and overbanks, heavy brush/grass/trees	Chow (1959)	Excavated or Dredged Channels: not maintained; weeds and brush uncut, clean bottom with brush on sides, highest stages of flow (average of minimum and normal value)	0.060
Concrete	Chow (1959)	Minimum value for any lined channel	0.010
Riprap	Chang (1992)	Calculated value for 1/2-ton riprap (dumped rock, d_{50} = 2.3 feet) using Manning-Strickler equation.	0.045
Roughened Channel – Boulder Weirs with maximum 2.5 feet protrusion from channel bed	NHC (2020)	Depth-Manning's rating curve developed using a physical model for a structure similar to the Bellota Weir Fish Passage ramp	0.035 – 0.060
Roughened Channel – Rock Clusters with maximum 1.5 feet protrusion from channel bed	NHC (2020)	Depth-Manning's rating curve developed using a physical model for a structure similar to the Bellota Weir Fish Passage ramp	0.035 – 0.060

Table 4-5. Depth-Manning's *n* Rating Curve developed by NHC (2020) for the Nelson Dam Removal Project

Depth (ft)	Manning's *n*
< 1.0	0.060
2.0	0.054
4.5	0.044
7.0	0.037
10.0	0.035

Table 4-6. Manning's Roughness Values for the Roughened Ramp based on NHC (2020) depth-varied Manning's *n* rating curve

Scenario	Discharge (cfs)	ESM with Boulder Weirs		ESM with Rock Clusters	
		Representative Depth (feet)	Manning's *n*	Representative Depth (feet)	Manning's *n*
FEMA 100-year	12,690	>10	0.035	8.0	0.035
FEMA 50-year	12,500	>10	0.035	8.0	0.035
USACE 25-year	10,640	9.0	0.035	6.5	0.035
USACE 10-year	9,530	8.5	0.035	6.0	0.035
High Fish Passage (migration)	1,735.2	4.5	0.041	1.2	0.054
5% Annual Exceedance	545.3	3.5	0.046	<1.0	0.060
High Fish Passage (irrigation)	150.9	3.0	0.049	<1.0	0.060
Low Fish Passage	20	<1.0	0.060	<1.0	0.060

ESM = engineered streambed material

4.2.3 Boundary Conditions

The finite-volume scheme used to solve for flow depths and velocities requires user-defined boundary conditions (i.e., hydrograph, normal depth) at the upstream and downstream boundaries of the model domain to initiate iterations and produce a final solution. A discharge hydrograph condition was selected at the upstream boundary, which extended from the high point on the southern channel bank to the high point on the eastern channel bank at the upstream end of the model domain. Discharge was distributed across the upstream boundary based on conveyance. A normal depth condition was selected at the downstream boundary, which extended from the high point on the southern channel bank to the high point on the northern channel bank at the downstream end of the model domain.

Discharge Data

Throughout the design process, numerous flow scenarios were considered for the fish passage ramp. Design flows regarding design of the low flow channel and weir sill, and fish passage ramp foundation rock and riprap are described in Section 5.2.

Normal Depth

HEC-RAS requires a user-defined energy slope to calculate normal depth for a given discharge. The energy slope at the downstream boundary was approximated as the slope of the channel invert. An average invert slope of 0.003 feet/feet was estimated from an elevation profile extracted from the terrain at the downstream boundary.

5 Roughened Channel Hydraulic Design

5.1 Design Overview

Hydraulic modeling results were used to design the proposed roughened channel. Through an iterative process, the slope, shape, and configuration of the roughened channel will be developed to meet the Project fish passage hydraulic objectives. The roughened channel is also integrated with adjacent Project features and proposed improvements to the channel area. During 35 percent design, the roughened channel sill and low flow channel were configured using normal depth calculation and 1D modeling. The initial slope, shape, and configuration were confirmed and iterated with 2D modeling for 65 percent design. Design parameters for the roughened channel foundation material and slope protection were also developed using 2D hydraulic modeling results for 65 percent design. For 90 percent design, foundation rock and slope protection design were refined, and a gradation was developed for the engineered streambed material layer based on 2D hydraulic modeling results revised to reflect an updated representation of the boulder weir geometry.

The roughened channel fishway and channel spanning weir crest was designed to conform to several performance and engineering criteria. Engineering criteria target the overall stability and resilience of the structure. Performance criteria target the biological performance and periodic costs associated with maximum acceptable damages that require modification or repairs over time. The various criteria and performance targets used to establish the overall design of this feature are detailed in the Draft Design Criteria Report (HDR 2021). Below are some of the key criteria:

- The structures (weir, channels, and high flow sections) will be designed to withstand, and provide sufficient conveyance for flood events up to the 100-year flood event (12,690 cfs).
- Maintain the minimum operational water surface of 115.4 feet upstream of the sill for the low flow discharge during irrigation season (HDR 2021).

- Limited damage that may occur during low frequency events (high flows) will not impede the structures' ability to achieve fish passage or surface water withdrawal performance criteria.
- Damage will be limited to surface scour of rock matrix, including loss of fine materials, and minor degradation of tailwater control (i.e., facility will be designed such that periodic adjustments in surface composition do not inhibit irrigation supply or fish passage, or negatively affect the Escalon Bellota Road Bridge).
- Dislodgment of large structural rocks will be limited to small areas where storm events create conditions that cause isolated areas of instability.
- Accumulation and removal of finer sediments and cobbles on the surface layer above the armor layer will be transitional and vary over time.
- The structure configuration and composition will be designed to meet the Project's fish passage performance objectives.

Fish passage performance objectives of the roughened channel will be continuous and met throughout the range of anticipated migration flows identified for various target species and life stages. Specific fish passage design criteria include:

- Provide a minimum depth of 0.9 feet for the low fish passage design discharge through the high fish passage design discharge (HDR 2021).
- Provide adequate velocity refugia for the high fish passage design discharge (HDR 2021).

5.2 Roughened Channel Low Flow and Concrete Sill Design

The low flow channel of the roughened ramp was designed to meet fish passage and operational hydraulic criteria. The low flow channel is characterized by exposed structural rock, engineered streambed material, boulder clusters, and natural debris deposits. The low flow notch at the concrete sill was designed with the same geometry as the low flow channel. The concrete sill at the upstream end of the roughened channel serves to retain the roughened channel material and provide a minimum depth of 24 inches above the adjacent weir crest. The water surface through the notch during low flows is controlled by the low flow channel and boulder weirs.

5.2.1 Design Discharges

A range of discharges were used to evaluate the proposed design as it relates to fish passage and operational criteria. As previously mentioned, fish passage design discharges were defined per NMFS (2011) guidelines (Section 3.1.3). However, given the range of fish passage design discharges and the magnitude of the flood discharges, a roughened ramp is not suited to meet fish passage hydraulic criteria at flows as low as 3 cfs. For the 65 percent design, the low flow section was sized to meet fish passage criteria for 20 cfs. According to SEWD, the irrigation season low flow is also 20 cfs (SEWD, KSN, and HDR 2020). Therefore, the sill, roughened ramp, and low flow

channel were configured to meet design criteria for discharges from 20 cfs to 1,735.2 cfs (Table 5-1).

Table 5-1. Roughened Channel, Low Flow Channel, and Notch Design Discharges

Scenario	Discharge (cfs)
High Fish Passage (migration season)	1,735.2
5% Annual Exceedance	545.3
High Fish Passage (irrigation season)	150.9
Low Fish Passage (irrigation season low flow)	20

5.2.2 Final Configuration

Multiple sill, low flow channel, and boulder weir configurations were tested using the 2D HEC-RAS model (Section 4.2) to meet the hydraulic criteria. The final configuration includes a notch depth of 1.2 feet (invert elevation = 114.3 feet), a notch/low flow channel bottom width of 5 feet, and notch/low flow channel side slopes of 8H:1V (Figure 5-1). The roughened channel and low flow channel extend approximately 17 feet downstream from the sill at a constant elevation of 114.3 feet prior to sloping downstream at an approximate 3 percent slope. This initial horizontal portion of the roughened channel provides a protective buffer against degradation at the downstream end of the weir by distancing the invert grade break beyond the sill transition. Weir boulders are located longitudinally along the both sides of the low flow channel at approximate 8-foot intervals. The weirs on the left and right side of the low flow channel are spaced to create an effective low flow bottom width of 3 feet; the weirs are laterally staggered for the 3-foot opening alternate between the left and right sides of the low flow channel invert. Figure 5-2 depicts the water surface at the sill for fish passage design discharges. Appendix D provides the HEC-RAS 2D velocities where fish passage depth criteria is met within the Project area for all design discharges.

Figure 5-1. Typical Roughened Channel Section with Weir Boulders

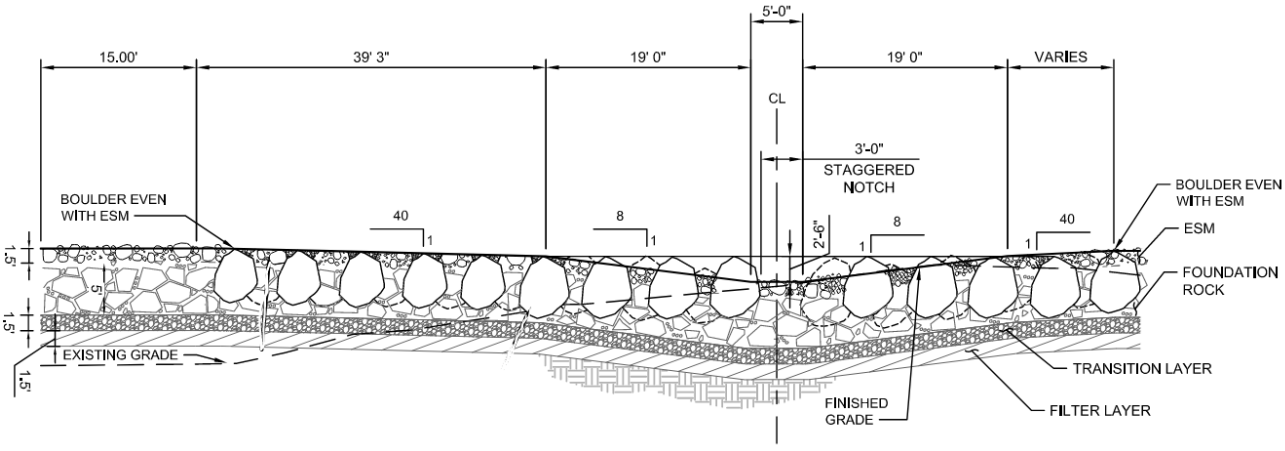
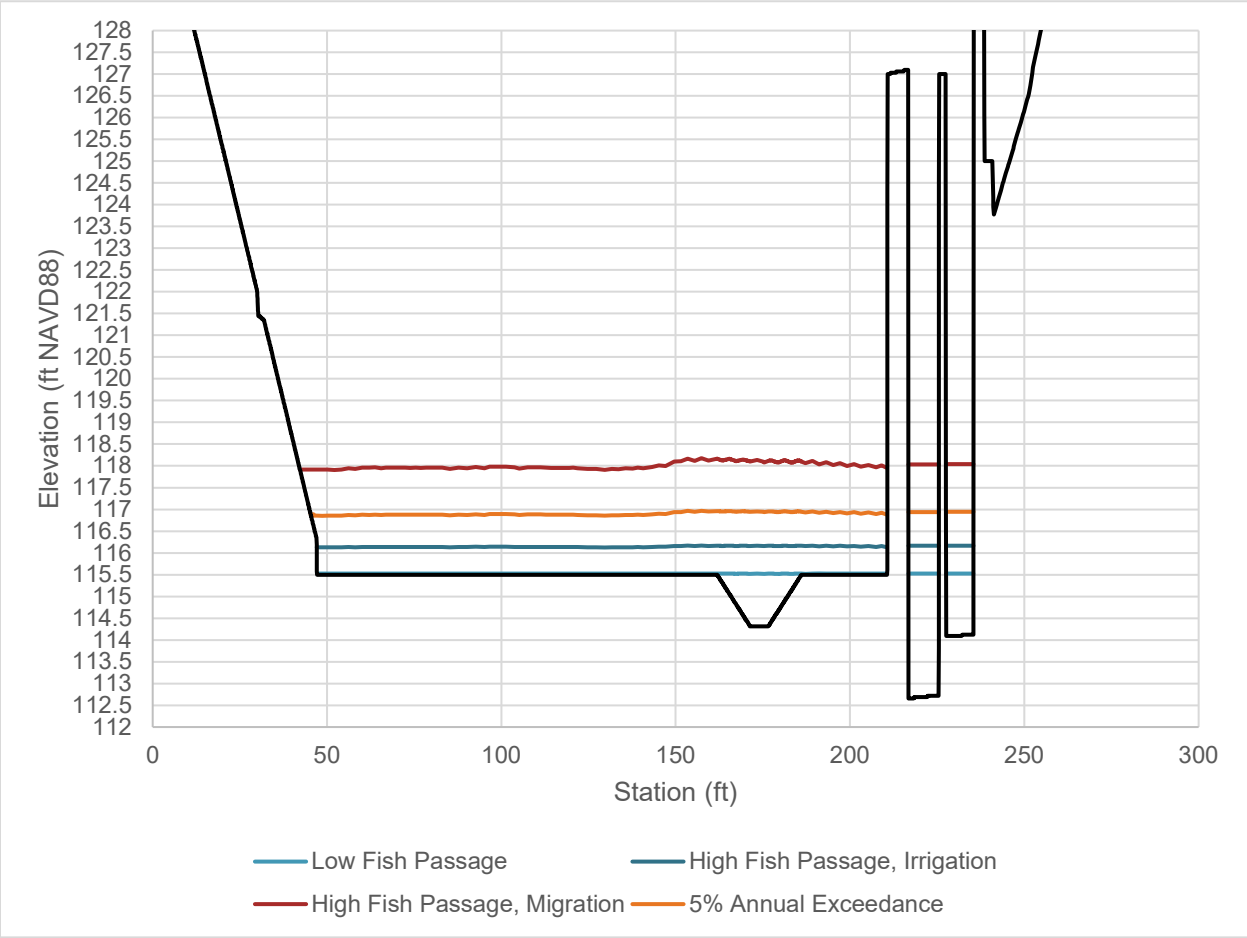


Figure 5-2. HEC-RAS 2D Water Surface for Fish Passage Design Flows at Roughened Channel Sill



5.3 Rock Material Sizing

The roughened channel slope, shape, and configuration must remain intact to function as intended. If the material comprising the roughened ramp is displaced or mobilized in an unintended manner, the hydraulic performance may be degraded and require maintenance beyond that identified as acceptable in Section 5.1. There are three primary mechanisms for degradation of the roughened ramp. First, the foundation rock, which constitutes the primary material of the channel, is displaced from the surface due to high flow hydraulics. This can affect the shape, configuration, and grade. Second, the underlaying layer, consisting of the natural streambed or placed fill, can be degraded by the winnowing or piping of fines up through the structural rock. This loss of substrate can lead to settling of the roughened channel, affecting the shape and grade. Finally, degradation at the downstream end, due to a variety of potential scour mechanisms, could cause mobilization of the roughened channel material and/or result in a perched condition not conducive to swim-through migration.

To guard against these three degradation mechanisms and support desired passage hydraulics, the typical roughened channel section was designed to include three individual layers installed above the existing riverbed material: a rock filter layer, structural foundation rock layer, and mobile bed layer composed of rounded river cobbles (engineered streambed material). Additionally, the roughened channel embankments will be protected with riprap to prevent degradation of the side slope.

Foundation rock and embankment protection sizing calculations are provided in Appendix E.

5.3.1 Design Discharges

The proposed fish passage ramp structures should be designed to withstand the 100-year discharge event. However, due to large variation in the distribution of flows across the fish passage ramp between the 100-year discharge and more frequent events, the 50-, 25-, and 10-year discharges were also evaluated to verify that the most conservative hydraulic variables are used to design the foundation and slope protection. Design discharges are summarized in Table 5-2.

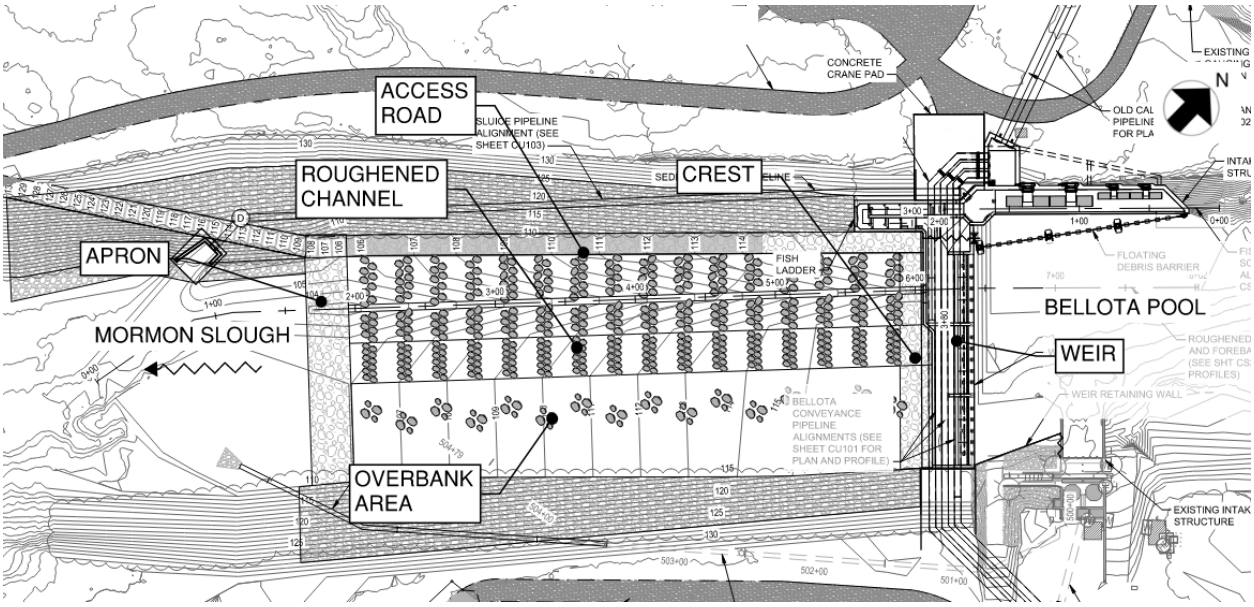
Table 5-2. Foundation Rock and Embankment Protection Design Discharges

Scenario	Discharge (cfs)
FEMA 100-year	12,690
FEMA 50-year	12,500
USACE 25-year	10,640
USACE 10-year	9,530

5.3.2 Fish Passage Ramp Foundation Rock

Results from the 2D HEC-RAS model were used to size the foundation rock for the fish passage ramp, including the crest, main channel, overbank, access road, and apron. Figure 5-3 shows a plan view of the different rock sizing zones.

Figure 5-3. Roughened Channel Foundation Rock Layer Zones



Approach

Two methodologies are recommended in the United States Bureau of Reclamation's *Rock Ramp Design Guidelines* (Reclamation 2007) for sizing riprap on rock ramps or, in this case, the foundation rock for the fish passage ramp:

- Steep Slope Riprap Design (USACE 1991)
- Abt and Johnson (1991)

All design flow scenarios were evaluated using both of the above rock sizing methods.

USACE (1991)

The USACE (1991) method for sizing riprap on steep slopes relates the bed slope and unit discharge to rock size using the following equation:

$$D_{30} = \frac{1.95\, S^{0.555} q^{2/3}}{g^{1/3}}$$

Where,

D_{30} = rock diameter for which 30 percent is smaller by mass; ft

S = slope of bed; ft/ft

q = design unit discharge; cfs/ft (recommended to multiply by safety factor of 1.25 to account for flow channelization)

g = acceleration due to gravity; ft/s^2

The equation is applicable for angular rock with a unit weight of 167 lbs/ft^3 , D_{85}/D_{15} ranging from 1.7 to 2.7, revetment thickness equal to 1.5*D_{100}, bed slopes ranging from 2 to 20 percent, and no tailwater.

Abt and Johnson (1991)

The Abt and Johnson (1991) method for sizing ramp bed material is based on empirical relationships developed from flume studies. Similar to USACE (1991) the method relates bed slope and unit discharge to rock size, but also includes additional coefficients to account for uncertainties related to regression fit, flow concentration, and material type:

$$D_{50} = \phi_e \cdot \phi_c \cdot a \cdot 5.23 \cdot S_0^{0.43} \cdot q_{design}^{0.56}$$

Where,

D_{50} = median diameter of the riprap layer; ft

ϕ_e = coefficient for the empirical envelope on the regression relationship (equal to 1.2)

ϕ_c = coefficient for flow concentration due to channelization within the revetment; equal to $r_q^{0.56}$, where r_q is the ratio of unit discharge in a channel versus uniform depth unit discharge (maximum value equal to 3)

a = shape factor for rounded material (equal to 1 for angular material)

S_0 = profile slope of the rock ramp; ft/ft

q_{design} = design unit discharge; cfs/ft (recommended to multiply by safety factor of 1.35)

The flume experiments used to develop the equation included bed slopes ranging from 1 to 20 percent, with stones having coefficient of uniformity ($C_u = D_{60}/D_{10}$) ranging from 1.62 to 2.15, and geometric standard deviations ($\sigma_g = (D_{84}/D_{16})^{0.5}$) ranging from 1.86 to 5.70.

Application

The primary inputs required for both of the methods described above are unit discharge and slope.

Unit discharges were estimated at 10 cross-sections along the fish passage ramp profile. The total discharge at each cross-section along the fish passage ramp invert was obtained from the 2D HEC-RAS model results. Unit discharge was then calculated by dividing the total discharges by the width of the fish passage ramp invert. The maximum unit discharge across all 10 locations was then selected for calculation of rock material size; unit discharge was multiplied by the recommended safety factors for their respective equations. Additionally, the maximum channelization factor ($r_q = 3$) was applied for the Abt and Johnson (1991) method to account for a significant preferential pathway down the center of the ramp during high flows (Appendix D provides 2D HEC-RAS results for design discharges).

The fish passage ramp has a bed slope of approximately 3 percent; however, a backwater condition exists at the downstream end of the fish passage ramp that limits the influence of the bed slope on hydraulics over the ramp. Because both rock sizing

methods were developed under the assumption of relatively uniform flow along the profile with no backwater, energy slope was considered to be an appropriate substitute for bed slope. Energy slope was determined for eight profiles across the fish passage ramp using water surface and velocity results produced by the 2D HEC-RAS model. The maximum energy slope across all eight profiles was used for the rock sizing calculations.

This method of estimating inputs for the rock sizing equations was repeated for all design discharges.

Results

The 100-year discharge event and the Abt and Johnson (1991) method produced the most conservative D_{50} (1.8 feet) for the fish passage ramp foundation rock. Although unit discharge was generally lower near upstream end of the ramp and would allow for a smaller rock size, the maximum rock size was conservatively selected for the entire ramp (main channel, overbank, crest, access road, apron). Rock gradations were selected based on NCHRP (2006) gradations. The United States Bureau of Reclamation (2007) recommends that the thickness of the foundation rock layer be calculated as the maximum of D_{100} and $1.5*D_{50}$. The resulting thickness given the selected riprap class would be 42 inches (3.5 feet). However, a conservative thickness of 60 inches (5 feet) was selected for the main channel, overbank, crest, and access road to ensure sufficient depth for boulder embedment. For the apron, a thickness of 108 inches (9 feet) was selected to account for increased erosion potential at the downstream end of the ramp, per the long-term degradation analysis (Section 6).

Table 5-3 provides a summary of the selected D_{50} and associated NCHRP (2006) gradation.

Table 5-3. Proposed Foundation Rock Gradation and Layer Thickness

Sub-Region	Nom. D50 (in)	D15 (in)		D50 (in)		D85 (in)		D100 (in)	Blanket Thickness (in)	Weight Class	Description
		Min	Max	Min	Max	Min	Max				
Access Road, Crest, Channel, Overbank	21	13	18.5	20	24	27.5	32.5	42	60	3/8-ton	NCHRP Class VI
Apron	21	13	18.5	20	24	27.5	32.5	42	108	3/8-ton	NCHRP Class VI

5.3.3 Embankment Protection

Approach

Riprap revetment protection of the channel embankments was sized and designed according to the USACE Engineering Manual 1110-2-1601, Hydraulic Design of Flood Control Channels (1994). This method is specifically used to size rock along a channel embankment, where the embankment side slope is considered. The USACE (1994) method for sizing riprap equates stone size with local velocity using empirical relationships using the following equation:

$$D_{30} = S_f C_s C_V C_T d \left[\left(\frac{\gamma_w}{\gamma_s - \gamma_w} \right)^{1/2} \frac{V}{\sqrt{K_1 g d}} \right]^{2.5}$$

Where,

D_{30} = riprap size for which 30 percent is smaller by weight, length

S_f = safety factor (1.25); ft/ft

C_s = stability coefficient for incipient failure (0.30 for angular rock); ft/ft

C_V = vertical velocity distribution coefficient (1.0 for straight channels)

C_T = thickness coefficient (1.0 for thickness = $1D_{100}$ (max) or $1.5D_{50}$ (max)

d = local depth of flow; ft

γ_w = unit weight of water; lbs/ft^2

γ_s = unit weight of riprap; lbs/ft^2

V = local depth averaged velocity; ft/sec

K_1 = side slope correction factor

g = gravitational constant; ft/s^2

Application

The primary input is the local depth average velocity. Similar to the approach for the foundation rock, velocity was evaluated in distinct channel zones, with the right and left channel embankments identified separately. The highest velocity result within these embankment zones was identified for the 100-year and more frequent events. Under the proposed channel configuration, the 100-year event produced the highest velocities, with a peak velocity of 11.7 ft/sec. Using the highest velocity provides a conservative rock size to accommodate changes to the local hydraulic condition during a large flood event. Angular rock is assumed for increased stability on the embankment slopes, which proposes 3H:1V.

Results

The NCHRP Class V riprap gradation was selected for design based on the rock size computations (Table 5-4). This gradation is recommended for both channel embankments bounding the roughened channel area. The recommended revetment thickness on slope is typically the greater of D_{100} or $1.5D_{50}$, which is 36 inches in this instance. The embankment revetments are to be tied into the foundation rock of the roughened channel. As the foundation rock gradation is larger in size, the embankment rock can terminate at the surface of the roughened channel. The roughened channel rock will span channel bank to channel bank.

Table 5-4. Proposed Embankment Rock Gradation and Layer Thickness

Location	Nom. D50 (in)	D15 (in)		D50 (in)		D85 (in)		D100 (in)	Blanket Thickness (in)	Weight Class	Description
		Min	**Max**	**Min**	**Max**	**Min**	**Max**				
Channel Embankments	18	11	15.5	17	20.5	23.5	27.5	36	26	1/4-ton	NCHRP Class V

5.3.4 Engineered Streambed Material

Approach

The engineered streambed material (ESM) was designed based on the D_{30} calculated using the Steep Slope Riprap Design equation (USACE 1991) discussed in Section 5.3.2. The resulting D_{30} (12.8 inches; 1.1 feet) was then used to determine an ESM gradation using the following relationships recommended by the California Department of Fish and Game (2009):

$$D_{84\text{-}ESM} = 1.5\ D_{30\text{-}USACE}$$

$$D_{50\text{-}ESM} = 0.4\ D_{84\text{-}ESM}$$

$$D_{100\text{-}ESM} = 2.5\ D_{84\text{-}ESM}$$

$$D_{16\text{-}ESM} = 0.32^{1/n}\ D_{50\text{-}ESM}$$

$$D_{8\text{-}ESM} = 0.16^{1/n}\ D_{50\text{-}ESM}$$

Where,

$D_{30\text{-}USACE}$ = the D_{30} calculated using the USACE (1991) Steep Slope Riprap Design equation

$D_{x\text{-}ESM}$ = Particle diameter at which the ESM mix is x-percent finer

n = density parameter that defines the coarseness of the mix

Results

This resulting gradation consisted of a D_{100}, D_{84}, D_{50}, D_{16}, and D_8 equal to 48.0, 19.2, 7.7, 0.4, and 0.08 inches, respectively. The California Department of Fish and Game recommends that the D_{84} – D_{100} should be separated from the ESM mix and reserved for construction of large stable rock features, where applicable. Since the proposed design includes large stable rock features (boulder weirs, rock clusters), a revised ESM gradation was developed for the remaining portion of the ESM, where the D_{100} was equal to the initially calculated D_{84}.

The Washington State Department of Transportation Standard Specifications (2021) for streambed aggregates were used to determine appropriate proportions of sediment and cobbles to create a well-graded size distribution that was reasonably consistent with the calculated gradation. The resulting mixture consists of 1 part streambed sediment and 1 parts cobble with gradations as presented in Table 5-5 and Table 5-6. Calculations for the ESM are provided in Appendix E.

Table 5-5. ESM - Streambed Sediment Gradation

Size Diameter	Percent Passing
2.5-inch	100
2.0-inch	80
1.5-inch	73
1.0-inch	65
0.75-inch	50
0.187-inch	35
No. 40	16
No. 200	7

Table 5-6. ESM - Cobbles Gradation

Size Diameter	
18-inch	100
15-inch	75
12-inch	50
10-inch	40
8-inch	34
6-inch	28
5-inch	23
4-inch	20
3-inch	17
2.5-inch	14
2-inch	11
1-inch	5.4

6 Scour Analysis

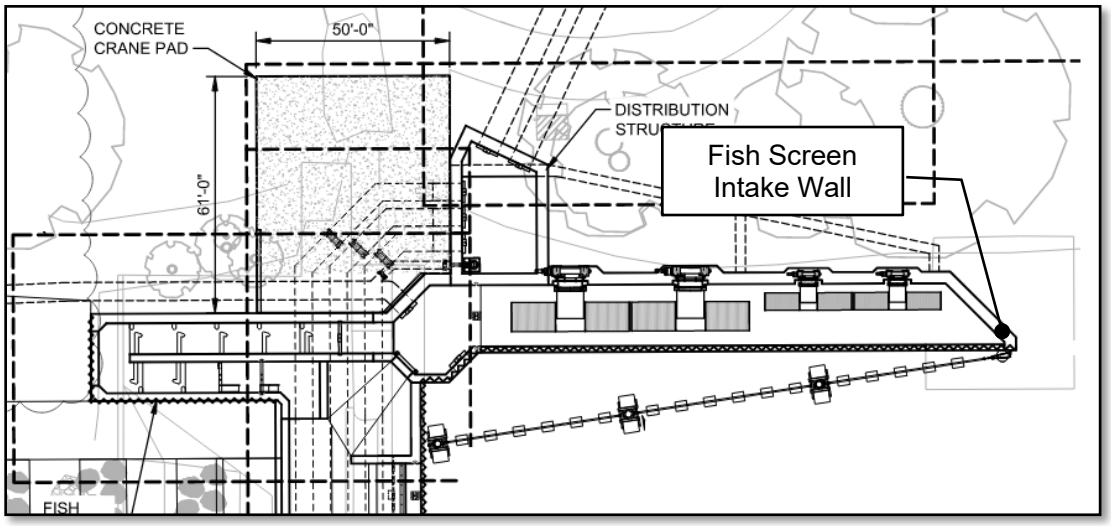
Contraction, impingement, and other applicable scour mechanisms were evaluated at appropriate locations, along with risk of bank erosion and long-term degradation. Long-term and local scour were assessed primarily by, but not limited to, the following guidelines:

- U.S. Department of Agriculture Technical Supplement 14B, Scour Calculations (2007).
- Federal Highway Administration (FHWA) Hydraulic Engineering Circular 18, Evaluating Scour at Bridges (2012).
- FHWA Hydraulic Engineering Circular 23, Bridge Scour and Stream Instability Countermeasures (2009).

6.1 Concrete Fish Screen Intake Wall

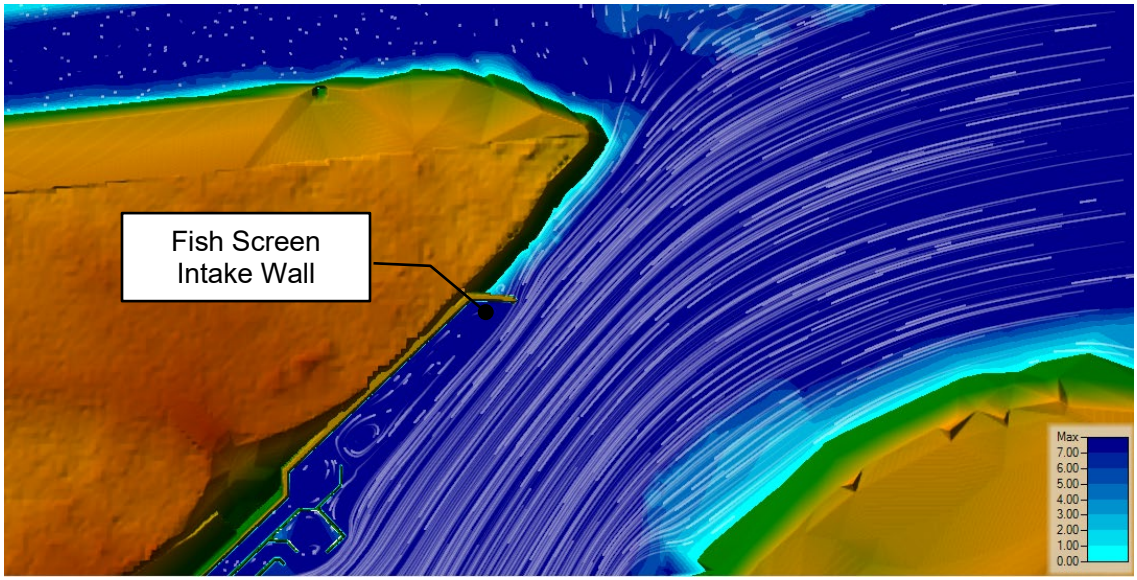
The proposed intake includes a concrete wall behind the screen assemblies, which wraps around the upstream extent of the intake apron (Figure 6-1). The upstream end of the wall extends into the inundated area of the 100-year discharge and is potentially subject to scour. The intake apron and wall are supported on piles. Sheet piling in place for construction purposes will be cut to final grade elevations and will remain along the wall face and in front of the apron, providing protection from undermining. Anticipated sheet pile depths below finish grade are approximately 20 feet. However, given the protruding nature of the wall (Figure 6-2), scour risk at the wall terminus was evaluated to assess the need for additional scour countermeasures elements.

Figure 6-1. Concrete Fish Screen Intake Wall



There were three scour mechanisms evaluated at this location. First, the channel is horizontally contracted from its normal upstream width to the width of the existing weir (Figure 6-3). This contraction of flow can cause general scour across the channel during a large flood event. Contraction scour was assessed through a comparison of hydraulics characteristics upstream of the channel contraction and at the point of interest within the contraction itself. While there is some contraction that occurs, results indicated that no scour is anticipated.

Figure 6-2. Fish Screen Intake Wall and 100-year Inundation Limits



Second, the wall terminus is located at the downstream end of a bend in the river, along the outer bank. Channel bends experience higher velocities along the outer bend, which can cause scour along the toe of the bank. Bendway scour was computed, as the effects of the channel bend could impact the wall terminus. A total bendway scour of 1.6 feet was computed.

Finally, impingement scour was assessed. Impingement occurs where flows are directed into a vertical wall or structure. The vertical Fish Screen Intake Wall intercepts upstream flow and has the potential to force flows downward in the water column, creating a localized scour condition. The scour depth is highly sensitive to the approach angle of flow, which can be variable. A 5-degree angle of approach was assumed, which yielded 10.3 feet of scour. Instead of designing the wall foundation to accommodate an anticipated scour depth, it's typical to counter the scouring effects with the use of a riprap apron.

The total scour depth anticipated is 11.9 feet (Table 6-1). While the sheet piling will remain well below this limit, a scour countermeasure at the upstream end of the intake, where flow impingement and bendway effects would be the most severe, was recommended.

Figure 6-3. Scour Mechanisms and 100-year Velocity

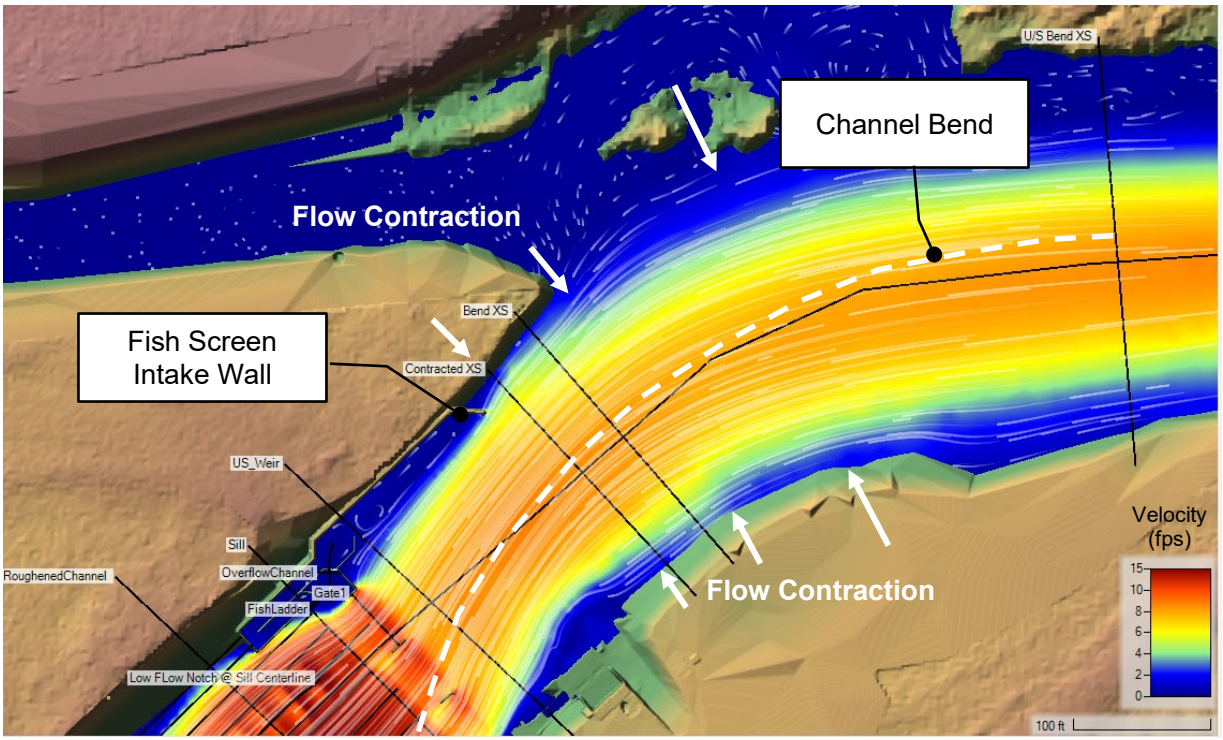


Table 6-1. Fish Screen Intake Wall Scour Potential (100-year)

Scour Mechanism	Scour Depth (ft)
Horizontal Contraction	0.0
Bendway	1.6
Impingement	10.3
Total	11.9

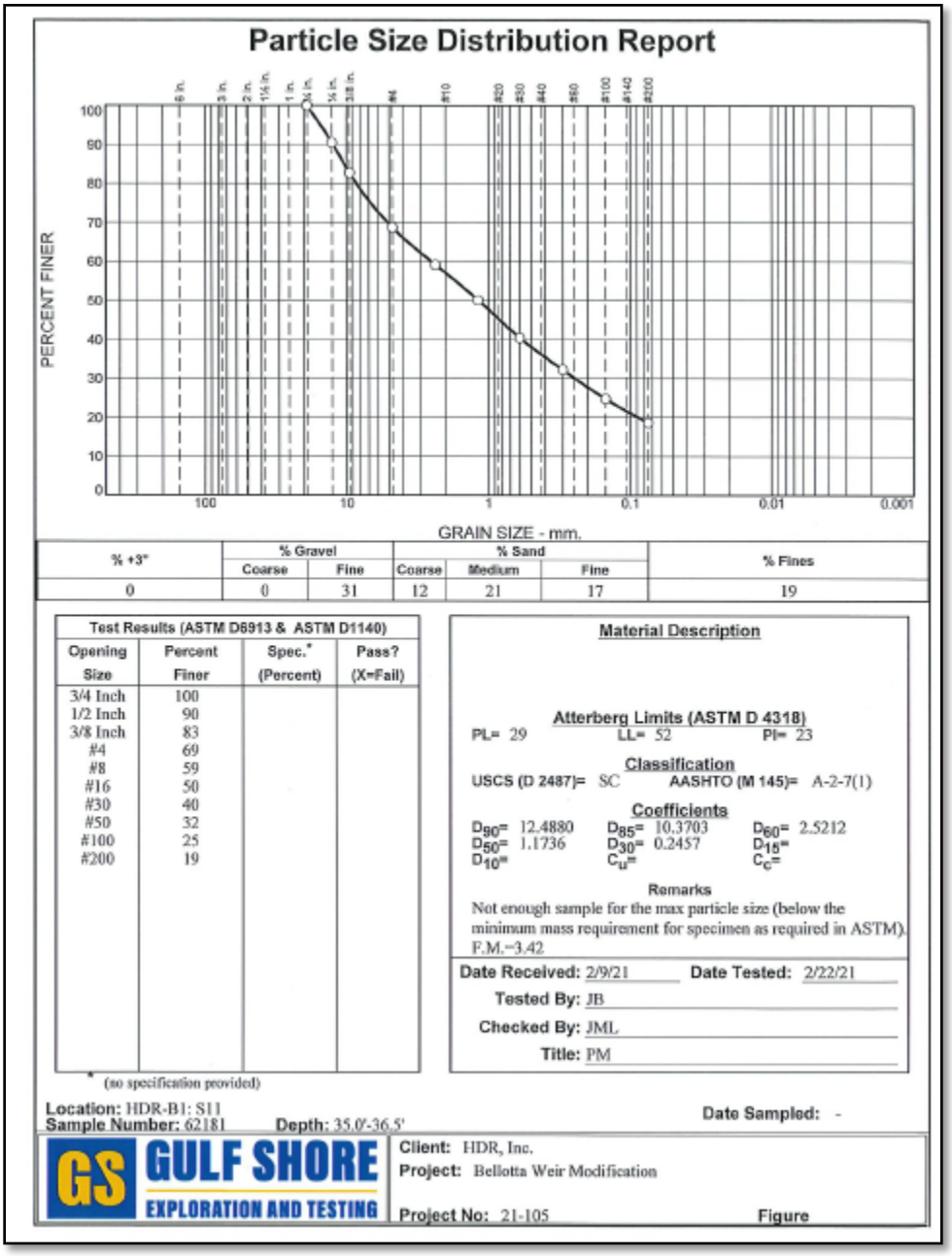
6.2 Roughened Channel Toe Down

The roughened channel is designed to be a permanent and stable feature within the main channel. The upstream end of the channel is protected by the proposed weir and sheet pile cutoff walls. The downstream end does not tie into a similar hardened feature, but rather transitions to the natural channel section of Mormon Slough. Degradation at the downstream end of any channel spanning structure is a likely mode of failure. Downstream degradation and the localized effects of slope or channel material transitions can result in a scour hole capable of undermining the structure. Therefore, the downstream end needs to be designed to accommodate the depth of this potential scour. With an in-stream, rock structure, this is typically done with a toe-down, where rock is extended to the depth of anticipated scour to prevent undermining. Two scour mechanisms were evaluated: long-term degradation, and the plunging effects of an exposed toe. These were evaluated using a1D sediment transport model based on the existing conditions 1-D HEC-RAS model.

The sediment model was run in a long-term scenario (approximately five months) with a channel-forming discharge on the order of the two-year flood event (Q_2 = 3,520 cfs), until no further bed change was observed. The bed material gradation data was obtained from boring HDR-B1 (HDR, 2021b) in the vicinity of the existing weir. Relatively fine sandy material sample (S11), with a median grain size D_{50} = 1 mm (Figure 6-4) was used to represent the entire bed gradation below the surface. Harder (clay) and coarser (gravel) layers of soil present in boring HDR-B1 above the sandy material were neglected to provide conservative scour results.

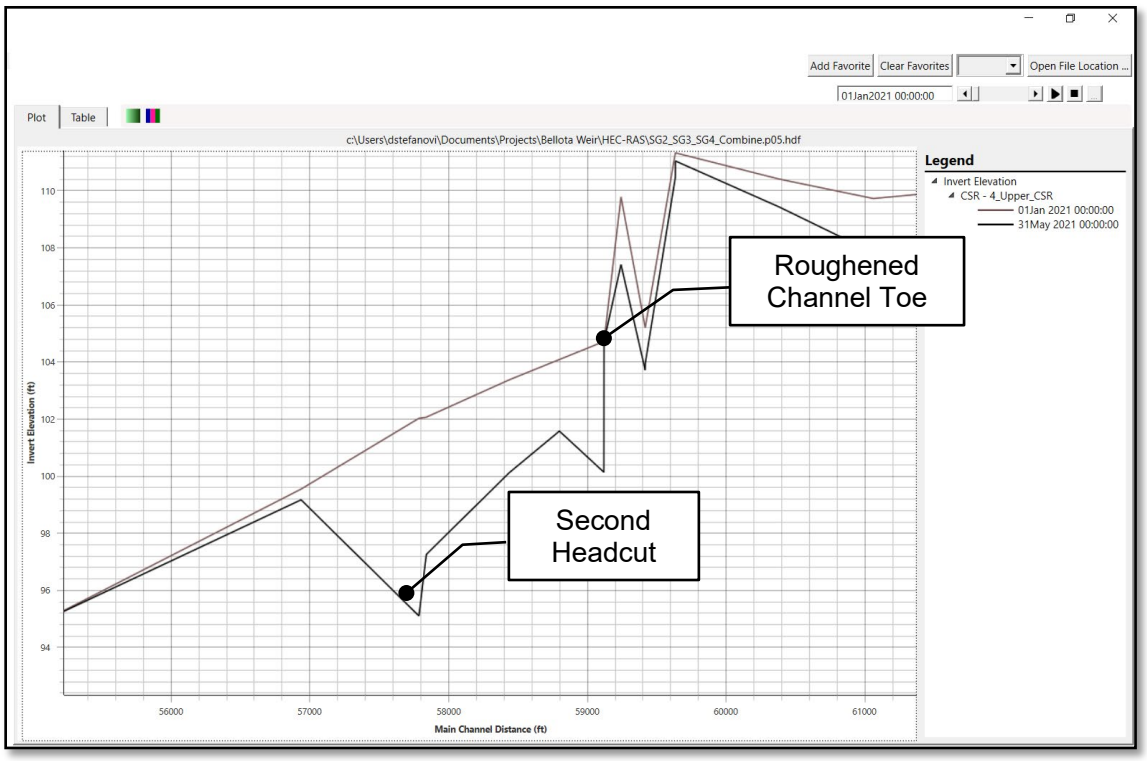
The upstream sediment load was approximated by a relatively low sediment concentration of 50 milligrams per liter (mg/l) to also maximize bed degradation. Sediment runs were based on the Yang transport function (recommended for sandy bed material) and Copeland (Exner 7) sorting method, which is known to be conservative for scour evaluation in finer systems.

Figure 6-4. Geotechnical Boring B1 with Sample S11



The long-term sediment transport results indicate maximum 5 ft of bed degradation immediately downstream of the weir (Figure 6-5) for the conservative modeling assumptions. Approximately 1000 feet downstream of the maximum bed degradation near the weir toe there is another potential headcut that may propagate farther upstream. This would cause a maximum bed degradation less than 10 feet in this worst-case scenario. Therefore, a toe down depth of 10-feet is recommended for the downstream end of the roughened channel.

Figure 6-5. Channel Bed Profile with Long-Term Bed Degradation



7 Scour Countermeasure Design

Scour countermeasures to the proposed structures were designed using the following guidelines:

- USACE Engineering Manual 1110-2-1601, Hydraulic Design of Flood Control Channels (1994).
- FHWA Hydraulic Engineering Circular 14, Hydraulic Design of Energy Dissipators for Culverts and Channels (2006).
- FHWA Hydraulic Engineering Circular 23, Bridge Scour and Stream Instability Countermeasures (2009).
- Reclamation Rock Ramp Design Guidelines (2007).
- FEMA Filters for Embankment Dams – Best Practices for Design and Construction (2011).

7.1 Fish Screen Intake Wall Riprap Apron

Given the assessed depth of potential scour at the end of the Fish Screen Intake Wall, a designed countermeasure in the form of a riprap apron has been incorporated into the proposed project. This rock apron is intended to resist the localized hydraulic effects of

the primary scour mechanism, impingement scour, and to handle any degradation due to a mild bendway scour condition.

Figure 7-1 depicts a plan view of the proposed riprap apron. During the phased construction process, the entire bank will be excavated to the depth of the intake floor (El.110' NAVD 88). The channel slope will be replaced and compacted to a 1.5H: 1V slope, per the darkened grading contours. Prior to this, the riprap apron will be constructed at the level of the intake floor and wall base. Subsequently, the apron will be partially buried below the reconstructed channel embankment. The apron will exist as a safeguard against further degradation in the event the new constructed embankment is scoured away during a high flow event.

The apron will extend 25-feet downstream and upstream of the wall terminus to ensure adequate coverage to mitigate for the local hydraulic effect of flow impingement. The upstream edge of the apron will include a thickened edge of twice the apron thickness per USACE Engineering Manual 1110-2-1601 (USACE 1994). This thickened end treatment will safeguard against the effects of bendway scour.

Rock sizing was developed utilizing methods presented in the USACE Engineering Manual 1110-2-1601 (USACE 1994). Table 7-1 summarizes the recommended riprap design parameters.

Figure 7-1. Fish Screen Intake Wall Apron

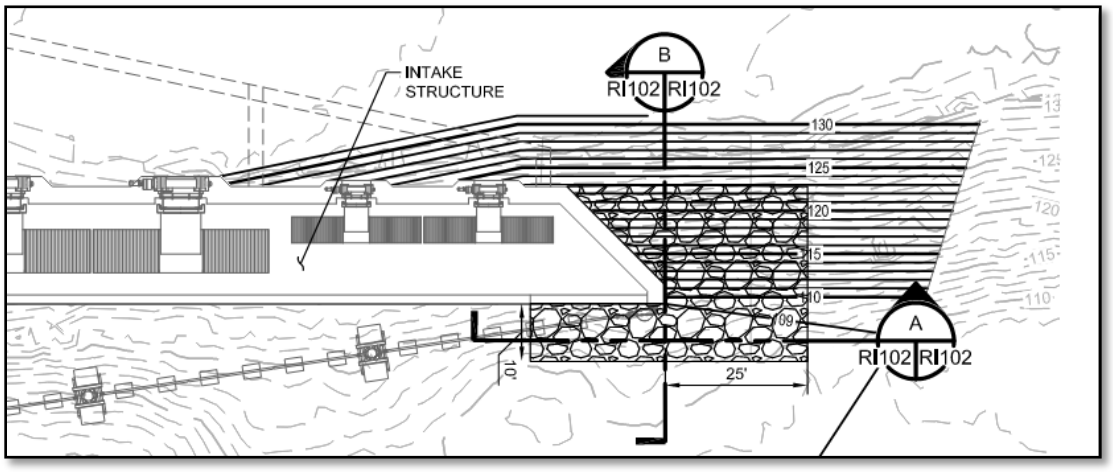


Table 7-1. Proposed Apron Rock Gradation and Layer Thickness

Location	Nom. D50 (in)	D15 (in)		D50 (in)		D85 (in)		D100 (in)	Apron Thickness (in)	Weight Class	Description
		Min	**Max**	**Min**	**Max**	**Min**	**Max**				
Fish Screen Intake Wall	12	7.3	10.5	11.5	14.0	15.5	18.5	24.0	24.0	150 lb	NCHRP Class III

7.2 Old Calaveras Outlet Structure Riprap Apron

Given the orientation of the Old Calaveras River pipelines to the Old Calaveras River, scouring velocities are expected and energy dissipation required. The Old Calaveras Outlet Structure was designed with a length to combat the expected scour length of the bend from the pipeline outlet orientation to the orientation of the Old Calaveras River (see DDR Section 7.7.2). Following this outlet structure, to provide further energy dissipation a countermeasure in the form of a riprap apron has been incorporated in the proposed project. This rock apron is intended to resist the effects of excess energy from the outlet of the Old Calaveras River pipelines.

Figure 7-2 depicts a plan view of the proposed riprap apron. During the construction process, the channel will be excavated to a depth of approximately 4 feet to a final elevation of 118.0 on each bank. This height accommodates 2 feet of freeboard over the expected channel depth. There will be a filter layer of 18-inch depth installed above existing fill, with a geotextile on top. Riprap will be installed above the geotextile with a 30-inch depth. The apron will extend 22.5 downstream of the structure.

Apron sizing was developed utilizing methods presented in the FHWA HEC 14 manual (FHWA 2006). Table 7-2 summarizes the recommended riprap design parameters.

Figure 7-2. Old Calaveras Outlet Structure Apron

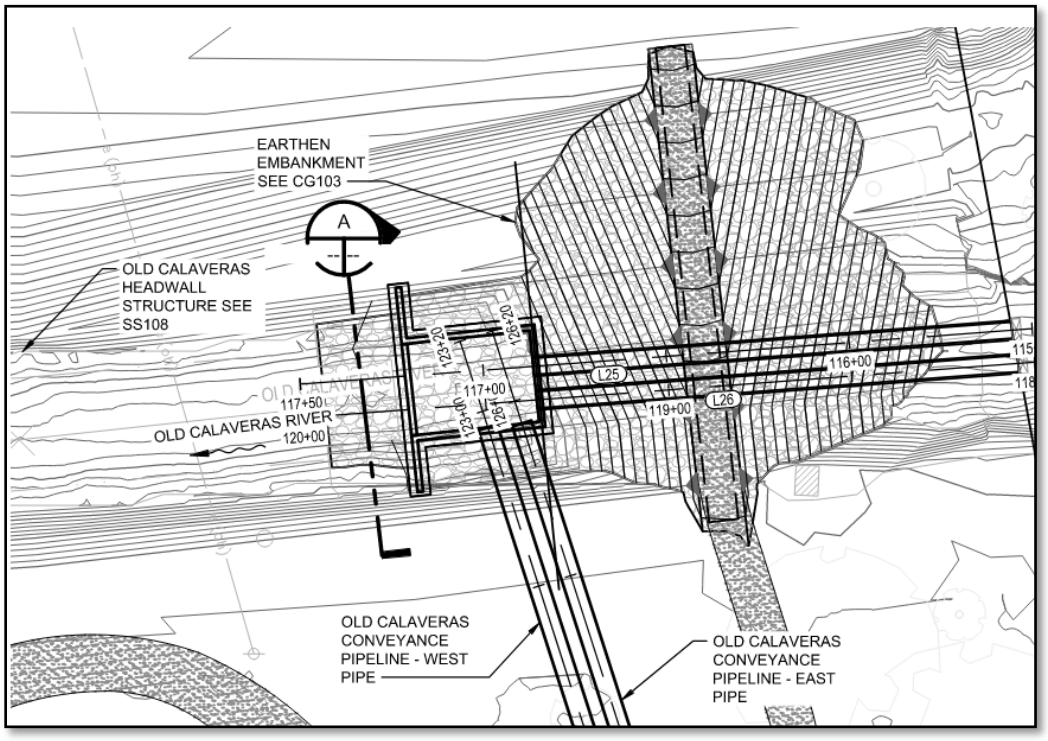


Table 7-2. Proposed Old Calaveras Outlet Structure Apron Rock Gradation and Layer Thickness

Location	Nom. D50 (in)	D15 (in)		D50 (in)		D85 (in)		D100 (in)	Apron Thickness (in)	Weight Class	Description
		Min	**Max**	**Min**	**Max**	**Min**	**Max**				
Old Calaveras River Outlet Structure	12	7.3	10.5	11.5	14.0	15.5	18.5	24.0	24.0	150 lb	NCHRP Class III

8 Next Steps and Recommendations

This report documents the current progress of the ongoing hydraulic modeling and analysis effort. The following actions are pending:

- Confirm with the San Joaquin County Floodplain Administrator that localized water surface increases within the Project area are acceptable. Continue to refresh the model based on design and surface changes.
- Complete the FEMA No-Rise Certificate, or Conditional Letter of Map Revision, if required, in preparation for delivery to the local floodplain administrator.

The Project may require authorization from USACE under 33 United States Code Section 408. A Section 408 authorization is mandated for any use or potential alteration of a USACE project. The Mormon Slough system is a federally authorized flood control facility constructed by the USACE and falls under USACE jurisdiction. USACE's Operation and Maintenance manual for the Mormon Slough flood control system is unclear as to whether USACE's jurisdiction extends to the Bellota Weir, therefore, an early determination of a potential Section 408 nexus is recommended.

9 References

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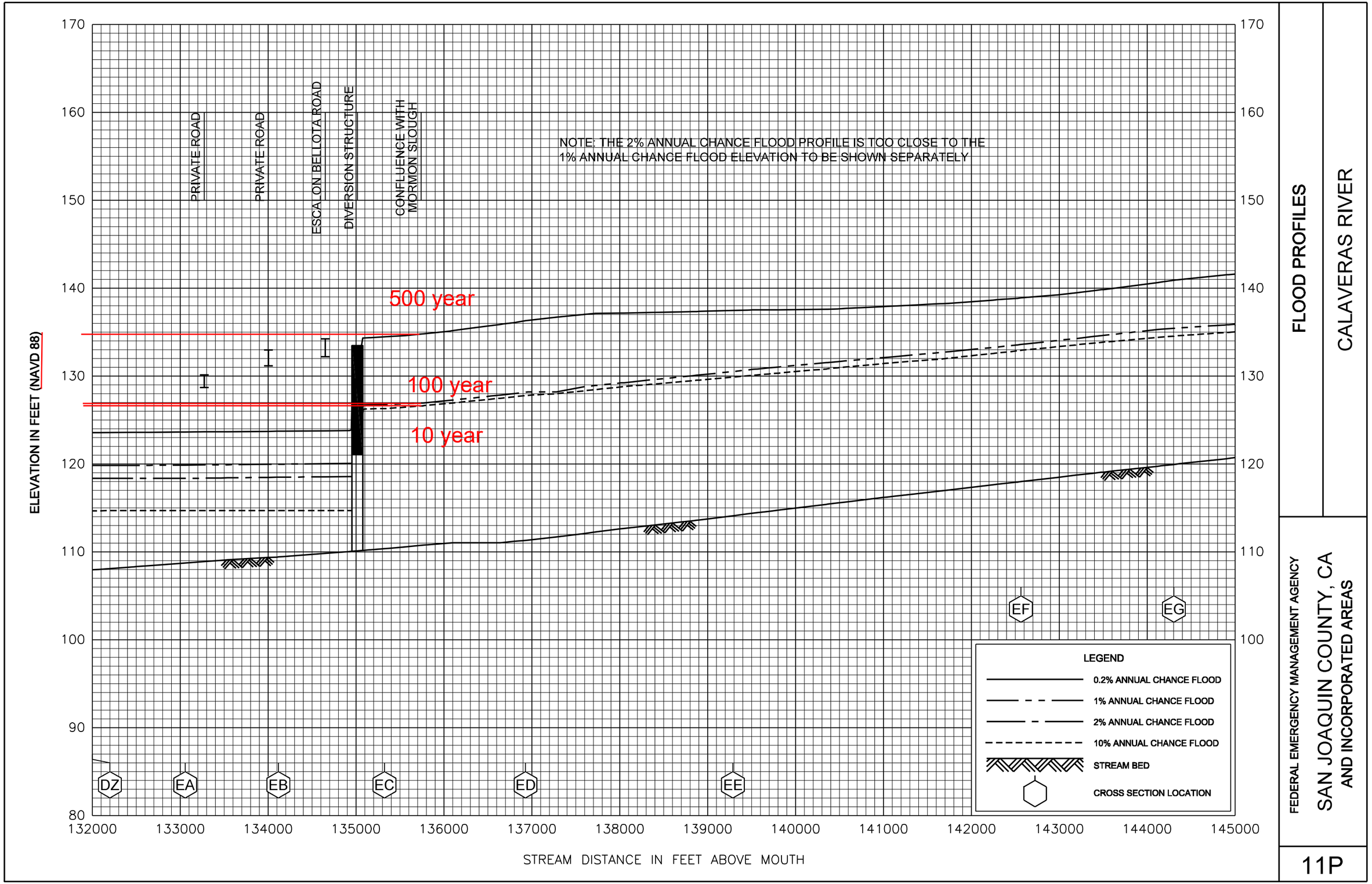
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Appendix A. FEMA FIS Data and FIRM Panels

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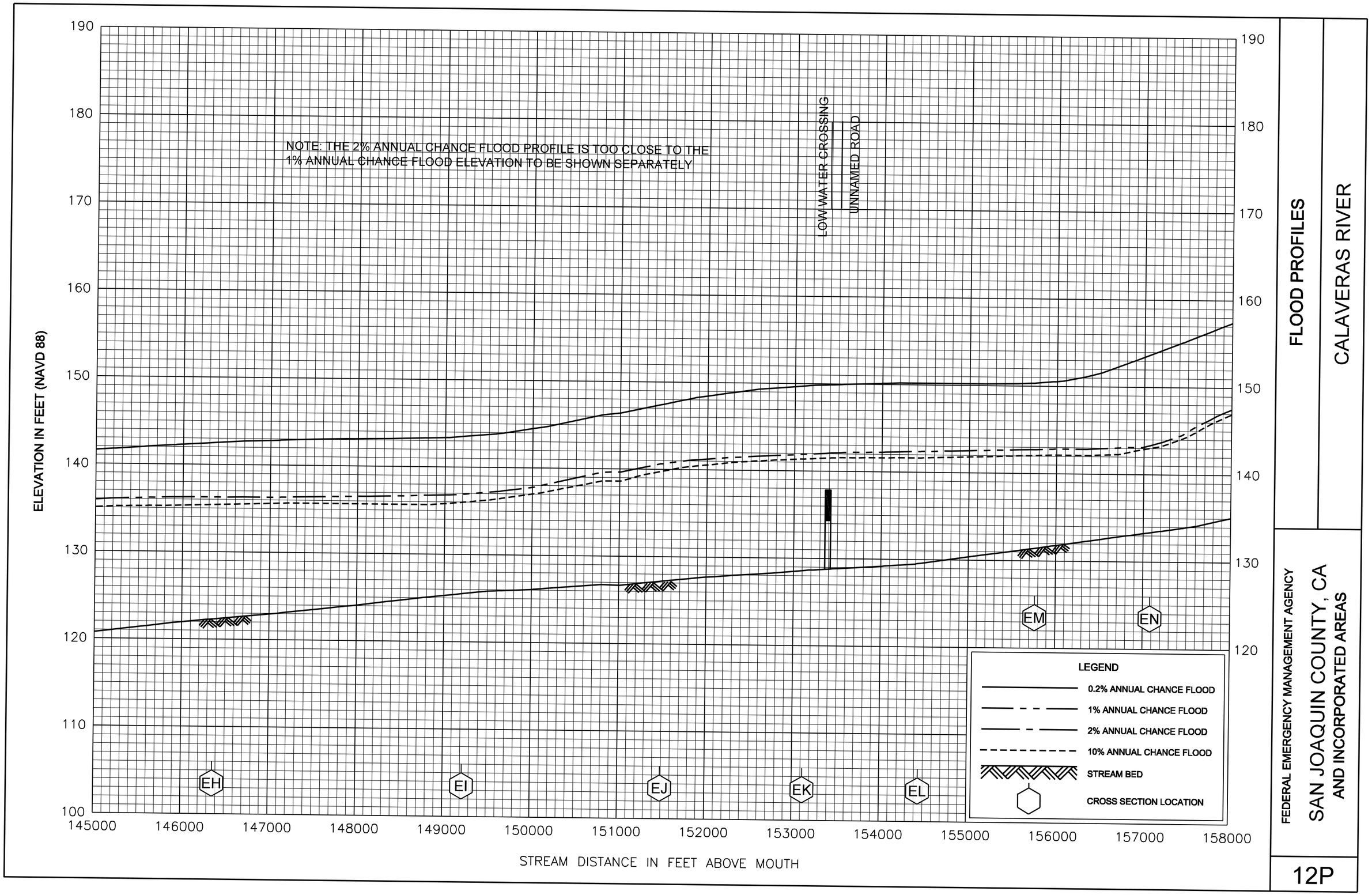
FLOOD PROFILES

CALAVERAS RIVER

FEDERAL EMERGENCY MANAGEMENT AGENCY

SAN JOAQUIN COUNTY, CA
AND INCORPORATED AREAS

11P



FLOOD PROFILES

CALAVERAS RIVER

FEDERAL EMERGENCY MANAGEMENT AGENCY

SAN JOAQUIN COUNTY, CA
AND INCORPORATED AREAS

12P

NOTE: THE 2% ANNUAL CHANCE FLOOD PROFILE IS TOO CLOSE TO THE 1% ANNUAL CHANCE FLOOD ELEVATION TO BE SHOWN SEPARATELY

SHELTON ROAD

COUNTY BOUNDARY

ELEVATION IN FEET (NAVD 88)

200
190
180
170
160
150
140
130
120
110

158000 159000 160000 161000 162000 163000 164000 165000 166000 167000

STREAM DISTANCE IN FEET ABOVE MOUTH

EO EP EQ ER ES ET EU

LEGEND

- 0.2% ANNUAL CHANCE FLOOD
- 1% ANNUAL CHANCE FLOOD
- 2% ANNUAL CHANCE FLOOD
- 10% ANNUAL CHANCE FLOOD
- STREAM BED
- CROSS SECTION LOCATION

FLOOD PROFILES

CALAVERAS RIVER

FEDERAL EMERGENCY MANAGEMENT AGENCY

SAN JOAQUIN COUNTY, CA
AND INCORPORATED AREAS

Table 10: Summary of Discharges

Flooding Source	Location	Drainage Area (Square Miles)	Peak Discharges (cfs)				
			10% Annual Chance	4% Annual Chance	2% Annual Chance	1% Annual Chance	0.2% Annual Chance
Bear Creek	Interstate Highway 5	102.5	*	*	*	7,630	*
Bear Creek	At confluence of Pixley Slough	100.7	*	*	*	7,540	*
Bear Creek	State Highway 99	84.2	*	*	*	6,730	*
Bear Creek	At confluence of Mosher Diversion	84.1	*	*	*	6,370	*
Bear Creek	At confluence of Paddy Creek	79.4	*	*	*	5,490	*
Bear Creek	At Lockeford gage	48.4	*	*	*	3,580	*
Calaveras River	Downstream of Stockton Diverting Canal[2]	614.2	*	*	*	15,818	*
Calaveras River	Upstream of Stockton Diverting Canal	23.7	*	*	*	1,159[3]	*
Calaveras River	Jack Tone Road	14.4	*	*	*	1,450	*
Corral Hollow Creek[1]	Chrisman Road	65.0	375	*	835	840	845
Corral Hollow Creek[1]	McArthur Road	64.0	400	*	1,200	1,300	1,380

Table 10: Summary of Discharges (continued)

[2]Flow is diverted from the Calaveras River to Mormon Slough at Bellota, carried by Mormon Slough to the Stockton Diverting Canal, and returned to the Calaveras River.
[3]Discharge decrease due to channel routing.
[4]Decrease in volume of 10-, 2-, and 1-percent annual chance floodflows in a downstream direction results from channel routing losses. Fluctuation in 0.2-percent annual chance floodflow results from channel routing losses and overbank gains (principally from Mormon Slough) and overbank losses.
[5]Flows for 10-, 2-, 1-, and 0.2-percent annual chance floods reflect overbank gains and losses. Source of slough is the confluence of the North and South Forks of South Littlejohns Creek.
[6]Decrease in volume of 1- and 0.2-percent annual chance floods in a downstream direction results from overbank and channel routing losses.
[7]Outflow will vary according to local conditions affecting project operation.
[8]Anabranch Channel of South Littlejohns Creek.
[9]Flows of 10-, 2-, and 1-percent annual chance floods at the upper end of the channel are estimated. Variations in flood flow downstream from Diamond Street result from storm drain diversions that return to the channel downstream and (for the 0.2-percent annual chance flood) from overbank gains and losses. The large increase in 0.2-percent annual chance flow between the upper end of the channel and State Highway 99 results from Mormon Slough inflow via the Duck Creek system.
[10]Volume of flow decreases in the downstream direction because of channel routing losses (1-percent annual chance flood) or overbank losses (0.2-percent annual chance flood).
[11]Controlled release.
[12]Decrease in 2-percent annual chance floodflow results from overbank losses. Fluctuation in 1- and 0.2-percent annual chance floodflows reflects overbank losses and gains, including overland inflow from South Fork South Littlejohns Creek.
[13]Volume of flow decreases in a downstream direction because of overbank losses and distributary-type flows from the study reach.
[14]Some of the drainage areas are estimates.
[15]Flows shown reflect overbank gains and losses and (for 0.2-percent annual chance flood) tributary inflow.
[16]Discharge increase due to incoming flow from overtopping right bank.
[17]Discharge decrease due to channel routing.
[18]Also known as Tempo Creek.
[19]Fluctuations in flows shown result from overbank losses and gains, including overland inflow from the upper reaches of Lone Tree Creek.

Table 10: Summary of Discharges (continued)

Flooding Source	Location	Drainage Area (Square Miles)	Peak Discharges (cfs)				
			10% Annual Chance	4% Annual Chance	2% Annual Chance	1% Annual Chance	0.2% Annual Chance
Lower Mosher Creek (Mosher Slough)	State Highway 99	3.9	*	*	*	430	*
Lower Mosher Creek (Mosher Slough)	Alpine Road	2.0	*	*	*	240	*
Middle Paddy Creek	Jack Tone Road	7.8	*	*	*	990	*
Mormon Channel[9]	Terminus (Upper Stockton Channel)	8.4	245	*	430	520	2,835
Mormon Channel[9]	Southern Pacific Railroad	6.4	100	*	195	240	2,615
Mormon Channel[9]	Diamond Street	4.5	175	*	300	365	2,720
Mormon Channel[9]	State Highway 99	2.5	175	*	300	365	3,525
Mormon Channel[9]	Upper end of channel	0.0	5	*	10	20	325
Mormon Slough	Terminus (head of Diverting Canal)	538.0	*	*	*	15,020[2]	*
Mormon Slough	Confluence of Potter Creek A	536.4	*	*	*	15,140	*

Table 10: Summary of Discharges (continued)

Flooding Source	Location	Drainage Area (Square Miles)	Peak Discharges (cfs)				
			10% Annual Chance	4% Annual Chance	2% Annual Chance	1% Annual Chance	0.2% Annual Chance
Mormon Slough	Confluence of Potter Creek B	513.7	*	*	*	13,240	*
Mormon Slough	At Bellota[1]	480.0	12,500	*	12,500	12,690	27,100
Mokelumne River[10]	Below Dry Creek	1,000	*	*	*	28,200	*
Mokelumne River[10]	Mouth of Dry Creek	696	4,000[11]	*	5,000[11]	14,500	34,400
Mokelumne River[10]	Peltier Road	680	4,000[11]	*	5,000[11]	15,400	35,200
Mokelumne River[10]	Woolbridge Dam	661	4,000[11]	*	5,000[11]	15,900	40,700
Mokelumne River[10]	Lodi Lake	658	4,000[11]	*	5,000[11]	16,000	41,800
Mokelumne River[10]	Southern Pacific Railroad Bridge	656	4,000[11]	*	5,000[11]	16,000	41,800
Mokelumne River[10]	State Highway 99	653	4,000[11]	*	5,000[11]	16,100	47,900
Mokelumne River[10]	Central Highway Traction Co. Railroad	650	4,000[11]	*	5,000[11]	16,100	49,400
Mokelumne River[10]	Elliot Road	644	4,000[11]	*	5,000[11]	18,100	57,300
Mokelumne River[10]	At Camanche Dam	627	4,000[11]	*	5,000[11]	19,000	60,200
Mosher Creek	Mouth (Bear Creek)	10.0	*	*	*	851	*

Table 24: Floodway Data

LOCATION		FLOODWAY			1% ANNUAL CHANCE FLOOD WATER SURFACE ELEVATION (FEET NAVD88)			
CROSS SECTION	DISTANCE[1]	WIDTH (FEET)	SECTION AREA (SQ. FEET)	MEAN VELOCITY (FEET/SEC)	REGULATORY	WITHOUT FLOODWAY	WITH FLOODWAY	INCREASE
A-AY[2]								
AZ	49,632	50	245	3.4	43.5	43.5	43.5	0.0
BA	50,688	50	250	3.3	44.8	44.8	44.8	0.0
BB	51,744	45	250	3.3	45.8	45.8	45.8	0.0
BC	52,800	45	260	3.2	47.3	47.3	47.3	0.0
BD	53,856	50	255	3.2	48.4	48.4	48.4	0.0
BE	54,912	50	270	3.4	49.8	49.8	49.8	0.0
BF	55,968	60	335	2.7	50.6	50.6	50.6	0.0
BG	57,024	60	335	2.4	51.4	51.4	51.4	0.0
BH	58,080	55	295	2.8	52.0	52.0	52.0	0.0
BI	59,136	55	285	2.9	53.0	53.0	53.0	0.0
BJ	60,192	55	265	3.1	54.1	54.1	54.1	0.0
BK	61,248	50	245	3.4	55.4	55.4	55.4	0.0
BL	62,304	50	275	3.0	56.6	56.6	56.6	0.0
BM	63,360	55	280	2.9	57.6	57.6	57.6	0.0
BN	64,416	55	290	2.9	58.6	58.6	58.6	0.0
BO	65,472	55	300	2.7	59.5	59.5	59.5	0.0
BP	66,528	60	315	2.6	60.3	60.3	60.3	0.0
BQ	67,584	55	315	2.6	61.1	61.1	61.1	0.0
BR	68,640	50	260	3.2	62.1	62.1	62.1	0.0
BS	69,696	50	270	3.1	63.2	63.2	63.2	0.0
BT	70,752	55	285	2.9	64.3	64.3	64.3	0.0
BU	71,808	55	295	2.8	65.2	65.2	65.2	0.0

[1]Feet above mouth
[2]Cross sections A-AY not included. Floodway superseded by revised analysis.

Table 24: Floodway Data (continued)

LOCATION		FLOODWAY			1% ANNUAL CHANCE FLOOD WATER SURFACE ELEVATION (FEET NAVD88)			
CROSS SECTION	DISTANCE[1]	WIDTH (FEET)	SECTION AREA (SQ. FEET)	MEAN VELOCITY (FEET/SEC)	REGULATORY	WITHOUT FLOODWAY	WITH FLOODWAY	INCREASE
BV	72,864	60	335	2.5	65.9	65.9	65.9	0.0
BW	73,920	55	320	2.6	66.6	66.6	66.6	0.0
BX	74,976	55	315	2.6	67.1	67.1	67.1	0.0
BY	76,032	55	315	2.6	67.9	67.9	67.9	0.0
BZ	76,982	55	315	2.6	68.5	68.5	68.5	0.0
CA	78,144	55	290	2.8	69.5	69.5	69.5	0.0
CB	79,200	55	275	3.0	70.2	70.2	70.2	0.0
CC	80,256	55	270	3.0	71.3	71.3	71.3	0.0
CD	81,312	60	300	2.8	72.5	72.5	72.5	0.0
CE	82,368	55	300	2.8	73.7	73.7	73.7	0.0
CF	83,424	55	310	2.7	74.7	74.7	74.7	0.0
CG	84,480	55	305	2.7	75.7	75.7	75.7	0.0
CH	85,536	55	295	2.8	76.5	76.5	76.5	0.0
CI	86,592	55	295	2.8	78.1	78.1	78.1	0.0
CJ	87,648	60	325	2.5	79.9	79.9	79.9	0.0
CK	88,704	60	340	2.4	81.5	81.5	81.5	0.0
CL	89,760	60	360	2.3	82.9	82.9	82.9	0.0
CM	90,816	65	370	2.2	84.0	84.0	84.0	0.0
CN	91,872	65	370	2.2	85.1	85.1	85.1	0.0
CO	92,928	65	380	2.2	86.1	86.1	86.1	0.0
CP	93,984	65	330	2.5	87.0	87.0	87.0	0.0
CQ	95,040	60	330	2.5	87.4	87.4	87.4	0.0
CR	96,096	60	310	2.6	88.0	88.0	88.0	0.0
CS	97,152	60	300	2.7	88.9	88.9	88.9	0.0
CT	98,208	55	285	2.9	89.8	89.8	89.8	0.0

[1]Feet above mouth

Table 24: Floodway Data (continued)

LOCATION		FLOODWAY			1% ANNUAL CHANCE FLOOD WATER SURFACE ELEVATION (FEET NAVD88)			
CROSS SECTION	DISTANCE[1]	WIDTH (FEET)	SECTION AREA (SQ. FEET)	MEAN VELOCITY (FEET/SEC)	REGULATORY	WITHOUT FLOODWAY	WITH FLOODWAY	INCREASE
CU	99,264	55	285	2.9	90.8	90.8	90.8	0.0
CV	100,320	55	260	3.1	91.8	91.8	91.8	0.0
CW	101,376	50	260	3.1	93.0	93.0	93.0	0.0
CX	102,432	55	255	3.1	94.1	94.1	94.1	0.0
CY	103,488	55	270	2.9	95.4	95.4	95.4	0.0
CZ	104,544	55	300	2.7	97.1	97.1	97.1	0.0
DA	105,600	50	255	3.1	98.4	98.4	98.4	0.0
DB	106,656	55	270	1.6	99.6	99.6	99.6	0.0
DC	107,712	50	240	1.8	100.2	100.2	100.2	0.0
DD	108,768	45	195	2.2	101.2	101.2	101.2	0.0
DE	109,824	40	205	2.1	102.2	102.2	102.2	0.0
DF	110,880	40	205	2.1	103.2	103.2	103.2	0.0
DG	111,936	40	205	2.1	103.9	103.9	103.9	0.0
DH	112,992	40	150	2.8	105.7	105.7	105.7	0.0
DI	114,048	40	170	2.5	107.3	107.3	107.3	0.0
DJ	115,104	45	230	3.6	109.5	109.5	109.5	0.0
DK	116,160	55	285	2.9	111.5	111.5	111.5	0.0
DL	117,216	60	320	2.6	113.3	113.3	113.3	0.0
DM	118,272	55	345	2.5	114.4	114.4	114.4	0.0
DN	119,328	55	320	2.4	115.5	115.5	115.5	0.0
DO	120,384	60	390	2.1	116.3	116.3	116.3	0.0
DP	121,440	65	480	1.7	116.9	116.9	116.9	0.0
DQ	122,496	75	625	1.3	117.2	117.2	117.2	0.0

[1]Feet above mouth

Table 24: Floodway Data (continued)

LOCATION		FLOODWAY			1% ANNUAL CHANCE FLOOD WATER SURFACE ELEVATION (FEET NAVD88)			
CROSS SECTION	DISTANCE[1]	WIDTH (FEET)	SECTION AREA (SQ. FEET)	MEAN VELOCITY (FEET/SEC)	REGULATORY	WITHOUT FLOODWAY	WITH FLOODWAY	INCREASE
DR	123,605	75	550	1.5	117.3	117.3	117.3	0.0
DS	124,608	75	565	1.5	117.6	117.6	117.6	0.0
DT	125,664	75	555	1.5	117.9	117.9	117.9	0.0
DU	126,878	70	520	1.6	118.3	118.3	118.3	0.0
DV	127,776	65	415	2.0	118.7	118.7	118.7	0.0
DW	128,832	60	425	2.0	119.3	119.3	119.3	0.0
DX	130,099	60	435	1.9	119.8	119.8	119.8	0.0
DY	130,944	60	435	0.0	120.0	120.0	120.0	0.0
DZ	132,000	50	335	0.0	120.0	120.0	120.0	0.0
EA	133,056	45	295	0.0	120.1	120.1	120.1	0.0
EB	134,112	50	350	0.0	120.1	120.1	120.1	0.0
EC	135,326	60	400	0.0	126.9	126.9	126.9	0.0
ED	136,910	120	1,745	8.1	128.2	128.2	128.2	0.0
EE	139,286	360	2,340	5.6	130.5	130.5	130.7	0.2
EF	142,560	125	1,815	7.0	133.6	133.6	134.0	0.4
EG	144,302	210	2,750	4.6	135.5	135.5	136.0	0.5
EH	146,362	820	8,130	1.6	136.4	136.4	136.7	0.3
EI	149,213	230	2,025	6.3	137.0	137.0	137.1	0.1
EJ	151,483	280	2,285	5.6	140.7	140.7	140.9	0.2
EK	153,120	340	3,930	3.3	141.9	141.9	142.1	0.2
EL	154,440	515	5,530	2.4	142.4	142.4	142.5	0.1
EM	155,760	325	3,320	4.0	142.7	142.7	142.8	0.1
EN	157,080	135	1,060	12.0	143.4	143.4	143.8	0.4
EO	158,400	200	2,135	6.0	148.6	148.6	148.6	0.0
EP	159,720	160	1,750	7.3	150.6	150.6	150.7	0.1

[1]Feet above mouth

Table 24: Floodway Data (continued)

LOCATION		FLOODWAY			1% ANNUAL CHANCE FLOOD WATER SURFACE ELEVATION (FEET NAVD88)			
CROSS SECTION	DISTANCE[1]	WIDTH (FEET)	SECTION AREA (SQ. FEET)	MEAN VELOCITY (FEET/SEC)	REGULATORY	WITHOUT FLOODWAY	WITH FLOODWAY	INCREASE
EQ	161,040	340	2,720	4.7	153.0	153.0	153.1	0.1
ER	162,360	370	1,835	7.9	155.4	155.4	155.4	0.0
ES	163,680	170	1,560	8.1	158.4	158.4	159.0	0.6
ET	165,000	180	1,980	6.4	161.6	161.6	162.2	0.6
EU	166,320	180	1,630	7.8	164.2	164.2	164.7	0.5

[1]Feet above mouth

NOTES TO USERS

This map is for use in administering the National Flood Insurance Program. It does not necessarily identify all areas subject to flooding, particularly from local drainage sources of small size. The **community map repository** should be consulted for possible updated or additional flood hazard information.

To obtain more detailed information in areas where **Base Flood Elevations** (BFEs) and/or **floodways** have been determined, users are encouraged to consult the Flood Profiles and Floodway Data and/or Summary of Stillwater Elevations tables contained within the Flood Insurance Study (FIS) report that accompanies this FIRM. Users should be aware that BFEs shown on the FIRM represent rounded whole-foot elevations. These BFEs are intended for flood insurance rating purposes only and should not be used as the sole source of flood elevation information. Accordingly, flood elevation data presented in the FIS report should be utilized in conjunction with the FIRM for purposes of construction and/or floodplain management.

Coastal Base Flood Elevations shown on this map apply only landward of 0.0' North American Vertical Datum of 1988 (NAVD 88). Users of this FIRM should be aware that coastal flood elevations are also provided in the Summary of Stillwater Elevations table in the Flood Insurance Study Report for this jurisdiction. Elevations shown in the Summary of Stillwater Elevations table should be used for construction, and/or floodplain management purposes when they are higher than the elevations shown on this FIRM.

Boundaries of the **floodways** were computed at cross sections and interpolated between cross sections. The floodways were based on hydraulic considerations with regard to requirements of the National Flood Insurance Program. Floodway widths and other pertinent floodway data are provided in the Flood Insurance Study report for this jurisdiction.

Certain areas not in Special Flood Hazard Areas may be protected by **flood control structures**. Refer to Section 2.4 "Flood Protection Measures" of the Flood Insurance Study report for information on flood control structures in this jurisdiction.

The **projection** used in the preparation of this map was Universal Transverse Mercator (UTM) Zone 10N. The **horizontal datum** was NAD83, GRS80 spheroid. Differences in datum, spheroid, projection or UTM zones used in the production of FIRMs for adjacent jurisdictions may result in slight positional differences in map features across jurisdiction boundaries. These differences do not affect the accuracy of this FIRM.

Flood elevations on this map are referenced to the North American Vertical Datum of 1988. These flood elevations must be compared to structure and ground elevations referenced to the same **vertical datum**. For information regarding conversion between the National Geodetic Vertical Datum of 1929 and the North American Vertical Datum of 1988, visit the National Geodetic Survey website at http://www.ngs.noaa.gov or contact the National Geodetic Survey at the following address:

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Silver Spring, Maryland 20910-3202
(301) 713-3242

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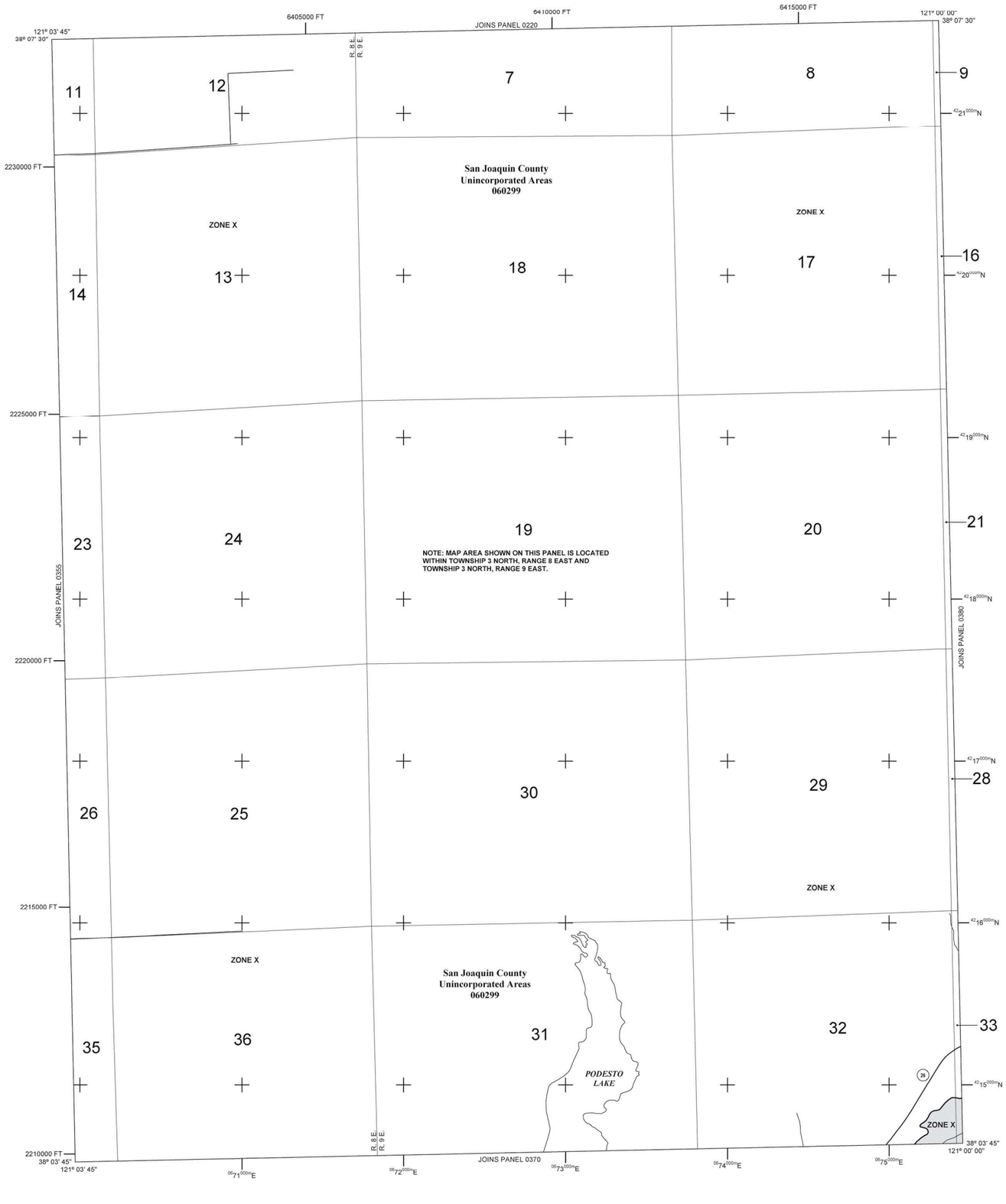
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LEGEND

SPECIAL FLOOD HAZARD AREAS (SFHAs) SUBJECT TO INUNDATION BY THE 1% ANNUAL CHANCE FLOOD

The 1% annual chance flood (100-year flood), also known as the base flood, is the flood that has a 1% chance of being equaled or exceeded in any given year. The Special Flood Hazard Area is the area subject to flooding by the 1% annual chance flood. Areas of Special Flood Hazard include Zones A, AE, AH, AO, AR, A99, V, and VE. The Base Flood Elevation is the water-surface elevation of the 1% annual chance flood.

- **ZONE A** No Base Flood Elevations determined.
- **ZONE AE** Base Flood Elevations determined.
- **ZONE AH** Flood depths of 1 to 3 feet (usually areas of ponding); base flood Elevations determined.
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- **ZONE AR** Special Flood Hazard Area formerly protected from the 1% annual chance flood by a flood control system that was subsequently decertified. Zone AR indicates that the former flood control system is being restored to provide protection from the 1% annual chance or greater flood.
- **ZONE A99** Area to be protected from 1% annual chance flood by a Federal flood protection system under construction; no Base Flood Elevations determined.
- **ZONE V** Coastal flood zone with velocity hazard (wave action); no Base Flood Elevations determined.
- **ZONE VE** Coastal flood zone with velocity hazard (wave action); Base Flood Elevations determined.

FLOODWAY AREAS IN ZONE AE

The floodway is the channel of a stream plus any adjacent floodplain areas that must be kept free of encroachment so that the 1% annual chance flood can be carried without substantial increases in flood heights.

OTHER FLOOD AREAS

- **ZONE X** Areas of 0.2% annual chance flood; areas of 1% annual chance flood with average depths of less than 1 foot or with drainage areas less than 1 square mile; and areas protected by levees from 1% annual chance flood.

OTHER AREAS

- **ZONE X** Areas determined to be outside the 0.2% annual chance floodplain.
- **ZONE D** Areas in which flood hazards are undetermined, but possible.

COASTAL BARRIER RESOURCES SYSTEM (CBRS) AREAS

OTHERWISE PROTECTED AREAS (OPAs)

CBRS areas and OPAs are normally located within or adjacent to Special Flood Hazard Areas.

- Floodplain boundary
- Floodway boundary
- Zone D Boundary
- CBRS and OPA Boundary
- Boundary dividing Special Flood Hazard Areas of different Base Flood Elevations, flood depths or flood velocities.
- 513 (EL 987) Base Flood Elevation line and value; elevation in feet* / Base Flood Elevation value where uniform within zone; elevation in feet*

*Referenced to the North American Vertical Datum of 1988

- A — A Cross section line
- 23 — 23 Transect line
- 97° 07' 30", 32° 22' 30" Geographic coordinates referenced to the North American Datum of 1983 (NAD 83), Western Hemisphere
- 6276000mE 1000-meter Universal Transverse Mercator grid values, zone 10
- 600000 FT 5000-foot grid ticks: California State Plane coordinate system, zone III (FIPSZONE 0403), Lambert Conformal Conic Projection
- DX5510 × Bench mark (see explanation in Notes to Users section of this FIRM panel)
- M1.5 River Mile

MAP REPOSITORIES
Refer to Map Repositories list on Map Index.

EFFECTIVE DATE OF COUNTYWIDE FLOOD INSURANCE RATE MAP PANEL
October 16, 2009

EFFECTIVE DATE(S) OF REVISION(S) TO THIS PANEL

For community map revision history prior to countywide mapping, refer to the Community Map History table located in the Flood Insurance Study report for this jurisdiction.

To determine if flood insurance is available in this community, contact your insurance agent or call the National Flood Insurance Program at 1-800-638-6620.

MAP SCALE 1" = 1000'

500 0 1000 2000 FEET

300 0 300 600 METERS

NFIP

NATIONAL FLOOD INSURANCE PROGRAM

PANEL 0360F

FIRM

FLOOD INSURANCE RATE MAP

SAN JOAQUIN COUNTY, CALIFORNIA AND INCORPORATED AREAS

PANEL 360 OF 950

(SEE MAP INDEX FOR FIRM PANEL LAYOUT)

CONTAINS:

COMMUNITY	NUMBER	PANEL	SUFFIX
SAN JOAQUIN COUNTY	060299	0360	F

Notice to User: The **Map Number** shown below should be used when placing map orders; the **Community Number** shown above should be used on insurance applications for the subject community.

MAP NUMBER
06077C0360F

EFFECTIVE DATE
OCTOBER 16, 2009

Federal Emergency Management Agency

NOTES TO USERS

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Boundaries of the **floodways** were computed at cross sections and interpolated between cross sections. The floodways were based on hydraulic considerations with regard to requirements of the National Flood Insurance Program. Floodway widths and other pertinent floodway data are provided in the Flood Insurance Study report for this jurisdiction.

Certain areas not in Special Flood Hazard Areas may be protected by **flood control structures**. Refer to Section 2.4 "Flood Protection Measures" of the Flood Insurance Study report for information on flood control structures in this jurisdiction.

The **projection** used in the preparation of this map was Universal Transverse Mercator (UTM) Zone 10N. The **horizontal datum** was NAD83, GRS80 spheroid. Differences in datum, spheroid, projection or UTM zones used in the production of FIRMs for adjacent jurisdictions may result in slight positional differences in map features across jurisdiction boundaries. These differences do not affect the accuracy of this FIRM.

Flood elevations on this map are referenced to the North American Vertical Datum of 1988. These flood elevations must be compared to structure and ground elevations referenced to the same **vertical datum**. For information regarding conversion between the National Geodetic Vertical Datum of 1929 and the North American Vertical Datum of 1988, visit the National Geodetic Survey website at http://www.ngs.noaa.gov or contact the National Geodetic Survey at the following address:

NGS Information Services
NOAA, N/NGS12
National Geodetic Survey, SSMC-3, #9202
1315 East-West Highway
Silver Spring, Maryland 20910-3202
(301) 713-3242

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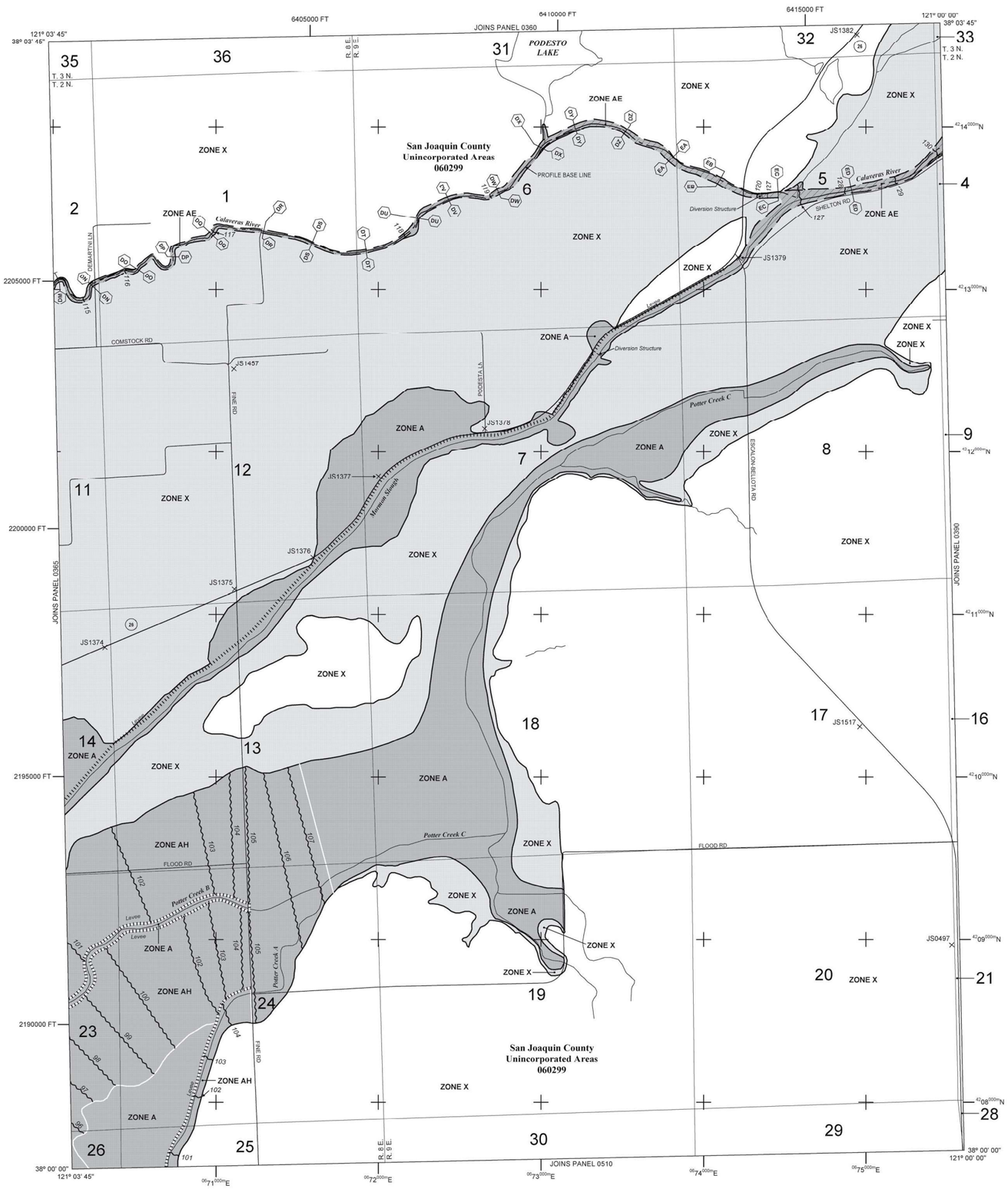
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LEGEND

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OTHER FLOOD AREAS

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MAP REPOSITORIES
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EFFECTIVE DATE OF COUNTYWIDE FLOOD INSURANCE RATE MAP PANEL
October 16, 2009

EFFECTIVE DATE(S) OF REVISION(S) TO THIS PANEL

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500 0 1000 2000 FEET

300 0 300 600 METERS

NFIP

NATIONAL FLOOD INSURANCE PROGRAM

PANEL 0370F

FIRM

FLOOD INSURANCE RATE MAP

SAN JOAQUIN COUNTY, CALIFORNIA AND INCORPORATED AREAS

PANEL 370 OF 950

(SEE MAP INDEX FOR FIRM PANEL LAYOUT)

CONTAINS:

COMMUNITY	NUMBER	PANEL	SUFFIX
SAN JOAQUIN COUNTY	060299	0370	F

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MAP NUMBER
06077C0370F

EFFECTIVE DATE
OCTOBER 16, 2009

Federal Emergency Management Agency

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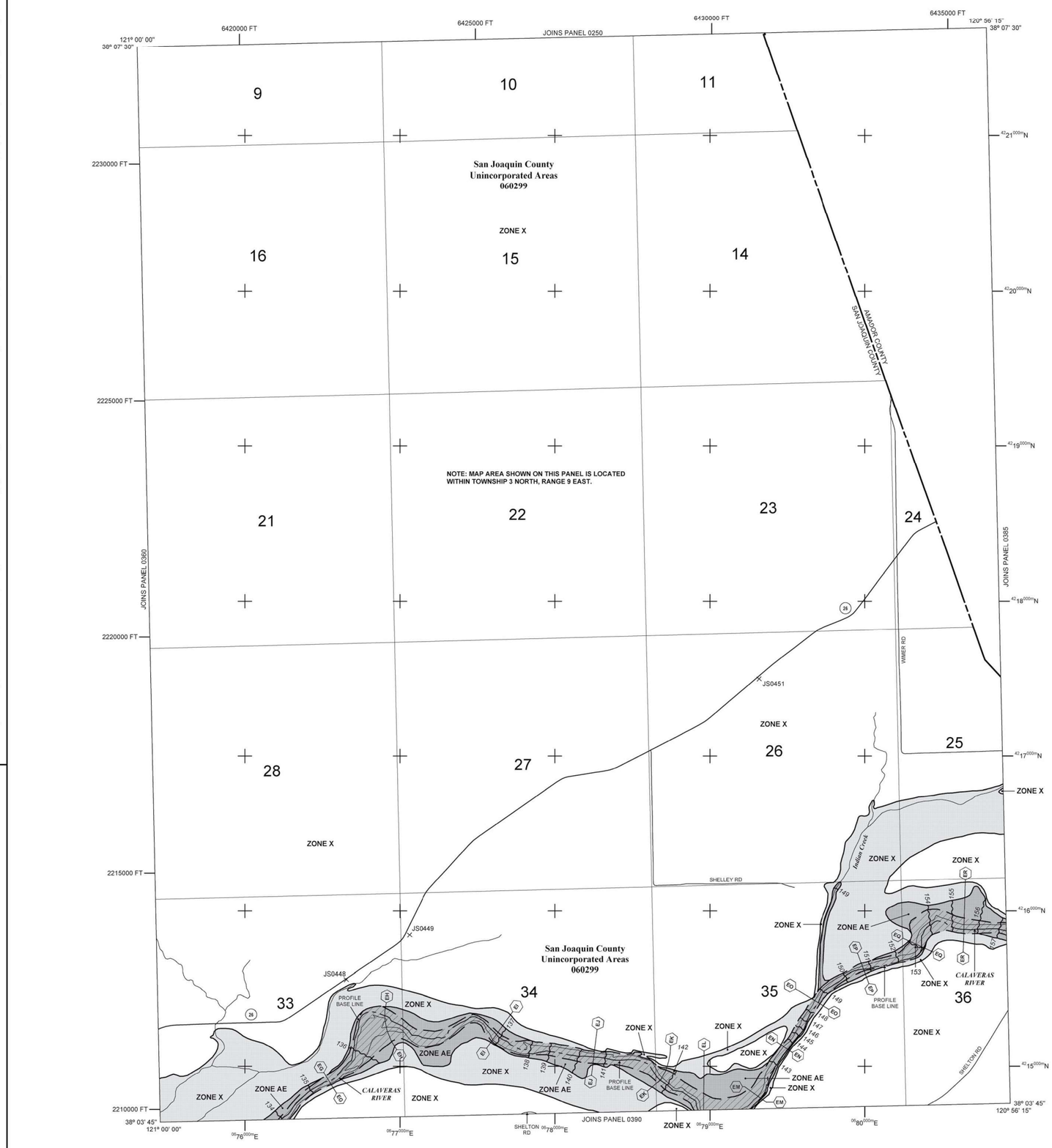
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LEGEND

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MAP REPOSITORIES
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EFFECTIVE DATE OF COUNTYWIDE FLOOD INSURANCE RATE MAP PANEL
October 16, 2009

EFFECTIVE DATE(S) OF REVISION(S) TO THIS PANEL

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500 0 1000 2000 FEET

300 0 300 600 METERS

NATIONAL FLOOD INSURANCE PROGRAM

PANEL 0380F

FIRM

FLOOD INSURANCE RATE MAP

SAN JOAQUIN COUNTY, CALIFORNIA AND INCORPORATED AREAS

PANEL 380 OF 950

(SEE MAP INDEX FOR FIRM PANEL LAYOUT)

CONTAINS:

COMMUNITY	NUMBER	PANEL	SUFFIX
SAN JOAQUIN COUNTY	060299	0380	F

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MAP NUMBER
06077C0380F

EFFECTIVE DATE
OCTOBER 16, 2009

Federal Emergency Management Agency

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Silver Spring, Maryland 20910-3282
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Base map information shown on this FIRM was derived from multiple sources. This information was compiled from the U.S. Geological Survey, 2002, National Geodetic Survey, 2002, City of Lathrop, 1997, City of Manteca, Department of Public Works, 2001, and San Joaquin County Community Development Department, 2008. Additional information was photogrammetrically compiled at a scale of 1:12,000 from U.S. Geological Survey aerial photography dated 1989, 1993 and 2002.

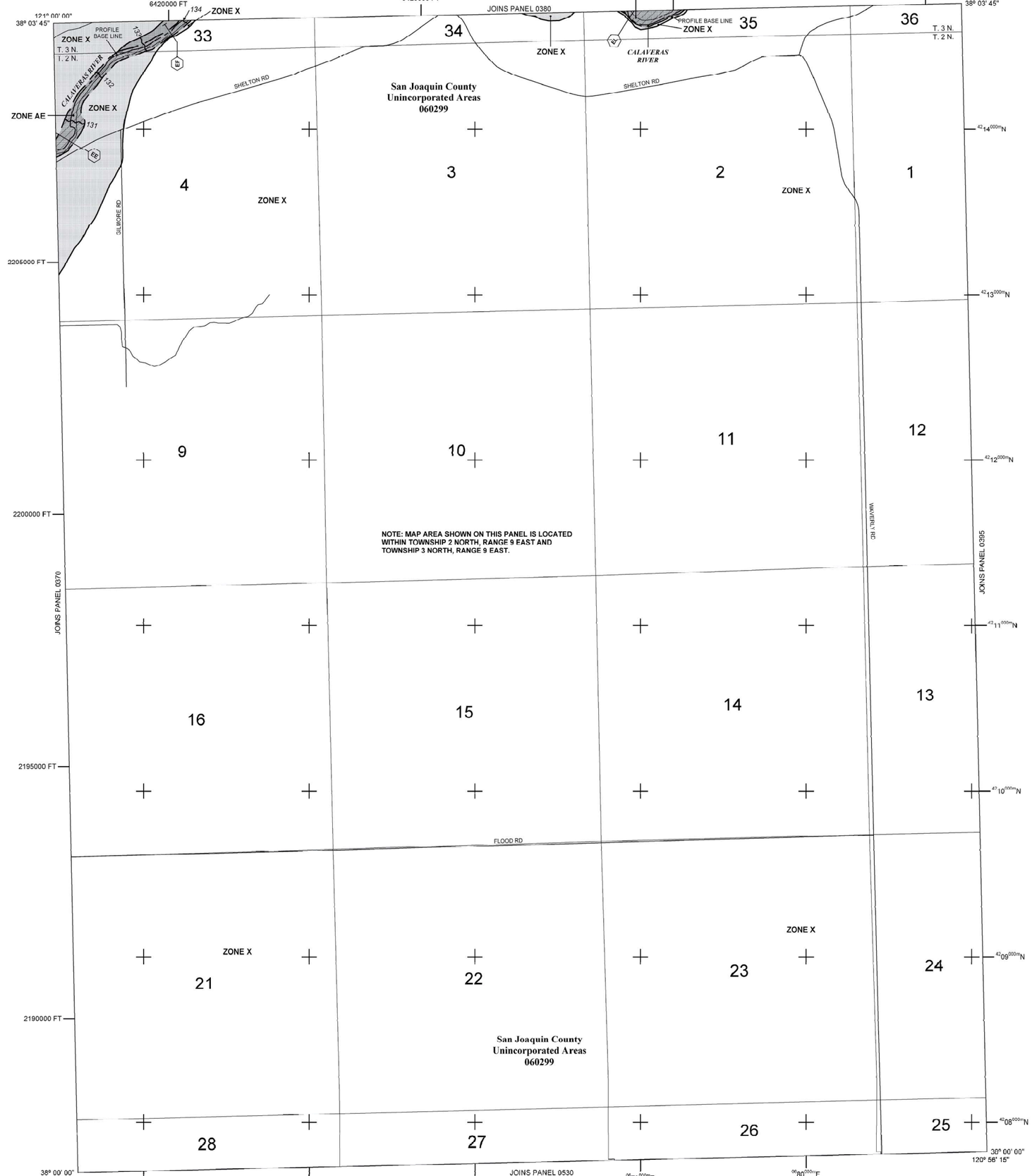
This map reflects more detailed and up-to-date **stream channel configurations** than those shown on the previous FIRM for this jurisdiction. The floodplains and floodways that were transferred from the previous FIRM may have been adjusted to conform to these new stream channel configurations. As a result, the Flood Profiles and Floodway Data tables in the Flood Insurance Study report (which contains authoritative hydraulic data) may reflect stream channel distances that differ from what is shown on this map.

Corporate limits shown on this map are based on the best data available at the time of publication. Because changes due to annexations or de-annexations may have occurred after this map was published, map users should contact appropriate community officials to verify current corporate limit locations.

Please refer to the separately printed **Map Index** for an overview map of the county showing the layout of map panels; community map repository addresses; and a Listing of Communities table containing National Flood Insurance Program dates for each community as well as a listing of the panels on which each community is located.

Contact the **FEMA Map Service Center** at 1-800-358-9616 for information on available products associated with this FIRM. Available products may include previously issued Letters of Map Change, a Flood Insurance Study report, and/or digital versions of this map. The FEMA Map Service Center may also be reached by Fax at 1-800-358-9620 and their website at http://www.msc.fema.gov/.

If you have **questions about this map** or questions concerning the National Flood Insurance Program in general, please call **1- 877- FEMA MAP** (1-877-336-2627) or visit the FEMA website at http://www.fema.gov/.



The 1% annual chance flood (100-year flood), also known as the base flood, is the flood that has a 1% chance of being equaled or exceeded in any given year. The Special Flood Hazard Area is the area subject to flooding by the 1% annual chance flood. Areas of Special Flood Hazard include Zones A, AE, AH, AO, AR, A99, V, and VE. The Base Flood Elevation is the water-surface elevation of the 1% annual chance flood.

ZONE A No Base Flood Elevations determined.

ZONE AE Base Flood Elevations determined.

ZONE AH Flood depths of 1 to 3 feet (usually areas of ponding); Base Flood Elevations determined.

ZONE AO Flood depths of 1 to 3 feet (usually sheet flow on sloping terrain); average depths determined. For areas of alluvial fan flooding, velocities also determined.

ZONE AR Special Flood Hazard Area formerly protected from the 1% annual chance flood by a flood control system that was subsequently decertified. Zone AR indicates that the former flood control system is being restored to provide protection from the 1% annual chance or greater flood.

ZONE A99 Area to be protected from 1% annual chance flood by a Federal flood protection system under construction; no Base Flood Elevations determined.

ZONE V Coastal flood zone with velocity hazard (wave action); no Base Flood Elevations determined.

ZONE VE Coastal flood zone with velocity hazard (wave action); Base Flood Elevations determined.

FLOODWAY AREAS IN ZONE AE

The floodway is the channel of a stream plus any adjacent floodplain areas that must be kept free of encroachment so that the 1% annual chance flood can be carried without substantial increases in flood heights.

OTHER FLOOD AREAS

ZONE X Areas of 0.2% annual chance flood; areas of 1% annual chance flood with average depths of less than 1 foot or with drainage areas less than 1 square mile; and areas protected by levees from 1% annual chance flood.

OTHER AREAS

ZONE X Areas determined to be outside the 0.2% annual chance floodplain.
ZONE D Areas in which flood hazards are undetermined, but possible.

COASTAL BARRIER RESOURCES SYSTEM (CBRS) AREAS

OTHERWISE PROTECTED AREAS (OPAs)

CBRS areas and OPAs are normally located within or adjacent to Special Flood Hazard Areas.

Floodplain boundary
Floodway boundary
Zone D Boundary
CBRS and OPA Boundary
Boundary dividing Special Flood Hazard Areas of different Base Flood Elevations, flood depths or flood velocities.
513 — Base Flood Elevation line and value; elevation in feet*
(EL 987) — Base Flood Elevation value where uniform within zone; elevation in feet*

*Referenced to the North American Vertical Datum of 1988

A — A — Cross section line
23 — 23 — Transect line
97° 07' 30", 32° 22' 30" — Geographic coordinates referenced to the North American Datum of 1983 (NAD 83), Western Hemisphere
4276000mE — 1000-meter Universal Transverse Mercator grid values, zone 10
600000 FT — 5000-foot grid ticks: California State Plane coordinate system, zone III (FIPSZONE 0403), Lambert Conformal Conic Projection
DX5510 × — Bench mark (see explanation in Notes to Users section of this FIRM panel)
M1.5 — River Mile

MAP REPOSITORIES
Refer to Map Repositories list on Map Index.

EFFECTIVE DATE OF COUNTYWIDE FLOOD INSURANCE RATE MAP PANEL
October 16, 2009

EFFECTIVE DATE(S) OF REVISION(S) TO THIS PANEL

For community map revision history prior to countywide mapping, refer to the Community Map History table located in the Flood Insurance Study report for this jurisdiction.

To determine if flood insurance is available in this community, contact your insurance agent or call the National Flood Insurance Program at 1-800-638-6620.

MAP SCALE 1" = 1000'
500 0 1000 2000 FEET
300 0 300 600 METERS

NFIP
NATIONAL FLOOD INSURANCE PROGRAM

PANEL 0390F

FIRM

FLOOD INSURANCE RATE MAP

SAN JOAQUIN COUNTY, CALIFORNIA AND INCORPORATED AREAS

PANEL 390 OF 950

(SEE MAP INDEX FOR FIRM PANEL LAYOUT)

CONTAINS:

COMMUNITY	NUMBER	PANEL	SUFFIX
SAN JOAQUIN COUNTY	060299	0390	F

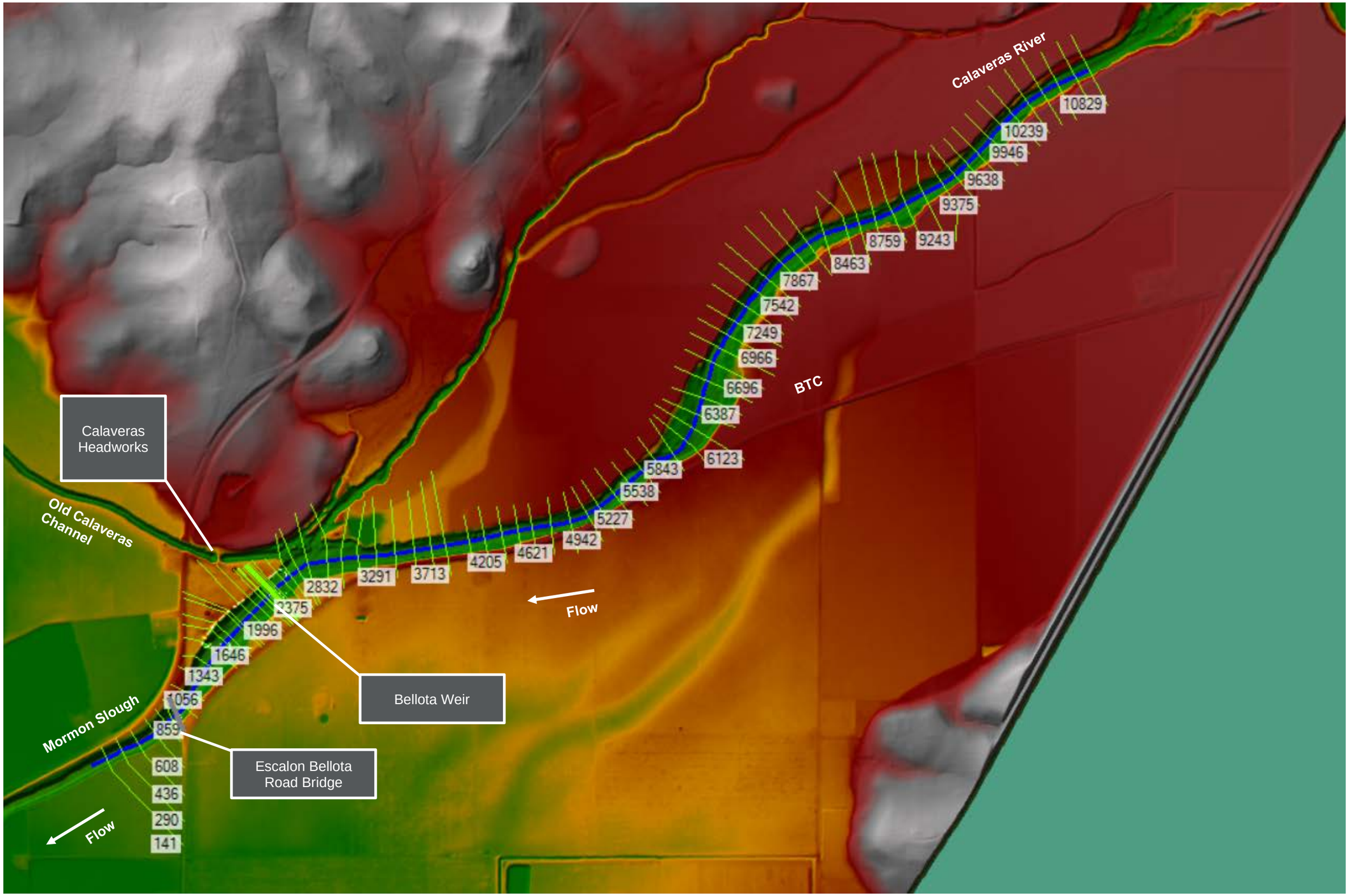
Notice to User: The **Map Number** shown below should be used when placing map orders; the **Community Number** shown above should be used on insurance applications for the subject community.

MAP NUMBER
06077C0390F

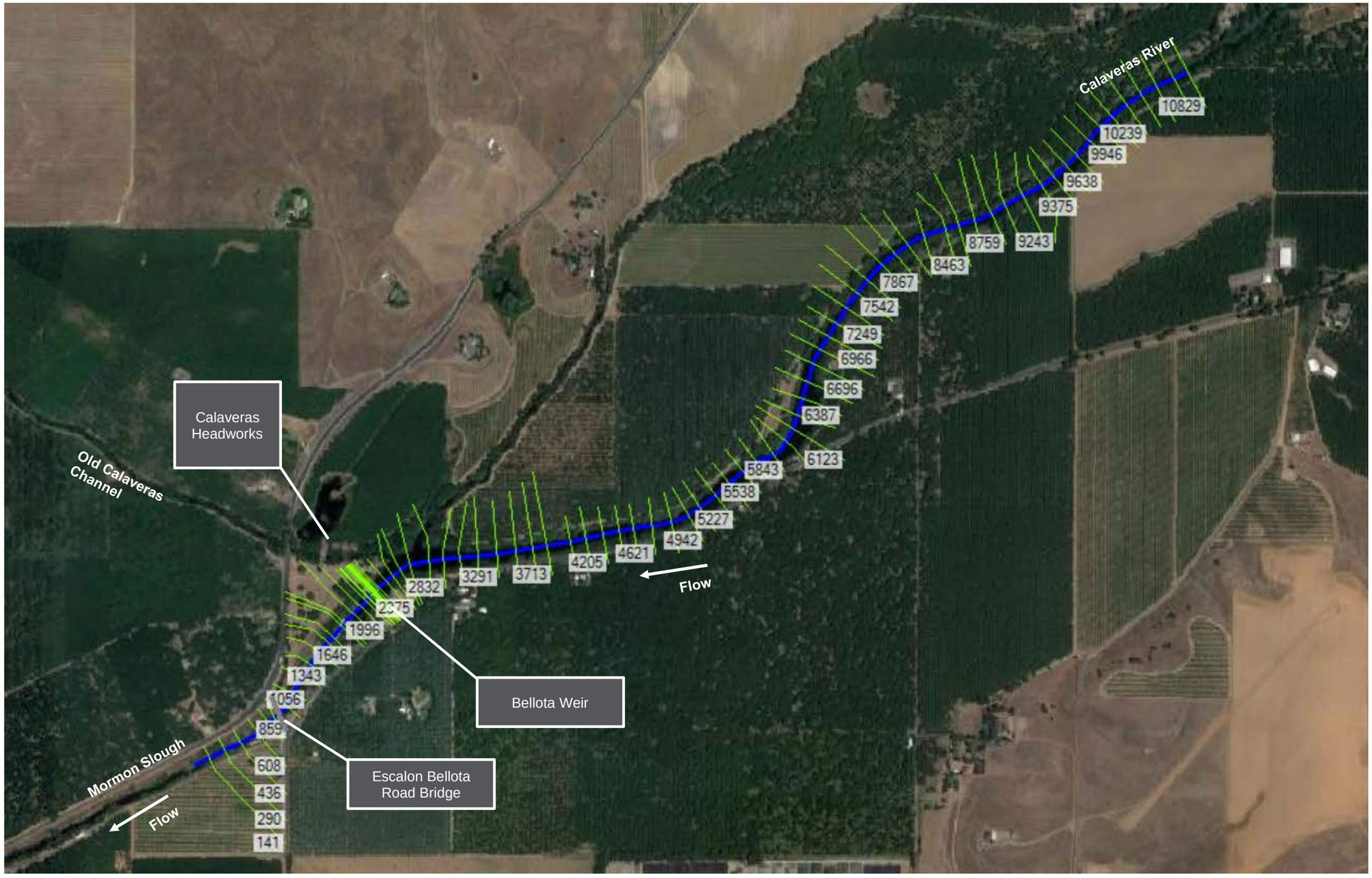
EFFECTIVE DATE
OCTOBER 16, 2009

Appendix B. Hydraulic Support Exhibits

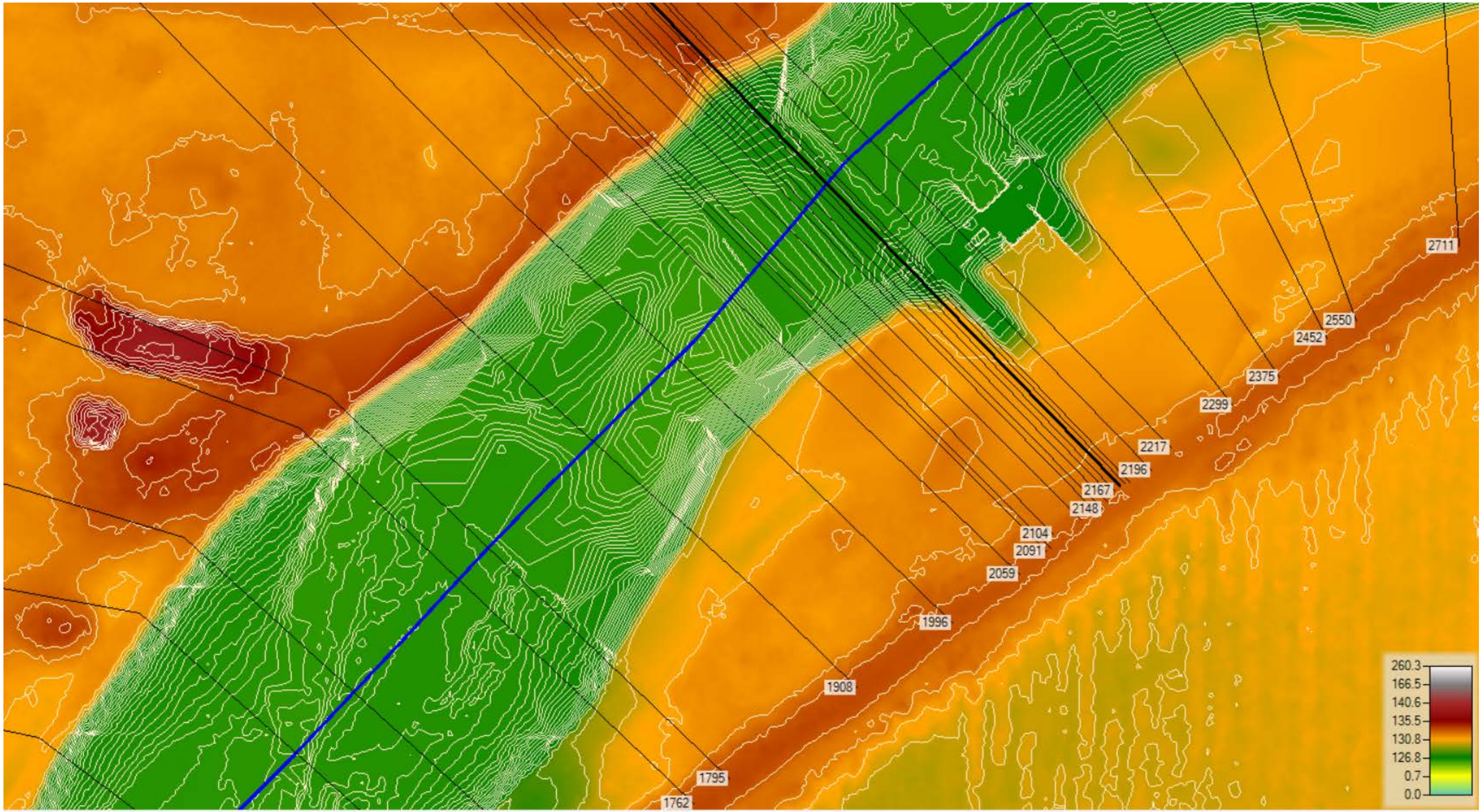
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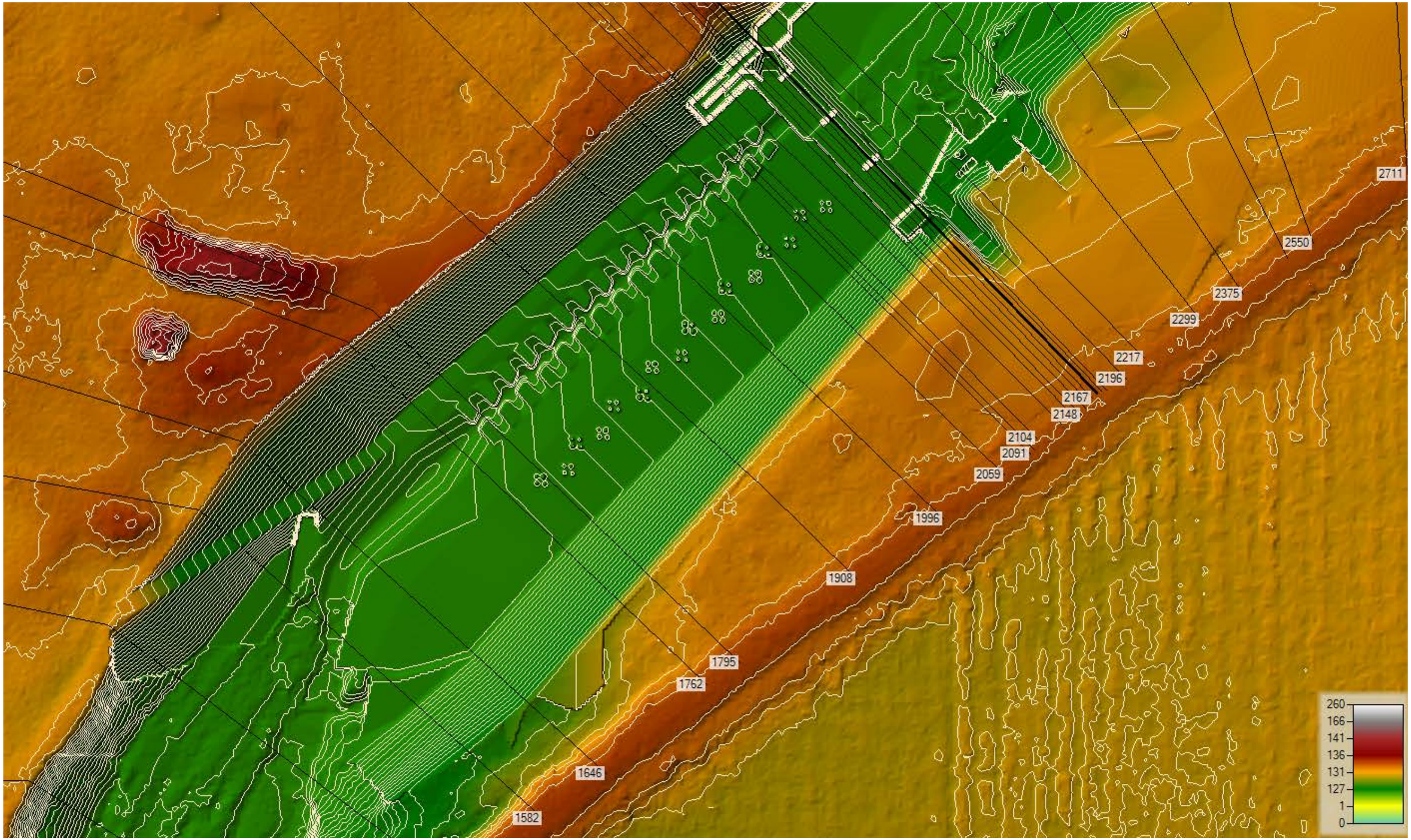
1D Modeling Layout and Existing Terrain Surface
EXHIBIT B1



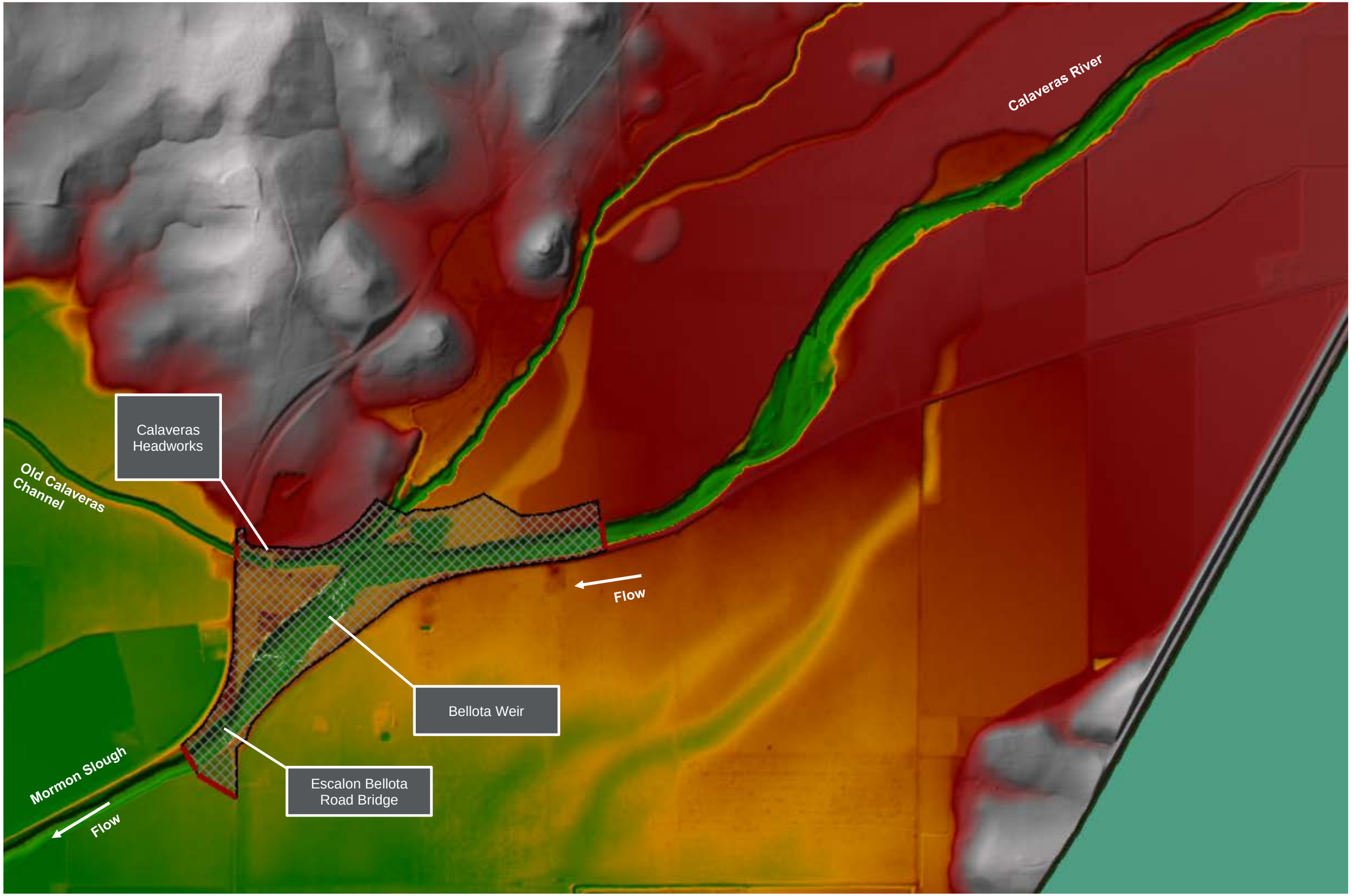
1D Modeling Layout and Aerial Image
EXHIBIT B2



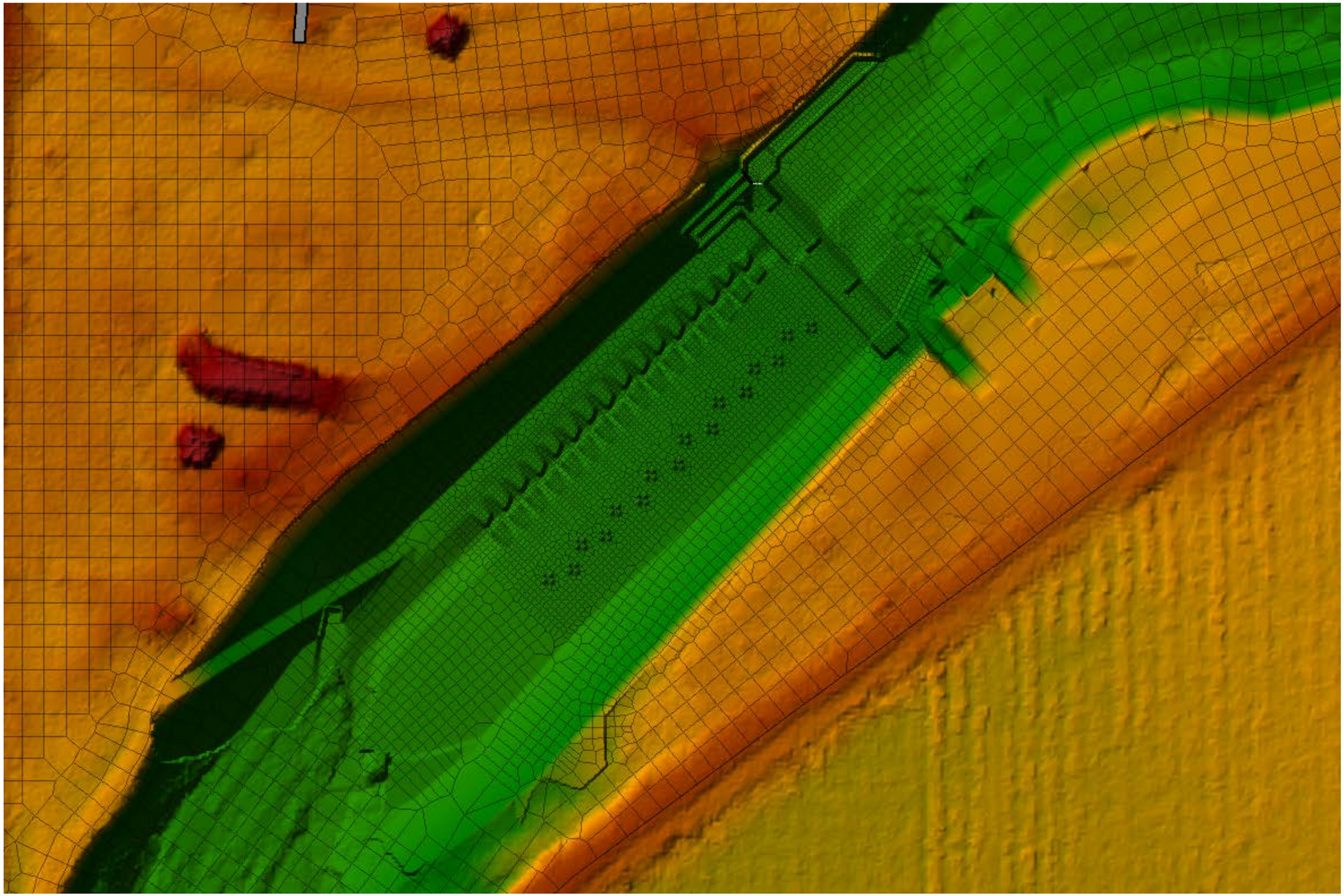
Project Area 1D Modeling Configuration, Existing Terrain
EXHIBIT B3



Project Area 1D Modeling Configuration, Proposed Terrain
EXHIBIT B4



2D Model Domain EXHIBIT B5



2D Computational Mesh in Project Area EXHIBIT B6

Appendix C. 1D HEC-RAS Results

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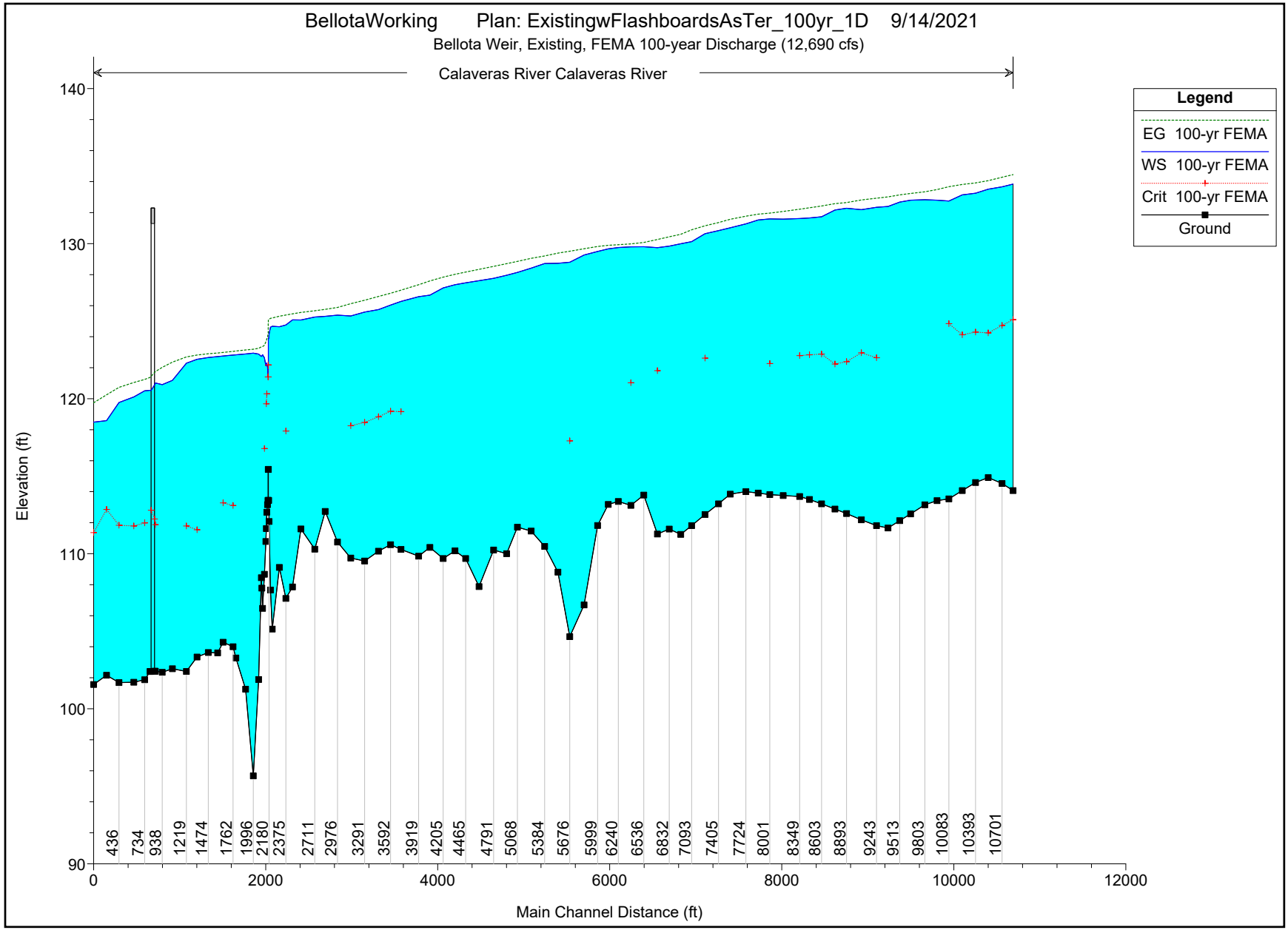
Existing Conditions: Hydraulic Output Table, Water Surface Profile, and Cross-Sections

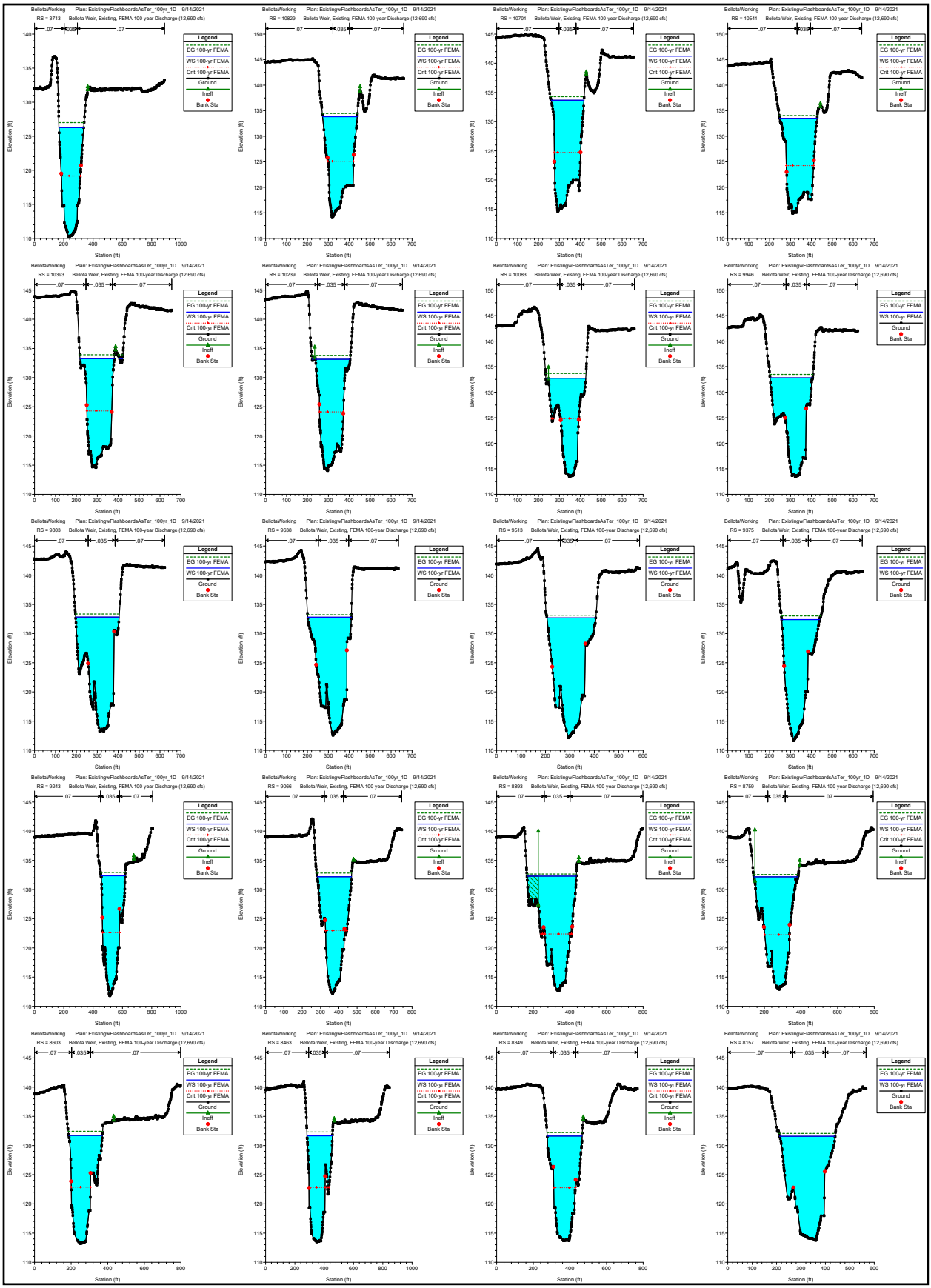
HEC-RAS Plan: ExistingwFlashboardsAsTer_100yr_1D River: Calaveras River Reach: Calaveras River Profile: 100-yr FEMA

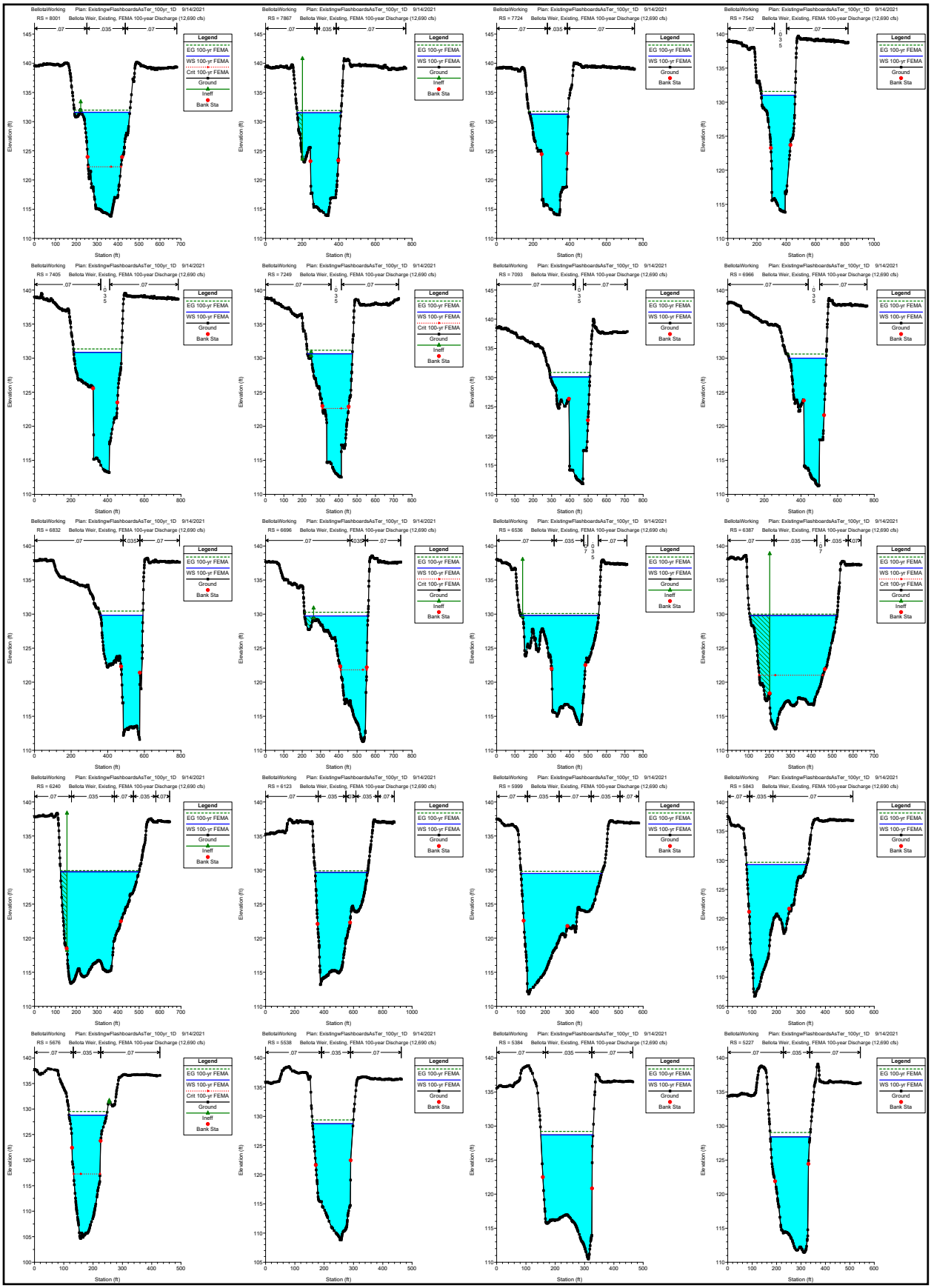
Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
Calaveras River	10829	100-yr FEMA	12690.00	114.07	133.839	125.09	134.45	0.001314	6.35	2124.25	165.34	0.28
Calaveras River	10701	100-yr FEMA	12690.00	114.53	133.665	124.72	134.29	0.001235	6.36	2074.13	156.17	0.28
Calaveras River	10541	100-yr FEMA	12690.00	114.91	133.512	124.24	134.06	0.001305	6.00	2238.73	179.23	0.27
Calaveras River	10393	100-yr FEMA	12690.00	114.59	133.254	124.30	133.92	0.000644	6.57	2009.46	177.49	0.29
Calaveras River	10239	100-yr FEMA	12690.00	114.07	133.145	124.12	133.82	0.000645	6.65	2013.05	171.63	0.29
Calaveras River	10083	100-yr FEMA	12690.00	113.54	132.745	124.84	133.67	0.000940	8.05	1911.10	190.89	0.35
Calaveras River	9946	100-yr FEMA	12690.00	113.42	132.804		133.48	0.000733	6.92	2272.82	203.68	0.30
Calaveras River	9803	100-yr FEMA	12690.00	113.16	132.829		133.34	0.000571	5.92	2480.30	207.96	0.26
Calaveras River	9638	100-yr FEMA	12690.00	112.57	132.803		133.23	0.000448	5.31	2548.49	208.35	0.23
Calaveras River	9513	100-yr FEMA	12690.00	112.13	132.679		133.14	0.001224	5.55	2446.19	199.94	0.24
Calaveras River	9375	100-yr FEMA	12690.00	111.66	132.398		133.02	0.000582	6.41	2190.78	181.37	0.28
Calaveras River	9243	100-yr FEMA	12690.00	111.81	132.343	122.64	132.93	0.000576	6.24	2218.77	165.40	0.27
Calaveras River	9066	100-yr FEMA	12690.00	112.19	132.189	122.96	132.82	0.000688	6.58	2275.29	186.73	0.28
Calaveras River	8893	100-yr FEMA	12690.00	112.59	132.282	122.39	132.65	0.000381	4.97	2798.14	269.54	0.22
Calaveras River	8759	100-yr FEMA	12690.00	112.88	132.167	122.23	132.58	0.000775	5.27	2710.12	240.15	0.23
Calaveras River	8603	100-yr FEMA	12690.00	113.22	131.733	122.87	132.43	0.000784	6.86	2143.78	183.73	0.30
Calaveras River	8463	100-yr FEMA	12690.00	113.50	131.654	122.83	132.31	0.000703	6.67	2168.60	173.20	0.29
Calaveras River	8349	100-yr FEMA	12690.00	113.69	131.618	122.77	132.22	0.000621	6.35	2265.47	191.15	0.28
Calaveras River	8157	100-yr FEMA	12690.00	113.75	131.577		132.08	0.000537	5.89	2547.21	227.41	0.26
Calaveras River	8001	100-yr FEMA	12690.00	113.81	131.593	122.26	131.97	0.000390	4.99	2689.11	257.17	0.23
Calaveras River	7867	100-yr FEMA	12690.00	113.91	131.523		131.90	0.000720	5.05	2743.15	239.59	0.23
Calaveras River	7724	100-yr FEMA	12690.00	114.00	131.277		131.77	0.000900	5.76	2416.90	208.90	0.26
Calaveras River	7542	100-yr FEMA	12690.00	113.84	131.020		131.57	0.001292	6.12	2328.30	230.39	0.28
Calaveras River	7405	100-yr FEMA	12690.00	113.22	130.839		131.35	0.001784	6.00	2439.96	261.54	0.28
Calaveras River	7249	100-yr FEMA	12690.00	112.53	130.637	122.62	131.15	0.000991	5.89	2410.00	245.60	0.28
Calaveras River	7093	100-yr FEMA	12690.00	111.81	130.128		130.90	0.002447	7.27	2013.34	217.11	0.32
Calaveras River	6966	100-yr FEMA	12690.00	111.25	129.988		130.60	0.001571	6.56	2234.40	203.92	0.29
Calaveras River	6832	100-yr FEMA	12690.00	111.59	129.836		130.44	0.000931	6.67	2458.57	232.17	0.29
Calaveras River	6696	100-yr FEMA	12690.00	111.27	129.741	121.81	130.27	0.001330	5.95	2437.84	346.13	0.28
Calaveras River	6536	100-yr FEMA	12690.00	113.78	129.799		130.07	0.000469	4.43	3500.82	419.28	0.21
Calaveras River	6387	100-yr FEMA	12690.00	113.12	129.794	121.03	129.99	0.000382	3.57	3647.77	416.62	0.18
Calaveras River	6240	100-yr FEMA	12690.00	113.37	129.748		129.93	0.000322	3.49	3862.18	375.60	0.17
Calaveras River	6123	100-yr FEMA	12690.00	113.18	129.666		129.89	0.000374	3.92	3587.20	357.41	0.19
Calaveras River	5999	100-yr FEMA	12690.00	111.82	129.498		129.82	0.000754	4.77	3114.14	329.48	0.23
Calaveras River	5843	100-yr FEMA	12690.00	106.70	129.264		129.67	0.000990	5.22	2652.75	246.27	0.24
Calaveras River	5676	100-yr FEMA	12690.00	104.65	128.804	117.28	129.51	0.000666	6.74	1939.52	128.05	0.27
Calaveras River	5538	100-yr FEMA	12690.00	108.81	128.730		129.38	0.000942	6.51	2002.05	138.38	0.28
Calaveras River	5384	100-yr FEMA	12690.00	110.47	128.714		129.21	0.000742	5.64	2272.86	177.91	0.27
Calaveras River	5227	100-yr FEMA	12690.00	111.46	128.419		129.05	0.001166	6.41	2046.30	162.34	0.30
Calaveras River	5068	100-yr FEMA	12690.00	111.71	128.145		128.86	0.001156	6.79	1918.31	140.60	0.31
Calaveras River	4942	100-yr FEMA	12690.00	110.00	127.960		128.70	0.001211	6.94	1879.04	136.13	0.31
Calaveras River	4791	100-yr FEMA	12690.00	110.24	127.763		128.53	0.001062	7.08	1878.06	143.75	0.32
Calaveras River	4621	100-yr FEMA	12690.00	107.88	127.606		128.34	0.001051	6.91	1889.73	142.94	0.31
Calaveras River	4465	100-yr FEMA	12690.00	109.69	127.469		128.17	0.001032	6.73	1944.34	152.39	0.31
Calaveras River	4342	100-yr FEMA	12690.00	110.19	127.349		128.02	0.001234	6.63	2008.94	165.95	0.31
Calaveras River	4205	100-yr FEMA	12690.00	109.69	127.144		127.84	0.001376	6.79	2018.94	186.52	0.32
Calaveras River	4049	100-yr FEMA	12690.00	110.41	126.679		127.60	0.001499	7.89	1815.87	178.91	0.38
Calaveras River	3919	100-yr FEMA	12690.00	109.84	126.578		127.35	0.001655	7.15	1912.52	169.78	0.34
Calaveras River	3713	100-yr FEMA	12690.00	110.28	126.269	119.17	127.00	0.001627	6.89	1924.14	168.65	0.33
Calaveras River	3592	100-yr FEMA	12690.00	110.58	126.033	119.19	126.80	0.001526	7.08	1880.73	163.80	0.34
Calaveras River	3452	100-yr FEMA	12690.00	110.16	125.745	118.83	126.59	0.001324	7.49	1822.52	156.23	0.35
Calaveras River	3291	100-yr FEMA	12690.00	109.53	125.589	118.47	126.35	0.001406	7.01	1866.74	161.01	0.34
Calaveras River	3130	100-yr FEMA	12690.00	109.72	125.330	118.26	126.13	0.001284	7.17	1808.37	152.37	0.34
Calaveras River	2976	100-yr FEMA	12690.00	110.75	125.383		125.88	0.000808	5.68	2263.38	217.57	0.28
Calaveras River	2832	100-yr FEMA	12690.00	112.73	125.308		125.76	0.000700	5.40	2365.26	265.62	0.28
Calaveras River	2711	100-yr FEMA	12690.00	110.29	125.263		125.67	0.000581	5.14	2547.25	404.72	0.27
Calaveras River	2550	100-yr FEMA	12690.00	111.60	125.074		125.56	0.000707	5.65	2404.79	346.35	0.30
Calaveras River	2452	100-yr FEMA	12690.00	107.86	125.083		125.47	0.000524	5.00	2577.47	352.18	0.25
Calaveras River	2375	100-yr FEMA	12690.00	107.12	124.754	117.91	125.39	0.000985	6.45	2075.19	286.92	0.32
Calaveras River	2299	100-yr FEMA	12690.00	109.12	124.639		125.32	0.000925	6.61	1944.17	205.36	0.33
Calaveras River	2217	100-yr FEMA	12690.00	105.14	124.667		125.20	0.000836	5.88	2216.23	204.56	0.28
Calaveras River	2196	100-yr FEMA	12690.00	107.66	124.605		125.18	0.000745	6.09	2154.91	228.66	0.30
Calaveras River	2180	100-yr FEMA	12690.00	112.08	124.160		125.12	0.001768	7.92	1690.54	203.02	0.44
Calaveras River	2176	100-yr FEMA	12690.00	113.44	123.868		125.09	0.000391	8.91	1447.69	192.56	0.54
Calaveras River	2172	100-yr FEMA	12690.00	113.44	123.865		125.09	0.000391	8.91	1447.13	192.54	0.54
Calaveras River	2171	100-yr FEMA	12690.00	115.44	122.300	122.17	124.95	0.001239	13.20	995.25	180.74	0.93
Calaveras River	2170.6	100-yr FEMA	12690.00	115.44	122.170	122.17	124.95	0.001333	13.50	971.87	179.76	0.96
Calaveras River	2170	100-yr FEMA	12690.00	113.44	121.421	121.40	124.20	0.001111	14.16	1002.80	169.01	0.91
Calaveras River	2167	100-yr FEMA	12690.00	113.44	121.415	121.40	124.20	0.001114	14.17	1001.81	169.01	0.91
Calaveras River	2162	100-yr FEMA	12690.00	113.17	122.058		123.91	0.000604	11.20	1211.20	181.45	0.70
Calaveras River	2154	100-yr FEMA	12690.00	112.67	122.147	120.32	123.87	0.000509	10.80	1276.19	180.18	0.65
Calaveras River	2148	100-yr FEMA	12690.00	111.62	122.300	119.67	123.71	0.000391	9.82	1399.33	179.92	0.56
Calaveras River	2143	100-yr FEMA	12690.00	110.79	122.131		123.67	0.000359	10.05	1457.63	178.11	0.56
Calaveras River	2127	100-yr FEMA	12690.00	108.67	122.631	116.78	123.45	0.000175	7.46	1861.14	181.17	0.37
Calaveras River	2104	100-yr FEMA	12690.00	106.48	122.837		123.36	0.000093	5.88	2289.67	183.20	0.27
Calaveras River	2096	100-yr FEMA	12690.00	107.78	122.749		123.35	0.000773	6.29	2174.25	183.61	0.30
Calaveras River	2091	100-yr FEMA	12690.00	108.45	122.724		123.34	0.001005	6.41	2147.42	184.67	0.31
Calaveras River	2059	100-yr FEMA	12690.00	101.89	122.876		123.25	0.000346	4.92	2614.93	187.21	0.20
Calaveras River	1996	100-yr FEMA	12690.00	95.67	122.940		123.19	0.000430	4.02	3252.38	190.67	0.16
Calaveras River	1908	100-yr FEMA	12690.00	101.26	122.880		123.14	0.000576	4.14	3143.89	207.78	0.18

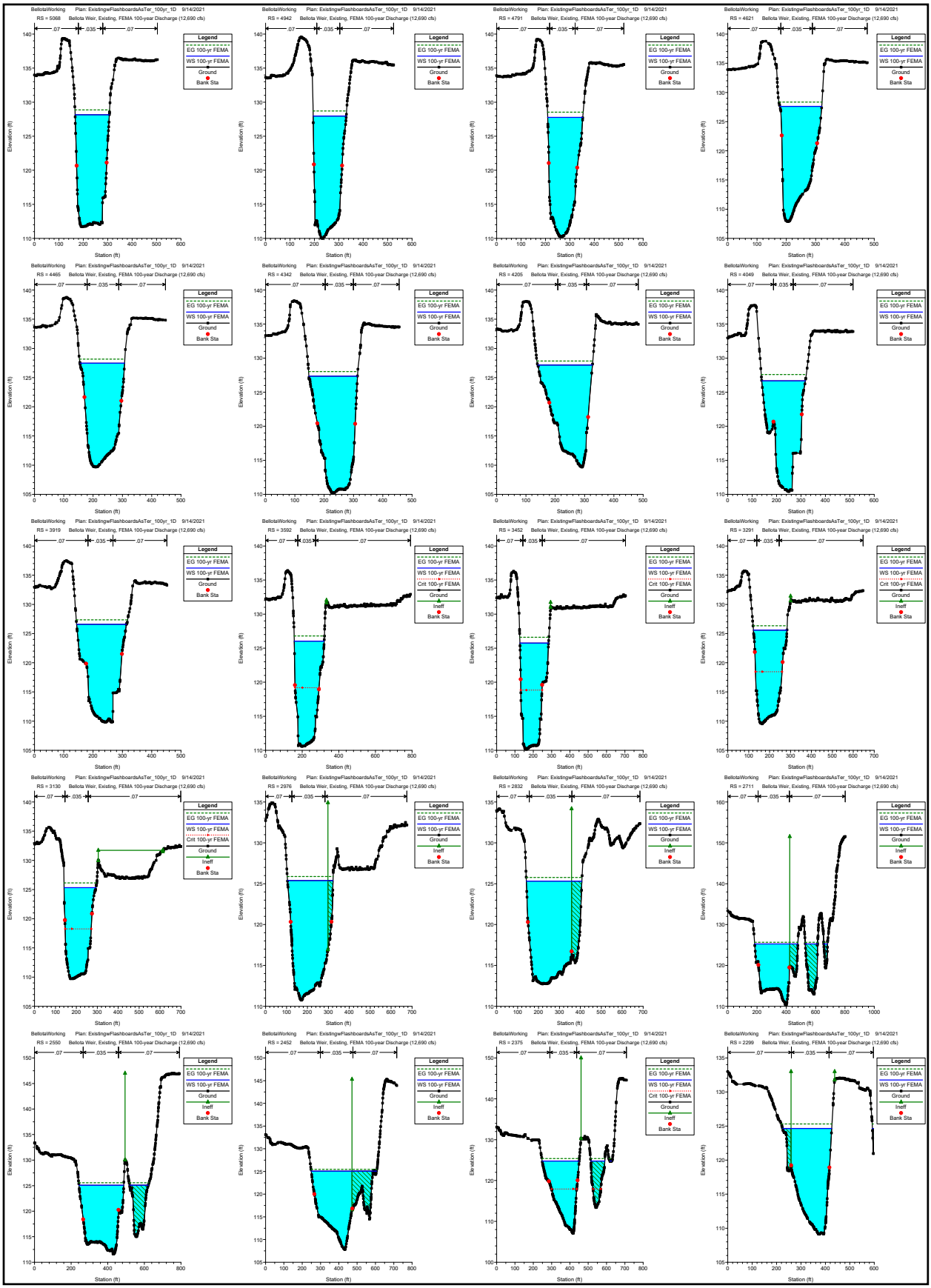
HEC-RAS Plan: ExistingwFlashboardsAsTer_100yr_1D River: Calaveras River Reach: Calaveras River Profile: 100-yr FEMA (Continued)

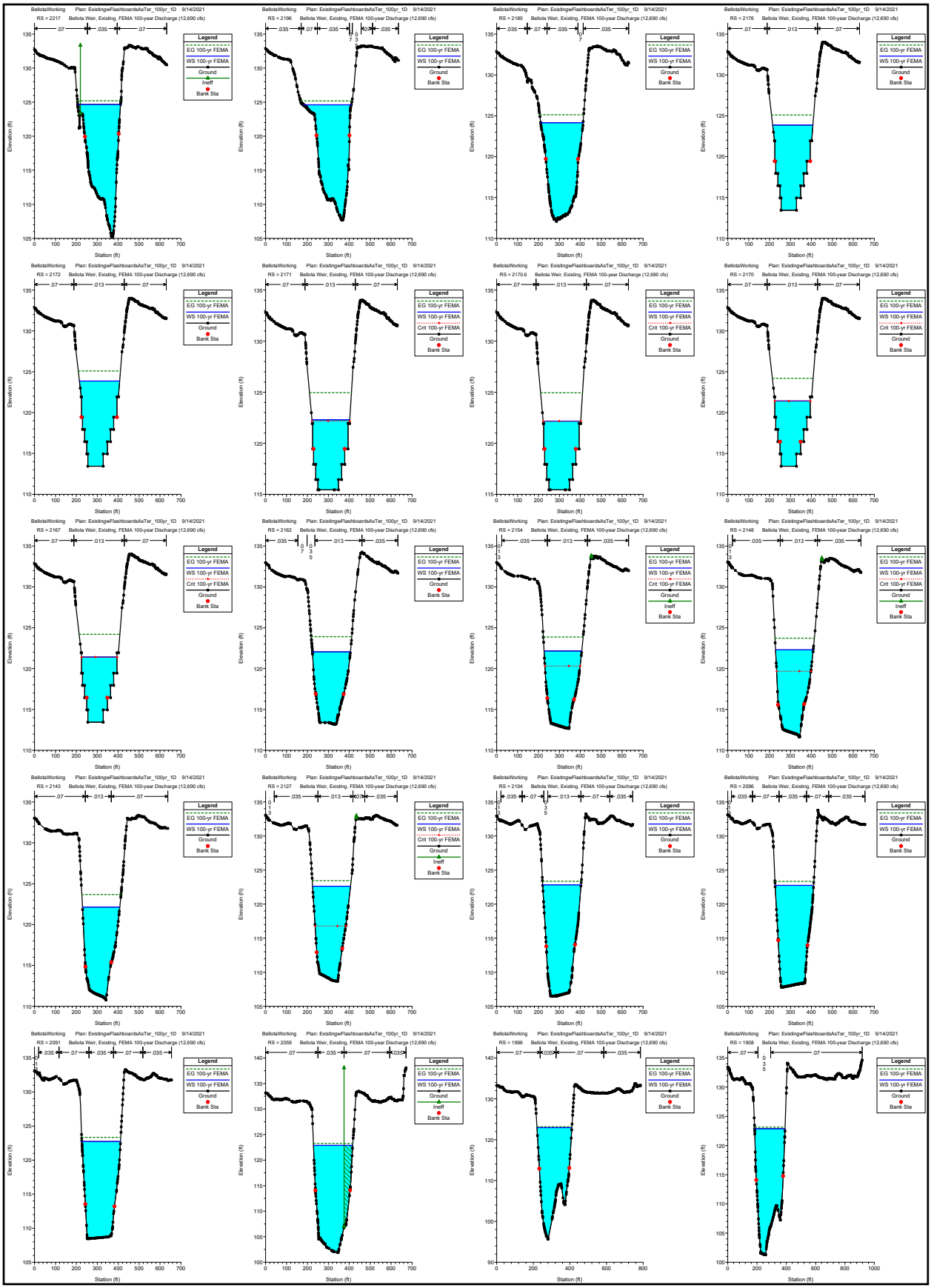
Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
Calaveras River	1795	100-yr FEMA	12690.00	103.28	122.823		123.07	0.000576	4.09	3292.97	255.84	0.19
Calaveras River	1762	100-yr FEMA	12690.00	104.00	122.817	113.11	123.05	0.000702	3.92	3390.04	267.57	0.18
Calaveras River	1646	100-yr FEMA	12690.00	104.29	122.750	113.28	122.98	0.000501	3.90	3446.61	277.62	0.18
Calaveras River	1582	100-yr FEMA	12690.00	103.61	122.717		122.95	0.000533	3.88	3377.75	261.48	0.18
Calaveras River	1474	100-yr FEMA	12690.00	103.63	122.658		122.89	0.000405	3.99	3308.95	261.97	0.18
Calaveras River	1343	100-yr FEMA	12690.00	103.34	122.537	111.55	122.82	0.000803	4.31	3104.78	253.25	0.19
Calaveras River	1219	100-yr FEMA	12690.00	102.41	122.291	111.79	122.69	0.001051	5.13	2570.77	196.98	0.22
Calaveras River	1056	100-yr FEMA	12690.00	102.58	121.182		122.36	0.002617	8.89	1564.85	126.21	0.39
Calaveras River	938	100-yr FEMA	12690.00	102.35	120.907		122.02	0.002795	8.59	1576.70	122.12	0.38
Calaveras River	859	100-yr FEMA	12690.00	102.42	121.012	111.87	121.75	0.001527	6.94	1926.24	141.77	0.30
Calaveras River	850		Bridge									
Calaveras River	796	100-yr FEMA	12690.00	102.40	120.538		121.37	0.001294	7.42	1811.56	145.90	0.33
Calaveras River	734	100-yr FEMA	12690.00	101.87	120.505	111.99	121.24	0.001360	6.97	1923.75	145.94	0.31
Calaveras River	608	100-yr FEMA	12690.00	101.71	120.114	111.79	121.05	0.001455	7.78	1671.00	111.49	0.34
Calaveras River	436	100-yr FEMA	12690.00	101.69	119.752	111.84	120.73	0.002317	7.98	1643.66	114.28	0.35
Calaveras River	290	100-yr FEMA	12690.00	102.16	118.588	112.86	120.25	0.003541	10.44	1283.00	100.13	0.47
Calaveras River	141	100-yr FEMA	12690.00	101.57	118.487	111.37	119.73	0.002001	9.00	1457.33	103.29	0.40

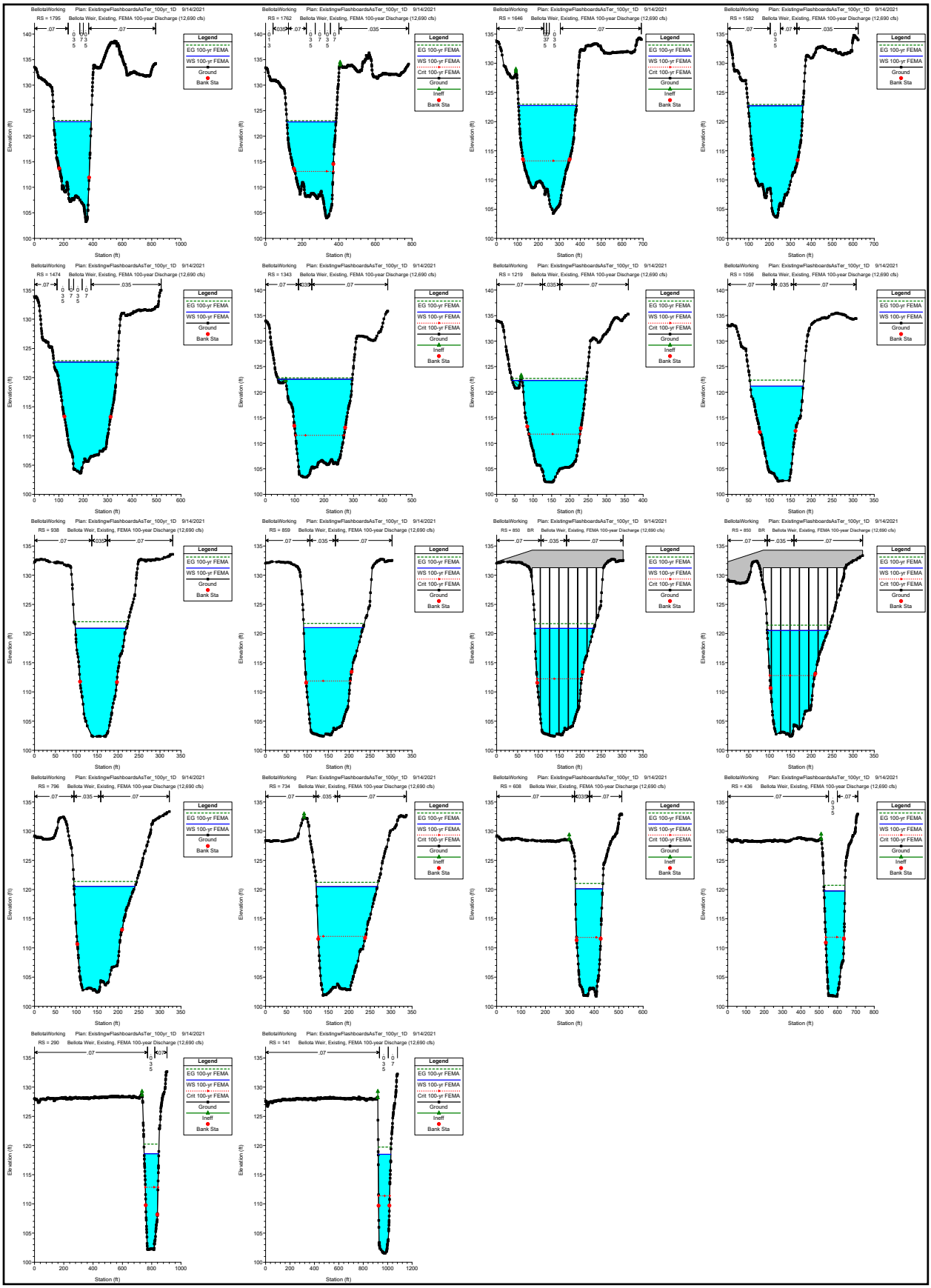

BellotaWorking Plan: ExistingwFlashboardsAsTer_100yr_1D 9/14/2021
Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
Calaveras River Calaveras River
Legend
EG 100-yr FEMA
WS 100-yr FEMA
Crit 100-yr FEMA
Ground
Elevation (ft)
Main Channel Distance (ft)
140
130
120
110
100
90
0
2000
4000
6000
8000
10000
12000
436
734
938
1219
1474
1762
1996
2180
2375
2711
2976
3291
3592
3919
4205
4465
4791
5068
5384
5676
5999
6240
6536
6832
7093
7405
7724
8001
8349
8603
8893
9243
9513
9803
10083
10393
10701


BellotaWorking Plan: ExistingwFlashboardsAsTer_100yr_1D 9/14/2021
RS = 3713 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 10829 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 10701 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 10541 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 10393 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 10239 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 10083 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 9946 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 9803 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 9638 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 9513 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 9375 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 9243 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 9066 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 8893 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 8759 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 8603 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 8463 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 8349 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 8157 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
.07
.035
.07
Legend
EG 100-yr FEMA
WS 100-yr FEMA
Crit 100-yr FEMA
Ground
Ineff
Bank Sta
Elevation (ft)
Station (ft)


BellotaWorking Plan: ExistingwFlashboardsAsTer_100yr_1D 9/14/2021
RS = 8001 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 7867 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 7724 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 7542 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 7405 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 7249 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 7093 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 6966 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 6832 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 6696 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 6536 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 6387 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 6240 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 6123 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 5999 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 5843 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 5676 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 5538 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 5384 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 5227 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
Legend
EG 100-yr FEMA
WS 100-yr FEMA
Crit 100-yr FEMA
Ground
Ineff
Bank Sta
Elevation (ft)
Station (ft)


BellotaWorking Plan: ExistingwFlashboardsAsTer_100yr_1D 9/14/2021
RS = 5068 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 4942 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 4791 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 4621 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 4465 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 4342 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 4205 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 4049 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 3919 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 3592 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 3452 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 3291 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 3130 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 2976 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 2832 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 2711 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 2550 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 2452 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 2375 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 2299 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
.07 .035 .07
Legend
EG 100-yr FEMA
WS 100-yr FEMA
Crit 100-yr FEMA
Ground
Ineff
Bank Sta
Elevation (ft)
Station (ft)


BellotaWorking Plan: ExistingwFlashboardsAsTer_100yr_1D 9/14/2021
RS = 2217 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 2196 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 2180 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 2176 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 2172 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 2171 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 2170.6 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 2170 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 2167 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 2162 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 2154 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 2148 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 2143 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 2127 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 2104 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 2096 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 2091 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 2059 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 1996 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 1908 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
Legend
EG 100-yr FEMA
WS 100-yr FEMA
Crit 100-yr FEMA
Ground
Ineff
Bank Sta
Elevation (ft)
Station (ft)


BellotaWorking Plan: ExistingwFlashboardsAsTer_100yr_1D 9/14/2021
RS = 1795 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 1762 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 1646 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 1582 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 1474 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 1343 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 1219 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 1056 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 938 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 859 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 850 BR Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 850 BR Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 796 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 734 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 608 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 436 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 290 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
RS = 141 Bellota Weir, Existing, FEMA 100-year Discharge (12,690 cfs)
Legend
EG 100-yr FEMA
WS 100-yr FEMA
Crit 100-yr FEMA
Ground
Ineff
Bank Sta
Elevation (ft)
Station (ft)

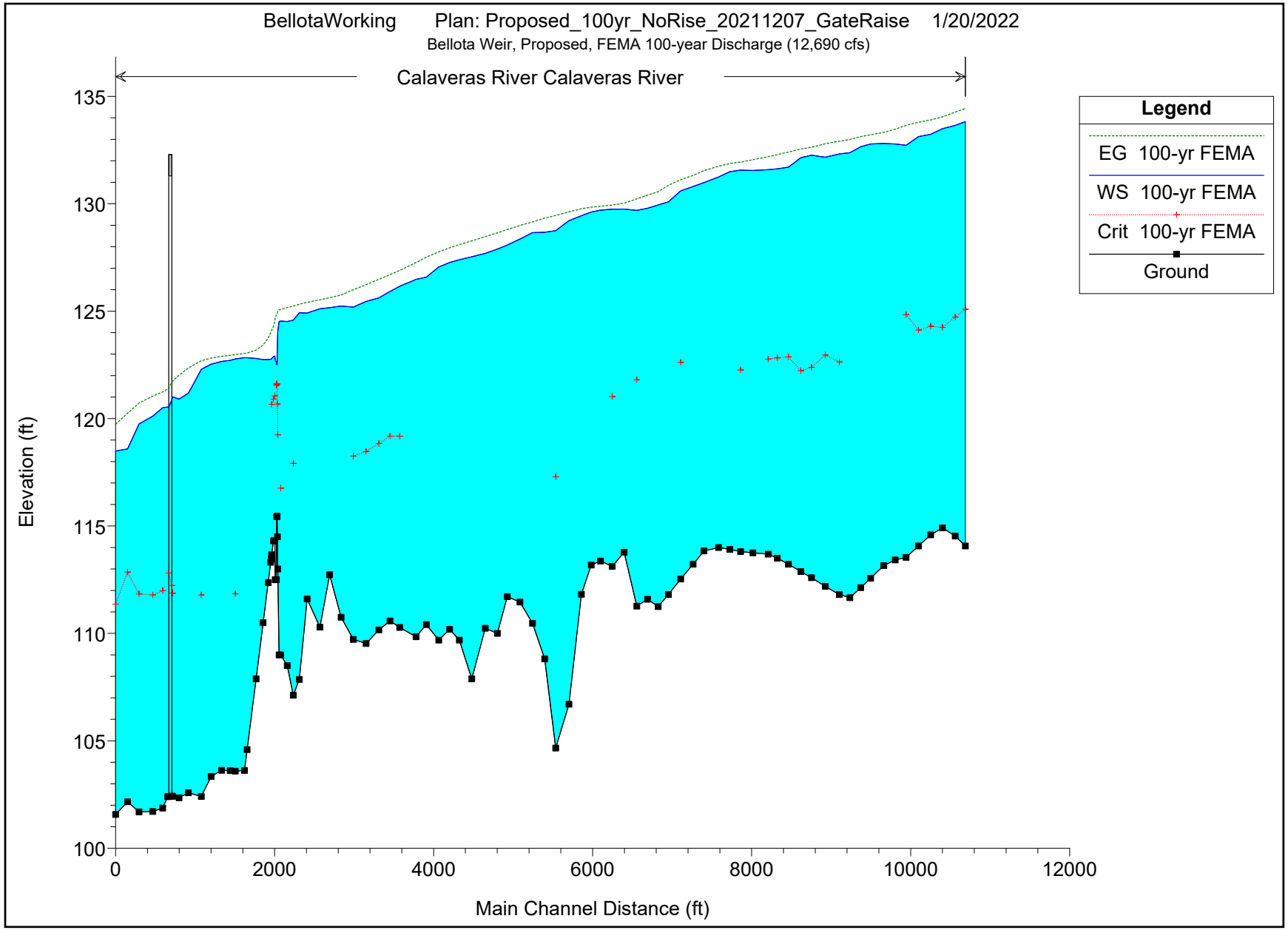
Proposed Conditions: Hydraulic Output Table, Water Surface Profile, and Cross-sections

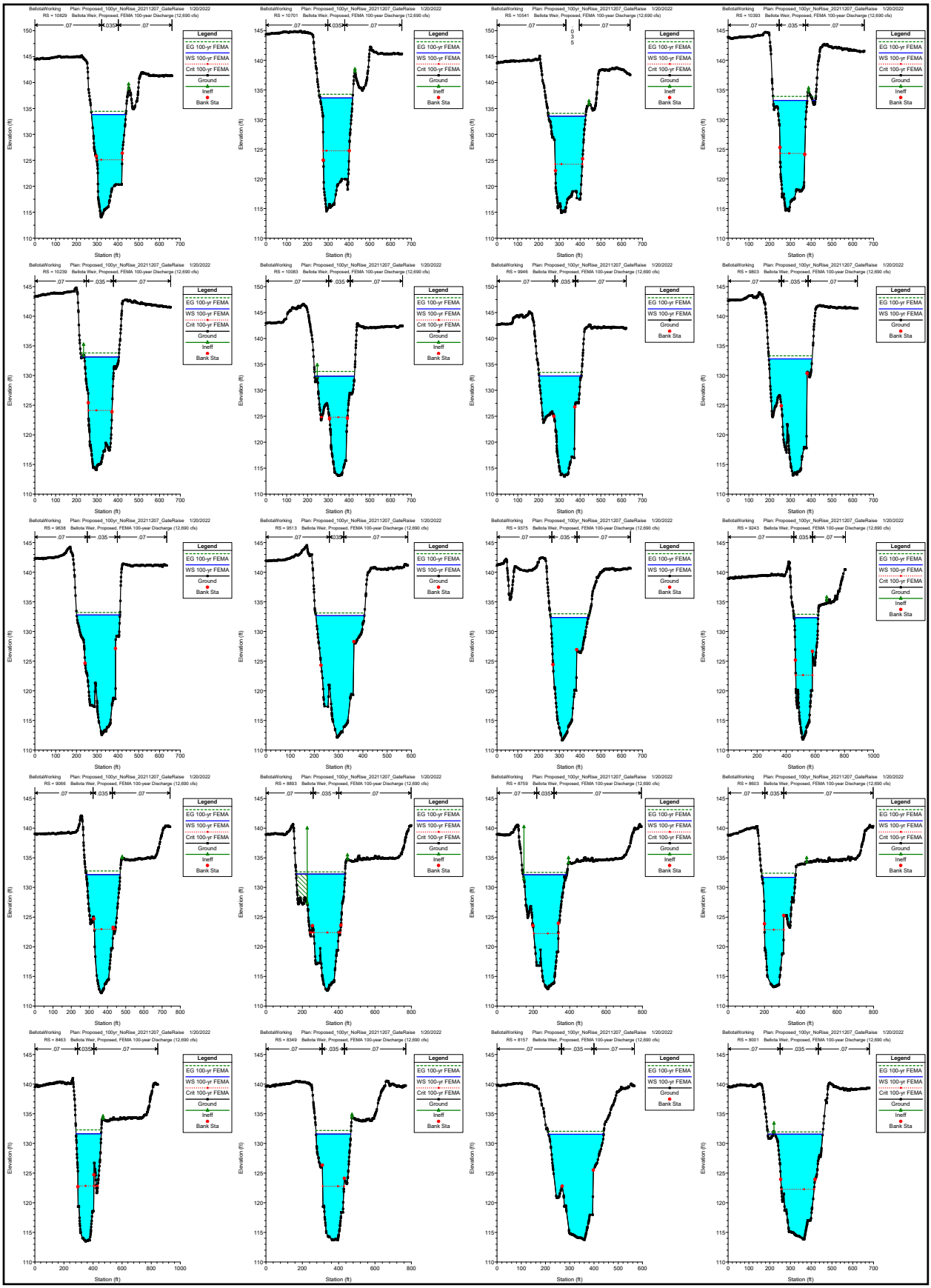
HEC-RAS Plan: Proposed_100yr_NoRise_20211207_GateRaise River: Calaveras River Reach: Calaveras River Profile: 100-yr FEMA

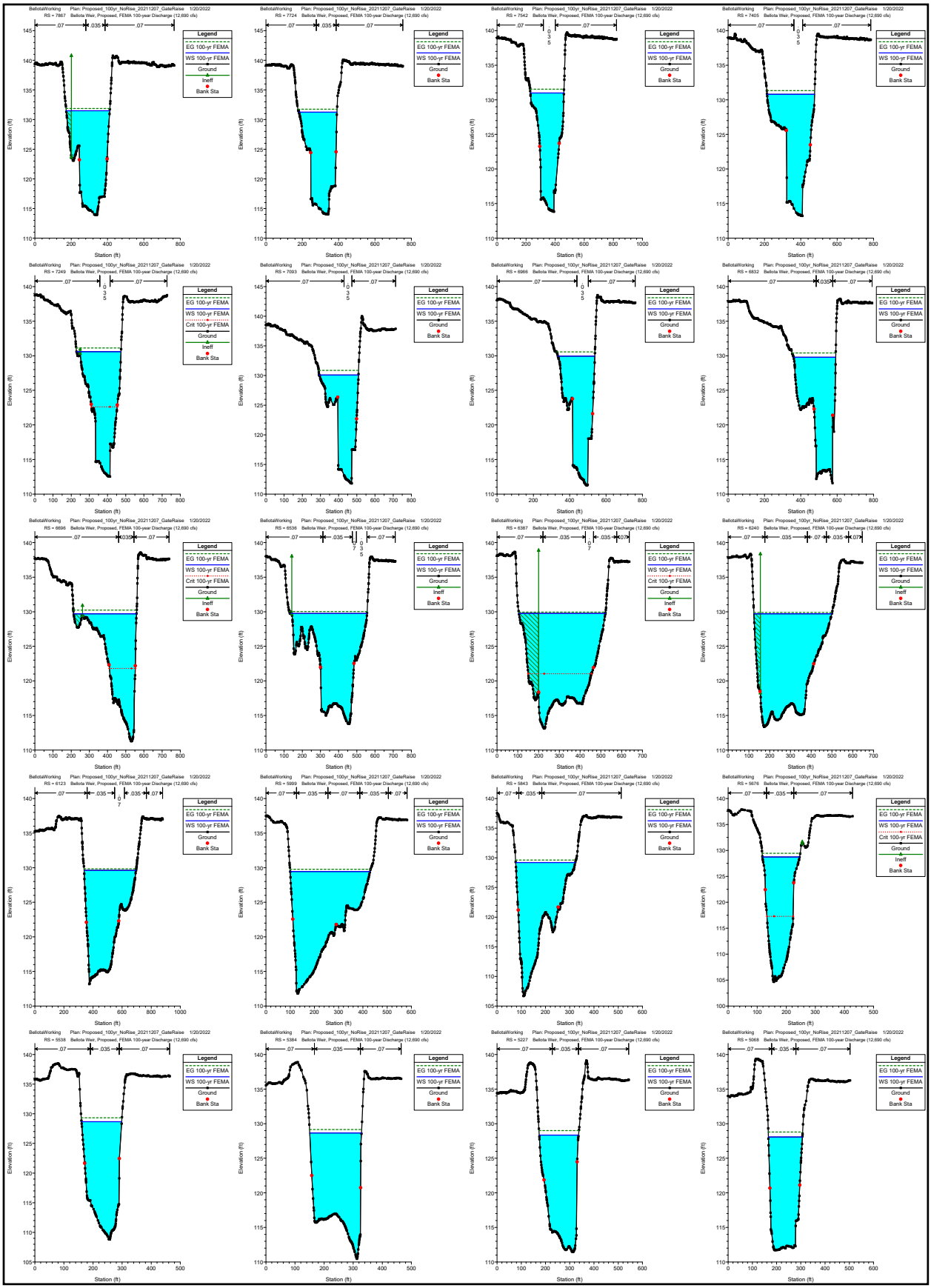
Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
Calaveras River	10829	100-yr FEMA	12690.00	114.07	133.82	125.09	134.44	0.001319	6.35	2121.69	165.28	0.28
Calaveras River	10701	100-yr FEMA	12690.00	114.53	133.65	124.72	134.27	0.001240	6.36	2071.63	156.08	0.28
Calaveras River	10541	100-yr FEMA	12690.00	114.91	133.50	124.24	134.04	0.001310	6.00	2235.67	179.12	0.27
Calaveras River	10393	100-yr FEMA	12690.00	114.59	133.24	124.30	133.90	0.000646	6.58	2006.54	177.27	0.29
Calaveras River	10239	100-yr FEMA	12690.00	114.07	133.13	124.12	133.80	0.000648	6.66	2010.00	171.21	0.29
Calaveras River	10083	100-yr FEMA	12690.00	113.54	132.72	124.84	133.65	0.000945	8.07	1907.43	190.83	0.35
Calaveras River	9946	100-yr FEMA	12690.00	113.42	132.78		133.47	0.000736	6.93	2268.67	203.55	0.30
Calaveras River	9803	100-yr FEMA	12690.00	113.16	132.81		133.32	0.000573	5.93	2476.07	207.88	0.26
Calaveras River	9638	100-yr FEMA	12690.00	112.57	132.78		133.22	0.000450	5.32	2544.22	208.26	0.23
Calaveras River	9513	100-yr FEMA	12690.00	112.13	132.66		133.12	0.001230	5.56	2441.96	199.84	0.24
Calaveras River	9375	100-yr FEMA	12690.00	111.66	132.38		133.00	0.000585	6.42	2186.75	181.20	0.28
Calaveras River	9243	100-yr FEMA	12690.00	111.81	132.32	122.64	132.91	0.000578	6.25	2215.07	165.15	0.27
Calaveras River	9066	100-yr FEMA	12690.00	112.19	132.17	122.96	132.79	0.000692	6.59	2270.92	186.61	0.28
Calaveras River	8893	100-yr FEMA	12690.00	112.59	132.26	122.39	132.63	0.000383	4.98	2793.31	269.46	0.22
Calaveras River	8759	100-yr FEMA	12690.00	112.88	132.14	122.23	132.55	0.000779	5.28	2704.49	239.82	0.23
Calaveras River	8603	100-yr FEMA	12690.00	113.22	131.71	122.87	132.40	0.000788	6.88	2139.04	183.47	0.30
Calaveras River	8463	100-yr FEMA	12690.00	113.50	131.63	122.83	132.29	0.000707	6.68	2164.04	173.11	0.29
Calaveras River	8349	100-yr FEMA	12690.00	113.69	131.59	122.77	132.19	0.000625	6.37	2260.35	191.07	0.28
Calaveras River	8157	100-yr FEMA	12690.00	113.75	131.55		132.05	0.000540	5.90	2541.02	227.23	0.26
Calaveras River	8001	100-yr FEMA	12690.00	113.81	131.57	122.26	131.94	0.000393	5.00	2682.78	256.51	0.23
Calaveras River	7867	100-yr FEMA	12690.00	113.91	131.50		131.87	0.000724	5.06	2737.29	239.46	0.23
Calaveras River	7724	100-yr FEMA	12690.00	114.00	131.25		131.75	0.000906	5.77	2410.77	208.69	0.26
Calaveras River	7542	100-yr FEMA	12690.00	113.84	130.99		131.54	0.001302	6.13	2320.99	230.23	0.28
Calaveras River	7405	100-yr FEMA	12690.00	113.22	130.81		131.32	0.001801	6.02	2431.10	261.40	0.28
Calaveras River	7249	100-yr FEMA	12690.00	112.53	130.60	122.62	131.12	0.000999	5.91	2402.02	245.43	0.28
Calaveras River	7093	100-yr FEMA	12690.00	111.81	130.09		130.86	0.002473	7.30	2004.46	216.86	0.33
Calaveras River	6966	100-yr FEMA	12690.00	111.25	129.95		130.57	0.001586	6.58	2225.80	203.56	0.29
Calaveras River	6832	100-yr FEMA	12690.00	111.59	129.79		130.40	0.000941	6.69	2448.31	231.80	0.30
Calaveras River	6696	100-yr FEMA	12690.00	111.27	129.70	121.81	130.23	0.001347	5.97	2424.08	345.89	0.28
Calaveras River	6536	100-yr FEMA	12690.00	113.78	129.75		130.03	0.000475	4.45	3481.61	416.63	0.21
Calaveras River	6387	100-yr FEMA	12690.00	113.12	129.75	121.03	129.94	0.000387	3.59	3632.99	416.25	0.18
Calaveras River	6240	100-yr FEMA	12690.00	113.37	129.70		129.89	0.000326	3.51	3846.03	375.13	0.17
Calaveras River	6123	100-yr FEMA	12690.00	113.18	129.62		129.84	0.000379	3.93	3570.16	356.94	0.19
Calaveras River	5999	100-yr FEMA	12690.00	111.82	129.45		129.77	0.000765	4.79	3097.60	329.08	0.23
Calaveras River	5843	100-yr FEMA	12690.00	106.70	129.21		129.62	0.001004	5.24	2639.76	245.91	0.25
Calaveras River	5676	100-yr FEMA	12690.00	104.66	128.75	117.29	129.46	0.000673	6.76	1932.25	127.79	0.27
Calaveras River	5538	100-yr FEMA	12690.00	108.81	128.67		129.33	0.000953	6.53	1994.20	138.22	0.29
Calaveras River	5384	100-yr FEMA	12690.00	110.47	128.66		129.15	0.000751	5.67	2262.62	177.77	0.27
Calaveras River	5227	100-yr FEMA	12690.00	111.46	128.36		128.99	0.001184	6.44	2036.24	162.06	0.30
Calaveras River	5068	100-yr FEMA	12690.00	111.71	128.08		128.80	0.001173	6.82	1909.08	140.44	0.31
Calaveras River	4942	100-yr FEMA	12690.00	110.00	127.89		128.64	0.001229	6.98	1869.90	135.96	0.31
Calaveras River	4791	100-yr FEMA	12690.00	110.24	127.69		128.47	0.001079	7.11	1867.96	143.53	0.32
Calaveras River	4621	100-yr FEMA	12690.00	107.88	127.53		128.27	0.001068	6.94	1879.39	142.54	0.32
Calaveras River	4465	100-yr FEMA	12690.00	109.69	127.39		128.10	0.001050	6.77	1932.69	152.05	0.31
Calaveras River	4342	100-yr FEMA	12690.00	110.19	127.27		127.95	0.001254	6.67	1995.66	165.73	0.31
Calaveras River	4205	100-yr FEMA	12690.00	109.69	127.06		127.77	0.001406	6.84	2003.12	185.79	0.33
Calaveras River	4049	100-yr FEMA	12690.00	110.41	126.58		127.52	0.001536	7.95	1798.77	178.28	0.39
Calaveras River	3919	100-yr FEMA	12690.00	109.84	126.48		127.26	0.001698	7.21	1895.71	169.31	0.34
Calaveras River	3713	100-yr FEMA	12690.00	110.28	126.16	119.18	126.90	0.001673	6.95	1905.64	168.14	0.33
Calaveras River	3592	100-yr FEMA	12690.00	110.58	125.92	119.19	126.70	0.001572	7.15	1861.85	163.47	0.34
Calaveras River	3452	100-yr FEMA	12690.00	110.16	125.62	118.84	126.49	0.001366	7.56	1803.14	155.78	0.36
Calaveras River	3291	100-yr FEMA	12690.00	109.53	125.46	118.47	126.23	0.001454	7.09	1845.63	160.48	0.34
Calaveras River	3130	100-yr FEMA	12690.00	109.72	125.19	118.26	126.00	0.001329	7.25	1787.03	151.69	0.35
Calaveras River	2976	100-yr FEMA	12690.00	110.75	125.24		125.75	0.000840	5.75	2236.06	217.09	0.29
Calaveras River	2832	100-yr FEMA	12690.00	112.73	125.16		125.63	0.000732	5.47	2333.05	265.01	0.29
Calaveras River	2711	100-yr FEMA	12690.00	110.29	125.11		125.53	0.000607	5.21	2511.45	403.19	0.27
Calaveras River	2550	100-yr FEMA	12690.00	111.60	124.92		125.42	0.000742	5.73	2365.86	343.70	0.30
Calaveras River	2452	100-yr FEMA	12690.00	107.86	124.92		125.32	0.000546	5.07	2542.14	348.97	0.26
Calaveras River	2375	100-yr FEMA	12690.00	107.12	124.58	117.92	125.24	0.001034	6.55	2040.50	276.99	0.33
Calaveras River	2299	100-yr FEMA	12690.00	108.50	124.52		125.17	0.000807	6.48	1961.68	203.48	0.32
Calaveras River	2217	100-yr FEMA	12690.00	109.00	124.54	116.76	125.08	0.000619	5.90	2168.49	212.38	0.28
Calaveras River	2196	100-yr FEMA	12690.00	109.00	124.52		125.07	0.000615	5.92	2143.27	233.88	0.28
Calaveras River	2180	100-yr FEMA	12690.00	113.00	124.06	119.25	125.01	0.001440	7.83	1621.32	204.64	0.42
Calaveras River	2176	100-yr FEMA	12690.00	114.50	123.51	120.68	124.96	0.000558	9.66	1313.77	165.97	0.57
Calaveras River	2172	100-yr FEMA	12690.00	115.44	122.52	121.61	124.86	0.001192	12.29	1032.88	179.00	0.81
Calaveras River	2171	100-yr FEMA	12690.00	115.44	122.51	121.61	124.86	0.001196	12.30	1031.91	178.23	0.82
Calaveras River	2170.6	100-yr FEMA	12690.00	115.44	122.51	121.62	124.86	0.001194	12.29	1032.82	177.96	0.81
Calaveras River	2170	100-yr FEMA	12690.00	115.44	122.51	121.61	124.86	0.001196	12.29	1032.21	178.12	0.82
Calaveras River	2167	100-yr FEMA	12690.00	115.44	122.50	121.61	124.86	0.001200	12.31	1031.01	178.63	0.82
Calaveras River	2162	100-yr FEMA	12690.00	112.50	122.63	121.54	124.80	0.000845	11.82	1074.00	184.53	0.78
Calaveras River	2154	100-yr FEMA	12690.00	112.50	122.69		124.76	0.000799	11.57	1097.03	177.71	0.76
Calaveras River	2148	100-yr FEMA	12690.00	112.50	122.78		124.67	0.000712	11.03	1150.92	184.54	0.72
Calaveras River	2143	100-yr FEMA	12690.00	114.31	122.91	121.06	124.51	0.007357	10.13	1251.75	200.18	0.65
Calaveras River	2127	100-yr FEMA	12690.00	114.31	122.89	120.91	124.30	0.007138	9.53	1330.91	206.43	0.62
Calaveras River	2104	100-yr FEMA	12690.00	113.66	122.79	120.66	124.11	0.006773	9.22	1377.09	210.14	0.61
Calaveras River	2096	100-yr FEMA	12690.00	113.47	122.76		124.05	0.006625	9.12	1391.06	210.33	0.60
Calaveras River	2091	100-yr FEMA	12690.00	113.31	122.76		124.00	0.006275	8.93	1420.88	212.13	0.58
Calaveras River	2059	100-yr FEMA	12690.00	112.36	122.75		123.75	0.004985	8.03	1579.41	218.08	0.53
Calaveras River	1996	100-yr FEMA	12690.00	110.50	122.75		123.43	0.002764	6.61	1919.56	226.35	0.40
Calaveras River	1908	100-yr FEMA	12690.00	107.88	122.80		123.19	0.001189	5.02	2527.33	239.94	0.27

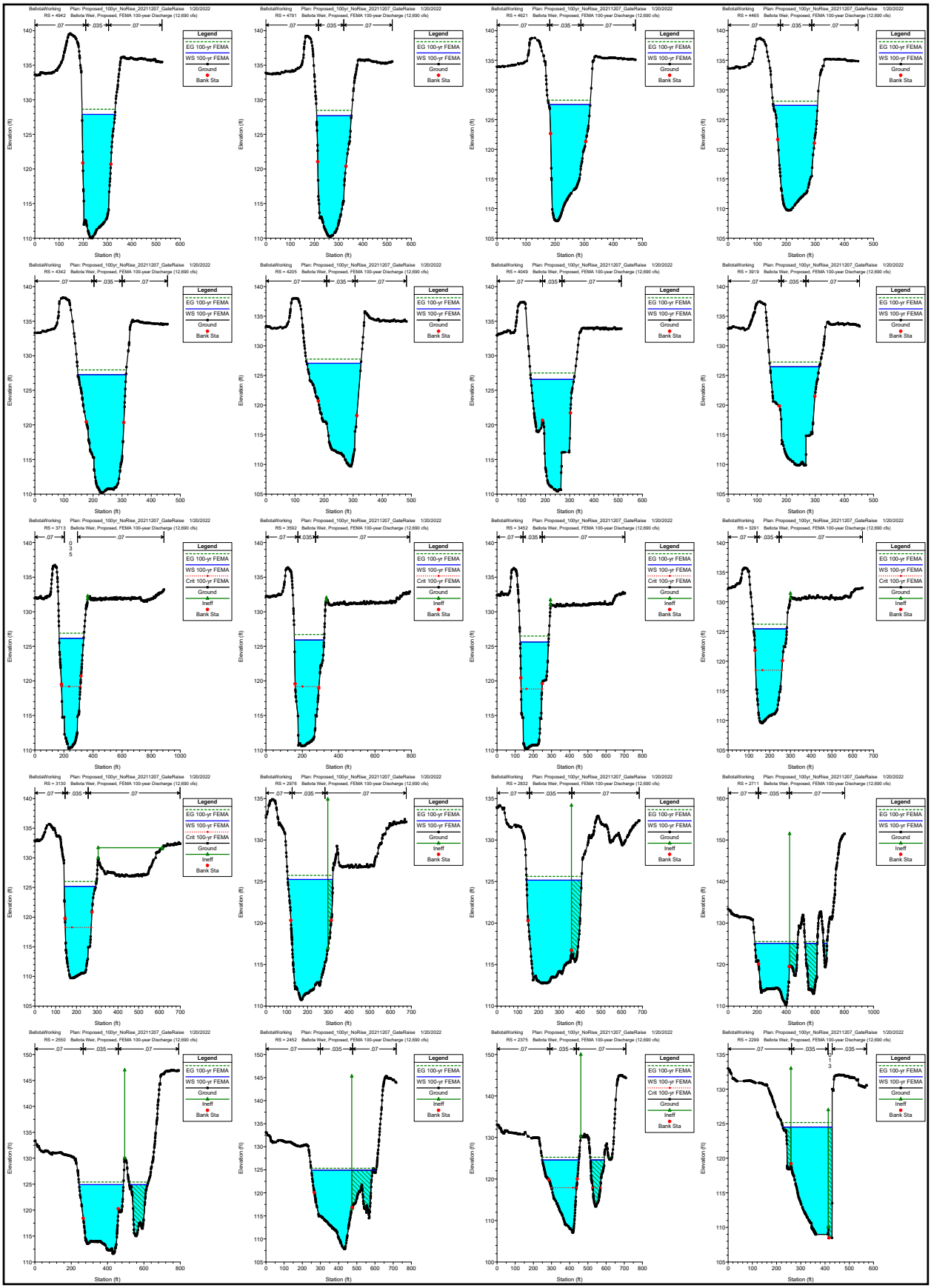
HEC-RAS Plan: Proposed_100yr_NoRise_20211207_GateRaise River: Calaveras River Reach: Calaveras River Profile: 100-yr FEMA (Continued)

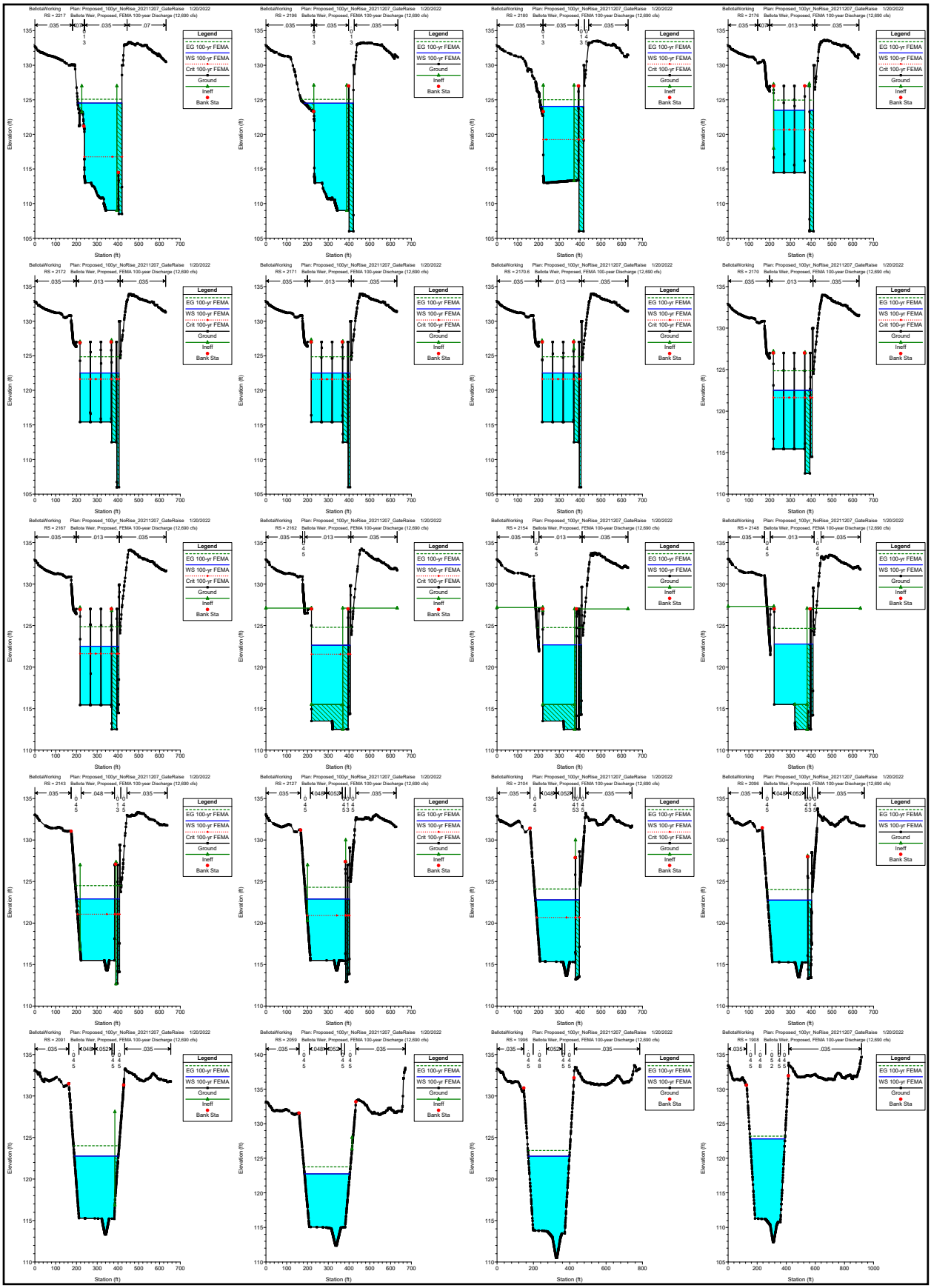
Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
Calaveras River	1795	100-yr FEMA	12690.00	104.59	122.83		123.05	0.000506	3.78	3361.16	258.94	0.18
Calaveras River	1762	100-yr FEMA	12690.00	103.62	122.84		123.03	0.000408	3.51	3615.26	264.64	0.17
Calaveras River	1646	100-yr FEMA	12690.00	103.59	122.78	111.84	122.98	0.000458	3.62	3506.36	269.65	0.18
Calaveras River	1582	100-yr FEMA	12690.00	103.61	122.71		122.95	0.000464	3.89	3263.03	255.27	0.19
Calaveras River	1474	100-yr FEMA	12690.00	103.63	122.66		122.90	0.000429	4.00	3315.63	261.05	0.18
Calaveras River	1343	100-yr FEMA	12690.00	103.34	122.54		122.82	0.000809	4.33	3119.19	253.24	0.19
Calaveras River	1219	100-yr FEMA	12690.00	102.41	122.29	111.79	122.69	0.001051	5.13	2570.77	196.98	0.22
Calaveras River	1056	100-yr FEMA	12690.00	102.58	121.18		122.36	0.002617	8.89	1564.85	126.21	0.39
Calaveras River	938	100-yr FEMA	12690.00	102.35	120.91		122.02	0.002795	8.59	1576.70	122.12	0.38
Calaveras River	859	100-yr FEMA	12690.00	102.42	121.01	111.87	121.75	0.001527	6.94	1926.24	141.77	0.30
Calaveras River	850		Bridge									
Calaveras River	796	100-yr FEMA	12690.00	102.40	120.54		121.37	0.001294	7.42	1811.56	145.90	0.33
Calaveras River	734	100-yr FEMA	12690.00	101.87	120.50	111.99	121.24	0.001360	6.97	1923.75	145.94	0.31
Calaveras River	608	100-yr FEMA	12690.00	101.71	120.11	111.79	121.05	0.001455	7.78	1671.00	111.49	0.34
Calaveras River	436	100-yr FEMA	12690.00	101.69	119.75	111.84	120.73	0.002317	7.98	1643.66	114.28	0.35
Calaveras River	290	100-yr FEMA	12690.00	102.16	118.59	112.86	120.25	0.003541	10.44	1283.00	100.13	0.47
Calaveras River	141	100-yr FEMA	12690.00	101.57	118.49	111.37	119.73	0.002001	9.00	1457.33	103.29	0.40

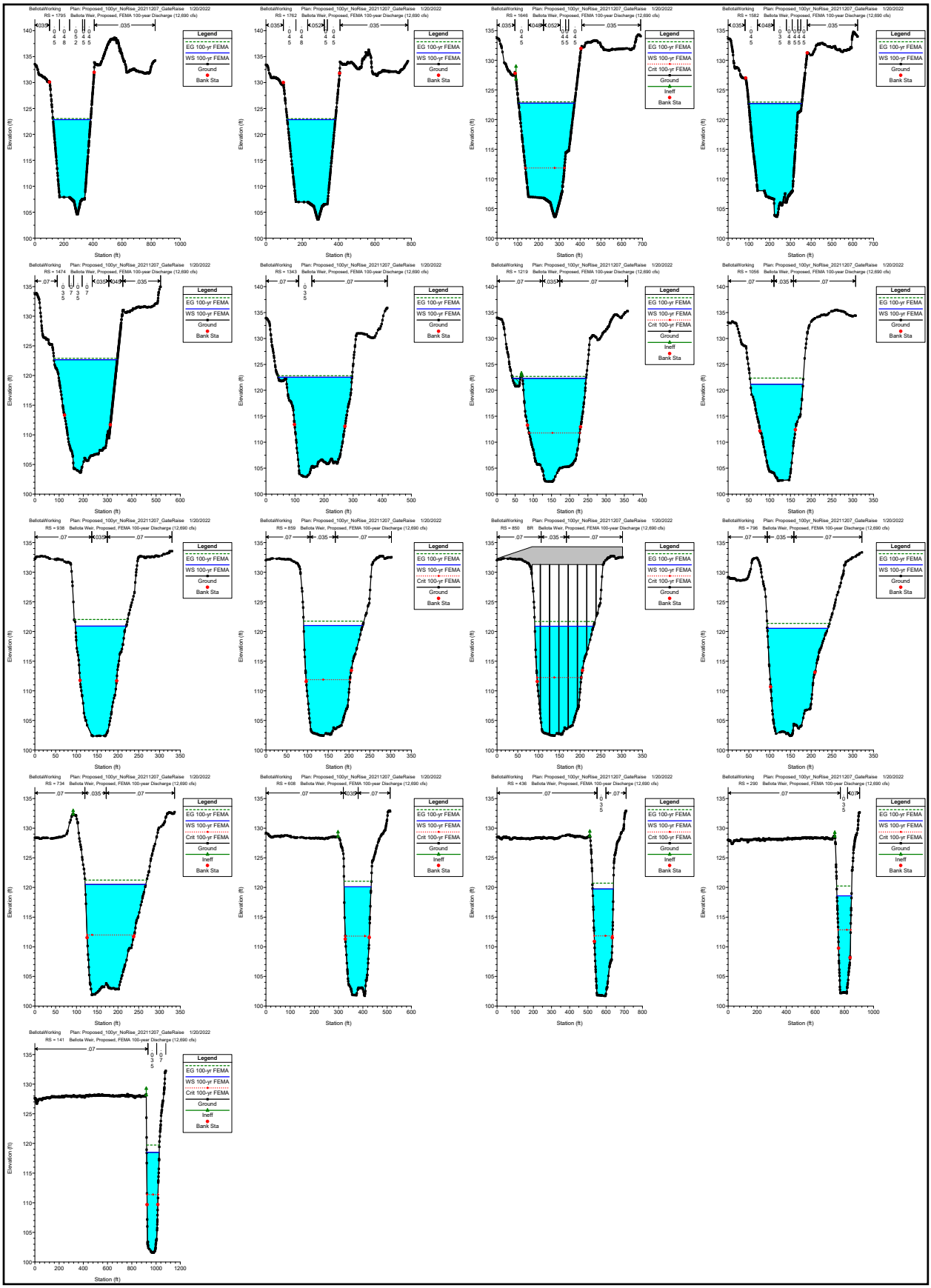

BellotaWorking Plan: Proposed_100yr_NoRise_20211207_GateRaise 1/20/2022
Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
Calaveras River Calaveras River
Legend
EG 100-yr FEMA
WS 100-yr FEMA
Crit 100-yr FEMA
Ground
Elevation (ft)
135
130
125
120
115
110
105
100
0
2000
4000
6000
8000
10000
12000
Main Channel Distance (ft)


BellotaWorking Plan: Proposed_100yr_NoRise_20211207_GateRaise 1/20/2022
RS = 10829 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 10701 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 10541 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 10393 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 10239 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 10083 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 9946 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 9803 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 9638 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 9513 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 9375 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 9243 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 9066 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 8863 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 8759 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 8603 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 8463 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 8349 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 8157 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 8001 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
.07
.035
Legend
EG 100-yr FEMA
WS 100-yr FEMA
Crit 100-yr FEMA
Ground
Ineff
Bank Sta
Elevation (ft)
Station (ft)


BellotaWorking Plan: Proposed_100yr_NoRise_20211207_GateRaise 1/20/2022
RS = 7867 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 7724 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 7542 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 7405 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 7249 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 7093 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 6966 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 6832 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 6696 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 6536 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 6387 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 6240 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 6123 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 5999 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 5843 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 5676 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 5538 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 5384 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 5227 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 5068 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
Legend
EG 100-yr FEMA
WS 100-yr FEMA
Crit 100-yr FEMA
Ground
Ineff
Bank Sta
Elevation (ft)
Station (ft)


BellotaWorking Plan: Proposed_100yr_NoRise_20211207_GateRaise 1/20/2022
RS = 4942 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 4791 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 4621 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 4465 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 4342 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 4205 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 4049 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 3919 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 3713 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 3592 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 3452 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 3291 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 3130 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 2976 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 2852 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 2711 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 2550 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 2452 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 2375 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 2299 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
.07
.035
.05
Legend
EG 100-yr FEMA
WS 100-yr FEMA
Crit 100-yr FEMA
Ground
Ineff
Bank Sta
Elevation (ft)
Station (ft)

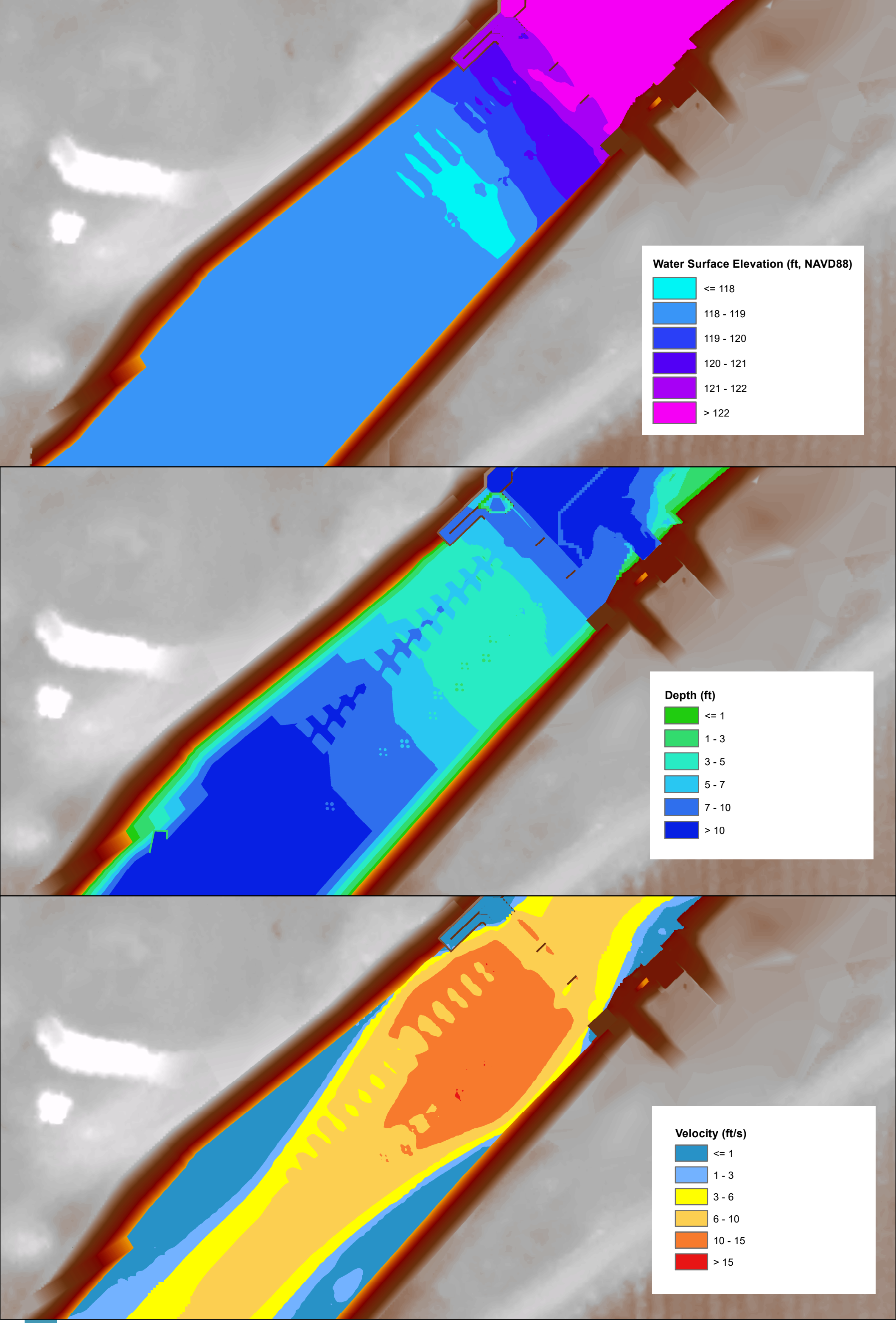

BellotaWorking Plan: Proposed_100yr_NoRise_20211207_GateRaise 1/20/2022
RS = 2217 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 2196 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 2180 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 2176 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 2172 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 2171 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 2170.6 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 2170 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 2167 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 2162 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 2154 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 2148 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 2143 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 2127 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 2104 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 2066 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 2091 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 2059 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 1996 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 1908 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
Legend
EG 100-yr FEMA
WS 100-yr FEMA
Crit 100-yr FEMA
Ground
Ineff
Bank Sta
Elevation (ft)
Station (ft)


BellotaWorking Plan: Proposed_100yr_NoRise_20211207_GateRaise 1/20/2022
RS = 1795 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 1762 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 1646 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 1582 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 1474 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 1343 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 1219 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 1056 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 938 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 859 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 850 BR Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 796 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 734 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 608 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 436 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 290 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
RS = 141 Bellota Weir, Proposed, FEMA 100-year Discharge (12,690 cfs)
Legend
EG 100-yr FEMA
WS 100-yr FEMA
Crit 100-yr FEMA
Ground
Ineff
Bank Sta
Elevation (ft)
Station (ft)

Appendix D. 2D HEC-RAS Results

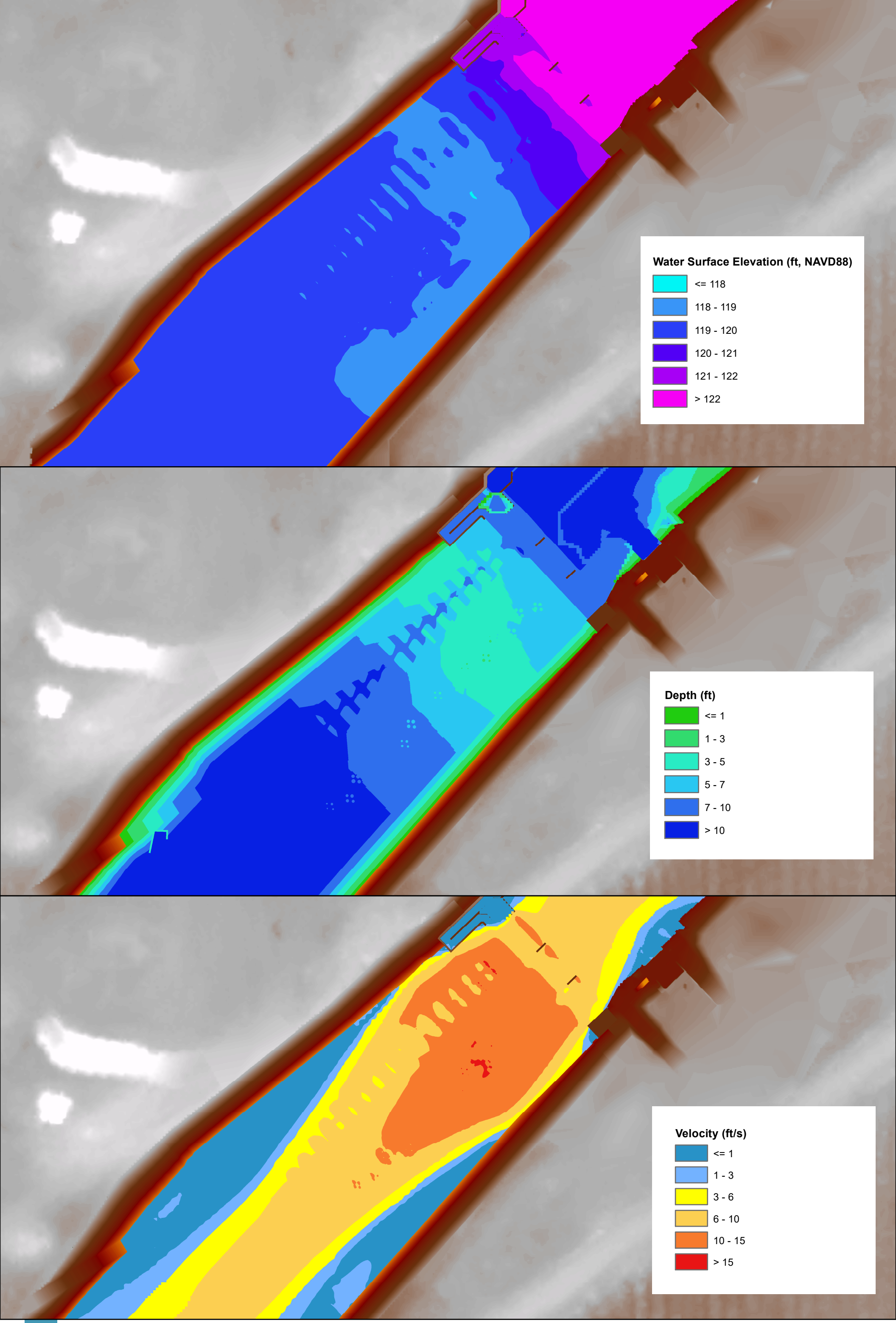
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Water Surface Elevations, Depths, and Velocities for the 10-, 25-, 50-, and 100-year Discharge Events



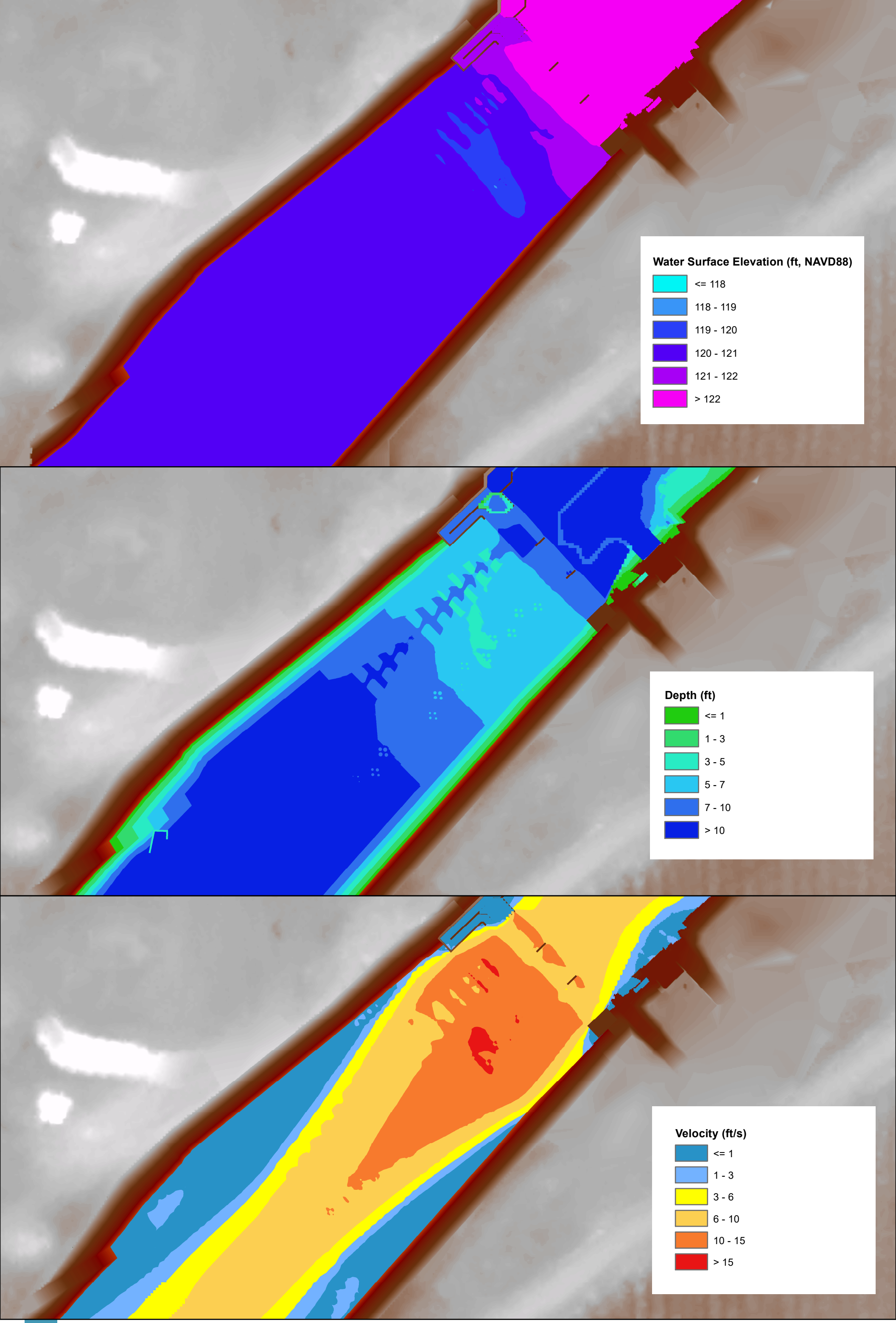
0 Feet 100

EXHIBIT D1
HEC-RAS 2D MODEL RESULTS
USACE 10-YEAR DISCHARGE (9,530 CFS)



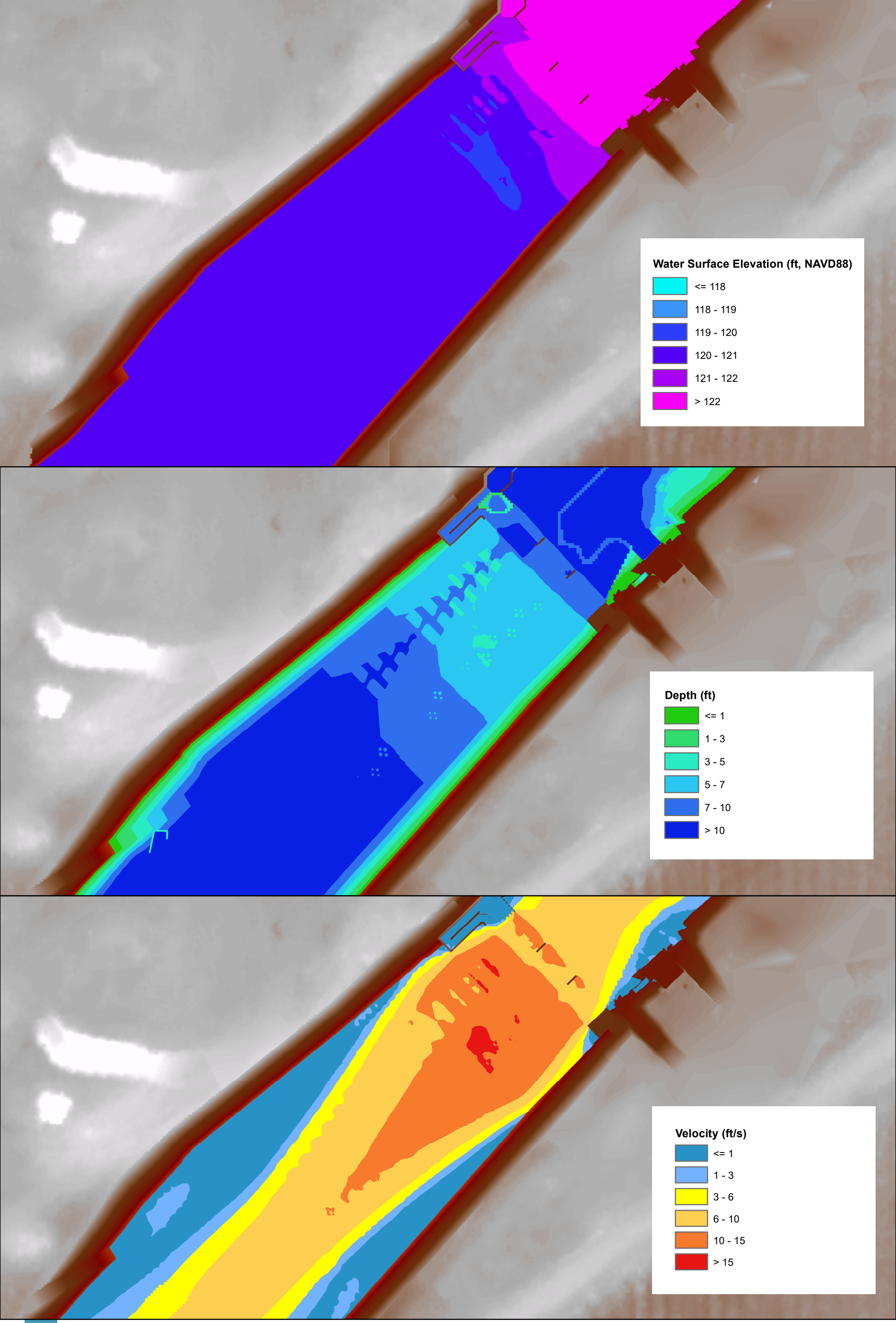
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EXHIBIT D2
HEC-RAS 2D MODEL RESULTS
USACE 25-YEAR DISCHARGE (10,640 CFS)



0 Feet 100

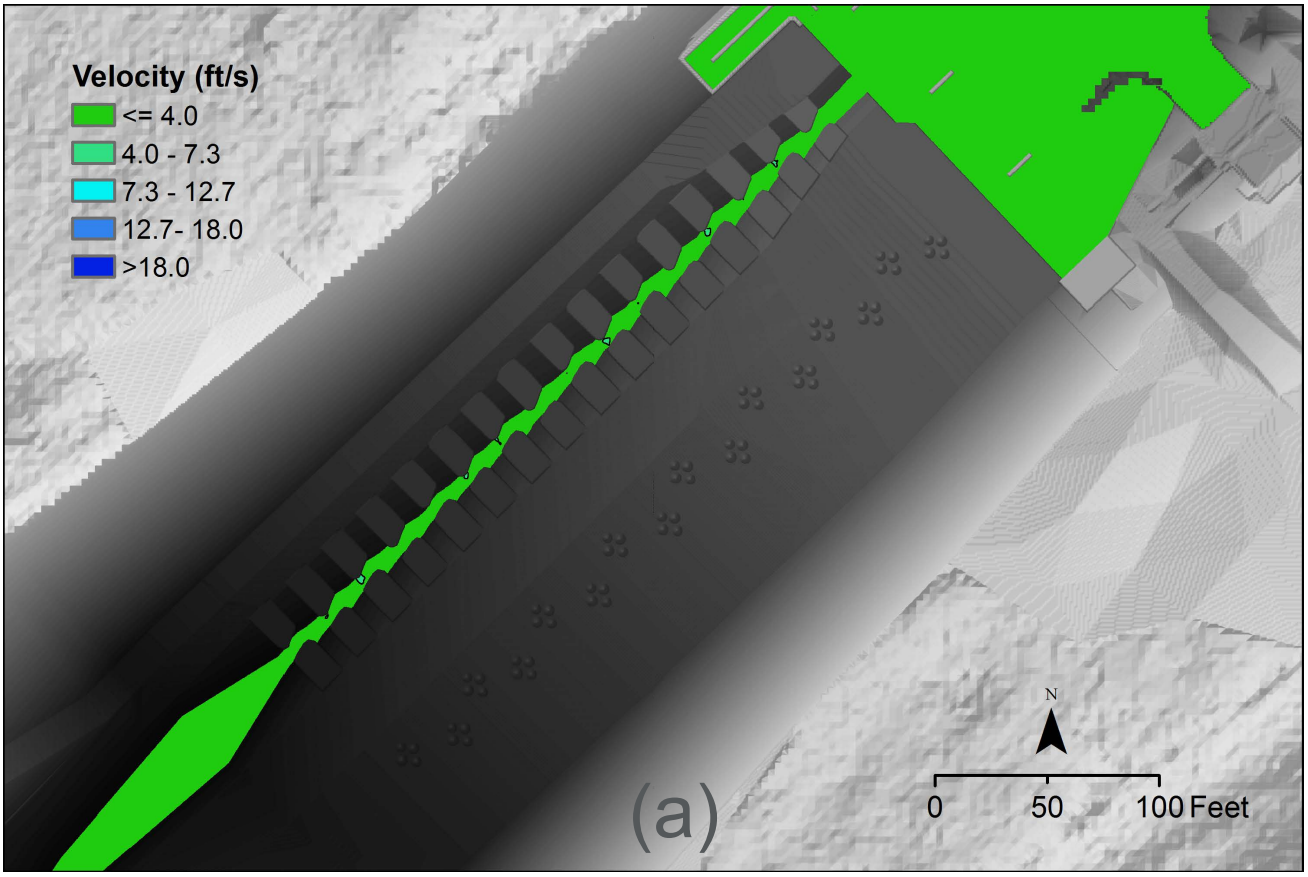
EXHIBIT D3
HEC-RAS 2D MODEL RESULTS
FEMA 50-YEAR DISCHARGE (12,500 CFS)

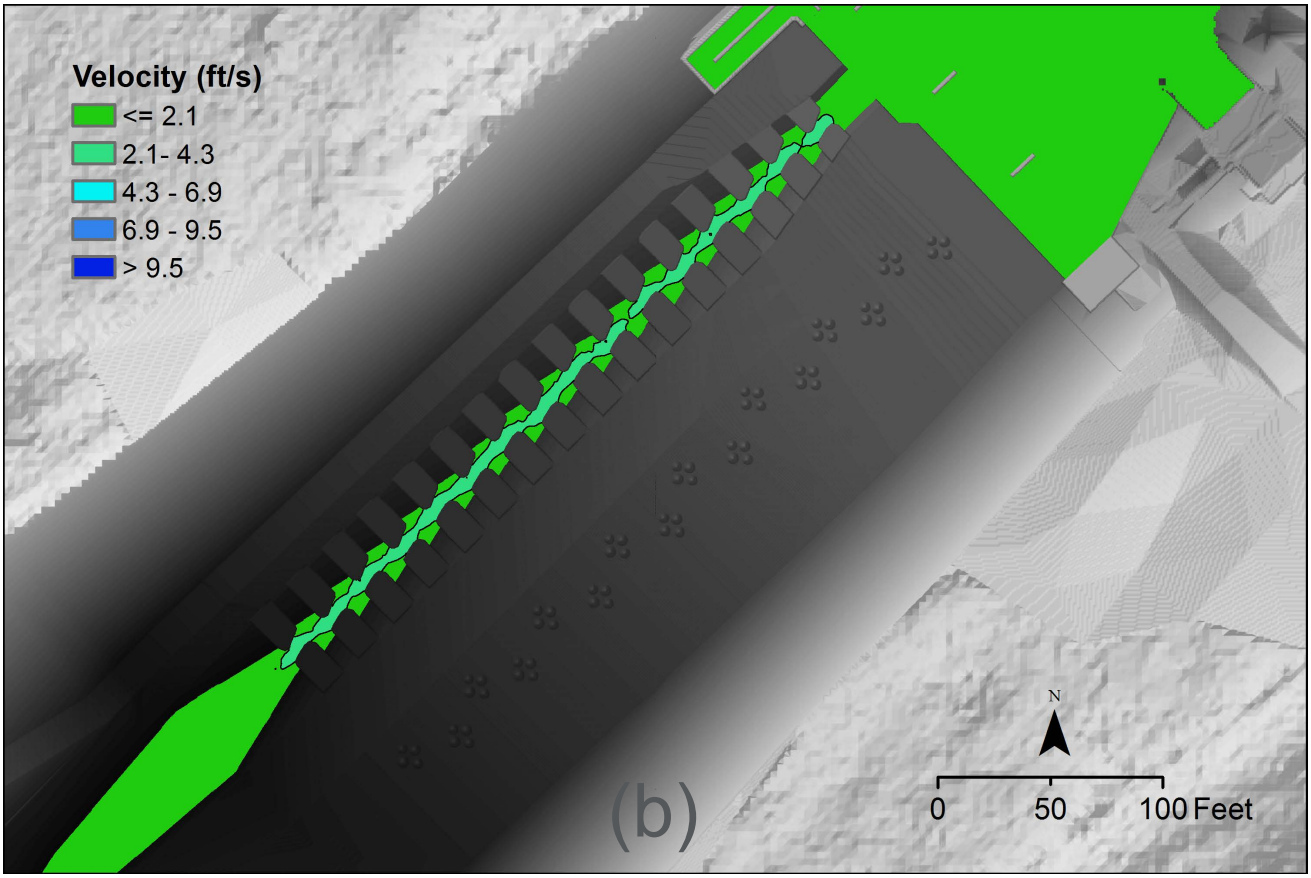


0 Feet 100

EXHIBIT D4
HEC-RAS 2D MODEL RESULTS
FEMA 100-YEAR DISCHARGE (12,690 CFS)

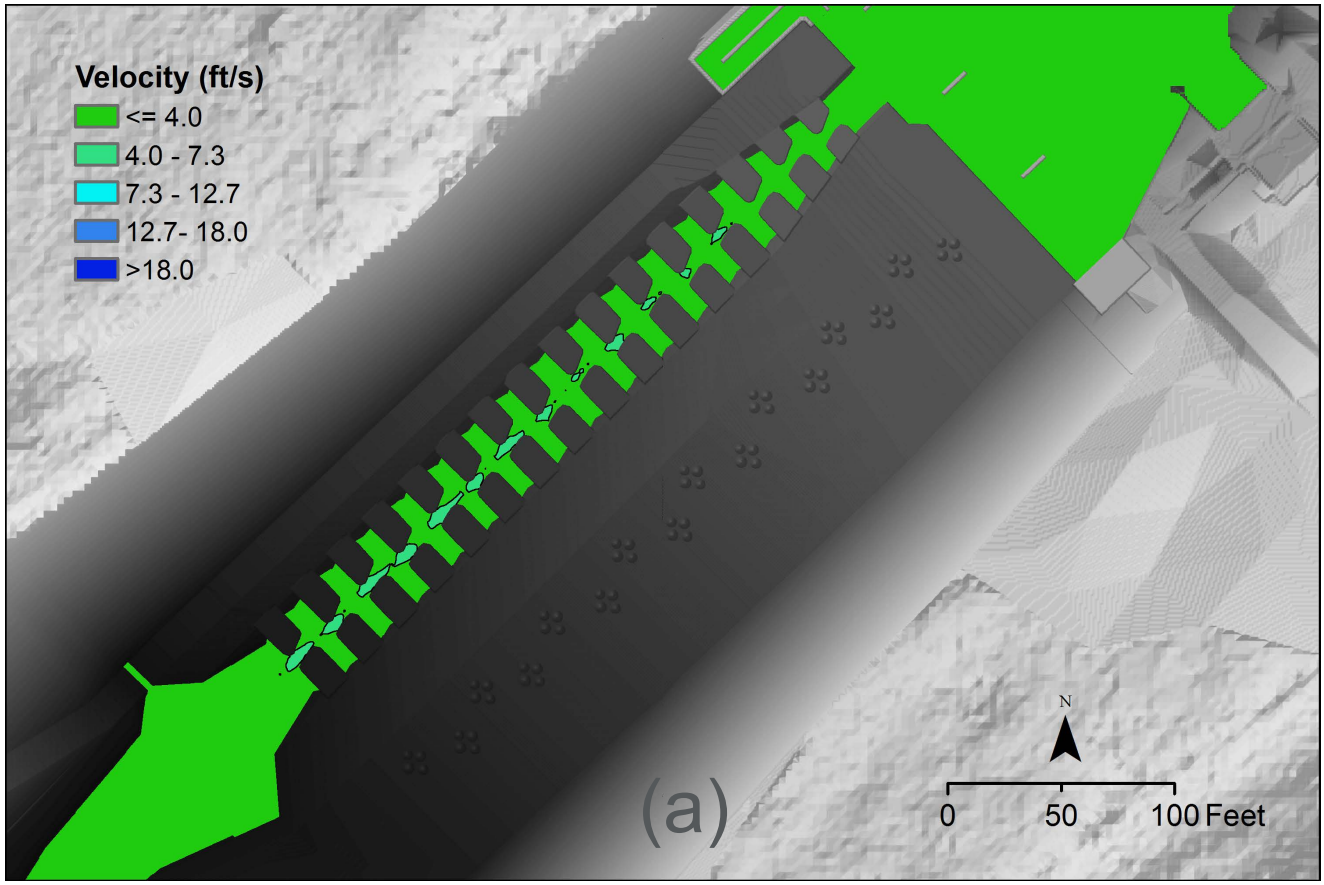
Velocities where Flow Meets Depth Criteria for Fish Passage for Fish Passage Design Flows

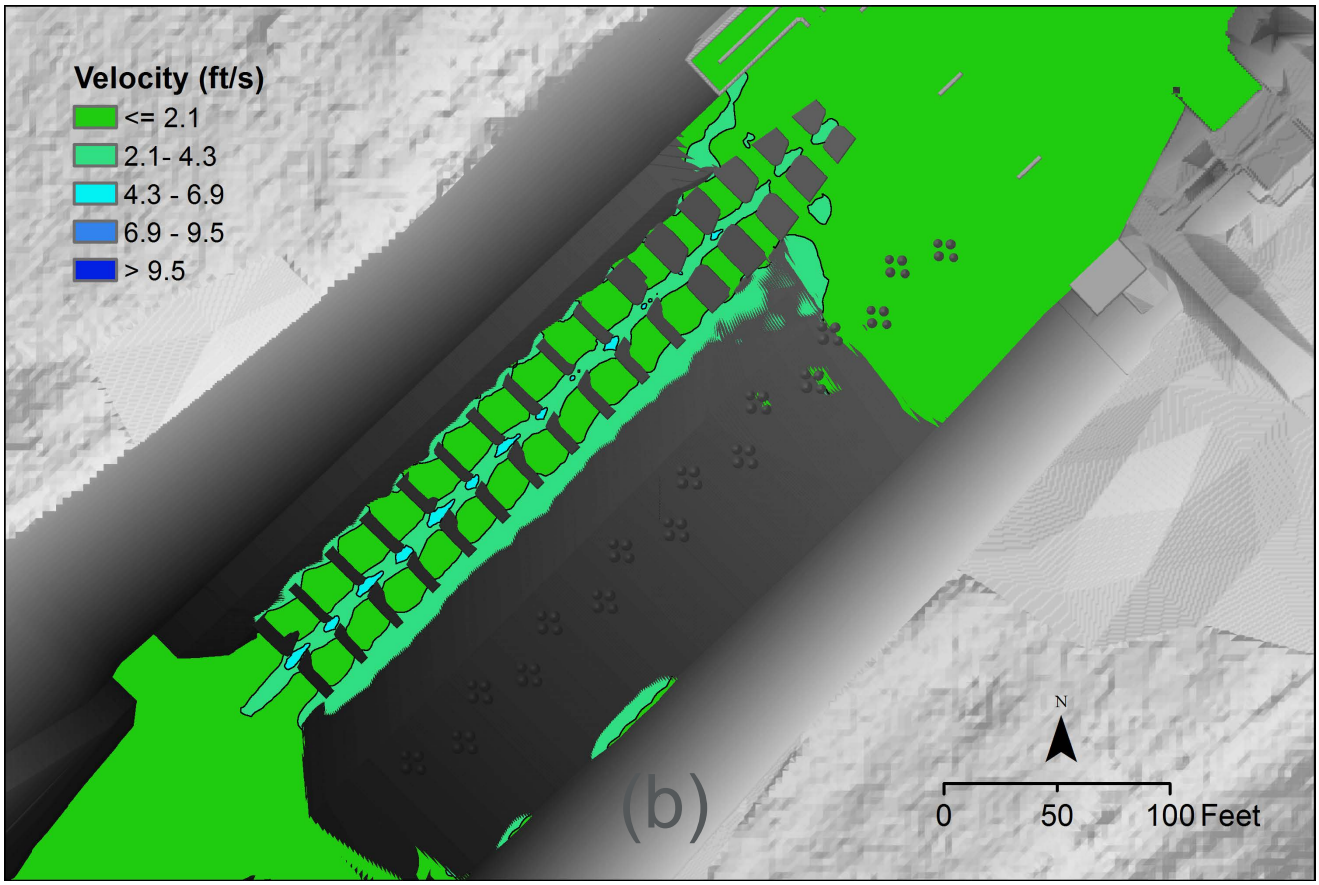




Panel (a) shows Katopodis and Gervais (2016) adult salmonid flow velocity categories where depth meets adult salmonid depth criteria (depth ≥ 0.9 ft); Panel (b) shows Katopodis and Gervais (2016) julvenile salmonid flow velocity categoeis where depth meets juvenile salmonid depth critiera (depth ≥ 0.4 ft)

EXHIBIT D5

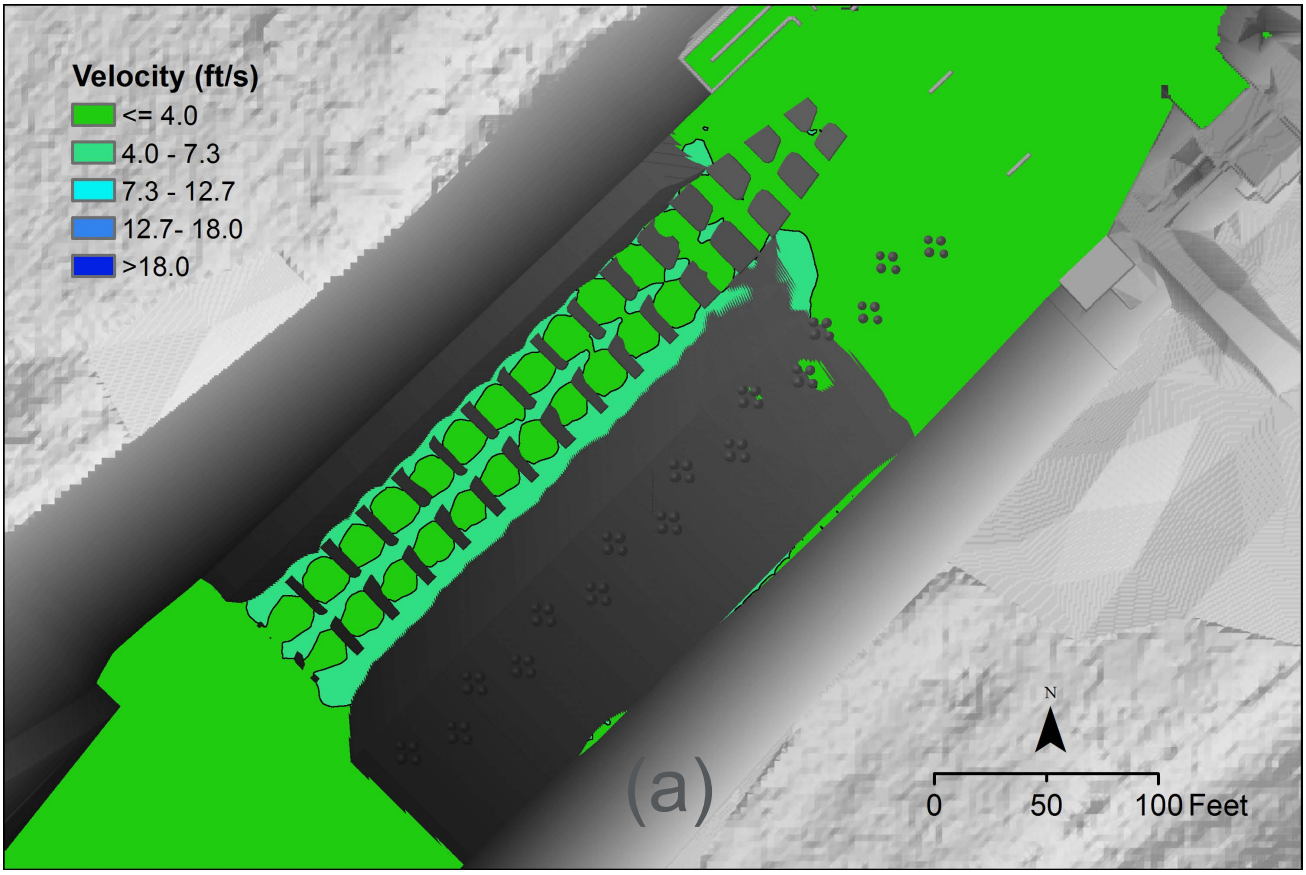


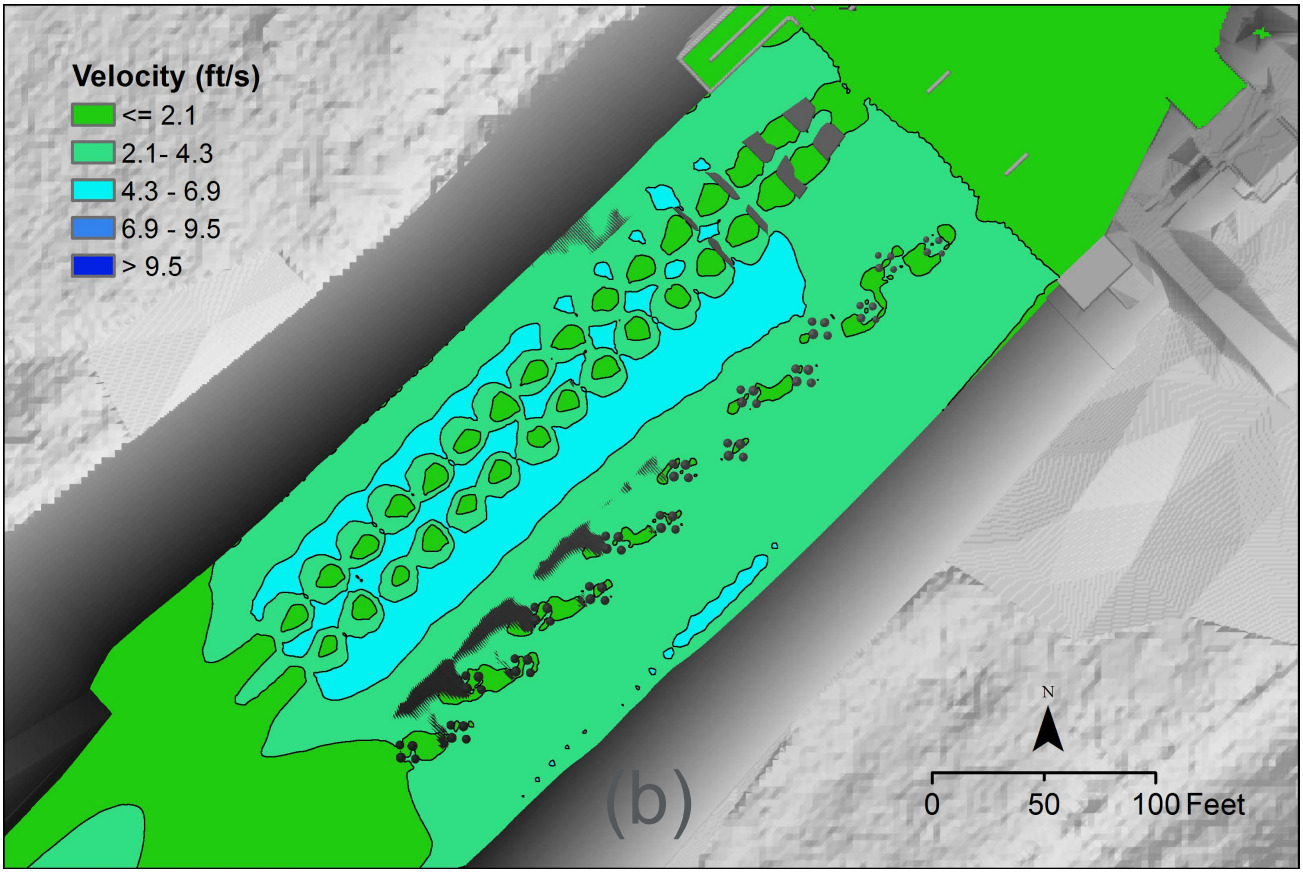


Panel (a) shows Katopodis and Gervais (2016) adult salmonid flow velocity categories where depth meets adult salmonid depth criteria (depth ≥ 0.9 ft); Panel (b) shows Katopodis and Gervais (2016) julvenile salmonid flow velocity categoeis where depth meets juvenile salmonid depth critiera (depth ≥ 0.4 ft)

EXHIBIT D6

HEC-RAS 2D MODEL RESULTS,
FISFISH PASSAGE DESIGN FLOW, HIGH FLOW - IRRIGATION SEASON (150.9 CFS)



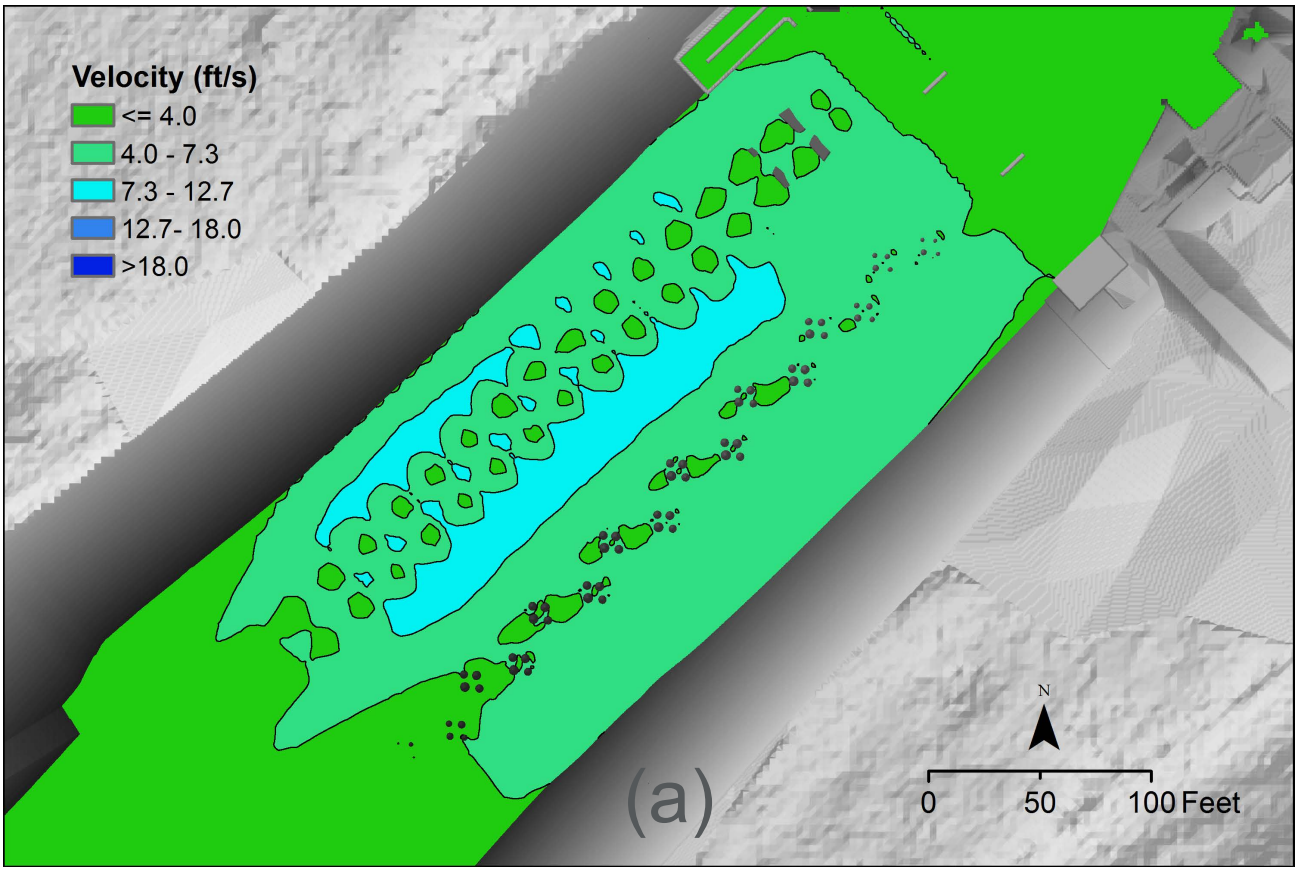


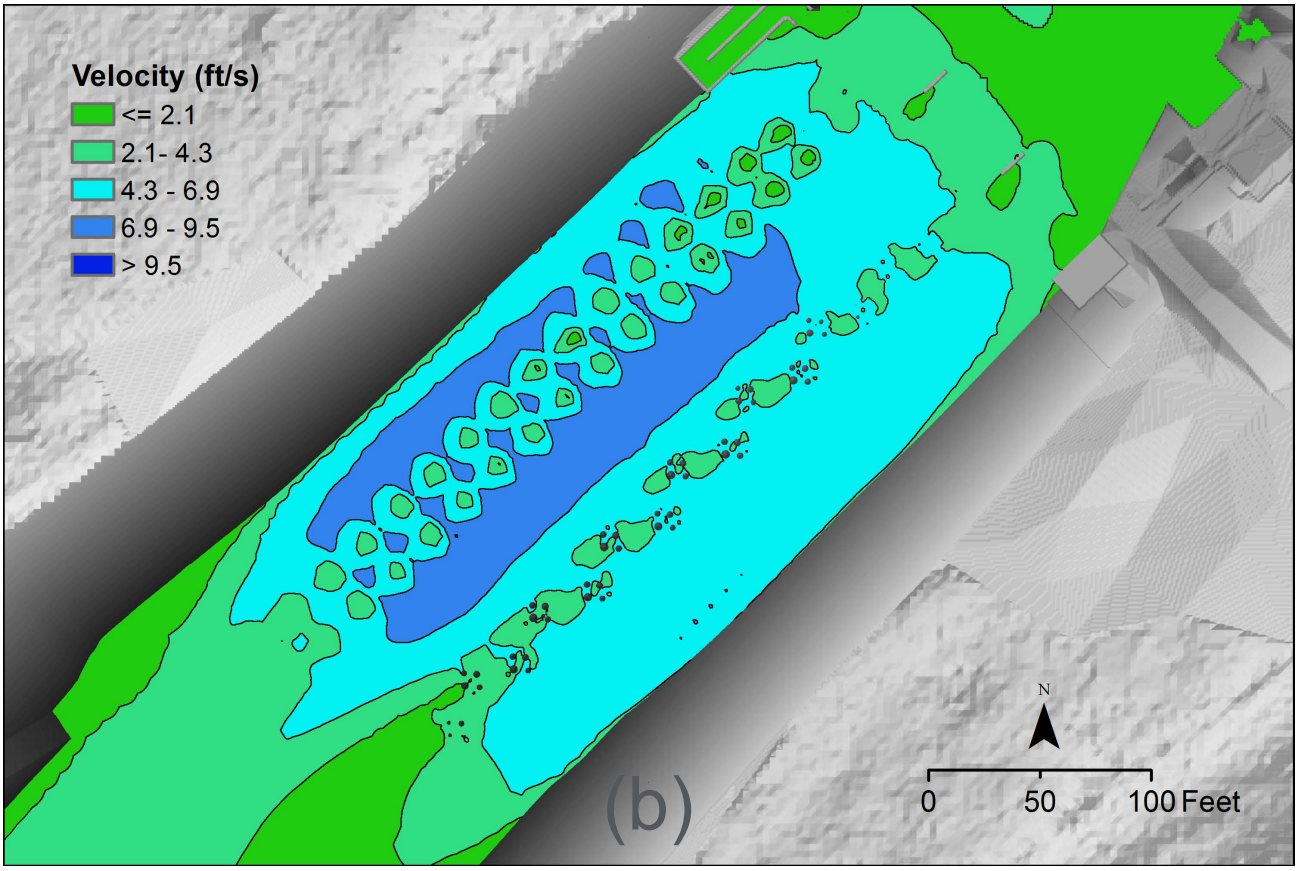
Panel (a) shows Katopodis and Gervais (2016) adult salmonid flow velocity categories where depth meets adult salmonid depth criteria (depth ≥ 0.9 ft); Panel (b) shows Katopodis and Gervais (2016) julvenile salmonid flow velocity categoeis where depth meets juvenile salmonid depth critiera (depth ≥ 0.4 ft)

EXHIBIT D7

HEC-RAS 2D MODEL RESULTS

, FISHFISH PASSAGE DESIGN FLOW, 5% ANNUAL EXCEEDANCE (545.3 CFS)





Panel (a) shows Katopodis and Gervais (2016) adult salmonid flow velocity categories where depth meets adult salmonid depth criteria (depth ≥ 0.9 ft); Panel (b) shows Katopodis and Gervais (2016) julvenile salmonid flow velocity categoeis where depth meets juvenile salmonid depth critiera (depth ≥ 0.4 ft)

EXHIBIT D8

HEC-RAS 2D MODEL RESULTS

, FISH FISH PASSAGE DESIGN FLOW, HIGH FLOW - MIGRATION SEASON (1,735.2 CFS)

Appendix E. Rock and ESM Sizing Calculations

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Roughened Ramp Slope Protection Calculations

EM 1110-2-1601: RIPRAP STONE SIZE CALCULATION

General Notes:

Calculations are based on EM 1110-2-1601 Equation 3-3

Calculations are for the Bellota Roughened Ramp bank riprap (Ramp Zones 3 and 4) design based on FEMA 100yr Discharge

SPREADSHEET LEGEND
HEADING
CONSTANT
USER FILL IN
AUTO-CALCULATES

$$D_{30} = S_f C_s C_V C_T d \left[\left(\frac{\gamma_w}{\gamma_s - \gamma_w} \right)^{1/2} \frac{V}{\sqrt{K_1 g d}} \right]^{2.5} \qquad (3\text{-}3)$$

where

D_{30} = riprap size of which 30 percent is finer by weight, length

S_f = safety factor (see *c* below)

* C_s = stability coefficient for incipient failure, D_{85}/D_{15} = 1.7 to 5.2

= 0.30 for angular rock

* = 0.375 for rounded rock

C_V = vertical velocity distribution coefficient

= 1.0 for straight channels, inside of bends

= 1.283 - 0.2 log (R/W), outside of bends (1 for (R/W) > 26)

= 1.25, downstream of concrete channels

= 1.25, ends of dikes

C_T = thickness coefficient (see d(1) below)

* = 1.0 for thickness = $1D_{100}$(max) or 1.5 D_{50}(max), whichever is greater

* d = local depth of flow, length (same location as *V*)

γ_w = unit weight of water, weight/volume

* V = local depth-averaged velocity, V_{SS} for side slope riprap, length/time

K_1 = side slope correction factor (see *d*(1) below)

g = gravitational constant, length/time^2

Safety Factor, *S_f*

-- Minimum SF is 1.1
-- SF may be increased for imposed impact forces resulting from logs, trees, loose vessels, ice, etc…
-- It is unlikely that an added SF should be used when blanket thickness exceeds 15"
-- SF may be increased to compensate for inaccuracies in velocity, unit weight of rock, and depth
-- SF should be increased where severe freeze-thaw is anticipated

Safety Factor, S_f =	1.25
Reasoning: SF of 1.1 is standard as recommended in EM 1110-2-1601	

Stability Coefficient for Incipient Failure, C_s

D_{85}/D_{15} > 1.7 for FHWA gradation	
Rock Shape: Angular	
Stability Coefficient C_s =	0.300

Vertical Velocity Distribution Coefficient, C_v

Channel Type: Downstream of Concrete Char	
Radius of Bend, Rc =	NA ft
Channel Width, W =	NA ft
Stability Coefficient C_v =	1.25

Blanket Thickness Coefficient, C_T

Thickness Coefficient C_T =	1.00	<-- Plate B-40, 1.00 is the most conservative C_T

Additional Inputs

Gravatational Constant, g =	32.20	ft/s^2
Unit weight of water, γ_w =	62.40	lb/ft^3
Unit weight of stone, γ_s =	165.00	lb/ft^3
Local depth of flow, d =	4.92	ft
Local depth averaged velocity, V =	11.67	ft/s
Side Slope (xH:1V) =	3.00	
Angle of Repose =	40.00	Degrees
Side slope correction factor, K_1 =	0.87	(Calculated by Eq. 3-4 in EM-1601)
Side slope correction factor, K_1 =	0.97	(Recommended for Design, Plate B-39)
Selected side slope correction factor, K_1 =	0.87	

Result

Computed D_{30} =	1.22 ft	
Computed D_{50} =	1.46 ft	(D_{50} = 1.2 D_{30} for FHWA gradation per NCHRP 568)
Computed D_{50} =	17.56 in	
Selected D_{50} =	18 in	(Table 4.1 from HEC-23 or Table 3.6 from NCHRP 568)

Blanket Thickness

1xD_{100}=	36.00	in
1.5xD_{50} =	27.00	in
Calculated Minimum Thicknesss =	36.00	in
Selected Minimum Thickness =	36.00	in

Gradation (Based on FHWA HEC-23 Table 4.1 in Design Guideline 4)

Nominal Riprap Class			D_{15}		D_{50}		D_{85}		D_{100}	
Class	Weight	Size	Min	Max	Min	Max	Min	Max	Max	
V	1/4 ton	18	11.0	15.5	17.0	20.5	23.5	27.5	36.0	(inch)

Roughened Ramp Foundation
Rock Calculations

Project:	**Bellota Weir Modifications**	Prepared:	KMF	Date: 12/22/2021
Subject:	**90% Design**	Checked:	SB	Date: 12/27/2021
Task:	**Fish Ramp Foundation Rock Design**			
Job#:				

Reference:

*Abt, S.R. and Johnson T.L. 1991. Riprap design for overtopping flow.

*U.S. Bureau of Reclamation (USBR), 2007. Rock Ramp Design Guidelines. Technical Service Center, Denver, CO. September.

*National Resource Conservation Service (NRCS). 2007. Stone Sizing Criteria. National Engineering Handbook, Park 654. Technical Supplement 14C.

Method 1: Abt and Johnson (1991)

EQUATION:

$$D_{50} = \phi_e \cdot \phi_c \cdot a \cdot 5.23 \cdot S_0^{0.43} \cdot q_{design}^{0.56}$$ **Equation 4-2**

INPUTS

ϕe =	1.2	coefficient for empirical envelope on the regression relationship
ϕc =	1.85	ϕc = rq 0.56 (USBR 2007, Eqn 4.3)
a =	1	angular material, a = 1; rounded material a = 1.40
r_q =	3	ratio of unit discharge in channel vs uniform depth; Max value = 3
Safety Factor	1.35	recommended 35% increase in design q (USBR 2007, Eqn. 4-1)
S_0	-0.03	channel invert slope

APPLICABILITY/ASSUMPTIONS

Embankment slopes:	1 - 20	%
Stone Sizes:	1 - 6	in
C_u (D60/D10):	1.62 - 2.15	
σ_g (D84/D16)$^{0.5}$:	1.86 - 5.70	
Unit Weight	167	pcf

DESIGN UNIT DISCHARGES, q, cfs/ft (MAX IN SUB-REGIONS)

Revised: 12/27/2021	**90% Proposed Conditions**			
Discharge	**USACE 10yr**	**USACE 25yr**	**FEMA 50yr**	**FEMA 100yr**
Ramp Location	**HEC-RAS 2D**	**HEC-RAS 2D**	**HEC-RAS 2D**	**HEC-RAS 2D**
Overbank	0	0	0	95
Main Channel	0	0	0	77
Main Channel & Overbank	0	0	0	82

DESIGN HYDRAULIC SLOPE (MAX IN CHANNEL)

Revised: 12/27/2021	**90% Proposed Conditions**			
Discharge	**USACE 10yr**	**USACE 25yr**	**FEMA 50yr**	**FEMA 100yr**
Ramp Location	**HEC-RAS 2D**	**HEC-RAS 2D**	**HEC-RAS 2D**	**HEC-RAS 2D**
Overbank	#DIV/0!	#DIV/0!	#DIV/0!	-0.002
Main Channel	#DIV/0!	#DIV/0!	#DIV/0!	-0.002
Main Channel & Overbank	#DIV/0!	#DIV/0!	#DIV/0!	-0.002

Note*: 65% design showed that 100-yr event produced most conservative rock sizing, thus only the 100-yr event was re-evaluated for 90% design

DESIGN ENERGY SLOPE (MAX IN CHANNEL)				
Revised: 12/27/2021	90% Proposed Conditions			
Discharge	USACE 10yr	USACE 25yr	FEMA 50yr	FEMA 100yr
Ramp Location	HEC-RAS 2D	HEC-RAS 2D	HEC-RAS 2D	HEC-RAS 2D
Overbank	#DIV/0!	#DIV/0!	#DIV/0!	-0.009
Main Channel	#DIV/0!	#DIV/0!	#DIV/0!	-0.009
Main Channel & Overbank	#DIV/0!	#DIV/0!	#DIV/0!	-0.009

***Note: Energy slope selected as design slope due to influence of backwater at the downstream end of the ramp; and energy slope produces more conservative rock sizing than hydraulic slope**

DESIGN D50, Equation 4-2 (USBR 2007); BASED ON HYDRAULIC SLOPE											
						See *Exhibit E.X* tab for gradation information					
Revised: 12/27/2021	90% Proposed Conditions				*Max of calculated D50*	*Min D50 based on NCHRP (2006) Table 3.6/3.7*	*Max D50 based on NCHRP (2006) Table 3.6/3.7*	*Max D100 based on NCHRP (2006) Table 3.6/3.7*	*Max of 1.5*D50, D100*	*Riprap Class based on NCHRP (2006) table 3.7*
Discharge	USACE 10yr	USACE 25yr	FEMA 50yr	FEMA 100yr	Calculated D50	MIN D50 (NCHRP 2006)	MAX D50 (NCHRP 2006)	MAX D100 (NCHRP 2006)	THICKNESS (NCHRP 2006)	WEIGHT (NCHRP 2006)
Ramp Location	HEC-RAS 2D	HEC-RAS 2D	HEC-RAS 2D	HEC-RAS 2D						
Overbank	#DIV/0!	#DIV/0!	#DIV/0!	0.95	0.95	0.71	0.88	1.50	1.50	60 lb
Main Channel	#DIV/0!	#DIV/0!	#DIV/0!	0.84	0.84	0.71	0.88	1.50	1.50	60 lb
Main Channel & Overbank	#DIV/0!	#DIV/0!	#DIV/0!	0.87	0.87	0.71	0.88	1.50	1.50	60 lb

DESIGN D50, Equation 4-2 (USBR 2007); BASED ON ENERGY SLOPE										
Revised: 12/27/2021	90% Proposed Conditions									
Discharge	USACE 10yr	USACE 25yr	FEMA 50yr	FEMA 100yr	Calculated D50	MIN D50 (NCHRP 2006)	MAX D50 (NCHRP 2006)	MAX D100 (NCHRP 2006)	THICKNESS (NCHRP 2006)	WEIGHT (NCHRP 2006)
Ramp Location	HEC-RAS 2D	HEC-RAS 2D	HEC-RAS 2D	HEC-RAS 2D						
Overbank	#DIV/0!	#DIV/0!	#DIV/0!	1.91	1.91	1.67	2.00	3.50	3.50	3/8 ton
Main Channel	#DIV/0!	#DIV/0!	#DIV/0!	1.69	1.69	1.67	2.00	3.50	3.50	3/8 ton
Main Channel & Overbank	#DIV/0!	#DIV/0!	#DIV/0!	1.76	1.76	1.67	2.00	3.50	3.50	3/8 ton

***Note: 6ft selected as conservative thickness to ensure adequate depth for embedment of weir**

DESIGN D50, Equation 4-2 (USBR 2007); BASED ON RAMP INVERT SLOPE										
Revised: 12/27/2021	90% Proposed Conditions									
Discharge	USACE 10yr	USACE 25yr	FEMA 50yr	FEMA 100yr	Calculated D50	MIN D50 (NCHRP 2006)	MAX D50 (NCHRP 2006)	MAX D100 (NCHRP 2006)	THICKNESS (NCHRP 2006)	WEIGHT (NCHRP 2006)
Ramp Location	HEC-RAS 2D	HEC-RAS 2D	HEC-RAS 2D	HEC-RAS 2D						
Overbank	0.00	0.00	0.00	3.24	3.24	2.83	3.46	6.00	6.00	2 ton
Main Channel	0.00	0.00	0.00	2.88	2.88	2.83	3.46	6.00	6.00	2 ton
Main Channel & Overbank	0.00	0.00	0.00	3.00	3.00	2.83	3.46	6.00	6.00	2 ton

Roughened Ramp Engineered Streambed Material Calculations

Project:	**Bellota Weir Modifications**	Prepared: KMF	Date:	20211215
Subject:	**90% Design**	Checked: SPB	Date:	12/27/2021
Task:	**Fish Ramp Engineered Streambed Material (ESM) Design**			
Job#:		No:		

FOR ENGINEERED STREAMBED MATERIAL SIZING

USACE. 1994. Hydraulic Design of Flood Control Channels (page 3-8)

CDFG. 2009. California Salmonid Stream Habitat Restoration Manual Part XII (page 67-70)

US Army Corps of Engineers Rock Sizing Equations

$D_{30\text{-ACOE}} = 1.95S^{0.555}(1.25q)^{0.667}/g^{0.333}$

Where:

Fill in yellow cells

$D_{30\text{-ACOE}}$ =			D_{30} stable particle size based on rock gradation provided in USACE (1994) (ft)
S (ft/ft)=	0.0087		S = Bed Slope (ft/ft) (EGL slope used due to presence of backwater/tailwater influence; See Foundation Rock Sizing Calculations for EG Slope data)
unit q (cfs/ft)=	95		q = unit discharge within the active channel at the stable bed design flow (cfs/ft) (Unit discharge is equal to the maximum 100-yr (12,960 cfs total) unit discharge in the roughened channel based on HEC-RAS 2D results)
g=	32.2		g = gravitational acceleration (32.2 ft/s^2)
n=	0.4	0.5	n should be between 0.45 and 0.70, and chosen such that the D8 is 2mm (0.0065')
$D_{30\text{-ACOE (ft)}}$ =	1.07		

CDFG (CFFW) ESM Ratios

	No Rock Structures	With Rock structures
$D_{84\ ESM\ (ft)}$ =	1.60	0.64
$D_{50\ ESM\ (ft)}$ =	0.64	0.26
$D_{100\ ESM\ (ft)}$ =	4.00	1.60
$D_{16\ ESM\ (ft)}$ =	0.037	0.026
$D_{8\ ESM\ (ft)}$ =	0.0066	0.0066

Bates

$$D_{84\text{-ESM}} = 1.5\ D_{30\text{-ACOE}}$$

$$D_{50\text{-ESM}} = 0.4\ D_{84\text{-ESM}}$$

$$D_{100\text{-ESM}} = 2.5\ D_{84\text{-ESM}}$$

Fuller-Thompson

$$D_{16-ESM} = 0.32^{\frac{1}{n}} D_{50-ESM}$$

$$D_{8-ESM} = 0.16^{\frac{1}{n}} D_{50-ESM}$$

Guidance for inclusion of large rock structures

Sizing of Rock Steps and other Large Rock Structures

For roughened channel steps, chutes or cascades, the larger rocks in the composite engineered streambed material, from $D_{84\text{-ESM}}$ to $D_{100\text{-ESM}}$ in size, are separated from the gradation and used to construct steps and other stable large rock features. The remainder of the material ($\leq D_{84\text{-ESM}}$) is used to form the bed between the steps. Although these large rock features are formed by material ranging in size ranging between the $D_{84\text{-ESM}}$ to $D_{100\text{-ESM}}$, the actual volume of material needed to construct these features should be calculated independently.

Engineered Streambed Material Mix

Calculated Mix					Material Mix (excluding D84 - D100)				
					From (ft)	To (ft)	Incremental %	Cumulative %	
D84 - D100	16%	1.60	to	4.00 (ft)					
D50 - D84	34%	0.64	to	1.60 (ft)	0.64	1.60	40.5%	100.0%	
D16 - D50	34%	0.037	to	0.64 (ft)	0.037	0.64	40.5%	59.5%	
D8 - D16	8%	0.0066	to	0.037 (ft)	0.0066	0.037	9.5%	19.0%	
to D8	8%	Fines			0.0000	0.0066	9.5%	9.5%	

D100 (ft)	D84 (ft)	D50 (ft)	D16 (ft)	D8 (ft)
1.60	1.22	0.50	0.027	0.002

Project:	**Bellota Weir Modifications**	Prepared:	KMF	Date:	12/15/2021
Subject:	**90% Design**	Checked:	SPB	Date:	12/27/2021
Task:	**Fish Ramp Engineered Streambed Material (ESM) Design**				
Job#:		No:			

Summary - Stream Simulation Bed Material Design

Reference WSDOT. 2022. *Standard Specifications*. Division 9-03.11 Streambed Aggregates.

Project:	
By:	

Design Gradation:				
Location:	Streambed Design			
	D_{100}	D_{84}	D_{50}	D_{16}
ft	**1.60**	**1.22**	**0.50**	**0.03**
in	**19.2**	**14.6**	**6.0**	**0.3**
mm	**488**	**372**	**151.9**	**8.3**

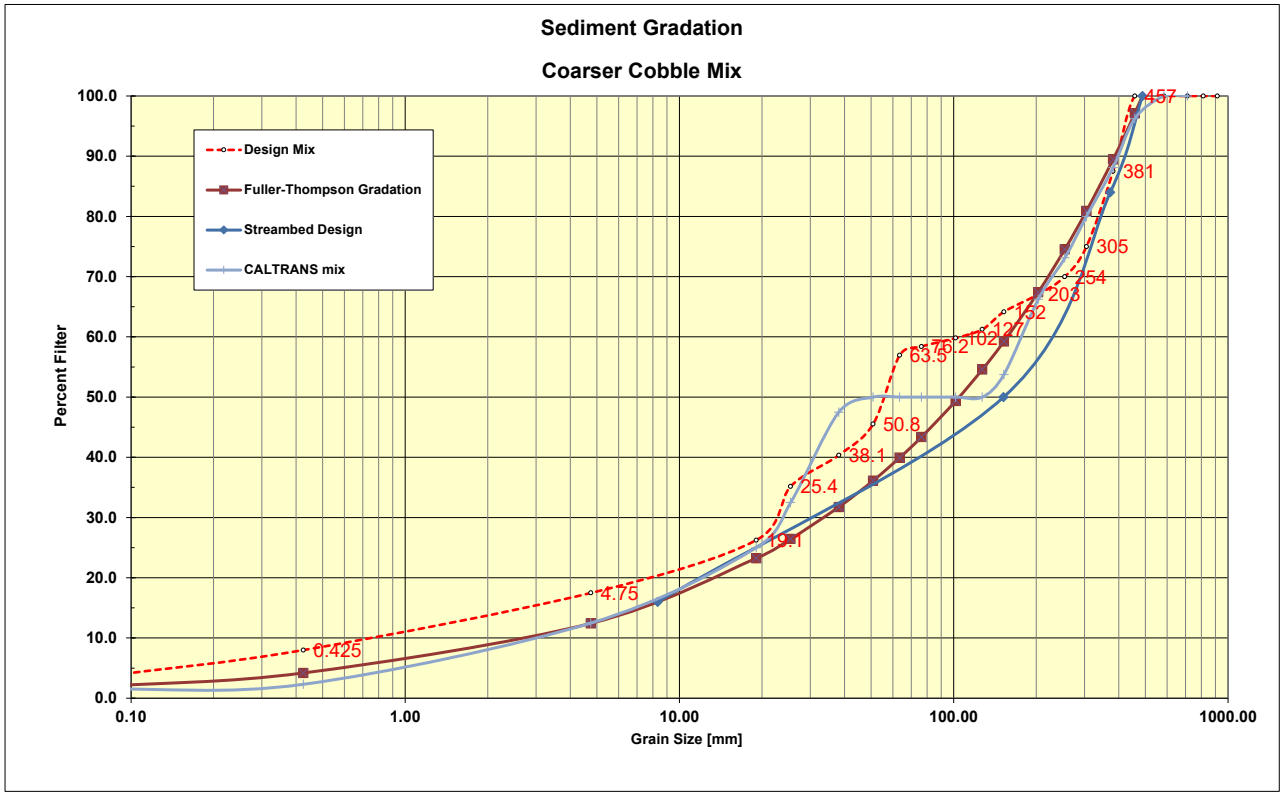
Design Gradation:				
Location:				
	D_{100}	D_{84}	D_{50}	D_{16}
ft				
in				
mm				

Determining Aggregate Proportions

Per WSDOT Standard Specifications 9-03.11

Rock Size		Streambed	Streambed Cobbles					Streambed Boulders			D_{size}
[in]	[mm]	Sediment	4"	6"	8"	10"	12"	12"-18"	18"-28"	28"-36"	
36.0	**914**									**100**	**100.0**
32.0	**813**	100	100	100	100	100	100	100	100	50	**100.0**
28.0	**711**	100							**100**		**100.0**
23.0	**584**	100							50		**100.0**
18.0	**457**	100						**100**			**100.0**
15.0	**381**	100						50			**87.5**
12.0	**305**	100					**100**				**75.0**
10.0	**254**	100				**100**	**80**				**70.0**
8.0	**203**	100			**100**	**80**	68				**67.1**
6.0	**152**	100		**100**	**80**	68	57				**64.2**
5.0	**127**	100		**80**	68	57	**45**				**61.3**
4.0	**102**	100	**100**	71	57	**45**	39				**59.8**
3.0	**76.2**	100	**80**	63	**45**	38	34				**58.4**
2.5	**63.5**	**100**	65	54	37	32	28				**57.0**
2.0	**50.8**	**80**	50	**45**	29	25	22				**45.5**
1.5	**38.1**	73	**35**	32	21	18	16				**40.4**
1.0	**25.4**	**65**	20	18	13	12	11				**35.2**
0.75	**19.1**	50	**5**	**5**	**5**	**5**	**5**				**26.3**
0.187	**4.75**	**35**									**17.5**
No. 40 =	**0.425**	**16**									**8.0**
No. 200 =	**0.0750**	**7**									**3.5**
% per category		**50**					**25**	**25**			*--> 100%*

	D95	D84	D50	D35	D16
Mix Size Interpolation	95	84	50	35	16
y1	87.5	75.0	45.5	26.3	8.0
y2	100.0	87.5	57.0	35.2	17.5
x1	381.0	304.8	50.8	19.1	0.4
x2	457.2	381.0	63.5	25.4	4.8
(mm)	**425**	**358**	**55**	**25**	**3**
(inches)	**16.7**	**14.1**	**2.2**	**1.0**	**0.1**
(feet)	**1.39**	**1.17**	**0.18**	**0.08**	**0.01**



Fuller-Thompson Gradation	
Dmax =	19.2
D[in]	
36.0	132.68
32.0	125.83
28.0	118.49
23.0	108.45
18.0	97.13
15.0	89.48
12.000	80.93
10.000	74.55
8.000	67.43
6.000	59.24
5.000	54.58
4.000	49.36
3.000	43.37
2.500	39.95
2.000	36.14
1.500	31.75
1.000	26.45
0.750	23.24
0.187	12.44
No. 40 =	4.20
No. 200 =	1.92

Project:	**Bellota Weir Modifications**	Prepared:	KMF	Date:	12/15/2021
Subject:	**90% Design**	Checked:	SPB	Date:	12/27/2021
Task:	**Fish Ramp Engineered Streambed Material (ESM) Design**				
Job#:		No:			

Streambed Sediment (50% of Total ESM Mix)

Rock Size		Streambed Sediment	Streambed Cobbles					Streambed Boulders			D_{size}
[in]	[mm]		4"	6"	8"	10"	12"	12"-18"	18"-28"	28"-36"	
36.0	**914**									**100**	**100.0**
32.0	**813**	100	100	100	100	100	100	100	100	50	**100.0**
28.0	**711**	100							**100**		**100.0**
23.0	**584**	100							50		**100.0**
18.0	**457**	100						**100**			**100.0**
15.0	**381**	100						50			**100.0**
12.0	**305**	100					**100**				**100.0**
10.0	**254**	100				**100**	**80**				**100.0**
8.0	**203**	100			**100**	**80**	68				**100.0**
6.0	**152**	100		**100**	**80**	68	57				**100.0**
5.0	**127**	100		**80**	68	57	**45**				**100.0**
4.0	**102**	100	**100**	71	57	**45**	39				**100.0**
3.0	**76.2**	100	**80**	63	**45**	38	34				**100.0**
2.5	**63.5**	**100**	65	54	37	32	28				**100.0**
2.0	**50.8**	**80**	50	**45**	29	25	22				**80.0**
1.5	**38.1**	73	**35**	32	21	18	16				**72.5**
1.0	**25.4**	**65**	20	18	13	12	11				**65.0**
0.75	**19.1**	50	**5**	**5**	**5**	**5**	**5**				**50.0**
0.187 (No 4)	**4.75**	**35**									**35.0**
No. 40 =	**0.425**	**16**									**16.0**
No. 200 =	**0.0750**	**7**									**7.0**
Percent Per Category		**100**									

Sediment Mix Size Interpolation

	D95	D84	D50	D35	D16
% cum.	95	84	50	35	16
y1	80.0	80.0	50.0	35.0	16.0
y2	100.0	100.0	50.0	35.0	16.0
x1	50.8	50.8	19.1	4.8	0.4
x2	63.5	63.5	19.1	4.8	0.4
(mm)	**60**	**53**	**19**	**5**	**0.43**
(inches)	**2.4**	**2.1**	**0.8**	**0.2**	**0.017**
(feet)	**0.20**	**0.17**	**0.06**	**0.02**	**0.0014**

Streambed Sediment Size Lookup Tables

% cum.	Diameter (mm)	% cum.	Diameter (mm)
100.0	63.5	0.0	0.000
80.0	50.8	7.0	0.075
72.5	38.1	16.0	0.425
65.0	25.4	35.0	4.75
50.0	19.1	50.0	19.1
35.0	4.75	65.0	25.4
16.0	0.425	72.5	38.1
7.0	0.075	80.0	50.8
0.0	0.000	100.0	63.5

Streambed Cobbles (50% of Total ESM Mix)											
Rock Size		Streambed Cobbles					Streambed Boulders			D_{size}	$D_{size,}$ ESM 50% sed, 50% cobble
[in]	[mm]	4"	6"	8"	10"	12"	12"-18"	18"-28"	28"-36"		
36.0	914	100	100	100	100	100	100	100	**100**	100.0	50.0
32.0	813	100	100	100	100	100	100	100	**50**	100.0	50.0
28.0	711	100	100	100	100	100	100	**100**		100.0	50.0
23.0	584	100	100	100	100	100	100	**50**		100.0	50.0
18.0	457	100	100	100	100	100	**100**			100.0	100.0
15.0	381	100	100	100	100	100	**50**			75.0	87.5
12.0	305	100	100	100	100	**100**				50.0	75.0
10.0	254	100	100	100	**100**	**80**				40.0	70.0
8.0	203	100	100	**100**	**80**	**68**				34.2	67.1
6.0	152	100	**100**	**80**	**68**	**57**				28.3	64.2
5.0	127	100	**80**	**68**	**57**	**45**				22.5	61.3
4.0	102	**100**	**71**	**57**	**45**	**39**				19.6	59.8
3.0	76.2	**80**	**63**	**45**	**38**	**34**				16.8	58.4
2.5	63.5	**65**	**54**	**37**	**32**	**28**				13.9	57.0
2.0	50.8	**50**	**45**	**29**	**25**	**22**				11.1	45.5
1.5	38.1	**35**	**32**	**21**	**18**	**16**				8.2	40.4
1.0	25.4	**20**	**18**	**13**	**12**	**11**				5.4	35.2
0.75	19.1	**5**	**5**	**5**	**5**	**5**				2.5	26.3
0.187 (No 4)	4.75									0.0	17.5
No. 40 =	0.425									0.0	8.0
No. 200 =	0.0750									0.0	3.5
Percent Per Category						50	50				

Cobble Mix Size Interpolation

	D95	D84	D50	D35	D16	D10
% cum.	95	84	50	35	16	10
y1	75.0	75.0	50.0	34.2	13.9	8.2
y2	100.0	100.0	50.0	40.0	16.8	11.1
x1	381.0	381.0	304.8	203.2	63.5	38.1
x2	457.2	457.2	304.8	254.0	76.2	50.8
(mm)	**441**	**407**	**305**	**210**	**72**	**46**
(inches)	**17.4**	**16.0**	**12.0**	**8.3**	**2.9**	**1.8**
(feet)	**1.45**	**1.33**	**1.00**	**0.69**	**0.24**	**0.15**

Cobble Size Lookup Tables

% cum.	Diameter (mm)	% cum.	Diameter (mm)
100.0	457	0.0	0
75.0	381	0.0	0
50.0	305	0.0	5
40.0	254	2.5	19
34.2	203	5.4	25
28.3	152	8.2	38
22.5	127	11.1	51
19.6	102	13.9	64
16.8	76	16.8	76
13.9	64	19.6	102
11.1	51	22.5	127
8.2	38	28.3	152
5.4	25	34.2	203
2.5	19	40.0	254
0.0	5	50.0	305
0.0	0	75.0	381
0.0	0	100.0	457

Appendix C. Geotechnical Exploration and Design Report

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DRAFT (90%) Geotechnical Investigation Report

Bellota Weir Modifications Project

Mormon Slough, Calaveras River, CA

February 7, 2022

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Appendix B. 2004-2005 Historic Boring Logs

Appendix C. 2021 Supplemental Boring Logs

Appendix D. Laboratory Testing

Appendix E. Weir Seepage and Stability Analysis

Appendix F. Filter Analysis

Acronyms

AOS	apparent opening size
ASCE	American Society of Civil Engineers
ASTM	American Society for Testing and Materials
CBC	California Building Code
El.	elevation
FEMA	Federal Emergency Management Agency
H:V	Horizontal:Vertical
HDR	HDR Engineering, Inc.
Horizontal Datum	CCS83, Zone 3 (horizontal datum)
IBC	International Building Code
NAVD88	North American Vertical Datum of 1988
OSHA	Occupational Safety and Health Administration
pcf	pounds per cubic foot
Project	Bellota Weir Modifications Project
psf	pounds per square foot
SEWD	Stockton East Water District
SPT	Standard Penetration Test
USACE	U.S. Army Corps of Engineers

1 Introduction

1.1 Project Description

The Stockton East Water District (SEWD or District) retained the professional services of KSN, Inc. and HDR Engineering, Inc. (HDR) to advance the Bellota Weir Modifications Project (Project) in San Joaquin County California (Figure 1; all figures provided in Appendix A). The Bellota Weir is located immediately downstream of the Calaveras River bifurcation between the Mormon Slough and Old Calaveras River, about 17 miles downstream of the New Hogan Dam and immediately northeast of the intersection of State Route 26 and Escalon - Bellota Road.

The Project includes weir and intake improvements that will provide more reliable water delivery systems for SEWD customers, improve migration of anadromous fish, and reduce fish entrainment at the facilities, thereby enhancing populations of anadromous salmonids present in the Calaveras River. Initial efforts for this Project included alternative refinement and conceptual design activities in collaboration with SEWD, resource agencies, regulatory authorities, and other Project partners. The Concept Formulation and Design prepared by KSN and HDR (2021) summarize the key background information and process used to identify potential viable options and select a conceptual design that balances the opportunities, constraints, and objectives of this Project.

The current design has progressed to the 90% design level (Figure 2). The Project consists of:

- A new pile supported gated weir (automatic Obermeyer gate system) replacing the existing weir and flashboard system. The weir will be constructed with sheet pile cutoff along the upstream edge of the weir to cutoff seepage flow through the underlying granular channel deposits. Downstream of the weir a gently sloping roughened fishway will be constructed.
- Constructing a combined fish ladder and fish exclusion facility along the north bank that will allow fish to pass the new weir structure and divert flow to the Old Calaveras River and Bellota Intake. All structures will be pile supported. A wall will be constructed along the northside of the roughened channel to direct sluice water past the roughened channel.
- The fish exclusion structure will transfer water north into the Old Calaveras River through two parallel 54-inch-diameter piles discharging through a new headwall structure. It will also discharge water south to the Bellota Intake through three steel pipes that will cross beneath the new weir structure, beneath Mormon Slough, and connect to a new transition structure south of the existing intake structure.
- A new fish barrier, consisting of an earthen berm, to preclude fish entering the Old Calaveras River Channel from the Mormon Slough.

- Constructing appurtenant infrastructure for the fish screen project such as the Control Building/shop, antenna pad, crane pad, and roadway improvements

1.2 Purpose and Scope

The geotechnical investigation was conducted to:

- Perform four supplemental exploratory borings and associated laboratory testing to characterize subsurface surface conditions along the north bank of Mormon Slough and one exploratory boring at the southern end of the existing weir to confirm conditions on the south bank of Mormon Slough.
- Characterize subsurface conditions and develop geotechnical recommendations for the design of the Project structures
- Prepare this characterization report.

1.3 Coordinates Units

The vertical datum for this work is the North American Vertical Datum of 1988 (NAVD88) and the horizontal datum is the CCS83, Zone 3 (Horizontal Datum).

1.4 Codes and Standards and Units

Because the Project site was determined not to be located within the U.S. Army Corps of Engineers (USACE) federal interest, it will be designed in accordance with SEWD criteria which is applicable sections of the 2018 International Building Code (IBC) and 2019 California Building Code.

2 Geotechnical Explorations

2.1 Past Investigations

Prior to conducting the field exploration and laboratory testing, the following geotechnical reports prepared for the site were reviewed:

- Preliminary Geotechnical Conditions and Recommendations – Bellota Weir Fish Passage Facilities for the Calaveras River Anadromous Fish Protection Project prepared by CH2MHILL in 2005. The study included advancing five borings along the Mormon Slough south bank to depths between 30 and 76 feet.

The borings were drilled in October 2004 and January 2005 for the then proposed fish screen location on the Mormon Slough south bank. Figure 3 shows the approximate locations of the previous explorations and Appendix B provides the exploration logs.

2.2 HDR Investigations

HDR conducted supplemental exploration in December 2020 focused on the Mormon Slough north bank in particular for Fish Weir option 7B. The explorations included

advancing five borings to depths of 70 feet, Boring B-1 on the south bank to confirm the conditions described by the CH2MHILL borings and four borings (B-2 through B-5) on the Mormon Slough north bank. A summary of the exploration program and supplemental exploration logs are included in Appendix C. The location of the borings is summarized in Table 1.

Table 1. Summary of Geotechnical Explorations

Explorations	Depth (feet)	Latitude	Longitude
HDR-B1	70	38.052124	-121.011166
HDR-B2	70	38.052257	-121.012052
HDR-B3	70	38.052581	-121.011907
HDR-B4	70	38.052946	-121.011322
HDR-B5	70	38.053055	-121.011818

2.3 Laboratory Testing

Geotechnical laboratory testing was conducted for selected soil samples obtained during each subsurface investigation program. The CH2MHILL study submitted samples to Taber Consultants Laboratory in West Sacramento, California for moisture content, dry density, gradation, and Atterberg limits testing.

Selected samples from the current investigation were submitted to Gulf Shores Laboratory in Rancho Cordova, California for testing, which included:

- Six Fine Sieve Analyses (American Society for Testing and Materials [ASTM] D6913)
- Eight - #200 Sieve Wash (ASTM D1140)
- Eleven Atterberg Limits (ASTM D4318)
- Six Moisture Density (ASTM D2937)
- Two Unconsolidated Undrained Triaxial Shear (ASTM 2850)

Laboratory test results are presented on the boring logs at the appropriate sample depths in 0 through Appendix D.

3 Geologic Hazards

3.1 Geologic Setting

3.1.1 Regional Geology

San Joaquin County lies in the region of the confluence of the San Joaquin and Sacramento rivers. The San Joaquin Valley is bordered on the west by the coast ranges and to the east by the foothills of the Sierra Nevada. The San Joaquin Valley basin has

been filled over time with up to a 6-mile-thick sequence of interbedded clay, silt, sand, and gravel deposits. The sediments range in age from more than 144 million years old (Jurassic Period) to less than 10,000 years (Holocene Period). The most recent sediments consist of coarse-grained (sand and gravel) deposits along river courses and fine-grained (clay and silt) deposits located in low lying areas or flood basins and are referred to as alluvial deposits. These deposits are loose and not well consolidated soils. Older alluvial deposits underlie the edges of the valley. The older alluvial deposits are exposed in the foothill regions in the eastern portion of the county.

3.1.2 Site Geology

The site is mapped as being underlain by the lower member of the Modesto Formation, a quaternary age alluvial deposit that has infilled a narrow valley between ridges (Figure 4; Wagener et al. 1981). The lower member of the Modesto Formation is composed primarily of sand, a substantial proportion of the deposits is well-stratified silt and fine sand, especially near the base of the unit and toward alluvial fan toes. Gravel also makes up a significant part of the lower member, particularly close to the foothills near or above the lower Modesto fan apexes (Marchand and Allwardt 1981). The ridges north and south of the site are mapped as Pleistocene and/or Pliocene sandstone, shale and gravel deposits (https://maps.conservation.ca.gov/cgs/gmc/). Wagner et al. mapped the ridges as comprising Laguna and Mehrten formations.

3.1.3 Regional Seismicity

The site is situated within a region traditionally characterized by numerous active faults and high seismic activity. Many faults exist in the region and are capable of producing earthquakes, which may cause strong ground shaking at the Project site. The nearest active faults are shown in Figure 5 and summarized in Table 2.

An active fault is a fault that has experienced seismic activity during historic time (since roughly 1800) or exhibits evidence of surface displacement during Holocene time (Bryant and Hart 2007). The definition of potentially active varies, however is generally a fault showing evidence of displacement that is older than 11,700 years (Holocene age) and younger than 2.6 million years (Pleistocene age). However, potentially active is no longer used as criteria for zoning by the California Geologic Society.

Table 2. Near Surface Faults within 100 km (62 miles) of the Site

Fault	Distance to Site km (miles)	M_{Max}
Foothill Fault Zone	22 (13)	6.8
Vernalis Fault	48 (30)	6.0
Great Valley Thrust Fault	63 (39)	6.7
Greenville Fault	74 (46)	6.8
Calaveras Fault	97 (60)	6.8
Green Valley Fault	96 (60)	6.7
Midland Fault	60 (37)	6.6

km = kilometers

3.2 Geologic Hazards

3.2.1 Fault Rupture

Based on the information provided in Bryant and Hart (2007), by the California Geologic Survey (2000), and by the Caltrans ARS on-line report (V2.3.06), no known active faults or Alquist-Priolo earthquake zones are present in San Joaquin County. Although the site is within the Central Valley part of California, which is considered to be seismically stable, earthquake activity in neighboring regions, namely the Sierra Nevada and San Francisco Bay area, could affect the proposed Project site with ground shaking. However, the closest active fault to the site is the Vernalis Fault which is located 30 miles west of the Project site. The Foothill Fault Zone is listed in Table 2, however, the segments of the foothills fault system located to the east are generally not considered active nor used as independent seismic sources by the United States Geological Survey (2013).

3.2.2 Liquefaction

Earthquake-induced soil liquefaction can be described as a significant loss of soil strength and stiffness caused by an increase in pore water pressure resulting from cyclic loading during shaking. Liquefaction is associated primarily with loose (low density), saturated, fine- to medium-grained, cohesionless soils below the groundwater table, but can also occur in non-plastic to low-plasticity finer grained soils. The potential consequences of liquefaction to engineered structures include loss of bearing capacity, buoyancy forces on underground structures, ground oscillations or cyclic mobility, increased lateral earth pressures on retaining walls, liquefaction settlement, and lateral spreading or flow failures in slopes.

According to the San Joaquin County Community Development Department online natural hazards disclosure page, no areas within the county have been identified as Seismic Hazard Zones that include the occurrence of liquefaction. The CH2MHILL report identified the south bank of Mormon Slough had the potential to experience liquefaction. The potential for liquefaction at the site is moderate. The liquefaction analysis is discussed in Section 5.2.

3.3 Floods

The Federal Emergency Management Agency (FEMA) Flood Insurance Rate Maps for the site (FEMA 2010) indicate the Project site is mapped as Zone X, which is an area where the flood hazard exceeds 0.2 percent annual chance of flood hazard (Figure 7). Mormon Slough is also identified as a regulated flood way. The slough, located near the Weir, is not leveed and is downstream of the USACE project levee system.

4 Site Conditions

4.1 Surface Conditions

The site is located at the confluence of the Old Calaveras River and Mormon Slough between California State Route 26 on the west, north of Shelton Road on the southeast, and Old Calaveras River on the north. Outside of the channels, the site is relatively flat with elevation range from El. 130 feet near the channel to El. 132 feet near the roadways. The site is covered with native grasses and trees. The elevations within the channels range from approximately El. 130 feet near the top of bank to El. 108 feet within the channel (Figure 2) with local scour holes upstream of the weir extending down to El. 104 feet and 95 feet downstream of the weir. The banks of the slough and river channel are generally over steepened at slope of 1.5H:1V (Horizontal:Vertical) or flatter.

Site development has generally been limited to the Bellota Intake on the south bank, Bellota Weir, and Old Calaveras River Headworks Gate across the old river channel (approximately 570 feet west of Mormon Slough). Review of available historic topographic maps and aerial photographs indicate the site has remained relatively unchanged since the reconstruction of the Bellota Weir in the late 1960s and early 1970s. Review of a 1967 aerial photograph indicates there were once medium sized buildings located on the south bank, setback from the river which were subsequently removed. A structure was also present adjacent to and south of the Old Calaveras Headworks Gate Structure. The structures are shown on the 2002 U.S Geological Survey Topographic 7.5-minute quadrangle but were removed from the maps in the 2012 update. A 1993 aerial photograph shows the buildings had been removed prior to 1993. An existing house is located approximately 400 feet west of the weir that is shown on the 2002 topography.

4.2 Subsurface Conditions

Subsurface conditions at the site have been interpreted based on the 2005 and 2021 subsurface investigations discussed in Section 2.0 and presented in Appendix B and Appendix C. The locations of the borings are shown on Figure 7 and three cross sections presented in Figure 8 through Figure 10. In general, site conditions consist of interbedded silts, silty sands, sands with variable percentages of gravel, silty sand, sandy gravels and clay typical for the riverine environment that can be divided into three layers: upper overbank deposits, a middle alluvial layer, and older, stiffer overbank deposits that extend to 70 feet, the exploration depth.

Overbank deposits generally consist of loose brown silty sand and sandy silt to a depth of 20 to 25 feet. The underlaying alluvial layer varies between 20 and 25 feet thick and is comprised of a 15-foot-thick upper fine grained layer consisting of interlayered loose silts, silty sands, silty clayey sands, and soft clays with discontinuous loose gravel deposits within the meandering river channel. The lower layer has a coarser grained layer consisting of loose silty to clayey sands with varying gravel content and more pronounced gravel zones. Older medium dense to hard alluvium is located at a depth of

40 feet and consists of alternating layers of silt, and silty sand with some poorly graded sand layers encountered at depth.

More recent alluvium is located within the channel with the south bank consisting of a gravel bar. The existing weir includes an upstream sheet pile cutoff wall to cutoff seepage flow beneath the existing weir. Review of as-built drawings show that the cutoff wall extends 8 feet below the weir abutments and increased in depth to 14 feet beneath the central portion of the weir. The cutoff wall was likely deepened to extend through more permeable channel deposits to maintain an 8-foot depth of wall penetration into the upper alluvial layer (Figure 9). This assumption indicates that up to 6 feet of recent alluvium is located below the weir and roughly correlates to the depth of the gravel bar encountered in the south bank borings.

4.3 Groundwater

Groundwater correlated with the water level in the slough and impounded by the weir. The Mormon Slough flow is controlled by releases from New Hogan Dam located 11.7 miles northeast of the weir. Groundwater was encountered between El. 108 and 110 feet during the recent explorations. The lowest groundwater level was measured in boring HDR-B2 located on the right bank downstream of the weir.

5 Discussion and Conclusion

5.1 General

Geotechnically, the Project is feasible with implementing the recommendations provided in this report. The key geotechnical considerations are the presence of loose granular deposits (Overbank and Upper Alluvium) above El. 100 feet and its effect on foundation selection and seismic performance; seismic ground shaking and the potential for limited liquefaction resulting in seismically induced settlement; presence of over steep river banks with respect to static and seismic stability; and considerations for construction phases when working in the river.

The proposed weir and fish passage structures will require support from deep foundations extending below El. 90 feet and loaded in tension (uplift) and compression. A sheetpile cutoff will be installed just upstream of the weir to provide a seepage cutoff through the permeable channel deposits and prevent through seepage erosion beneath the weir foundation. The existing north bank of the slough and south bank of the Old Calaveras River channel are over steepened and subject to instability under both static and dynamic loading, which needs to be accounted for in the design. The Project will be phased as follows:

Phase 1 – Construct dam sill and roughened channel (including sheetpile cutoff)

Phase 2 – Construct dam super structure and mechanical gates

Phase 3 – Existing intake, Old Calaveras River Improvements

Phase 4 – Fish ladder and sluiceway construction

Phase 5 – Remaining portions of Bellota Conveyance Structure and tying into the existing intake/conveyance pipeline

Relevant geotechnical discussion on each of these items follows and specific geotechnical recommendations are provided in Section 6 of this report

5.2 Liquefaction

Liquefaction evaluations were based on the data presented in the 2005 boring logs and mud rotary wash borings performed for this investigation. For the soil layers encountered in the mud rotary borings, the soil composition criteria in Seed et al. (2003) was used to evaluate the liquefaction susceptibility. For soil layers determined to be potentially liquefiable, a liquefaction triggering analysis was performed using the methodologies in Idriss and Boulanger (2008). The liquefaction induced settlements were estimated using volumetric strain criteria in Tokimatsu and Seed (1987). The depth to groundwater measured at the time of drilling was used in the analysis cyclic strength resistance (Table 3). The groundwater at the time of the 2005 explorations was not documented in available reports, but was assumed approximately equal at El. 112 feet, the approximate bottom elevation of the weir, which is slightly above the highest groundwater elevation recorded in borings HDR-B3 and HDR-B4.

Earthquake magnitude and peak ground acceleration were determined using the 2008 Probabilistic Seismic Hazard Analysis Interactive Deaggregation website (https://geohazards.usgs.gov/deaggint/2008/). The analysis used an earthquake magnitude of 6.5 and a peak ground acceleration of 0.287g for the liquefaction analysis. A site-specific ground motion hazard analysis was not performed. The analysis was performed with groundwater at both the maximum and minimum pool elevation assuming that the groundwater would equal the pool water surface elevation at each boring. Review of the groundwater elevations at the time of drilling summarized in Table 3 suggest that Boring HDR-B2, located downstream of the weir, would have a lower groundwater elevation than assumed in the analysis.

Minimum factor of safety for a liquefaction hazard assessment are provide in the 2009 National Earthquake Hazards Reduction Program. Recommended seismic provisions that reference work published by Martin and Lew (1999). In general, the corrected equivalent clean sand Standard Penetration Test (SPT) blow counts in the potential liquefiable layers at the site are less than 15. Therefore, the following minimum factors of safety are recommended:

Settlement	1.1
Surfacing Manifestation	1.2
Lateral Spread	1.3

The site is comprised predominately of loose to medium dense sands silts and gravels above El. 90 feet. Below El. 90 feet the soils consist of alternating layers of medium dense to hard silts and silty sands. The soil composition of the upper soils meets the criterial of potentially liquefiable by the criteria presented in Seed et al. (2003). The liquefaction triggering analyses confirmed the 2005 investigation finding that the south bank contains soils with high potential for liquefaction and, if not mitigated, would result

in seismically induced surface settlements from soil layers that have factors of safety against liquefaction between 1.0 and 1.3. These layers range in thickness from 5 to 10 feet and are located between El. 100 and 115 feet, 10 to 20 feet below the ground surface. Analysis of the current investigation of the north bank found that the site has less potential for liquefaction, however, would also experience seismically induced settlements due to layers with factors of safety against liquefaction between 1.0 and 1.3.

No layers had a factor of safety against liquefaction less than 1.0 and the risk of strength loss is judged to be low, the risk of surface manifestation (sand boils) on the ground surface is judged to be extremely low. However, some surface manifestation could occur within the channel or south bank gravel bar deposits due to low confining pressures.

Excavation and replacement as a means to mitigate the low-density and saturated layers may not be a preferred alternative due to their depth below the top of bank. Alternatively, the layers could possibly be densified by performing compaction grouting; however, the high fines content layers could result in limited effective densification. Stone columns could also be installed on a grid pattern to improve and strengthen the entire soil mass. A typical stone column grid pattern could be 5 to 8 feet on center, in either a triangular or square grid, and focused along the riverbanks and beneath the fish ladder footprint, box culvert alignments, Bellota Intake Structure, and Bellota Weir. Stone columns would not replace the deep foundation needed to support the fish passage and weir.

Settlements were calculated using the volumetric strain procedure (Ishihara and Yoshimine 1992). The probability of liquefaction was compared to the relationship developed by Cetin et al. (2018). Post seismic settlement was estimated for soil layers calculated to have a factor of safety less than 1.2 for the design peak ground acceleration. Calculated post-liquefaction settlements for each boring are provided in Table 3 and range from no settlement in boring HDR-B4 to 5.8 inches in Boring BI-1 located adjacent to the Bellota Intake Structure. The settlements are greatest for the maximum pool elevation (El. 121.4 feet) and the design input acceleration (0.287g) with the probability of settlement occurring from 50 to 80 percent. The probability of seismically induced settlement reduces dramatical for accelerations less than 0.2g.

Table 3. Summary of Calculated Liquefaction Settlement

Boring ID	Location	Ground El. (NAVD88)	Depth to Groundwater at Time of Drilling (feet)	Approx Groundwater El. at Time of Drilling (feet)	Min Pool Liquefaction Settlement[1] (inches)	Max Pool Liquefaction Settlement[1] (inches)	Depth to Liquefiable Layers (feet)
HDR-B1	Left Abutment	124	18	106	0.2	0.2	2.5 ft thick layer at El. 108.2 ft
HDR-B2	North Bank Downstream	132	30	102	1.1	1.1	5 ft thick layer at El. 101.2 ft
HDR-B3	North Bank Right Abutment	135	25	110	2.5	2.5	10 ft thick layer at El. 104.2 ft
HDR-B4	Upstream North Bank	131	21	110	0	0	N/A
HDR-B5	South Bank of Old	131	25	106	0.1	0.1	5 ft thick layer at El. 105.2 ft

Boring ID	Location	Ground El. (NAVD88)	Depth to Groundwater at Time of Drilling (feet)	Approx Groundwater El. at Time of Drilling (feet)	Min Pool Liquefaction Settlement[1] (inches)	Max Pool Liquefaction Settlement[1] (inches)	Depth to Liquefiable Layers (feet)
	Calaveras River						
BD-2	Weir Right Abutment	113.9	1.9	112 [2]	2.4	2.4	4 ft thick layer at El. 104.1 ft and 5 ft thick layer at El. 74.1 ft
BF-1	Weir Left Abutment	122.4	10.4	112 [2]	2.2	2.2	5 ft thick layer at El. 96.6 ft, 5 ft thick layer at El. 76.6 ft, and 5.7 ft thick layer at El. 45.9 ft
BI-1	SEWD Intake	123.0	11	112 [2]	5.8	5.8	10 ft thick layer at El. 102.2 ft (3.5 inches), 5 ft thick layer at El. 92.2 ft (1.1 inches), and 10 ft thick layer at El. 72.2 ft (1.2 inches)
BI-2	South Bank Gravel Bar	116.4	4.4	112 [2]	1.1	1.1	5 ft thick layer at El. 100.6 ft and 3.5 ft thick layer at El. 87.1 ft
BI-3	South Bank Gravel Bar	115.9	3.9	112 [2]	0.5	0.5	5 ft thick layer at El. 107.6 ft and 5 ft thick layer at El. 92.6 ft

1. Minimum Pool El. 115.44 feet and Maximum Pool El. 121.44 feet

2. Historic Borings performed by others. Groundwater elevation at time of drilling not reported and is assumed to be equal to the base of the existing weir for these analyses.

3. Peak ground acceleration (PGAm) of 0.287g for Site Class D and earthquake magnitude (Mw) of 6.5

NOTE: Depth to Liquefiable Layer is the depth to layers with factor of safety < 1.2 for liquefaction (Elevation of bottom of the liquefiable layers shown).

The calculated post seismic settlement of specific structures are as follows:

Weir Abutments	2.2 to 2.5 inches
Right Bank downstream of Weir	Approximately 1 inch
Bellota Intake	5.8 inches
Proposed Fish Screen Intake	less than ½ inch to 2.5 inches
Old Calaveras River Bypass Outfall	less than ½ inch

Due to wide variation in calculated settlements and the proximity of the proposed structures to the river, differential settlements are anticipated to be significant. The structures should be supported on deep foundations to gain support in the stiff to hard soils in the older alluvium, below El. 90 feet.

5.3 Slope Stability

The Old Calaveras River Bank adjacent to boring HDR-B5 was selected for static and seismic stability analysis as this slope is overly steep and will have limited modifications by the Project. Soil strength properties were developed based on correlations with SPT equivalent clean sand blow counts ($N1_{60}cs$). The results for the stability analyses are shown in Figure 11 through Figure 14. The failure surface with a factor of safety equal to 1.0 is shown on Figure 12. The soil wedge with a factor of safety of 1.5 is a linear wedge projecting up from the toe of slope and setback a distance of 18 feet from the top of bank (44.4 feet from the toe of slope). This figure shows that overtime the upper portion of the slope will continue to erode to meet the factor of safety 1.5 projection line (Figure 13). Therefore, any structure supported with shallow foundations should be setback a distance of 18 feet from the top of slope.

Seismic pseudo – static stability was evaluated with a horizontal acceleration of 0.15g (approximately 50 percent of the peak ground acceleration) assuming the river bank would act as a rigid block (Figure 14). The setback of 18 feet results in a slope with a minimum factor of safety of 1.1 which is acceptable for the seismic case.

5.4 Roadways, Control Building, Antenna Pad, and Crane Pad Subgrade

The upper soils at the site are comprised of loose sand and non-plastic silts that vary in density across the Project site. Therefore, it is recommended that the upper 6 feet of the site be excavated, the material stockpiled and then placed back into the excavation as properly moisture conditioned and compacted engineered fill to provide a suitable foundation support for spread footings, slab-on-grade, and roadways. The excavations should extend a minimum of 5 feet beyond the edge of buildings or slabs. The intake structure and fish passage should be supported on deep foundations.

Care should be taken for the crane pad where a portion of the slab will be located over the deep excavation to install the Bellota Conveyance pipelines. The shape of the excavation to construct the intake structure distribution box and fish ladder is not specified and, depending on how the excavations are backfilled, could result in large differential settlements beneath the pad leading to cracking and potential large vertical and horizontal offsets. Therefore, the subgrade for the entire crane pad should be removed to EL 112 feet at the edge of the conveyance structure and fish ladder and uniformly sloped up to EL 120 feet, 5 feet beyond the north and west edges of the crane pad. The excavation should be backfilled with properly moisture conditioned and compacted native soils as recommended in Section 5.8 to control abrupt differential settlement of the slab. The slab could settle over time likely sloping away from the fish passage structure. Gapping between the slab and the structure could occur overtime and would require routine maintenance to ensure a watertight seal between the slab and the adjacent structures.

5.5 Weir Foundation, Sheetpile Cutoff, Roughen Channel Fill

5.5.1 Sheetpile Cutoff

Review of available as-built drawings show that the weir had been damaged in the late 1960s and the current weir constructed as a replacement. The site was regraded using channel deposits from upstream of the weir. The new weir included installing a sheet pile wall along the upstream edge of the foundation varying in depth from 8 feet at the abutments to 14 feet in the center of the channel (Figure 9). At its deepest point, the sheetpile cutoff appears to have tipped into the lower coarse-grained alluvial layer. The wall appears to cutoff seepage and reduce potential erosion of material placed to support the weir, which may have been the failure mode of the original weir. The basis for the wall depth is not stated on the as-built drawings, however it may be that the wall depth was selected to maintain 8 feet of penetration into the underlying alluvium located below the channel deposits. The method of sheet pile installation also is unknown as well as the current condition of the existing sheetpile wall.

To construct the new weir, the existing weir should be removed, and the foundation excavated to expose at least the top 7 feet of the existing sheetpile wall. Material excavated from underneath the weir likely consists of channel deposits and should be placed as fill within the scour holes upstream and downstream of the weir alignment. The existing sheetpile wall could then be exposed and either extracted and/or cut to the excavated subgrade. New free draining engineered fill could be placed and properly compacted to establish the new weir foundation subgrade for the new sheetpile wall and weir foundation piling to be installed. The new sheetpile wall should extend to a depth of 20 feet below the gated section of the weir and the tip elevation held constant as the wall progresses up the abutments terminating above the full pool water surface. A test pile program should be performed to confirm the appropriate method of pile installation to reach the design depth.

5.5.2 Roughened Channel

The fish weir includes a roughened approach channel downstream of the weir. This area will be subject to underseepage and end around flows, therefore the gradation of the fill should be specified to reduce the potential for transport of finer grained foundation or abutment material into new fill. Migration of finer grained material could, over time, lead to the formation of sinkholes or depressions. A drainage layer should also be included in the fill to provide a drainage pathway to the downstream toe. Section 6.9 provides the analysis and recommendations for the gradation of the filter layer and transition layer section below the roughened channel. It also provides the riprap to be placed on the channel slopes above the roughened channel and within the Old Calaveras River channel.

A seepage model was developed for the weir and roughened channel section (Appendix E). Values of hydraulic conductivity and drained strength for the granular soils were estimated based on material classification and particle size analyses. The inclination of

the roughened channel is less than 2 degrees; thus the channel has a high factor of safety against slope failure during both low and high pool conditions. The cutoff sheetpile wall primary function is to prevent undermining of erodible material beneath the weir slab during high flow conditions. Key to the performance of the weir and the roughened slab is extending the sheetpile cutoff into the underlying older alluvial layer as previously discussed. This modeling shows that the cutoff is effective in reducing downstream underseepage flow into the roughened channel fill. The banks of the slough are comprised of permeable materials and lateral/end around flow will occur, as had occurred for the current weir structure. Seepage flow also will be greater than indicated by the seepage models. Estimating the three-dimensional seepage effects around the weir was beyond the scope of these studies.

5.6 Old Calaveras Conveyance Pipeline

The fish ladder will include fish screens that connect to an afterbay to control flow velocity through the fish screens. Water will then flow into a water distribution structure connecting to the Old Calaveras River and/or the Bellota Intake structure through two systems of pipes. The Old Calaveras River diversion consists of two 54-inch-diameter pipes that extend below grade and discharge into the channel through an outfall structure. The pipe installation would provide a net unloading to the foundation, however post seismic settlement could vary from zero to 2.5 inches between the weir and the outfall structure (Table 3). Options to mitigate the settlement could include ground improvements to reduce settlements to tolerable levels supporting the box culverts on a rigid foundation or deep foundations. The box culvert will likely be constructed by the cut and cover methods or a combination of cut and cover with shoring. Dewatering of the excavation will also be likely. To reduce the potential for ground settling away from the base of the box culvert and provide a stable base to construct on is to install pattern short stone columns to support the culverts (Section 5.2). The width of treatment should be two times the width of the box culverts and extend from the entire length of the pipes. If ground settlement away from the base structure is not a concern for the performance of the diversion, the culvert can be supported on deep foundations.

The Bellota Intake diversion consists of three 54-inch pipes that extend beneath the weir and fish ladder structure and connect to a junction structure with the existing Bellota Distribution Pipeline . The pipelines will be constructed in three phases consisting of a 90 degree bend extending from the fish passage distribution structure on the north bank, extending beneath the weir structure along the downstream side of the sheetpile cutoff, and then extending up the south bank connecting to the Bellota junction structure. Table 3 indicates that the weir could experience 2.5 inches of settlement where it crosses the channel and less than 1 inch at the abutments. Also, the Bellota Intake Structure is founded on a thick sequence of loose granular soils and could experience up to 6 inches of settlement. Therefore, foundation options include supporting the Bellota Intake box culvert on pile foundations similar to those supporting the fish screen structures or treating the foundation with stone column ground improvements when adding deep foundations or soil anchors (such as helical anchors) to provide uplift support. The structural design team has selected installing steel cross members that will span between the weir pile foundations to support the pipelines where they cross under

the weir and encapsulating the pipes in controlled density backfill. The pile will provide support against settlement and uplift displacements.

5.7 Bellota Intake

Analysis of the one 2005 boring located adjacent to the intake structure indicates locally as much as 5.8 inches of post seismic settlement could occur. However, boring BI-1 calculated settlement is more than twice the amount of estimated for the next adjacent boring BF-1. It is recommended that additional cone penetrometer testing be performed around the intake to confirm that the subsurface conditions shown in boring BI-1 are accurate, specifically the 2.3 inches calculated at depth as no other boring drilled at this site calculated post seismic settlements at this depth. The current design has placed the Bellota junction structure further back from the south bank away from the thick loose granular deposits that were encountered along the south bank in an area that has not been investigated.

One approach for designing connections at this junction structure is to not mitigate all settlement but provide flexible couplings that could accommodate up to 1 inch of deferential settlement.

5.8 Construction Sequencing

The Project construction is proposed to occur in five sequential phases to minimize environmental impacts and optimize constructability, cost effectiveness, and operational continuity. The current sequencing strategy and level of effort is based on a minimum 2-year construction period; however, project elements may be constructed in a different order or over a longer period of time to accommodate costs or permitting restrictions.

A construction sequencing summary is provided in this section.

Sequence 1A (Non-Irrigation season, out of water work)

- Complete construction of access roads
- Remove trees

Sequence 1B-1D (Irrigation season, in-water work window)

Construction of sequences 1B-1D will take place during the irrigation season (mid-April to mid-October), during the in-water work window of mid-June to early October. Stream diversion will be provided using the proposed 54-inch sluiceway on the north bank and the existing 36-inch bypass on the south bank.

Sequence 1B

- Complete demolition of existing weir
- Complete removal and salvage of existing riprap
- Installation of sheet pile walls

- Installation of 54-inch sluiceway on north bank and extension of 36-inch bypass pipeline on south bank

Sequence 1C

- Complete in-water grading in Mormon Slough
- Partial construction of Bellota conveyance pipelines (under concrete weir structure)
- Complete construction of weir slab
- Complete pile installation for the weir and debris boom
- Complete construction of the roughened channel

Sequence 1D

- Complete Installation of debris boom components
- Remove temporary access road to roughened channel
- Installation of sheet pile wall S-3A and removal of sheet pile wall S2
- Complete installation of riverbank armoring
- Removal of in-river dewatering systems

Sequence 2 (Non-irrigation season, out-of-water work)

- Complete construction of the concrete fish ladder structure
- Completion of the downstream section of the intake used for sediment sluicing and installation of sluice gates
- Connection of 54-inch sluiceway pipeline to structure
- Complete construction of the diversion structure
- Complete construction of the control building

Sequence 3 (Irrigation season, in-water work window)

- Complete construction of the intake structure
- Complete construction of weir superstructure and installation of Obermeyer weirs

Sequence 4 (Non-irrigation season, out-of-water work)

- Complete construction of the Old Calaveras River conveyance pipelines
- Complete construction of the Bellota conveyance pipelines
- Complete remaining site work

Sequence 5 (Irrigation season, in-water work window)

- Completion of in-water grading in the Old Calaveras River

The Project is constrained to a relatively confined area and the effects of shoring installation, dewatering systems, and excavations immediately adjacent to other structures needs to be carefully evaluated to assess changes in loading conditions.

The phased construction could result in temporary unbalanced loads that may govern the design of a specific element. Portions of the permanent work may serve as support for temporary excavations and impose additional loading conditions that should be evaluated by the design team. Including the effects of differential dewatering, excavation induced lateral soil loads, surcharges, and structural connection between phases of work.

5.8.2 Seasonal Construction and Dewatering

Calaveras Dam and seasonal releases may affect construction. Specifically, the top elevation of temporary sheetpiles, cofferdams, or berms should be specified to provide adequate protection to the work areas and prevent inundation during infrequent releases that could overtop systems and flood the site. Also, seasonal shutdowns may affect the next season's work specifically the removal and re-installation of dewatering systems. The channel deposits and underlying alluvial deposit have a relatively high permeability and dewatering operations may account for high water conditions that continue during winter shutdowns

6 Recommendations

6.1 Seismic Design Criteria

The proposed structures should be designed to resist the lateral forces generated by earthquake shaking in accordance with local design practices. The seismic design parameters based on the American Society of Civil Engineers (ASCE) 7-16 as modified by Supplement #1 and the 2019 California Building Code for a Latitude of 38.052325 and a Longitude of -121.011499 are presented in Table 4.

Table 4. Seismic Design Parameters

Parameter/Coefficient Description	Parameter/Coefficient	Value
MCE_R for 0.2 sec Spectral Response Acceleration, Site Class D	S_s	0.490g
MCE_R for 1.0 sec Spectral Response Acceleration, Site Class D	S_1	0.287g
Soil Profile Type	Site Class	D
Site Coefficient	F_a	1.4
Site Coefficient	F_v	2.15
Adjusted MCE Spectral Design Parameters	S_{MS}	0.690g
	S_{M1}	0.480g
Design Spectral Acceleration Parameters	S_{DS}	0.460g
	S_{D1}	0.320g

Table 4. Seismic Design Parameters

Parameter/Coefficient Description	Parameter/Coefficient	Value
Seismic Design Category	Occupancy Category I to IV	III

MCR_R = Risk targeted maximum considered earthquake

6.2 Earthwork

6.2.1 Stripping and Demolition

The proposed site improvement areas currently consist of existing structures and undeveloped areas. Initial preparation of the site will include demolition of existing facilities (including demolition of existing structures, foundation systems, trees, and vegetation. Existing underground utilities that interfere with the proposed construction should be identified and properly abandoned or relocated.

Areas to receive fill and support structures should be stripped of debris and any other deleterious materials prior to over-excavation or placement of engineered fill. Stripping and removal of debris should extend laterally a minimum of 5 feet beyond the perimeters of roadways, concrete flatwork, and any other facilities supported on grade. Any stripped debris or organic matter should not be reused as engineered fill, but can be reused as non-engineered fill in designated areas or removed offsite. Excavations from removal of foundations, underground utilities, or other below-ground obstructions should be cleaned of loose soil and any potentially deleterious material and backfilled with compacted engineered fill to the requirements provided herein. Excavation backfilling operations should be carried out under the observation of an HDR geotechnical engineer. In addition, active or inactive utilities within the construction area should be protected, relocated, or abandoned.

During construction, soft or loose areas may be encountered especially near or below the existing groundwater level (roughly El. 110 feet). Loose or soft subgrade soils encountered during construction may require over-excavation. Assessment of these areas to determine the extent of over-excavation should be performed by the geotechnical engineer at the time of construction when conditions are exposed.

6.2.2 Densification of Potentially Liquefiable Soils

Granular soils within the riverbanks were found to have factors of safety greater than 1.0 for the peak ground acceleration and although not subject to the occurrence of full liquefaction and strength loss, they will develop pore pressures and experience post-seismic settlements. Total and differential settlements will vary across the site, and ground improvements to densify these soils are recommended. Two institute densification methods that could be performed at this site are compaction grouting and vibro-replacement (atone columns). Compaction grouting is performed by injecting grout at targeted depths creating a pressure bulb that compacts the soil. Vibro-replacement consists of advancing a vibratory probe to the specified depth and placing the constructing a compacted stone column by lifting the probe in increments, introducing

gravel through the bottom feed method and then repenetrating the gravel to compact it and surrounding soils. Both methods require an experienced specialty Contractor to perform the insitu densification.

Due to the variability in the fine-grained soil content compaction grouting may not be a feasible means to perform targeted densification. Instead short stone columns should be installed in a 5- to 8-foot pattern to densify the soils and strengthen the site. Stone columns should be constructed using the bottom up construction method with casing or a vibrating probe to support the soil column as the column is built Gravel is placed in the column in lifts and vibrated to densify the stone end loose granular soil layers. Densification should be performed prior to preparing the site subgrade as discussed in the following section so that disturbed surface soils can be properly prepared.

6.2.3 Subgrade Preparation

The site is blanketed with a loose silt and silty sand layer. Therefore, after stripping and performing any necessary removals, the upper 6 feet of the site should be excavated, moisture conditioned, and compacted to make the subgrade below any footings and slab-on-grades. The excavation should extend 5 feet beyond the limits of the proposed improvements. The material should be placed in accordance with the recommendations provided in Section 6.2.5.

Concrete for foundations and exterior flatwork (e.g., sidewalks or walkways) should be placed neat against undisturbed soil or properly-prepared engineered fill as described in Section 6.2.4. Foundation subgrades should not be allowed to dry out before placing concrete. If shrinkage cracks appear in the excavations, the excavations should be thoroughly moistened to close all cracks prior to concrete placement. The foundation excavations should be observed by the geotechnical engineer for compliance with appropriate moisture control and to confirm the adequacy of the supporting soils. Soft, loose, or otherwise deleterious materials encountered at the bottoms of footing excavations should be removed and replaced with compacted engineered fill per the requirements below.

6.2.4 Engineered Fill Materials

Material used as engineered fill to support structures should have an organic content of less than 3 percent by volume, be free of debris and other deleterious materials, be onsite lean clay with a plasticity index less than 15 and liquid limit less than 40 or granular non-expansive and have a maximum particle size less than 3 inches in dimension. Excavated onsite materials not meeting these requirements should be properly disposed of offsite or, if approved by San Joaquin County, placed as a wide, level, engineered fill. The fill should be setback a minimum 50 feet from the slough banks to not appear as a levee and should not concentrate flow onto adjacent properties. High moisture content soils that meet these requirements are not considered as unsuitable material but will require proper moisture conditioning prior to placement and compaction.

Imported fill should be predominantly granular, meet the size requirements previously discussed, have an organic content of less than 3 percent by volume, have a liquid limit less than 40 percent, have a plasticity index not exceeding 15, should be non-corrosive,

and should contain no environmental contaminants or debris. Imported materials for use as engineered fill should be sampled and tested by the geotechnical engineer prior to being transported to the site. Submitting representative samples of imported materials proposed for use as engineered fill should be submitted to the geotechnical engineer for testing and approval at least one week prior to the start of grading and import of the material.

6.2.5 Fill Placement and Compaction

Prior to fill placement, the subgrade should be prepared as discussed above. Engineered fill placed beneath or around structures should be placed in loose horizontal lifts less than 8 inches, moisture conditioned to 0 to +3 percent above optimum moisture content and compacted to at least 90 percent of relative compaction per ASTM D1557. The upper 2 feet of fill placed as structure, slab-on-grade, or pavement subgrade should be compacted to 95 percent per ASTM D1557.

Additional fill lifts should not be placed if the previous lift did not meet the required relative compaction or if soil conditions are not stable. Disking or blending may be required to uniformly moisture condition soils used for engineered fill. Ponding or jetting compaction methods should not be allowed. Jetting for placement of the roughened channel rock interfill material may be used in a controlled manner to sluice materials between the roughened channel rock fill while not plugging the underlying transition layers. All site preparation and fill placement should be observed by the geotechnical engineer. It is important that during the stripping and scarification processes, a geotechnical engineer representative should be present to observe whether any undesirable material is encountered in the construction area and exposed soils are similar to those encountered during the geotechnical site exploration.

6.3 Temporary Excavations

All excavations should comply with applicable local, state, and federal safety regulations including the current Occupational Safety and Health Administration (OSHA) Excavation and Trench Safety Standards. Construction site safety is the responsibility of the Contractor, who is also solely responsible for the means, methods, and sequencing of construction operations. The Contractor should prepare a temporary support plan for review by the design team to check that it does not impose additional loads on the installed work that was not anticipated during design.

6.3.1 Excavation

Slope height, slope inclination, or excavation depths (including utility trench excavations) should not exceed those specified in local, state, and/or federal safety regulations (e.g., OSHA Health and Safety Standards for Excavations, 29 Code of Federal Regulations Part 1926, or successor regulations). Such regulations are strictly enforced, and, if not followed, the owner, Contractor, or earthwork and utility subcontractors could be liable for substantial penalties. Based on interpretation of the OSHA regulations and the findings from this investigation, the existing near surface soils will likely be predominantly Type C soils. For planning purposes, excavations less than about 20 feet deep should have

maximum allowable slopes of 1.5H:1V. The actual OSHA soil type classification and sloping requirements should be determined by the Contractor's responsible person based on the materials exposed during construction. OSHA requires that excavated slope heights in excess of 20 feet be designed by a professional engineer.

Underground utilities should be located above a 1H:1V plane projected downward from the bottoms of new foundations to avoid undermining the foundations during the excavation of the utility trench. Given the permeable nature of the foundation soils, the groundwater elevations are anticipated to vary with the water surface elevation in the slough. Excavations are anticipated to extend below the low water slough elevation and will require dewatering to localize lower groundwater below the excavation subgrade elevation. Groundwater should be lowered to at least 2 feet below the bottom of any excavation.

6.3.2 Dewatering

Groundwater control may be required for the site excavations, depending on groundwater conditions encountered during construction, or to address seepage zones and/or locally perched groundwater. The extent of dewatering required and the dewatering system used will depend on the excavation support system selected and the Contractor's construction methods. The design, installation, and maintenance of all necessary systems for groundwater control and dewatering during construction are the responsibility of the Contractor, including testing of groundwater and permitting for proper disposal. Large scale dewatering outside excavation areas should not be permitted. Existing structures that may be sensitive to settlement induced by dewatering should be monitored during construction.

6.3.3 Pipe Culvert Backfill

Pipe backfill material should be placed under, on the sides, and a minimum of 1 foot above any pipes. The requirements of the pipe backfill material should be provided by the pipe supplier to include uniform and adequate support for the applied loads. The pipe backfill should consist of fill that is well-graded, free-draining, angular, and durable material. Pipe backfill material should be placed in loose horizontal lifts less than 8 inches and compacted to at least 90 percent relative compaction per ASTM D1557. Alternatively, the pipes can be backfilled with flowable controlled low strength material that is placed around the pipe. Alternatively, the pipes can be backfilled with flowable controlled low strength material that is placed around the pipe. Care must be taken to place the controlled low strength material in controlled sections and not allow it to flow down a significant length of the pipeline to provide full haunch support.

6.3.4 Trench Backfill

Trench backfill material will be placed above the pipe backfill material. Trench backfill material should be placed in 8-inch loose lifts, moisture conditioned to 0 to +3 percent above optimum moisture content and compacted to at least 90 percent of relative compaction per ASTM D1557. The upper 3 feet of trench backfill should be compacted to at least 95 percent of relative compaction per ASTM D1557.

6.4 Shallow Foundations

6.4.1 Allowable Bearing Capacity

The proposed Electrical/Control/Equipment Building can be supported by conventional spread and continuous wall foundations founded on engineered fill. Continuous perimeter foundation can be designed with an allowable bearing pressure of 2,000 pounds per square foot (psf) provided site preparation is prepared as recommended in Section 6.2.

A one-third increase may be applied to this value when considering the effects of transient loads such as wind or seismic. The recommended net allowable bearing pressure includes a safety factor of at least 3 for shear failure of the foundation soils.

Where footings are located near and parallel to existing or new underground utilities, the footings should extend below a plane projected at a slope of 1H:1V upward from the utility to avoid surcharging the underground utility. Where footings are located adjacent to below-grade structures or near major underground utilities, the footings should extend below a 2H:1V plane projected upward from the structure footing or bottom of the underground utility to avoid surcharging the below grade structure and underground utility.

6.4.2 Lateral Resistance

Resistance to lateral loads are developed from the frictional resistance of the footing bottom with the subgrade and from passive resistance against the buried portions of walls and footings. The buildings slab-on-grade should not be considered for providing lateral resistance since it will be structurally separated from the building footing foundations. A coefficient of friction of 0.4 may be used for the friction between the footing bottoms and the underlying soil. This coefficient of friction can be increased to 0.55 when evaluating the earthquake loading condition. These friction coefficients can be mobilized with lateral deformations of less than ¼-inch.

An allowable passive pressure equal to an equivalent fluid pressure of 350 pounds per cubic foot (pcf) can be used for lateral resistance against the buried portions of walls and footing foundations. This passive resistance value has a factor of safety of over 2. However, relatively large deformations would be required to mobilize the ultimate passive resistance. In order to limit deformations to generally accepted magnitudes of about ½-inch, we recommend that this passive resistance be considered an ultimate value. The upper 1 foot of soil should be ignored for passive resistance unless it is confined by a pavement or slab.

A HDR geotechnical representative should observe the foundation excavations prior to placing reinforcing steel and concrete, to check that they are founded on appropriate material. All foundation excavations should be cleaned of loose material and should be free of water prior to steel and concrete placement.

6.5 Mat Foundations

Mat foundations may be designed using a preliminary modulus of subgrade reaction (K) of 30kips per square foot per inch of deflection (for a 1 square-foot bearing plate). The modulus should be adjusted for the actual slab size using appropriate formulas or software.

6.6 Slab-on-Grade

The backfill against the slab should be compacted and graded to shed away from the slab perimeter using non-expansive soil. The slab should be found on minimum 18 inches compacted fill consisting of Aggregate Subbase (AS). If moisture migration through the slabs is undesirable, a moisture barrier and capillary break may be provided between the slab and the compacted layer of engineered fill. We recommend that the system consist of 4 inches of free-draining gravel, such as 3/4-inch, clean, crushed, uniformly graded gravel with less than 3 percent passing No. 200 sieve, or equivalent, overlain by a relatively impermeable vapor retarder that is placed between the subgrade soil and the slab. The vapor retarder should be at least 10-mil thick and should conform to the requirements for ASTM E 1745 Class A, B, or C Underslab Vapor Retarders.

Minor cracking and maintenance problems may be anticipated for slab-on-grade construction; however, separation from structure with concentrated loads and keeping the subgrade moist during construction can greatly reduce the amount of eventual cracking. Prior to concrete placement, any dry cracked subgrade should be sealed with appropriate watering or should be excavated and re-compacted. These conditions and construction procedures should be observed by the Project Geotechnical Engineer.

6.7 Deep Foundations

6.7.1 Axial Capacity

As discussed above, recommended foundation types include driven piles and Cast-in-Drilled Hole (CIDH) piles. In addition, the structure coup be supported on auger cast displacement piles (ADCP) if coupled with ground improvement to provide additional lateral support as the pile reinforcement is inserted after the pile concrete has been placed. Supplemental geotechnical recommendations for ADCP can be provided if this pile type is being considered. Figures 15 and 16 are plots of ultimate axial capacities for 24-inch diameter open ended pipe piles and 30-inch CIDH piles, respectively. Plots of axial capacity pipe piles where and internal plug does or does not form within the pile. For the condition where there an internal plug does form, it is assumed that the plug only forms below Elevation 90 feet, where the pile is driven into the underlying older alluvium. Due to the potential for liquefaction induced settlement downdrag forces should be included assuming the soil above Elevation 100 ft induces downdrag forces. Recommendation for soil properties to analyses pile lateral capacity is provided in Table 6-1. A specialty Contractor should be retained to construct the piles.

6.7.2 Group Effects

The recommended axial capacity applies to single, isolated piles with center-to-center spacings of at least three effective pile diameters. To avoid group effects, all piles should be spaced at a minimum center-to-center spacing of 2.5 times the pile diameter. Pile interaction factors for pile groups are provided in Table 5. Input parameters for Lpile Analyses for the Fish Passage Structure and the Weir are provided in Table 6 and Table 7.

Table 5. P-Multipliers[1] for Fish Passage Structure Lateral Pile Design using LPILE

Center-to-Center Pile Spacing (Number of Pile Widths, B)	Pile Row 1	Pile Row 2	Pile Row 3 or higher
3	0.75	0.55	0.4
4	0.875	0.7	0.55
5	1	0.85	0.7
6	1	0.93	0.8
7	1	1	0.9
8	1	1	1

P-multipliers are adapted from Section 10.7.2.4 of the AASHTO LRFD Bridge Design Specifications, Eighth Edition (AASHTO 2017) with California Amendments (Caltrans 2019). The pile cap example at right is taken from this reference.

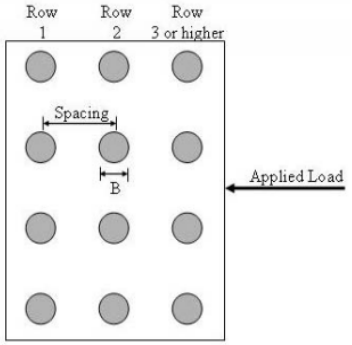


Table 6. L-pile Input Parameters Fish Screen Intake Structure

Layer and USCS Soil Type	Elevation (Feet, NAVD88)	Unit Wight (pcf)		Cohesionless Parameters	Cohesive Parameters	P-Y Curve Model for LPILE/Group
		Total	Effective	Friction Angel (degrees)	Undrain Strength (psf)	
1. SM	131 to 125	120	57.6	45	--	Sand (Reese)
2. CL/SM/ML	125 to 115	115	52.6	30		Soft Clay (Matlock)
3. SM/SC	115 to 95	120	57.6	33		Sand (Reese)
4. Silt	95 to 61	115	52.6	40	--	Stiff Clay (Reese)

Table 7. L-pile Input Parameters Weir and Fish Passage Structures

Layer and USCS Soil Type	Elevation (Feet, NAVD88)	Unit Wight (pcf)		Cohesionless Parameters	Cohesive Parameters	P-Y Curve Model for LPILE/Group
		Total	Effective	Friction Angel (degrees)	Undrain Strength (psf)	
1. ML	135 to 131	120	57.6	32	--	Sand (Reese)
2. SM/ML	125 to 115	115	52.6	30	--	Soft Clay (Matlock)
3. SM/SC/GC	115 to 92	120	57.6	28	--	Sand (Reese)
4. Silt	95 to 61	115	52.6	36	--	Stiff Clay (Reese)

6.7.3 Construction Considerations

All piles should be driven vertically to refusal in order to develop adequate vertical pile capacities. Preliminary refusal criteria should be developed. To limit the potential for damaging the piles, it is the responsibility of the pile-driving Contractor to select a pile-driving hammer appropriate to the soil conditions at the site. HDR recommends that a review of the Contractor-proposed hammer type be conducted. In addition, the method of handling and picking up of the piles should be established by the pile-driving Contractor and should be reviewed by HDR prior to construction.

HDR recommends that the final foundation plans and specifications be reviewed to confirm the recommendations presented in this report have been properly incorporated into the contract documents. To confirm that piles are installed to adequate depths and they have encountered sufficient resistance to develop the required supporting capacities, HDR also recommends that qualified staff observe the driving of all piles at the site on a full-time basis.

6.8 Retaining Structures

The fish passage structure and retaining walls are being designed as restrained walls that are not free to move or rotate. Therefore, at-rest pressures are being assumed for static loading conditions. To limit wall pressure, walls should be backfilled with compacted native granular soils. Based on the available geotechnical information at the overall Project area, onsite soil available for borrow and structure backfill is predominantly sandy sit and silty sand. High plasticity soil should be avoided for use as wall backfill. Refer to Section 6.7.4 for specific requirements for backfill. On this basis the walls may be designed assuming the following soil parameters and earth pressure coefficients appropriate for compacted lean clay backfill:

- Backfill total unit weight of 125 pcf above the phreatic surface for each design case.
- Backfill buoyant unit weight of 62.6 pcf below the phreatic surface.
- At-rest earth pressure coefficient (K_O) of 0.60 (for static loading).
- Active earth pressure coefficient (K_A) of 0.43 (for seismic loading).

Under static loading conditions, the walls are being designed to resist hydrostatic and lateral earth pressure, as well as additional loads caused by surcharging. Under seismic conditions, the active pressure is combined with uniform seismic pressure increment of 6H psf, where H is the height of the backfill. This seismic earth pressure was developed using the design peak ground acceleration of 0.28g. Earth pressure diagrams for design of the walls are presented Figure 17 and Figure 18 for static and seismic loading conditions, respectively.

Because the walls will be backfilled with granular soils, back drainage may not be required.

6.9 Filter Design

The Project includes constructing a roughened rock channel fill immediately downstream of the weir with strategically placed boulders to allow fish to swim up to the fish ladder (Figure 19). The roughened channel will be comprised of open graded rock infilled with finer coarse-grained material (rock infill material). The rock infill material will be washed into the rock fill as it is placed. Filter and transition layers will be required beneath the rock fill to prevent foundation material from being transported/eroded into the rock fill that could undermine the weir. A sheetpile cutoff will be located directly upstream of the weir, extending into the underlying older alluvium layer that will prevent lateral migration of material into the rock fill. The filter layer and transition material will need to extend across the entire bottom of the channel and up both banks to prevent material loss due to end around seepage flow through the permeable weir abutments/creek bank. These calculations determine gradations of filter material that are compatible with the natural granular stream bed. Riprap will be placed on the slopes as erosion protection and require a filter layer to prevent loss of soil into more riprap material. However, a geotextile with a sufficient apparent opening size (AOS) will be required to retain the filter layer and have sufficient permeability to allow water to flow into the channel.

6.9.1 Base Soil and Rock Infill Gradation

Gradation for the base soil (native riverbed soil) was determined using the available grain size distribution test data for the material near EL 100 feet from the existing and supplemental explorations. The base soil gradation bounds were selected based on the five grain size distribution test results shown in Table 8. The gradation for the rock infill material is provided in Table 9.

Table 8. Base Soil Gradation Limits:

Particle size (mm)	Sieve #	Percent Passing					Base Soil Gradation Limits	
		BI-1 6-SS	BI-2 2-SS	BI-3 4-SS	BD-2 3-SS	HDR-B2 S-10	Upper Bound	Lower Bound
37.5	--	--	--	--	--	100	100	100
25	--	100	--	--	100	95	90	100
19.05	--	66	--	100	78	86	--	--
12.5	--	58	--	88	69	71	--	--
9.5	--	50	100	83	66	62	--	--

Particle size (mm)	Sieve #	Percent Passing					Base Soil Gradation Limits	
		BI-1 6-SS	BI-2 2-SS	BI-3 4-SS	BD-2 3-SS	HDR-B2 S-10	Upper Bound	Lower Bound
4.75	4	38	97	67	55	46	40	97
2.36	8	--	--	--	--	34	--	--
2	10	28	95	47	47	--	30	95
1.18	16	--	--	--	--	27	--	--
0.6	30	--	--	--	--	20	--	--
0.425	40	16	83	18	37	--	16	85
0.3	50	--	--	--	--	15	--	--
0.15	100	--	--	--	--	11	--	--
0.075	200	8	42	7	30	9	0	45

Table 9. Rock Infill Gradation and Transition Layer Gradation

Particle size (mm)	Sieve #	Rock infill Material (original), % passing	Proposed Transition Layer Gradation (%)
		Upper Bound	Lower Bound
63	--	100.0	100.0
50	--	80.0	80.0
25.0	--	65.0	65.0
19.0	--	50.0	50.0
4.75	4	35.0	16
2.36	8	--	10
1.00	18	--	5
0.425	40	16.0	--
0.075	200	7.0	3
0.053	270	--	--
0.0001	--	0.0	0.0

6.9.2 Methodology and Criteria

Filter and transition layer gradation specifications were determined based on the range of particle sizes of the native riverbed soil (Table 1) using criteria by the USACE (1994) and the U.S. Bureau of Reclamation (Reclamation 2011). A spreadsheet tool developed by the Virginia Polytechnical Institute and State University (VPI 2004) was used to the evaluate the compatibility of candidate filter and drain material gradations relative to the respective base soils/materials (i.e., filter material with respect to the backfill soils and drain materials with respect to the filter material).

The need for a transition zone between the filter layer and the Caltrans Class III and Class V riprap selected for the Project. The Cistern-Ziems Chart from NCHRP Report 568 (NCHRP 2006) was used to calculate the transition layer gradation based on the ration of the D_{50} ratio of the riprap to transition layer material. The report includes a flow chart for selection of geotechnical filter fabric that is based on the 0_{95} value which is the

aperture size where 95 percent of the openings are smaller and the size is equivalent to the AOS of the geotextile filter. The AOS is based on the filter layer gradation which is a blend of coarse sand and fine gravel. There are available geotextile fabrics with adequate AOS. Use of filter fabrics along both the Mormon Slough and Old Calaveras River are comprised of overbank materials consisting of fine sands and silts. The use of a geotextile alone would not meet the requirements for a filter.

6.9.3 Recommendations

The calculation results indicate the standard ASTM C 33 sand gradation does not provide an adequate gradation range to act as a filter for the channel bed. A 40 to 60 percent blend of ASTM C33 -02a Size 9 sand and AASHTO M 43-99 Size #57 sand (by weight) provides a gradation that meets filter criteria with the gradation presented in Table 10. The Rock infill material gradation was found to satisfy the transition zone gradation; however, the gradation provided in Table 9 is recommended to provide a lower coefficient of uniformity (Cu = d60/d10) by limiting the percent finer than the #4 sieve to 16 percent and the percent passing the #200 sieve to 3 percent. This will provide more stable gradation for transport and placement. The filter design calculation for Base Soil to Filter and for Rock Infill to Filter is provided in Appendix F.

The Class III and Class V Riprap have a D_{50} of 12.5 and 18 inches respectively per the Caltrans new riprap classes. Using the Cistern-Ziems Chart from NCHRP Report 568 and assuming the riprap and riprap bedding layers have a Cu = 2.4 and 2.0, the A value of the D_{50r} to D_{50b} (riprap:base layer) is 10. The D_{50b} of the Class III and Class V riprap is 1.25 and 1.8 inches, respectively. Therefore, a coarser backing layer is needed. For both Class III and Class V riprap the D_{50} of the backing layer should be a minimum of 8 inches in thickness and a gradation $1.5 < D_{50} < 2$ and the backing layer can replace the upper 8 inches of the transition layer. If a geotextile is used in lieu of a transition/backing layer, a geotextile a fabric with an AOS of 0.2 mm (#70 sieve) would be sufficient permeability for placement over the filter layer. It should be a woven geotextile with sufficient strength to not be damaged by the proposed placement method.

Table 10: Filter Material Gradation:

Particle size (mm)	Sieve size	ASTM C33 - 02a	Table 2: Size # 9	AASHTO M 43-88	Table 1: Size # 57	Blended Mix	
		Percent Passing		Percent Passing		Percent Passing	
		Upper Bound	Lower Bound	Upper Bound	Lower Bound	Upper Bound	Lower Bound
50.0	--	(100.0%)	(100.0%)	(100.0%)	(100.0%)	100%	100%
37.5	--	--	--	100.0%	(100.0%)	--	--
25.0	--	(100.0%)	(100.0%)	95.0%	100.0%	97%	100%
12.5	--	(100.0%)	(100.0%)	25.0%	60.0%	55%	76%
9.5	--	100.0%	--	--	--	--	--
4.75	4	85.0%	100.0%	0.0%	10.0%	34%	46%
2.36	8	10.0%	40.0%	0.0%	5.0%	4%	19%
1.40	14	--	--	(0.0%)	(0.0%)	--	--
1.18	16	0.0%	10.0%	(0.0%)	(0.0%)	0%	4%
0.600	30	--	--	(0.0%)	(0.0%)	--	--
0.300	50	0.0%	5.0%	--	--	--	--
0.075	200	(0.0%)	(0.0%)	(0.0%)	(0.0%)	0%	0%
0.0001	--	0.0%	0.0%	0.0%	0.0%	--	--

6.10 Additional Geotechnical Services

A geotechnical representative should review the geotechnical aspects of the Project plans and specifications to check for conformance with the intent of the recommendations provided in this report. During construction, a geotechnical representative should observe the following:

- Soil conditions exposed by foundation excavations, to check for consistency with those encountered during explorations
- Subgrade preparation prior to placement of engineered fill
- Engineered fill placement and compaction
- Pile installation

7 Limitations

This report has been prepared by the design team for specific application to the Bellota Fish Passage Project in accordance with generally accepted geotechnical engineering practice. No warranty, express or implied, is made. The analyses and recommendations submitted are based on the data available to HDR at the time of this geotechnical investigation and on the data collected during the field investigation. This report does not reflect subsurface soil variations that may occur between the locations of the explorations or variations in groundwater conditions which may occur over a period of time. Variations in conditions may become evident during construction, at which time re-evaluation of the conclusions may be necessary. In the event the Project design changes

after the final report is submitted, the recommendations should be reviewed and possibly modified by HDR's .

Historical explorations and testing were not performed by HDR, and HDR cannot vouch for the accuracy of data and information obtained by others. The historic explorations and subsurface data by others are presented herein to include data relevant to this Project in one document. Data by others should not be relied upon unless the originator of that data is available to confirm its accuracy.

This geotechnical study did not include an investigation regarding the existence, location, or type of possible hazardous materials. If any hazardous materials are encountered during construction of the Project, the proper regulatory officials should be notified immediately.

8 References and Referenced Documents

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Appendix A. Figures

Figure 1. Site Location

Figure 2. Preferred Fish Passage Alternative

Figure 3. Boring Location Plan

Figure 4. Geologic Map

Figure 5. Seismic Hazard

Figure 6. FEMA Flood Map

Figure 7. Geotechnical Profiles – Plan View

Figure 8. Profile Section A-A'

Figure 9. Profile Section B-B'

Figure 10. Profile Figure C-C'

Figure 11. Slope Stability Model - Old Calaveras River

Figure 12. Static Stability Existing Conditions

Figure 13. Static Stability Slope Flattening

Figure 14. Seismic Stability (pseudo-Static K_h=0.15g)

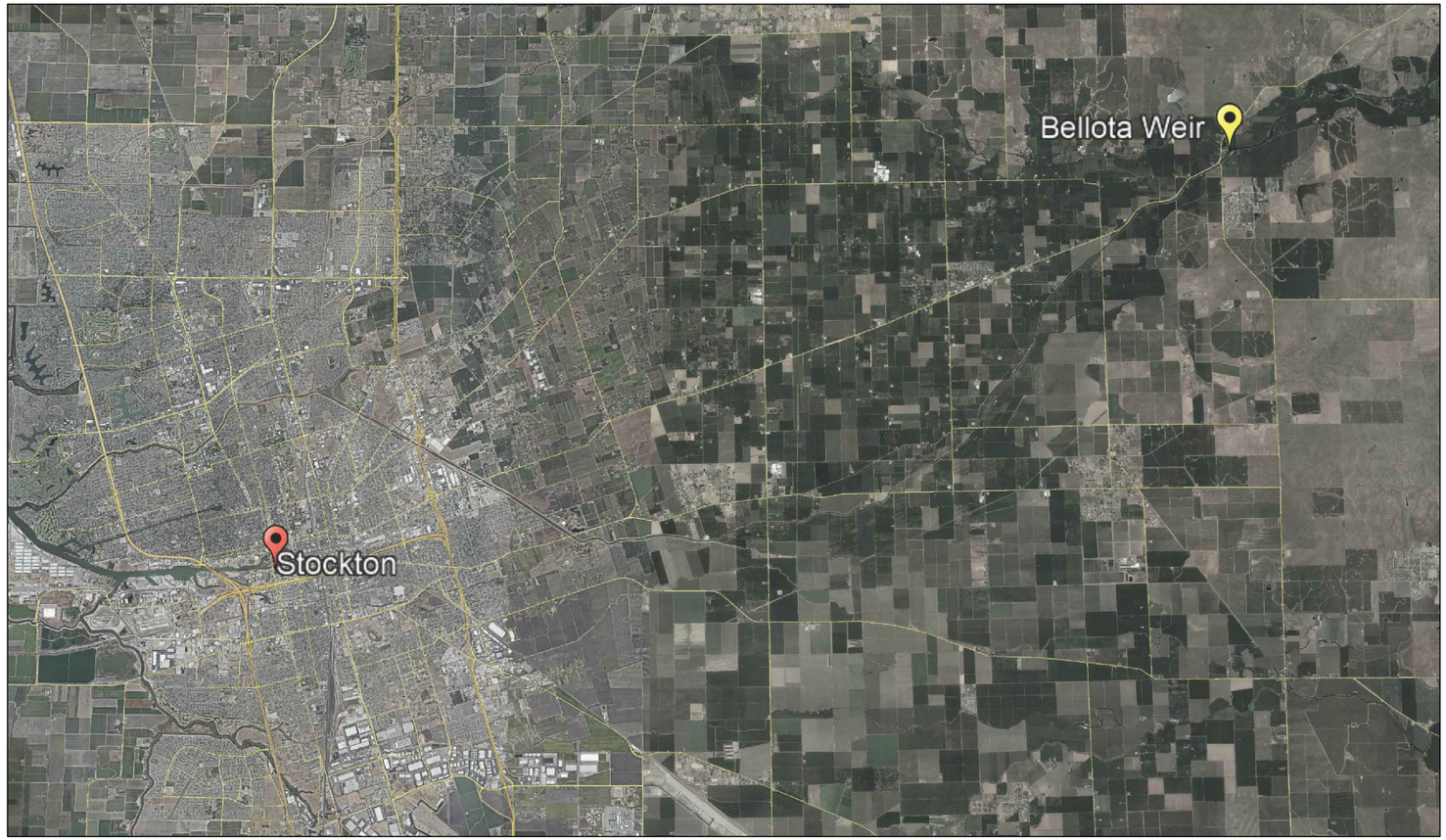
Figure 15. Ultimate Axial Pile Capacity Curve – 24-inch Pipe Pile 0.5-inch Thick

Figure 16. Ultimate Axial Pile Capacity Curve – 36-inch CIDH Pile

Figure 17. Static Lateral Earth Pressures on Structure Walls

Figure 18. Seismic Lateral Earth Pressures on Structure Walls

Figure 19. Roughened Channel Profile





12,000 0 12,000 24,000

SCALE: 1 INCH = 12,000 FEET

NOTE
Image from Google Earth Pro

Site Location

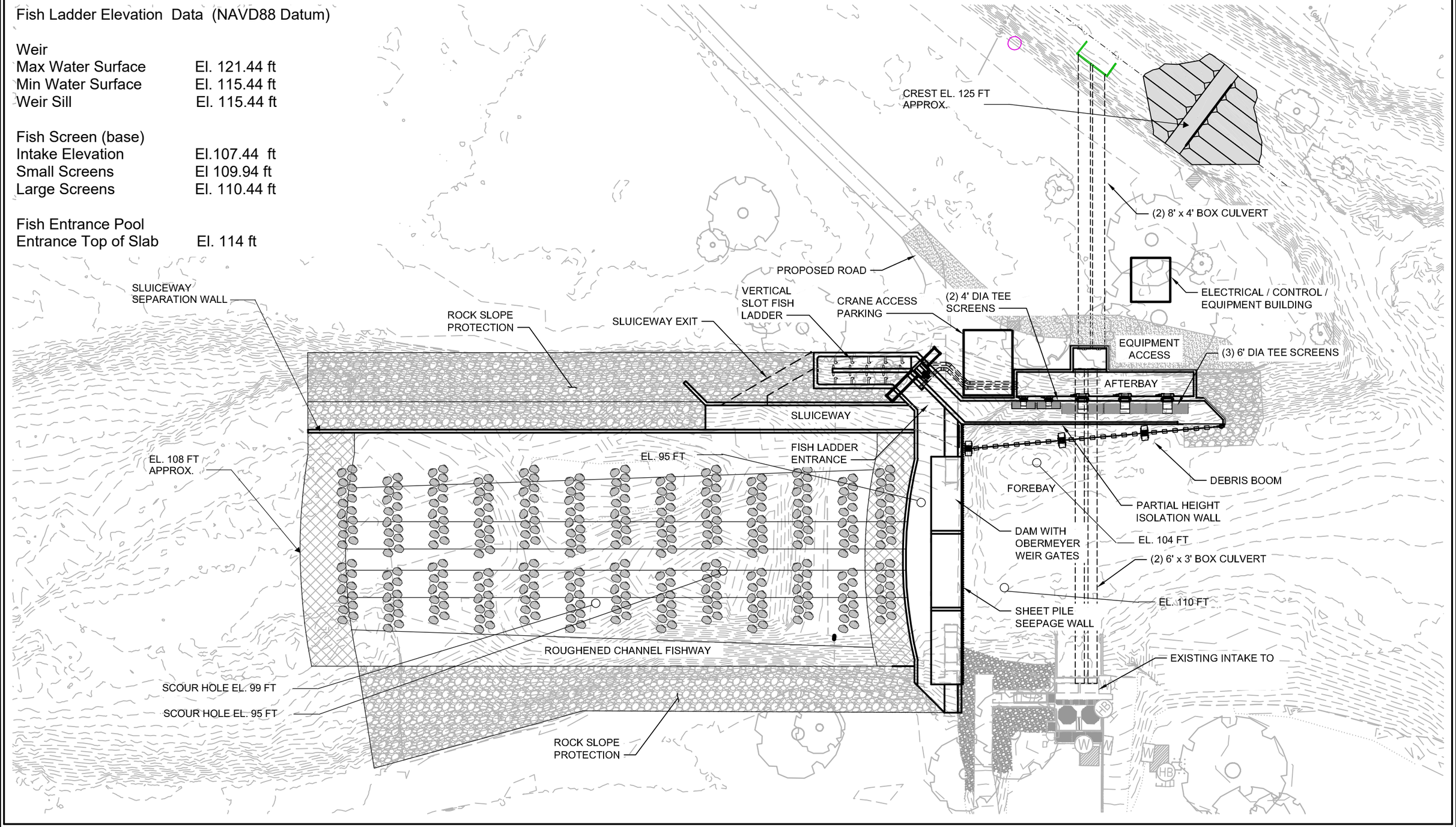
BELLOTA WEIR MODIFICATION PROJECT
SAN JOAQUIN, CALIFORNIA

Date
APRIL 2021

Figure
1

Fish Ladder Elevation Data (NAVD88 Datum)

Weir	
Max Water Surface	El. 121.44 ft
Min Water Surface	El. 115.44 ft
Weir Sill	El. 115.44 ft
Fish Screen (base)	
Intake Elevation	El.107.44 ft
Small Screens	El 109.94 ft
Large Screens	El. 110.44 ft
Fish Entrance Pool	
Entrance Top of Slab	El. 114 ft



N

60 0 60 120

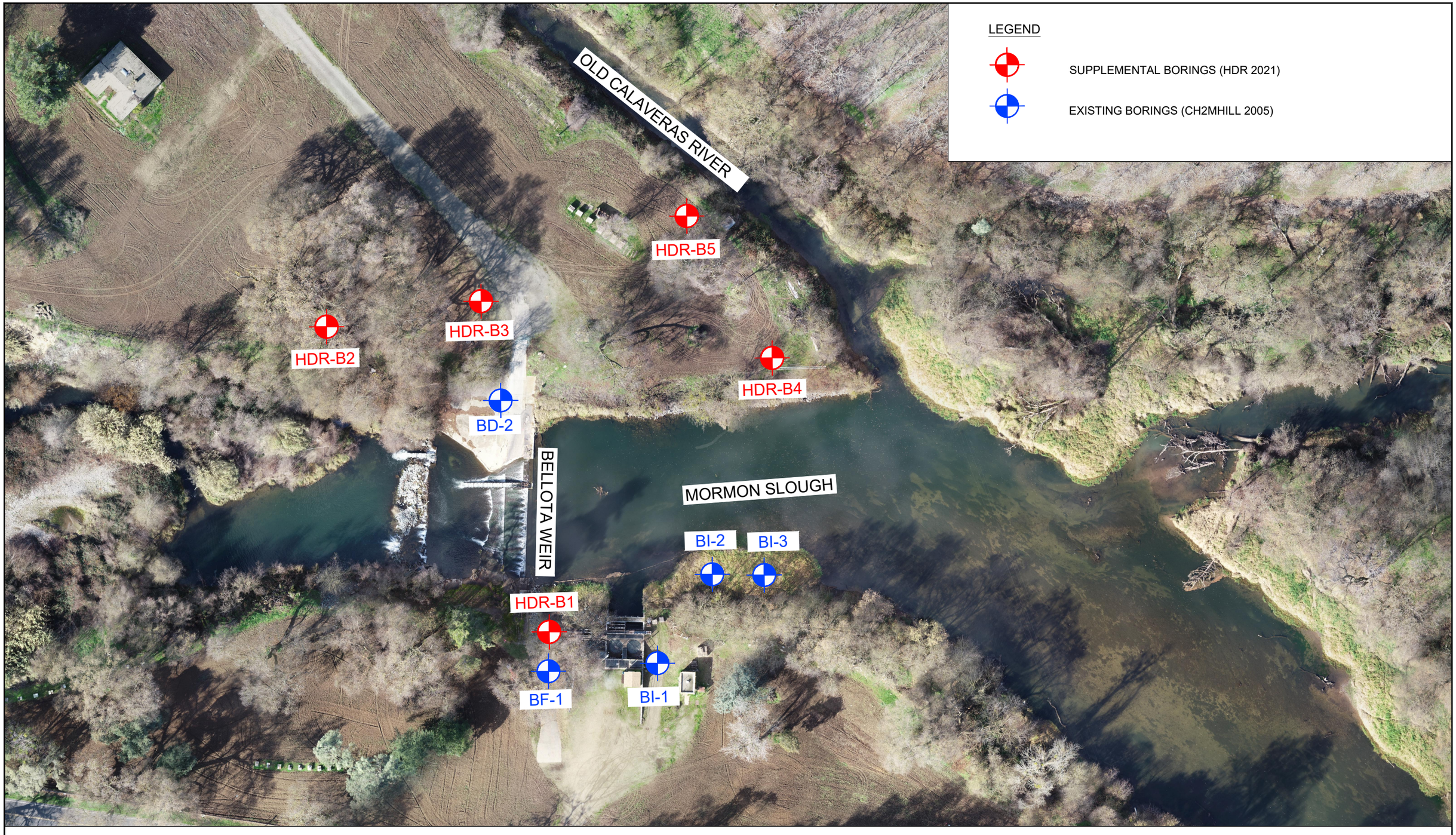
SCALE IN FEET

Preferred Fish Passage Alternative

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APRIL 2021

Figure
2



NOTE
EXPLORATION LOCATIONS SHOWN ARE APPROXIMATE.
EXPLORATION LOCATIONS WERE NOT SURVEYED.

75 0 75 150
SCALE: 1 INCH = 75 FEET

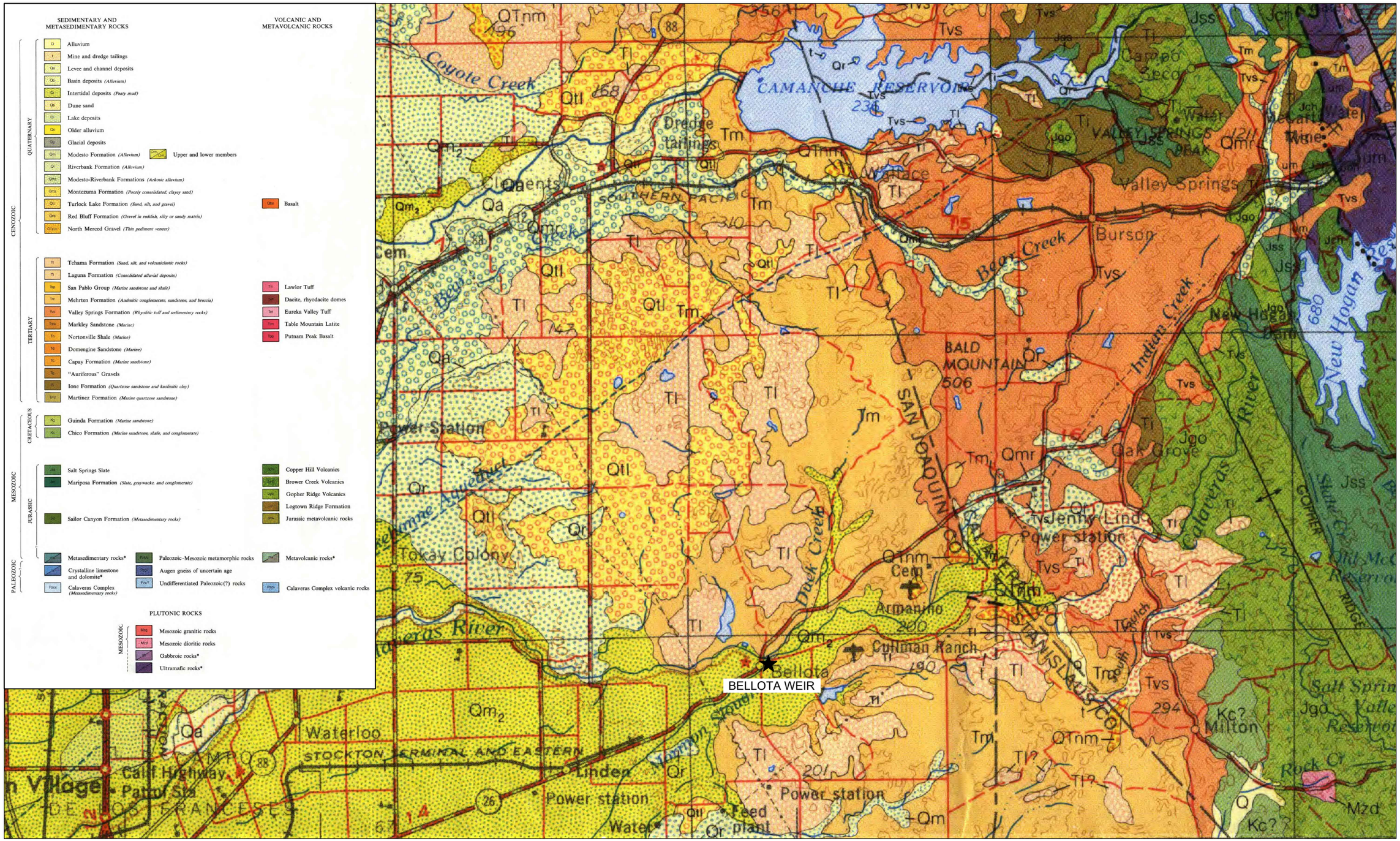
N

BORING LOCATION

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APRIL 2021

Figure
3



NOTE

GEOLOGIC MAP SOURCE: GEOLOGIC MAP OF THE SACRAMENTO QUADRANGLE, CALIFORNIA, COMPILATION BY D.L. WAGNER, C.W. JENNINGS, AND E.J. BORTUGNO (1981)

10,000 0 10,000 20,000

SCALE: 1 INCH = 10,000 FEET

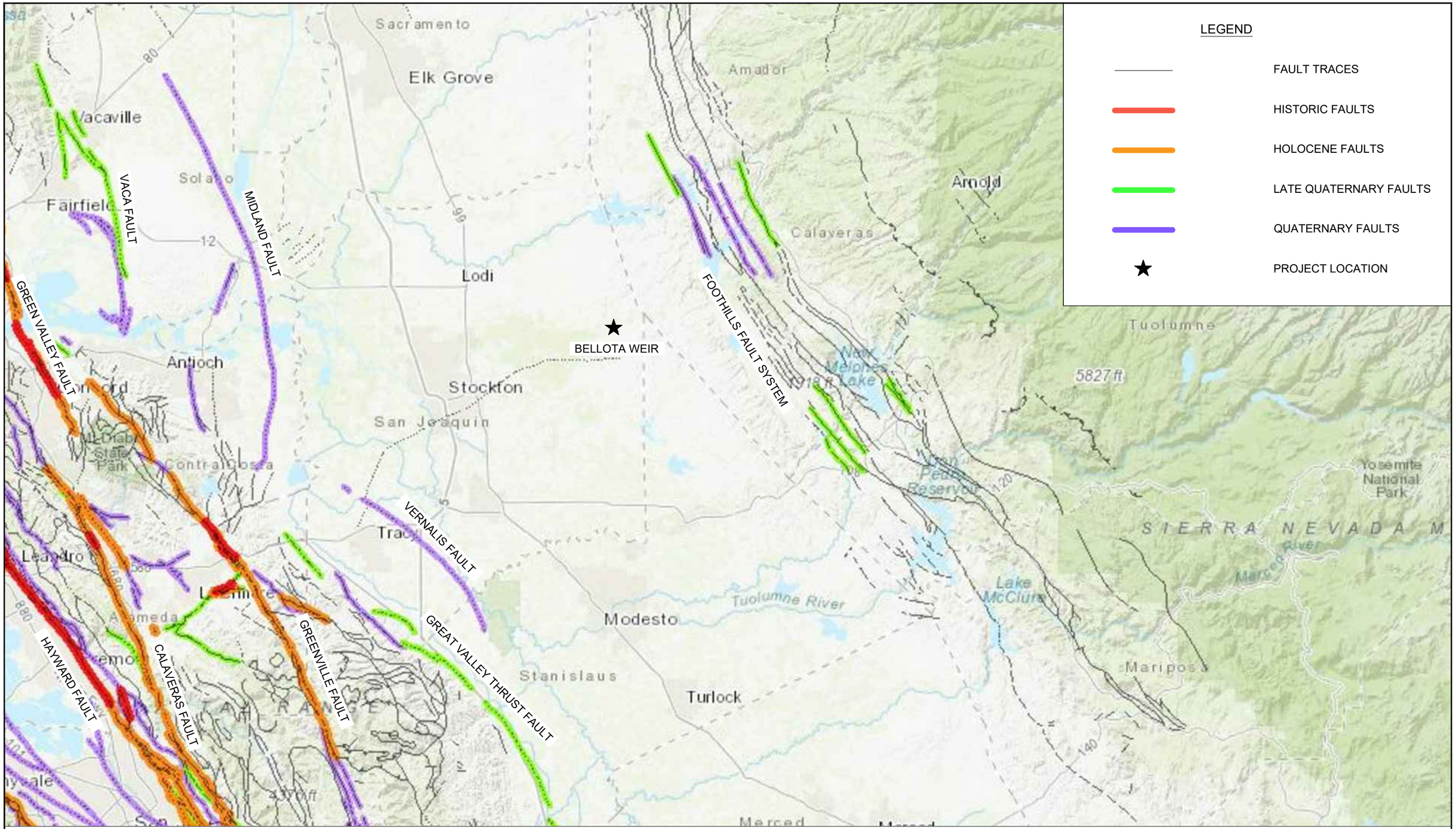
N

GEOLOGIC MAP

BELLOTA WEIR MODIFICATION PROJECT
SAN JOAQUIN, CALIFORNIA

Date
APRIL 2021

Figure
4



NOTE

FAULT TRACES DATA FROM CALIFORNIA GEOLOGICAL SURVEY - FAULT ACTIVITY MAP OF CALIFORNIA.

50,000 0 50,000 100,000

SCALE: 1 INCH = 50,000 FEET

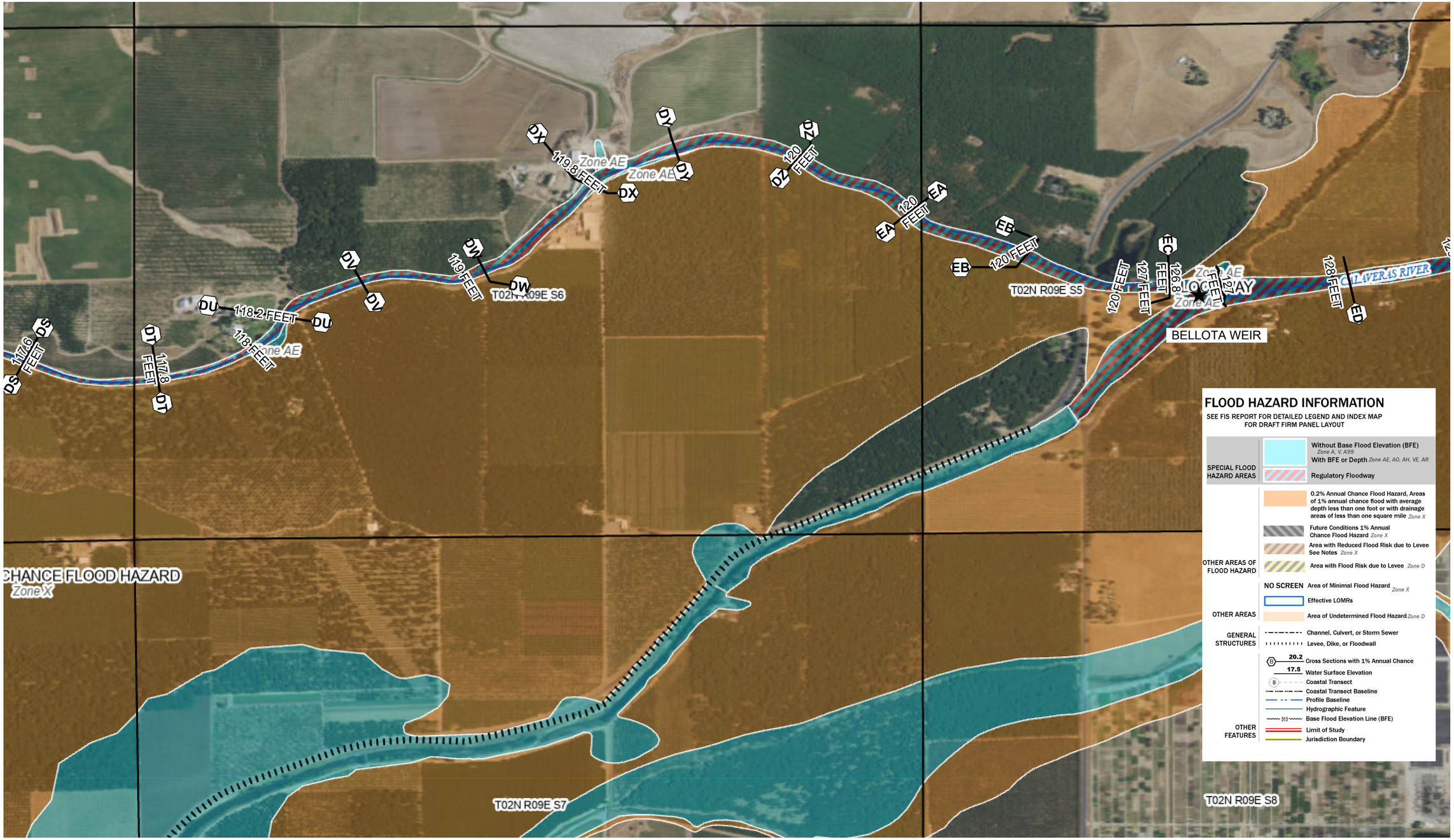
N

SEISMIC HAZARD

BELLOTA WEIR MODIFICATION PROJECT
SAN JOAQUIN, CALIFORNIA

Date
APRIL 2021

Figure
5



NOTE

Flood Map from: FEMA's National Flood Hazard Layer (NFHL) Viewer

1000 0 1000 2000

SCALE: 1 INCH = 1000 FEET

N

FEMA Flood Map

BELLOTA WEIR MODIFICATION PROJECT
SAN JOAQUIN, CALIFORNIA

Date
APRIL 2021

Figure
6

LEGEND

SUPPLEMENTAL BORINGS (HDR 2021)

EXISTING BORINGS (CH2MHILL 2005)

OLD CALAVERAS RIVER

A

B

C

C'

A'

B'

10+00

11+00

12+00

13+00

14+00

15+00

16+00

16+24.07

1+00

2+00

3+00

00

5+00

6+00

7+00

7+24.07

20+00

21+00

22+00

23+00

24+00

24+46.22

HDR-B1

HDR-B2

HDR-B3

HDR-B4

HDR-B5

BD-2

BF-1

BI-1

BI-2

BI-3

BELLOTA WEIR

MORMON SLOUGH

EXISTING WEIR AND FISH LADDER

EXISTING WEIR CENTERLINE

DIVERSION INTAKE STRUCTURE

NOTE

EXPLORATION LOCATIONS SHOWN ARE APPROXIMATE.
EXPLORATION LOCATIONS WERE NOT SURVEYED.

75 0 75 150

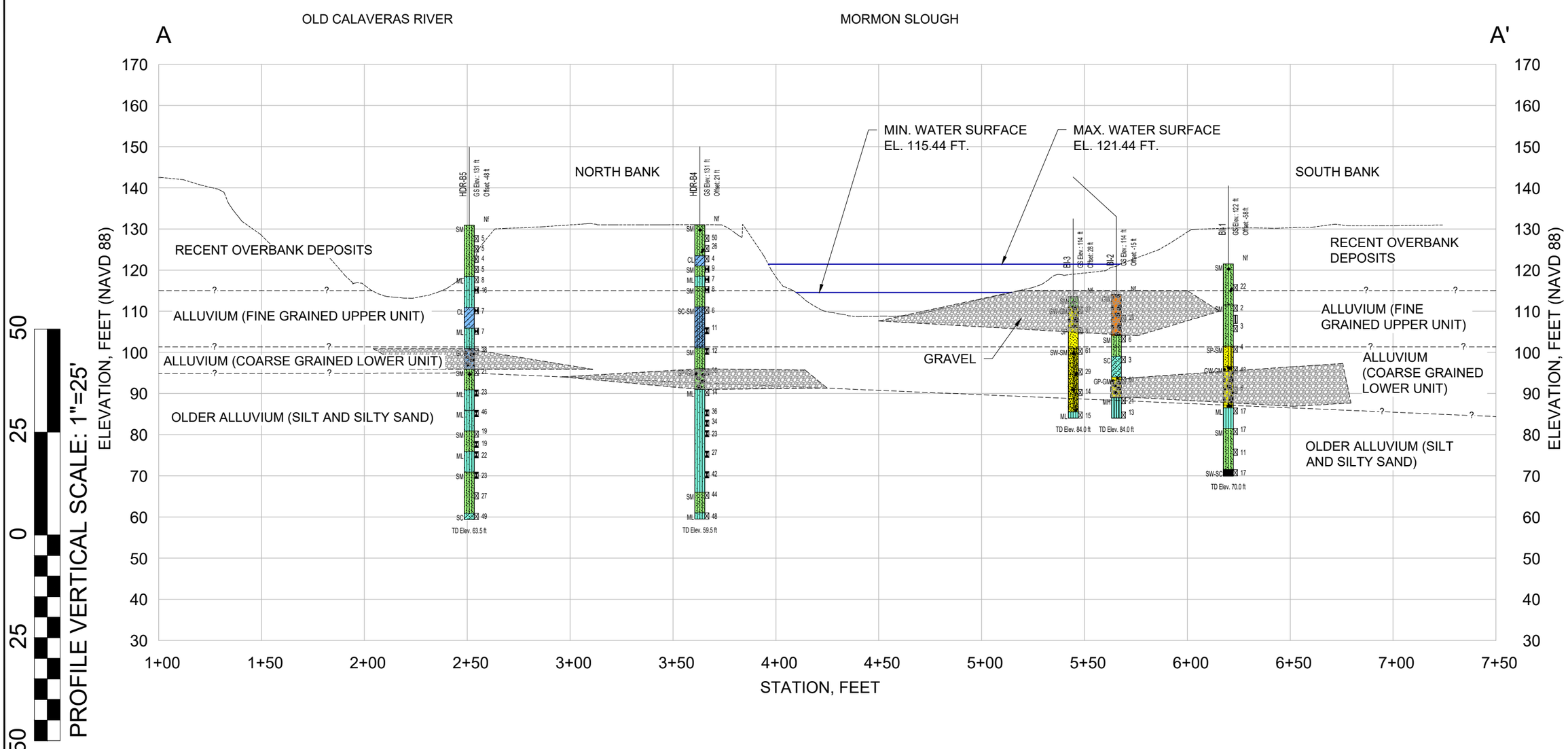
SCALE: 1 INCH = 75 FEET

GEOTECHNICAL PROFILES - PLAN VIEW

BELLOTA WEIR MODIFICATION PROJECT
SAN JOAQUIN, CALIFORNIA

Date
APRIL 2021

Figure
7



NOTE

EXPLORATION LOCATIONS SHOWN ARE APPROXIMATE.
EXPLORATION LOCATIONS WERE NOT SURVEYED.

PROFILE HORIZONTAL SCALE: 1"=50'

100 50 0 50 100

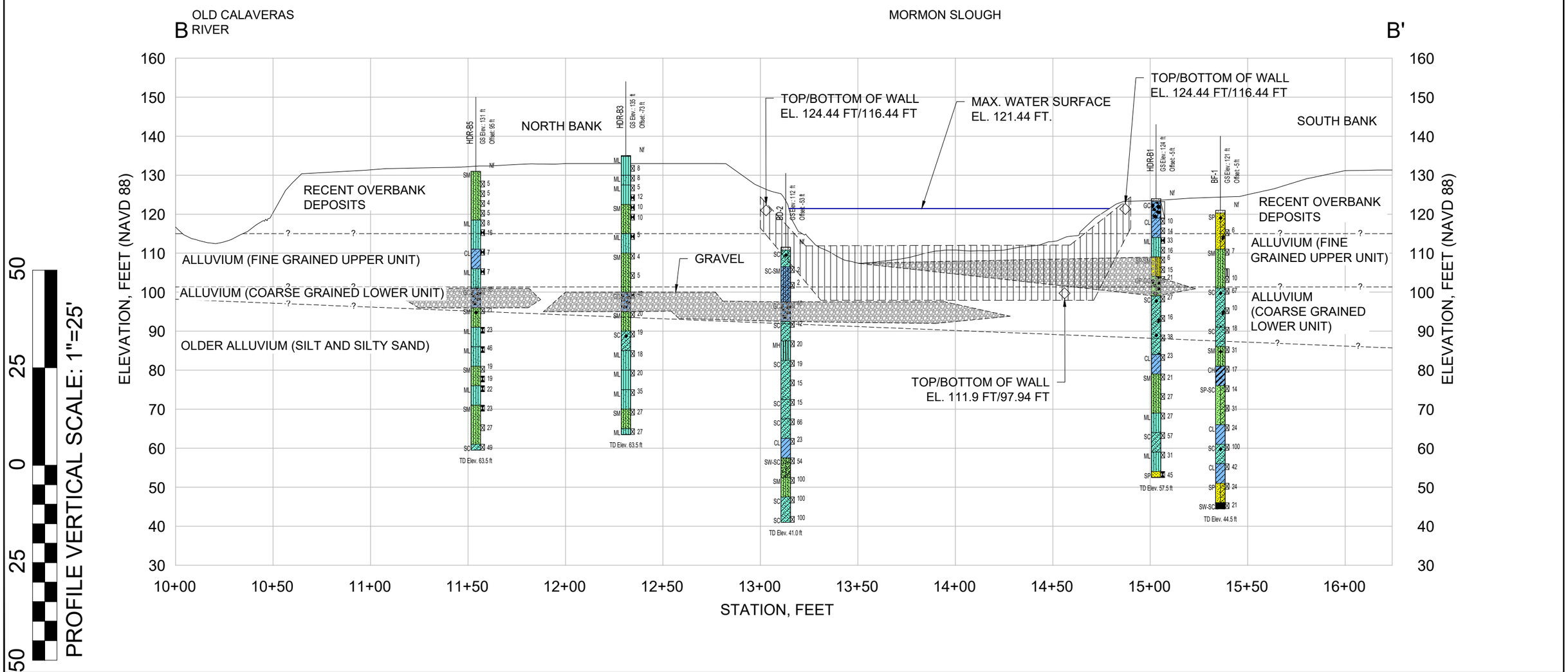
SCALE IN FEET

PROFILE A-A'
Looking Upstream 170 feet upstream of Weir

BELLOTA WEIR MODIFICATION PROJECT
SAN JOAQUIN, CALIFORNIA

Date
APRIL 2021

Figure
8



PROFILE VERTICAL SCALE: 1"=25'

50 25 0 25 50

NOTE

EXPLORATION LOCATIONS SHOWN ARE APPROXIMATE.
EXPLORATION LOCATIONS WERE NOT SURVEYED.

PROFILE HORIZONTAL SCALE: 1"=50'

100 50 0 50

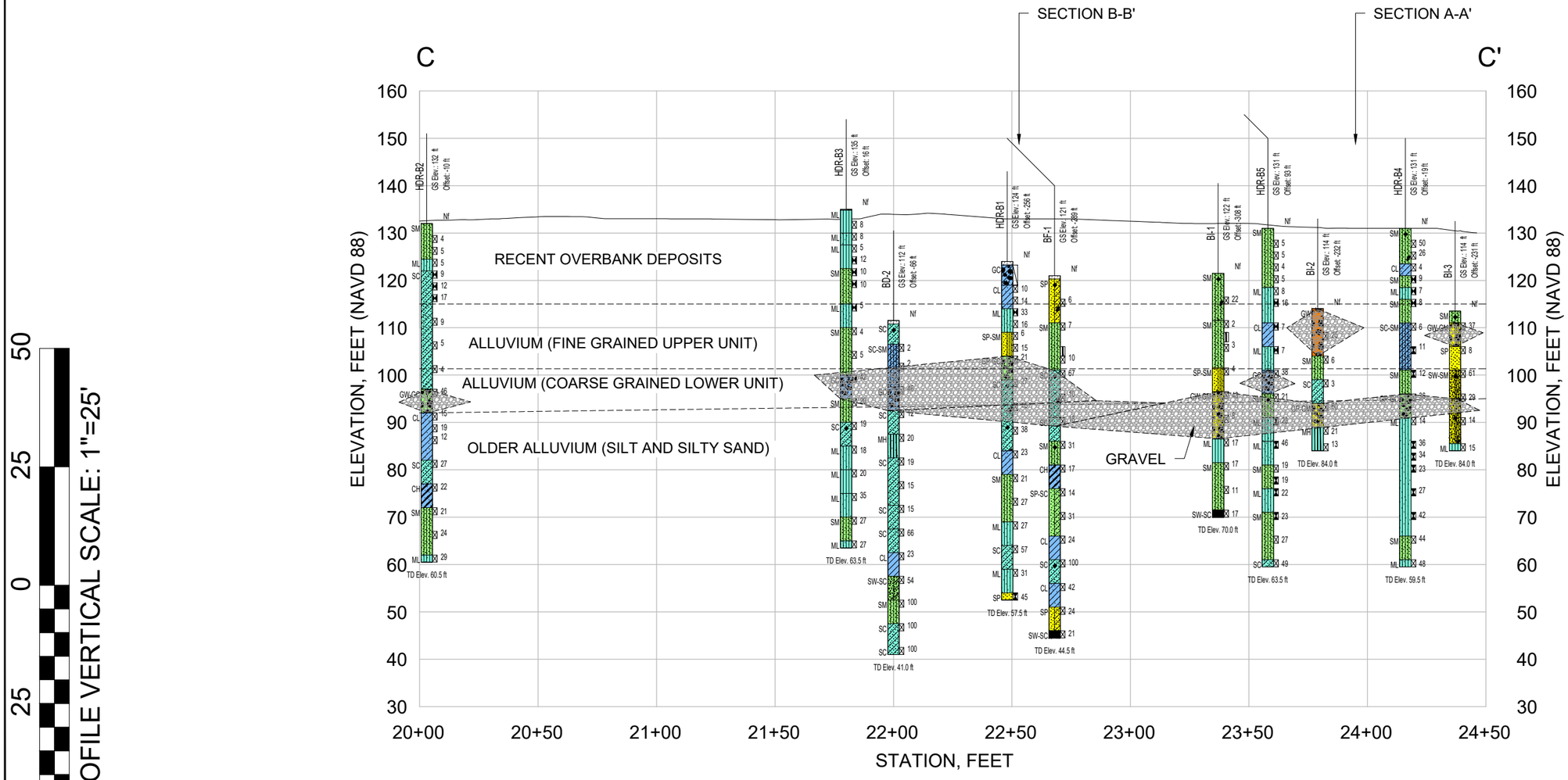
SCALE IN FEET

PROFILE B-B'
Looking upstream 28 Feet from Existing Weir

BELLOTA WEIR MODIFICATION PROJECT
SAN JOAQUIN, CALIFORNIA

Date
APRIL 2021

Figure
9



PROFILE VERTICAL SCALE: 1"=25'

50 25 0 25 50

NOTE

EXPLORATION LOCATIONS SHOWN ARE APPROXIMATE.
EXPLORATION LOCATIONS WERE NOT SURVEYED.

PROFILE HORIZONTAL SCALE: 1"=50'

100 50 0 50

SCALE IN FEET

PROFILE C-C'
NORTH BANK

BELLOTA WEIR MODIFICATION PROJECT
SAN JOAQUIN, CALIFORNIA

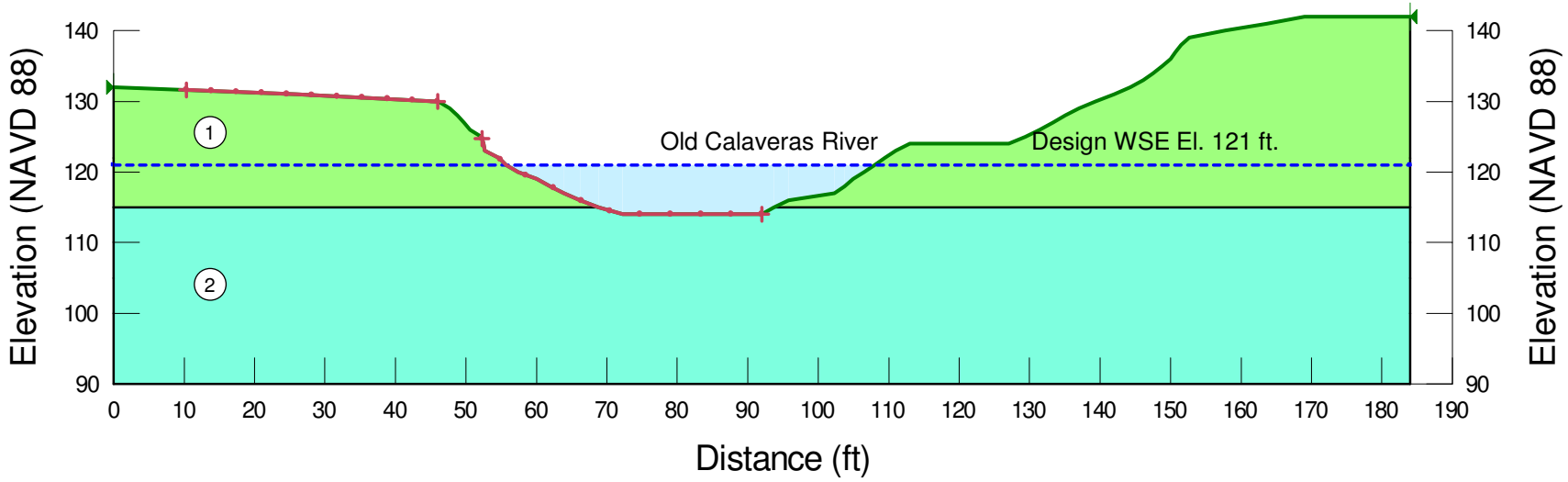
Date
APRIL 2021

Figure
10

Layer	Geology	Material Type	Total Unit Weight (pcf)	Effection friction angle (deg)	Effective Cohesion (psf)
1	Recent Overbank Deposits	SM/ML	115	30	-
2	Alluvium	SC/SM	120	32	-

Project Area Side

North Bank



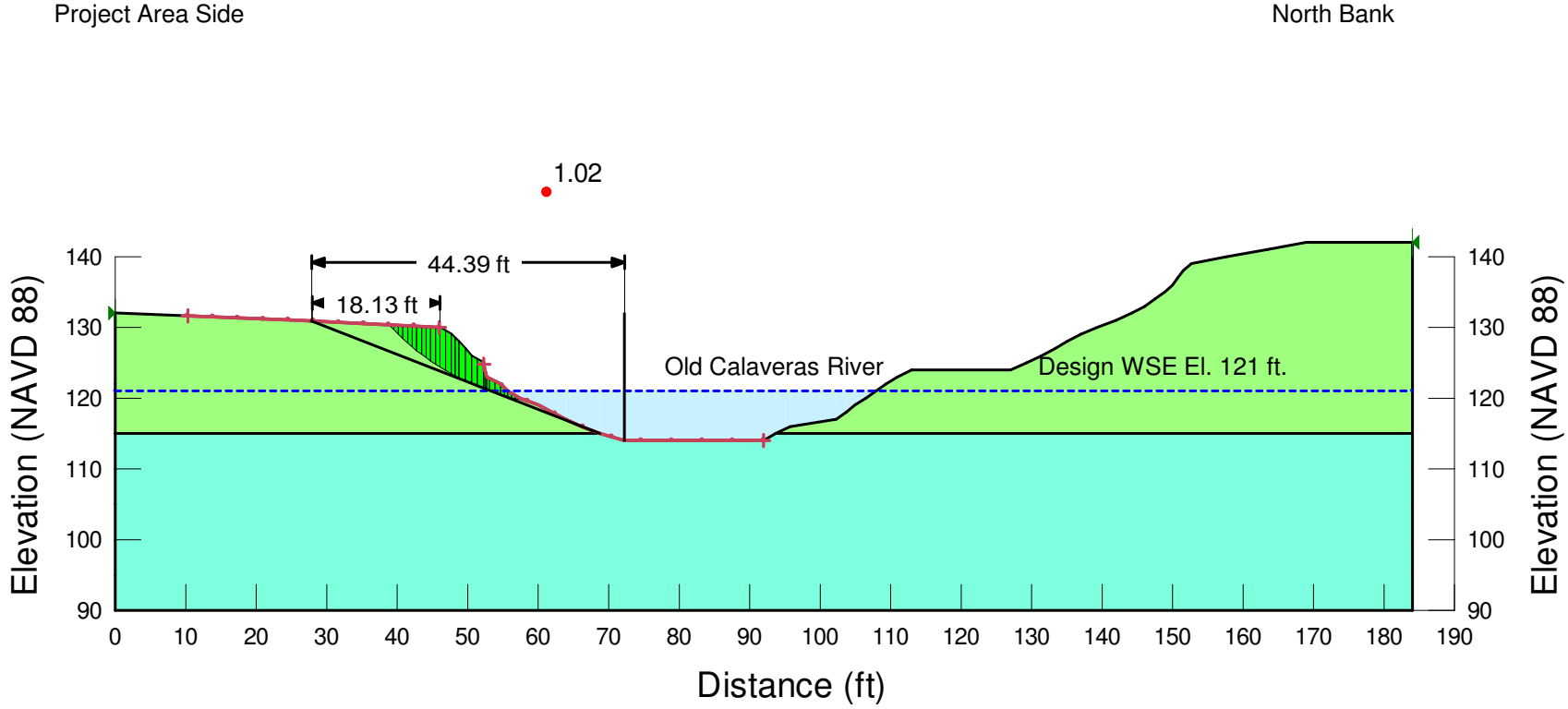
NOTES:

Bellota Weir Modification Project

Slope Stability Model – Old Calaveras River

April 2021

FIGURE 11



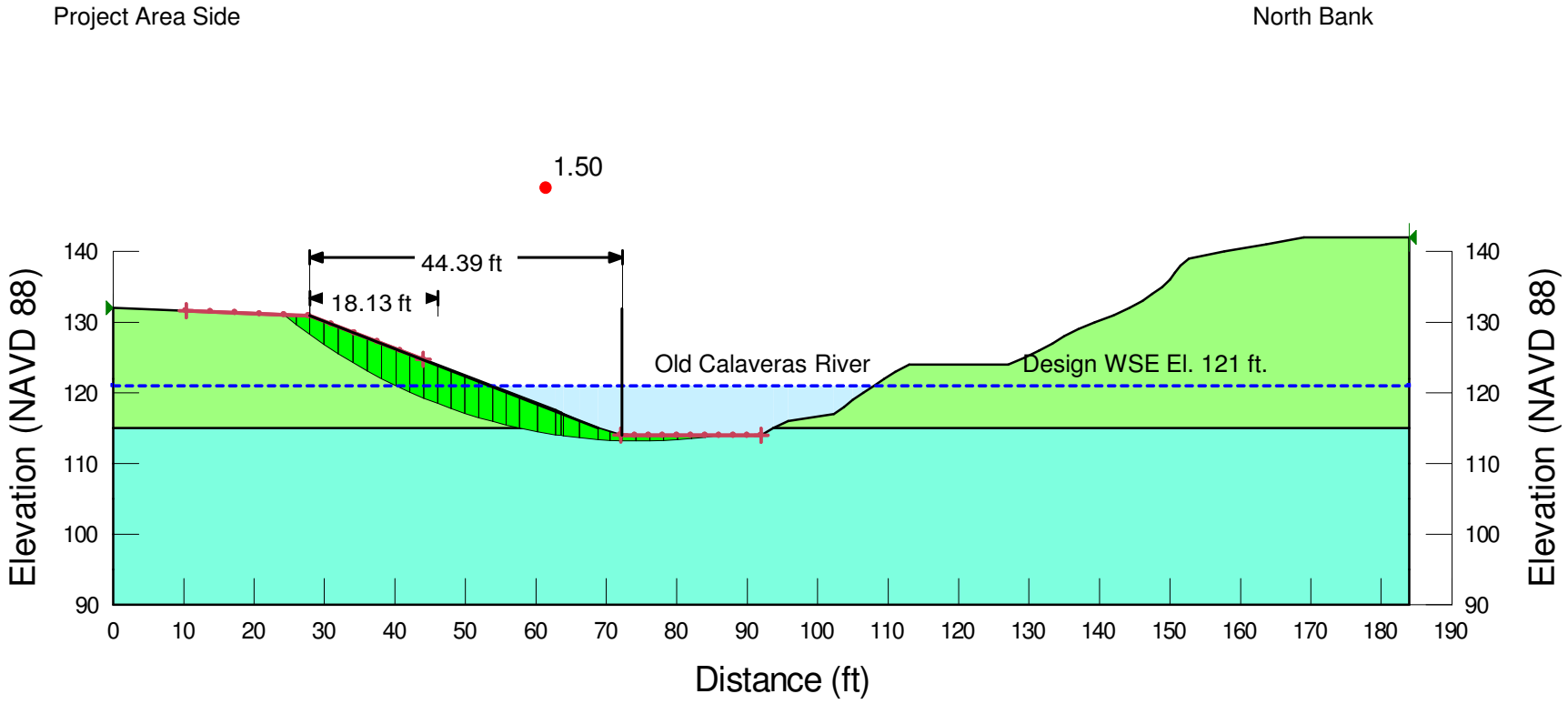
NOTES:

Bellota Weir Modification Project

Static Stability Existing Conditions

April 2021

FIGURE 12



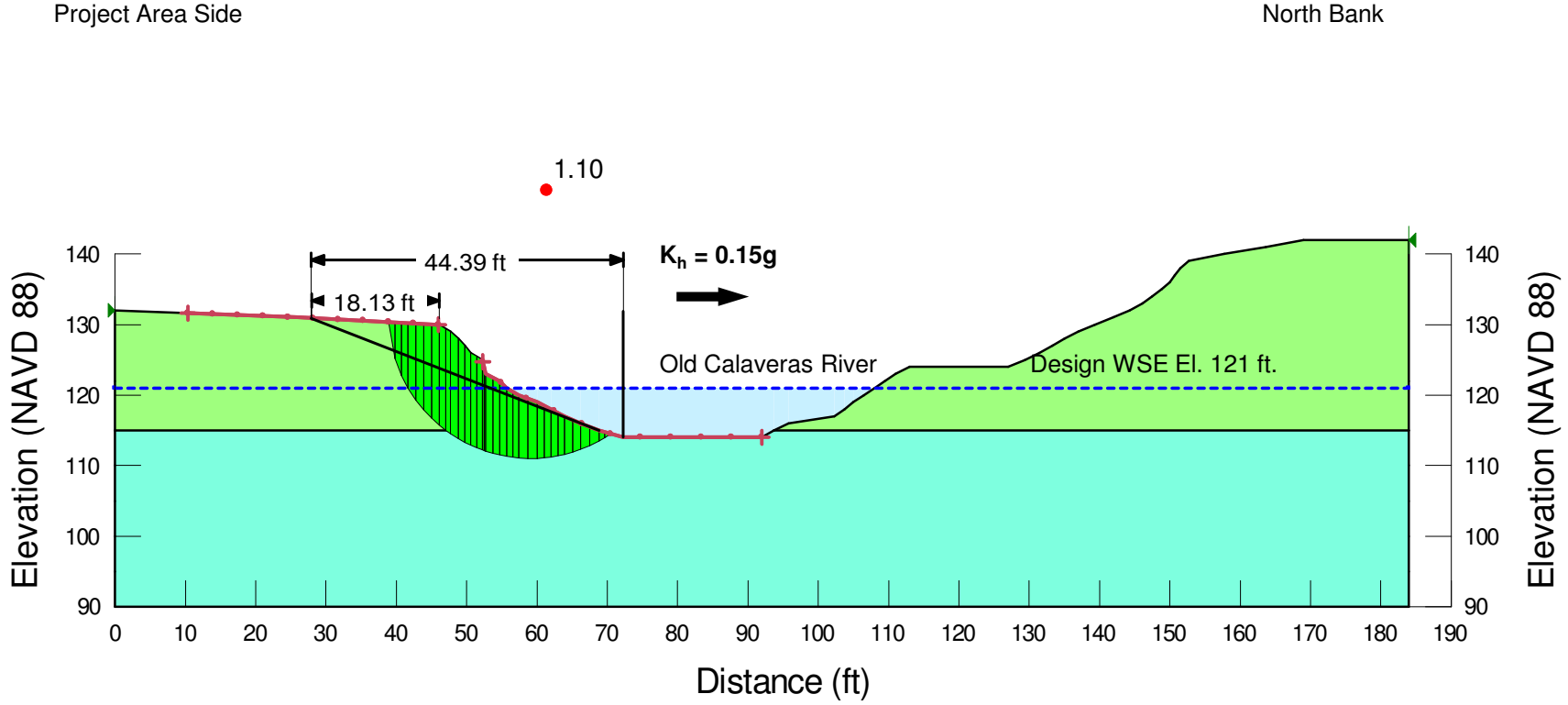
NOTES:

Bellota Weir Modification Project

Static Stability Slope Flattening

April 2021

FIGURE 13



NOTES:

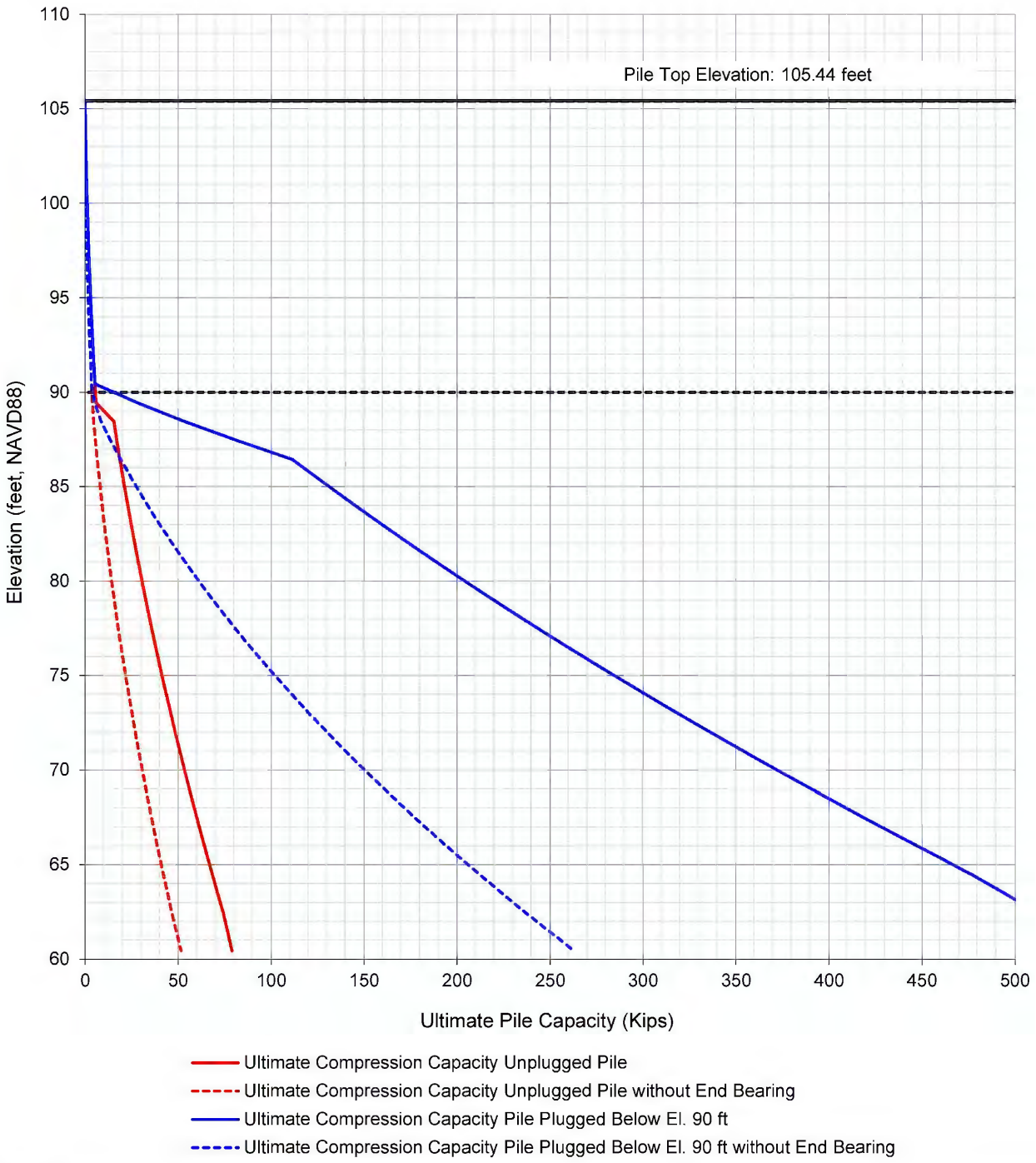
Bellota Weir Modification Project

Seismic Stability (Pseudo-Static K_h=0.15g)

April 2021

FIGURE 14

Bellota Weir Modification Project
24-inch 0.5-inch thick Pipe Pile
Ultimate Axial Capacity



Notes:
1) The allowable pile capacity for service load cases should be taken as the ultimate pile capacity shown divided by the required factor of safety of 2.0.
2) The allowable pile capacity can be increased by one-third for all load combinations that include wind or seismic.

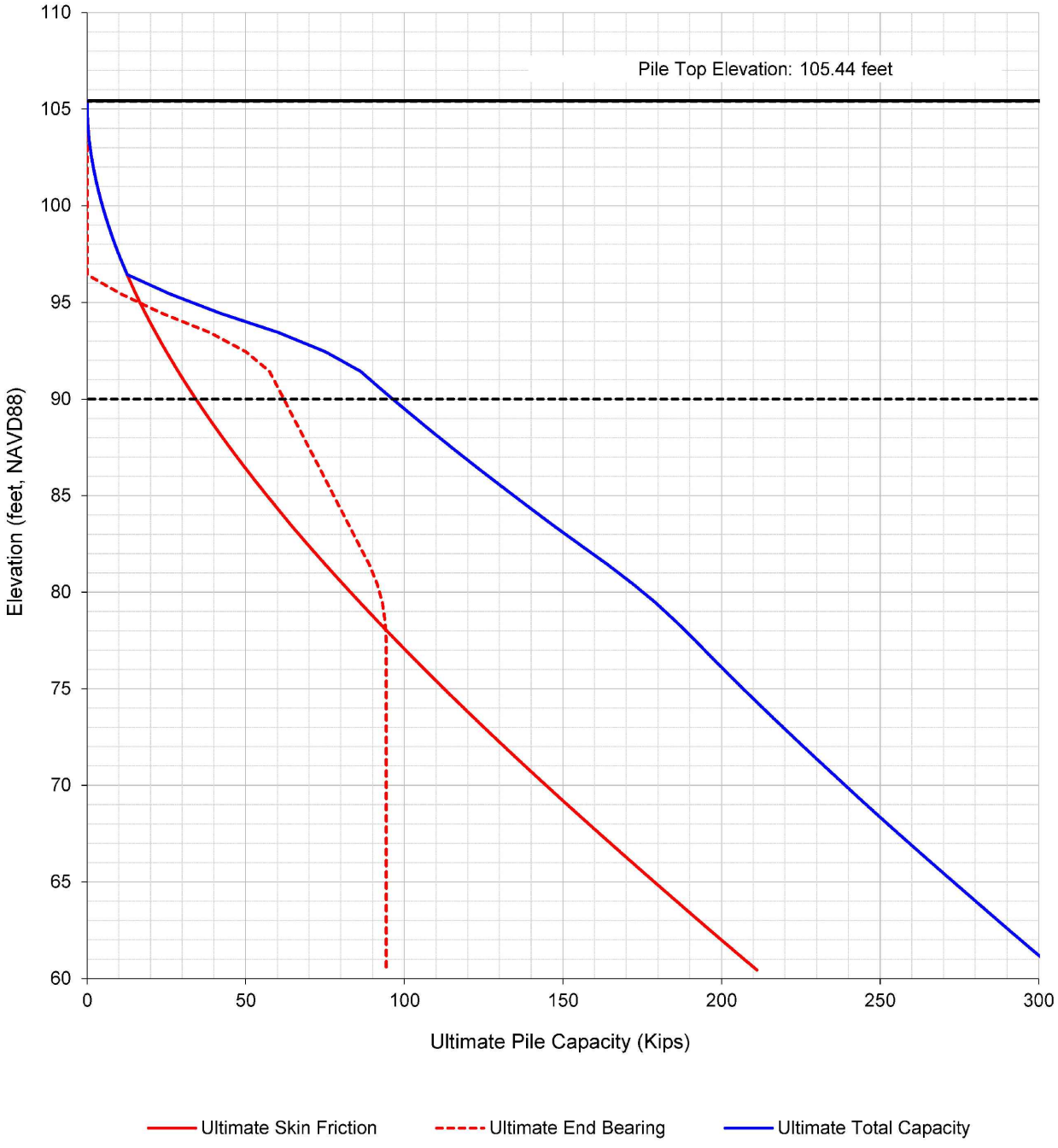
ULTIMATE AXIAL PILE CAPACITY
24 INCH PIPE PILE 0.5 INCH THICK

BELLOTA WEIR MODIFICATION PROJECT
SAN JOAQUIN, CALIFORNIA

Date
APRIL 2021

Figure
15

Bellota Weir Modification Project
36-inch CIDH Pile
Ultimate Axial Capacity



Notes:
1) The allowable pile capacity for service load cases should be taken as the ultimate pile capacity shown divided by the required factor of safety of 2.0.
2) The allowable pile capacity can be increased by one-third for all load combinations that include wind or seismic.

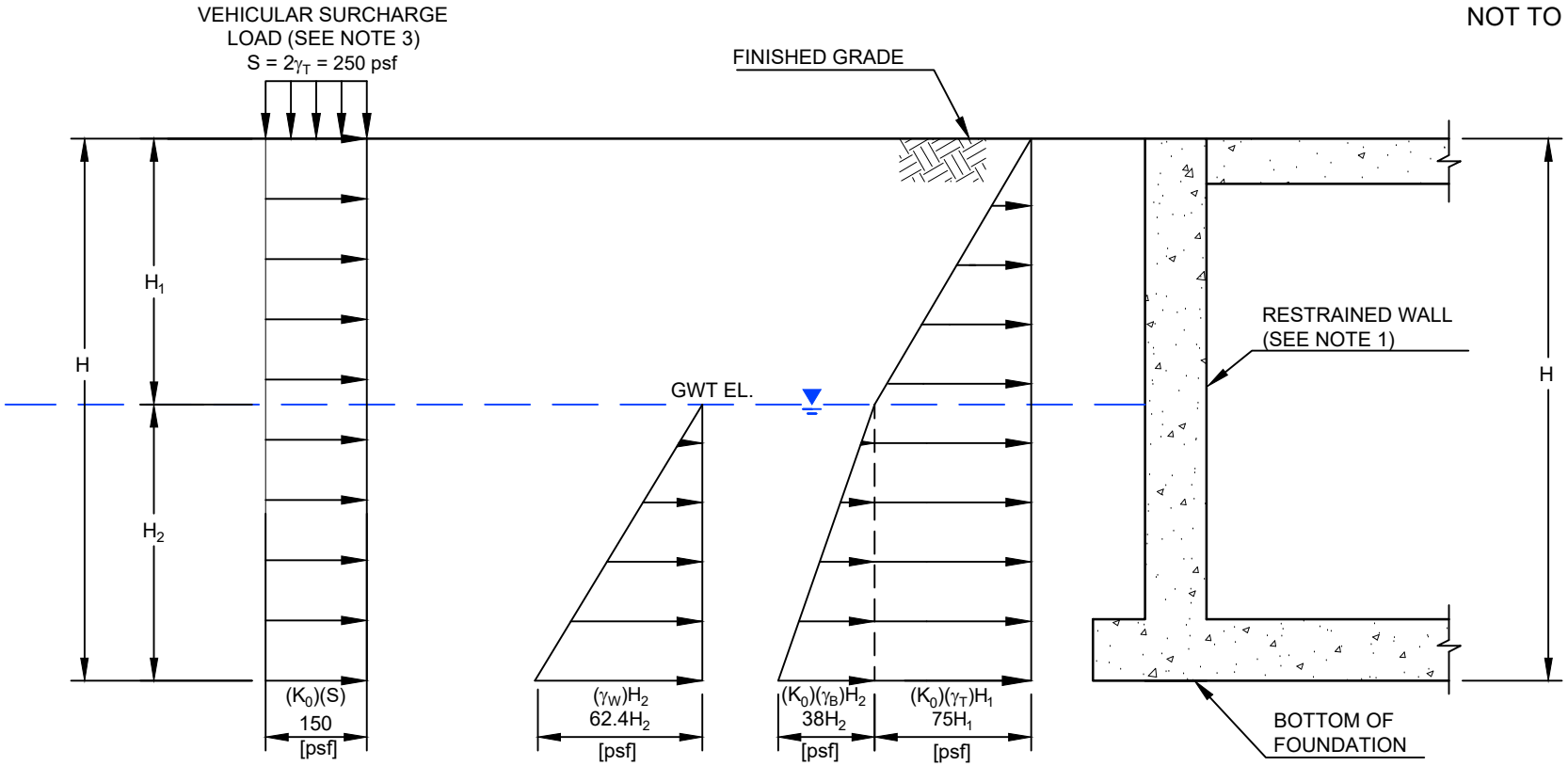
ULTIMATE AXIAL PILE CAPACITY
36 INCH CIDH PILE

BELLOTA WEIR MODIFICATION PROJECT
SAN JOAQUIN, CALIFORNIA

Date
APRIL 2021

Figure
16

NOT TO SCALE



PARAMETER	VALUE
AT-REST EARTH PRESSURE COEFFICIENT (K_0)	0.60
TOTAL UNIT WEIGHT (γ_T)	125 pcf
UNIT WEIGHT OF WATER (γ_W)	62.4 pcf
BUOYANT UNIT WEIGHT ($\gamma_B = \gamma_T - \gamma_W$)	62.6 pcf

NOTES:

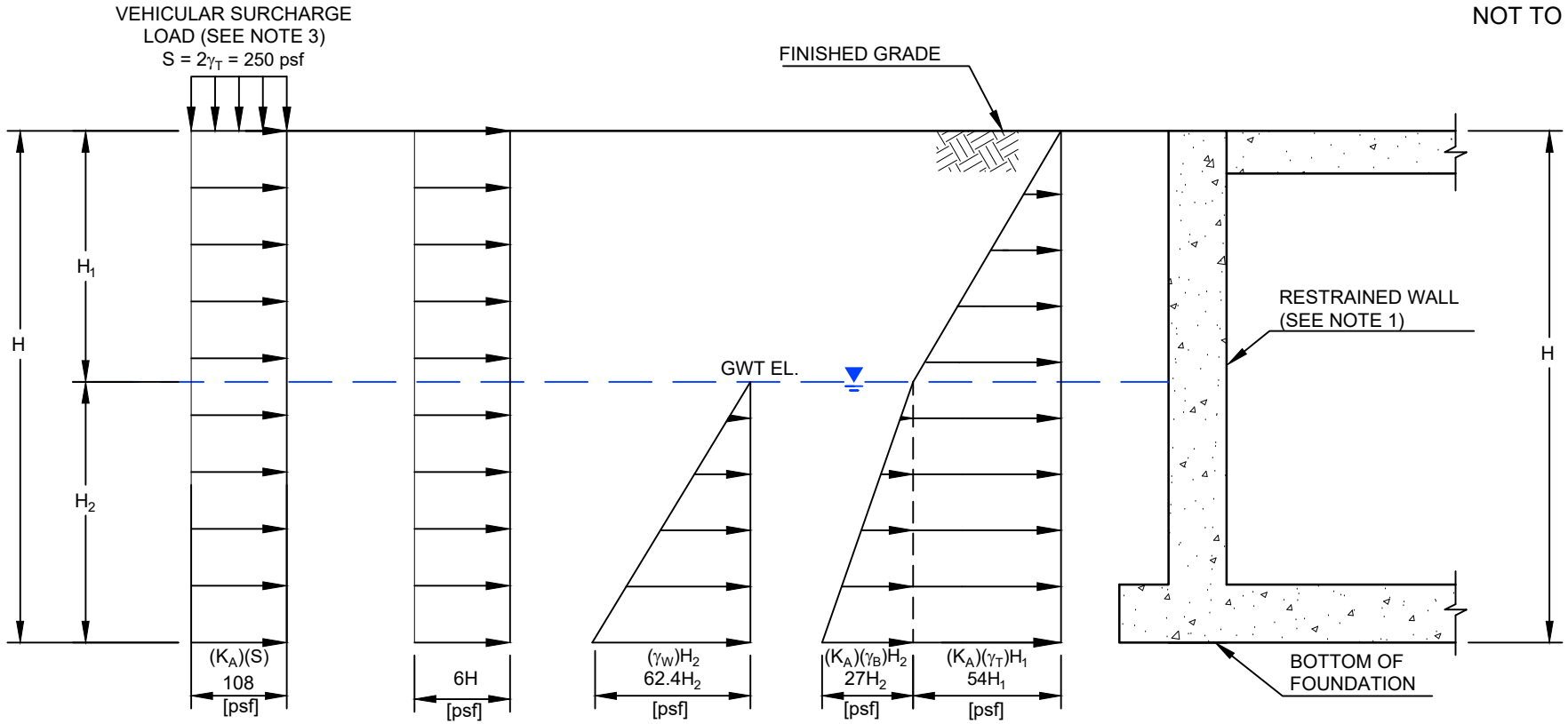
1. RESTRAINED WALLS ARE NOT FREE TO MOVE/ROTATE. RESTRAINED WALLS INCLUDE BRACED WALLS AND RIGID CANTILEVER WALLS.
2. THE HEIGHT OF THE WALL (H) IS FROM THE FINISHED GRADE TO THE BOTTOM OF THE FOUNDATION IN FEET (I.E. $H_1 + H_2$). H_1 REPRESENTS THE PORTION OF THE WALL ABOVE THE GROUNDWATER TABLE (GWT), AND H_2 REPRESENTS THE PORTION OF THE WALL BELOW THE GWT.
3. VEHICULAR SURCHARGE (S) SHOULD BE TAKEN AS EQUAL TO THE SURCHARGE LOAD FROM 2 FEET OF FILL. THE HORIZONTAL PRESSURE DUE TO VEHICULAR SURCHARGE IS THE SURCHARGE LOAD MULTIPLIED BY THE AT-REST EARTH PRESSURE COEFFICIENT (K_0) FOR THE FULL HEIGHT OF THE WALL.
4. FOR THE STATIC LOADING CASE FOR RESTRAINED WALLS, THE EARTH PRESSURE COMPONENTS USED ARE THE AT-REST VALUES.
5. THE EARTH PRESSURE SHOWN HEREIN IS BASED ON THE ASSUMPTION OF LEVEL BACKFILL.
6. THE HYDROSTATIC PRESSURE DISTRIBUTION SHOWN IS BASED ON THE ASSUMPTION THAT A PROPERLY DESIGNED FILTER AND BACK DRAIN IS PLACED BEHIND THE WALL.

STATIC LATERAL EARTH PRESSURES ON STRUCTURE WALLS

APRIL 2021

BELLOTA WEIR MODIFICATION PROJECT
SAN JOAQUIN, CALIFORNIA

NOT TO SCALE



PARAMETER	VALUE
ACTIVE EARTH PRESSURE COEFFICIENT (K_A)	0.43
TOTAL UNIT WEIGHT (γ_T)	125 pcf
UNIT WEIGHT OF WATER (γ_W)	62.4 pcf
BUOYANT UNIT WEIGHT ($\gamma_B = \gamma_T - \gamma_W$)	62.6 pcf

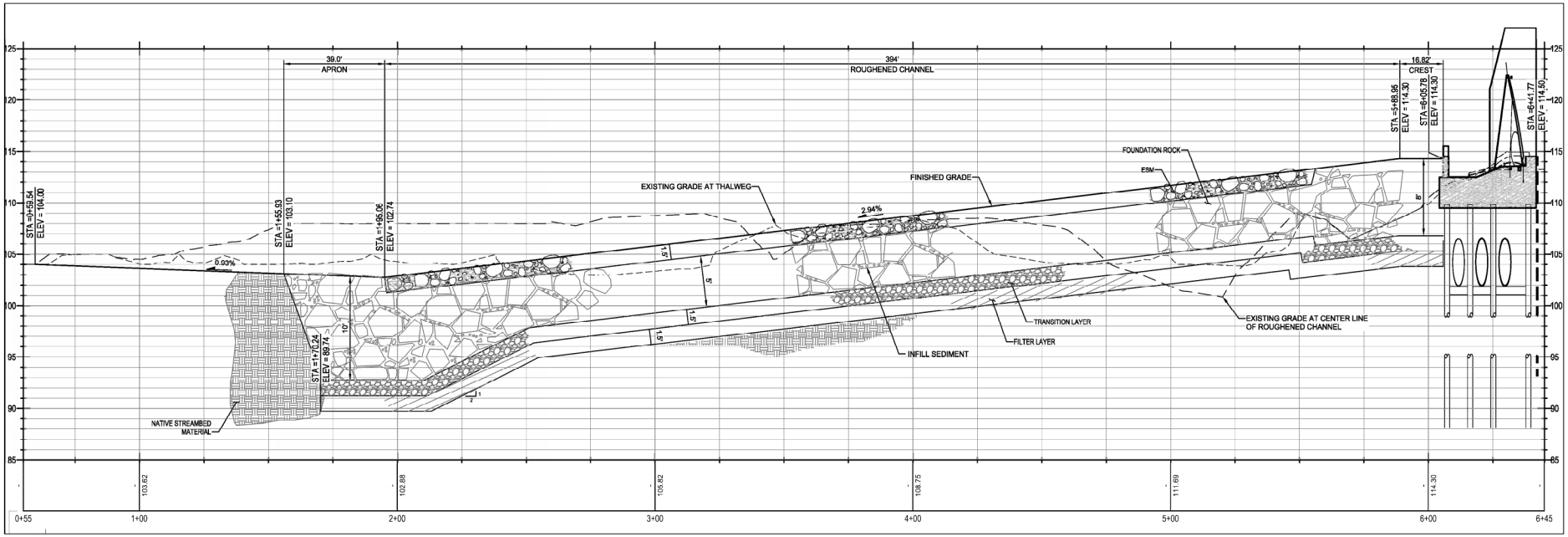
NOTES:

1. RESTRAINED WALLS ARE NOT FREE TO MOVE/ROTATE. RESTRAINED WALLS INCLUDE BRACED WALLS AND RIGID CANTILEVER WALLS.
2. THE HEIGHT OF THE WALL (H) IS FROM THE FINISHED GRADE TO THE BOTTOM OF THE FOUNDATION IN FEET (I.E. $H_1 + H_2$). H_1 REPRESENTS THE PORTION OF THE WALL ABOVE THE GROUNDWATER TABLE (GWT), AND H_2 REPRESENTS THE PORTION OF THE WALL BELOW THE GWT.
3. THE STRUCTURAL ENGINEER SHALL DETERMINE IF A VEHICULAR SURCHARGE SHOULD BE APPLIED FOR THE SEISMIC LOADING CASE. IF A VEHICULAR SURCHARGE (S) IS APPLIED, IT SHOULD BE TAKEN AS EQUAL TO THE SURCHARGE LOAD FROM 2 FEET OF FILL. THE HORIZONTAL PRESSURE DUE TO VEHICULAR SURCHARGE IS THE SURCHARGE LOAD MULTIPLIED BY THE ACTIVE EARTH PRESSURE COEFFICIENT (K_A) FOR THE FULL HEIGHT OF THE WALL.
4. FOR THE SEISMIC LOADING CASE FOR RESTRAINED WALLS, THE EARTH PRESSURE COMPONENTS USED ARE THE ACTIVE VALUES.
5. THE EARTH PRESSURE SHOWN HEREIN IS BASED ON THE ASSUMPTION OF LEVEL BACKFILL.
6. THE HYDROSTATIC PRESSURE DISTRIBUTION SHOWN IS BASED ON THE ASSUMPTION THAT A PROPERLY DESIGNED FILTER AND BACK DRAIN IS PLACED BEHIND THE WALL.

SEISMIC LATERAL EARTH PRESSURES ON STRUCTURE WALLS

APRIL 2021

BELLOTA WEIR MODIFICATION PROJECT
SAN JOAQUIN, CALIFORNIA



NOTES: Source: Sheet RI201 of Bellota Weir Modifications Project, 90% design submittal

Bellota Weir Modification Project

Bellota Weir Roughened Channel Downstream Profile

Feb 2022 FIGURE 19

Appendix B. 2004-2005 Historic Boring Logs

Appendix B - Historic Boring Logs
Bellota weir Modification Project

Background:

Bellota Weir is located where water flowing down the Calaveras River splits between the Mormon Slough and the Old Calaveras River channel. The project includes weir and intake improvements that will provide more reliable water delivery systems for Stockton East Water District (SEWD) customers, improve migration of anadromous fish, and reduce fish entrainment at the facilities, thereby enhancing populations of anadromous salmonids present in the Calaveras River.

Existing Geotechnical Explorations:

Existing geotechnical explorations includes previously conducted drilling at the site by CH2M Hill between October 2004 and January 2005. Five borings were visually observed and logged during the previous exploration program. These logs and laboratory testing indicate that the project site is generally underlain by riverine alluvial deposits to the depths explored (CH2M Hill 2005). The geologic map indicates presence of Holocene alluvium channel and levee deposits and Upper Modesto formation (Marchand and Bartow 1979).

Summary of Explorations:

The CH2M Hill borings were drilled in two phases to accommodate in-river work-window restrictions. Borings BI-2 and BI-3 were drilled within the river channel on October 25 and 26, 2004, using a Simco 2400 SK-1 drill rig. Drilling was advanced with a 6-inch-diameter hollow-stem and continuous-flight auger. The remaining borings were drilled on January 17, 18, and 19, 2005 using a Mobile B-80 drill rig. Drilling was advanced with a 5.5-inch-diameter tricone bit and mud-rotary drilling techniques. During both phases of the investigation, a 2-inch-outside-diameter split-spoon sampler was driven using a safety hammer of 140 pounds falling 30 inches, in accordance with the American Society for Testing Materials (ASTM) Standard Penetration Test (SPT; ASTM D1586). The hammer was lifted with a rope and cathead system for the initial borings, and an automatic trip hammer system for the latter borings. The number of blows required to drive the sampler 18 inches in 6-inch increments was recorded, and the number of blows required to drive the last two 6-inch intervals was summed to get the “N” value. Sampling was performed at approximate 5-foot intervals. Additional sampling was performed by hydraulically pushing thin-walled steel Shelby tubes (3-inch-diameter) into the soil at selected depths.

Select samples obtained during drilling were transported to a soil testing laboratory and tested for moisture content, dry density, gradation, and Atterberg limits.

Borings BI-1 through BI-3 were drilled near the proposed intake and fish screen structure. The presence of buried bank-protection riprap within the footprint of the proposed intake structure

necessitated that the borings be relocated away from the riverbank during drilling to a gravel bar within the river. Borings BF-1 and BD-2 were drilled on the south and north banks of the river, respectively, near the proposed inflatable crest-gate weir and fishway facilities. The presence of deep tailwater on the downstream limit of the existing apron prevented drill rig access to the portion of the apron south of the existing fish ladder structure. No borings were drilled within the river south of the fish ladder or on the sloping abutment.

Soil samples were classified using the Unified Soil Classification System in accordance with ASTM D2487. The relative compactness of the soil included in the soil description was determined from the SPT blow count obtained during the subsurface investigation. To determine the relative density and consistency of the soils encountered in the boring, an N60 blow count, or a corrected blow count, was calculated. The N60 blow count is standardized to account for samplers and hammers other than the standard 2-inch split spoon sampler and the standard safety hammer (140 pounds falling 30 inches). For classifying relative density and consistency to evaluate liquefaction potential, the uncorrected field blow counts shown on the logs were corrected for hammer type by multiplying the field blow count by the applicable correction factors (Attachment A.

Subsurface conditions were determined using the boring log information prepared during the investigation. Subsurface soil generally consists of very loose to loose silty or clayey sand underlain by medium-dense to dense sand interbedded by stiff clay, silt, and gravel lenses.

References

CH2M Hill

2005 Phase 2 Preliminary Design Report, Calaveras River Anadromous Fish Protection Project. April, 2005.

American Society for Testing and Material

2011 ASTM Standard D1586 “Standard Test Method for Standard Penetration Test (SPT) and Split-Barrel Sampling of Soils,” ASTM International, West Conshohocken, PA, DOI: 10.1520/D1586-11, www.astm.org.

2011 ASTM Standard D2487 “Standard Practice for Classification of Soils for Engineering Purposes (Unified Soil Classification System),” ASTM International, West Conshohocken, PA, DOI: 10.1520/D2487-11, www.astm.org.

Attachment A. Historic Boring Logs

CH2MHILL

PROJECT NUMBER:	BORING NUMBER:
175404.AA.P2.03	BD-2 SHEET 1 OF 3

SOIL BORING LOG

PROJECT : Calaveras River Fish Passage Improvement Project, Bellota, CA LOCATION : North side of Apron, 14 feet West of Existing Dam

ELEVATION : Approximately 111.5 feet DRILLING CONTRACTOR : Gregg Drilling

DRILLING METHOD AND EQUIPMENT : Mobile B-80 Drill Rig, Mud Rotary, Automatic Hammer

WATER LEVELS : --- START : END : LOGGER : A. Evans

DEPTH BELOW GROUND SURFACE (ft)	INTERVAL (ft)	RECOVERY (ft)	#TYPE	STANDARD PENETRATION TEST RESULTS 6"-6"-6" (N)	SOIL DESCRIPTION: SOIL NAME, USCS GROUP SYMBOL, COLOR, MOISTURE CONTENT, RELATIVE DENSITY OR CONSISTENCY, SOIL STRUCTURE, MINERALOGY	COMMENTS: DEPTH OF CASING, DRILLING RATE, DRILLING FLUID LOSS, TESTS, AND INSTRUMENTATION
5	5.0				0.75 feet - Concrete CLAYEY SAND WITH GRAVEL (SC), light brown, wet, with coarse gravel and fine cobble	9:00 am - Start time Encountered gravel and cobbles. Very hard to clear the hole. Drove 8-inch casing to 5 feet bgs.
	6.5	1.5	1-SS	0-0-2 (2)	SILTY CLAYEY SAND (SC-SM), gray with orange brown oxide staining, wet, very loose, fine-grained sand	Very soft. Sampler sank 1 foot with weight of hammer P200 = 45.0%; LL = 25, PL = 21, PI = 4
	9.0					
10	10.5	0.0	2-SS	1-1-1 (2)	SILTY CLAYEY SAND (SC-SM), similar to above	No recovery Very soft. Sampler tip plugged with gravel. Soil description was logged from soil encountered in cuttings.
	14.0					
15	15.5	0.5	3-SS	1-14-32 (46)	CLAYEY GRAVEL WITH SAND (GC), gray with reddish and yellowish gravel, wet, dense, coarse sand, with 1-inch diameter gravel in sample	Drill bouncing P200 = 29.8%; SA; %Gravel = 45.3%
	19.0					
20	20.5	1.0	4-SS	5-6-6 (12)	CLAYEY SAND (SC), light brown with horizontal orangish oxide stains in layers, wet, medium dense, medium- to fine-grained sand	
	24.0					
25	25.5	1.5	5-SS	7-8-12 (20)	ELASTIC SILT WITH SAND (MH), light brown with orangish stains, wet, medium dense, fine-grained sand	Unable to clear hole, lower down Modified California sampler to pull up gravels P200% = 76.2%; LL = 57, PL = 41, PI = 16
	29.0					
30		1.5	6-SS	6-8-11 (19)	CLAYEY SAND (SC), light gray with pinkish blue, moist, medium dense, fine-grained sand	

CH2MHILL

PROJECT NUMBER:	BORING NUMBER:
175404.AA.P2.03	BD-2 SHEET 2 OF 3

SOIL BORING LOG

PROJECT : Calaveras River Fish Passage Improvement Project, Bellota, CA LOCATION : North side of Apron, 14 feet West of Existing Dam

ELEVATION : Approximately 111.5 feet DRILLING CONTRACTOR : Gregg Drilling

DRILLING METHOD AND EQUIPMENT : Mobile B-80 Drill Rig, Mud Rotary, Automatic Hammer

WATER LEVELS : --- START : END : LOGGER : A. Evans

DEPTH BELOW GROUND SURFACE (ft)	INTERVAL (ft)	RECOVERY (ft)	#TYPE	STANDARD PENETRATION TEST RESULTS 6"-6"-6" (N)	SOIL DESCRIPTION: SOIL NAME, USCS GROUP SYMBOL, COLOR, MOISTURE CONTENT, RELATIVE DENSITY OR CONSISTENCY, SOIL STRUCTURE, MINERALOGY	COMMENTS: DEPTH OF CASING, DRILLING RATE, DRILLING FLUID LOSS, TESTS, AND INSTRUMENTATION
	30.5					
	34.0					
35	35.5	1.5	7-SS	5-8-7 (15)	CLAYEY SAND (SC), light brown, wet, medium dense, fine-grained sand, becoming gray medium-grained and less clayey in bottom 6 inches of the sample	
	39.0					
40	40.5	1.4	8-SS	7-8-7 (15)	CLAYEY SAND/WELL-GRADED SAND (SC/SW), gray, wet, medium dense	12:00 pm
	44.0					
45	45.5	1.5	9-SS	8-17-49 (66)	CLAYEY SAND/SANDY LEAN CLAY (SC/CL), light brown to light gray, moist, very dense, fine-grained sand	
	49.0					
50	50.5	0.5	10-SS	6-11-12 (23)	LEAN CLAY WITH SAND (CL), gray, moist, very stiff	12:30 pm P200 = 70.8%
	54.0					
55	55.5	1.5	11-SS	11-21-33 (54)	CLAYEY SAND/WELL-GRADED SAND (SC/SW), light brown with some pinkish brown zones, moist, very dense, some horizontal layering, very hard zones	
	59.0					
60		0.7	12-SS	37-50/6" (>50)	SILTY SAND (SM), brown, moist, very dense, fine-grained sand	Broken gravel in sample P200 = 35.0%; SA; %Gravel = 1.1%

CH2MHILL

PROJECT NUMBER:	BORING NUMBER:
175404.AA.P2.03	BD-2 SHEET 3 OF 3

SOIL BORING LOG

PROJECT : Calaveras River Fish Passage Improvement Project, Bellota, CA LOCATION : North side of Apron, 14 feet West of Existing Dam

ELEVATION : Approximately 111.5 feet DRILLING CONTRACTOR : Gregg Drilling

DRILLING METHOD AND EQUIPMENT : Mobile B-80 Drill Rig, Mud Rotary, Automatic Hammer

WATER LEVELS : --- START : END : LOGGER : A. Evans

DEPTH BELOW GROUND SURFACE (ft)	INTERVAL (ft)	RECOVERY (ft)	#TYPE	STANDARD PENETRATION TEST RESULTS 6"-6"-6" (N)	SOIL DESCRIPTION: SOIL NAME, USCS GROUP SYMBOL, COLOR, MOISTURE CONTENT, RELATIVE DENSITY OR CONSISTENCY, SOIL STRUCTURE, MINERALOGY	COMMENTS: DEPTH OF CASING, DRILLING RATE, DRILLING FLUID LOSS, TESTS, AND INSTRUMENTATION
60.5						
64.0						
65	65.5	0.5	13-SS	22-42-50/4" (>92)	CLAYEY SAND (SC), yellowish brown, moist, very dense, fine-grained sand, with some hard black nodules less than 1/8-inch diameter	1:38 pm Gravels in top of sample
69.0						
70	70.5	0.6	14-SS	17-50/6" (>50)	CLAYEY SAND (SC), similar to above, no nodules	2:00 pm Gravels in top of sample Hole grouted at 3:13 pm
75					Bottom of Hole at 70.5 ft below ground surface	
80						
85						
90						

CH2MHILL

PROJECT NUMBER:	BORING NUMBER:
175404.AA.P2.03	BF-1 SHEET 1 OF 3

SOIL BORING LOG

PROJECT : Calaveras River Fish Passage Improvement Project, Bellota, CA LOCATION : Slab at Roll Off Dumpster

ELEVATION : Approximately 121.0 feet DRILLING CONTRACTOR : Gregg Drilling

DRILLING METHOD AND EQUIPMENT : Mobile B-80 Drill Rig, Mud Rotary, Automatic Hammer

WATER LEVELS : --- START : 1/17/05 15:15 END : 1/18/05 12:55 LOGGER : A. Evans

DEPTH BELOW GROUND SURFACE (ft)	INTERVAL (ft)	RECOVERY (ft)	#TYPE	STANDARD PENETRATION TEST RESULTS 6"-6"-6" (N)	SOIL DESCRIPTION: SOIL NAME, USCS GROUP SYMBOL, COLOR, MOISTURE CONTENT, RELATIVE DENSITY OR CONSISTENCY, SOIL STRUCTURE, MINERALOGY	COMMENTS: DEPTH OF CASING, DRILLING RATE, DRILLING FLUID LOSS, TESTS, AND INSTRUMENTATION
					0.73 feet - Concrete with rebar	Very hard drilling with gravel and cobble
5	5.0					Transition at 4.5 feet bgs.
	6.5	0.0	1-SS	3-3-3 (6)	POORLY GRADED SAND WITH GRAVEL/CLAYEY SAND WITH GRAVEL (SP/SC), light brown, wet, loose, fine- to medium-grained sand	No recovery Very hard drilling. No catcher installed. Logged from rotary wash.
10	10.0					
	11.5	0.7	2-SS	3-4-3 (7)	SILTY SAND (SM), light orangish brown and gray, wet, loose, fine-grained sand	3:45 pm PP = 0.75 tsf.
15	15.0					
	17.0	1.6	3-ST	push	SILTY SAND (SM), similar to above	4:10 pm Pushed tube at 200 psi to refusal at 1.9 feet. Tube was possibly full. P200 = 19.3%; WC = 31.7%; UWD = 90 pcf; LL = 25, PL = NP, PI = NP
	18.5	0.7	4-SS	4-4-6 (10)	SILTY SAND (SM), similar to above, more medium-grained sand	PP = 0.25 tsf. Very wet
20	20.0					
	21.5	1.2	5-SS	6-32-35 (67)	CLAYEY SAND WITH GRAVEL (SC), orangish brown and gray, wet, very dense, coarse- to medium-grained sand, with fine- to medium-grained gravel	Stopped drilling at 4:48pm 1/17/05 Started drilling at 8:20am 1/18/05 Rough drilling, bouncing.
25	25.0					
	26.5	0.0	6-SS	7-5-5 (10)	CLAYEY SAND WITH GRAVEL (SC), similar to above, loose, with subrounded gravels (less than 1.5-inch diameter)	No recovery Gravel in sampler tip dropped from cuttings. Having trouble clearing gravels from hole.
30						

CH2MHILL

PROJECT NUMBER:	BORING NUMBER:
175404.AA.P2.03	BF-1 SHEET 2 OF 3

SOIL BORING LOG

PROJECT : Calaveras River Fish Passage Improvement Project, Bellota, CA LOCATION : Slab at Roll Off Dumpster

ELEVATION : Approximately 121.0 feet DRILLING CONTRACTOR : Gregg Drilling

DRILLING METHOD AND EQUIPMENT : Mobile B-80 Drill Rig, Mud Rotary, Automatic Hammer

WATER LEVELS : --- START : 1/17/05 15:15 END : 1/18/05 12:55 LOGGER : A. Evans

DEPTH BELOW GROUND SURFACE (ft)	INTERVAL (ft)	RECOVERY (ft)	#TYPE	STANDARD PENETRATION TEST RESULTS 6"-6"-6" (N)	SOIL DESCRIPTION: SOIL NAME, USCS GROUP SYMBOL, COLOR, MOISTURE CONTENT, RELATIVE DENSITY OR CONSISTENCY, SOIL STRUCTURE, MINERALOGY	COMMENTS: DEPTH OF CASING, DRILLING RATE, DRILLING FLUID LOSS, TESTS, AND INSTRUMENTATION
30.0						
	30.0 - 31.5	0.3	7-SS	10-11-7 (18)	CLAYEY SAND/SANDY LEAN CLAY (SC/CL), orange brown, moist, medium dense	Large gravels at bottom. Used Modified California sampler to try and remove rocks. 2" - 2.5" Max size and some soil PP = 2.0 tsf. Drilled without mud between 30 and 33 feet bgs.
35	35.0 - 36.5	1.5	8-SS	9-12-19 (31)	SILTY SAND WITH GRAVEL (SM), very dark brown with yellow and white and pink gravels, moist, dense, with fine-grained gravel	PP = 4.0 tsf. Started drilling with mud at 35 feet bgs.
40	40.0 - 41.5	1.5	9-SS	4-9-8 (17)	FAT CLAY WITH SAND (CH), light gray with pinkish blue, moist, very stiff	P200 = 75.9%; LL = 59, PL = 26, PI = 33
45	45.0 - 46.5	1.5	10-SS	5-6-8 (14)	POORLY GRADED SAND/CLAYEY SAND (SP/SC), grayish brown, wet, medium dense, medium- to fine-grained sand	PP = 1.5 tsf.
50	50.0 - 51.5	1.5	11-SS	6-15-16 (31)	POORLY GRADED SAND/CLAYEY SAND (SP/SC), similar to above, becoming CLAYEY SAND (SC), light gray, wet, dense at bottom 6 inches of sample	
55	55.0 - 56.5	1.5	12-SS	5-10-14 (24)	SANDY LEAN CLAY (CL), light brown transitioning to light gray, wet, very stiff, with coarse- to fine-grained sand	P200 = 69.3%
60						

CH2MHILL

PROJECT NUMBER:	BORING NUMBER:
175404.AA.P2.03	BF-1 SHEET 3 OF 3

SOIL BORING LOG

PROJECT : Calaveras River Fish Passage Improvement Project, Bellota, CA LOCATION : Slab at Roll Off Dumpster

ELEVATION : Approximately 121.0 feet DRILLING CONTRACTOR : Gregg Drilling

DRILLING METHOD AND EQUIPMENT : Mobile B-80 Drill Rig, Mud Rotary, Automatic Hammer

WATER LEVELS : --- START : 1/17/05 15:15 END : 1/18/05 12:55 LOGGER : A. Evans

DEPTH BELOW GROUND SURFACE (ft)	INTERVAL (ft)	RECOVERY (ft)	#TYPE	STANDARD PENETRATION TEST RESULTS 6"-6"-6" (N)	SOIL DESCRIPTION: SOIL NAME, USCS GROUP SYMBOL, COLOR, MOISTURE CONTENT, RELATIVE DENSITY OR CONSISTENCY, SOIL STRUCTURE, MINERALOGY	COMMENTS: DEPTH OF CASING, DRILLING RATE, DRILLING FLUID LOSS, TESTS, AND INSTRUMENTATION
60.0						
	60.0 – 61.5	1.0	13-SS	23-50/5.5" (>50)	CLAYEY SAND WITH GRAVEL/WELL-GRADED SAND (SC/SW), wet, very dense, well graded sand, with less than 5% fine-grained gravel	12:00 pm
65	65.0 – 66.5	1.5	14-SS	11-17-25 (42)	SANDY LEAN CLAY/LEAN CLAY (CL), very light brown with pinkish blue structure, dry, hard, soft-rock like, some orange oxidation veins	Drill bouncing
70	70.0 – 71.5	1.5	15-SS	8-12-12 (24)	POORLY GRADED SAND/CLAYEY SAND (SP/SC), grayish brown, wet, medium dense, fine-grained sand	Drill bouncing
75	75.0 – 76.5	1.2	16-SS	12-13-8 (21)	WELL-GRADED SAND/CLAYEY SAND (SW/SC), grayish brown, wet, medium dense	12:55 pm
					Bottom of Hole at 76.5 ft below ground surface 1/18/05 12:55	
80						
85						
90						

CH2MHILL

PROJECT NUMBER:	BORING NUMBER:	
175404.AA.P2.03	BI-1	SHEET 1 OF 2

SOIL BORING LOG

PROJECT : Calaveras River Fish Passage Improvement Project, Bellota, CA LOCATION : 25 feet East of Existing Intake

ELEVATION : Approximately 121.5 feet DRILLING CONTRACTOR : Gregg Drilling

DRILLING METHOD AND EQUIPMENT : Mobile B-80 Drill Rig, Mud Rotary, Automatic Hammer

WATER LEVELS : --- START : 1/17/05 10:00 END : 1/17/05 14:00 LOGGER : A. Evans

DEPTH BELOW GROUND SURFACE (ft)	INTERVAL (ft)	RECOVERY (ft)	#TYPE	STANDARD PENETRATION TEST RESULTS 6"-6"-6" (N)	SOIL DESCRIPTION: SOIL NAME, USCS GROUP SYMBOL, COLOR, MOISTURE CONTENT, RELATIVE DENSITY OR CONSISTENCY, SOIL STRUCTURE, MINERALOGY	COMMENTS: DEPTH OF CASING, DRILLING RATE, DRILLING FLUID LOSS, TESTS, AND INSTRUMENTATION
						Hard drilling, with small cobble/rip rap
5	5.0					
	6.5	0.8	1-SS	7-10-12 (22)	SILTY SAND WITH GRAVEL (SM), gray, wet, medium dense, well graded sand, with fine-grained gravel	PP = 1.0, 1.5 tsf.
						7.5 feet - Smooth drilling
10	10.0					
	11.5	1.2	2-SS	1-1-1 (2)	SILTY SAND (SM), light brown, wet, very loose, fine-grained sand	P200 = 46.3%
	12.5					
	14.5	0.0	3-ST	push	SILTY SAND (SM), similar to above	Pushed tube at 500 psi End crushed on gravel sluff No recovery, too sandy
15	15.0					
	16.5	0.0	4-SS	2-1-2 (3)	SILTY SAND (SM), similar to above	No recovery
20	20.0					
	21.5	0.3	5-SS	1-2-2 (4)	POORLY GRADED SAND/SILTY SAND (SP/SM), gray and white, wet, very loose, medium-grained sand	
						Rough drilling, bouncing on rocks
25	25.0					
	26.5		6-SS	17-22-21 (43)	WELL-GRADED GRAVEL WITH SILT AND SAND (GW-GM), gray and yellow brown gravels and quartz, wet, dense, approximately 1 inch diameter gravel	P200 = 8.0%; SA; %Gravel = 62.2%
30						

CH2MHILL

PROJECT NUMBER:	BORING NUMBER:
175404.AA.P2.03	BI-1 SHEET 2 OF 2

SOIL BORING LOG

PROJECT : Calaveras River Fish Passage Improvement Project, Bellota, CA LOCATION : 25 feet East of Existing Intake

ELEVATION : Approximately 121.5 feet DRILLING CONTRACTOR : Gregg Drilling

DRILLING METHOD AND EQUIPMENT : Mobile B-80 Drill Rig, Mud Rotary, Automatic Hammer

WATER LEVELS : --- START : 1/17/05 10:00 END : 1/17/05 14:00 LOGGER : A. Evans

DEPTH BELOW GROUND SURFACE (ft)	INTERVAL (ft)	RECOVERY (ft)	#TYPE	STANDARD PENETRATION TEST RESULTS 6"-6"-6" (N)	SOIL DESCRIPTION: SOIL NAME, USCS GROUP SYMBOL, COLOR, MOISTURE CONTENT, RELATIVE DENSITY OR CONSISTENCY, SOIL STRUCTURE, MINERALOGY	COMMENTS: DEPTH OF CASING, DRILLING RATE, DRILLING FLUID LOSS, TESTS, AND INSTRUMENTATION
	30.0					Probably gravel in tip of the sample
	31.5	0.0	7-SS	4-4-4 (8)		No recovery
						Smooth drilling between 33 and 33.5 feet bgs.
35	35.0					
	36.5	1.5	8-SS	7-9-8 (17)	SANDY SILT (ML), gray to light gray with orange oxidation stains, wet, very stiff	Rough drilling, not typical for what is in the sample P200 = 50.1%; LL = 45, PL = 42, PI = 3
40	40.0					
	41.5	0.4	9-SS	8-8-9 (17)	SILTY SAND (SM), light brown, wet, medium dense, fine-grained sand	12:49 pm Staff gage 1.4 feet PP = 1.5 tsf.
45	45.0					
	46.5	1.5	10-SS	2-5-6 (11)	SILTY SAND (SM), similar to above, light brown with thin white veins, with some medium-grained sand zones	PP = 2.0 tsf.
50	50.0					
	51.5	1.5	11-SS	4-9-8 (17)	WELL-GRADED SAND/CLAYEY SAND (SW/SC), light brown, wet, medium dense	2:00 pm - End drilling
55					Bottom of Hole at 51.5 ft below ground surface 1/17/05 14:00	
60						

CH2MHILL

PROJECT NUMBER:	BORING NUMBER:
175404.AA.P2.03	BI-2 SHEET 1 OF 1

SOIL BORING LOG

PROJECT : Calaveras River Fish Passage Improvement Project, Bellota, CA LOCATION :

ELEVATION : Approximately 114.0 feet DRILLING CONTRACTOR : Taber

DRILLING METHOD AND EQUIPMENT : Simco 2400 SK-1, Hollow Stem Auger, 6-inch, Safety Hammer with Rope and Cathead

WATER LEVELS : --- START : 10/26/04 10:15 END : 10/26/04 16:00 LOGGER : A. Finney

DEPTH BELOW GROUND SURFACE (ft)	INTERVAL (ft)	RECOVERY (ft)	#TYPE	STANDARD PENETRATION TEST RESULTS 6"-6"-6" (N)	SOIL DESCRIPTION: SOIL NAME, USCS GROUP SYMBOL, COLOR, MOISTURE CONTENT, RELATIVE DENSITY OR CONSISTENCY, SOIL STRUCTURE, MINERALOGY	COMMENTS: DEPTH OF CASING, DRILLING RATE, DRILLING FLUID LOSS, TESTS, AND INSTRUMENTATION
						Driller was unable to advance auger. Rip rap buried 12 inches below surface. Moved 3-4 feet to the north. Restarted drilling at 11 am. Moved and restarted 4 times to avoid rip rap.
5	5.0					
	6.5	1.0	1-SS	15-16-9 (25)	WELL-GRADED GRAVEL WITH SAND (GW), dark yellowish brown, wet, medium dense	1:30 pm P200 = 4.7%; SA
						Driller noted material changed at 7.5 feet bgs.
10	10.0					
	11.5	1.5	2-SS	1-3-3 (6)	SILTY SAND (SM), dark yellowish brown, wet, loose	LL = 24, PL = NP, PI = NP; P200 = 41.8%; SA; %Gravel = 53.3%
						Driller noted material changed at 12.5 feet bgs.
15	15.0					
	16.5	1.5	3-SS	1-1-2 (3)	CLAYEY SAND (SC), reddish brown, wet, very loose, very fine-grained sand	2:30 pm Driller added water to ensure material does not enter when plugs pulled
						Drilling much harder below 17.5 feet, rods chattering
20	20.0					
	21.5	1.1	4-SS	14-25-35 (60)	POORLY GRADED GRAVEL WITH SAND AND SILT (GP-GM), dark yellow brown, wet, very dense	
						Driller noted material changed at 22.5 feet bgs.
25	25.0					
	26.5	1.5	5-SS	5-9-12 (21)	SANDY ELASTIC SILT (MH), pinkish brown with rust and black mottling, moist, very stiff, grading to silty clayey sand	PP = 3.0 tsf. LL = 50, PL = 39, PI = 11; P200 = 66.2%
	28.5					
30	30.0		6-SS	5-5-8 (13)	SANDY ELASTIC SILT (MH), pinkish brown and gray, moist, stiff	4:00 pm

Bottom of Hole at 30.0 ft below ground surface
10/26/04 16:00

CH2MHILL

PROJECT NUMBER:	BORING NUMBER:
175404.AA.P2.03	BI-3 SHEET 1 OF 1

SOIL BORING LOG

PROJECT : Calaveras River Fish Passage Improvement Project, Bellota, CA LOCATION : Gravel Bar

ELEVATION : Approximately 113.5 feet DRILLING CONTRACTOR : Taber

DRILLING METHOD AND EQUIPMENT : Simco 2400 SK-1, Hollow Stem Auger, 6-inch, Safety Hammer with Rope and Cathead

WATER LEVELS : --- START : 10/25/04 14:45 END : 10/26/04 09:15 LOGGER : A. Finney

DEPTH BELOW GROUND SURFACE (ft)	INTERVAL (ft)	RECOVERY (ft)	#TYPE	STANDARD PENETRATION TEST RESULTS 6"-6"-6" (N)	SOIL DESCRIPTION: SOIL NAME, USCS GROUP SYMBOL, COLOR, MOISTURE CONTENT, RELATIVE DENSITY OR CONSISTENCY, SOIL STRUCTURE, MINERALOGY	COMMENTS: DEPTH OF CASING, DRILLING RATE, DRILLING FLUID LOSS, TESTS, AND INSTRUMENTATION
2.5					SILTY SAND WITH GRAVEL (SM), dark yellowish brown, wet, medium dense	Surface covered with cobbles
4.0	1.0	1	SS	10-17-20 (37)	WELL GRADED GRAVEL WITH SILT AND SAND (GW-GM), dark yellowish brown, wet, dense	
5						
7.5						
9.0	1.5	2	SS	2-1-7 (8)	POORLY GRADED SAND (SP), dark yellowish brown, wet, loose, with some fine gravel and wood fragments at 8.5 feet bgs.	3:20 pm Rock in tip of sampler P200 = 4.3%; SA; LL = 31, PL = NP, PI = NP
10						Gravel and harder drilling at 10 feet bgs.
12.5						
14.0	1.2	3	SS	45-32-29 (61)	WELL-GRADED SAND WITH SILT AND GRAVEL (SW-SM), gray, wet, dense	4:30 pm
15						
17.5						
19.0	1.2	4	SS	17-12-17 (29)	WELL-GRADED SAND WITH SILT AND GRAVEL (SW-SM), similar to above	P200 = 7.0%; SA; %Gravel = 33.4%
20						
22.5						
24.0	0.9	5	SS	7-9-5 (14)	WELL-GRADED SAND WITH SILT AND GRAVEL (SW-SM), similar to above, gray, medium dense	5:45 pm
25						Driller noted material changed at 24 feet bgs
28.0						
29.5	1.5	6	SS	7-6-9 (15)	SANDY SILT (ML), tan, moist, stiff	P200 = 53.0%; LL = 42, PL = 40, PI = 2 6:30 pm Too dark to continue. Left auger in ground. Returned at 8:00 am. Drillers on site at 9:15 am.
30						

Bottom of Hole at 29.5 ft below ground surface
10/26/04 09:15

Appendix C. 2021 Supplemental Boring Logs

Appendix C - Supplemental Boring Logs
Bellota weir Modification Project

Thursday, April 01, 2021

To: Mark Stanley, P.E., G.E., D.GE

From: Sagar Raj Satyal, EIT

Background

Bellota Weir is located where water flowing down the Calaveras River splits between the Mormon Slough and the Old Calaveras River channel. The project includes weir and intake improvements that will provide more reliable water delivery systems for Stockton East Water District (SEWD) customers, improve migration of anadromous fish, and reduce fish entrainment at the facilities, thereby enhancing populations of anadromous salmonids present in the Calaveras River.

Existing Geotechnical Explorations

Existing geotechnical explorations includes previously conducted drilling at the site by CH2M Hill between October 2004 and January 2005. Five borings were visually observed and logged during the previous exploration program. These logs and laboratory testing indicate that the project site is generally underlain by riverine alluvial deposits to the depths explored (CH2M Hill 2005). The geologic map indicates presence of Holocene alluvium channel and levee deposits and Upper Modesto formation (Marchand and Bartow 1979).

Supplemental Geotechnical Explorations

Exploration Preparation

HDR developed exploration and health and safety work plans prior to site mobilization. The exploration work plan included a description of the location and proposed depth of explorations and type of drilling method/equipment to use for advancing the boring.

As required by San Joaquin County, a well (drilling) permit was obtained for the explorations. As required prior to performing explorations, the proposed exploration locations were marked in the field and California 811 Service Alert was contacted to check for the presence of underground utilities in the proposed locations.

Summary of Explorations

The field investigation consisted of advancing a total of five exploratory borings to depths of 70 feet adjacent to the weir. Four mud-rotary borings were advanced to depths of 70 feet along the northern bank of Mormon Slough to provide a better understanding of the materials, help interpret the subsurface stratigraphy of the area, and characterize the soil layers. And one mud-rotary boring was advanced to 70 feet deep along the south bank of the Mormon Slough to confirm the subsurface condition.

The boring locations were determined by measuring the distance from existing site features by pacing and are accurate only to the degree implied by the method used. The boring locations were not surveyed, and boring coordinates and ground surface elevations were obtained from Google Earth imagery and an existing topographic survey map of the site.

The borings were drilled by Taber Drilling of West Sacramento California using mud-rotary drilling methods. Subsurface encountered in each boring were logged by an HDR engineer with the coordinates and depth for each boring included in the boring logs (Attachment A). T.

Drilling Methods

As previously discussed, Taber Drilling advanced the borings using track-mounted drill rig with mud-rotary drilling methods detailed on the boring logs. The mud-rotary method employs a rotating drill bit that grinds and pulverizes the soil as the bit advances. The drill cuttings are transported up to the ground surface using a circulating drilling fluid composed of water and bentonite, referred to as mud. This method can incorporate the use of casing to help reinforce the drilled borehole and prevent borehole collapse, while facilitating transportation of the cuttings. This method (either cased or open hole), is usually preferred to retrieve undisturbed samples below the ground water table as the drilling fluid (mud) serves to equalize the hydrostatic pressure between the open borehole and the surrounding area reducing the potential for heaving.

Sampling

Samples were collected by driving an 18-inch-long split spoon sampler into the subsurface material using an automatic trip 140-pound hammer with a 30-inch free fall in general accordance with ASTM D1586. After the sampler was driven, it was removed from the borehole and the exploration was advanced to the next sample interval. Samples were collected at 2.5-foot intervals for the first 15-foot depth and every 5-foot thereafter. Soil samplers included 2.5-inch inside diameter Modified California (Mod Cal) split barrel sampler with liners (6-inch-long stainless steel liners), 2-inch inside diameter California (Cal) split barrel sampler with liners (6-inch long stainless steel liners), and a 1.375-inch inside diameter Standard Penetration Test (SPT) split barrel sampler without liners.

The Mod Cal, Cal, and SPT samplers were used in general accordance with ASTM D1586. The type of sampler used at each sampling interval were based on the material encountered or expected to be encountered. Mod Cal and Cal samples were capped to retain moisture while

SPT samples were placed in sealed “Ziploc” bags. Each sample was labeled with the project number, date, sample number, and depth. If more than one sample was taken at each interval, an A/B designation was added with A being the lower sample and B the upper.

An HDR field engineer maintained a boring log and visually classified the soils encountered in the samples and cuttings in accordance with ASTM D2488. The field classifications were updated based on laboratory test results in accordance with ASTM D2487.

Upon completion of drilling, the samples were transported to the HDR Folsom, CA office for further review.

Hammer

Samplers were advanced by dropping a 140-pound, automatic hammer through a 30-inch free fall and the number of blows required to drive the sampler a total of 18 inches, or a shorter distance where the number of blows encountered for a 6-inch interval exceeded 50 (hard resistance). The number of blows were recorded for each 6 inches of penetration. The vertically-stacked blows recorded on the boring logs represent the individual blows for each 6 inches of driving. The blows per foot is the accumulated number of blows that were required to drive the last 12 inches, or the number of inches indicated where hard resistance was encountered. The field blow counts (Nf) are shown for each sample on the boring logs. The Nf has not been corrected for the sampler size.

The hammer energy was not measured at the site during the exploration program. The latest hammer calibration report from Taber Drilling was obtained to be used for the hammer efficiency. The calibration report dated 7/1/2020 indicated an average energy transfer ratio of 76.5 percent.

Drill Cuttings and Restoration

During each boring, drill cuttings were collected and stored in 55-gallon drums and the borings backfilled with neat cement grout in accordance with San Joaquin County well permit requirements, with an approximate ratio of 94 pounds of Portland cement for every 5 gallons of water. Grout inspector from the county was present to observe grouting of all boreholes. All equipment, materials, and trash was removed from the site and the area restored to its prior condition.

Laboratory Testing

Soil samples were selected from the samples obtained during the subsurface investigation for testing and were delivered to Gulf Shores Exploration and Testing of Rancho Cordova, California for geotechnical laboratory testing, which consisted of:

- Moisture Content and Dry Density (ASTM D2937)
- Moisture Content (ASTM D2216)
- Coarse Sieve (ASTM D6913, 3/4” to - #200 Sieve)

- Fine Sieve (ASTM D6913, #4 to #200 Sieve)
- #200 Wash (ASTM D1140)
- Atterberg Limits (ASTM D 4318 Wet Method)
- Triaxial Shear Unconsolidated Undrained (UU; ASTM D2850)

The results of the laboratory tests are presented on the boring logs at the appropriate sample depths (Attachment A).

Subsurface Condition

Four borings on the north bank of the Mormon Slough were advanced to the depth of 70 feet. Subsurface encountered recent overbank deposits consisting of approximately 15 feet of loose to medium stiff silty sands and silt layers overlying alternating layers of loose to medium dense silt, silty sands, and clayey sands to the depth of 35 feet. Coarse grained gravel was encountered at depths of 30 to 40 feet. Below a depth of 40 feet, older alluvial deposits consisting of medium dense to dense silty sands, silt, clay, and clayey sands were encountered to the depth of 70 feet. The borings indicated that the thickness of the gravel layer varies along the north bank and the gravel deposit is not continuous.

One boring on the south bank of the Mormon Slough was advanced to the depth of 70 feet. The boring was advanced by coring through a 9-inch-thick concrete slab overlying a 5-foot-thick clayey gravel layer. Underlaying the gravel layer, hard silty sand was encountered to the depth of 15 feet. Clayey sand, silty sand, and clayey gravel layers were encountered to the depth of 40 feet. Below 40 feet to a depth of 70 feet, older alluvial deposits consisting of medium dense to dense silty sands, silt, clay, and clayey sands were encountered.

The ground water table was encountered at depths ranging from 18 feet to 30 feet. Given the proximity to the river, the groundwater level would probably match closely with the river water level. Piezometers were not installed in the project area as part of this effort.

References

CH2M Hill

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2015 ASTM D2850 “Standard Test Method for Unconsolidated-Undrained Triaxial Compression Test on Cohesive Soils”, ASTM International, West Conshohocken, PA, 2015, www.astm.org.

Attachment A. Supplemental Boring Logs

UNIFIED SOIL CLASSIFICATION SYSTEM (ASTM D-2487)

MATERIAL TYPES	CRITERIA FOR ASSIGNING SOIL GROUP NAMES			GROUP SYMBOL	SOIL GROUP NAMES & LEGEND
COARSE-GRAINED SOILS >50% RETAINED ON NO. 200 SIEVE	GRAVELS >50% OF COARSE FRACTION RETAINED ON NO 4. SIEVE	CLEAN GRAVELS <5% FINES	$C_u \geq 4$ AND $1 \leq C_c \leq 3$	GW	WELL-GRADED GRAVEL
			$C_u < 4$ AND/OR $1 > C_c > 3$	GP	POORLY-GRADED GRAVEL
		GRAVELS WITH FINES >12% FINES	FINES CLASSIFY AS ML OR MH	GM	SILTY GRAVEL
			FINES CLASSIFY AS CL OR CH	GC	CLAYEY GRAVEL
	SANDS >50% OF COARSE FRACTION PASSES NO 4. SIEVE	CLEAN SANDS <5% FINES	$C_u \geq 6$ AND $1 \leq C_c \leq 3$	SW	WELL-GRADED SAND
			$C_u < 6$ AND/OR $1 > C_c > 3$	SP	POORLY-GRADED SAND
		SANDS AND FINES >12% FINES	FINES CLASSIFY AS ML OR MH	SM	SILTY SAND
			FINES CLASSIFY AS CL OR CH	SC	CLAYEY SAND
FINE-GRAINED SOILS >50% PASSES NO. 200 SIEVE	SILTS AND CLAYS LIQUID LIMIT<50	INORGANIC	PI>7 AND PLOTS>"A" LINE	CL	LEAN CLAY
			PI<4 OR PLOTS<"A" LINE	ML	SILT
		ORGANIC	LL (oven dried)/LL (not dried)<0.75	OL	ORGANIC CLAY OR SILT
	SILTS AND CLAYS LIQUID LIMIT>50	INORGANIC	PI PLOTS >"A" LINE	CH	FAT CLAY
			PI PLOTS <"A" LINE	MH	ELASTIC SILT
		ORGANIC	LL (oven dried)/LL (not dried)<0.75	OH	ORGANIC CLAY OR SILT
HIGHLY ORGANIC SOILS		PRIMARILY ORGANIC MATTER, DARK IN COLOR, AND ORGANIC ODOR		PT	PEAT

OTHER SYMBOLS

MATERIALS

- Asphalt
- Aggregate Base
- Topsoil
- Bedrock

PIEZOMETER

- Grout Seal or Fill
- Bentonite Seal or Fill
- Sand Pack + Solid Pipe
- Sand Pack + Slotted Pipe

SAMPLERS

- SPT (2" OD)
- Modified California (3" OD)
- California (2.5" OD)
- Shelby Tube
- Pitcher Barrel
- HQ Core
- Grab/Bulk

- INITIAL WATER LEVEL MEASUREMENT(WITH DATE)
- STABILIZED WATER LEVEL MEASUREMENT(WITH DATE)

GRAIN SIZES

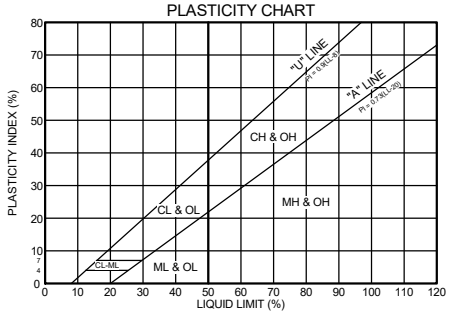
U.S. STANDARD SIEVE	200	40	10	4	3/4"	3"	12"
SILTS AND CLAYS	SAND			GRAVEL		COBBLES	BOULDERS
	FINE	MEDIUM	COARSE	FINE	COARSE		

PENETRATION RESISTANCE

SAND & GRAVEL		SILT & CLAY		
RELATIVE DENSITY	BLOWS/FOOT*	CONSISTENCY	BLOWS/FOOT*	UNC. COMP. STRENGTH (KSF)
VERY LOOSE	0 - 4	VERY SOFT	0 - 1	0 - 1/2
LOOSE	5 - 10	SOFT	2 - 4	1/2 - 1
MEDIUM DENSE	11 - 30	MEDIUM STIFF	5 - 8	1 - 2
DENSE	31 - 50	STIFF	9 - 15	2 - 4
VERY DENSE	OVER 50	VERY STIFF	16 - 30	4 - 8
		HARD	OVER 30	OVER 8

* NUMBER OF BLOWS OF 140 LB HAMMER FALLING 30 INCHES TO DRIVE A 2 INCH O.D. (1-3/8 INCH I.D.) SPLIT-BARREL SAMPLER THE LAST 12 INCHES OF AN 18-INCH DRIVE (ASTM-1586 STANDARD PENETRATION TEST).

PLASTICITY CHART



LABORATORY TESTS

AT	ATTERBERG LIMITS
CD	CONSOLIDATED DRAINED TRIAXIAL
CN	CONSOLIDATION
CR	CORROSIVITY
CU	CONSOLIDATED UNDRAINED TRIAXIAL
DS	DIRECT SHEAR
HY	HYDROMETER
PR	PERMEABILITY
RV	R-VALUE
SA	SIEVE ANALYSIS
TC	CYCLIC TRIAXIAL
UC	UNCONFINED COMPRESSION
UU	UNCONSOLIDATED UNDRAINED TRIAXIAL
-200	% PASSING NO. 200 SIEVE

PROPERTIES

c	COHESION
DD	DRY DENSITY
EI	EXPANSION INDEX
LL	LIQUID LIMIT
MC	MOISTURE CONTENT
N_f	FIELD BLOW COUNT
PI	PLASTICITY INDEX
Su	UNDRAINED STRENGTH

UNDRAINED SHEAR STRENGTH

V	FIELD VANE
P	POCKET PENETROMETER
T	TORVANE
Q	UNCONFINED COMPRESSION
U	UNCONSOLIDATED UNDRAINED TRIAXIAL

INCREASING VISUAL MOISTURE CONTENT

WET
MOIST
DRY

Boring and Test Pit Legend

Bellota Weir Modification Project
Linden, CA

Date
3/8/2021

Figure
A-1

LEGEND_JULY 2016; 10171169_KIEFER_LANDFILL_LEACHATE_PIPELINE.GPJ; HDR_FOLSOM_OAKLAND_MARCH 2017_WIP.GLB; 11/2/20

Project: Bellota Weir Modification Project		Boring ID: HDR-B1	Sheet 1 of 3 Sheets
Project Location: Linden, CA			
Project Number: 10209137			

Start Date:	End Date:	Logged By:	Checked By:	Date Checked:
1/21/2021	1/21/2021	Sagar Raj Satyal	Mark Stanley	04/02/2021

Drilling Company (Rig Type): Taber Drilling (CME 55 Track Mounted)	Inspector: Aaron - San Joaquin County	Weather Conditions: warm/cloudy
Drill Method: Auger / Mud	Drilled By: Toby/Ernesto/Jacob	Elevation Top of Boring: 124.0 ft. Vertical Datum: NAVD88
Drill Bit (Type/Size): Auger / 4" : Coring Bit / 94 mm	Total Depth Drilled: 71.5 ft.	Latitude: 38.052124° Longitude: -121.011166° Horizontal Datum: WGS84
Hammer Type: Automatic	Hole Backfill: Neat Cement Grout	Northing: Easting: Coordinate System:
Hammer Efficiency: 76.5 % / Rod Type: NWJ	Total Number of Samples: 18 Disturbed: 18 Undisturbed: 0	Initial Groundwater Depth: 18 ft Static Groundwater Depth:

ELEV	DEPTH	SAMPLE	Blows/6" or Press.	N_f	LEGEND	DESCRIPTION OF MATERIALS	% REC	Samp No.	Laboratory: Fines	LL	PI	DD	MC	Su (ksf)	REMARKS
						Light gray, 9" concrete.									Started at 10:45 AM
						CLAYEY GRAVEL with Sand (GC): medium dense, brown, moist, max. gravel size up to 1.5", sub-rounded, fine sand, less gravel with depth.		S1							Cored concrete slab with 6" bucket drill Hand Augered to 5 ft
125	5		5 4 6	10		LEAN CLAY with Sand (CL): stiff, brown, moist, 71% fines.	100	S2	71						4" Auger
			6 6 8	14		Approx. 30% sand.	100	S3		31	12		15		
120	10		11 14 19	33		SANDY SILT (ML): hard, grayish brown, moist, 61% non-plastic, fines.	78	S4	61			107	15		
			10 7 9	16			100	S5		NP	NP				
115	15		4 3 3	6		Poorly Graded SAND with Silt (SP-SM): loose, brown, moist.	11	S6							Low recovery, sampling at 17.5 ft. with catcher
			4 6 9	15		Medium dense, brownish gray, moist, 13% non-plastic fines and approx. 10% gravel, gravel up to 1", sub-rounded, fine to medium sand.	100	S7	13	NP	NP		28		Ground water at 18 ft Switched to mud-rotary
110	20		5 8 13	21		Poorly Graded GRAVEL with Clay and Sand (GP-GC): medium dense, gray, wet, 6% fines, 15% sand, sub-angular gravel, up to 1.5".	28	S8	6						Rig chattering
105	25		14 17 10	27		CLAYEY SAND with Gravel (SC): medium dense, brownish gray, wet, sub-angular gravel up to 1", approx. 10% fines, 20% gravel, medium to fine sub-angular sand.	33	S9					17		Cuttings contain sand and gravel up to 1"
100															

DRAFT

Project: Bellota Weir Modification Project	Boring ID: HDR-B1	Sheet 2 of 3 Sheets
Project Location: Linden, CA		
Project Number: 10209137		

ELEV	DEPTH	SAMPLE	Blows/6" or Press.	N_f	LEGEND	DESCRIPTION OF MATERIALS	% REC	Samp No.	Laboratory: Fines	LL	PI	DD	MC	Su (ksf)	REMARKS
			7 9 7	16			45	S10							
95	35		34 22 16	38		Dense, dark gray, medium to fine sand, sub-rounded gravel, up to 1", 31% gravel, 19% fines.	55	S11	19	52	23		28		
90	40		8 13 10	23		LEAN CLAY with Sand (CL): very stiff, light gray, wet, medium plasticity, very stiff, fine sub-rounded sand, approx. 20% sand.	28	S12							
85	45		7 10 11	21		SILTY SAND (SM): medium dense, light grayish brown, wet, medium to fine sand, approx. 30% fines.	100	S13							
80	50		10 12 15	27			100	S14		50	12				
75	55		6 14 13	27		SILT with Sand (ML): very stiff, light gray, wet, approx. 20% sand.	100	S15							
70	60		8 21 36	57		CLAYEY SAND (SC): very dense, gray, wet, medium to fine sand, sub-rounded, approx. 20% fines.	100	S16							
65	65		10 12 19	31		SILT with Sand (ML): hard, light gray, wet, dark gray sand, trace gravel up to 0.5", approx. 20% sand.	100	S17							

DRAFT

HDR SOIL BORING LOG_2017_MARCH_R1; 10209137_BELLOTA_WEIR_EXPLORATIONS-SRS-V2.GPJ; HDR_FOLSOM_OAKLAND_FEB 2018_WIP.GLB; 3/8/21

Project: Bellota Weir Modification Project	Boring ID:	Sheet
Project Location: Linden, CA	HDR-B1	3 of 3 Sheets
Project Number: 10209137		

ELEV	DEPTH	SAMPLE	Blows/6" or Press.	N_f	LEGEND	DESCRIPTION OF MATERIALS	% REC	Samp No.	Laboratory: Fines	LL	PI	DD	MC	Su (ksf)	REMARKS
60	70		14 21 24	45		Poorly Graded SAND (SP): dense, gray, wet.	100	S18							Terminated at 71.5 ft. Grouted with neat cement grout using 10 bags of cement. Completed at 5:30 PM

DRAFT

HDR SOIL BORING LOG_2017_MARCH_R1; 10209137_BELLOTA_WEIR_EXPLORATIONS-SRS-V2.GPJ; HDR_FOLSOM_OAKLAND_FEB 2018_WIP.GLB; 3/8/21

Project: Bellota Weir Modification Project		Boring ID: HDR-B2	Sheet 1 of 3 Sheets
Project Location: Linden, CA			
Project Number: 10209137			
Start Date: 1/19/2021	End Date: 1/19/2021	Logged By: Sagar Raj Satyal	Checked By: Mark Stanley / Date Checked: 04/02/2021
Drilling Company (Rig Type): Taber Drilling (CME 55 Track Mounted)		Inspector: Aaron - San Joaquin County	Weather Conditions: sunny/clear/windy
Drill Method: Auger / Mud		Drilled By: Toby/Ernesto/Jacob	Elevation Top of Boring: 132.0 ft. Vertical Datum: NAVD88
Drill Bit (Type/Size): Auger / 4" : Coring Bit / 94 mm		Total Depth Drilled: 71.5 ft.	Latitude: 38.052257° Longitude: -121.012052° Horizontal Datum: WGS84
Hammer Type: Automatic		Hole Backfill: Neat Cement Grout	Northing: Easting: Coordinate System:
Hammer Efficiency: 76.5 %	Rod Type: NWJ	Total Number of Samples: 18 Disturbed: 18 Undisturbed: 0	Initial Groundwater Depth: 30 ft Static Groundwater Depth:

ELEV	DEPTH	SAMPLE	Blows/6" or Press.	N_f	LEGEND	DESCRIPTION OF MATERIALS	% REC	Samp No.	Laboratory: Fines	LL	PI	DD	MC	Su (ksf)	REMARKS
						SILTY SAND (SM): loose, light brown, moist, fine sand, approx. 30% fines.									Started at 9:40 AM Started with 4" Auger
130			3 2 2	4			89	S1	34						
	5		3 2 3	5			94	S2					8		
125			3 2 3	5		SILT with Sand (ML): medium stiff, dark brown, moist, some roots, approx. 20% sand, fine sand.	100	S3							
	10		6 4 5	9		CLAYEY SAND (SC): loose, dark grayish brown, moist, fine sand, trace gravel up to 0.5", sub-rounded, approx. 30% fines.	83	S4		35	15	90	16	2.50 P	
120			5 4 8	12		Approx. 40% fines.	78	S5						2.50 P	
	15		6 7 10	17		Approx. 30% fines.	67	S6		35	14	97	16		
115															
	20		5 4 5	9		Dark brown, 46% fines.	100	S7	46						
110															
	25		2 2 3	5		Dark gray, sub-rounded sand, approx. 20% fines.	28	S8							Low recovery, gravel in shoe
105															

DRAFT

Project: Bellota Weir Modification Project	Boring ID: HDR-B2	Sheet 2 of 3 Sheets
Project Location: Linden, CA		
Project Number: 10209137		

ELEV	DEPTH	SAMPLE	Blows/6" or Press.	N_f	LEGEND	DESCRIPTION OF MATERIALS	% REC	Samp No.	Laboratory: Fines	LL	PI	DD	MC	Su (ksf)	REMARKS
			2 2 2	4		Dark gray with mottled brown.	100	S9							Ground water at 30 ft Switched to mud-rotary
100	35		25 24 22	46		Well-Graded GRAVEL with Clay and Sand (GW-GC): dense, dark gray, wet, sub-rounded to sub-angular gravel up to 1", sub-rounded sand, 54% gravel and 37% sand.	39	S10	9						Rig chattering
95	40		10 8 7	15		LEAN CLAY with Sand (CL): very stiff, light gray with mottled brown, wet, trace gravel, sub-rounded gravel up to 1", coarse to medium sub-rounded sand.	0	S11							No recovery, sampling at 42.5 ft. after cleaning the hole
90			18 11 8	19			67	S12	82						
85	45		9 6 6	12			0	S13							No recovery, gravel in shoe
80	50		16 10 17	27		CLAYEY SAND (SC): medium dense, light gray, wet, fine sand, sub-rounded, low to medium plasticity fines, approx. 30% fines.	94	S14							
75	55		6 10 12	22		FAT CLAY with Sand (CH): very stiff, light gray, wet, fine sand, sub-rounded, 26% sand.	72	S15	74	71	39				
70	60		9 10 11	21		SILTY SAND (SM): medium dense, light gray, wet, fine sand, sub-rounded, approx. 40% fines.	100	S16							
	65		7 11 13	24		Approx. 20% fines.	39	S17							

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Project: Bellota Weir Modification Project	Boring ID: HDR-B2	Sheet 3 of 3 Sheets
Project Location: Linden, CA		
Project Number: 10209137		

ELEV	DEPTH	SAMPLE	Blows/6" or Press.	N_f	LEGEND	DESCRIPTION OF MATERIALS	% REC	Samp No.	Laboratory: Fines	LL	PI	DD	MC	Su (ksf)	REMARKS
	70		7 13 16	29		SILT with Sand (ML): very stiff, light gray, wet, fine, sub-rounded sand, approx. 20% sand.	39	S18							Terminated at 71.5 ft. Grouted with neat cement grout using 6 bags of cement. Completed at 2:30 PM

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Project: Bellota Weir Modification Project		Boring ID: HDR-B3	Sheet 1 of 3 Sheets
Project Location: Linden, CA			
Project Number: 10209137			

Start Date:	End Date:	Logged By:	Checked By:	Date Checked:
1/18/2021	1/18/2021	Sagar Raj Satyal	Mark Stanley	04/02/2021

Drilling Company (Rig Type): Taber Drilling (CME 55 Track Mounted)	Inspector: Aaron - San Joaquin County	Weather Conditions: sunny/clear/windy
Drill Method: Auger / Mud	Drilled By: Toby/Ernesto/Jacob	Elevation Top of Boring: 135.0 ft. Vertical Datum: NAVD88
Drill Bit (Type/Size): Auger / 4" : Coring Bit / 94 mm	Total Depth Drilled: 71.5 ft.	Latitude: 38.052581° Longitude: -121.011907° Horizontal Datum: WGS84
Hammer Type: Automatic	Hole Backfill: Neat Cement Grout	Northing: Easting: Coordinate System:
Hammer Efficiency: 76.5 % / Rod Type: NWJ	Total Number of Samples: 17 Disturbed: 17 Undisturbed: 0	Initial Groundwater Depth: 25 ft Static Groundwater Depth:

ELEV	DEPTH	SAMPLE	Blows/6" or Press.	N_f	LEGEND	DESCRIPTION OF MATERIALS	% REC	Samp No.	Laboratory: Fines	LL	PI	DD	MC	Su (ksf)	REMARKS
						Aggregate base 2" thick.									Started at 9:00 AM
			3 4 4	8		SILT (ML): loose, light brown, moist, trace gravel up to 0.5", approx. 10% sand.	89	S1							Started with 4" Auger
130	5		4 3 5	8		SILT with Sand (ML): loose, light brown, moist, approx. 20% fine sand, non-plastic fines.	89	S2		NP	NP				
			2 3 2	5		SILT (ML): loose, light brown, moist, trace fine sand.	67	S3							
125	10		6 6 6	12		Trace gravel up to 0.5".	67	S4						4.5+ P	
			5 5 5	10		SILTY SAND (SM): medium dense, light gray, 39% fines.	78	S5	39						
120	15		5 5 5	10		32% fines.	100	S6	32			99	15		
115	20		3 2 3	5		SANDY SILT (ML): loose, dark gray, moist, approx. 30% fine sand.	28	S7							
110	25		1 2 2	4		SILTY SAND (SM): loose, light gray, wet, fine, sub-rounded, sand, 33% fines.	100	S8	33						Ground water at 25 ft Switched to mud-rotary

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Project: Bellota Weir Modification Project	Boring ID: HDR-B3	Sheet 2 of 3 Sheets
Project Location: Linden, CA		
Project Number: 10209137		

ELEV	DEPTH	SAMPLE	Blows/6" or Press.	N_f	LEGEND	DESCRIPTION OF MATERIALS	% REC	Samp No.	Laboratory: Fines	LL	PI	DD	MC	Su (ksf)	REMARKS
			1 2 3	5		Light gray with mottled brown, approx. 40% fines.	83	S9							
100	35		26 20 22	42		CLAYEY GRAVEL with Sand (GC): dense, gray, wet, sub-rounded gravel up to 0.5", approx. 20% fines and 30% sand.	45	S10							Rig chattering
95	40		9 10 10	20		SILTY SAND (SM): medium dense, light brown, wet, sub-rounded sand, approx. 40% fines, trace gravel up to 0.5".	45	S11		49	17				
90	45		8 10 9	19		CLAYEY SAND with Gravel (SC): medium dense, light gray, wet, sub-rounded sand, approx. 30% gravel, up to 0.5", approx. 30% fines.	22	S12							Low recovery, sand catcher added
85	50		6 8 10	18		SILT with Sand (ML): very stiff, light gray, wet, fine sub-rounded sand, approx. 30% sand.	100	S13							
80	55		8 8 12	20		SILT (ML): very stiff, light gray, wet, approx. 10% fine sand.	89	S14		48	17				
75	60		15 20 15	35		SILT with Sand (ML): hard, light gray, wet, approx. 30% sub-rounded sand.	45	S15							
70	65		12 15 12	27		SILTY SAND (SM): medium dense, light gray, wet, fine sand, approx. 40% fines.	89	S16							

DRAFT

HDR SOIL BORING LOG_2017_MARCH_R1; 10209137_BELLOTA_WEIR_EXPLORATIONS-SRS-V2.GPJ; HDR_FOLSOM_OAKLAND_FEB 2018_WIP.GLB; 3/8/21

Project: Bellota Weir Modification Project	Boring ID: HDR-B3	Sheet 3 of 3 Sheets
Project Location: Linden, CA		
Project Number: 10209137		

ELEV	DEPTH	SAMPLE	Blows/6" or Press.	N_f	LEGEND	DESCRIPTION OF MATERIALS	% REC	Samp No.	Laboratory: Fines	LL	PI	DD	MC	Su (ksf)	REMARKS
65	70		14 13 14	27		SANDY SILT (ML): medium dense, light gray, wet, 54% fines.	94	S17	54						Terminated at 71.5 ft. Grouted with neat cement grout using 8 bags of cement. Completed at 3:00 PM. Grouted next morning

DRAFT

HDR SOIL BORING LOG_2017_MARCH_R1; 10209137_BELLOTA_WEIR_EXPLORATIONS-SRS-V2.GPJ; HDR_FOLSOM_OAKLAND_FEB 2018_WIP.GLB; 3/8/21

Project: Bellota Weir Modification Project		Boring ID: HDR-B4	Sheet 1 of 3 Sheets
Project Location: Linden, CA			
Project Number: 10209137			

Start Date:	End Date:	Logged By:	Checked By:	Date Checked:
1/20/2021	1/20/2021	Sagar Raj Satyal	Mark Stanley	04/02/2021

Drilling Company (Rig Type): Taber Drilling (CME 55 Track Mounted)	Inspector: Aaron - San Joaquin County	Weather Conditions: sunny/warm
Drill Method: Auger / Mud	Drilled By: Toby/Ernesto/Jacob	Elevation Top of Boring: 131.0 ft. Vertical Datum: NAVD88
Drill Bit (Type/Size): Auger / 4" : Coring Bit / 94 mm	Total Depth Drilled: 71.5 ft.	Latitude: 38.052946° Longitude: -121.011322° Horizontal Datum: WGS84
Hammer Type: Automatic	Hole Backfill: Neat Cement Grout	Northing: Easting: Coordinate System:
Hammer Efficiency: 76.5 %	Rod Type: NWJ	Total Number of Samples: 18 Disturbed: 18 Undisturbed: 0
		Initial Groundwater Depth: 21 ft Static Groundwater Depth:

ELEV	DEPTH	SAMPLE	Blows/6" or Press.	N_f	LEGEND	DESCRIPTION OF MATERIALS	% REC	Samp No.	Laboratory: Fines	LL	PI	DD	MC	Su (ksf)	REMARKS
130			22 25 25	50		SILTY SAND with Gravel (SM): dense, light brown, moist, approx. 30% fines, and 20% gravel, sub-rounded gravel up to 1", some roots.	83	S1					4		Started at 11:30 AM Started with 4" Auger
125	5		17 16 10	26			22	S2							
			2 2 2	4		SANDY LEAN CLAY (CL): soft, brown, moist, fine sand, trace gravel up to 0.5", sub-rounded, some roots, approx. 30% sand.	83	S3		28	9				
120	10		3 4 5	9		SILTY SAND (SM): loose, brown, moist, sub-rounded fine sand, 41% fines.	67	S4	41			98	14		
			3 4 3	7		SILT with Sand (ML): soft, brown, moist, fine sand, trace gravel up to 0.5", sub-rounded, some roots, approx. 30% sand.	78	S5							
115	15		3 4 4	8		SILTY SAND (SM): loose, brown, moist, sub-rounded fine sand, 28% fines.	67	S6	28						
110	20		3 3 3	6		SILTY CLAYEY SAND (SC-SM): loose, light brownish gray, moist, approx. 40% fines, fine sub-rounded sand.	94	S7		25	4				Ground water at 21 ft Switched to mud-rotary
105	25		5 6 5	11			0	S8							

DRAFT

Project: Bellota Weir Modification Project	Boring ID: HDR-B4	Sheet 2 of 3 Sheets
Project Location: Linden, CA		
Project Number: 10209137		

ELEV	DEPTH	SAMPLE	Blows/6" or Press.	N_f	LEGEND	DESCRIPTION OF MATERIALS	% REC	Samp No.	Laboratory: Fines	LL	PI	DD	MC	Su (ksf)	REMARKS
100			5 4 8	12		SILTY SAND (SM): medium dense, brownish gray, wet, approx. 30% fines, medium to fine sand, trace, sub-rounded gravel up to 0.5".	72	S9							
	35		15 22 14	36		Poorly Graded GRAVEL with Clay and Sand (GP-GC): dense, dark gray, wet, sub-rounded gravel up to 1.5", 73% gravel, 20% sand.	45	S10	7						
95															
	40		11 7 7	14		SILT with Sand (ML): stiff, brownish gray, wet, approx. 20% fine sand.	72	S11							
90															
	45		14 18 18	36		Hard, light gray.	33	S12							Low recovery, sampling at 47.5 ft
85			19 15 19	34		Medium to coarse white sand, trace gravel up to 1".	33	S13							
	50		12 9 14	23		Fine sand, no gravel.	100	S14						0.87 U	
80															
	55		11 13 14	27		Coarser sand, approx. 20% sand.	33	S15							
75															
	60		11 20 22	42		Fine sand, approx. 20% sand.	67	S16						0.62 U	Sample S16 B highly disturbed
70															
	65		16 25 19	44		SILTY SAND (SM): dense, gray, wet, medium to fine sand, sub-angular to sub-rounded, 40% fines.	61	S17	40						
65															

DRAFT

HDR SOIL BORING LOG_2017_MARCH_R1; 10209137_BELLOTA_WEIR_EXPLORATIONS-SRS-V2.GPJ; HDR_FOLSOM_OAKLAND_FEB 2018_WIP.GLB; 3/8/21

Project:	Bellota Weir Modification Project	Boring ID:	Sheet
Project Location:	Linden, CA	HDR-B4	3 of 3 Sheets
Project Number:	10209137		

ELEV	DEPTH	SAMPLE	Blows/6" or Press.	N_f	LEGEND	DESCRIPTION OF MATERIALS	% REC	Samp No.	Laboratory: Fines	LL	PI	DD	MC	Su (ksf)	REMARKS
60	70		9 18 30	48		SANDY SILT (ML): hard, dark brown, wet, approx. 30% sand, fine sand, trace gravel, sub-angular, up to 1".	100	S18							Terminated at 71.5 ft. Grouted with neat cement grout using 12 bags of cement. Completed at 4:40 PM. Grouted next morning

DRAFT

HDR SOIL BORING LOG_2017_MARCH_R1; 10209137_BELLOTA_WEIR_EXPLORATIONS-SRS-V2.GPJ; HDR_FOLSOM_OAKLAND_FEB 2018_WIP.GLB; 3/8/21

Project: Bellota Weir Modification Project			Boring ID: HDR-B5	Sheet 1 of 3 Sheets
Project Location: Linden, CA				
Project Number: 10209137				
Start Date: 1/19/2021	End Date: 1/21/2021	Logged By: Sagar Raj Satyal	Checked By: Mark Stanley	Date Checked: 04/02/2021
Drilling Company (Rig Type): Taber Drilling (CME 55 Track Mounted)		Inspector: Aaron - San Joaquin County	Weather Conditions: sunny/warm/windy	
Drill Method: Auger / Mud		Drilled By: Toby/Ernesto/Jacob	Elevation Top of Boring: 131.0 ft. Vertical Datum: NAVD88	
Drill Bit (Type/Size): Auger / 4" : Coring Bit / 94 mm		Total Depth Drilled: 71.5 ft.	Latitude: 38.053055° Longitude: -121.011818° Horizontal Datum: WGS84	
Hammer Type: Automatic		Hole Backfill: Neat Cement Grout	Northing: Easting: Coordinate System:	
Hammer Efficiency: 76.5 %	Rod Type: NWJ	Total Number of Samples: 18 Disturbed: 18 Undisturbed: 0	Initial Groundwater Depth: 25 ft Static Groundwater Depth:	

ELEV	DEPTH	SAMPLE	Blows/6" or Press.	N_f	LEGEND	DESCRIPTION OF MATERIALS	% REC	Samp No.	Laboratory: Fines	LL	PI	DD	MC	Su (ksf)	REMARKS
			2 2 3	5		SILTY SAND (SM): loose, light brown, moist, fine sand, approx. 20% non-plastic fines, trace gravel, sub-rounded, up to 0.5", some roots.	78	S1							Started at 3:00 PM Started with 4" Auger
130	5		4 2 3	5			78	S2		NP	NP				
			2 2 2	4			72	S3							
125	10		3 2 3	5		Increased fines, approx. 20%.	83	S4							
			3 3 5	8		SILT with Sand (ML): medium stiff, brown, moist, approx. 20% fine sand, sub-rounded sand.	100	S5							
120	15		4 7 9	16			94	S6							
115	20		3 4 3	7		LEAN CLAY with Sand (CL): medium stiff, brownish gray, wet, approx. 20% fine sand, sub-rounded sand.	100	S7		31	10	89	32		
110	25		4 3 4	7		SANDY SILT (ML): medium stiff, dark gray, wet, approx. 40% fine sand.	94	S8							Ground water at 25 ft Switched to mud-rotary

DRAFT

Project: Bellota Weir Modification Project	Boring ID: HDR-B5	Sheet 2 of 3 Sheets
Project Location: Linden, CA		
Project Number: 10209137		

ELEV	DEPTH	SAMPLE	Blows/6" or Press.	N_f	LEGEND	DESCRIPTION OF MATERIALS	% REC	Samp No.	Laboratory: Fines	LL	PI	DD	MC	Su (ksf)	REMARKS
			8 15 23	38		CLAYEY GRAVEL with Sand (GC): dense, dark gray, wet, sub-rounded gravel, up to 1", approx. 50% gravel and 20% fines.	33	S9							Rig chattering
100	35		15 9 12	21		SILTY SAND with Gravel (SM): dense, grayish brown, wet, approx. 50% sand, 30% fines, and 20% gravel, sub-rounded gravel up to 1".	67	S10							Stopped for the day Started at 7:30 AM 1/20/2021
95	40		16 9 14	23		SILT with Sand (ML): very stiff, light brownish gray, wet, 31% sand, trace gravel up to 0.5".	67	S11	68			72	59		
90	45		12 18 28	46		SANDY SILT (ML): hard, dark brownish gray, wet, approx. 30% fine sand.	33	S12							1.5" gravel at the shoe, low recovery, highly disturbed sample
85	50		9 9 10	19		SILTY SAND (SM): medium dense, light brownish gray, wet, approx. 40% fines, trace gravel, up to 1.5".	0	S13							
			9 11 8	19		43% fines.	67	S14	43			90	39		
80	55		8 9 13	22		SILT with Sand (ML): very stiff, light brownish gray, wet, approx. 20% fine sand, trace gravel up to 0.5".	100	S15							
75	60		9 6 17	23		SILTY SAND (SM): medium dense, light gray, wet, approx. 20% fines.	100	S16							
70	65		8 12 15	27			100	S17							

DRAFT

Project: Bellota Weir Modification Project	Boring ID: HDR-B5	Sheet 3 of 3 Sheets
Project Location: Linden, CA		
Project Number: 10209137		

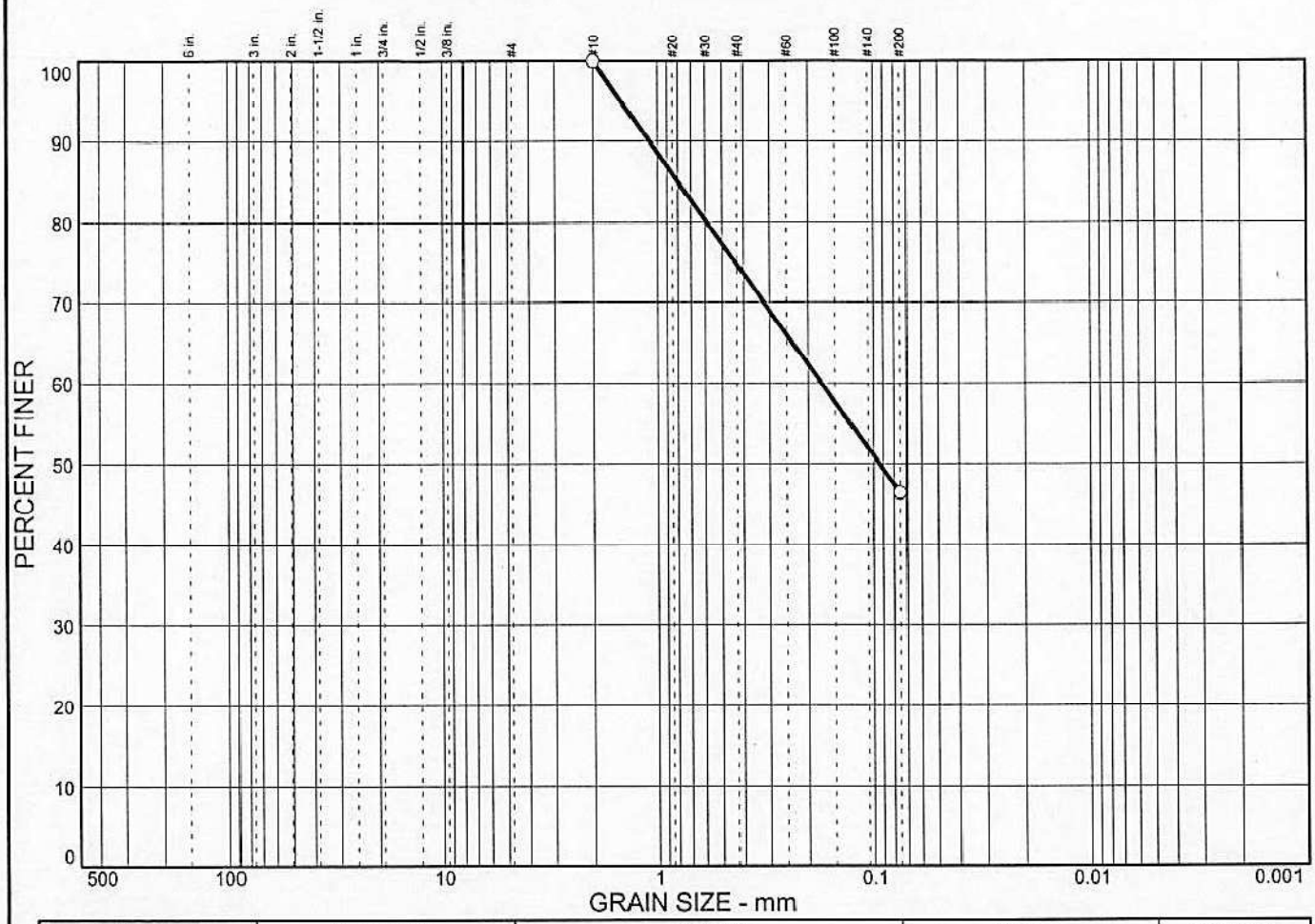
ELEV	DEPTH	SAMPLE	Blows/6" or Press.	N_f	LEGEND	DESCRIPTION OF MATERIALS	% REC	Samp No.	Laboratory: Fines	LL	PI	DD	MC	Su (ksf)	REMARKS
65	70		13 19 30	49		CLAYEY SAND (SC): dense, light gray, wet, approx. 15% fines, trace gravel up to 0.5".	100	S18							Terminated at 71.5 ft. Grouted with neat cement grout using 11 bags of cement. Copleted at 10:50 AM

DRAFT

Appendix D. Laboratory Testing

Attachment A-2
Laboratory Test Results

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	0.0	53.7	46.3	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#10	100.0		
#200	46.3		

* (no specification provided)

Material Description

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= 0.799 D_{60}= 0.173 D_{50}= 0.0940
D_{30}= D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= AASHTO=

Remarks

Total dry weight tested= 480 grams.

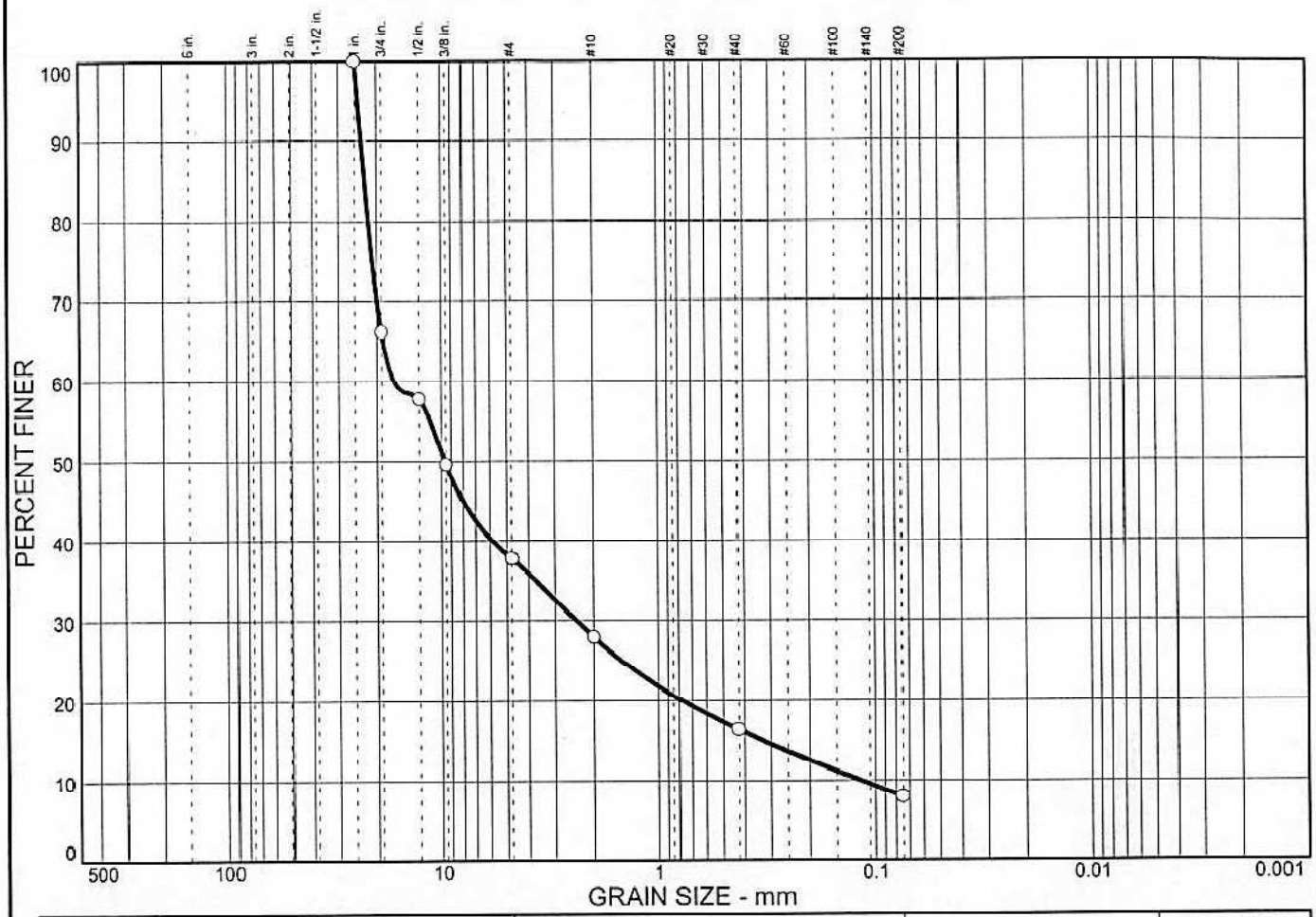
Sample No.: BI-1 **Source of Sample:** **Date:** 1-27-05
Location: **Elev./Depth:** 10'

Taber Since 1954

Client: CH2M Hill
Project: Bellota Weir / #175404.AA.P2.03

Project No: 2T2/305/19 **Figure:** 1

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	62.2	29.8	8.0	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1 in.	100.0		
0.75 in.	66.1		
0.5 in.	57.8		
0.375 in.	49.6		
#4	37.8		
#10	28.0		
#40	16.4		
#200	8.0		

* (no specification provided)

Material Description

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= 22.8 D_{60}= 16.6 D_{50}= 9.65
D_{30}= 2.40 D_{15}= 0.328 D_{10}= 0.117
C_u= 142.50 C_c= 2.97

Classification

USCS= AASHTO=

Remarks

Total dry weight tested= 463 grams.

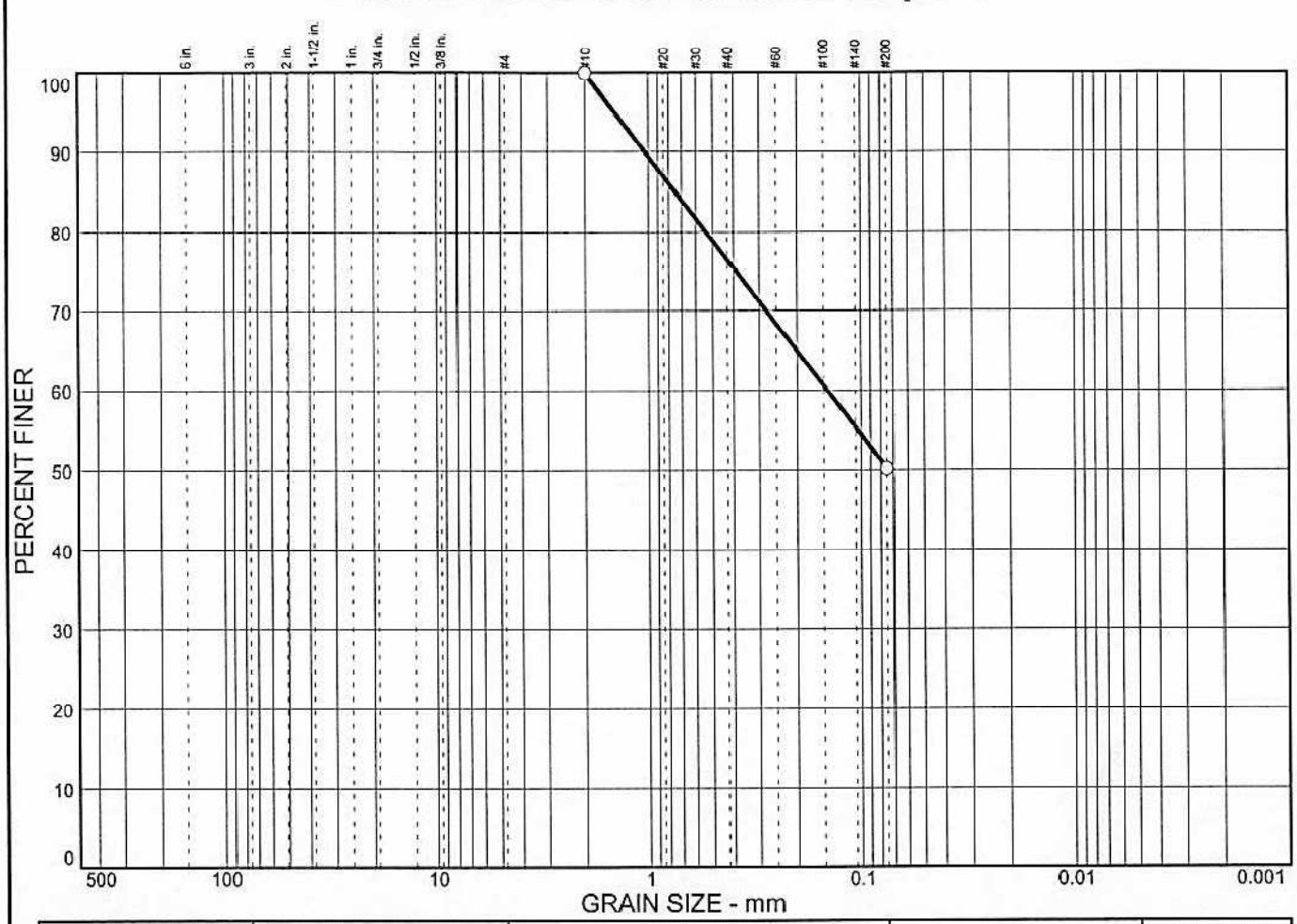
Sample No.: BI-1 **Source of Sample:** **Date:** 1-27-05
Location: **Elev./Depth:** 25'

Taber Since 1954

Client: CH2M Hill
Project: Bellota Weir / #175404.AA.P2.03

Project No: 2T2/305/19 **Figure:** 1

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	0.0	49.9	50.1	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#10	100.0		
#200	50.1		

* (no specification provided)

Material Description

Sandy silt

Atterberg Limits

PL= 42 LL= 45 PI= 3

Coefficients

D_{85}= 0.745 D_{60}= 0.144 D_{50}=
D_{30}= D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= ML AASHTO= A-5(1)

Remarks

Total dry weight tested= 491 grams.

Sample No.: BI-1 **Source of Sample:** **Date:** 1-27-05
Location: **Elev./Depth:** 35'

Taber
Since 1954

Client: CH2M Hill
Project: Bellota Weir / #175404.AA.P2.03

Project No: 2T2/305/19 **Figure:** 1

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	53.3	42.0	4.7	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
0.75 in.	100.0		
0.5 in.	78.2		
0.375 in.	64.9		
#4	46.7		
#10	33.3		
#40	15.0		
#200	4.7		

* (no specification provided)

Material Description

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= 14.5 D_{60}= 8.32 D_{50}= 5.65
D_{30}= 1.57 D_{15}= 0.425 D_{10}= 0.215
C_u= 38.68 C_c= 1.38

Classification

USCS= AASHTO=

Remarks

Total Dry weight tested= 493 grams.

Sample No.: BI-2 **Source of Sample:** **Date:** 1-27-05
Location: **Elev./Depth:** 5'

Taber Since 1954

Client: CH2M Hill
Project: Bellota Weir / #175404.AA.P2.03

Project No: 2T2/305/19 **Figure:** 1

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	2.9	55.3	41.8	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
0.375 in.	100.0		
#4	97.1		
#10	95.2		
#40	82.8		
#200	41.8		

* (no specification provided)

Material Description

Silty sand

Atterberg Limits

PL= NP LL= 24 PI= NP

Coefficients

D_{85}= 0.492 D_{60}= 0.147 D_{50}= 0.101
D_{30}= D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= SM AASHTO= A-4(0)

Remarks

Total dry weight tested= 532 grams.

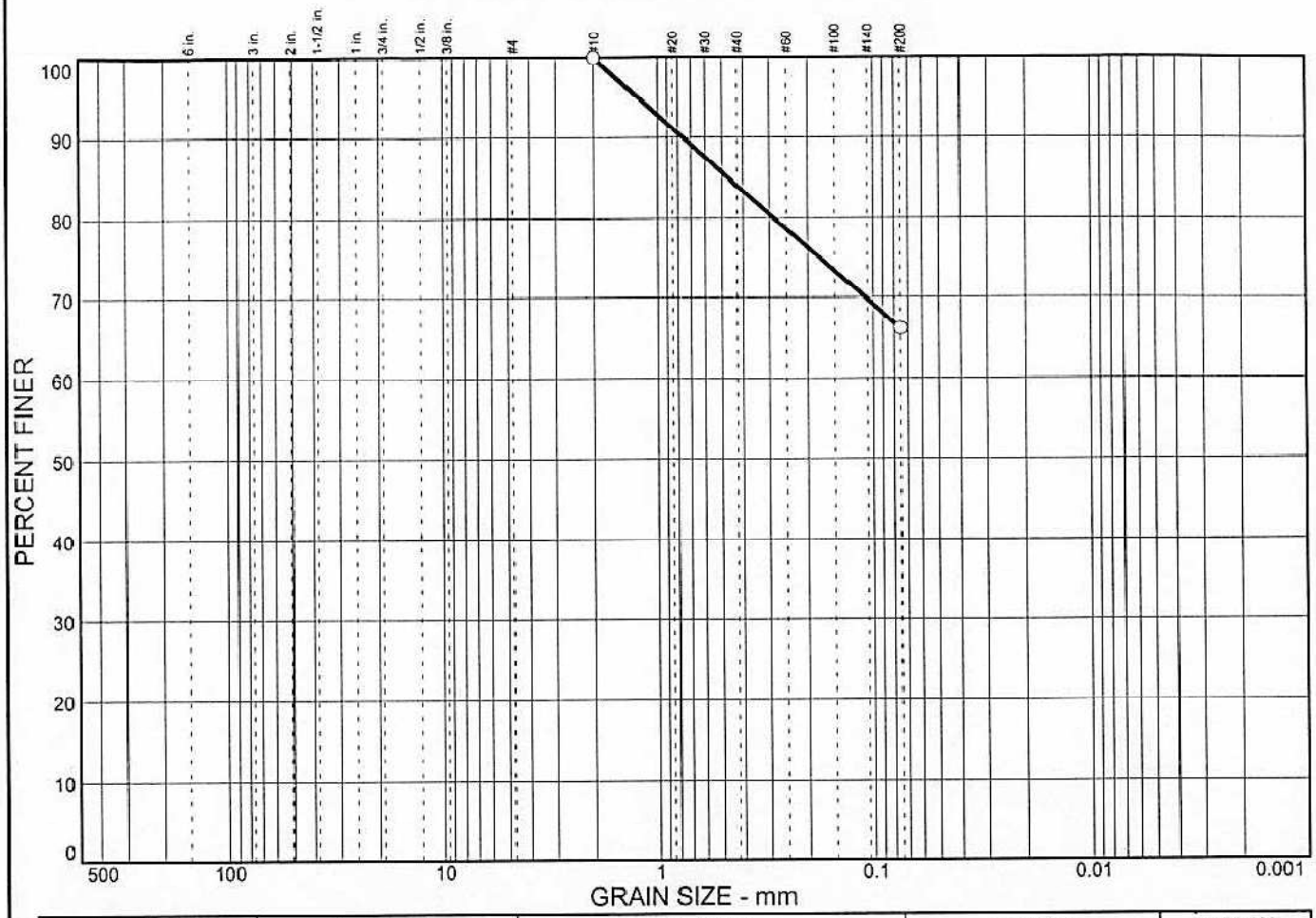
Sample No.: BI-2 **Source of Sample:** **Date:** 1-27-05
Location: **Elev./Depth:** 10'

Taber Since 1954

Client: CH2M Hill
Project: Bellota Weir / #175404.AA.P2.03

Project No: 2T2/305/19 **Figure:** 1

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	0.0	33.8	66.2	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#10	100.0		
#200	66.2		

* (no specification provided)

Material Description

Sandy elastic silt

Atterberg Limits

PL= 39 LL= 50 PI= 11

Coefficients

D_{85}= 0.466 D_{60}= D_{50}=
D_{30}= D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= MH AASHTO= A-7-5(8)

Remarks

Total dry weight tested= 406 grams.

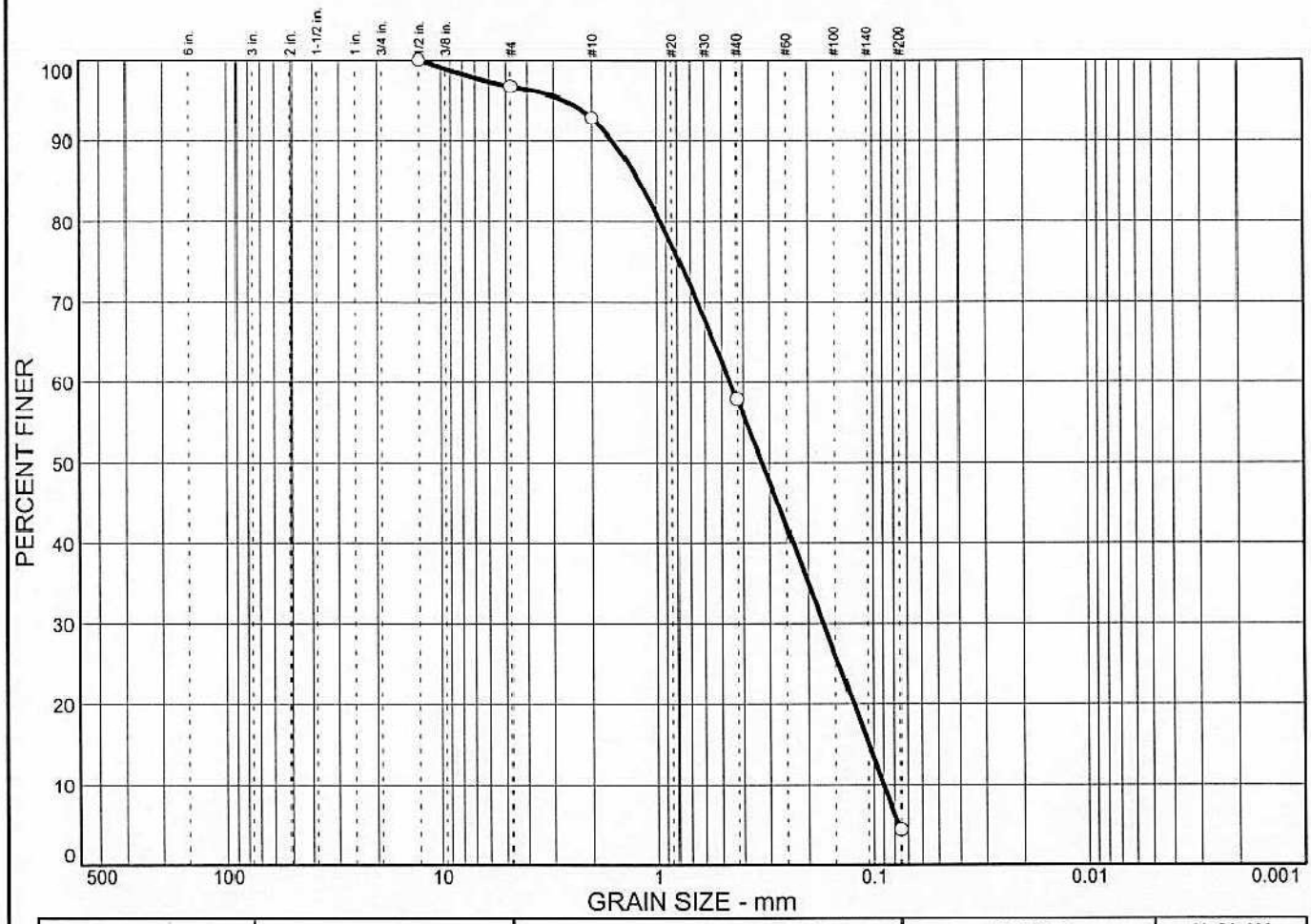
Sample No.: BI-2 **Source of Sample:** **Date:** 1-27-05
Location: **Elev./Depth:** 25'

Taber Since 1954

Client: CH2M Hill
Project: Bellota Weir / #175404.AA.P2.03

Project No: 2T2/305/19 **Figure:** 1

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	3.3	92.4	4.3	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
0.5 in.	100.0		
#4	96.7		
#10	92.8		
#40	57.9		
#200	4.3		

* (no specification provided)

Material Description

Poorly graded sand

Atterberg Limits

PL= NP LL= 31 PI= NP

Coefficients

D_{85}= 1.21 D_{60}= 0.456 D_{50}= 0.326
D_{30}= 0.170 D_{15}= 0.105 D_{10}= 0.0898
C_u= 5.08 C_c= 0.70

Classification

USCS= SP AASHTO= A-3

Remarks

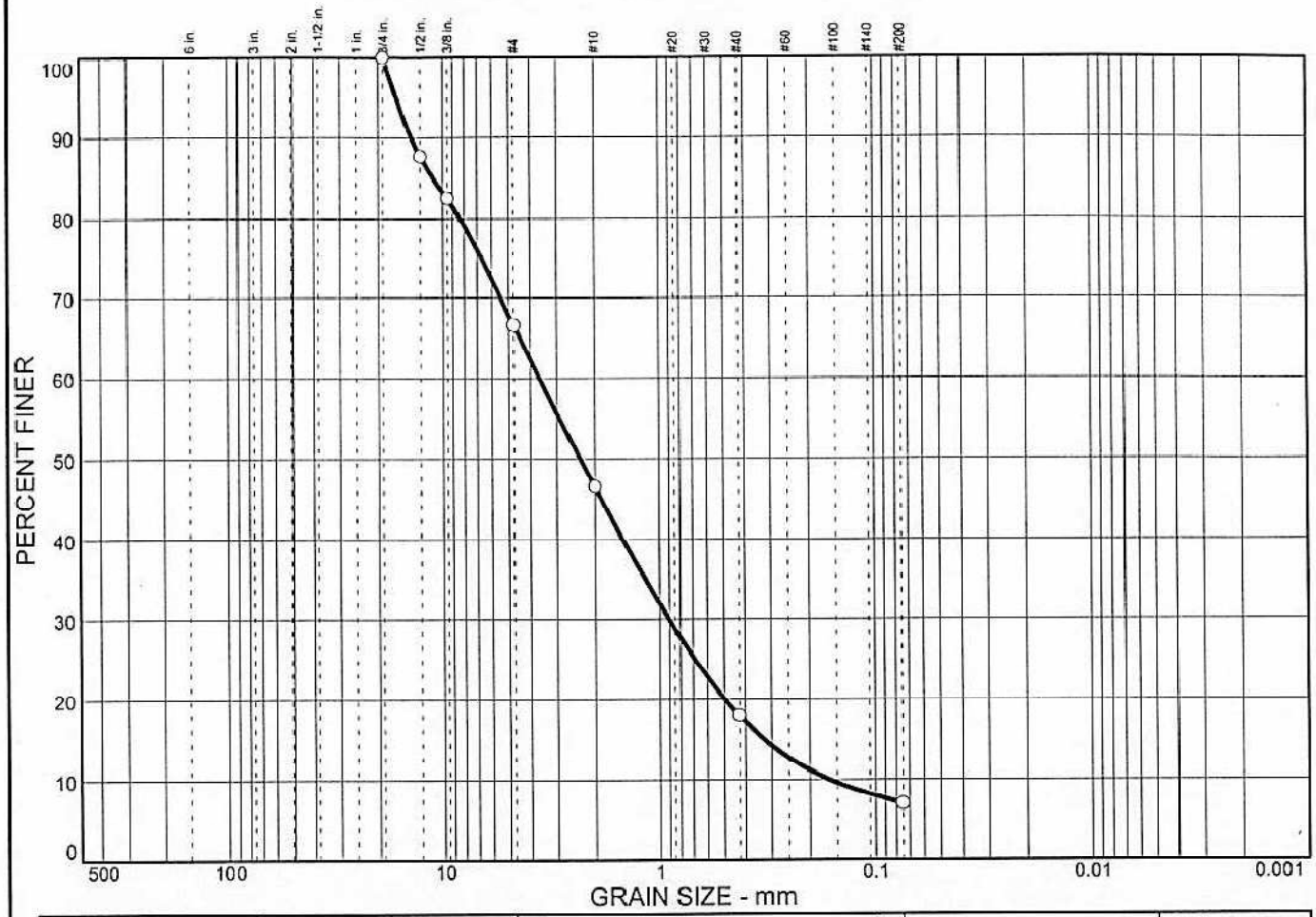
Total dry weight tested= 654 grams.

Sample No.: BI-3 **Source of Sample:** **Date:** 1-27-05
Location: **Elev./Depth:** 7.5'

Taber Since 1954

Client: CH2M Hill
Project: Bellota Weir / #175404.AA.P2.03
Project No: 2T2/305/19 **Figure:** 1

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	33.4	59.6	7.0	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
0.75 in.	100.0		
0.5 in.	87.7		
0.375 in.	82.5		
#4	66.6		
#10	46.5		
#40	18.0		
#200	7.0		

* (no specification provided)

Material Description

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= 11.0 D_{60}= 3.62 D_{50}= 2.34
D_{30}= 0.903 D_{15}= 0.324 D_{10}= 0.162
C_u= 22.28 C_c= 1.39

Classification

USCS= AASHTO=

Remarks

Total dry weight tested= 759 grams.

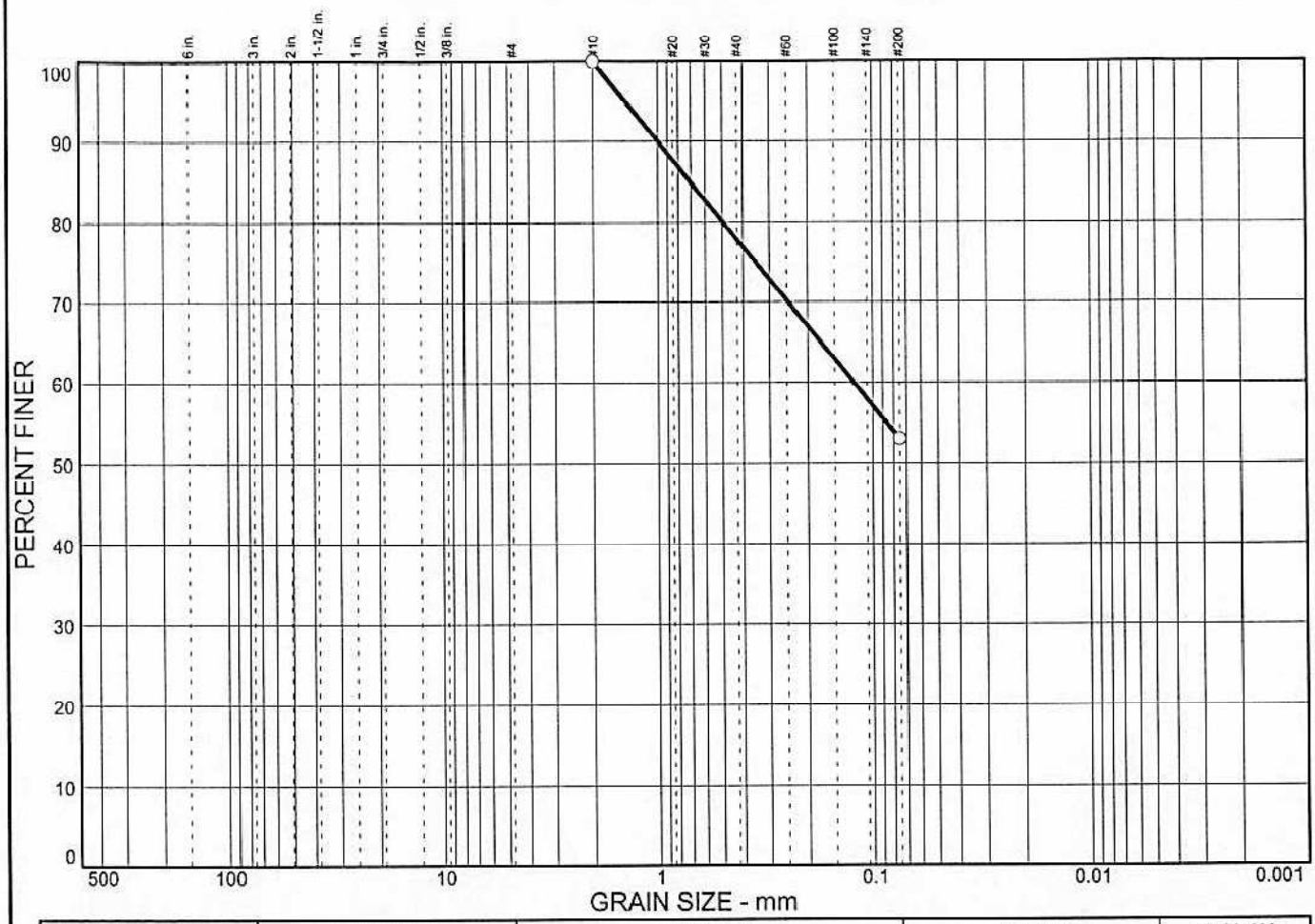
Sample No.: BI-3 **Source of Sample:** **Date:** 1-27-05
Location: **Elev./Depth:** 17.5'

Taber Since 1954

Client: CH2M Hill
Project: Bellota Weir / #175404.AA.P2.03

Project No: 2T2/305/19 **Figure:** 1

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	0.0	47.0	53.0	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#10	100.0		
#200	53.0		

* (no specification provided)

Material Description

Sandy silt

Atterberg Limits

PL= 40 LL= 42 PI= 2

Coefficients

D_{85}= 0.701 D_{60}= 0.122 D_{50}=
D_{30}= D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= ML AASHTO= A-5(1)

Remarks

Total dry weight tested= 532 grams.

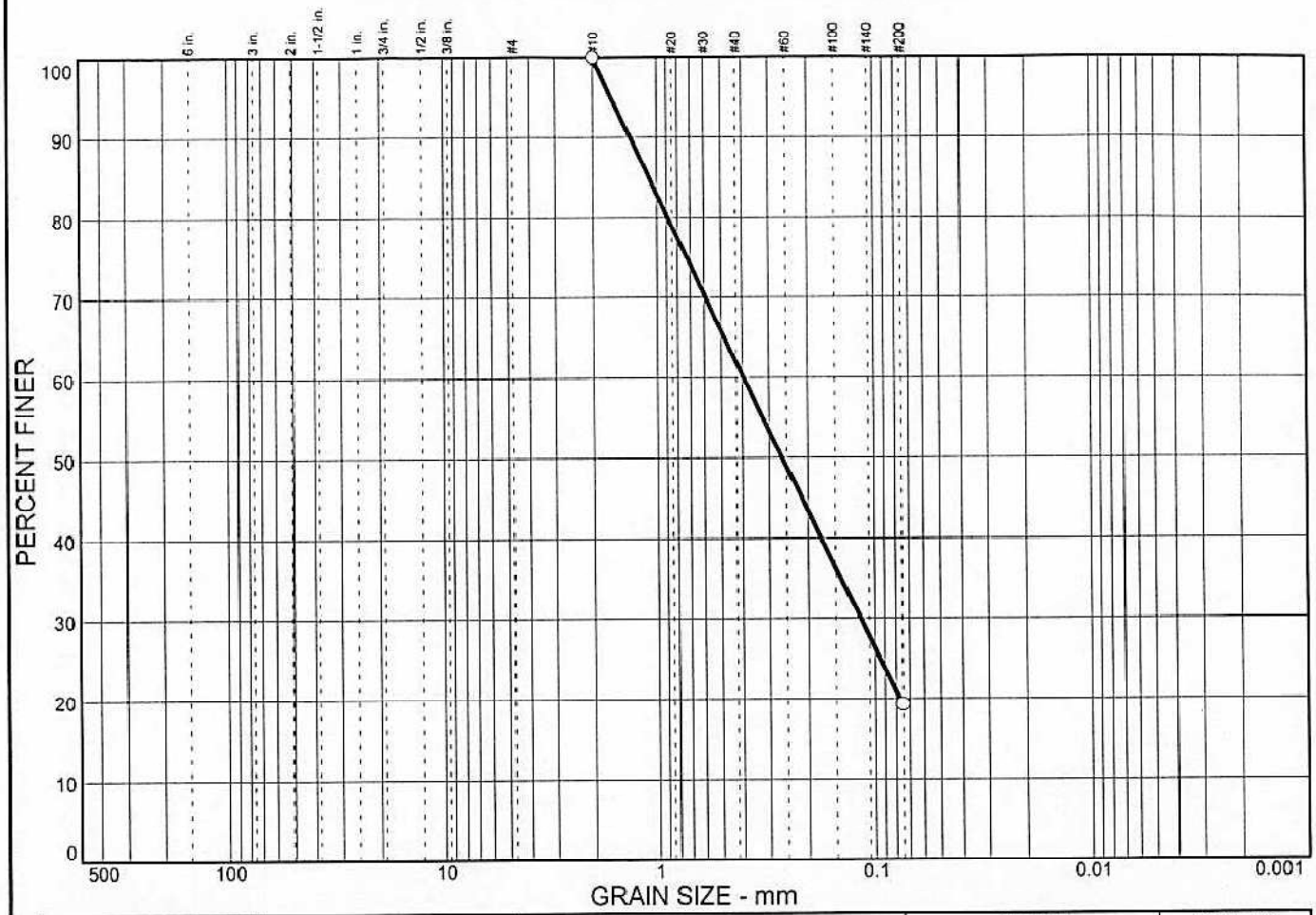
Sample No.: BI-3 **Source of Sample:** **Date:** 1-27-05
Location: **Elev./Depth:** 28.5'

Taber Since 1954

Client: CH2M Hill
Project: Bellota Weir / #175404.AA.P2.03

Project No: 2T2/305/19 **Figure:** 1

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	0.0	80.7	19.3	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#10	100.0		
#200	19.3		

* (no specification provided)

Material Description

Silty sand

Atterberg Limits

PL= NP LL= 25 PI= NP

Coefficients

D_{85}= 1.09 D_{60}= 0.393 D_{50}= 0.262
D_{30}= 0.116 D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= SM AASHTO= A-2-4(0)

Remarks

Total dry weight tested= 1042 grams.

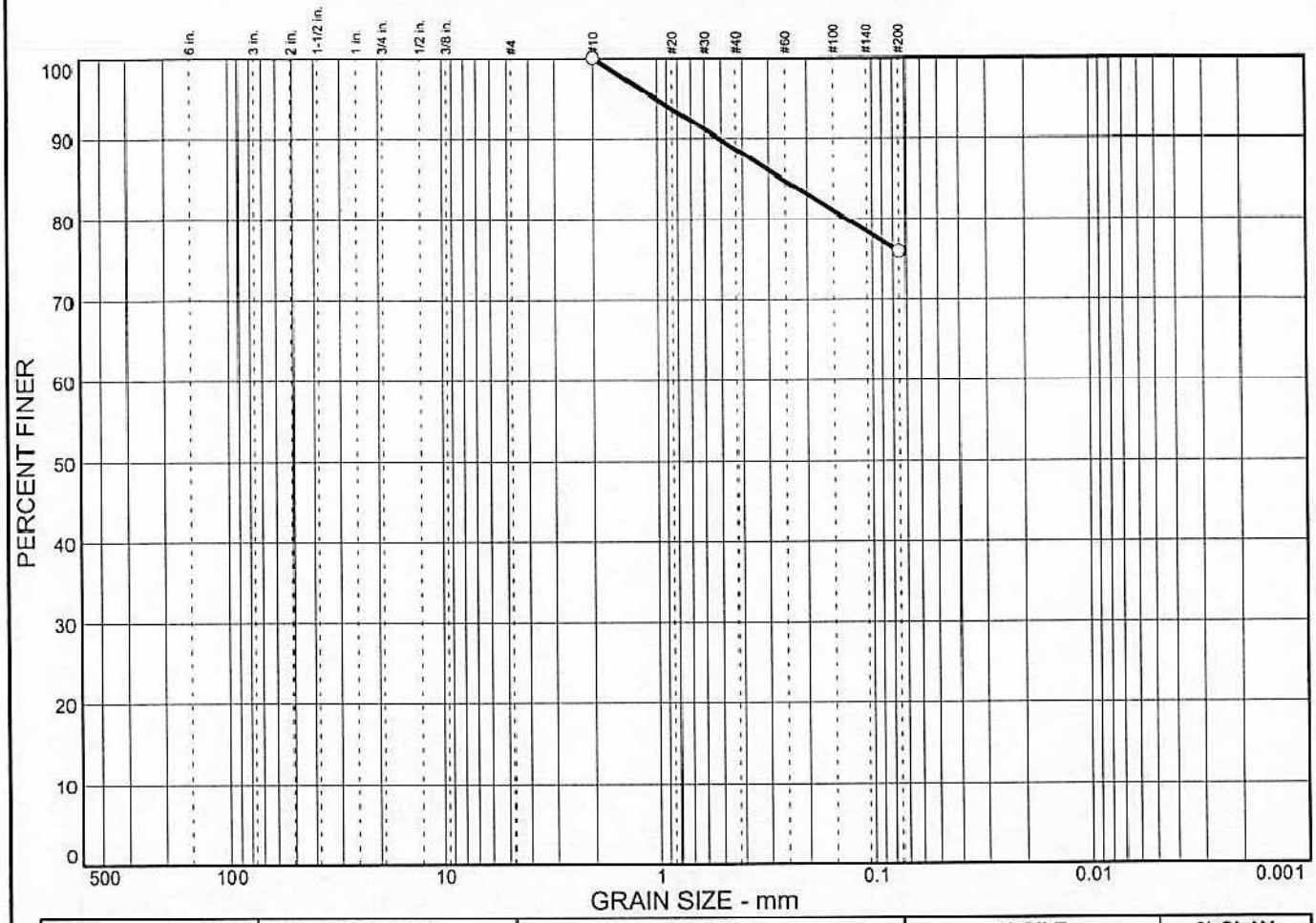
Sample No.: BF-1 (Shelby) **Source of Sample:** **Date:** 1-27-05
Location: **Elev./Depth:** 15'

Taber Since 1954

Client: CH2M Hill
Project: Bellota Weir / #175404.AA.P2.03

Project No: 2T2/305/19 **Figure:** 1

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	0.0	24.1	75.9	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#10	100.0		
#200	75.9		

* (no specification provided)

Material Description

Fat clay with sand

Atterberg Limits

PL= 26 LL= 59 PI= 33

Coefficients

D_{85}= 0.259 D_{60}= D_{50}=
D_{30}= D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= CH AASHTO= A-7-6(26)

Remarks

Total dry weight tested= 292 grams.

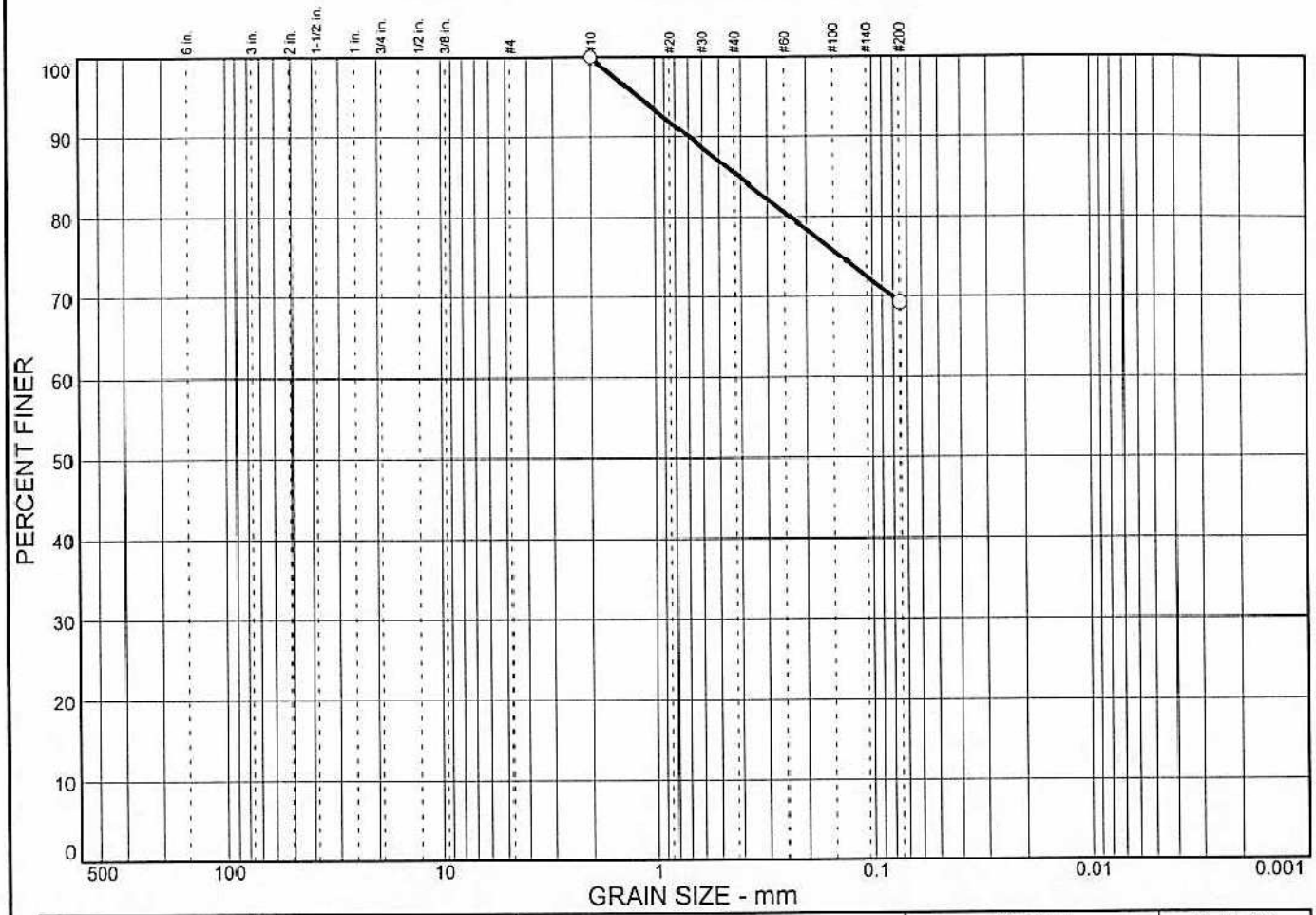
Sample No.: BF-1 **Source of Sample:** **Date:** 1-27-05
Location: **Elev./Depth:** 40'

Taber
Since 1954

Client: CH2M Hill
Project: Bellota Weir / #175404.AA.P2.03

Project No: 2T2/305/19 **Figure:** 1

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	0.0	30.7	69.3	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#10	100.0		
#200	69.3		

* (no specification provided)

Material Description

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= 0.402 D_{60}= D_{50}=
D_{30}= D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= AASHTO=

Remarks

Total dry weight tested= 444 grams.

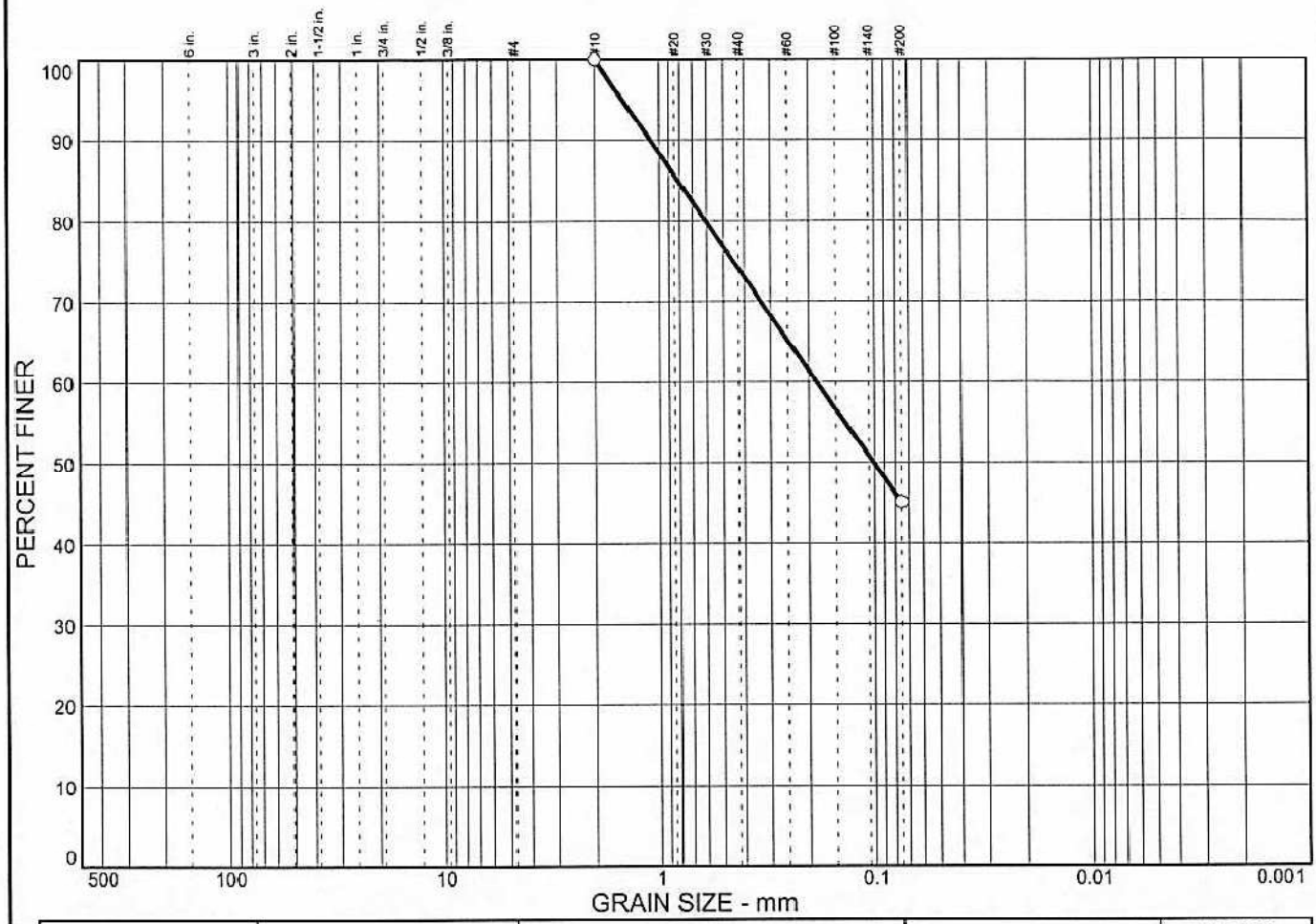
Sample No.: BF-1 **Source of Sample:** **Date:** 1-27-05
Location: **Elev./Depth:** 55'

Taber Since 1954

Client: CH2M Hill
Project: Bellota Weir / #175404.AA.P2.03

Project No: 2T2/305/19 **Figure:** 1

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	0.0	55.0	45.0	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#10	100.0		
#200	45.0		

* (no specification provided)

Material Description

Silty, clayey sand

Atterberg Limits

PL= 21 LL= 25 PI= 4

Coefficients

D_{85}= 0.817 D_{60}= 0.184 D_{50}= 0.101
D_{30}= D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= SC-SM AASHTO= A-4(0)

Remarks

Total dry weight tested= 807 grams.

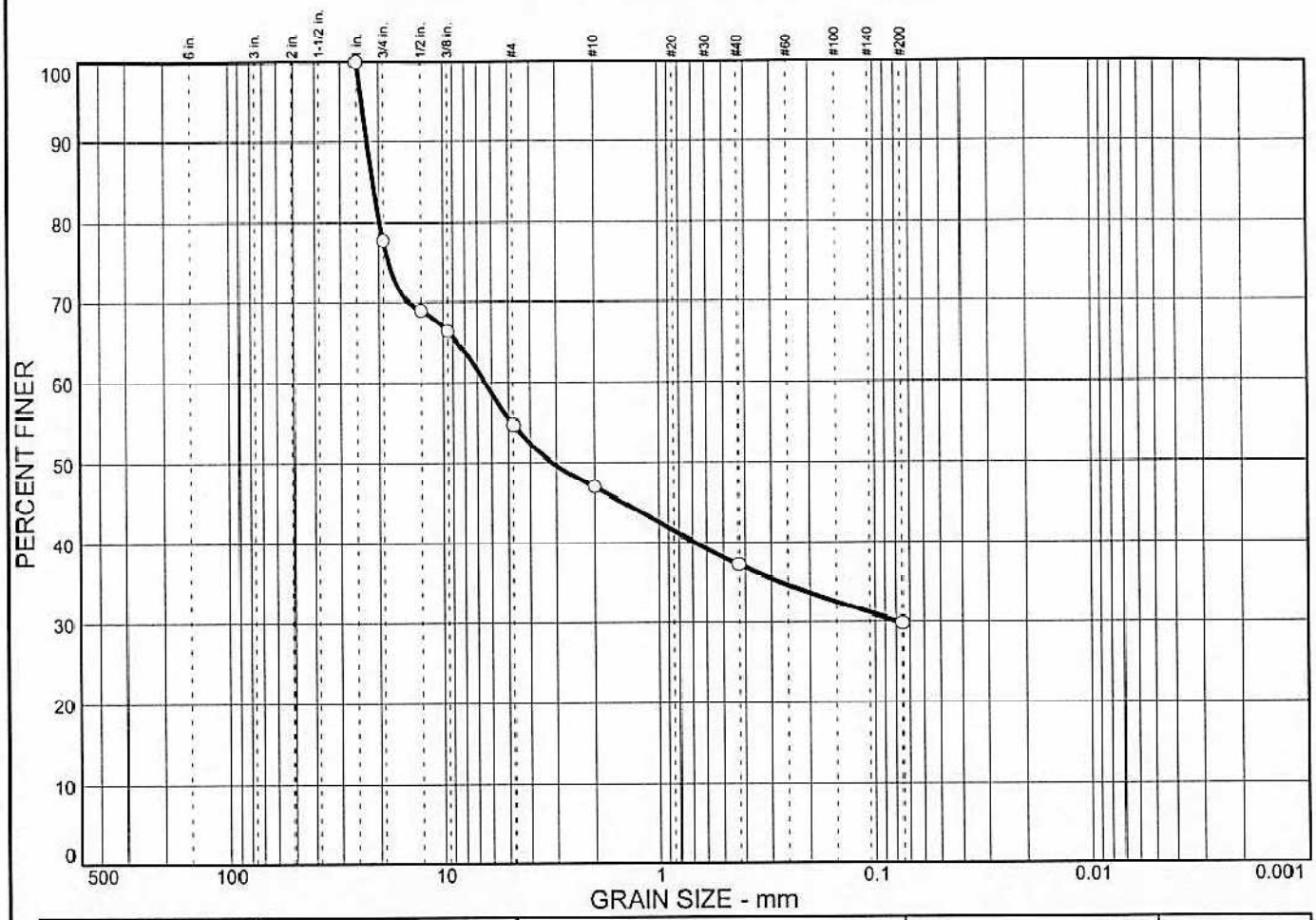
Sample No.: BD-2 **Source of Sample:** **Date:** 1-27-05
Location: **Elev./Depth:** 5'

Taber
Since 1954

Client: CH2M Hill
Project: Bellota Weir / #175404.AA.P2.03

Project No: 2T2/305/19 **Figure:** 1

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	45.3	24.9	29.8	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1 in.	100.0		
0.75 in.	77.7		
0.5 in.	69.0		
0.375 in.	66.4		
#4	54.7		
#10	46.9		
#40	37.2		
#200	29.8		

* (no specification provided)

Material Description

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= 21.3 D_{60}= 6.44 D_{50}= 3.15
D_{30}= 0.0792 D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= AASHTO=

Remarks

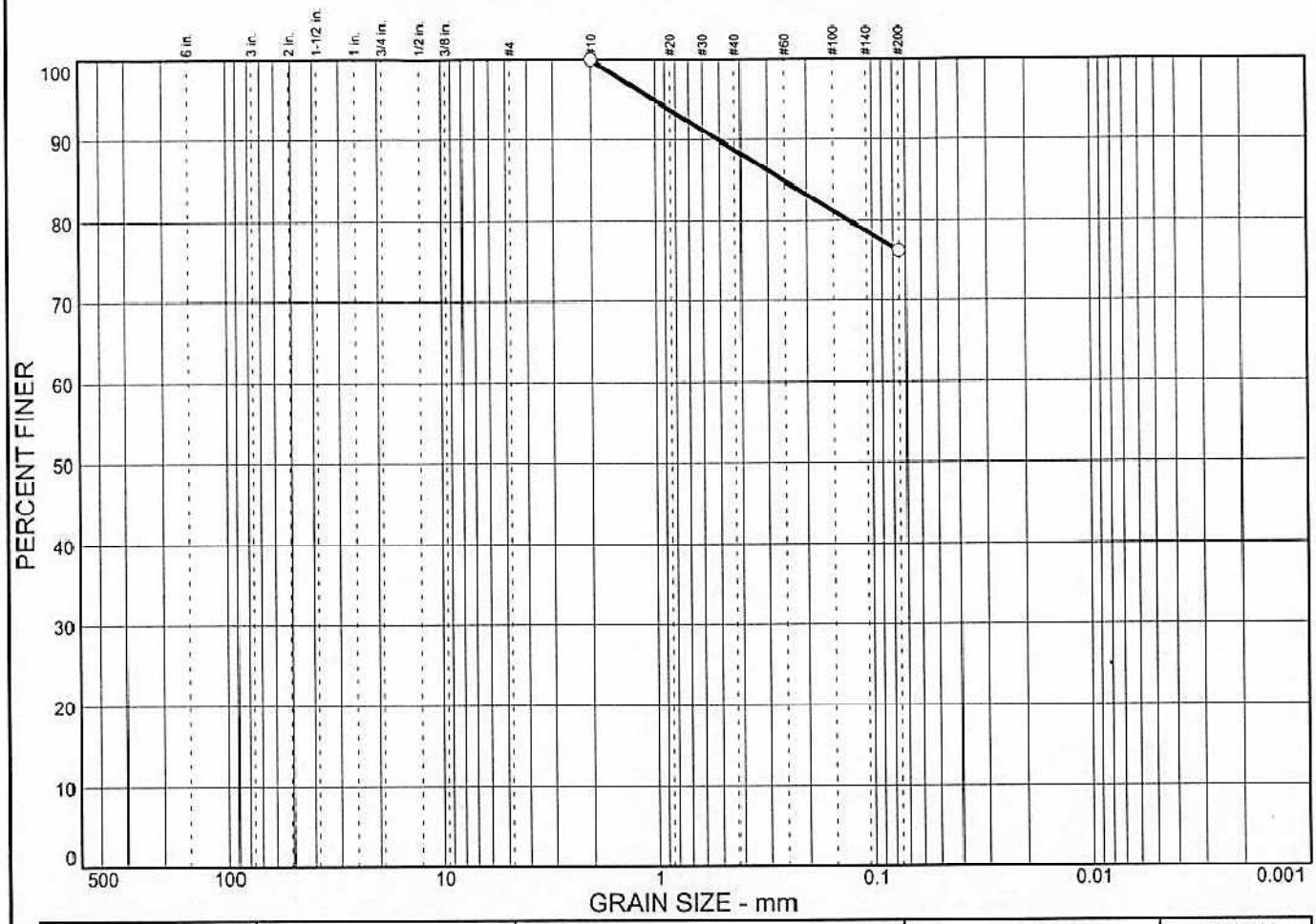
Total dry weight tested= 412 grams.

Sample No.: BD-2 **Source of Sample:** **Date:** 1-27-05
Location: **Elev./Depth:** 15'

Taber
Since 1954

Client: CH2M Hill
Project: Bellota Weir / #175404.AA.P2.03
Project No: 2T2/305/19 **Figure:** 1

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	0.0	23.8	76.2	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#10	100.0		
#200	76.2		

* (no specification provided)

Material Description

Elastic silt with sand

Atterberg Limits

PL= 41 LL= 57 PI= 16

Coefficients

D_{85}= 0.253 D_{60}= D_{50}=
D_{30}= D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= MH AASHTO= A-7-5(15)

Remarks

Total dry weight tested= 318 grams.

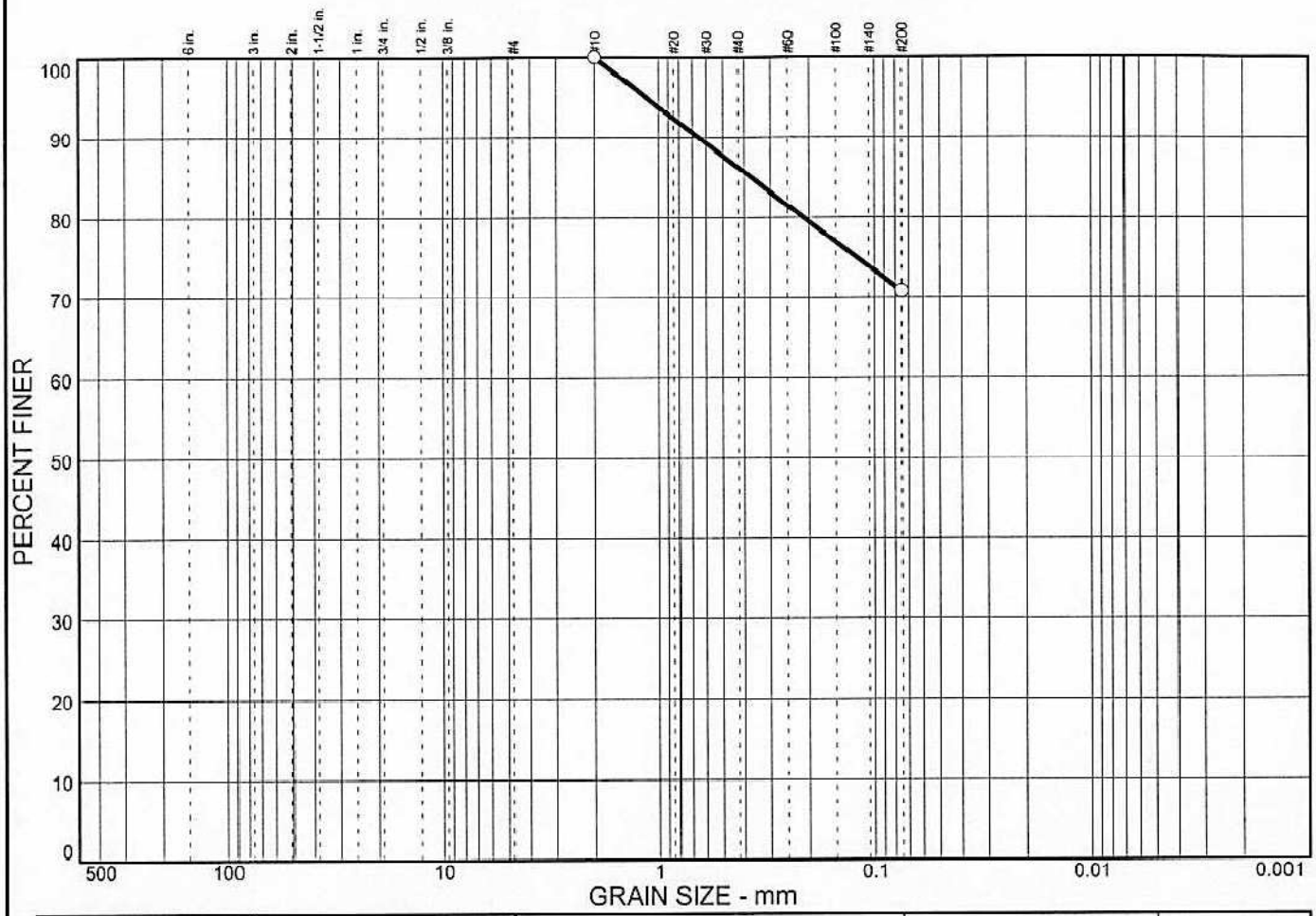
Sample No.: BD-2 **Source of Sample:** **Date:** 1-27-05
Location: **Elev./Depth:** 25'

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Client: CH2M Hill
Project: Bellota Weir / #175404.AA.P2.03

Project No: 2T2/305/19 **Figure:** 1

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	0.0	29.2	70.8	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#10	100.0		
#200	70.8		

* (no specification provided)

Material Description

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= 0.370 D_{60}= D_{50}=
D_{30}= D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= AASHTO=

Remarks

Total dry weight tested= 141 grams.

Sample No.: BD-2 **Source of Sample:** **Date:** 1-27-05
Location: **Elev./Depth:** 50'

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Client: CH2M Hill
Project: Bellota Weir / #175404.AA.P2.03

Project No: 2T2/305/19 **Figure:** 1

Particle Size Distribution Report



% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
0.0	1.1	63.9	35.0	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
0.375 in.	100.0		
#4	98.9		
#10	95.4		
#40	85.0		
#200	35.0		

* (no specification provided)

Material Description

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= 0.425 D_{60}= 0.157 D_{50}= 0.116
D_{30}= D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= AASHTO=

Remarks

Total dry weight tested= 241 grams.

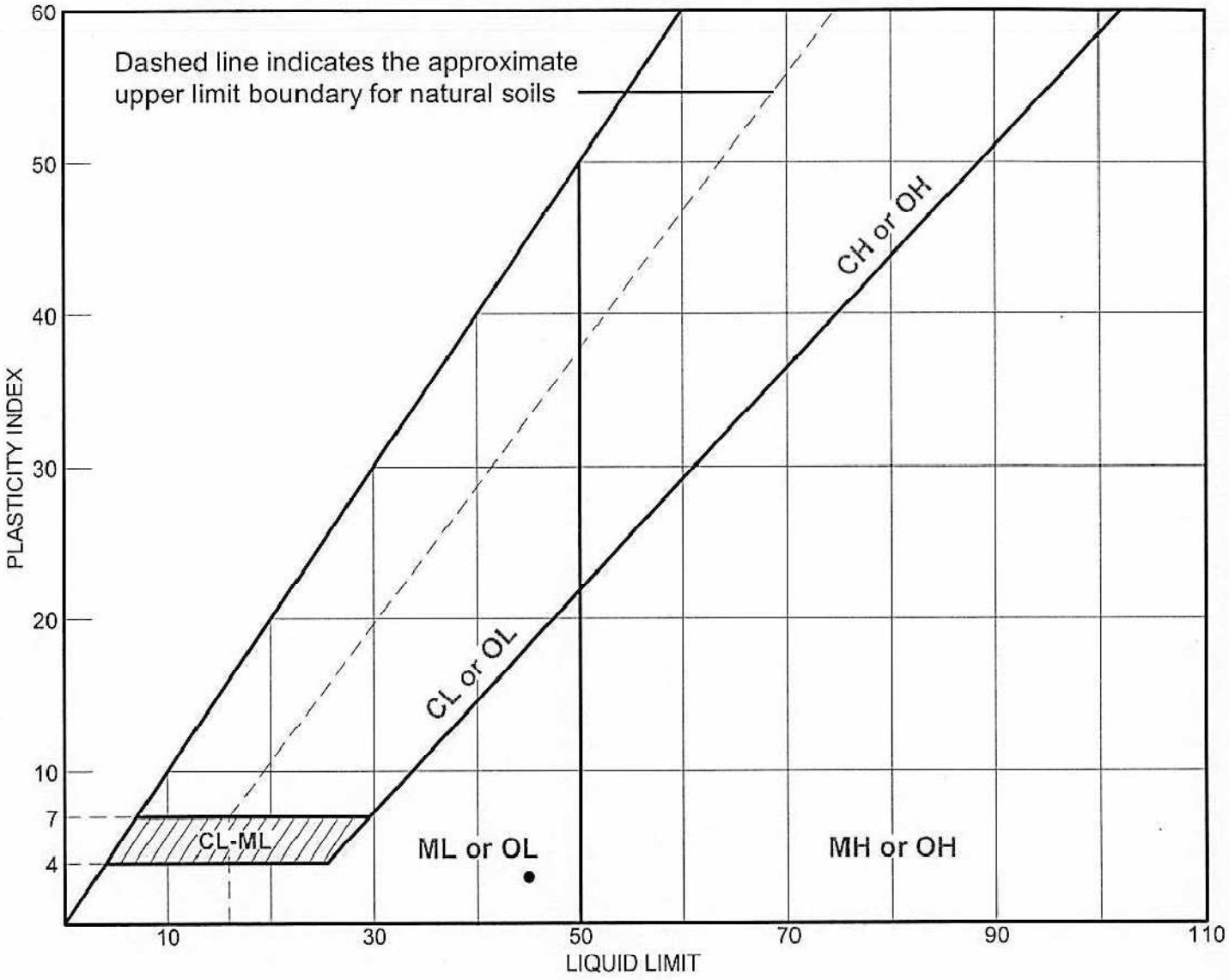
Sample No.: BD-2 **Source of Sample:** **Date:** 1-27-05
Location: **Elev./Depth:** 60'

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Client: CH2M Hill
Project: Bellota Weir / #175404.AA.P2.03

Project No: 2T2/305/19 **Figure:** 1

LIQUID AND PLASTIC LIMITS TEST REPORT



SOIL DATA								
SYMBOL	SOURCE	SAMPLE NO.	DEPTH (ft.)	NATURAL WATER CONTENT (%)	PLASTIC LIMIT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	USCS
●		BI-1	35'		42	45	3	ML

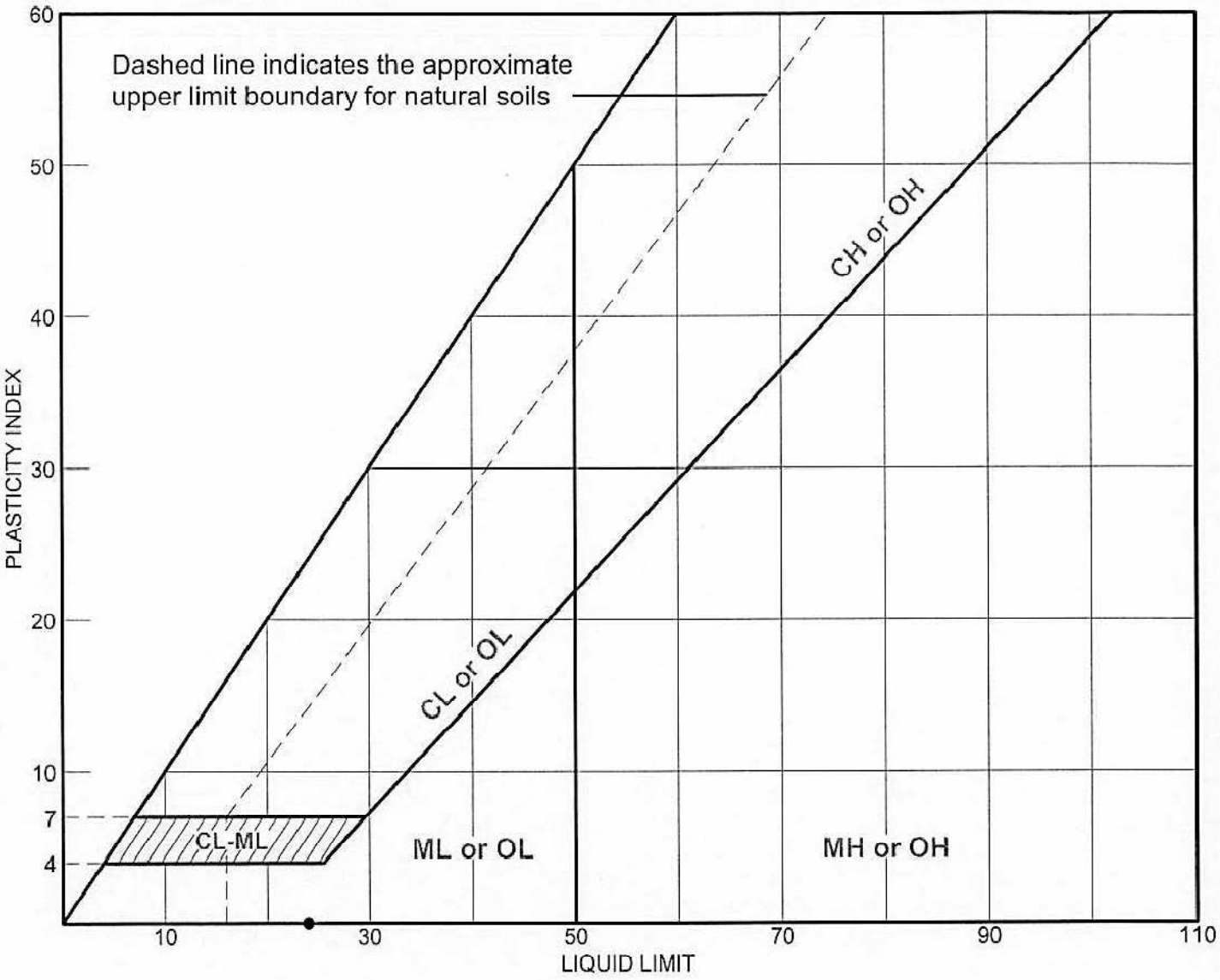
Taber Since 1954

Client: CH2M Hill

Project: Bellota Weir / #175404.AA.P2.03

Project No.: 2T2/305/19 **Figure:** 2

LIQUID AND PLASTIC LIMITS TEST REPORT

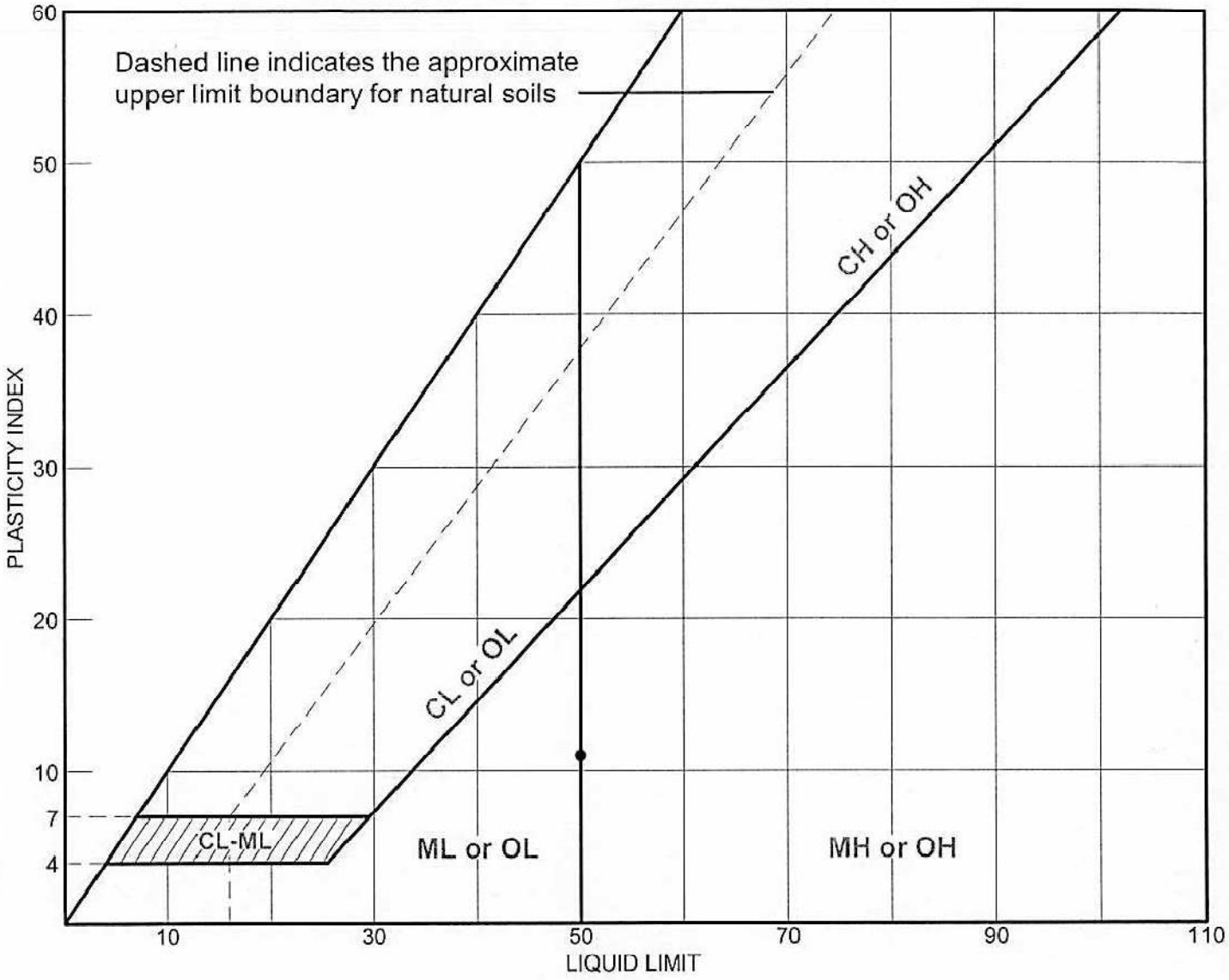


SOIL DATA								
SYMBOL	SOURCE	SAMPLE NO.	DEPTH (ft.)	NATURAL WATER CONTENT (%)	PLASTIC LIMIT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	USCS
●		BI-2	10'		NP	24	NP	SM

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Since 1954

Client: CH2M Hill
Project: Bellota Weir / #175404.AA.P2.03
Project No.: 2T2/305/19 **Figure:** 2

LIQUID AND PLASTIC LIMITS TEST REPORT



SOIL DATA								
SYMBOL	SOURCE	SAMPLE NO.	DEPTH (ft.)	NATURAL WATER CONTENT (%)	PLASTIC LIMIT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	USCS
●		BI-2	25'		39	50	11	MH

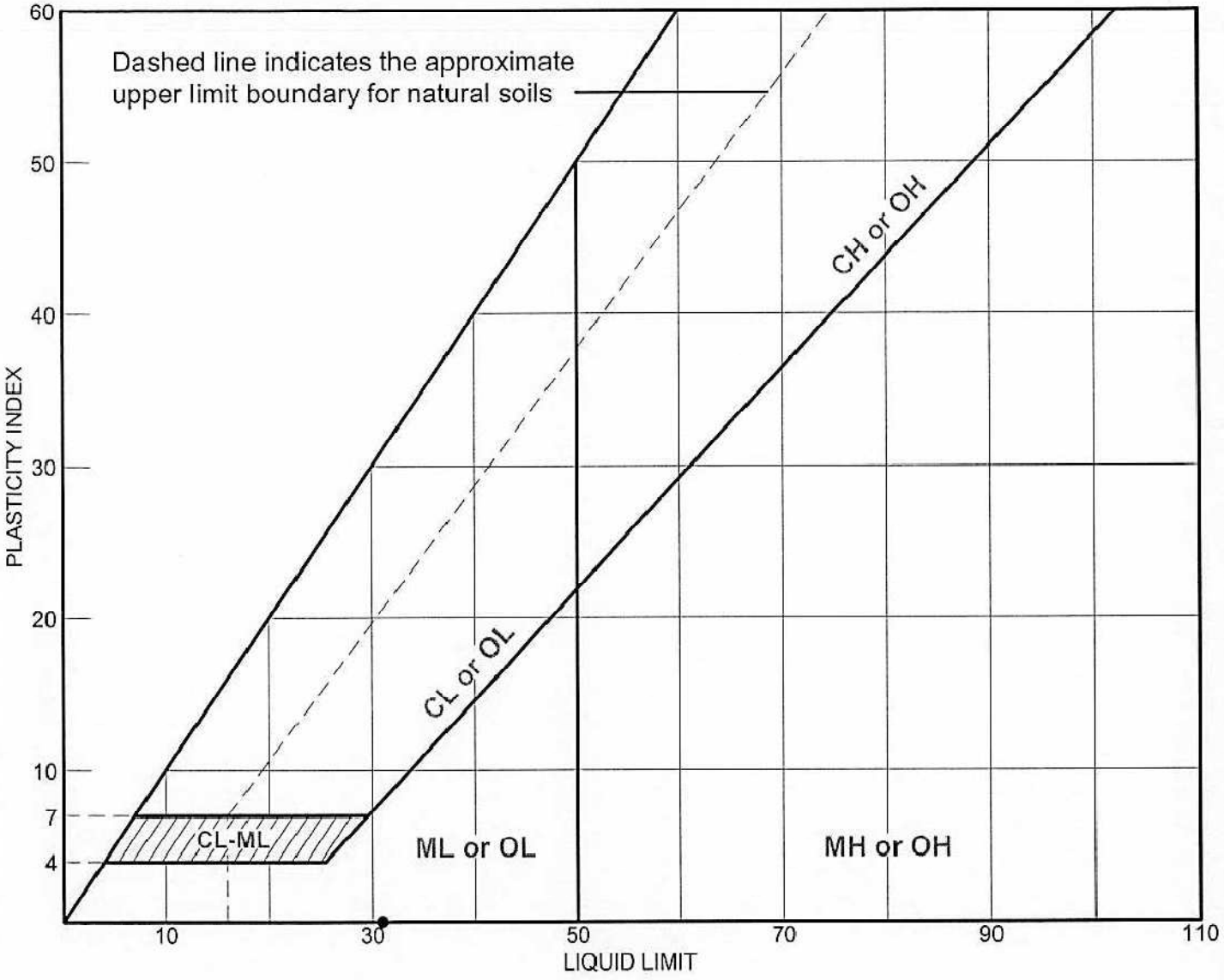
Taber Since 1954

Client: CH2M Hill

Project: Bellota Weir / #175404.AA.P2.03

Project No.: 2T2/305/19 **Figure:** 2

LIQUID AND PLASTIC LIMITS TEST REPORT



SOIL DATA								
SYMBOL	SOURCE	SAMPLE NO.	DEPTH (ft.)	NATURAL WATER CONTENT (%)	PLASTIC LIMIT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	USCS
●		BI-3	7.5'		NP	31	NP	SP

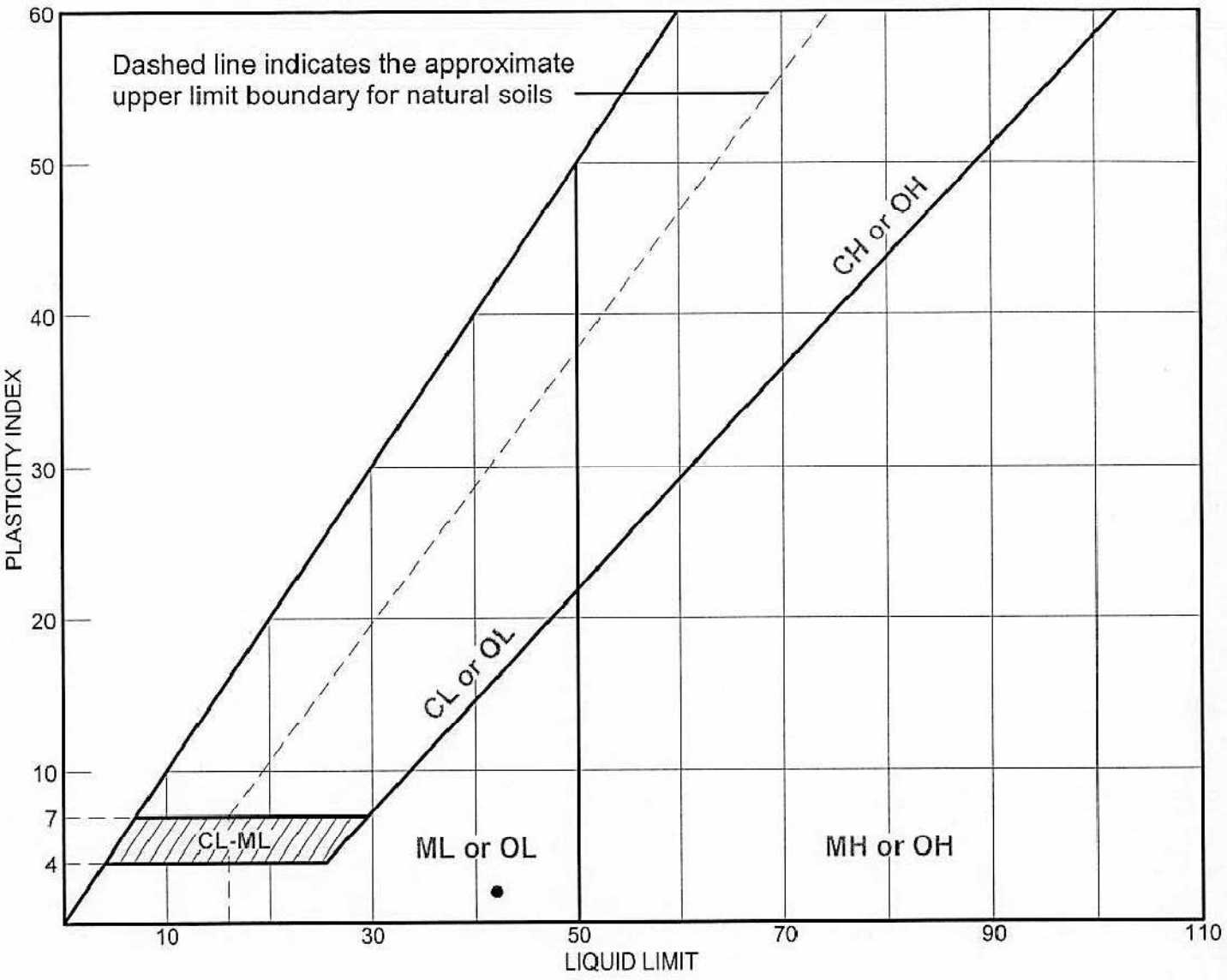
Taber Since 1954

Client: CH2M Hill

Project: Bellota Weir / #175404.AA.P2.03

Project No.: 2T2/305/19 **Figure:** 2

LIQUID AND PLASTIC LIMITS TEST REPORT



SOIL DATA								
SYMBOL	SOURCE	SAMPLE NO.	DEPTH (ft.)	NATURAL WATER CONTENT (%)	PLASTIC LIMIT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	USCS
●		BI-3	28.5'		40	42	2	ML

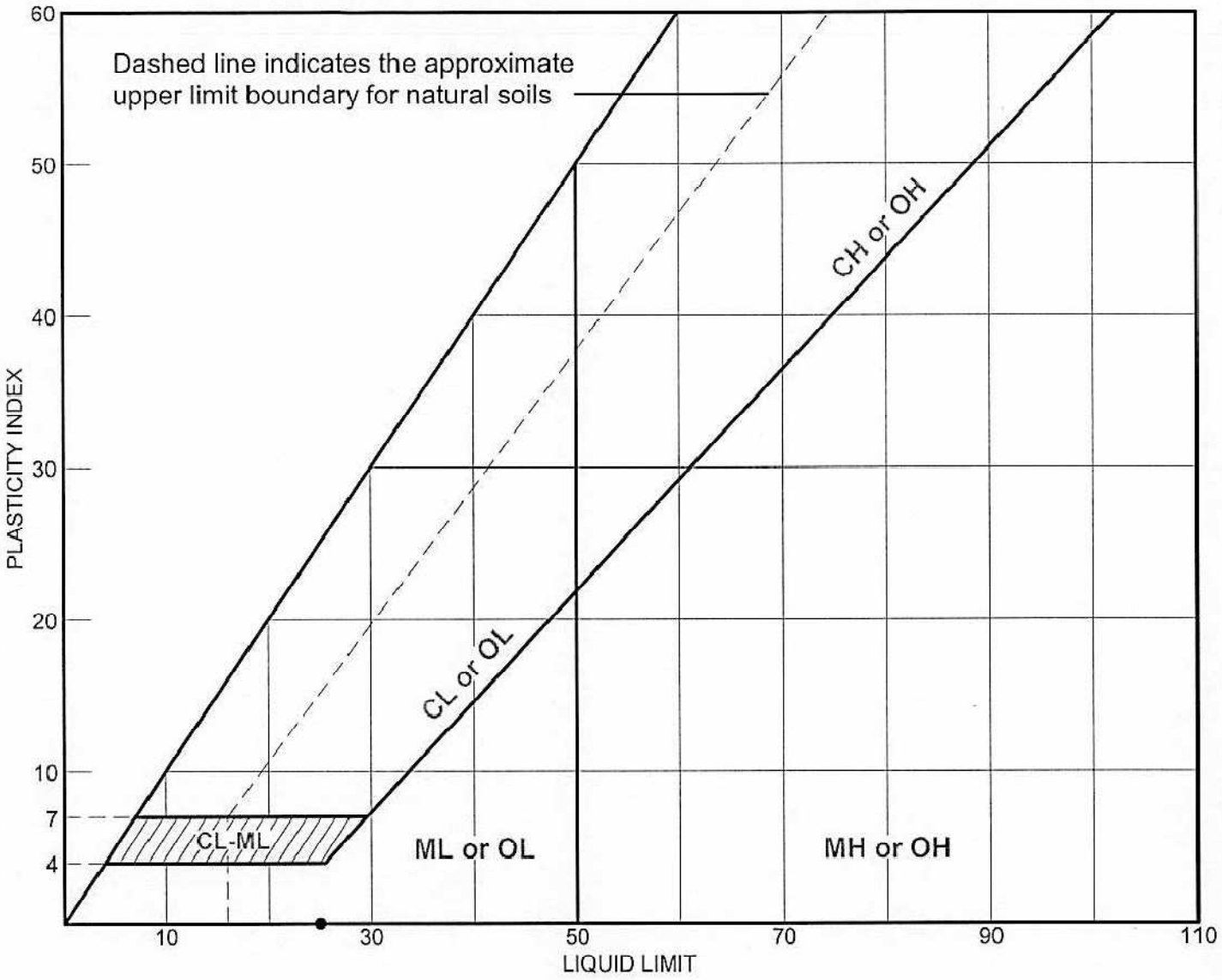
Taber Since 1954

Client: CH2M Hill

Project: Bellota Weir / #175404.AA.P2.03

Project No.: 2T2/305/19 **Figure:** 2

LIQUID AND PLASTIC LIMITS TEST REPORT

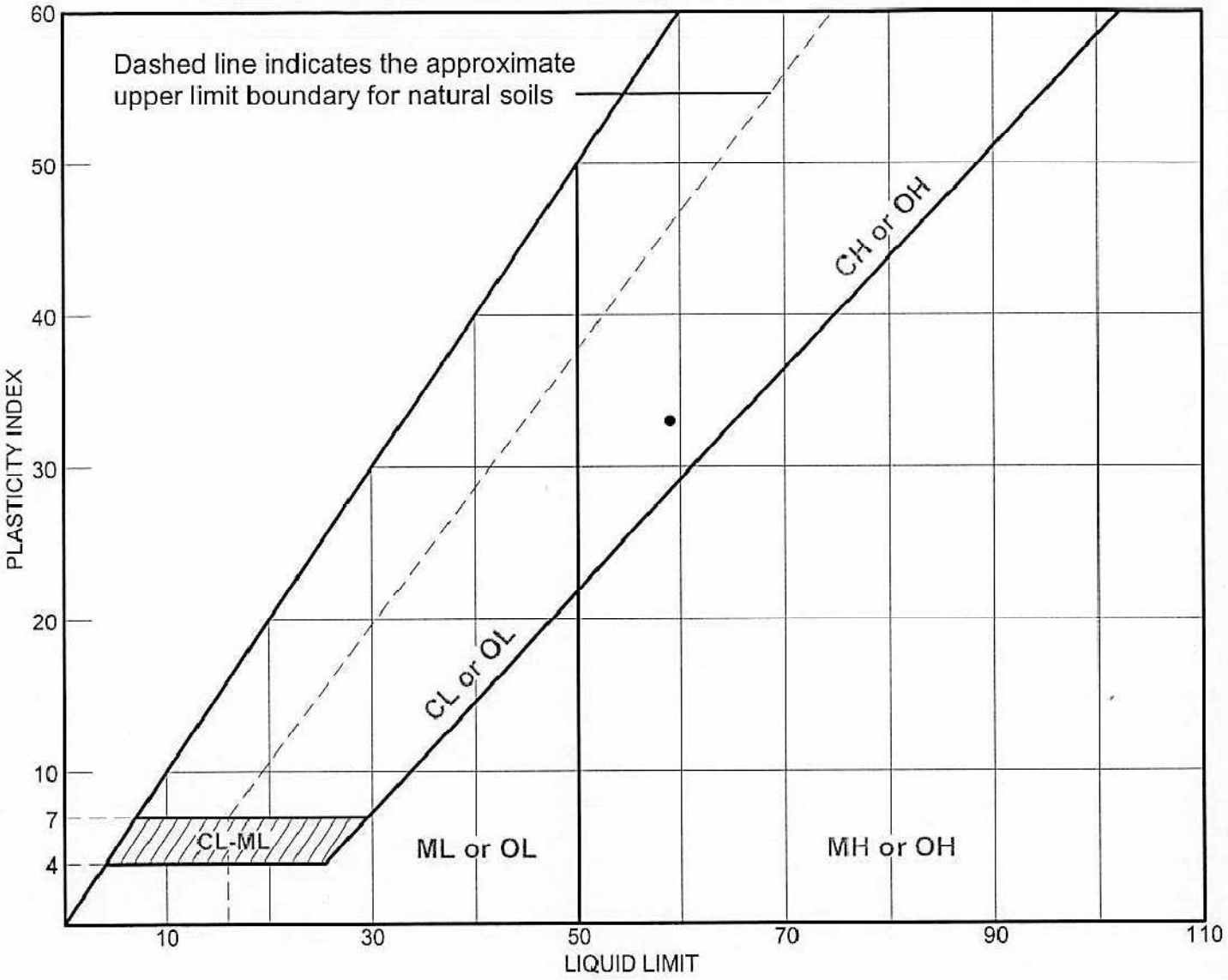


SOIL DATA								
SYMBOL	SOURCE	SAMPLE NO.	DEPTH (ft.)	NATURAL WATER CONTENT (%)	PLASTIC LIMIT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	USCS
●		BF-1	15'		NP	25	NP	SM

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Client: CH2M Hill
Project: Bellota Weir / #175404.AA.P2.03
Project No.: 2T2/305/19 **Figure:** 2

LIQUID AND PLASTIC LIMITS TEST REPORT

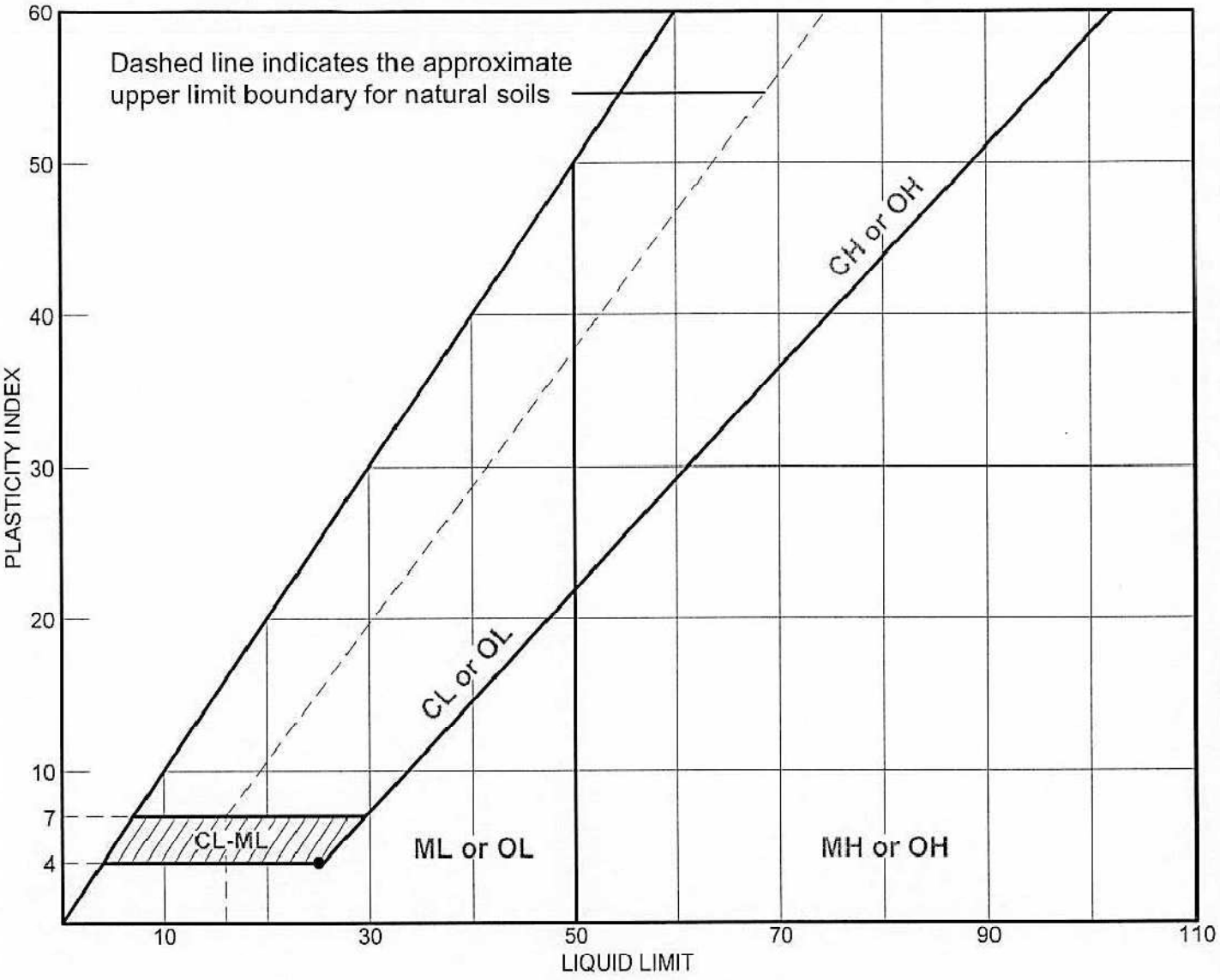


SOIL DATA								
SYMBOL	SOURCE	SAMPLE NO.	DEPTH (ft.)	NATURAL WATER CONTENT (%)	PLASTIC LIMIT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	USCS
●		BF-1	40'		26	59	33	CH

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Client: CH2M Hill
Project: Bellota Weir / #175404.AA.P2.03
Project No.: 2T2/305/19 **Figure:** 2

LIQUID AND PLASTIC LIMITS TEST REPORT

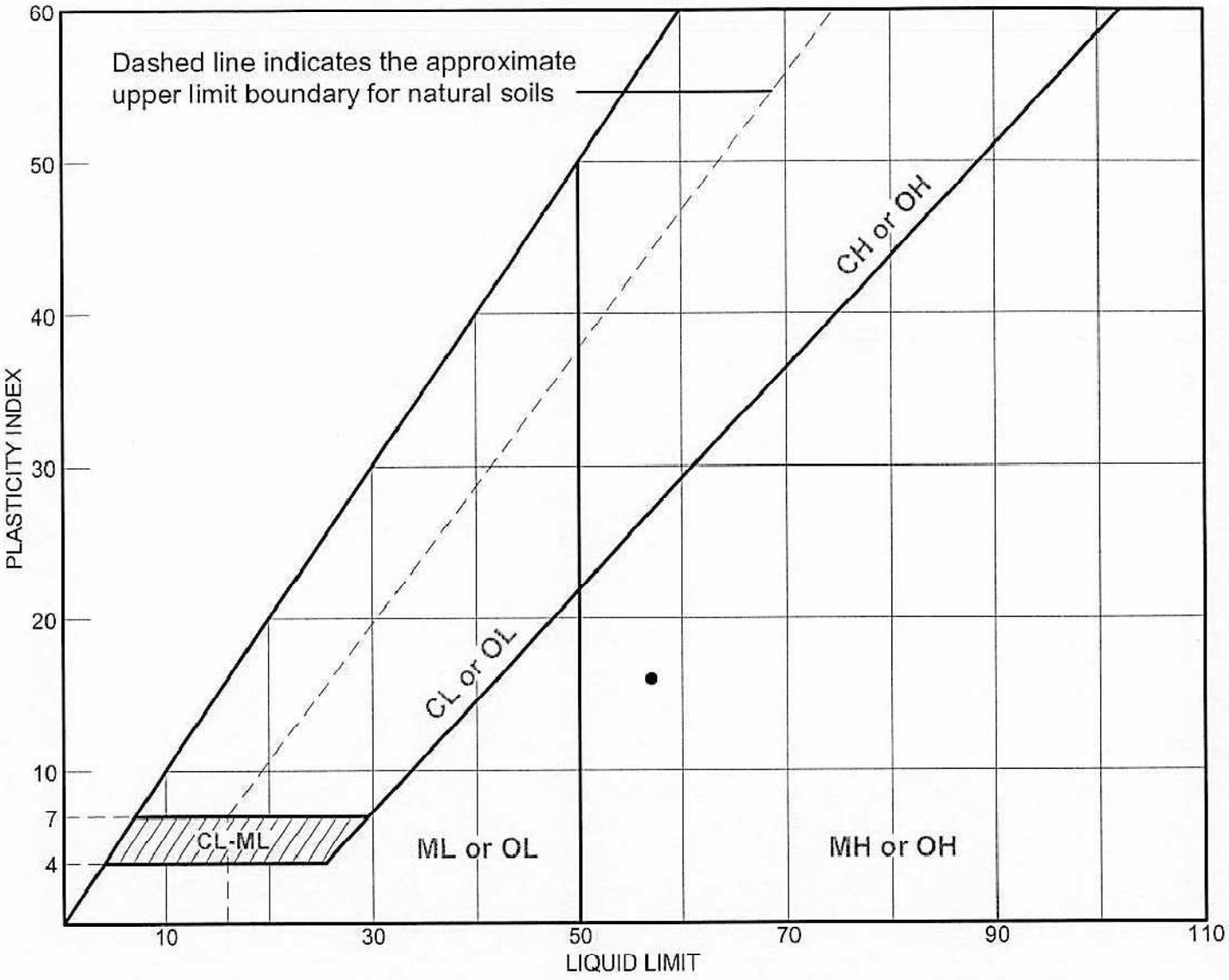


SOIL DATA								
SYMBOL	SOURCE	SAMPLE NO.	DEPTH (ft.)	NATURAL WATER CONTENT (%)	PLASTIC LIMIT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	USCS
●		BD-2	5'		21	25	4	SC-SM

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Client: CH2M Hill
Project: Bellota Weir / #175404.AA.P2.03
Project No.: 2T2/305/19 **Figure:** 2

LIQUID AND PLASTIC LIMITS TEST REPORT



SOIL DATA								
SYMBOL	SOURCE	SAMPLE NO.	DEPTH (ft.)	NATURAL WATER CONTENT (%)	PLASTIC LIMIT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	USCS
●		BD-2	25'		41	57	16	MH

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Client: CH2M Hill
Project: Bellota Weir / #175404.AA.P2.03
Project No.: 2T2/305/19 **Figure:** 2

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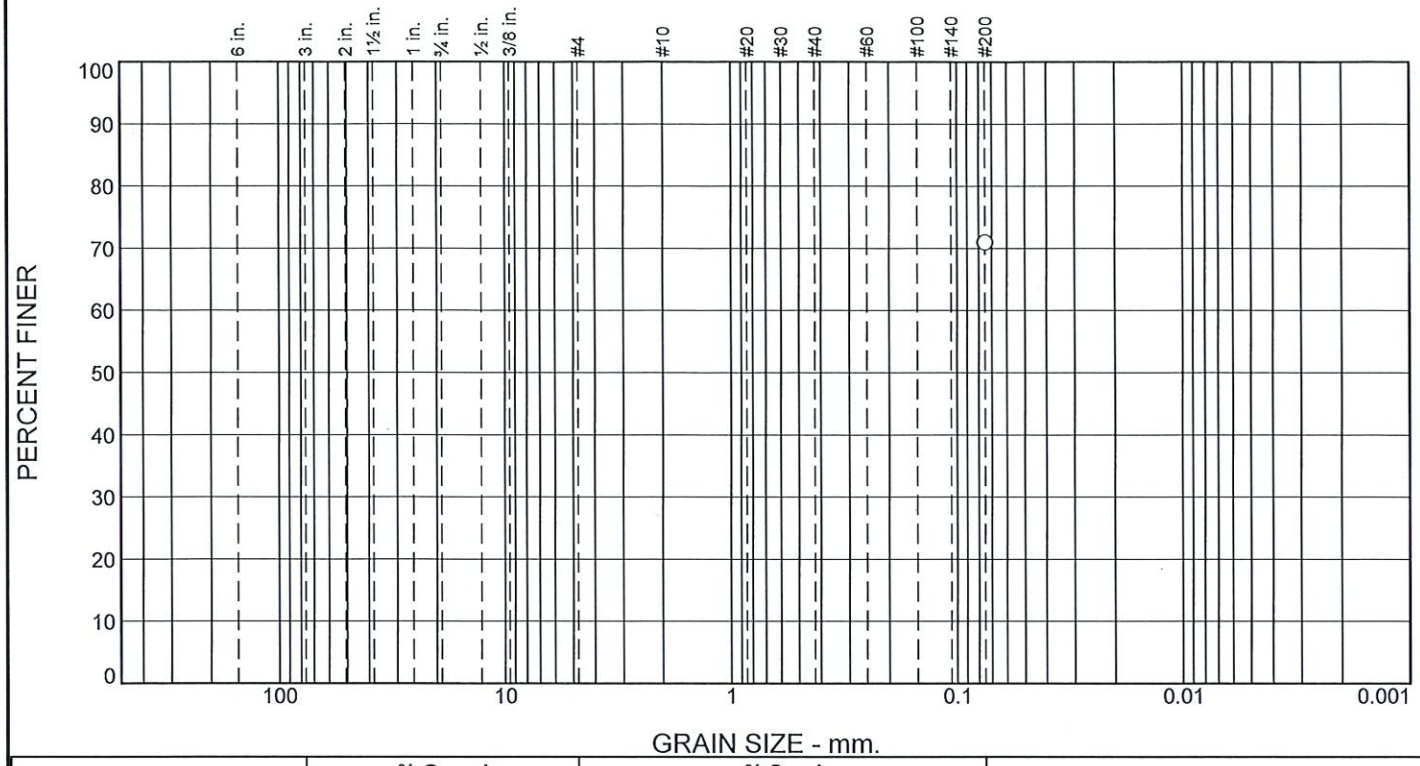
DENSITY TESTS

Client CH2M Hill / #175404.AA.P2.03 Date 1/25/2005 Job # 2T2/305/19

Sample No.	BF-1									
Sample Length (in)	8.00									
Sample Diameter (in)	2.870									
Sample weight	1608.6									
Tare										
Wet Weight	1608.6									
Corrected weight	1608.6									
Container #	C									
Wet Weight+tare	1822.6									
Dry Weight+tare	1435.0									
Water	387.6									
Tare	214.0									
Dry Weight	1221.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
% Water	**31.7%**									
Dry Density (pcf)	**90**									
Dry Density (kN/m^3)	**14.12**									
Unit Weight (pcf)	**118**									
Unit Weight (kN/m^3)	**19**									

Figure-3

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines
	Coarse	Fine	Coarse	Medium	Fine	
						71

Test Results (ASTM D6913 & ASTM D1140)

Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
#200	71		

* (no specification provided)

Material Description

Atterberg Limits (ASTM D 4318)

PL= LL= PI=

Classification

USCS (D 2487)= AASHTO (M 145)=

Coefficients

D_{90}= D_{85}= D_{60}=
D_{50}= D_{30}= D_{15}=
D_{10}= C_u= C_c=

Remarks

Date Received: 2/9/21 **Date Tested:** 2/22/21

Tested By: JB

Checked By: JML

Title: PM

Location: HDR-B1: S2 **Date Sampled:** -

Sample Number: 62174 **Depth:** 5.0'-7.5'

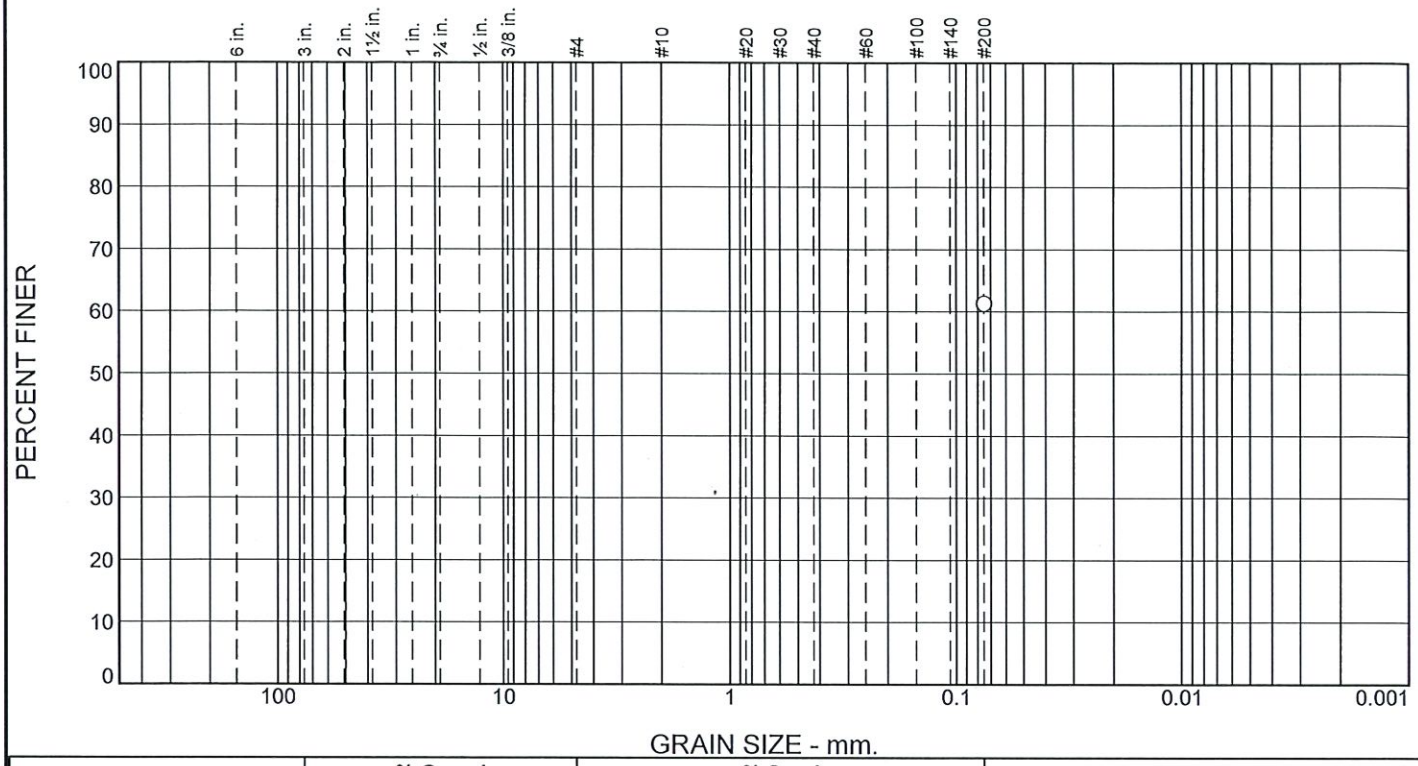
GULF SHORE EXPLORATION AND TESTING

Client: HDR, Inc.

Project: Bellotta Weir Modification

Project No: 21-105 **Figure**

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines
	Coarse	Fine	Coarse	Medium	Fine	
						61

Test Results (ASTM D6913 & ASTM D1140)

Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
#200	61		

* (no specification provided)

Material Description

Atterberg Limits (ASTM D 4318)

PL= LL= PI=

Classification

USCS (D 2487)= AASHTO (M 145)=

Coefficients

D_{90}= D_{85}= D_{60}=
D_{50}= D_{30}= D_{15}=
D_{10}= C_u= C_c=

Remarks

Date Received: 2/9/21 **Date Tested:** 2/22/21

Tested By: JB

Checked By: JML

Title: PM

Location: HDR-B1: S4
Sample Number: 62176 **Depth:** 10.0'-11.5'

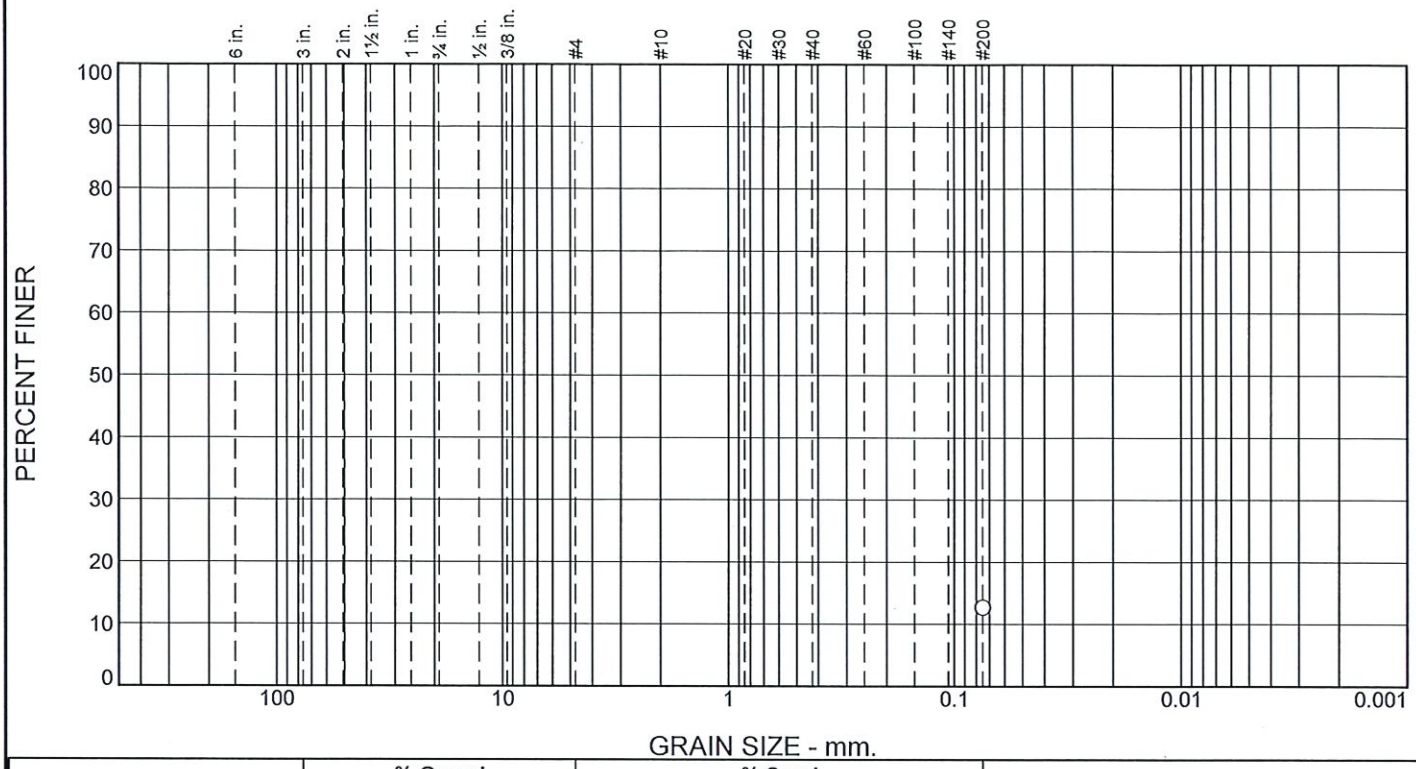
Date Sampled: -

GULF SHORE EXPLORATION AND TESTING

Client: HDR, Inc.
Project: Bellotta Weir Modification

Project No: 21-105 **Figure**

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines
	Coarse	Fine	Coarse	Medium	Fine	
						13

Test Results (ASTM D6913 & ASTM D1140)

Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
#200	13		

* (no specification provided)

Material Description

Atterberg Limits (ASTM D 4318)

PL= NP LL= NV PI= NP

Classification

USCS (D 2487)= AASHTO (M 145)=

Coefficients

D_{90}= D_{85}= D_{60}=
D_{50}= D_{30}= D_{15}=
D_{10}= C_u= C_c=

Remarks

Date Received: 2/9/21 **Date Tested:** 2/22/21

Tested By: JB

Checked By: JML

Title: PM

Location: HDR-B1: S7
Sample Number: 62178 **Depth:** 17.5'-19.0'

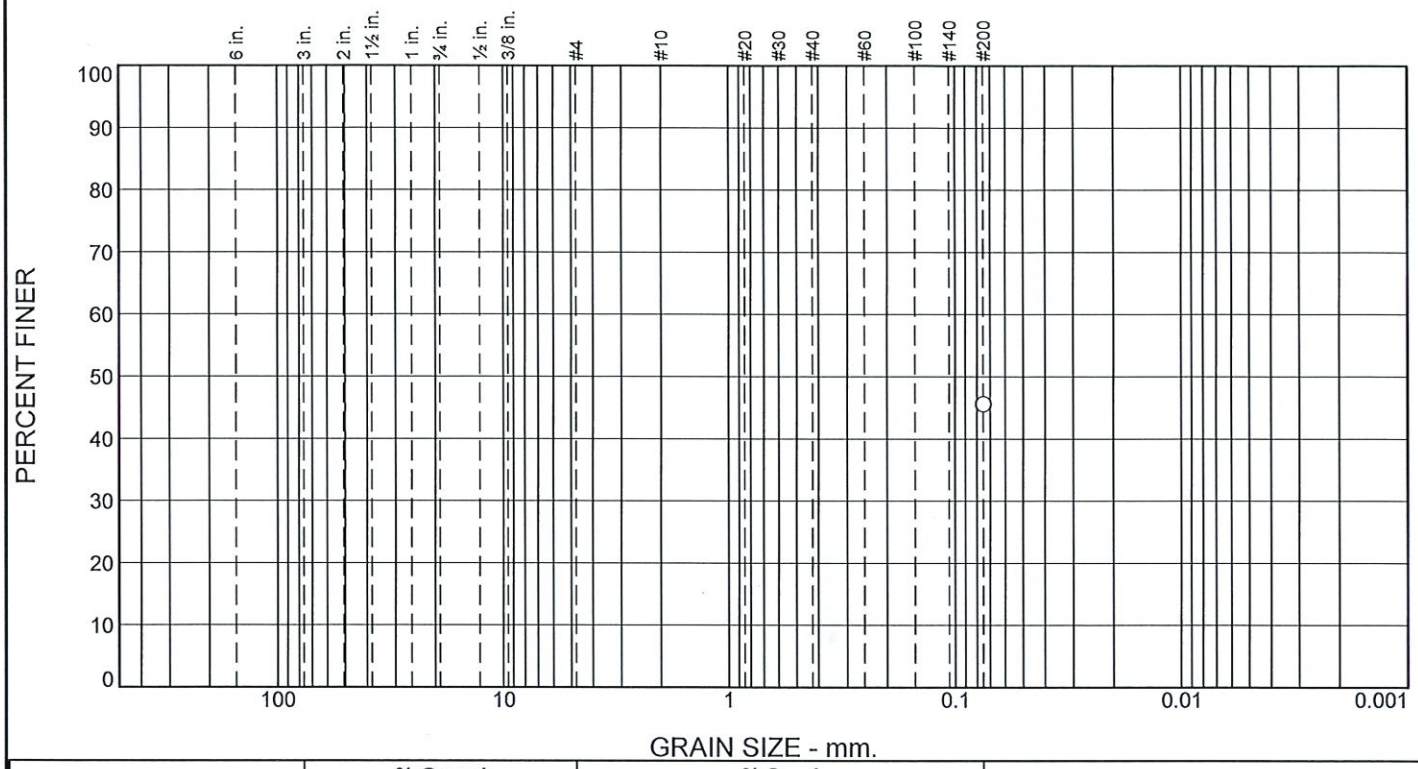
Date Sampled: -

GULF SHORE EXPLORATION AND TESTING

Client: HDR, Inc.
Project: Bellotta Weir Modification

Project No: 21-105 **Figure**

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines
	Coarse	Fine	Coarse	Medium	Fine	
						46

Test Results (ASTM D6913 & ASTM D1140)

Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
#200	46		

* (no specification provided)

Material Description

Atterberg Limits (ASTM D 4318)

PL= LL= PI=

Classification

USCS (D 2487)= AASHTO (M 145)=

Coefficients

D_{90}= D_{85}= D_{60}=
D_{50}= D_{30}= D_{15}=
D_{10}= C_u= C_c=

Remarks

Date Received: 2/9/21 **Date Tested:** 2/22/21

Tested By: JB

Checked By: JML

Title: PM

Location: HDR-B2: S7 **Date Sampled:** -

Sample Number: 62187 **Depth:** 20.0'-21.5'

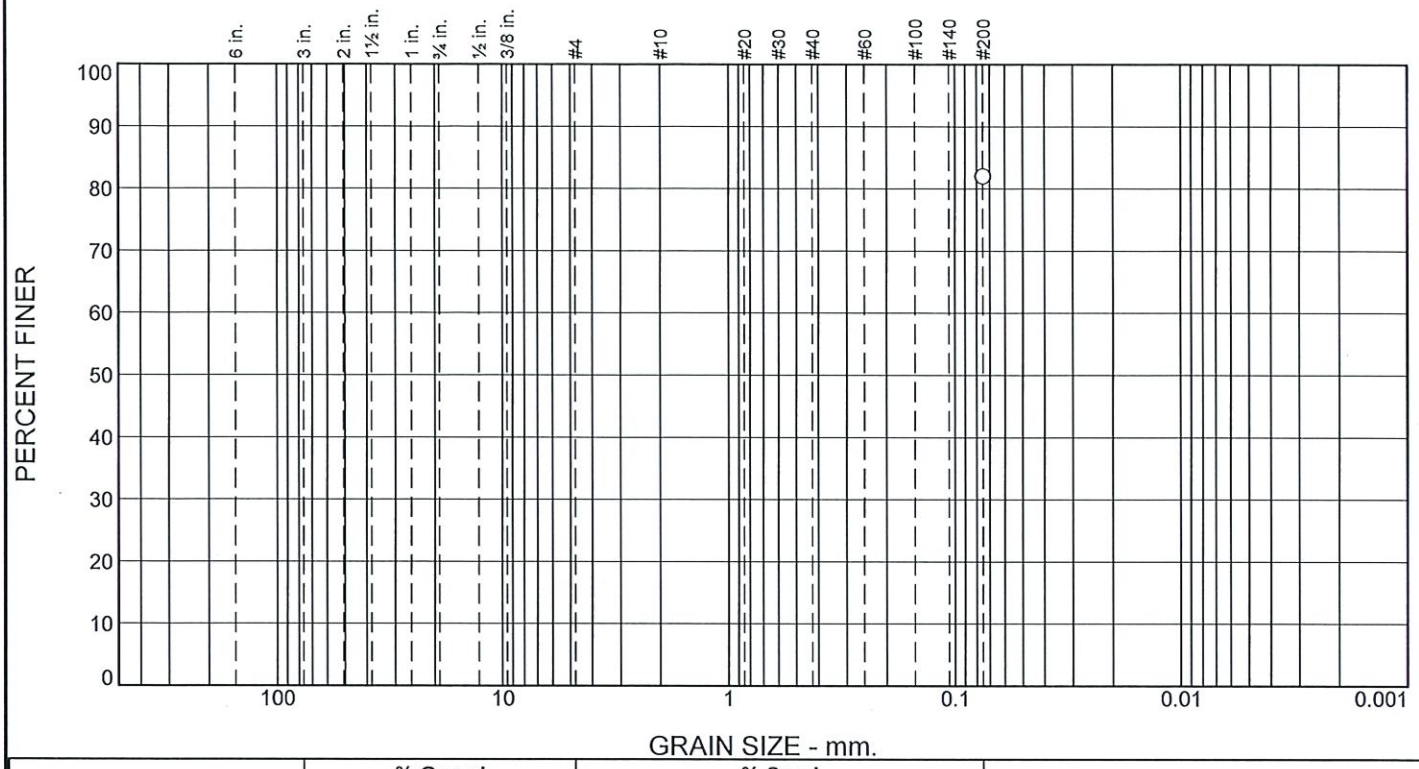
GULF SHORE EXPLORATION AND TESTING

Client: HDR, Inc.

Project: Bellotta Weir Modification

Project No: 21-105 **Figure**

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines
	Coarse	Fine	Coarse	Medium	Fine	
						82

Test Results (ASTM D6913 & ASTM D1140)

Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
#200	82		

* (no specification provided)

Material Description

Atterberg Limits (ASTM D 4318)

PL= LL= PI=

Classification

USCS (D 2487)= AASHTO (M 145)=

Coefficients

D_{90}= D_{85}= D_{60}=
D_{50}= D_{30}= D_{15}=
D_{10}= C_u= C_c=

Remarks

Date Received: 2/9/21 **Date Tested:** 2/22/21

Tested By: JB

Checked By: JML

Title: PM

Location: HDR-B2: S12 **Date Sampled:** -

Sample Number: 62189 **Depth:** 42.5'-44.0'

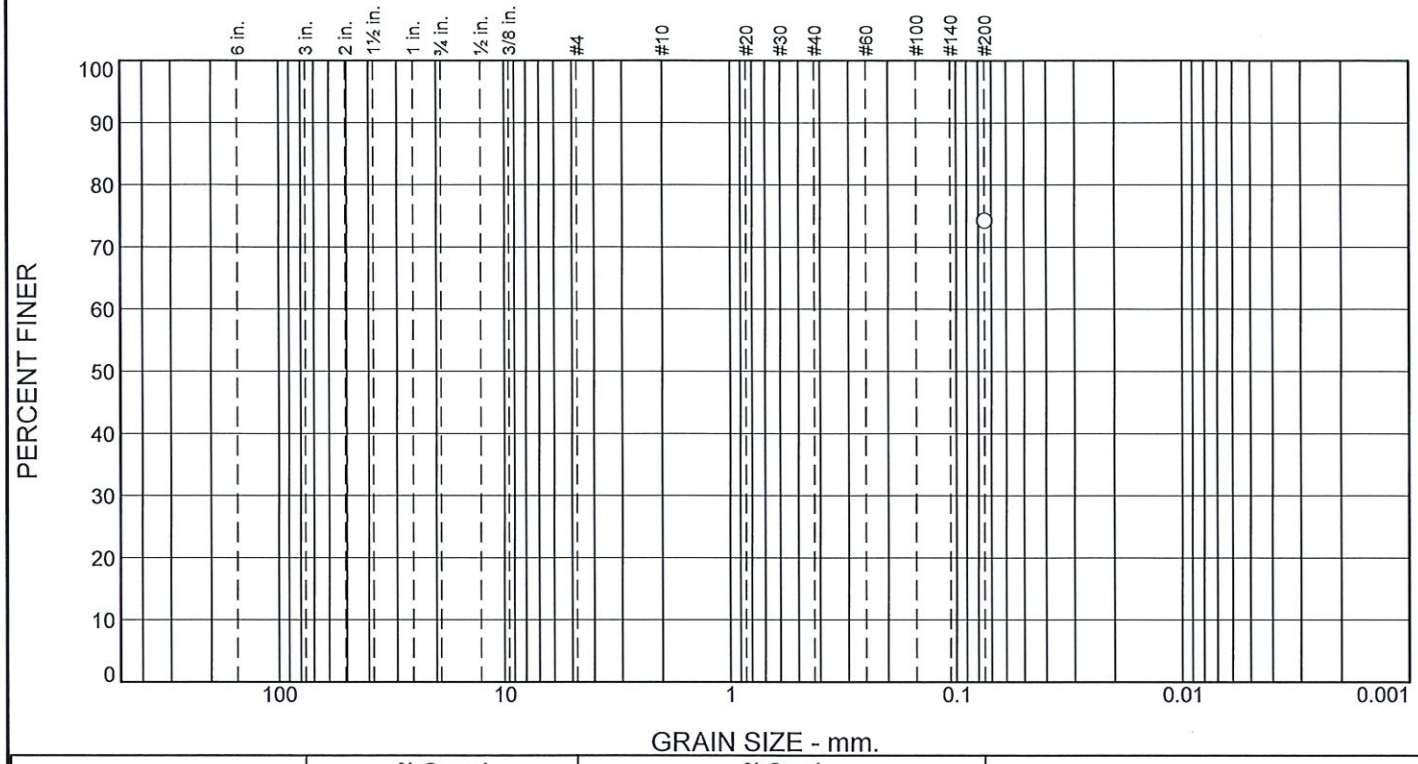
GULF SHORE EXPLORATION AND TESTING

Client: HDR, Inc.

Project: Bellotta Weir Modification

Project No: 21-105 **Figure**

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines
	Coarse	Fine	Coarse	Medium	Fine	
						74

Test Results (ASTM D6913 & ASTM D1140)			
Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
#200	74		

* (no specification provided)

Material Description

Atterberg Limits (ASTM D 4318)

PL= LL= PI=

Classification

USCS (D 2487)= AASHTO (M 145)=

Coefficients

D_{90}= D_{85}= D_{60}=
D_{50}= D_{30}= D_{15}=
D_{10}= C_u= C_c=

Remarks

Date Received: 2/9/21 **Date Tested:** 2/22/21

Tested By: JB

Checked By: JML

Title: PM

Location: HDR-B2: S15
Sample Number: 62190 **Depth:** 55.0'-56.5'

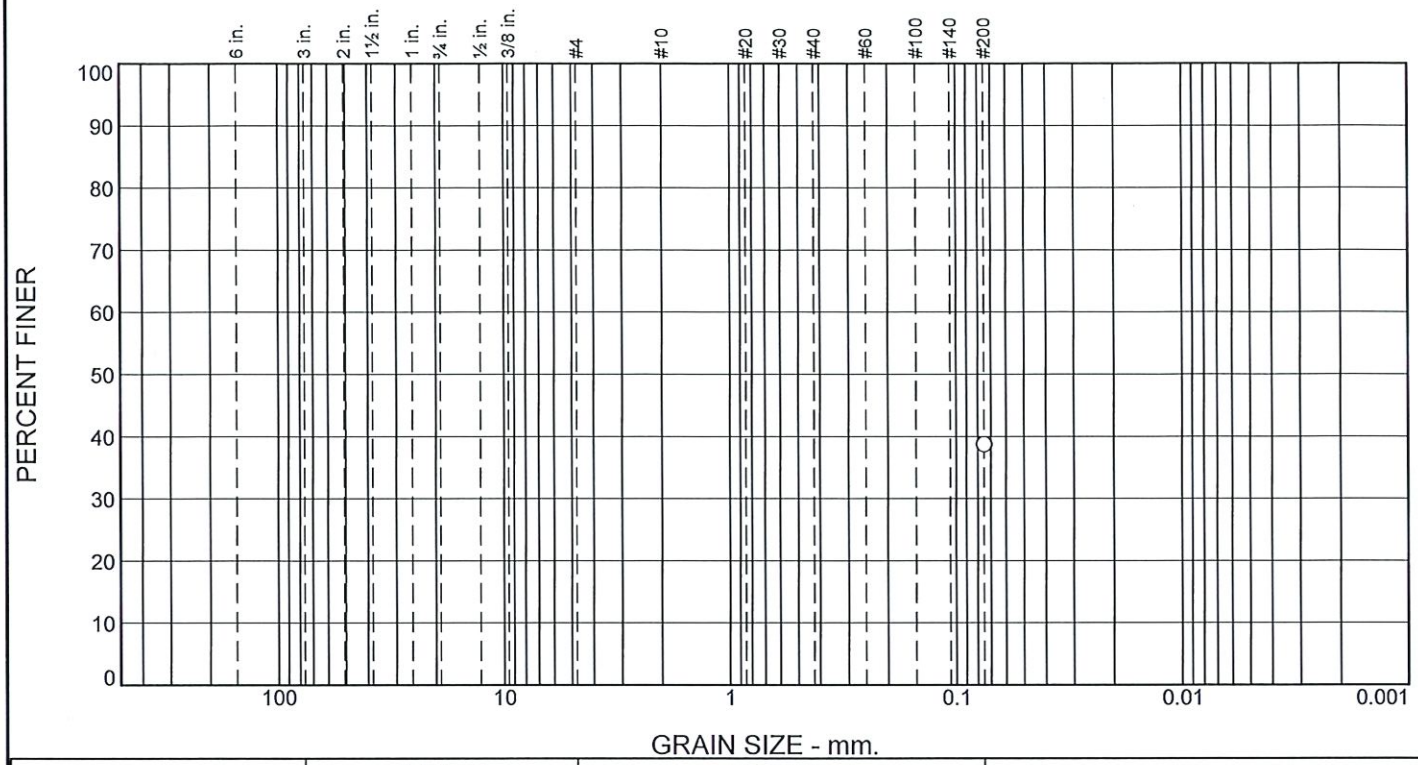
Date Sampled: -

GULF SHORE EXPLORATION AND TESTING

Client: HDR, Inc.
Project: Bellotta Weir Modification

Project No: 21-105 **Figure**

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines
	Coarse	Fine	Coarse	Medium	Fine	
						39

Test Results (ASTM D6913 & ASTM D1140)

Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
#200	39		

* (no specification provided)

Material Description

Atterberg Limits (ASTM D 4318)

PL= LL= PI=

Classification

USCS (D 2487)= AASHTO (M 145)=

Coefficients

D_{90}= D_{85}= D_{60}=
D_{50}= D_{30}= D_{15}=
D_{10}= C_u= C_c=

Remarks

Date Received: 2/9/21 **Date Tested:** 2/22/21

Tested By: JB

Checked By: JML

Title: PM

Location: HDR-B3: S5 **Date Sampled:** -

Sample Number: 62192 **Depth:** 12.5'-14.0'

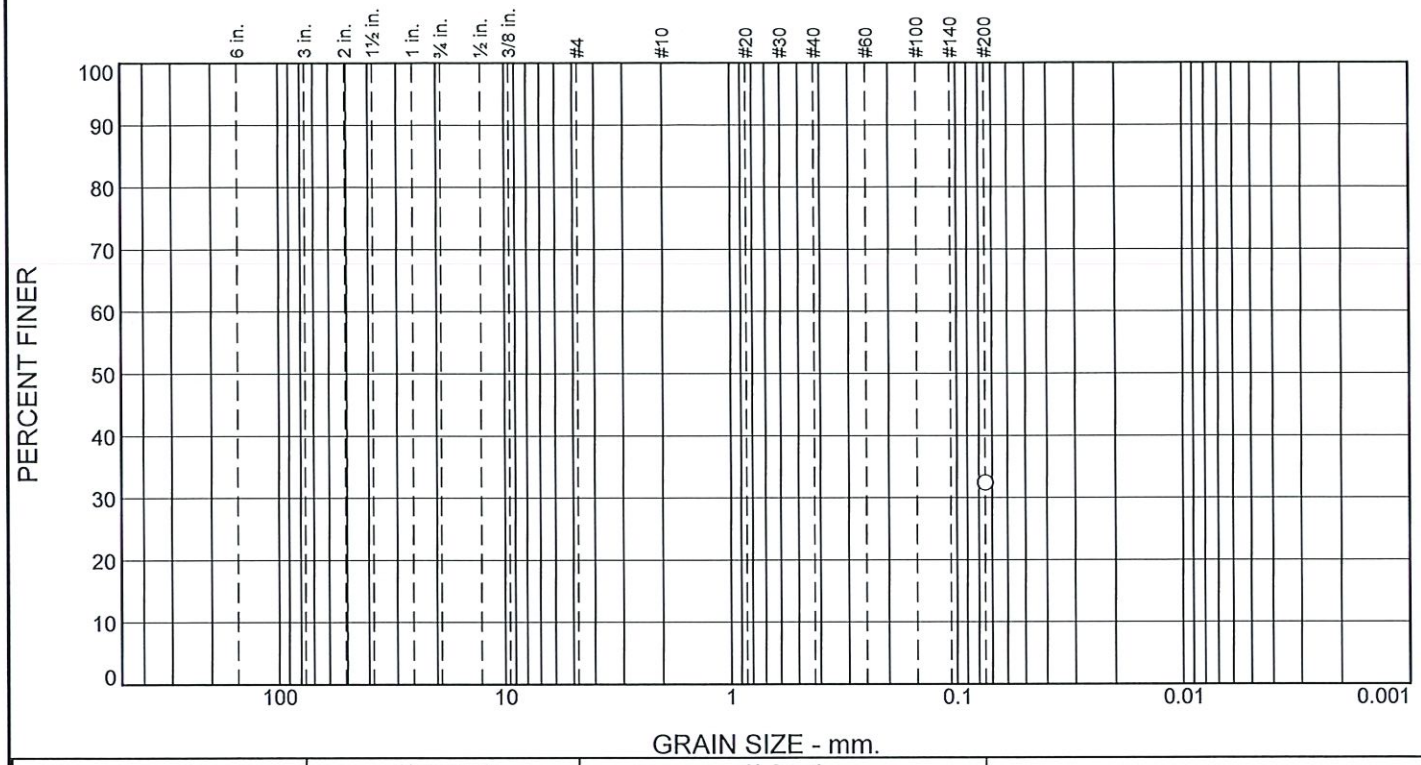
GULF SHORE EXPLORATION AND TESTING

Client: HDR, Inc.

Project: Bellotta Weir Modification

Project No: 21-105 **Figure**

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines
	Coarse	Fine	Coarse	Medium	Fine	
						32

Test Results (ASTM D6913 & ASTM D1140)

Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
#200	32		

* (no specification provided)

Material Description

Atterberg Limits (ASTM D 4318)

PL= LL= PI=

Classification

USCS (D 2487)= AASHTO (M 145)=

Coefficients

D_{90}= D_{85}= D_{60}=
D_{50}= D_{30}= D_{15}=
D_{10}= C_u= C_c=

Remarks

Date Received: 2//9/21 **Date Tested:** 2/22/21

Tested By: JB

Checked By: JML

Title: PM

Location: HDR-B3: S6 **Date Sampled:** -

Sample Number: 62193 **Depth:** 15.0'-16.5'

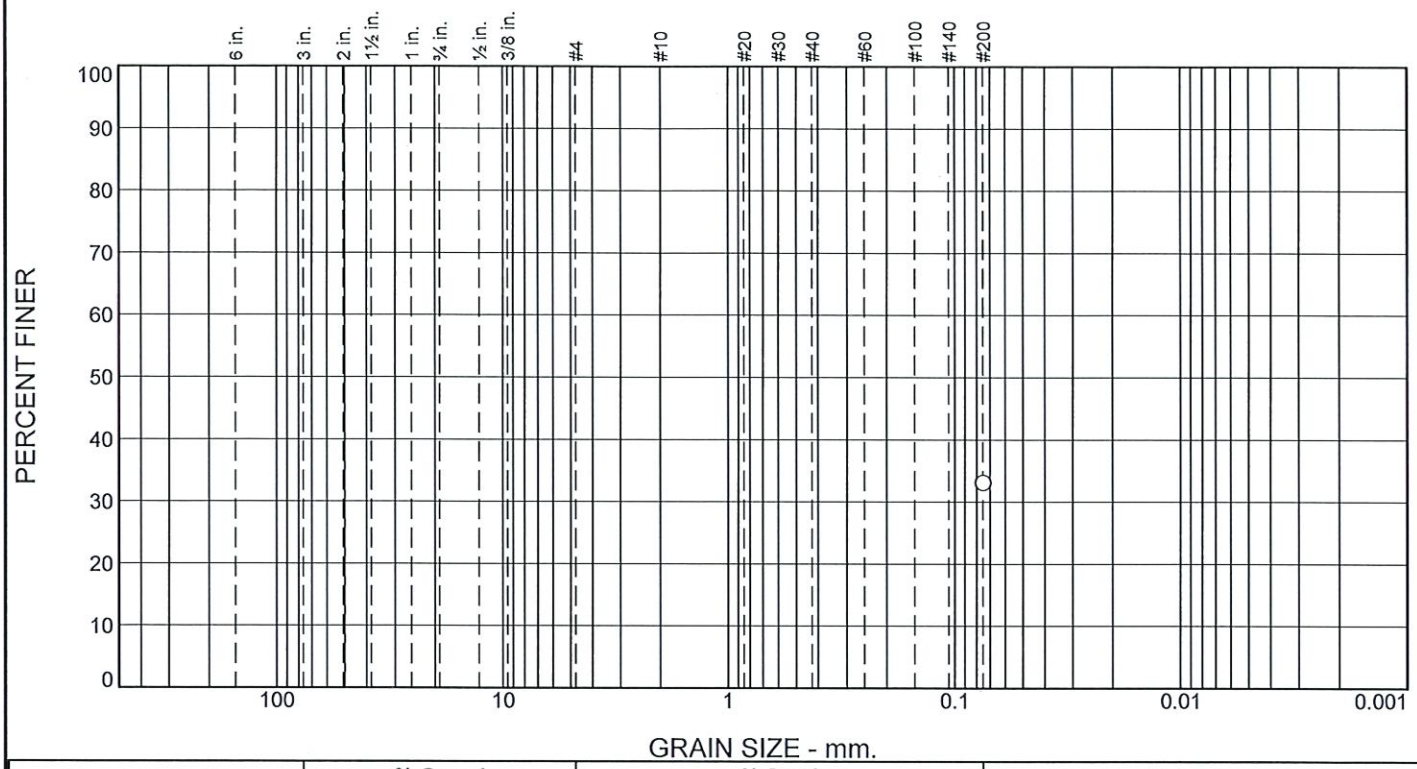
GULF SHORE EXPLORATION AND TESTING

Client: HDR, Inc.

Project: Bellotta Weir Modification

Project No: 21-105 **Figure**

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines
	Coarse	Fine	Coarse	Medium	Fine	
						33

Test Results (ASTM D6913 & ASTM D1140)			
Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
#200	33		

* (no specification provided)

Material Description

Atterberg Limits (ASTM D 4318)

PL= LL= PI=

Classification

USCS (D 2487)= AASHTO (M 145)=

Coefficients

D_{90}= D_{85}= D_{60}=
D_{50}= D_{30}= D_{15}=
D_{10}= C_u= C_c=

Remarks

Date Received: 2/9/21 **Date Tested:** 2/22/21

Tested By: JB

Checked By: JML

Title: PM

Location: HDR-B3: S8
Sample Number: 62194 **Depth:** 25.0'-26.5'

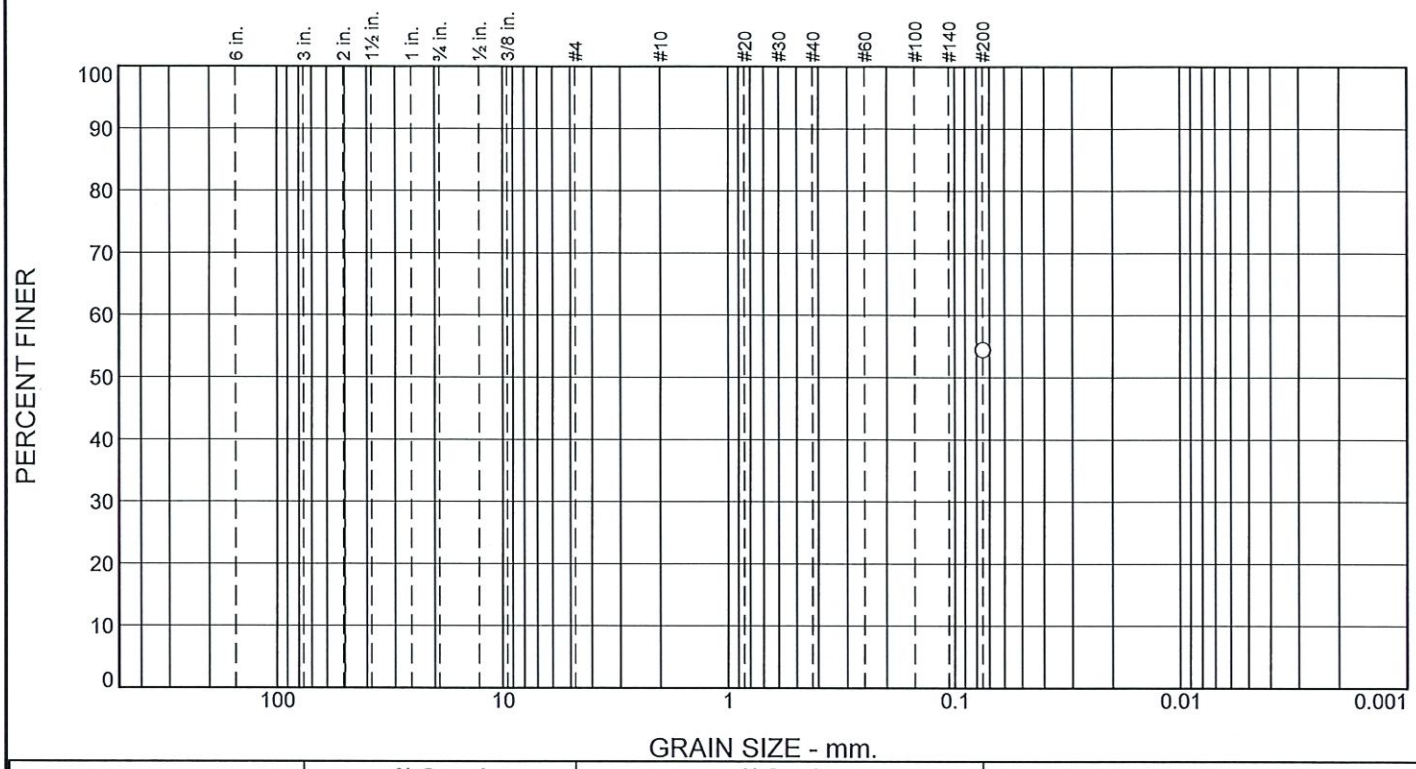
Date Sampled: -

GULF SHORE EXPLORATION AND TESTING

Client: HDR, Inc.
Project: Bellotta Weir Modification

Project No: 21-105 **Figure**

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines
	Coarse	Fine	Coarse	Medium	Fine	
						54

Test Results (ASTM D6913 & ASTM D1140)

Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
#200	54		

* (no specification provided)

Material Description

Atterberg Limits (ASTM D 4318)

PL= LL= PI=

Classification

USCS (D 2487)= AASHTO (M 145)=

Coefficients

D_{90}= D_{85}= D_{60}=
D_{50}= D_{30}= D_{15}=
D_{10}= C_u= C_c=

Remarks

Date Received: 2/9/21 **Date Tested:** 2/22/21

Tested By: JB

Checked By: JML

Title: PM

Location: HDR-B3: S17
Sample Number: 62198 **Depth:** 70.0'-71.5'

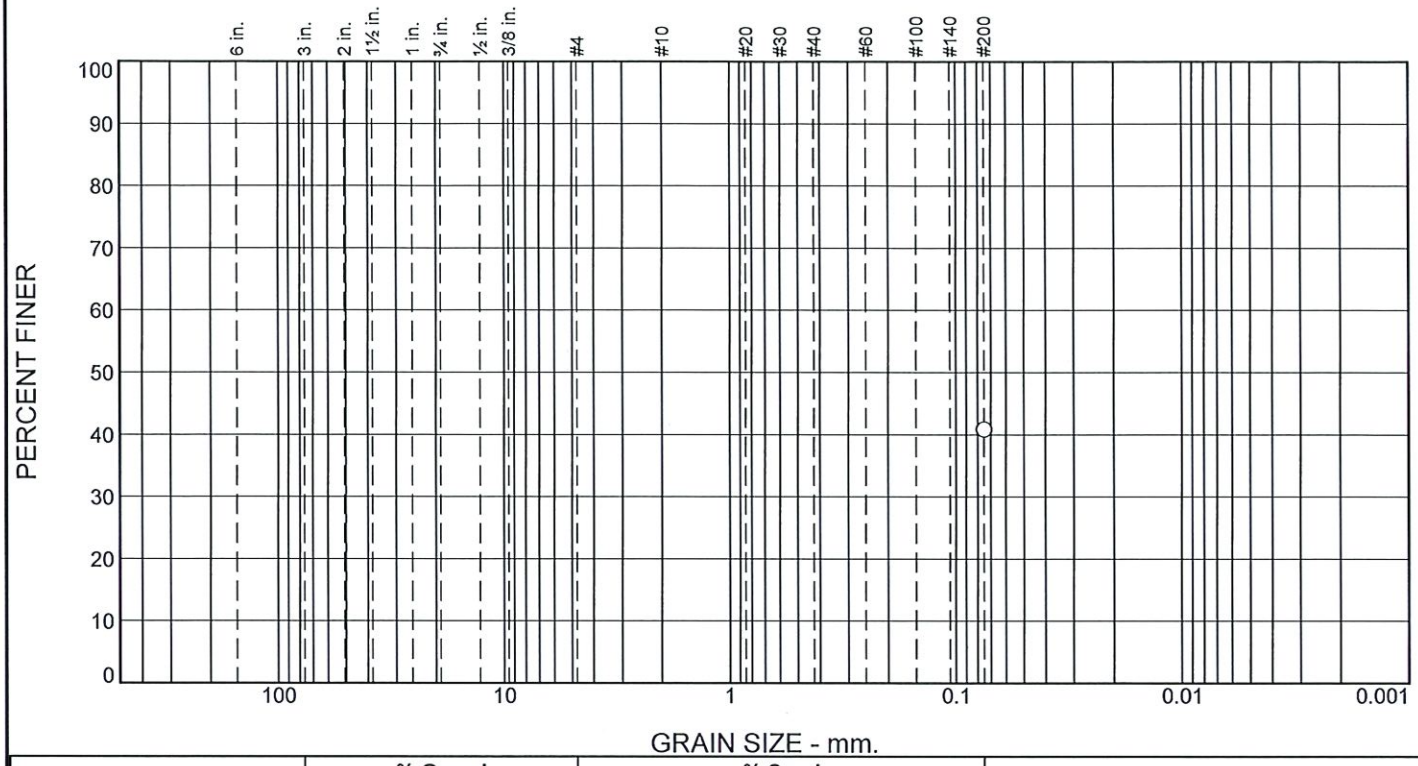
Date Sampled: -

GULF SHORE EXPLORATION AND TESTING

Client: HDR, Inc.
Project: Bellotta Weir Modification

Project No: 21-105 **Figure**

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines
	Coarse	Fine	Coarse	Medium	Fine	
						41

Test Results (ASTM D6913 & ASTM D1140)

Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
#200	41		

* (no specification provided)

Material Description

Atterberg Limits (ASTM D 4318)

PL= LL= PI=

Classification

USCS (D 2487)= AASHTO (M 145)=

Coefficients

D_{90}= D_{85}= D_{60}=
D_{50}= D_{30}= D_{15}=
D_{10}= C_u= C_c=

Remarks

Date Received: 2/9/21 **Date Tested:** 2/22/21

Tested By: JB

Checked By: JML

Title: PM

Location: HDR-B4: S4 **Date Sampled:** -

Sample Number: 62201 **Depth:** 10.0'-11.5'

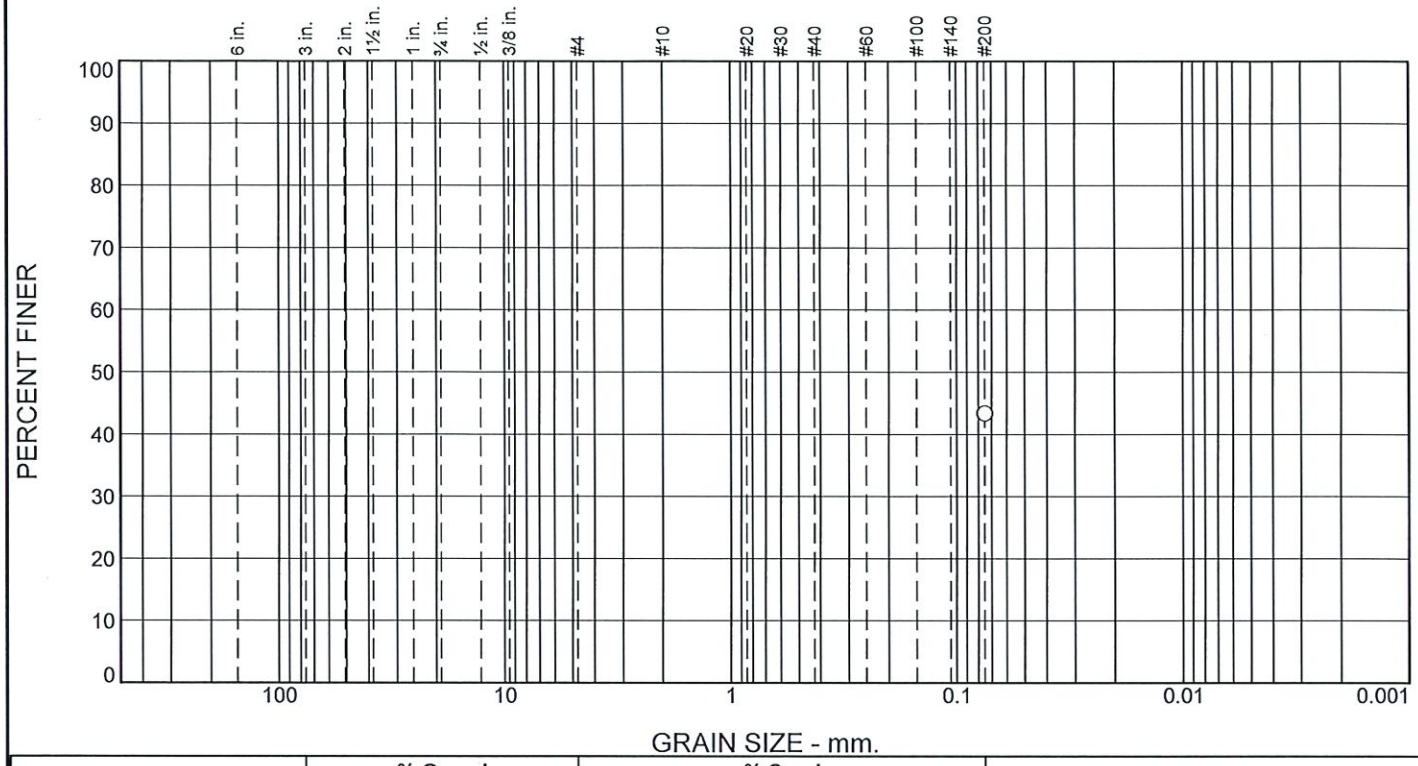
GULF SHORE EXPLORATION AND TESTING

Client: HDR, Inc.

Project: Bellotta Weir Modification

Project No: 21-105 **Figure**

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines
	Coarse	Fine	Coarse	Medium	Fine	
						43

Test Results (ASTM D6913 & ASTM D1140)

Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
#200	43		

* (no specification provided)

Material Description

Atterberg Limits (ASTM D 4318)

PL= LL= PI=

Classification

USCS (D 2487)= AASHTO (M 145)=

Coefficients

D_{90}= D_{85}= D_{60}=
D_{50}= D_{30}= D_{15}=
D_{10}= C_u= C_c=

Remarks

Date Received: 2/9/21 **Date Tested:** 2/22/21

Tested By: JB

Checked By: JML

Title: PM

Location: HDR-B5: S14
Sample Number: 62212 **Depth:** 52.5'-54.0'

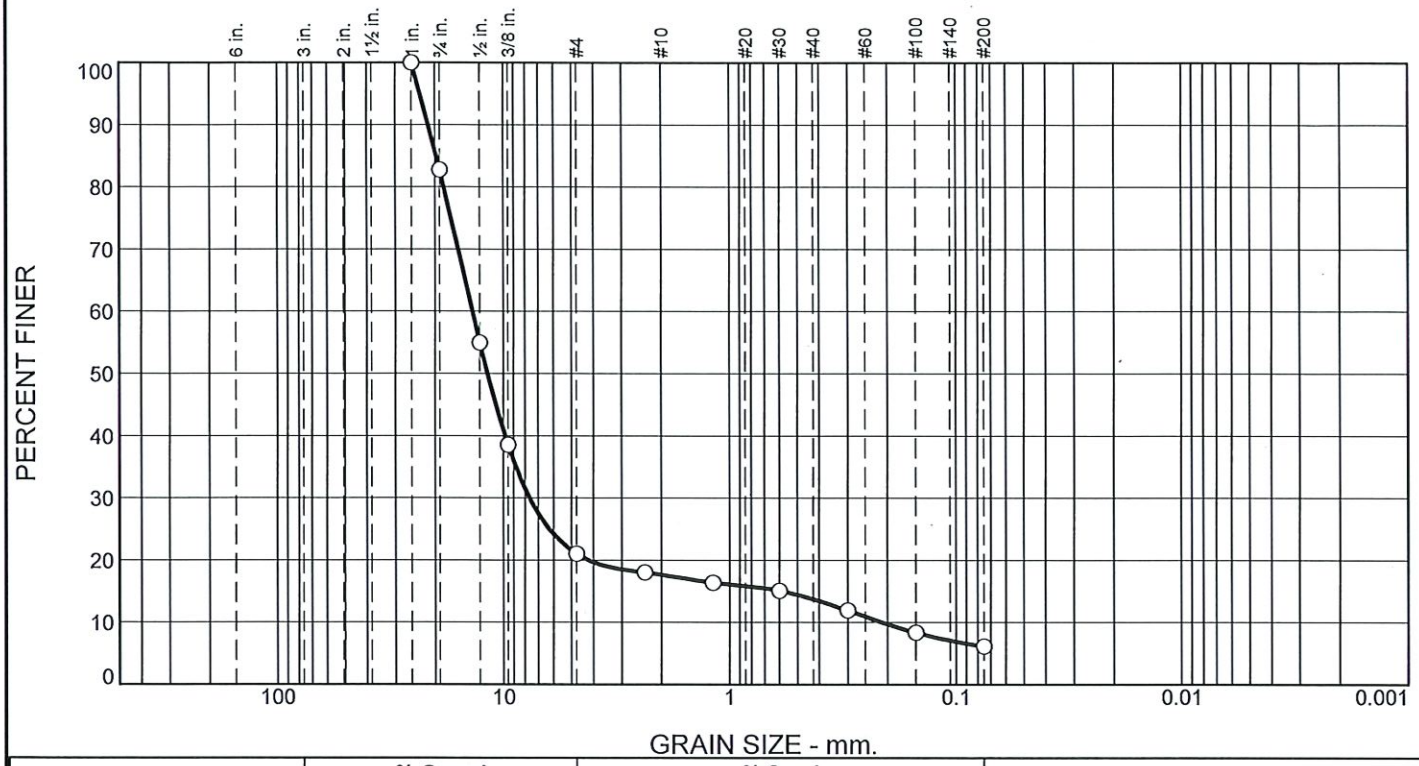
Date Sampled: -

GULF SHORE EXPLORATION AND TESTING

Client: HDR, Inc.
Project: Bellotta Weir Modification

Project No: 21-105 **Figure**

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines
	Coarse	Fine	Coarse	Medium	Fine	
0	17	62	3	4	8	6

Test Results (ASTM D6913 & ASTM D1140)

Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
1 Inch	100		
3/4 Inch	83		
1/2 Inch	55		
3/8 Inch	39		
#4	21		
#8	18		
#16	16		
#30	15		
#50	12		
#100	8		
#200	6.1		

* (no specification provided)

Material Description

Atterberg Limits (ASTM D 4318)

PL= LL= PI=

Classification

USCS (D 2487)= AASHTO (M 145)=

Coefficients

D_{90}= 21.4017 D_{85}= 19.7318 D_{60}= 13.6905
D_{50}= 11.7471 D_{30}= 7.6396 D_{15}= 0.5863
D_{10}= 0.2114 C_u= 64.77 C_c= 20.17

Remarks

Not enough sample for the max particle size (below the minimum mass requirement for specimen as required in ASTM). F.M.=5.88

Date Received: 2/9/21 **Date Tested:** 2/22/21

Tested By: JB

Checked By: JML

Title: PM

Location: HDR-B1: S8 **Date Sampled:** -

Sample Number: 62179 **Depth:** 20.0'-21.5'

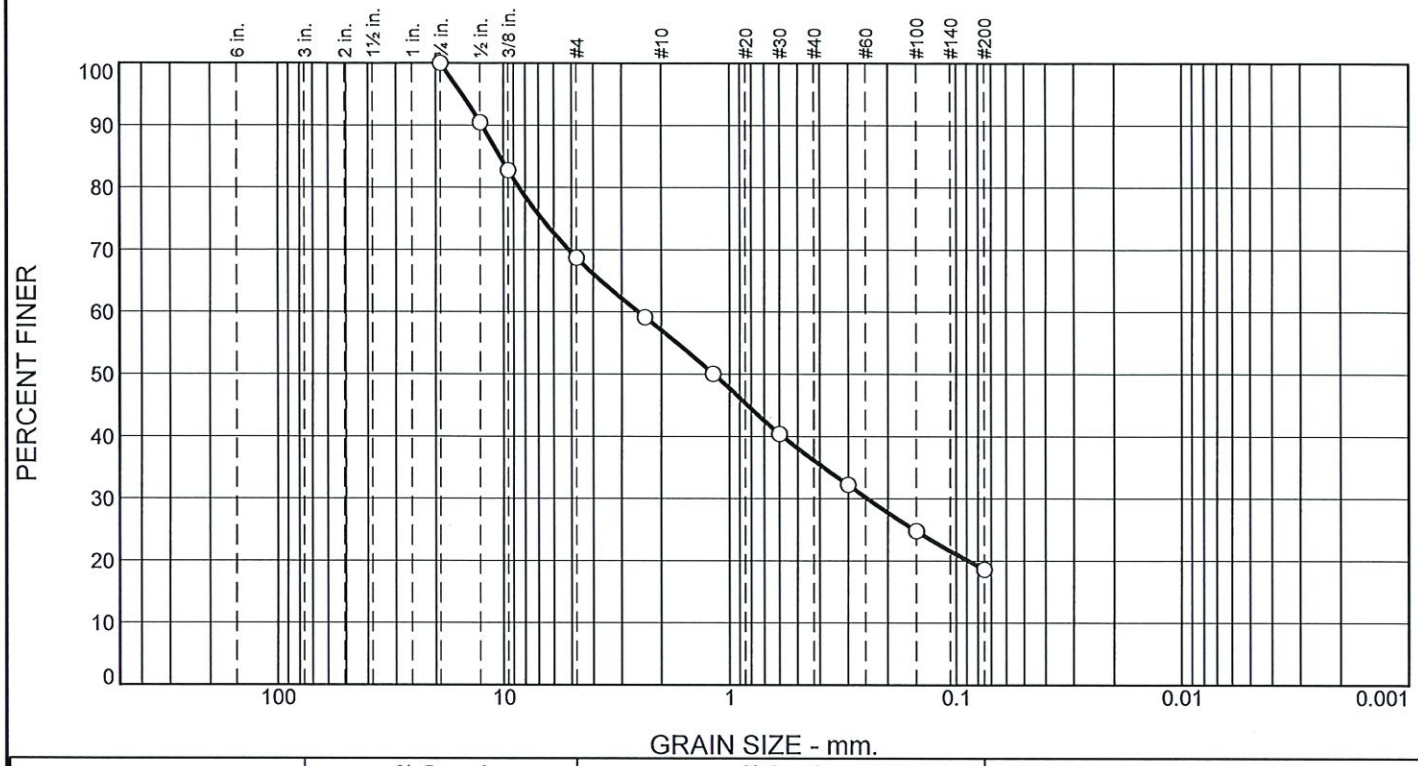
GULF SHORE EXPLORATION AND TESTING

Client: HDR, Inc.

Project: Bellotta Weir Modification

Project No: 21-105 **Figure**

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines
	Coarse	Fine	Coarse	Medium	Fine	
0	0	31	12	21	17	19

Test Results (ASTM D6913 & ASTM D1140)

Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
3/4 Inch	100		
1/2 Inch	90		
3/8 Inch	83		
#4	69		
#8	59		
#16	50		
#30	40		
#50	32		
#100	25		
#200	19		

* (no specification provided)

Material Description

Atterberg Limits (ASTM D 4318)

PL= 29 LL= 52 PI= 23

Classification

USCS (D 2487)= SC AASHTO (M 145)= A-2-7(1)

Coefficients

D_{90}= 12.4880 D_{85}= 10.3703 D_{60}= 2.5212
D_{50}= 1.1736 D_{30}= 0.2457 D_{15}=
D_{10}= C_u= C_c=

Remarks

Not enough sample for the max particle size (below the minimum mass requirement for specimen as required in ASTM). F.M.=3.42

Date Received: 2/9/21 **Date Tested:** 2/22/21

Tested By: JB

Checked By: JML

Title: PM

Location: HDR-B1: S11 **Date Sampled:** -

Sample Number: 62181 **Depth:** 35.0'-36.5'

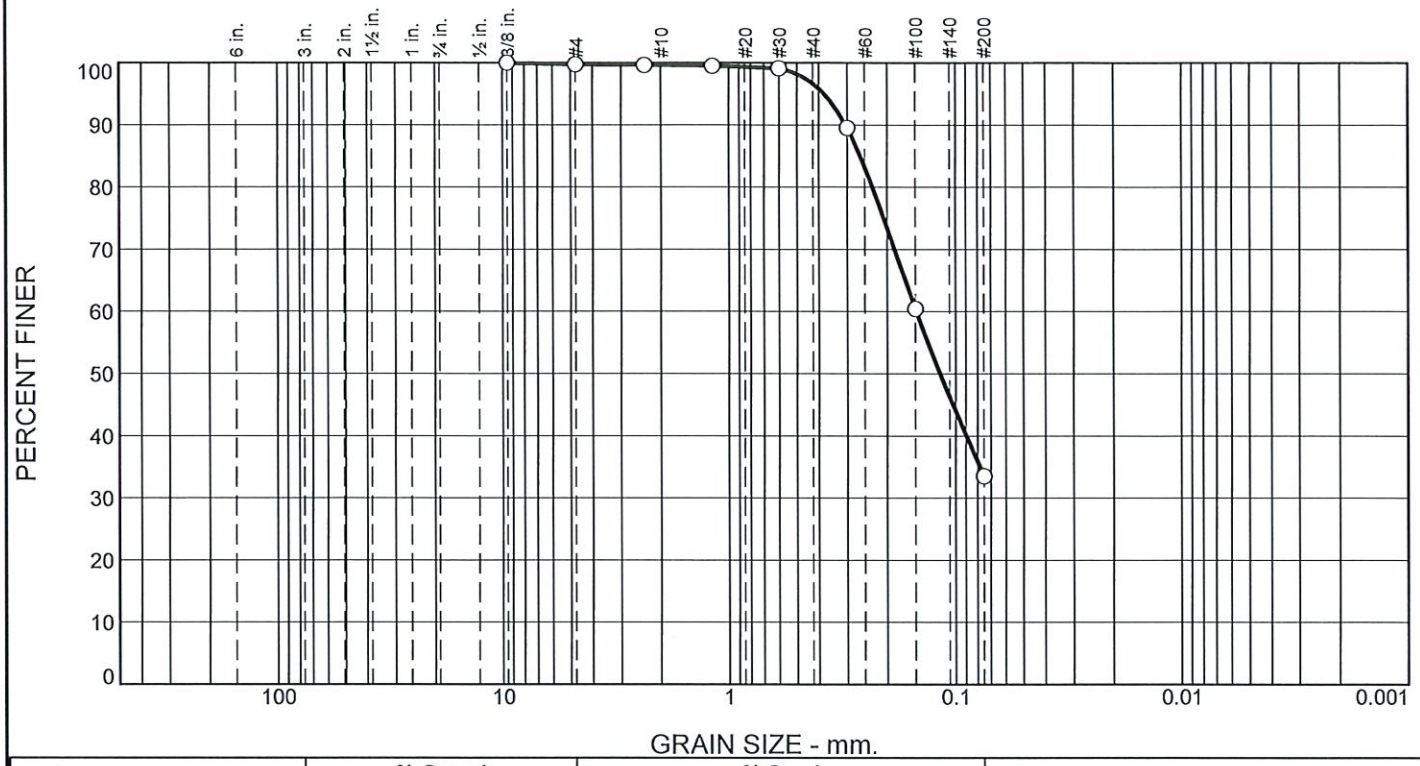
GULF SHORE EXPLORATION AND TESTING

Client: HDR, Inc.

Project: Bellotta Weir Modification

Project No: 21-105 **Figure**

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines
	Coarse	Fine	Coarse	Medium	Fine	
0	0	0	0	3	63	34

Test Results (ASTM D6913 & ASTM D1140)

Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
3/8 Inch	100		
#4	100		
#8	100		
#16	100		
#30	99		
#50	90		
#100	60		
#200	34		

* (no specification provided)

Material Description

Atterberg Limits (ASTM D 4318)

PL= LL= PI=

Classification

USCS (D 2487)= AASHTO (M 145)=

Coefficients

D_{90}= 0.3048 D_{85}= 0.2624 D_{60}= 0.1486
D_{50}= 0.1167 D_{30}= D_{15}=
D_{10}= C_u= C_c=

Remarks

F.M.=0.52

Date Received: 2/9/21 **Date Tested:** 2/22/21

Tested By: JB

Checked By: JML

Title: PM

Location: HDR-B2: S1 **Date Sampled:** -

Sample Number: 62183 **Depth:** 2.5'-3.0'

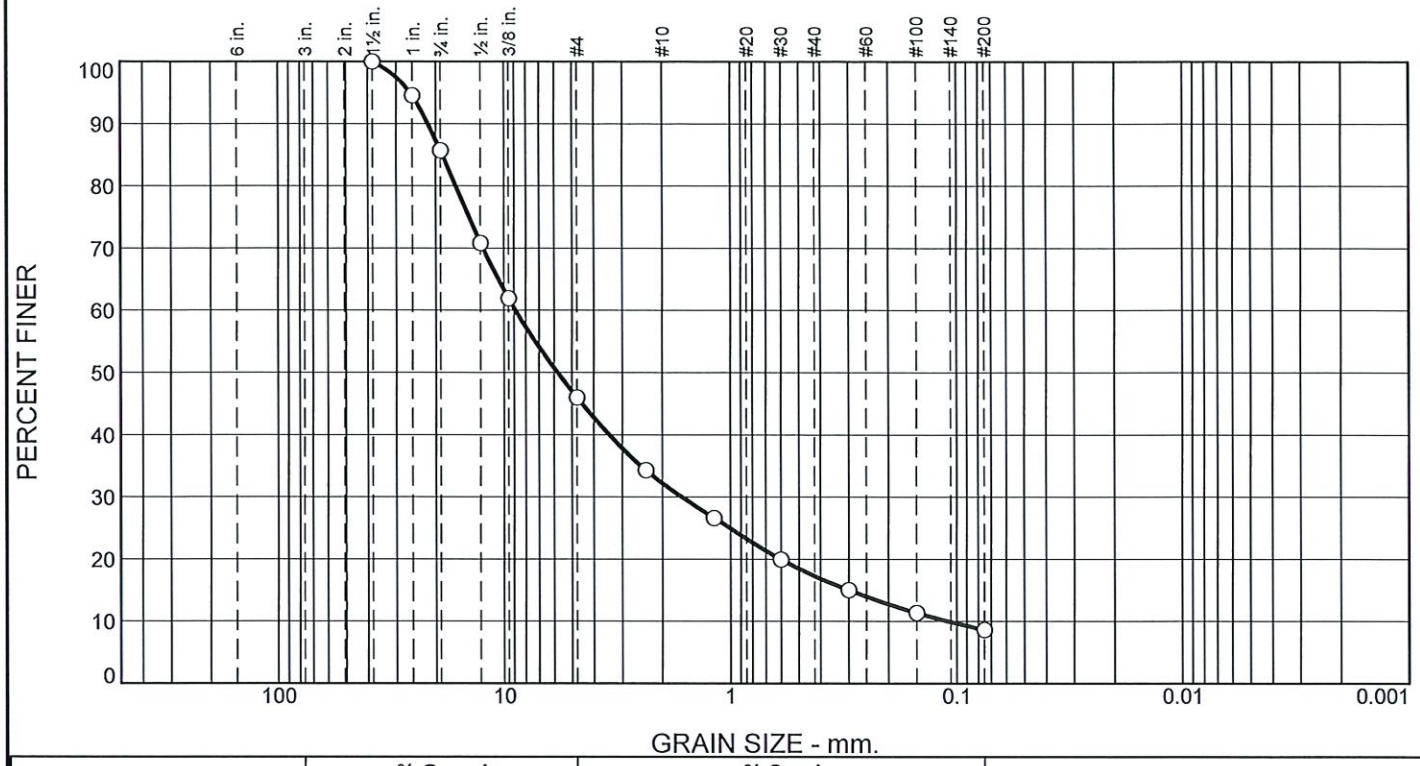
GULF SHORE EXPLORATION AND TESTING

Client: HDR, Inc.

Project: Bellotta Weir Modification

Project No: 21-105 **Figure**

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines
	Coarse	Fine	Coarse	Medium	Fine	
0	14	40	14	15	8	9

Test Results (ASTM D6913 & ASTM D1140)			
Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
1 1/2 Inch	100		
1 Inch	95		
3/4 Inch	86		
1/2 Inch	71		
3/8 Inch	62		
#4	46		
#8	34		
#16	27		
#30	20		
#50	15		
#100	11		
#200	8.6		

* (no specification provided)

Material Description

Atterberg Limits (ASTM D 4318)

PL= LL= PI=

Classification

USCS (D 2487)= AASHTO (M 145)=

Coefficients

D_{90}= 21.5995 D_{85}= 18.6961 D_{60}= 8.8784
D_{50}= 5.8017 D_{30}= 1.6425 D_{15}= 0.2991
D_{10}= 0.1087 C_u= 81.69 C_c= 2.80

Remarks

Not enough sample for the max particle size (below the minimum mass requirement for specimen as required in ASTM). F.M.=4.99

Date Received: 2/9/21 **Date Tested:** 2/23/21

Tested By: JB

Checked By: JML

Title: PM

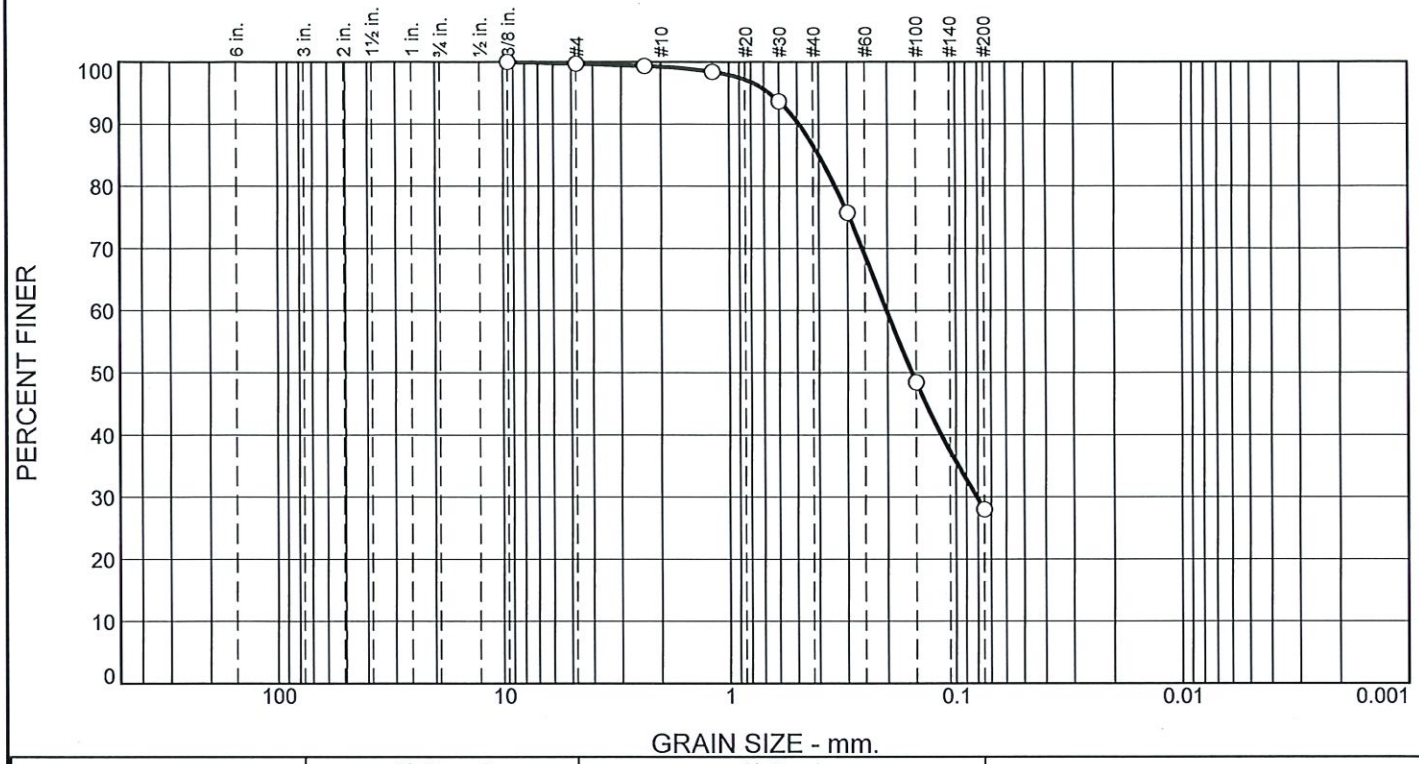
Location: HDR-B2: S10 **Date Sampled:** -
Sample Number: 62188 **Depth:** 35.0'-36.5'

GULF SHORE EXPLORATION AND TESTING

Client: HDR, Inc.
Project: Bellotta Weir Modification

Project No: 21-105 **Figure**

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines
	Coarse	Fine	Coarse	Medium	Fine	
0	0	0	1	12	59	28

Test Results (ASTM D6913 & ASTM D1140)

Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
3/8 Inch	100		
#4	100		
#8	99		
#16	98		
#30	94		
#50	76		
#100	48		
#200	28		

* (no specification provided)

Material Description

Atterberg Limits (ASTM D 4318)

PL= LL= PI=

Classification

USCS (D 2487)= AASHTO (M 145)=

Coefficients

D_{90}= 0.4907 D_{85}= 0.4008 D_{60}= 0.2016
D_{50}= 0.1564 D_{30}= 0.0809 D_{15}=
D_{10}= C_u= C_c=

Remarks

F.M.=0.85

Date Received: 2/9/21 **Date Tested:** 2/22/21

Tested By: JB

Checked By: JML

Title: PM

Location: HDR-B4: S6 **Date Sampled:** -

Sample Number: 62202 **Depth:** 15.0'-16.5'

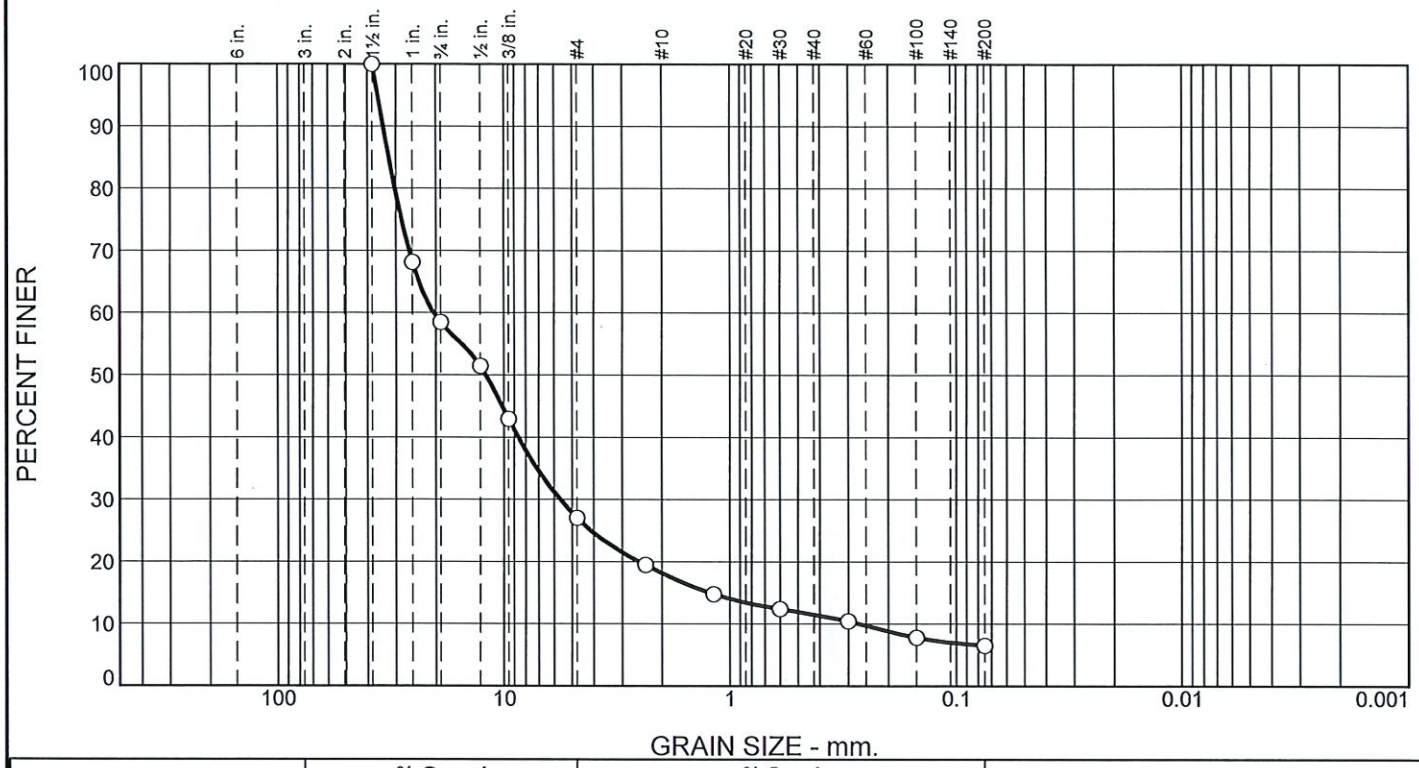
GULF SHORE EXPLORATION AND TESTING

Client: HDR, Inc.

Project: Bellotta Weir Modification

Project No: 21-105 **Figure**

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines
	Coarse	Fine	Coarse	Medium	Fine	
0	42	31	9	7	4	7

Test Results (ASTM D6913 & ASTM D1140)

Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
1 1/2 Inch	100		
1 Inch	68		
3/4 Inch	58		
1/2 Inch	51		
3/8 Inch	43		
#4	27		
#8	19		
#16	15		
#30	12		
#50	10		
#100	8		
#200	6.5		

* (no specification provided)

Material Description

Atterberg Limits (ASTM D 4318)

PL= LL= PI=

Classification

USCS (D 2487)= AASHTO (M 145)=

Coefficients

D_{90}= 34.0913 D_{85}= 32.1742 D_{60}= 20.4269
D_{50}= 12.0038 D_{30}= 5.6494 D_{15}= 1.2289
D_{10}= 0.2685 C_u= 76.07 C_c= 5.82

Remarks

Not enough sample for the max particle size (below the minimum mass requirement for specimen as required in ASTM). F.M.=6.07

Date Received: 2/9/21 **Date Tested:** 2/22/21

Tested By: JB

Checked By: JML

Title: PM

Location: HDR-B4: S10 **Date Sampled:** -

Sample Number: 62204 **Depth:** 35.0'-36.5'

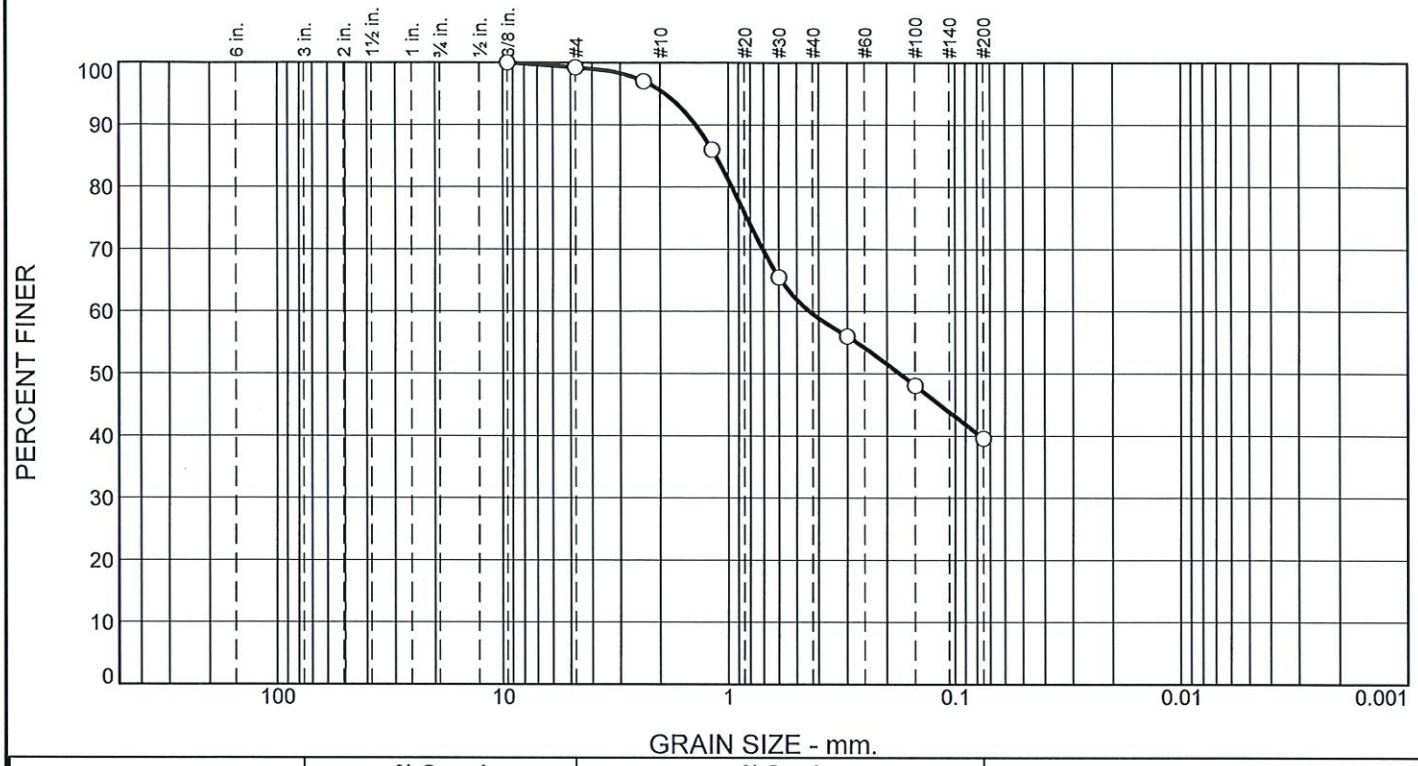
GULF SHORE EXPLORATION AND TESTING

Client: HDR, Inc.

Project: Bellotta Weir Modification

Project No: 21-105 **Figure**

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines
	Coarse	Fine	Coarse	Medium	Fine	
0	0	1	3	36	20	40

Test Results (ASTM D6913 & ASTM D1140)

Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
3/8 Inch	100		
#4	99		
#8	97		
#16	86		
#30	66		
#50	56		
#100	48		
#200	40		

* (no specification provided)

Material Description

Atterberg Limits (ASTM D 4318)

PL= LL= PI=

Classification

USCS (D 2487)= AASHTO (M 145)=

Coefficients

D_{90}= 1.3937 D_{85}= 1.1366 D_{60}= 0.4400
D_{50}= 0.1754 D_{30}= D_{15}=
D_{10}= C_u= C_c=

Remarks

F.M.=1.48

Date Received: 2/9/21 **Date Tested:** 2/22/21

Tested By: JB

Checked By: JML

Title: PM

Location: HDR-B4: S17 **Date Sampled:** -

Sample Number: 62207 **Depth:** 65.0'-66.5'

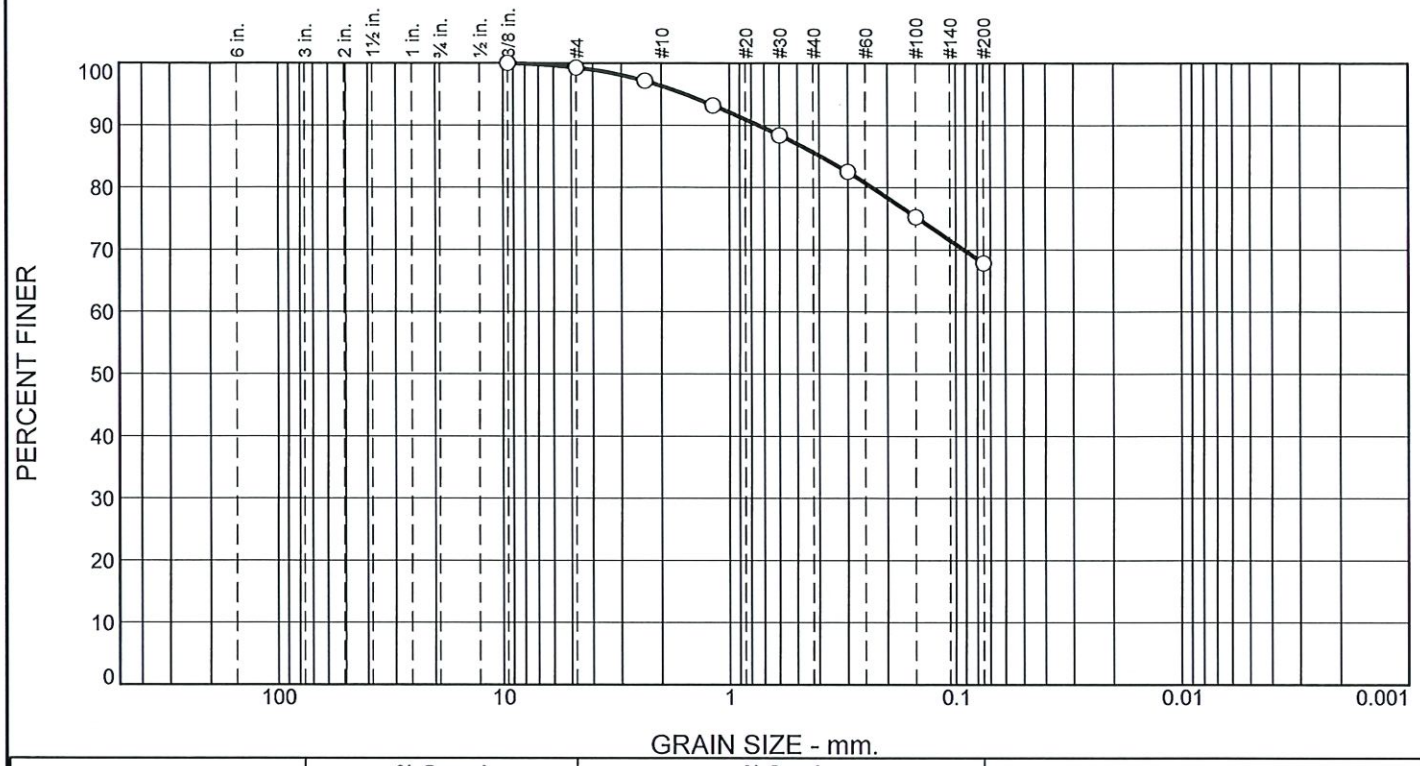
GULF SHORE EXPLORATION AND TESTING

Client: HDR, Inc.

Project: Bellotta Weir Modification

Project No: 21-105 **Figure**

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines
	Coarse	Fine	Coarse	Medium	Fine	
0	0	1	3	10	18	68

Test Results (ASTM D6913 & ASTM D1140)

Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
3/8 Inch	100		
#4	99		
#8	97		
#16	93		
#30	88		
#50	83		
#100	75		
#200	68		

* (no specification provided)

Material Description

Atterberg Limits (ASTM D 4318)

PL= LL= PI=

Classification

USCS (D 2487)= AASHTO (M 145)=

Coefficients

D_{90}= 0.7435 D_{85}= 0.3940 D_{60}=
D_{50}= D_{30}= D_{15}=
D_{10}= C_u= C_c=

Remarks

F.M.=0.64

Date Received: 2/9/21 **Date Tested:** 2/23/21

Tested By: JB

Checked By: JML

Title: PM

Location: HDR-B5: S11 **Date Sampled:** -

Sample Number: 62211 **Depth:** 40.0'-41.5'

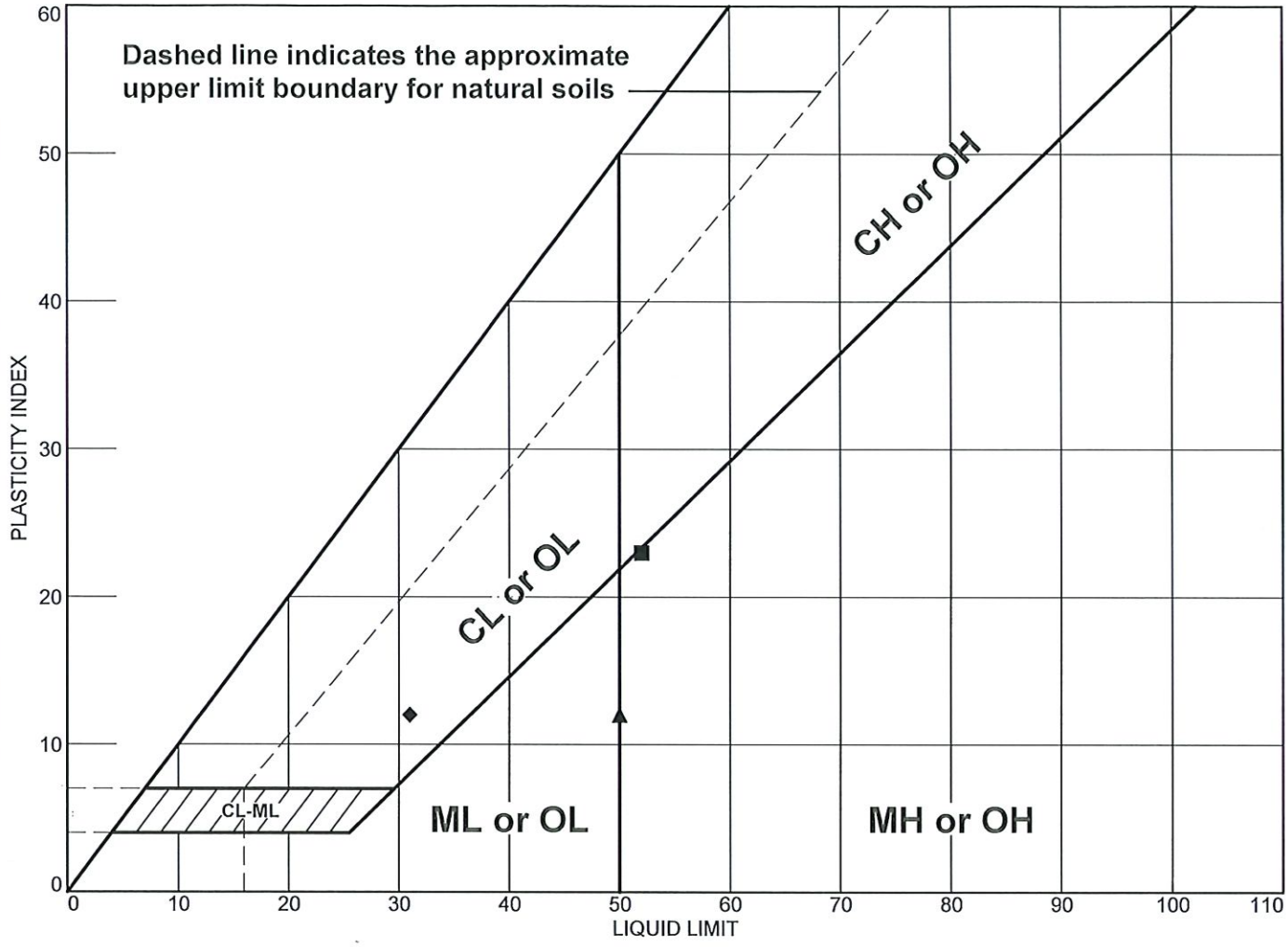
GULF SHORE EXPLORATION AND TESTING

Client: HDR, Inc.

Project: Bellotta Weir Modification

Project No: 21-105 **Figure**

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●		NV	NP	NP			
■		52	29	23	36	19	SC
▲		50	38	12			
◆		31	19	12			

Project No. 21-105 **Client:** HDR, Inc.

Project: Bellotta Weir Modification

● **Location:** HDR-B1: S5 **Depth:** 12.5'-14.0' **Sample Number:** 62177

■ **Location:** HDR-B1: S11 **Depth:** 35.0'-36.5' **Sample Number:** 62181

▲ **Location:** HDR-B1: S14 **Depth:** 50.0'-51.5' **Sample Number:** 62182

◆ **Location:** HDR-B1: S3 **Depth:** 7.5'-9.0' **Sample Number:** 62175

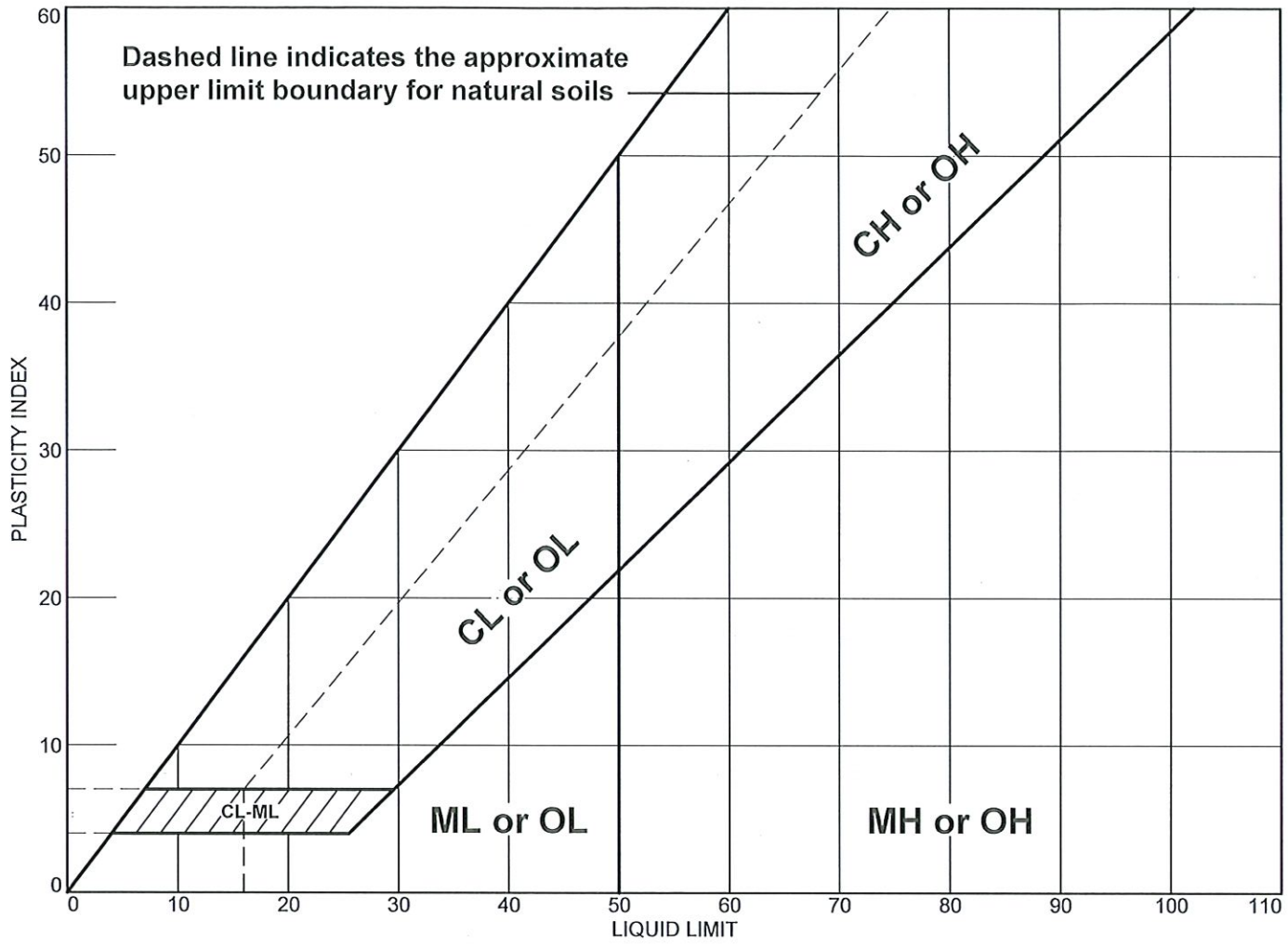
GS GULF SHORE EXPLORATION AND TESTING

Remarks:

Figure

Tested By: BGS **Checked By:** JML

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●		NV	NP	NP		13	

Project No. 21-105 **Client:** HDR, Inc.

Project: Bellotta Weir Modification

● **Location:** HDR-B1: S7 **Depth:** 17.5'-19.0' **Sample Number:** 62178

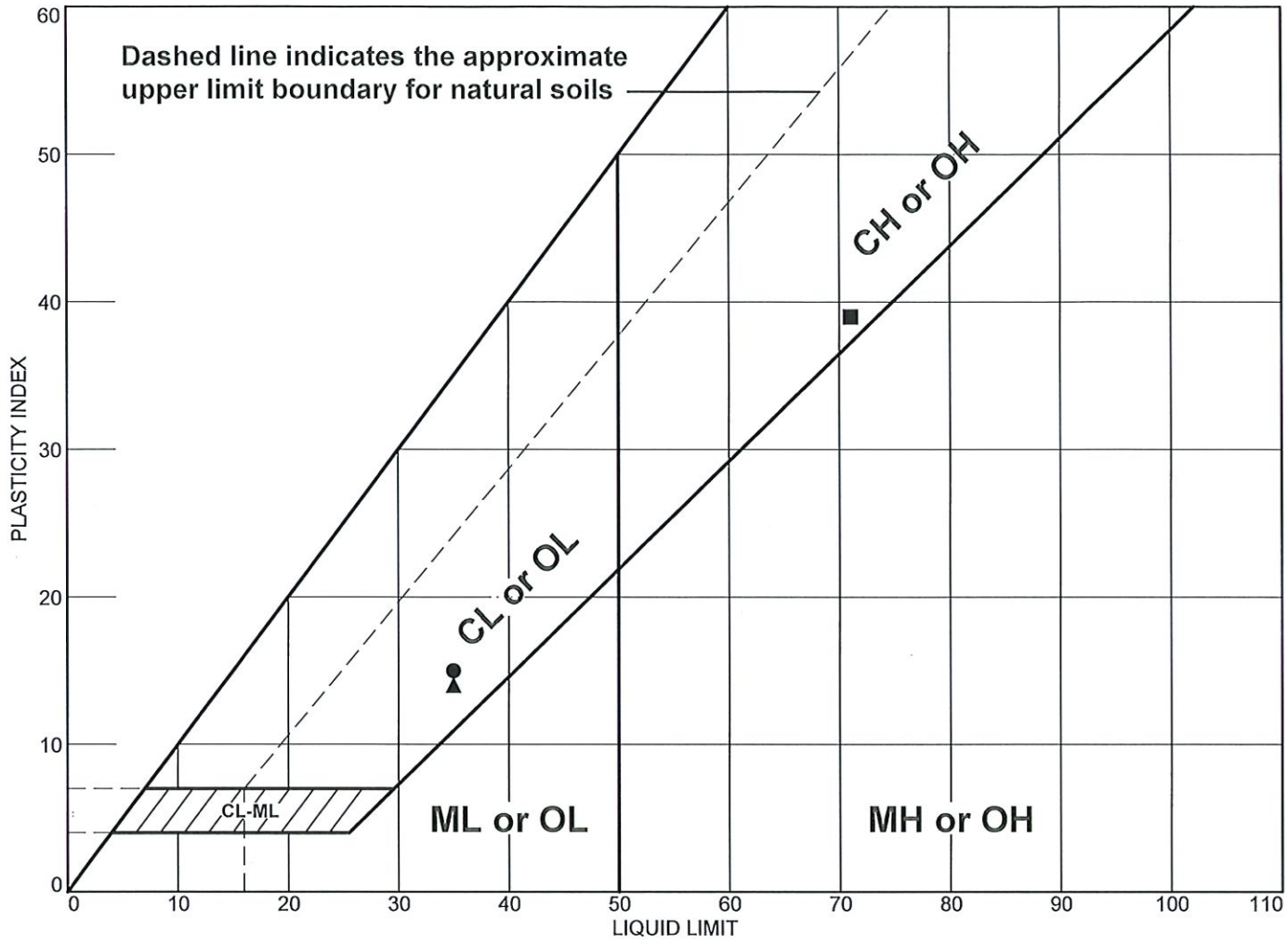
Remarks:

GULF SHORE EXPLORATION AND TESTING

Figure

Tested By: BGS **Checked By:** JML

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●		35	20	15			
■		71	32	39		74	
▲		35	21	14			

Project No. 21-105 **Client:** HDR, Inc.

Project: Bellotta Weir Modification

● **Location:** HDR-B2: S4 **Depth:** 10.0'-11.5' **Sample Number:** 62185

■ **Location:** HDR-B2: S15 **Depth:** 55.0'-56.5' **Sample Number:** 62190

▲ **Location:** HDR-B2: S6 **Depth:** 15.0'-16.5' **Sample Number:** 62186

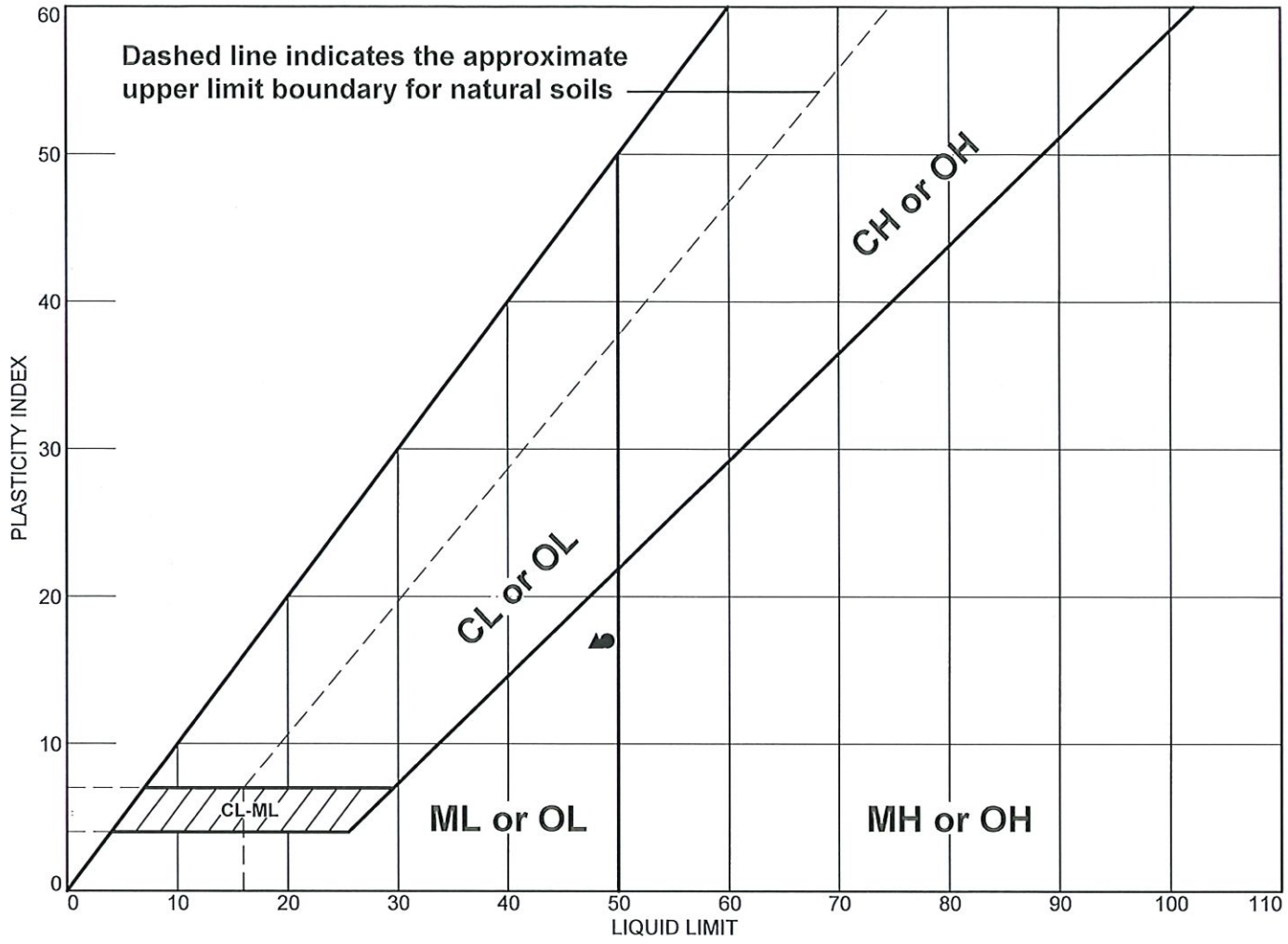
Remarks:

GS GULF SHORE EXPLORATION AND TESTING

Figure

Tested By: BGS **Checked By:** JML

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●		49	32	17			
■		NV	NP	NP			
▲		48	31	17			

Project No. 21-105 **Client:** HDR, Inc.

Project: Bellota Weir Modification

● **Location:** HDR-B3: S11 **Depth:** 40.0'-41.5' **Sample Number:** 62196

■ **Location:** HDR-B3: S2 **Depth:** 5.0'-6.5' **Sample Number:** 62191

▲ **Location:** HDR-B3: S14 **Depth:** 55.0'-56.5' **Sample Number:** 62197

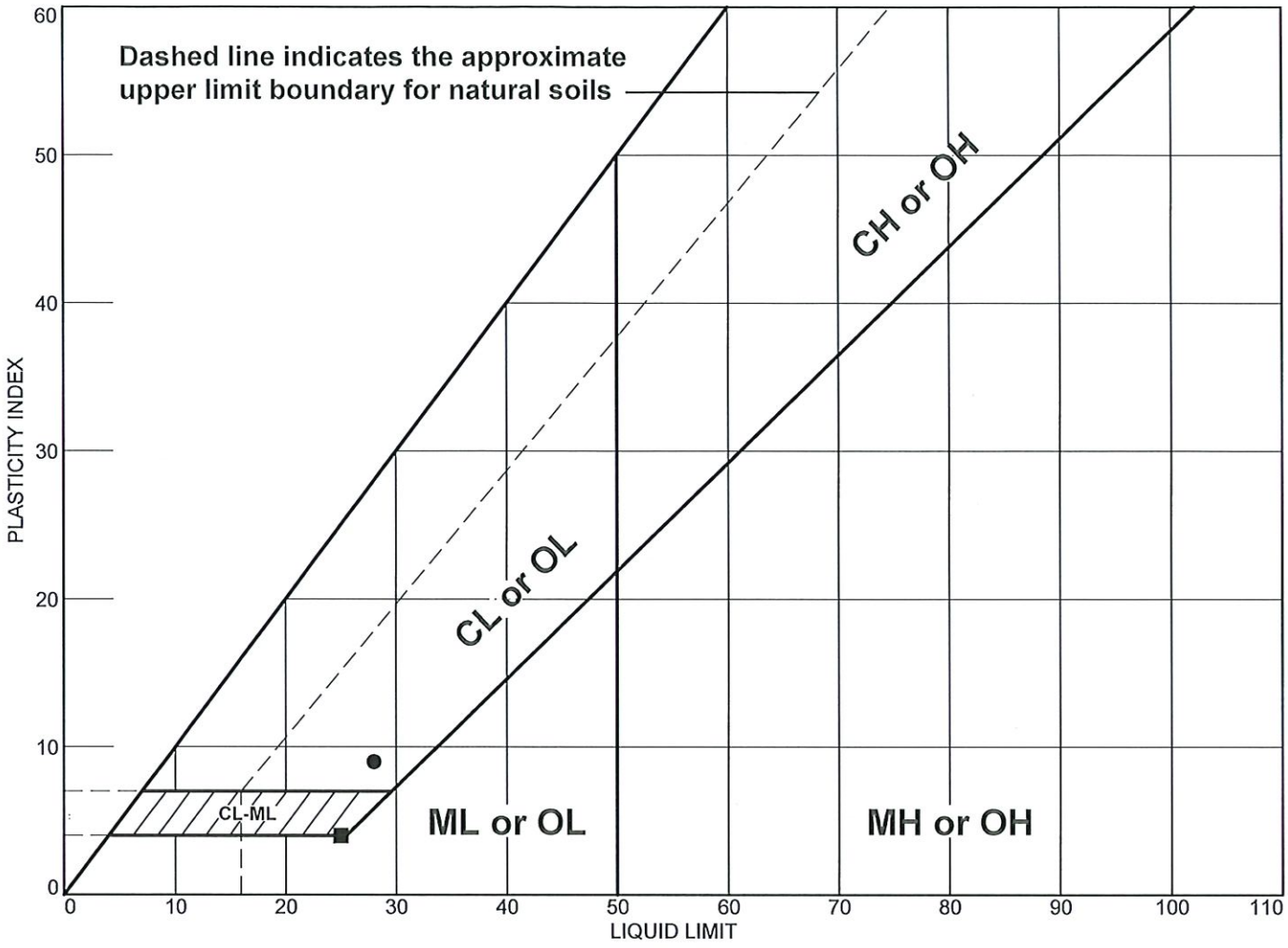
GS GULF SHORE EXPLORATION AND TESTING

Remarks:

Figure

Tested By: BGS **Checked By:** JML

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●		28	19	9			
■		25	21	4			

Project No. 21-105 **Client:** HDR, Inc.

Project: Bellotta Weir Modification

● **Location:** HDR-B4: S3 **Depth:** 7.5'-9.0' **Sample Number:** 62200

■ **Location:** HDR-B4: S7 **Depth:** 20.0'-21.5' **Sample Number:** 62203

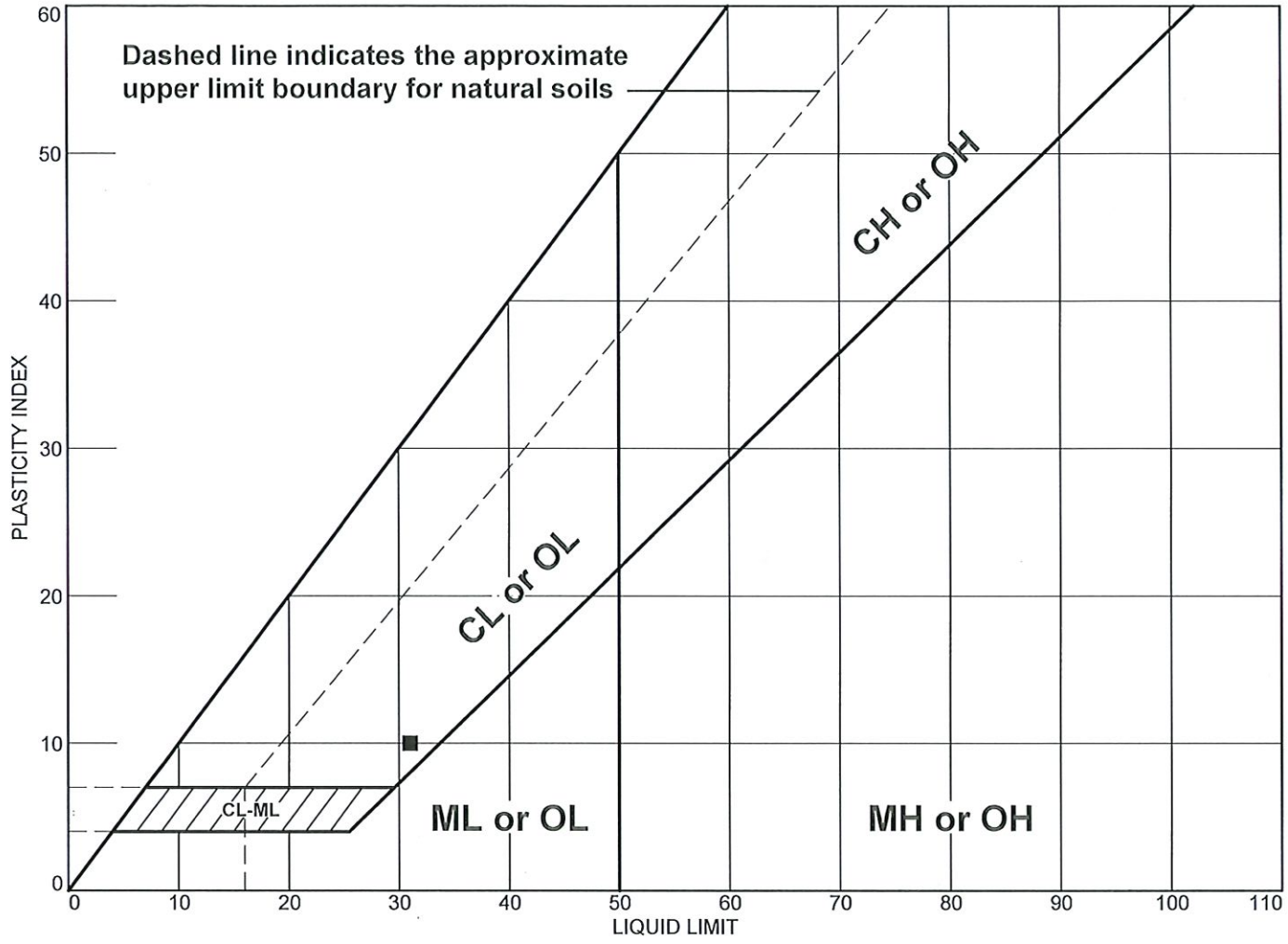
Remarks:

GULF SHORE EXPLORATION AND TESTING

Figure

Tested By: BGS **Checked By:** JML

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●		NV	NP	NP			
■		31	21	10			

Project No. 21-105 **Client:** HDR, Inc.

Project: Bellotta Weir Modification

● **Location:** HDR-B5: S2 **Depth:** 5.0'-6.5' **Sample Number:** 62209

■ **Location:** HDR-B5: S7 **Depth:** 20.0'-21.5' **Sample Number:** 62210

Remarks:

GULF SHORE EXPLORATION AND TESTING

Figure

Tested By: BGS **Checked By:** JML

MOISTURE CONTENT & UNIT WEIGHT TEST RESULTS

Sample Identification	Depth, ft.	Wet Unit Weight, lb/ft.^{3}	Dry Unit Weight, lb/ft.^{3}	Moisture Content, %
HDR-B1: S3	7.5'-9.0'			15.0
HDR-B1: S4	10.0'-11.5'	122.7	106.6	15.1
HDR-B1: S7	17.5'-19.0'			27.6
HDR-B1: S9	25.0'-26.5'			16.9
HDR-B1: S11	35.0'-36.5'			28.1
HDR-B2: S2	5.0'-6.5'			8.1
HDR-B2: S4	10.0'-11.5'	104.9	90.3	16.2
HDR-B2: S6	15.0'-16.5'	112.6	97.3	15.7
HDR-B3: S6	15.0'-16.5'	114.0	98.9	15.2

Test Method: ASTM D2216, ASTM D2937

PROJECT NUMBER:	21-105	February 18, 2021	Bellota Weir Modification

GULF SHORE EXPLORATION AND TESTING
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Rancho Cordova, CA 95742
Phone: (916) 939-4117
FAX: (916) 635-4315

MOISTURE CONTENT & UNIT WEIGHT TEST RESULTS

Sample Identification	Depth, ft.	Wet Unit Weight, lb/ft.^{3}	Dry Unit Weight, lb/ft.^{3}	Moisture Content, %
HDR-B4: S4	10.0'-11.5'	111.8	98.0	14.1
HDR-B5: S7	20.0'-21.5'	117.9	89.4	31.9
HDR-B5: S11	40.0'-41.5'	114.6	71.9	59.4
HDR-B5: S14	52.5'-54.0'	124.9	89.7	39.2

Test Method: ASTM D2216, ASTM D2937

PROJECT NUMBER:	21-105	February 18, 2021	Bellota Weir Modification

GULF SHORE EXPLORATION AND TESTING
3362 Fitzgerald Road
Rancho Cordova, CA 95742
Phone: (916) 939-4117
FAX: (916) 635-4315

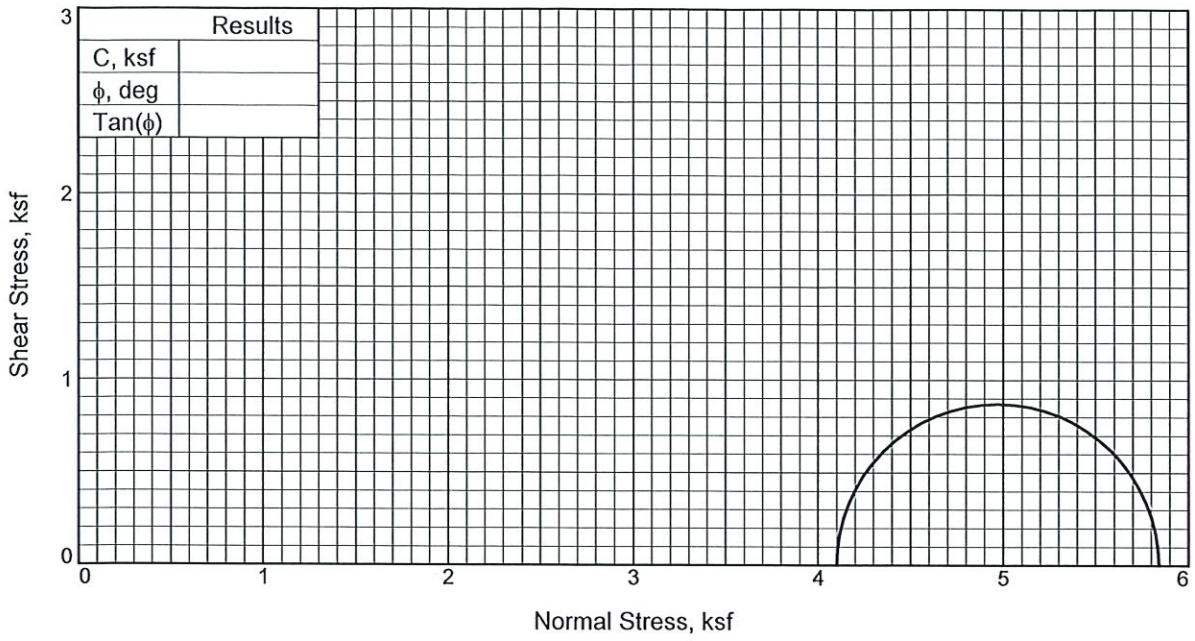
MOISTURE AND ORGANIC CONTENT TEST RESULTS

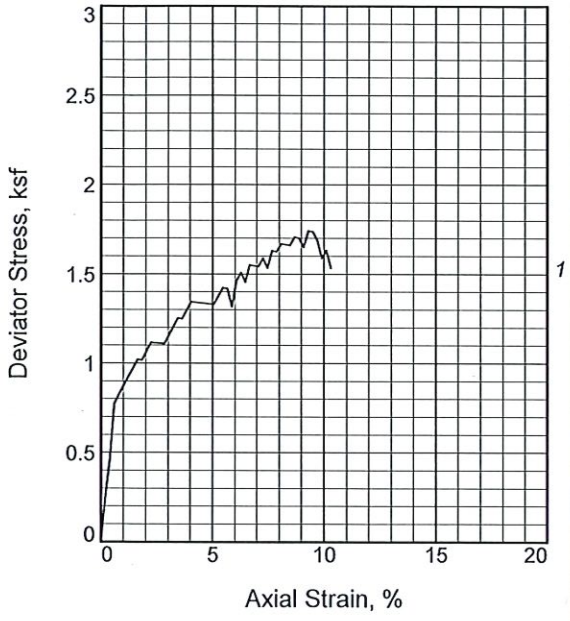
Sample Identification	Depth, ft.	Organic Content, %	Moisture Content, %
HDR-B4: S1	2.5'-4.0'	1.5	3.8

Test Method: ASTM D2974

PROJECT NUMBER:	21-105	February 18, 2021	Bellota Weir Modification

GULF SHORE EXPLORATION AND TESTING
3362 Fitzgerald Road
Rancho Cordova, CA 95742
Phone: (916) 939-4117
FAX: (916) 635-4315





Sample No.		1
Initial	Water Content, %	41.9
	Dry Density, pcf	78.9
	Saturation, %	99.6
	Void Ratio	1.1354
	Diameter, in.	1.88
	Height, in.	4.95
At Test	Water Content, %	42.1
	Dry Density, pcf	78.9
	Saturation, %	100.0
	Void Ratio	1.1354
	Diameter, in.	1.88
	Height, in.	4.95
Strain rate, in./min.		0.008
Back Pressure, psi		0.00
Cell Pressure, psi		28.47
Fail. Stress, ksf		1.74
Ult. Stress, ksf		
σ_1 Failure, ksf		5.84
σ_3 Failure, ksf		4.10

Type of Test:
Unconsolidated Undrained

Sample Type: -

Description:

Assumed Specific Gravity= 2.70

Remarks:

Figure _____

Client: HDR, Inc.

Project: Bellotta Weir Modification

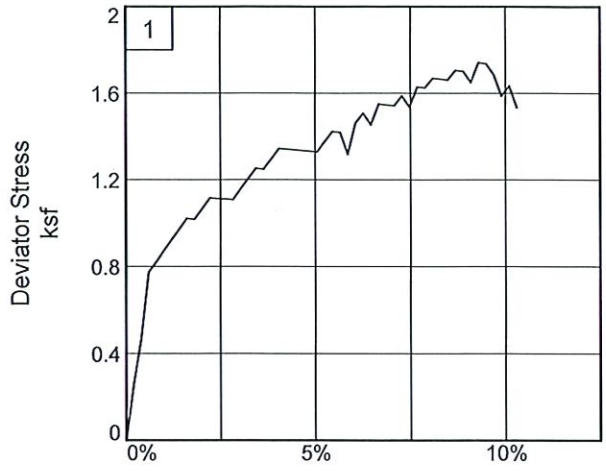
Location: HDR-B4: S14

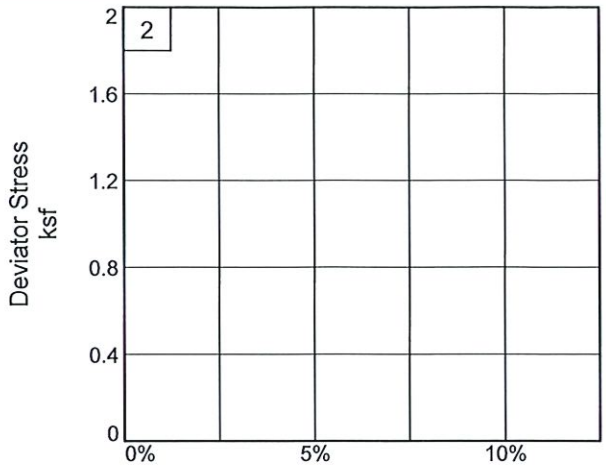
Sample Number: 62205 **Depth:** 50.0'-51.5'

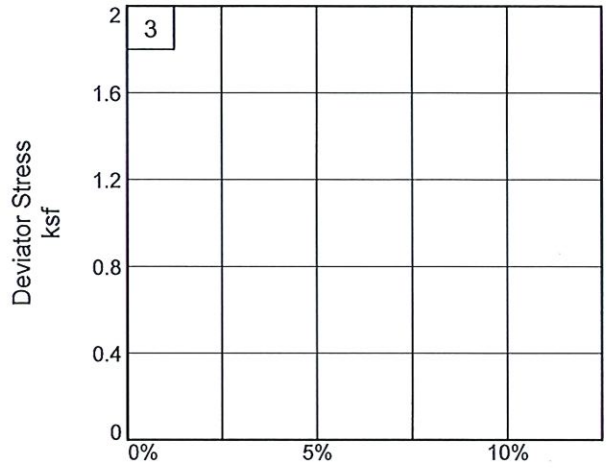
Proj. No.: 21-105 **Date Sampled:** -

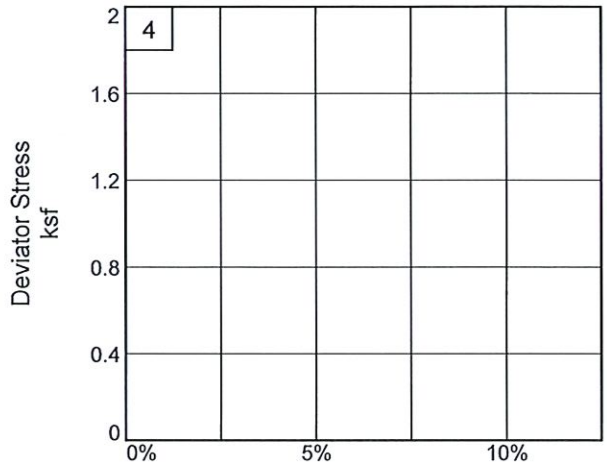
GS GULF SHORE EXPLORATION AND TESTING

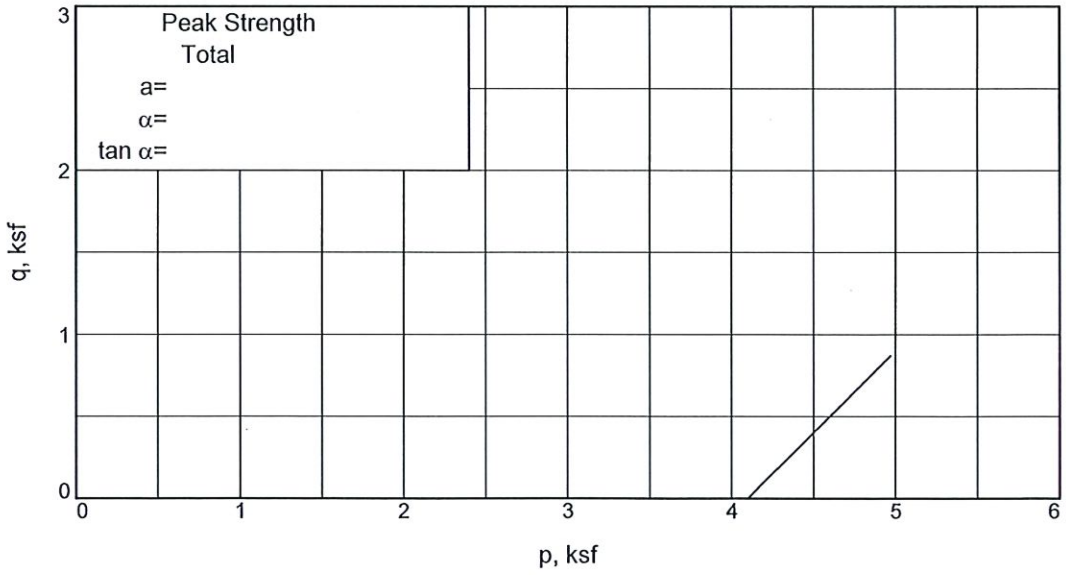
Tested By: MPW **Checked By:** JML







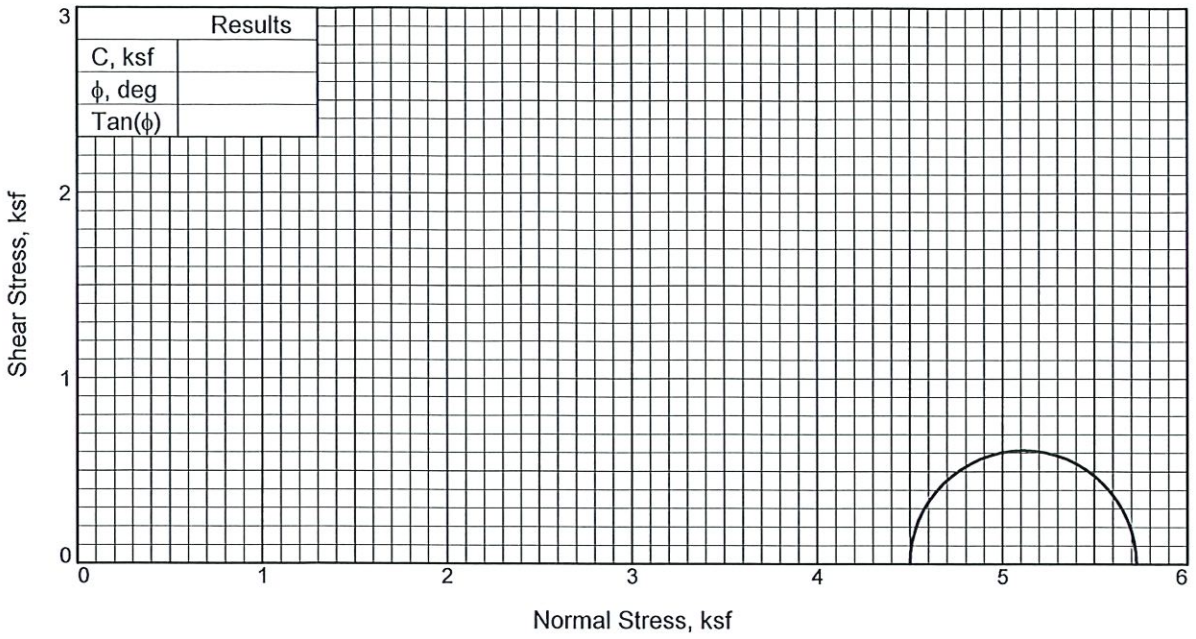


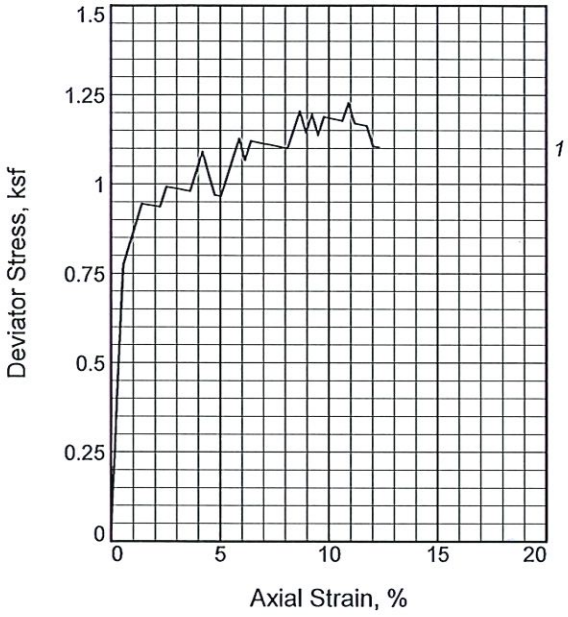


Client: HDR, Inc.
Project: Bellotta Weir Modification
Location: HDR-B4: S14 **Depth:** 50.0'-51.5' **Sample Number:** 62205
Project No.: 21-105 **Figure** _____

Gulf Shore Construction Services, LLC

Tested By: MPW **Checked By:** JML





Sample No.		1
Initial	Water Content, %	32.3
	Dry Density, pcf	89.9
	Saturation, %	99.6
	Void Ratio	0.8750
	Diameter, in.	1.75
	Height, in.	3.58
At Test	Water Content, %	32.4
	Dry Density, pcf	89.9
	Saturation, %	100.0
	Void Ratio	0.8750
	Diameter, in.	1.75
	Height, in.	3.58
Strain rate, in./min.		0.008
Back Pressure, psi		0.00
Cell Pressure, psi		31.25
Fail. Stress, ksf		1.23
Ult. Stress, ksf		
σ_1 Failure, ksf		5.73
σ_3 Failure, ksf		4.50

Type of Test:
Unconsolidated Undrained

Sample Type: -

Description:

Assumed Specific Gravity= 2.70

Remarks:

Figure _____

Client: HDR, Inc.

Project: Bellotta Weir Modification

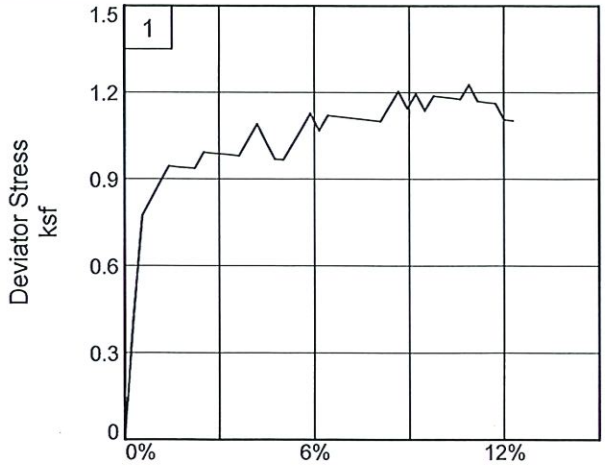
Location: HDR-B4: S16

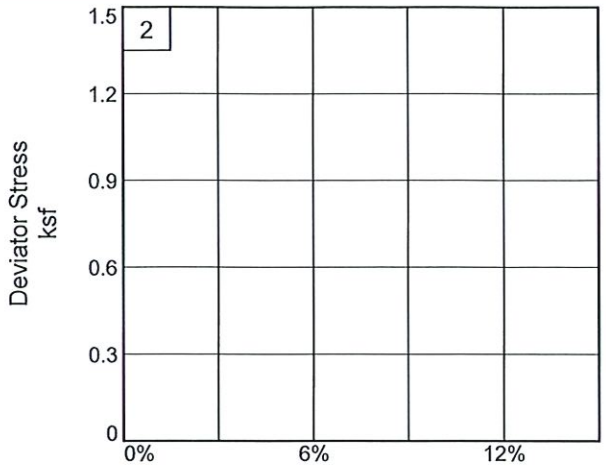
Sample Number: 62206 **Depth:** 60.0'-61.5'

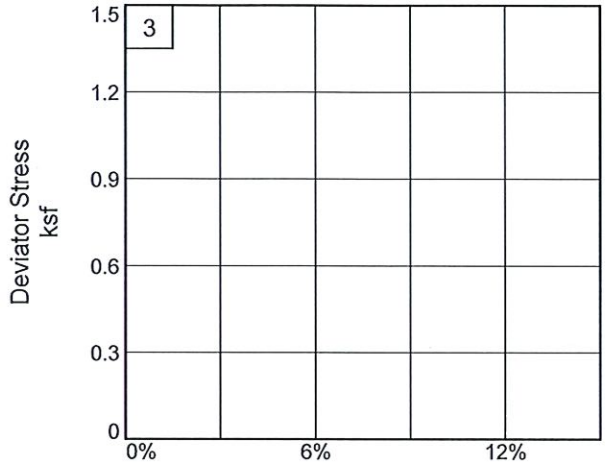
Proj. No.: 21-105 **Date Sampled:** -

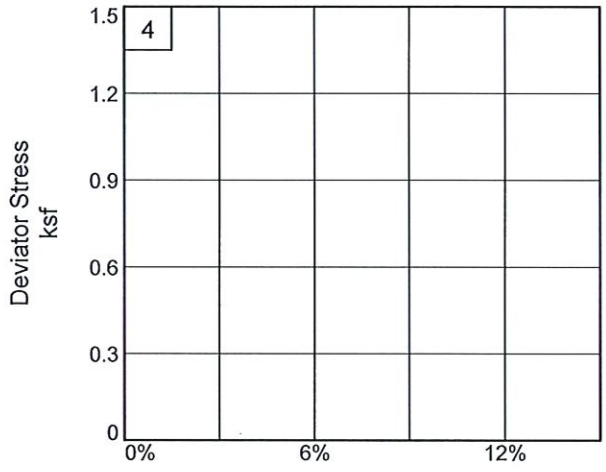
GS GULF SHORE EXPLORATION AND TESTING

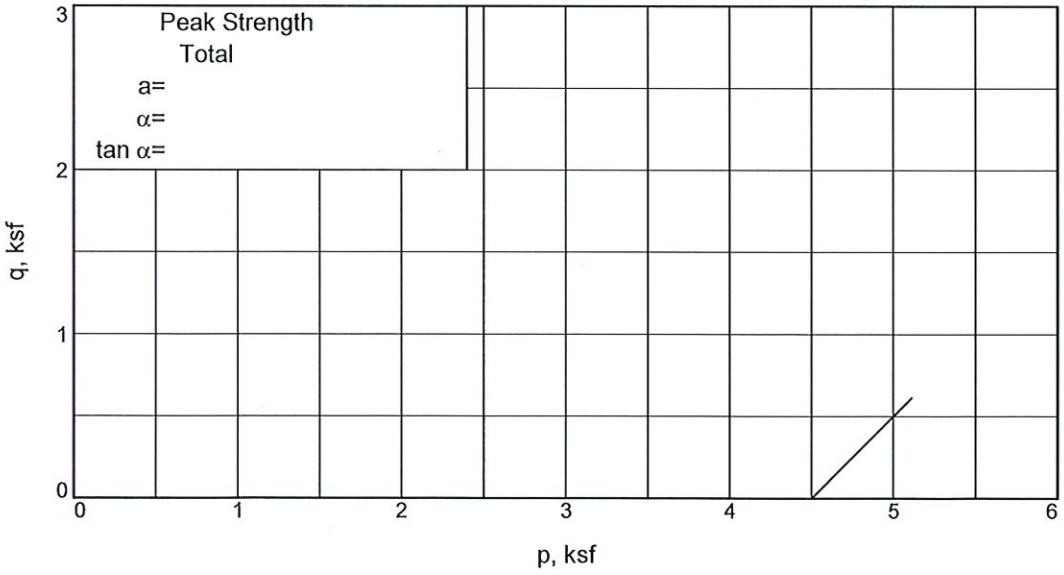
Tested By: MPW **Checked By:** JML









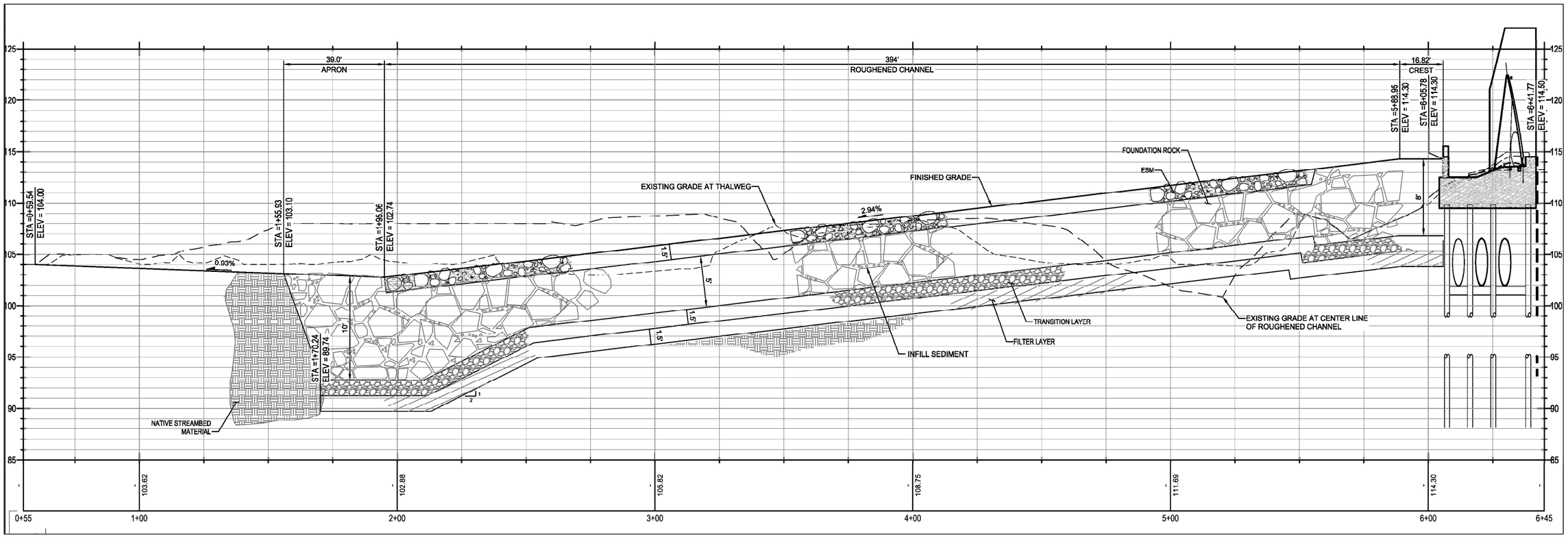


Client: HDR, Inc.
Project: Bellotta Weir Modification
Location: HDR-B4: S16 **Depth:** 60.0'-61.5' **Sample Number:** 62206
Project No.: 21-105 **Figure** _____

Gulf Shore Construction Services, LLC

Tested By: MPW **Checked By:** JML

Appendix E. Weir Seepage and Stability Analysis



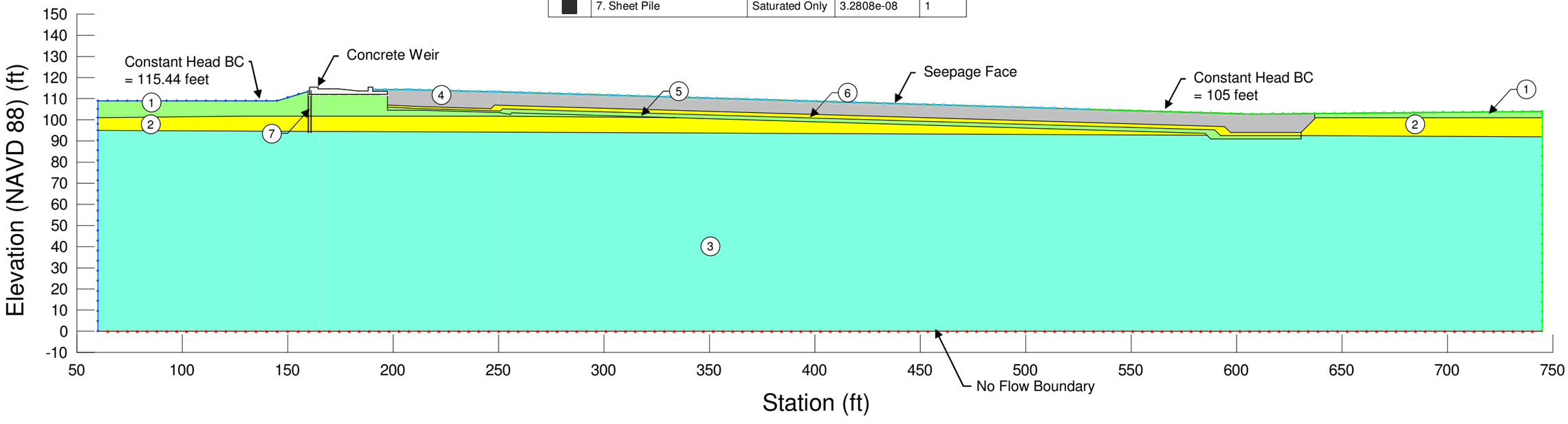
NOTES: Source: Sheet RI201 of Bellota Weir Modifications Project, 90% design submittal

Bellota Weir Modification Project

Bellota Weir Roughened Channel Downstream Profile

Feb 2022 FIGURE E-1

Color	Name	Material Model	Sat Kx (ft/sec)	Ky'/Kx' Ratio
	1. Alluvium (Upper Unit) SM	Saturated Only	3.28084e-06	0.25
	2. Alluvium (Lower Unit) SP	Saturated Only	0.000328084	0.25
	3. Older Alluvium ML/SM	Saturated Only	3.2808e-07	0.25
	4. Foundation Rocks Cobbles	Saturated Only	0.328084	1
	5. Filter SP	Saturated Only	0.0164042	1
	6. Transition SM	Saturated Only	3.28084e-06	0.25
	7. Sheet Pile	Saturated Only	3.2808e-08	1



NOTES: BC = Boundary Condition

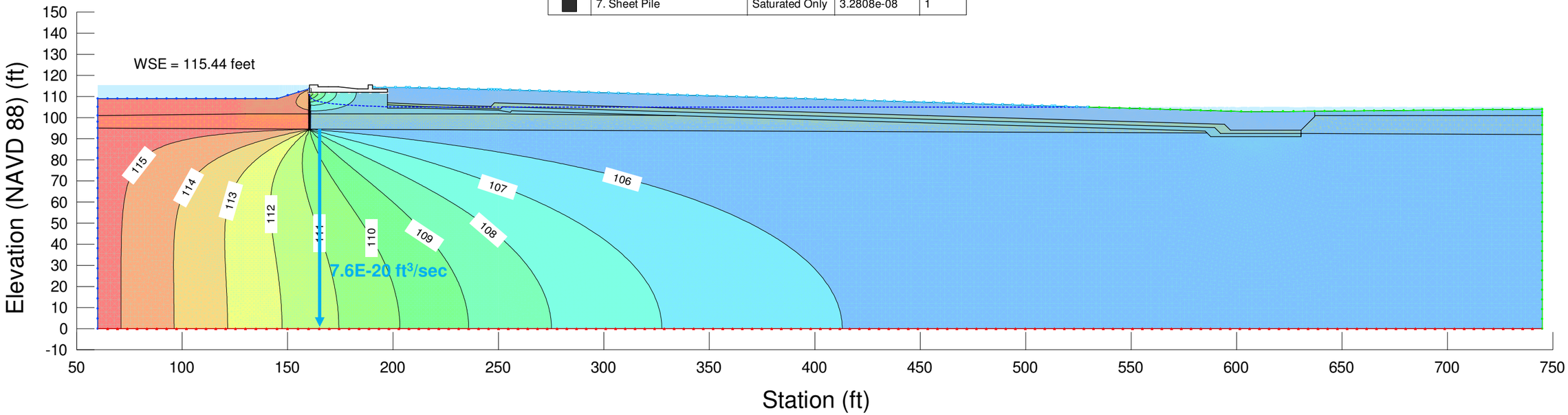
Bellota Weir Modification Project

Steady State Seepage

Bellota Weir Steady State Seepage Model D/S WSE 105 ft

Feb 2022 FIGURE E-2

Color	Name	Material Model	Sat Kx (ft/sec)	Ky'/Kx' Ratio
	1. Alluvium (Upper Unit) SM	Saturated Only	3.28084e-06	0.25
	2. Alluvium (Lower Unit) SP	Saturated Only	0.000328084	0.25
	3. Older Alluvium ML/SM	Saturated Only	3.2808e-07	0.25
	4. Foundation Rocks Cobbles	Saturated Only	0.328084	1
	5. Filter SP	Saturated Only	0.0164042	1
	6. Transition SM	Saturated Only	3.28084e-06	0.25
	7. Sheet Pile	Saturated Only	3.2808e-08	1



NOTES:

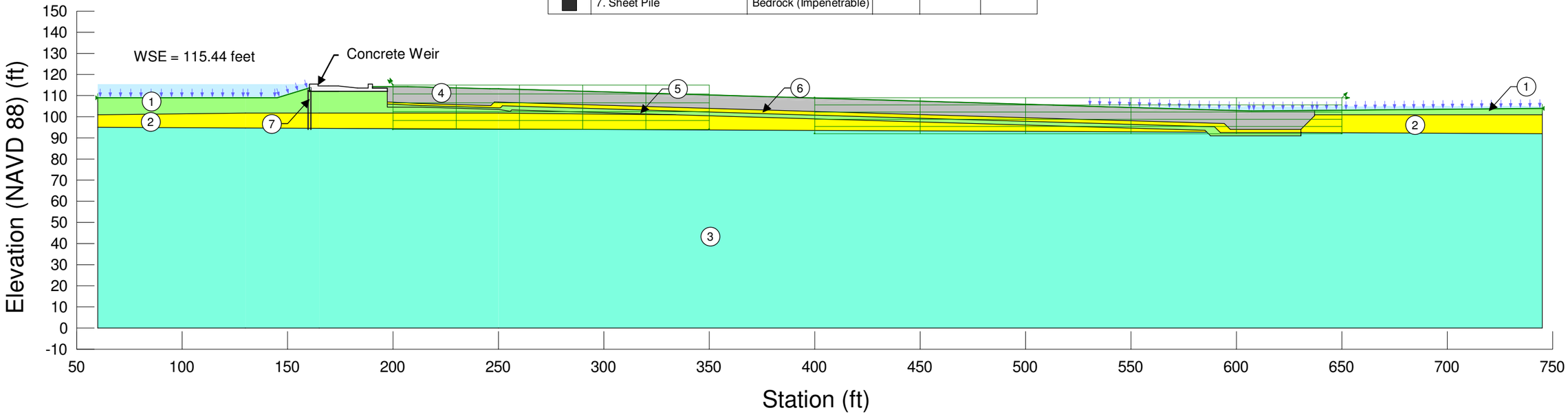
Bellota Weir Modification Project

Steady State Seepage

Bellota Weir Steady State Seepage Result D/S WSE 105 ft

Feb 2022 FIGURE E-3

Color	Name	Material Model	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)
	1. Alluvium (Upper Unit) SM	Mohr-Coulomb	120	50	30
	2. Alluvium (Lower Unit) SP	Mohr-Coulomb	120	0	32
	3. Older Alluvium ML/SM	Mohr-Coulomb	120	100	30
	4. Foundation Rocks Cobbles	Mohr-Coulomb	140	0	40
	5. Filter SP	Mohr-Coulomb	120	0	33
	6. Transition SM	Mohr-Coulomb	120	0	33
	7. Sheet Pile	Bedrock (Impenetrable)			



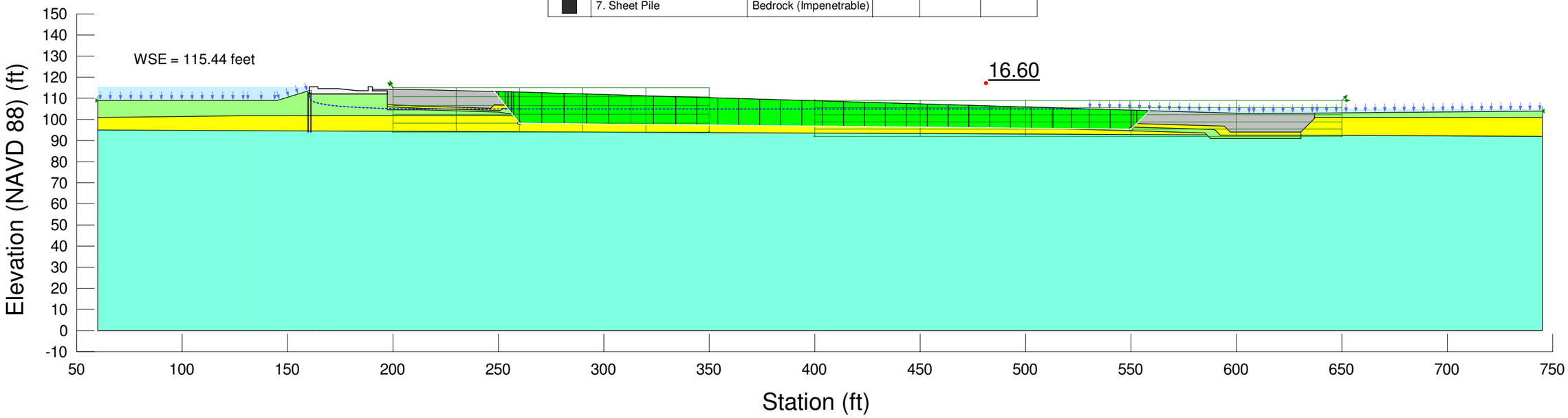
NOTES: Blocks used for Slip Surface search

Bellota Weir Modification Project

Bellota Weir Steady State Stability Model D/S WSE 105 ft

Feb 2022 FIGURE E-4

Color	Name	Material Model	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)
	1. Alluvium (Upper Unit) SM	Mohr-Coulomb	120	50	30
	2. Alluvium (Lower Unit) SP	Mohr-Coulomb	120	0	32
	3. Older Alluvium ML/SM	Mohr-Coulomb	120	100	30
	4. Foundation Rocks Cobbles	Mohr-Coulomb	140	0	40
	5. Filter SP	Mohr-Coulomb	120	0	33
	6. Transition SM	Mohr-Coulomb	120	0	33
	7. Sheet Pile	Bedrock (Impenetrable)			



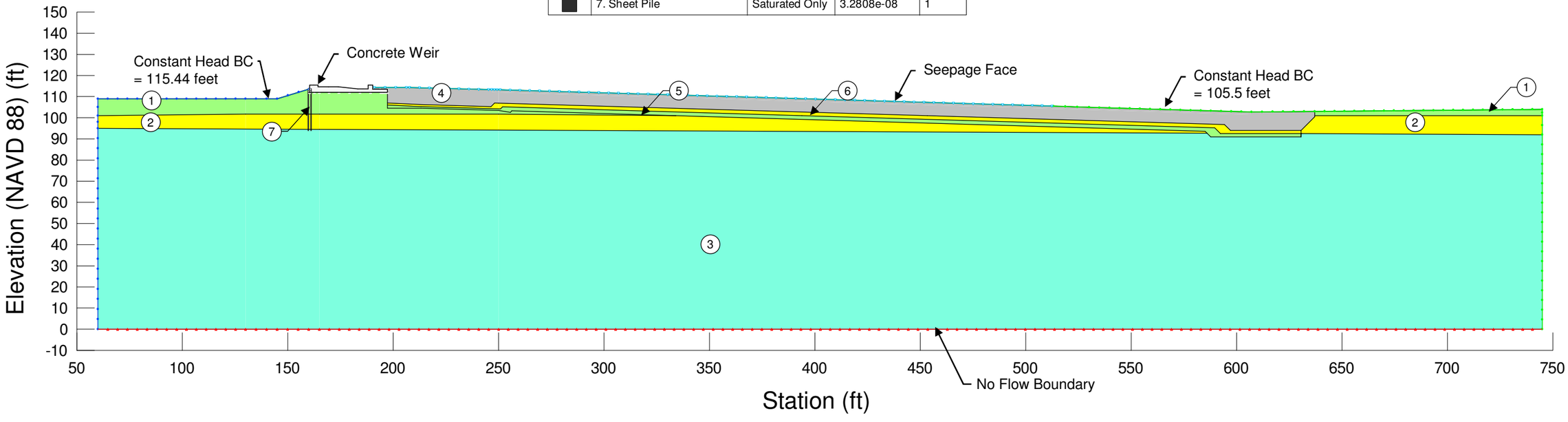
NOTES:

Bellota Weir Modification Project

Bellota Weir Steady State Stability Result D/S WSE 105 ft

Feb 2022 FIGURE E-5

Color	Name	Material Model	Sat Kx (ft/sec)	Ky'/Kx' Ratio
	1. Alluvium (Upper Unit) SM	Saturated Only	3.28084e-06	0.25
	2. Alluvium (Lower Unit) SP	Saturated Only	0.000328084	0.25
	3. Older Alluvium ML/SM	Saturated Only	3.2808e-07	0.25
	4. Foundation Rocks Cobbles	Saturated Only	0.328084	1
	5. Filter SP	Saturated Only	0.0164042	1
	6. Transition SM	Saturated Only	3.28084e-06	0.25
	7. Sheet Pile	Saturated Only	3.2808e-08	1



NOTES: BC = Boundary Condition

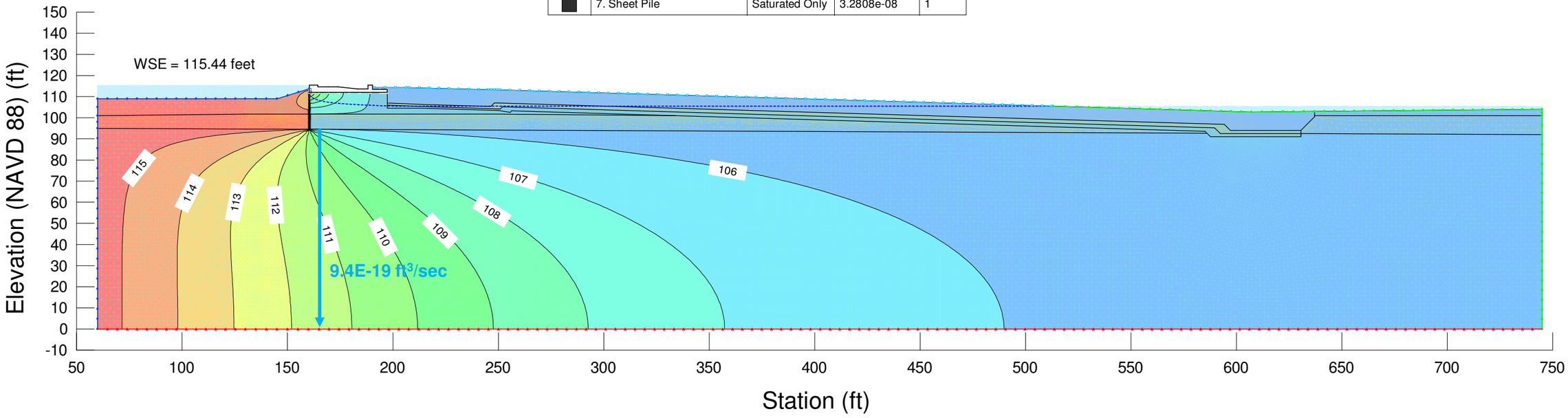
Bellota Weir Modification Project

Steady State Seepage

Bellota Weir Steady State Seepage Model D/S WSE 105.5 ft

Feb 2022 FIGURE E-6

Color	Name	Material Model	Sat Kx (ft/sec)	Ky'/Kx' Ratio
	1. Alluvium (Upper Unit) SM	Saturated Only	3.28084e-06	0.25
	2. Alluvium (Lower Unit) SP	Saturated Only	0.000328084	0.25
	3. Older Alluvium ML/SM	Saturated Only	3.2808e-07	0.25
	4. Foundation Rocks Cobbles	Saturated Only	0.328084	1
	5. Filter SP	Saturated Only	0.0164042	1
	6. Transition SM	Saturated Only	3.28084e-06	0.25
	7. Sheet Pile	Saturated Only	3.2808e-08	1



NOTES:

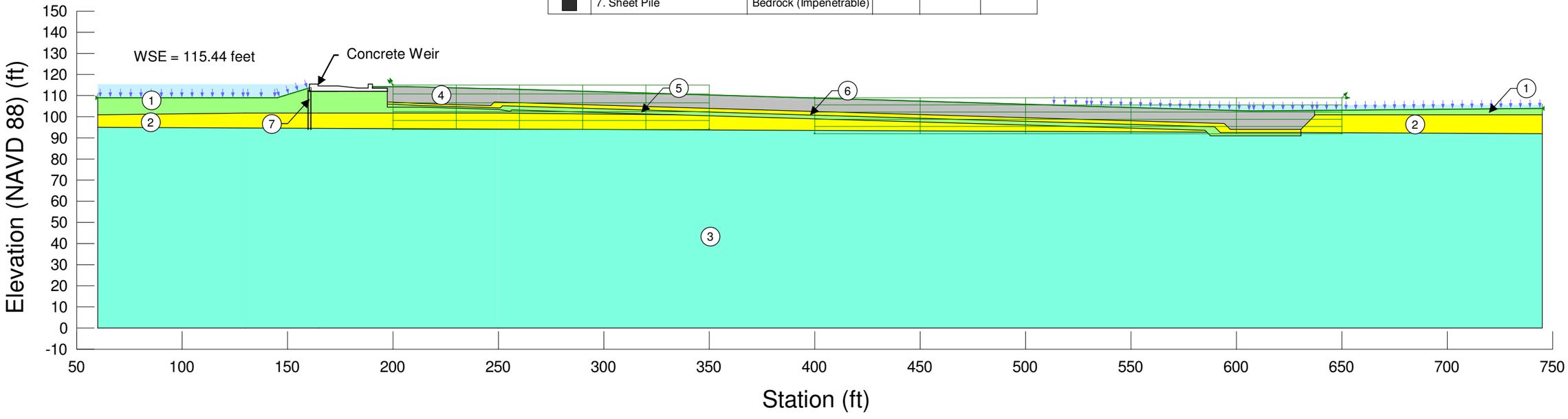
Bellota Weir Modification Project

Steady State Seepage

Bellota Weir Steady State Seepage Result D/S WSE 105.5 ft

Feb 2022 FIGURE E-7

Color	Name	Material Model	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)
	1. Alluvium (Upper Unit) SM	Mohr-Coulomb	120	50	30
	2. Alluvium (Lower Unit) SP	Mohr-Coulomb	120	0	32
	3. Older Alluvium ML/SM	Mohr-Coulomb	120	100	30
	4. Foundation Rocks Cobbles	Mohr-Coulomb	140	0	40
	5. Filter SP	Mohr-Coulomb	120	0	33
	6. Transition SM	Mohr-Coulomb	120	0	33
	7. Sheet Pile	Bedrock (Impenetrable)			



NOTES: Blocks used for Slip Surface search

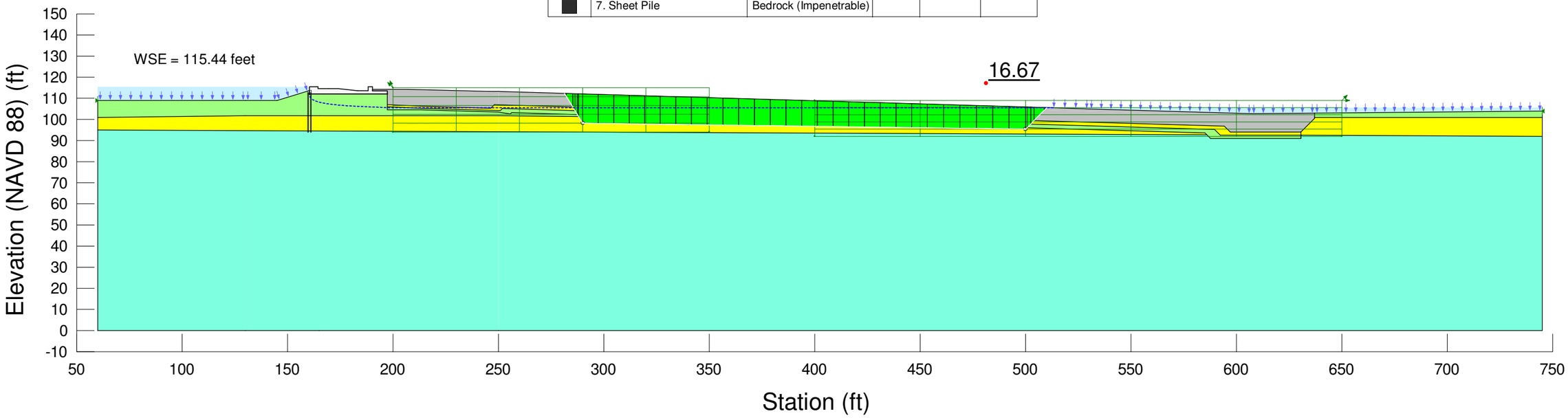
Bellota Weir Modification Project

Bellota Weir Steady State Stability Model D/S WSE 105.5 ft

Feb 2022

FIGURE E-8

Color	Name	Material Model	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)
	1. Alluvium (Upper Unit) SM	Mohr-Coulomb	120	50	30
	2. Alluvium (Lower Unit) SP	Mohr-Coulomb	120	0	32
	3. Older Alluvium ML/SM	Mohr-Coulomb	120	100	30
	4. Foundation Rocks Cobbles	Mohr-Coulomb	140	0	40
	5. Filter SP	Mohr-Coulomb	120	0	33
	6. Transition SM	Mohr-Coulomb	120	0	33
	7. Sheet Pile	Bedrock (Impenetrable)			



NOTES:

Bellota Weir Modification Project

Bellota Weir Steady State Stability Result D/S WSE 105.5 ft

Feb 2022 FIGURE E-9

Appendix F. Filter Analysis

Base Material

Determine the gradation curves of the base soil. Use enough samples to define the range of grain size for the base soil. Design the filter gradation based on the base soil that requires the smallest D_{15F} size. If soil has particles larger than the #4 sieve, an adjusted gradation is calculated. Input values below for the base soil (original) gradation (in red):

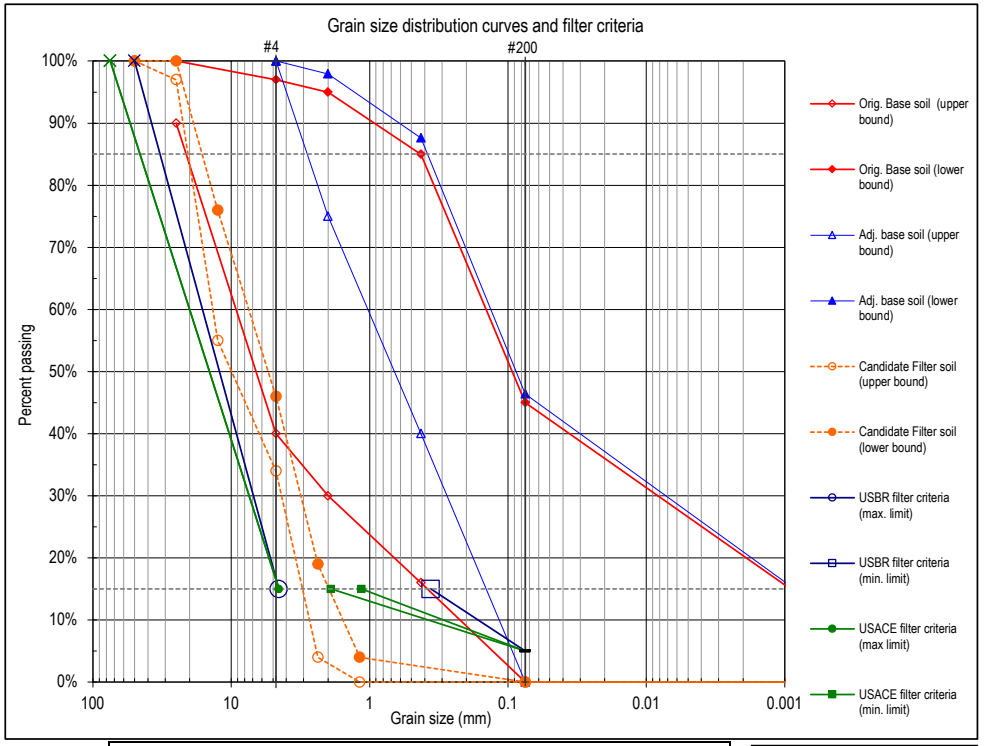
Project: Bellota Weir
Base Soil to Filter

Name: SRS
Date: 12/23/2021

Particle size (mm)	Sieve #	Base soil (original), % passing (upper bound)	(lower bound)	Adjusted gradation, % passing upper bound	lower bound
25	-	90.0%	100.0%		
19.05	-				
12.5	-				
9.5	-				
4.75	4	40.0%	97.0%	100.0%	100.0%
4.00	5				
3.35	6				
2.80	7				
2.36	8				
2.00	10	30.0%	95.0%	75.0%	97.9%
1.70	12				
1.40	14				
1.18	16				
1.00	18				
0.850	20				
0.710	25				
0.600	30				
0.500	35				
0.425	40	16.0%	85.0%	40.0%	87.6%
0.300	50				
0.250	60				
0.212	70				
0.180	80				
0.150	100				
0.125	120				
0.106	140				
0.090	170				
0.075	200	0.0%	45.0%	0.0%	46.4%
0.053	270				
0.037	-				
0.017	-				
0.009	-				
0.005	-				
0.002	-				
0.001	-				
0.0001	-	0.0%	0.0%	0.0%	0.0%

Required entry values are defined below. *

Average % passing #200 after regarding (if any) = A = 23.2%



Properties of base soil	Upper bound	Lower bound
D_{85B} of original base soil =	21.175 mm	0.425 mm
D_{85B} of adjusted base soil =	2.827 mm	0.381 mm
D_{15B} of original base soil =	0.3813 mm	0.0009 mm
D_{15B} of adjusted base soil =	0.1437 mm	0.0009 mm
D_{60B} of adjusted base soil =	1.030 mm	0.133 mm
D_{10B} of adjusted base soil =	0.1157 mm	0.0004 mm
C_u of adjusted base soil =	8.9	319.1

Fixed points on graph:
- <5% passing #200
- 50mm max. grain size (USBR)
- 75mm max. grain size (USACE)

Filter Material

Filter criteria required by the USBR as published in *Design Standards - Embankment Dams No. 13 (1994):*

D_{85B} used in filter design	1.604
Average Passing #200 sieve of base soil (from adjusted gradation)	23.2%
Base soil category	3
Base soil description	Silty and clayey sands and gravels
Filter criteria (mm)	Maximum: $D_{15F} \leq$ 4.54 To ensure sufficient permeability: Minimum: $D_{15F} \geq$ 0.36
Maximum particle size of filter (mm)	50
Maximum % passing # 200 sieve	5%
PI of material passing #40	0 when tested in accordace with USBR 5360, Earth Manual, on material passing #40

Filter criteria required by the US Army Corps of Engineers as published in EM 1110-2-2300 (31 Jul 94):

D_{85B} used in filter design	1.604
Average Passing #200 sieve of base soil (from adjusted gradation)	23.2%
Base soil category	3
Filter criteria (mm)	Maximum: $D_{15F} \leq$ 4.54 to 4.54 To ensure sufficient permeability: Minimum: $D_{15F} \geq$ 1.14 to 1.91
Maximum particle size of filter (mm)	75
Maximum % passing # 200 sieve	5%
PI of material passing #40	0 when tested in accordance with EM 1110-2-1906

**If the base soil is in category 4, use the lower of the two 'max. D_{15F}' values when the filter is beneath riprap subject to wave action or beneath drains which may be subject to violent surging and/or vibration.

*Required entry values for base soil & candidate filter gradations:

1. Particle size for 100% passing.
2. % Passing the #4 sieve.
3. % Passing the #200 sieve.
4. One point in the 85% - 90% range and another point in the 80% - 85% range, or the 85% point.
5. One point in the 15% - 20% range and another point in the 10% - 15% range, or the 15% point.
6. No duplicate entries; if D100<#4, enter 101% for #4 and 100% for appropriate size.

USBR filter gradation limits:

Maximum limit	
Grain size (mm)	% Passing
50.00	100.0%
4.54	15.0%

Minimum limit	
Grain size (mm)	% Passing
0.36	15.0%
0.075	5.0%

USACE filter gradation limits:

Maximum limit	
Grain size (mm)	% Passing
75.00	100.0%
4.54	15.0%
4.54	15.0%

Minimum limit	
Grain size (mm)	% Passing
1.14	15.0%
1.91	15.0%
0.075	5.0%

Candidate filter soil gradation. Values shown in red in the left column, and all values in the two right columns, can be changed.

Particle size *mm*	Sieve #	% Passing (upper bound)	% Passing (lower bound)
150.0	-		
100.0	-		
90.0	-		
75.0	-		
63.0	-		
50.0	-	(100.0%)	(100.0%)
37.5	-		
25.0	-	(97.0%)	(100.0%)
19.0	-		
12.5	-	55.0%	(76.0%)
9.5	-		
4.75	4	34.0%	46.0%
3.35	6		
2.36	8	4.0%	19.0%
2.00	10		
1.70	12		
1.40	14		
1.18	16	0.0%	4.0%
0.850	20		
0.600	30		
0.425	40		
0.300	50		
0.250	60		
0.212	70		
0.180	80		
0.150	100		
0.125	120		
0.106	140		
0.090	170		
0.075	200	(0.0%)	(0.0%)
0.053	270		
0.037	-		
0.019	-		
0.009	-		
0.0001	-	0.0%	0.0%

Required entry values are defined above. *

Properties of candidate filter soil (CF). D sizes are in mm:

	D_{85CF}	D_{15CF}	D_{60CF}	D_{10CF}	C_u
upper bound	20.51	3.05	13.58	2.71	5.00
lower bound	16.21	1.96	7.46	1.56	4.79

Acceptibility of candidate filter (CF) soil:

USBR criteria	Upper bound	Lower bound
Max % passing #200:	OK	OK
Max particle size (mm):	OK	OK
Maximum D_{15CF}:	OK	OK
Minimum D_{15CF}:	OK	OK
To minimize segregation (from *Table 2*)***		
Max allowable D_{90CF} =	30	OK
Max D_{90CF} =	22.27	

USACE criteria	Upper bound	Lower bound
Max % passing #200:	OK	OK
Max particle size (mm):	OK	OK
Maximum D_{15CF}:	OK	OK
Minimum D_{15CF} ($3\times D_{15B}$):	OK	OK
Minimum D_{15CF} ($5\times D_{15B}$):	OK	OK
To minimize segregation (from *Table B-3*)***		
Max allowable D_{90CF} =	30	OK
Max D_{90CF} =	22.27	

Filters should be relatively uniform (see the C_U value of the candidate filter soil.). Also, filters should not be gap-graded.

*** Generally, this requirement is only necessary for coarse filters and gravel zones that serve as both filters and drains. For sand filters with D_{90} < ~20mm, these limitations are usually not necessary.

Base Material

Determine the gradation curves of the base soil. Use enough samples to define the range of grain size for the base soil. Design the filter gradation based on the base soil that requires the smallest D_{15F} size. If soil has particles larger than the #4 sieve, an adjusted gradation is calculated. Input values below for the base soil (original) gradation (in red):

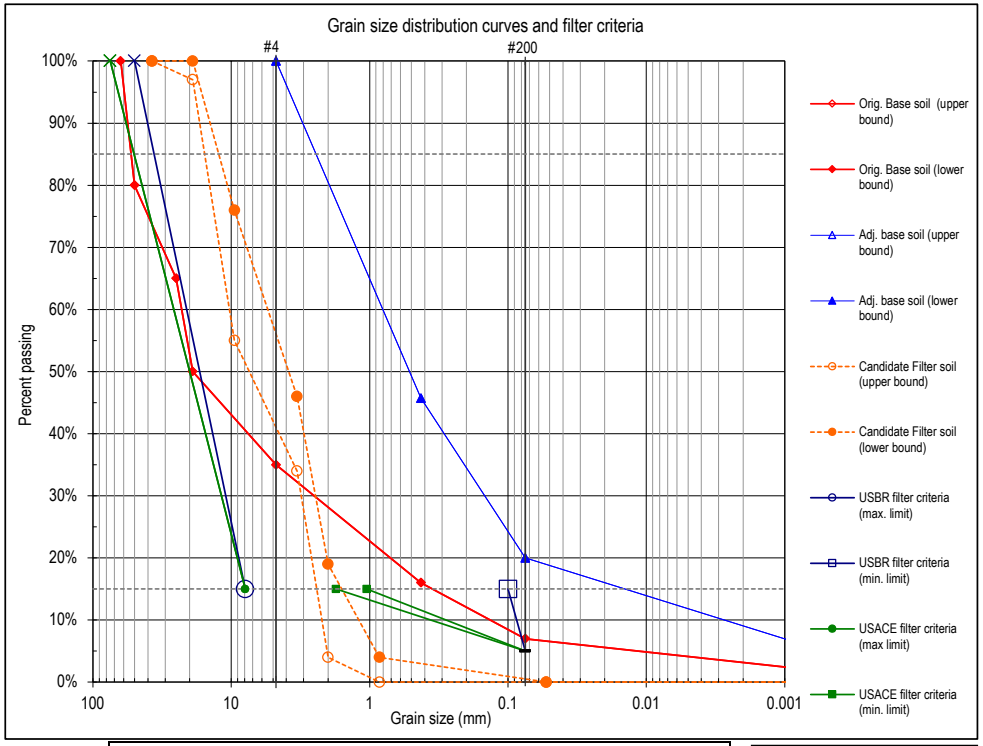
Project: Bellota Weir
Rock Infill to Filter

Name: SRS
Date: 12/23/2021

Particle size (mm)	Sieve #	Base soil (original), % passing (upper bound)	(lower bound)	Adjusted gradation, % passing upper bound	lower bound
63	-	100.0%	100.0%		
50	-	80.0%	80.0%		
25.0	-	65.0%	65.0%		
19.0	-	50.0%	50.0%		
4.75	4	35.0%	35.0%	100.0%	100.0%
4.00	5				
3.35	6				
2.80	7				
2.36	8				
2.00	10				
1.70	12				
1.40	14				
1.18	16				
1.00	18				
0.850	20				
0.710	25				
0.600	30				
0.500	35				
0.425	40	16.0%	16.0%	45.7%	45.7%
0.300	50				
0.250	60				
0.212	70				
0.180	80				
0.150	100				
0.125	120				
0.106	140				
0.090	170				
0.075	200	7.0%	7.0%	20.0%	20.0%
0.053	270				
0.037	-				
0.017	-				
0.009	-				
0.005	-				
0.002	-				
0.001	-				
0.0001	-	0.0%	0.0%	0.0%	0.0%

Required entry values are defined below. *

Average % passing #200 after regarding (if any) = A = 20.0%



Properties of base soil	Upper bound	Lower bound
D_{85B} of original base soil =	52.974 mm	52.974 mm
D_{85B} of adjusted base soil =	2.438 mm	2.438 mm
D_{15B} of original base soil =	0.3505 mm	0.3505 mm
D_{15B} of adjusted base soil =	0.0143 mm	0.0143 mm
D_{60B} of adjusted base soil =	0.802 mm	0.802 mm
D_{10B} of adjusted base soil =	0.0027 mm	0.0027 mm
C_u of adjusted base soil =	292.9	292.9

Fixed points on graph:
- <5% passing #200
- 50mm max. grain size (USBR)
- 75mm max. grain size (USACE)

Filter Material

Filter criteria required by the USBR as published in *Design Standards - Embankment Dams No. 13 (1994):*

D_{85B} used in filter design	2.438
Average Passing #200 sieve of base soil (from adjusted gradation)	20.0%
Base soil category	3
Base soil description	Silty and clayey sands and gravels
Filter criteria (mm)	Maximum: $D_{15F} \leq$ 7.94 To ensure sufficient permeability: Minimum: $D_{15F} \geq$ 0.10
Maximum particle size of filter (mm)	50
Maximum % passing # 200 sieve	5%
PI of material passing #40	0 when tested in accordace with USBR 5360, Earth Manual, on material passing #40

Filter criteria required by the US Army Corps of Engineers as published in EM 1110-2-2300 (31 Jul 94):

D_{85B} used in filter design	2.438
Average Passing #200 sieve of base soil (from adjusted gradation)	20.0%
Base soil category	3
Filter criteria (mm)	Maximum: $D_{15F} \leq$ 7.94 to 7.94 To ensure sufficient permeability: Minimum: $D_{15F} \geq$ 1.05 to 1.75
Maximum particle size of filter (mm)	75
Maximum % passing # 200 sieve	5%
PI of material passing #40	0 when tested in accordance with EM 1110-2-1906

**If the base soil is in category 4, use the lower of the two 'max. D_{15F}' values when the filter is beneath riprap subject to wave action or beneath drains which may be subject to violent surging and/or vibration.

*Required entry values for base soil & candidate filter gradations:

1. Particle size for 100% passing.
2. % Passing the #4 sieve.
3. % Passing the #200 sieve.
4. One point in the 85% - 90% range and another point in the 80% - 85% range, or the 85% point.
5. One point in the 15% - 20% range and another point in the 10% - 15% range, or the 15% point.
6. No duplicate entries; if D100<#4, enter 101% for #4 and 100% for appropriate size.

USBR filter gradation limits:

Maximum limit	
Grain size (mm)	% Passing
50.00	100.0%
7.94	15.0%

Minimum limit	
Grain size (mm)	% Passing
0.10	15.0%
0.075	5.0%

USACE filter gradation limits:

Maximum limit	
Grain size (mm)	% Passing
75.00	100.0%
7.94	15.0%
7.94	15.0%

Minimum limit	
Grain size (mm)	% Passing
1.05	15.0%
1.75	15.0%
0.075	5.0%

Candidate filter soil gradation. Values shown in red in the left column, and all values in the two right columns, can be changed.

Particle size *mm*	Sieve #	% Passing (upper bound)	% Passing (lower bound)
150.0	-		
100.0	-		
90.0	-		
75.0	-		
63.0	-		
50.0	-		
37.5	-	(100.0%)	(100.0%)
25.0	-		
19.0	-	(97.0%)	(100.0%)
12.5	-		
9.5	-	55.0%	76.0%
4.75	4		
3.35	6	(34.0%)	(46.0%)
2.36	8		
2.00	10	(4.0%)	(19.0%)
1.70	12		
1.40	14		
1.18	16		
0.850	20	(0.0%)	(4.0%)
0.600	30		
0.425	40		
0.300	50		
0.250	60		
0.212	70		
0.180	80		
0.150	100		
0.125	120		
0.106	140		
0.090	170		
0.075	200		
0.053	270	(0.0%)	(0.0%)
0.037	-		
0.019	-		
0.009	-		
0.0001	-	0.0%	0.0%

Required entry values are defined above. *

Properties of candidate filter soil (CF). D sizes are in mm:

	D_{85CF}	D_{15CF}	D_{60CF}	D_{10CF}	C_u
upper bound	15.59	2.42	10.32	2.22	4.65
lower bound	15.90	1.59	5.45	1.20	4.55

Acceptibility of candidate filter (CF) soil:

USBR criteria	Upper bound	Lower bound
Max % passing #200:	OK	OK
Max particle size (mm):	OK	OK
Maximum D_{15CF}:	OK	OK
Minimum D_{15CF}:	OK	OK
To minimize segregation (from *Table 2*)***		
Max allowable D_{90CF} =	30	OK
Max D_{90CF} =	16.93	

USACE criteria	Upper bound	Lower bound
Max % passing #200:	OK	OK
Max particle size (mm):	OK	OK
Maximum D_{15CF}:	OK	OK
Minimum D_{15CF} ($3\times D_{15B}$):	OK	OK
Minimum D_{15CF} ($5\times D_{15B}$):	OK	NOT MET
To minimize segregation (from *Table B-3*)***		
Max allowable D_{90CF} =	30	OK
Max D_{90CF} =	16.93	

Filters should be relatively uniform (see the C_U value of the candidate filter soil.). Also, filters should not be gap-graded.

*** Generally, this requirement is only necessary for coarse filters and gravel zones that serve as both filters and drains. For sand filters with D_{90} < ~20mm, these limitations are usually not necessary.

Appendix D. Cost Data

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STOCKTON EAST WATER DISTRICT
BELLOTA WEIR MODIFICATIONS
PRELIMINARY OPINION OF PROBABLE CONSTRUCTION COSTS

BID ITEM	DESCRIPTION	QUANTITY	UNIT	COST	AMOUNT	TOTAL
	DEMO					
	EXTG PIPELINE					**$654,587.19**
1	REMOVE REINFORCED CONCRETE 54" PIPE (8FT SECTIONS)	24	LF	$88.00	$2,112.00	
2	REMOVE EXTG 36" BYPASS PIPE (8FT SECTIONS)	24	LF	$70.00	$1,680.00	
	EXTG DAM					
3	CONCRETE	1,481	CY	$202.00	$299,259.26	
4	DISPOSAL, DUMP CHARGES	2,222	TONS	$81.00	$180,000.00	
	EXTG SHEET PILE WALLS					
5	SHEET PILE WALLS	988	SF	$25.00	$24,700.00	
6	DISPOSAL, DUMP CHARGES (ASSUME SOME SALVAGE)	13	TONS	$81.00	$1,080.38	
7	EXTG BUILDINGS	3	LS	$5,000.00	$15,000.00	
	EXTG FENCING					
8	EXISTING FENCE	420	LF	$10.00	$4,200.00	
9	EXISTING GATE	2	EA	$500.00	$1,000.00	
	EXTG WELL					
10	REMOVE EXISTING WELL 5' BELOW BOTTOM OF IMPROVEMENTS	1	LS	$3,500.00	$3,500.00	
11	ABANDON EXISTING WELL	1	LS	$22,000.00	$22,000.00	
12	WELL ABANDONMENT PERMIT	1	LS	$10,000.00	$10,000.00	
	EXTG BUILDING					
13	ABOVE-GRADE STRUCTURE	1	LS	$15,000.00	$15,000.00	
14	CONCRETE FOUNDATION	28	CY	$32.00	$888.89	
15	UTILITY CONNECTIONS	1	LS	$10,000.00	$10,000.00	
16	SEPTIC TANK & LEACH FIELD	1	LS	$10,000.00	$10,000.00	
17	DISPOSAL, DUMP CHARGES	52	TONS	$80.00	$4,166.67	
18	ASBESTOS REMOVAL AND DISPOSAL	1	LS	$50,000.00	$50,000.00	
	SITEWORK					**$2,102,075.30**
19	CLEAR & GRUB - CONSTRUCTION STAGING AREAS	4.8	ACRE	$17,950.00	$85,591.75	
20	CLEAR & GRUB - CONSTRUCTION ACCESS ROAD (LANDSIDE)	0.55	ACRE	$17,950.00	$9,898.05	
21	CLEAR & GRUB - PERMANENT ACCESS ROAD (LANDSIDE)	0.83	ACRE	$17,950.00	$14,852.84	
22	CLEAR & GRUB - OLD CALAVERAS RIVER EMBANKMENT	0.39	ACRE	$17,950.00	$7,023.25	
23	REMOVE TREE	2	EA	$219.00	$438.00	
24	DEWATERING	520	DY	$2,950.00	$1,534,000.00	
25	FINISHED GRADING	1	LS	$20,000.00	$20,000.00	
26	EXCAVATION (MISC)	250	CY	$30.00	$7,500.00	
27	BACKFILL (MISC)	250	CY	$40.00	$10,000.00	
28	STRUCTURAL BACK FILL (MISC)	250	CY	$65.00	$16,250.00	
29	CONCRETE, SLAB (GENERATOR PAD)	10	CY	$1,055.00	$10,550.00	
30	CONCRETE, SLAB (BUILDING ENTRY PADS)	3	CY	$1,055.00	$2,972.76	
31	CONCRETE, SLAB (ANTENNA PAD)	38	CY	$1,055.00	$40,011.85	
32	CONCRETE, SLAB (AIR CONDITIONER PAD)	0.69	CY	$1,055.00	$732.70	
33	CONCRETE, SLAB (SWITCHBOARD HOUSEKEEPING PAD)	2.0	CY	$1,055.00	$2,110.00	
34	CONCRETE, SLAB (ELECTRICAL TRANSFORMER PAD)	6	CY	$1,055.00	$5,861.11	
35	BOLLARD	27	EA	$850.00	$22,950.00	
36	SITE SIGNAGE	10	EA	$350.00	$3,500.00	
37	6' H CHAINLINK FENCE W/ BARB WIRE TOP	740	LF	$80.00	$59,200.00	
38	10' W DBL CHAINLINK GATE, 6' H	1	EA	$5,400.00	$5,400.00	
39	12' W DBL CHAINLINK GATE, 6' H	1	EA	$6,400.00	$6,400.00	
40	20' W ROLLING GATE, 6' H	1	EA	$7,000.00	$7,000.00	
41	24' W ROLLING GATE, 6' H	2	EA	$8,000.00	$16,000.00	
42	AUTOMATIC GATE OPERATOR	3	EA	$4,500.00	$13,500.00	
43	TESC MEASURES	1	LS	$160,000.00	$160,000.00	
44	SITE CLEAN-UP	1	LS	$10,000.00	$10,000.00	
45	LANDSCAPING / GROUND IMPROVEMENTS	1	LS	$30,333.00	$30,333.00	
	ACCESS					
	CRANE ACCESS PARKING					**$662,471.96**
46	CRUSHED BASE, 6"	46	CY	$55.00	$2,546.50	
47	COMPACT FINISH ROCK, 3"	23	CY	$5.00	$115.63	
48	SLAB	185	CY	$750.00	$138,750.00	
49	EXCAVATION	583	CY	$6.00	$3,496.50	
50	BACKFILL	583	CY	$40.00	$23,310.00	
	EQUIPMENT ACCESS					
51	RSP INSTALLATION	111	CY	$120.00	$13,333.33	
52	TEMPORARY ACCESS ROAD FOR ROUGHENED CHANNEL AND WEIR					
53	QUARRY SPALLS	455	CY	$20.00	$9,100.00	
	CONSTRUCTION ACCESS ROAD					
54	AGGREGATE BASE, 11.5"	2,700	SY	$55.00	$148,500.00	
55	SUBGRADE, 8" COMPACTED TO 90%	2,700	SY	$5.00	$13,500.00	
56	ROAD GRADING	880	CY	$22.00	$19,360.00	
	PERMANENT ACCESS ROAD					
57	AGGREGATE BASE, 11.5"	3,200	SY	$55.00	$176,000.00	
58	SUBGRADE, 8" COMPACTED TO 90%	3,200	SY	$5.00	$16,000.00	
59	ROAD GRADING	1,270	CY	$22.00	$27,940.00	
	REPAIR EXISTING AC PAVEMENT AT TRANSITION TO AB ROAD					
60	ASPHALT CONCRETE, 4"	820	SY	$45.00	$36,900.00	
61	AGGREGATE BASE, 8"	820	SY	$36.00	$29,520.00	
62	SUBGRADE, 8" COMPACTED TO 90%	820	SY	$5.00	$4,100.00	

STOCKTON EAST WATER DISTRICT
BELLOTA WEIR MODIFICATIONS
PRELIMINARY OPINION OF PROBABLE CONSTRUCTION COSTS

BID ITEM	DESCRIPTION	QUANTITY	UNIT	COST	AMOUNT	TOTAL
	INTAKE/FISHWAY					**$9,207,242.54**
63	SHORING AND PROTECTIVE STRUCTURES (BANK ONLY)	4,900	SF	$60.00	$294,000.00	
64	EXCAVATION (INTAKE)	3,824	CY	$6.00	$22,944.44	
65	STRUCTURAL BACKFILL (INTAKE)	584	CY	$40.00	$23,348.15	
66	EXCAVATION (FISHWAY)	1,921	CY	$2.00	$3,842.59	
67	BACKFILL (FISHWAY)	160	CY	$40.00	$6,400.00	
68	CONCRETE, WALL (INTAKE)	1,297	CY	$750.00	$972,693.78	
69	CONCRETE, SLAB (INTAKE)	794	CY	$1,000.00	$793,650.00	
70	CONCRETE INSIDE PILE (6 FT TOP)	106	CY	$750.00	$79,786.55	
71	CONCRETE, WALL (FISH LADDER)	298	CY	$750.00	$223,381.95	
72	CONCRETE, WALL (VERTICAL SLOTS AND WEIRS)	23	CY	$750.00	$16,928.18	
73	CONCRETE, SLAB (FISH LADDER)	258	CY	$1,000.00	$257,508.90	
74	FISH LADDER WEIR STOPLOGS	8	EA	$80.00	$640.00	
75	TEE SCREENS, ISI 83", INCLUDES ALL CONTROLS	2	EA	$875,300.00	$1,750,600.00	
76	TEE SCREENS, ISI 48", INCLUDES ALL CONTROLS	2	EA	$587,800.00	$1,175,600.00	
77	TRASH RACK FOR FOREBAY SLIUCEWAY, 20 X 5'	1,750	LBS	$10.00	$17,500.00	
78	TRASH RACK, 12.5' X 8'	1,500	LBS	$10.00	$15,000.00	
79	TRASH RACK CLEANING SYSTEM FOR LADDER	1	EA	$385,000.00	$385,000.00	
80	WATER JET SYSTEM	1	LS	$10,000.00	$10,000.00	
81	GUARDRAIL, 42"H, TWO-RAIL, GS	379	LF	$75.00	$28,425.00	
82	FISH LADDER GATE, SIZE 96" X 84"	1	EA	$12,000.00	$12,000.00	
83	OVERFLOW BYPASS GATE, SIZE 72" X 72"	1	EA	$25,000.00	$25,000.00	
84	STOP GATE ON FISH LADDER WEIR, 12" X 15"	1	EA	$2,500.00	$2,500.00	
85	DEEP FOUNDATION (151X 55 FT PILES)	8,305	LF	$154.00	$1,278,970.00	
86	LAMPREY RAMPS (4.5' x 2')	2	EA	$1,200.00	$2,400.00	
87	REMOVABLE 6" SILLS AT VERTICAL SLOTS	7	EA	$360.00	$2,520.00	
88	RSP TYPE C	135	CY	$112.00	$15,120.00	
89	GEOTEXTILE	134	SY	$2.00	$268.00	
90	REMOVABLE 12" SILLS AT VERTICAL SLOTS	7	EA	$410.00	$2,870.00	
91	SHEET PILE WALL S2 (TEMPORARY)	5,250	SF	$90.00	$472,500.00	
92	SHEET PILE WALL S3-A (PERMENANT)	3,880	SF	$90.00	$349,200.00	
93	SHEET PILE WALL S3-B (PERMENANT)	10,340	SF	$90.00	$930,600.00	
94	GRATING	801	SF	$45.00	$36,045.00	
	SLUICEWAY					**$1,725,834.33**
95	TEMPORARY SHORING AND PROTECTIVE STRUCTURES (BANK PROTECTION ONLY)	22,555	SF	$60.00	$1,353,300.00	
96	SLUICEWAY NO. 1 SLIDE GATE, 54" X 54" WITH ELECTRIC OPERATOR	1	EA	$25,000.00	$25,000.00	
97	FISHWAY SLUICEWAY SLIDE GATE, 54" X 54" WITH ELECTRIC OPERATOR	1	EA	$25,000.00	$25,000.00	
98	EXCAVATION (36" EXTENTION SLUICEWAY)	200	CY	$12.00	$2,400.00	
99	EXCAVATION (54" SLUICEWAY)	650	CY	$12.00	$7,800.00	
100	BACKFILL (36" EXTENTION SLUICEWAY)	140	CY	$30.00	$4,200.00	
101	BACKFILL (54" SLUICEWAY)	385	CY	$40.00	$15,385.63	
102	36" BYPASS PIPELINE (CONNECT TO EXTG 36" BYPASS PIPELINE FOR OVERFLOW)	275	LF	$75.00	$20,625.00	
103	8" STEEL PIPELINE (BLACK STEEL EXTRA HEAVY DUTY)	40	LF	$200.00	$8,000.00	
104	REINFORCED CONCRETE PIPE (54" RCP)	440	LF	$115.00	$50,600.00	
105	CONCRETE PIPE DELIVERY	1	LS	$98,000.00	$98,000.00	
106	SLUICEWAY HEADWALL STRUCTURE	110	CY	$1,000.00	$109,523.70	
107	RSP APRON FOR 54" SLUICEWAY	33	CY	$120.00	$4,000.00	
108	RSP APRON FOR 36" BYPASS	17	CY	$120.00	$2,000.00	
	BUILDING					**$133,531.17**
	EQUIPMENT AND CONTROLS BUILDING					
109	EQUIPMENT FOUNDATIONS	45	CY	$750.00	$33,482.87	
110	BUILDING SLAB	10	CY	$750.00	$7,135.22	
111	BUILDING FOUNDATION	19	CY	$750.00	$14,163.60	
112	BUILDING CMU	1,276	SF	$30.00	$38,280.00	
113	BUILDING ROOF	561	SF	$8.00	$4,488.00	
114	BUILDING ALUM DOORS	160	SF	$100.00	$16,000.00	
115	BUILDING GYPSUM BOARD	900	SF	$1.00	$900.00	
116	EXCAVATION	202	CY	$6.00	$1,210.63	
117	BACKFILL	202	CY	$40.00	$8,070.85	
118	HVAC FAN	1	LS	$2,600.00	$2,600.00	
119	LOUVERS	4	EA	$400.00	$1,600.00	
120	DUCTLESS MINI SPLIT AC & HEAT PUMP	1	EA	$4,000.00	$4,000.00	
121	SUN TUBES	2	EA	$800.00	$1,600.00	
	BELLOTA CONVEYANCE PIPELINES AND ASSOCIATED STRUCTURES					**$2,017,638.96**
122	SHORING AND PROTECTIVE STRUCTURES	5,750	SF	$60.00	$345,000.00	
123	EXCAVATION	10,222	CY	$6.00	$61,333.33	
124	BACKFILL	9,701	CY	$40.00	$388,029.63	
125	GATES, SLIDE (THREE 54" X 54" WITH MOTOR OPERATOR)	3	EA	$12,000.00	$36,000.00	
126	DEEP FOUNDATION WITH GEOTECHNICAL IMPROVEMENTS (PILES) (116X50FT LONG)	5,800	LF	$41.00	$237,800.00	
127	54" BELLOTA CONVEYANCE STEEL PIPELINE	1,020	LF	$768.00	$783,360.00	
128	54" BELLOTA CONVEYANCE STEEL PIPELINE WELDS	27	EA	$1,100.00	$29,700.00	
129	54" BELLOTA CONVEYANCE STEEL CATHORIC PROTECTION	1	LS	$30,000.00	$30,000.00	
130	54" BELLOTA CONVEYANCE SUPPORT CHANNEL	12	EA	$150.00	$1,800.00	
131	54" BELLOTA CONVEYANCE NEOPRENE STRIPS	12	EA	$118.00	$1,416.00	
132	54" FLOW METER, ULTRASONIC	2	LS	$51,600.00	$103,200.00	

STOCKTON EAST WATER DISTRICT
BELLOTA WEIR MODIFICATIONS
PRELIMINARY OPINION OF PROBABLE CONSTRUCTION COSTS

BID ITEM	DESCRIPTION	QUANTITY	UNIT	COST	AMOUNT	TOTAL
	OLD CALAVERAS IMPROVEMENTS AND CONVEYANCE PIPELINES					**$1,568,246.57**
133	SHORING AND PROTECTIVE STRUCTURES	10,500	SF	$60.00	$630,000.00	
134	EXCAVATION	7,156	CY	$2.00	$14,311.11	
135	BACKFILL	6,634	CY	$40.00	$265,362.96	
136	REINFORCED CONCRETE PIPE (54" RCP)	360	LF	$368.00	$132,480.00	
137	FLOW METER	1	LS	$5,000.00	$5,000.00	
138	SLIDE GATE FOR OLD CALAVERAS BYPASS PIPE, SIZE 72", MANUAL	1	EA	$15,000.00	$15,000.00	
139	GATES, SLIDE (TWO 96" X 48" WITH MOTOR OPERATOR)	2	EA	$25,000.00	$50,000.00	
	OLD CALAVERAS OUTLET STRUCTURE					
140	CONCRETE, WALL	106	CY	$750.00	$79,642.50	
141	CONCRETE, SLAB	120	CY	$750.00	$89,910.00	
142	RSP TYPE C	120	CY	$120.00	$14,400.00	
143	FILTER LAYER	70	CY	$102.00	$7,140.00	
144	GEOTEXTILE	133	SY	$95.00	$12,635.00	
145	EARTHEN EMBANKMENT DAM	2,100	CY	$22.00	$46,200.00	
146	EARTHEN EMBANKMENT DAM - CLAY CORE (IMPORT)	400	CY	$222.00	$88,800.00	
147	EARTHEN EMBANKMENT DAM - RSP INSTALLATION, SLOPE	520	CY	$131.00	$68,120.00	
148	EARTHEN EMBANKMENT DAM - RSP INSTALLATION, SCOUR PROTECTION	270	CY	$131.00	$35,370.00	
149	16' AGGREGATE BASE ACCESS ROAD EXTENSION	185	LF	$75.00	$13,875.00	
	CONCRETE WEIR STRUCTURE					**$4,448,646.30**
150	SHORING AND PROTECTIVE STRUCTURES	8,800	SF	$60.00	$528,000.00	
151	EXCAVATION	2,600	CY	$20.00	$52,000.00	
152	BACKFILL	538	CY	$40.00	$21,511.11	
153	BACKFILL (CONTROL DENSITY FILL OR FLOWABLE FILL)	923	CY	$187.00	$172,601.00	
154	CONCRETE, SILL	1,023	CY	$1,000.00	$1,023,279.40	
155	CONCRETE, GRAVITY WALL (SOUTH BANK AND NORTH BANK)	83	CY	$750.00	$62,316.79	
156	CONCRETE, KEY WALL (WEIR CENTER WALLS)	36	CY	$750.00	$26,862.00	
157	OBERMEYER WEIR, 10' WEIR HT X 30' L, INCLDS CNTRLS (x3)	3	EA	$351,792.00	$1,055,376.00	
158	DRIVEN STEEL PIPE PILES, PP 71X40 ft	2,840	LF	$154.00	$437,360.00	
159	SHEET PILE WALL S1 (PERMENANT)	7,000	SF	$90.00	$630,000.00	
160	KING PILE WALL S4 _SHEET PILE PORTION (PERMENANT)	1,590	SF	$90.00	$143,100.00	
161	KING PILE WALL S4 _10 HZ PILE PORTION-55 FT LONG (PERMENANT)	550	LF	$100.00	$55,000.00	
162	SHEET PILE WALL S5 (PERMENANT)	5,680	SF	$40.00	$227,200.00	
163	STEEL CHANNEL C10X30 (18X26FT)	468	LF	$30.00	$14,040.00	
	SOUTHSIDE DIVERSION STRUCTURE					**$371,203.25**
163	CONCRETE WALL	247	CY	$750.00	$184,981.50	
164	CONCRETE SLAB	109	CY	$750.00	$81,501.75	
165	DEEP FOUNDATION PILES 17X40FT	680	LF	$154.00	$104,720.00	
	MORMON SLOUGH IN-CHANNEL WORK (ROUGHENED CHANNEL)					**$5,688,535.09**
166	EXCAVATION	33,433	CY	$20.00	$668,650.20	
167	FOUNDATION ROCK (ARMOR LAYER)	11,940	CY	$170.00	$2,029,800.00	
168	ENGINEERED STREAMBED MATERIAL	3,590	CY	$109.00	$391,310.00	
169	FILTER LAYER	3,590	CY	$102.00	$366,180.00	
170	TRANSITION LAYER	3,590	CY	$95.00	$341,050.00	
171	GEOTEXTILE	5,975	SY	$2.00	$11,950.00	
172	48"-54" BOULDERS	1,464	CY	$187.00	$273,768.00	
173	60"-72" BOULDERS	2,190	CY	$204.00	$446,846.89	
174	INFILL SEDIMENT	1,200	CY	$95.00	$114,000.00	
175	RSP TYPE A	5,980	CY	$170.00	$1,016,600.00	
176	FINISHED GRADING	14,190	SY	$2.00	$28,380.00	
	BELLOTA POOL IN-CHANNEL WORK					**$89,216.67**
177	FLOATING DEBRIS BARRIER	1	EA	$80,000.00	$80,000.00	
178	EXCAVATION	301	CY	$20.00	$6,016.67	
179	FINISHED GRADING	1,600	SY	$2.00	$3,200.00	
	ELECTRICAL SYSTEM					**$1,398,900.00**
180	ELECTRICAL UTILITY SERVICE	1	LS	$120,000.00	$120,000.00	
181	GROUNDING AND BONDING	1	LS	$41,600.00	$41,600.00	
182	HANGERS AND SUPPORTS	1	LS	$25,000.00	$25,000.00	
183	WIRING DEVICES	1	LS	$15,000.00	$15,000.00	
184	POWER DISTRIBUTION FEEDERS	1	LS	$368,000.00	$368,000.00	
185	SWITCHBOARD, PANELBOARDS, AND MCC	1	LS	$194,800.00	$194,800.00	
186	450 KW ENGINE GENERATOR	1	LS	$250,000.00	$250,000.00	
187	ACCEPTANCE TESTING AND POWER SYSTEMS ANALYSIS	1	LS	$74,000.00	$74,000.00	
188	BUILDING & PROCESS AREA LIGHTING	1	LS	$89,700.00	$89,700.00	
189	SITE LIGHTING	1	LS	$220,800.00	$220,800.00	
	INSTRUMENTATION AND CONTROLS					**$857,662.00**
190	TELEMETRY SYSTEMS	1	LS	$193,162.00	$193,162.00	
191	CONTROL SYSTEM EQUIPMENT PANELS AND RACKS	1	LS	$113,500.00	$113,500.00	
192	PLC AND SCADA PROGRAMMING	1	LS	$450,000.00	$450,000.00	
193	INSTRUMENTATION	1	LS	$101,000.00	$101,000.00	
					SUBTOTAL	**$ 30,926,000**
	MOBILIZATION (5%)					$ 1,547,000
	GENERAL CONDITIONS (10%)					$ 3,093,000
	BOND (2%)					$ 619,000
	GENERAL CONTRACTORS OH AND PROFIT (15%)					$ 4,639,000
					SUBTOTAL	**$ 40,824,000**
	TAXES (9%)					$ 2,783,000
					TOTAL BASE CONSTRUCTION COSTS	***$ 43,607,000***
					DESIGN CONTINGENCY (10%)	$ 4,361,000
					TOTAL WITH CONTINGENCY	***$ 47,970,000***
					ESCALATION (ASSUME 5% PER YEAR OVER 5 YEARS)	$ 13,253,000
					TOTAL WITH ESCALATION	***$ 61,223,000***
					CONSTRUCTION PHASING COMPLEXITY (4%)	$ 2,449,000
					MARKET VOLATILITY ADJUSMENT FACTOR (3%)	$ 1,837,000
					SUBTOTAL WITH MARKET VOLITILITY	***$ 65,509,000***
					ALLOWANCE FOR CHANGE ORDERS DURING CONSTRUCTION (5%)	$ 3,275,000
					TOTAL DIRECT CONSTRUCTION COSTS	***$ 68,784,000***

STOCKTON EAST WATER DISTRICT
BELLOTA WEIR MODIFICATIONS
SOFT COST SUMMARY

ITEM	DESCRIPTION	PERCENTAGE OF CONSTRUCTION COST	COST	TOTAL
	SOFT COSTS			**$11,005,440**
1	TESTING, MATERIAL AND GENERAL	1%	$ 687,840	
2	CONSTRUCTION MANAGEMENT SERVICES	5%	$ 3,439,200	
3	CONSTRUCTION INSPECTION SERVICES	5%	$ 3,439,200	
4	ENGINEERING SERVICES DURING CONSTRUCTION	5%	$ 3,439,200	
	TOTAL INDIRECT COSTS PERCENTAGE	16%	$ 11,005,440	

Project: BELLOTA WEIR MODIFICATIONS
Subject: 90% DESIGN
Task: OPCC
Job #: 10209137

Computed by: Theo Malone
Created: 2/4/2022
Printed: 2/7/2022
Checked by: Joy Terry
Date: 2/4/2022

STOCKTON EAST WATER DISTRICT
BELLOTA WEIR MODIFICATIONS PROJECT
PRELIMINARY OPINION OF PROBABLE CONSTRUCTION COST
CONSTRUCTION SEQUENCE COSTS

Row Labels	Sum of SUBTOTAL	ESCALATION (5%/YR)	TOTAL w/ ESCALATION	Sequence 1	Sequence 2	Sequence 3	Sequence 4	Sequence 5
1	$ 18,583,417	$ 929,171	$ 19,512,588	$ 19,512,588				
2	$ 3,951,574	$ 405,036	$ 4,356,610		$ 4,356,610			
3	$ 1,428,554	$ 225,176	$ 1,653,730			$ 1,653,730		
4	$ 9,727,320	$ 2,096,298	$ 11,823,618				$ 11,823,618	
5	$ 2,581,608	$ 713,251	$ 3,294,859					$ 3,294,859
1-5	$ 2,473,251	$ 396,666	$ 2,869,917	$ 519,383	$ 545,352	$ 572,619	$ 601,250	$ 631,313
2,4	$ 1,688,240	$ 268,435	$ 1,956,676		$ 930,643		$ 1,026,033	
4,5	$ 388,080	$ 95,427	$ 483,507				$ 235,857	$ 247,650
Grand Total	**$ 40,822,045**	**$ 5,130,000**	**$ 45,952,000**	**$ 20,032,000**	**$ 5,833,000**	**$ 2,227,000**	**$ 13,687,000**	**$ 4,174,000**
DESIGN CONTINGENCY (10%)			*$ 4,595,000*	*$ 2,003,000*	*$ 583,000*	*$ 223,000*	*$ 1,369,000*	*$ 417,000*
CONSTRUCTION PHASING COMPLEXITY (4%)			*$ 1,838,000*	*$ 801,000*	*$ 233,000*	*$ 89,000*	*$ 547,000*	*$ 167,000*
MARKET VOLATILITY ADJUSMENT FACTOR (3%)			*$ 1,379,000*	*$ 601,000*	*$ 175,000*	*$ 67,000*	*$ 411,000*	*$ 125,000*
SUBTOTAL WITH MARKET VOLITILITY			***$ 53,764,000***	***$ 23,437,000***	***$ 6,824,000***	***$ 2,606,000***	***$ 16,014,000***	***$ 4,883,000***
ALLOWANCE FOR CHANGE ORDERS DURING CONSTRUCTION (5%)			*$ 2,688,000*	*$ 1,172,000*	*$ 341,000*	*$ 130,000*	*$ 801,000*	*$ 244,000*
TOTAL DIRECT CONSTRUCTION COSTS			$ 56,452,000	$ 24,609,000	$ 7,165,000	$ 2,736,000	$ 16,815,000	$ 5,127,000
INDIRECT CONSTRUCTION COSTS			$ 9,032,320	$ 3,937,440	$ 1,146,400	$ 437,760	$ 2,690,400	$ 820,320
TOTAL DIRECT AND INDIRECT CONSTRUCTION COSTS			$ 65,484,320	$ 28,546,440	$ 8,311,400	$ 3,173,760	$ 19,505,400	$ 5,947,320

Project: BELLOTA WEIR MODIFICATIONS
Subject: 90% DESIGN
Task: OPCC
Job #: 10209137

Computed by: Joy Terry
Created: 5/30/2019
Printed:2/7/2022
Checked by: Theo Malone
Date: 2/7/2022

STOCKTON EAST WATER DISTRICT
BELLOTA WEIR MODIFICATIONS
SUMMARY OF OPERATION AND MAINTENANCE COSTS

Item		Quantity		Unit Cost	Amount	Total
LABOR						
Normal Operation and Inspection						**$67,257**
Biology technician direct labor cost	0.09808	FTE	$48,100	$4,717.50	
(one 7 hour visit per week during period of migration)					
Biology technician benefits @1.15 labor cost	0.09808	FTE	$55,315	$5,425	
Engineering or mechanical technician direct labor cost	0.32308	FTE	$70,200	$22,680	
(three 8 hour visits per week during period of migration)					
Engineering or mechanical technician benefits @1.15 labor cost	0.32308	FTE	$80,730	$26,082	
Maintenance Technician / laborer direct labor cost	0.08077	FTE	$48,100	$3,885	
(one 6 hour visit per week during period of migration)					
Maintenance tech / laborer benefits @ 1.15 labor cost	0.08077	FTE	$55,315	$4,468	
Annual Inspection and Maintenance						**$55,201**
Engineering technician direct labor cost	0.05769	FTE	$70,200	$4,050	
(assume 8 hours a day for 15 days)					
Engineering technician benefits @1.15 labor cost	0.05769	FTE	$80,730	$4,658	
Mechanical technician direct labor cost	0.05769	FTE	$70,200	$4,050	
(assume 8 hours a day for 15 days)					
Mechanical technician benefits @1.15 labor cost	0.05769	FTE	$80,730	$4,658	
Maintenance technician direct labor cost	0.36538	FTE	$48,100	$17,575	
(assume 8 hours a day for 95 days)					
Laborer benefits @1.15 labor cost	0.36538	FTE	$55,315	$20,211	
Monitoring and Reporting						**$20,705**
Biology lead direct labor cost	0.07692	FTE	$57,850	$4,450	
(assume 160 hours a year)					
Biology lead benefits @1.15 labor cost	0.07692	FTE	$66,528	$5,118	
Biology technician direct labor cost	0.10769	FTE	$48,100	$5,180	
(8 hrs per week during period of migration)					
Biology technician benefits @1.15 labor cost	0.10769	FTE	$55,315	$5,957	
FTE = Full time equivalent						
Subcontractors						**$5,264**
Annual rock manipulation and repair	1	LS	5,264	5,264	
EXPENDABLES						**$20,003**
Electrical					
General	3600	kWh	$0.230	$828	
Obermeyer weir low pressure air systems	10460.8	kWh	$0.230	$2,406	
Vehicles					
Mileage	15250	MI	$0.575	$8,769	
Equipment repair and service materials	1	LS	$6,000	$6,000	
Disposal Costs	1	LS	$2,000	$2,000	
TOTAL ANTICIPATED OPERATIONS AND MAINTENANCE COSTS (ROUNDED $100):					**$169,000**

Appendix E. Hydraulic Calculations

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FISH PASSAGE CALCULATIONS

HYDRAULIC PROFILE

FOR THE

BELLOTA WEIR MODIFICATIONS PROJECT

PREPARED FOR

STOCKTON EAST WATER DISTRICT

CALCULATIONS PREPARED BY

HDR ENGINEERING, INC.

RACHEL AINSLIE, E.I.T.

DATE PREPARED: 12/10/21

QC REVIEWER: SHAUN BEVAN, P.E.

QC REVIEW DATE: 12/21/21

DATE SUBMITTED: 2/04/21

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Exp. MM-DD-YYYY

Project:	**Bellota**	Computed:	**RA**	Date:	
Subject:		Checked:	**SB / MP**	Date:	
Task:	**Hydraulic Profile Calculations**	Page:		of:	
Job #:		No:			

As of CEQA 11/12/21
Accepted by SEWD as best representation of agreed upon flows with Board

	Diversion Demands (in cfs)	
	Irrigation Season	Non-Irrigation Season
Demand/ Flow Description	April 10 to 15-Oct	October 16 to 9-Apr
Current Demands/Flows		
Bellota Diversion	70	46
OCR Diversion	150	15
Future Demands/Flows		
Bellota Diversion	200	60
OCR Diversion	150	15

Design WSELs per HDR Internal Design Charette 6/29/2021
req'd at Reservoir Pool for 46 cfs Bellota (SEWD) demand: 115.44
req'd at Reservoir Pool for 70 cfs Bellota (SEWD) demand: 121.44

*Assumption: Scale flow distribution at high flow between T-screens by T-screen surface area so have uniform approach velocity.

Intake Screen Ratios

	Screen Area	Area Ratio
ScrA: 48" dia T-screens with each screen cylinder 72" wide:	151 sf	12.8%
ScrB: 48" dia T-screens with each screen cylinder 72" wide:	151 sf	12.8%
crC: 83" dia T-screens with each screen cylinder 120" wide:	440 sf	37.2%
crD: 83" dia T-screens with each screen cylinder 120" wide:	440 sf	37.2%
Total:	1181 sf	100.0%

**Ratio flow only for high flow design cases.

Design Cases

Scenario 2a Current, non-irrigation

Pool WSEL:	115.44
Bellota (SEWD) current:	46 cfs
OCR (Calaveras) current:	15 cfs
Total current:	61 cfs
T-Screen A:	31 cfs
T-Screen B:	31 cfs
T-Screen C:	closed
T-Screen D:	closed
Headloss to OCR:	**1.4 ft**
Headloss to SEWD:	**1.6 ft**
HGL at Manhole:	**113.8 ft**

Scenario 4a Current, irrigation

Pool WSEL:	121.44
Bellota (SEWD) current:	70 cfs
OCR (Calaveras) current:	150 cfs
Total current:	220 cfs
T-Screen A:	56 cfs
T-Screen B:	closed
T-Screen C:	164 cfs
T-Screen D:	closed
Headloss to OCR:	**5.5 ft**
Headloss to SEWD:	**2.5 ft**
HGL at Manhole:	**119.0 ft**

Scenario 2b Future, non-irrigation

Pool WSEL:	115.44
Bellota (SEWD) future:	60 cfs
OCR (Calaveras) future:	15 cfs
Total future:	75 cfs
T-Screen A:	38 cfs
T-Screen B:	38 cfs
T-Screen C:	closed
T-Screen D:	closed

Scenario 4b Future, irrigation

Pool WSEL:	121.44
Bellota (SEWD) future:	200 cfs
OCR (Calaveras) future:	150 cfs
Total future:	350 cfs
T-Screen A:	45 cfs
T-Screen B:	45 cfs
T-Screen C:	130 cfs
T-Screen D:	130 cfs

Headloss to OCR:	**1.4 ft**	***Headloss to OCR:***	**5.5 ft**
Headloss to SEWD:	**2.0 ft**	***Headloss to SEWD:***	**2.4 ft**
HGL at Manhole:	**113.4 ft**	***HGL at Manhole:***	**119.0 ft**

Scenario 5 Current, irrigation, use all 3 Bellota Conveyance Pipelines

Pool WSEL:	121.44
Bellota (SEWD) current:	70 cfs
OCR (Calaveras) current:	150 cfs
Total current:	220 cfs
T-Screen A:	56 cfs
T-Screen B:	closed
T-Screen C:	164 cfs
T-Screen D:	closed
Headloss to OCR:	**5.5 ft**
Headloss to SEWD:	**0.9 ft**
HGL at Manhole:	**120.5 ft**

From spreadsheet Cylindrical Screen Calcs; Folder 5.2/09_Calcs/65%_Intermediate_Review/Civil

Scenario	Current or Future	Time of Year	Max or Min	Bellota flow *cfs*	OCR flow *cfs*	Total *cfs*
1a	Current	Non-irrigation	Min	46	0	46
2a	Current	Non-irrigation	Max	46	15	61
3a	Current	Irrigation	Min	46	0	46
4a	Current	Irrigation	Max	70	150	220
1b	Future	Non-irrigation	Min	60	0	60
2b	Future	Non-irrigation	Max	60	15	75
3b	Future	Irrigation	Min	60	0	60
4b	Future	Irrigation	Max	200	150	350

Bellota Weir Modifications
System Hydraulic Profile
Scenario 2a

Color Key	Manual Entry
	Formula

Assumptions:
- Do not use Hazen-Williams for surface water pipe because Manning's equation provides more conservative headloss for long lengths of pipe.
- 6" air vacuum/air release valves on 36" pipe create negligible headloss.

T-Screens to D-Box

											Steel Pipe							Old Pipe		
			Invert EL	Upstream	Downstream	Pipe	Slope	Flow	Flow	Nominal	Inside	Area	Vel	V^2	Loss Coef	Minor	Pipe	Hydraulic Grade Line	Energy Grade Line
	Element	Component	(US; ft)	Pipe STA	Pipe STA	Length		(cfs)	(gpm)	Size (in)	Dia. (in)	(SF)	(ft/s)	2g	C or k	Loss (ft)	Loss	(Head) (ft)	(Head)(ft)
	Scenario 2a Current Non-irrigation: T-screens to wet well																	115.44	115.44
	a	T-screen losses	109.44	0+00.0	-	0	0.0000	15.25	6844.2	60	60	19.63	0.78	0.01	-	1.00	0.00	114.43	114.44
	d	gate valve full open	109.44	0+10.0	-	0	0.0000	30.50	13688	60	60	19.63	1.55	0.04	0.19	0.01	0.00	114.40	114.43
	e	exit loss	109.44	0+10.0	-	0	0.0000	30.50	13688	60	60	19.63	1.55	0.04	1	0.04	0.00	114.36	114.40
	f	pipe length	108.05	0+10.0	0+50.0	40	0.0223	30.50	13688	9.335	9.335	79	0.39	0.00	0.013	0.00	0.00	114.39	114.39
	g	combining flow	108.12	0+33.0	-	0	0.0223	61.00	27377	15	15	107	0.57	0.01	0.68	0.00	0.00	114.38	114.39
	h	pipe length	106.85	0+50.0	1+02.3	52.25	0.0223	61.00	27377	19.835	19.835	260.78	0.23	0.00	0.013	0.00	0.00	114.39	114.39
	i	exit loss	106.27	1+02.3	-	0	0.0000	61.00	27377	9.335	9.335	233.38	0.26	0.00	1	0.00	0.00	114.39	114.39
	j	D-Box HGL & EGL	100.50	1+02.3	-	0	0.0000	61.00	27377	0	0	0.00	0.00	0.00	0	0.00	0.00	114.39	114.39

Top of Pipe @ End of Section:	100.50
Total Section Loss:	1.05
Max Internal Pressure:	3.18
Static Internal Pressure:	6.47

Invert of pipe in dist box	112.6	ft
Invert of pipe in OCR structure	112	ft

See OCRheadloss tab.

Top of Pipe @ End of Section:	116.50
HGL in OCR channel:	114.00
Channel IE @ End of Section:	113.00
Total Section Loss:	0.39
Max Internal Pressure:	-0.92
Static Internal Pressure:	1.03

High flow case A: T-screens to wet well

											Steel Pipe							Old Pipe		
			Invert EL	Upstream	Downstream	Pipe	Slope	Flow	Flow	Nominal	Inside	Area	Vel	V^2	Loss Coef	Minor	Pipe	Hydraulic Grade Line	Energy Grade Line
	Element	Component	(US; ft)	Pipe STA	Pipe STA	Length		(cfs)	(gpm)	Size (in)	Dia. (in)	(SF)	(ft/s)	2g	C or k	Loss (ft)	Loss	(Head) (ft)	(Head)(ft)
																		114.39	114.39
	ba	ent loss	101.00	100+04.7	-	0	0.0000	46.00	20645	54	54	15.90	2.89	0.13	0.5	0.06	0.00	114.19	114.32
D-	bb	chem injection port loss	101.00	100+04.7	-	0	0.0000	46.00	20645	54	54	15.90	2.89	0.13	0.2	0.03	0.00	114.17	114.30
	bc	pipe length	101.00	100+04.7	100+22.2	17.5	0.0000	46.00	20645	54	54	15.90	2.89	0.13	0.013	0.00	0.01	114.16	114.29
	bd	45 deg bend (horz)	101.00	100+22.2	-	0	0.0000	46.00	20645	54	54	15.90	2.89	0.13	0.18	0.02	0.00	114.13	114.26
	be	pipe length	101.00	100+22.2	100+38.4	16.2	0.0000	46.00	20645	54	54	15.90	2.89	0.13	0.013	0.00	0.01	114.12	114.25
	bf	45 deg bend (horz)	101.00	100+38.4	-	0	0.0000	46.00	20645	54	54	15.90	2.89	0.13	0.18	0.02	0.00	114.10	114.23
	bg	pipe length	101.00	100+38.4	102+93.4	255	0.0163	46.00	20645	54	54	15.90	2.89	0.13	0.013	0.00	0.14	113.96	114.09

Bellota Weir Modifications
System Hydraulic Profile
Scenario 2a

Color Key	Manual Entry
	Formula

Assumptions:
- Do not use Hazen-Williams for surface water pipe because Manning's equation provides more conservative headloss for long lengths of pipe.
- 6" air vacuum/air release valves on 36" pipe create negligible headloss.

bh	30 deg bend (vert)	101.00	100+45.6	-	0	0.0163	46.00	20645	54	54	15.90	2.89	0.13	0.1	0.01	0.00	113.95	114.08
bj	45 deg bend (horz)	106.00	102+93.4	-	0	0.0171	46.00	20645	54	54	15.90	2.89	0.13	0.18	0.02	0.00	113.93	114.05
bk	pipe length	106.00	102+93.4	103+88.4	95	0.0171	46.00	20645	54	54	15.90	2.89	0.13	0.013	0.00	0.05	113.87	114.00
bl	flow meter	106.00	103+88.4	-	0	0.0171	46.00	20645	54	54	15.90	2.89	0.13	0.2	0.03	0.00	113.85	113.98
bm	exit loss	106.32	103+88.4	-	0	0.0171	46.00	20645	54	54	15.90	2.89	0.13	1	0.13	0.00	113.72	113.85
bn	existing conveyance pipe	106.32	103+88.4	-	0	0.0000	46.00	20645	54	54	15.90	2.89	0.13	0	0.00	0.00	113.72	113.85

Top of Pipe @ End of Section:	110.82
Total Section Loss:	0.54
Max Internal Pressure:	3.77
Static Internal Pressure:	3.49
HGL at MH:	113.72

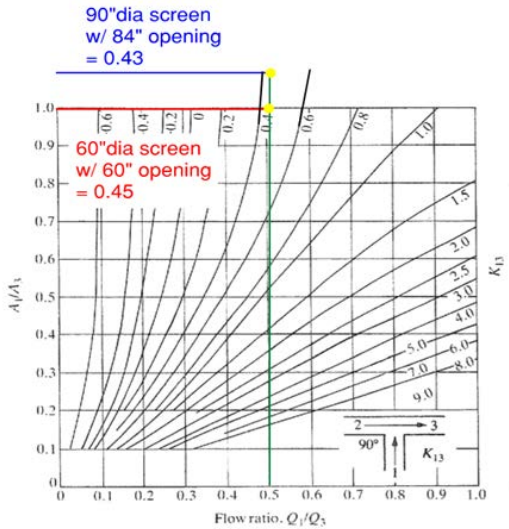


Fig. 13.10†. Combining flow: branch angle 90°, loss coefficient K_{13}

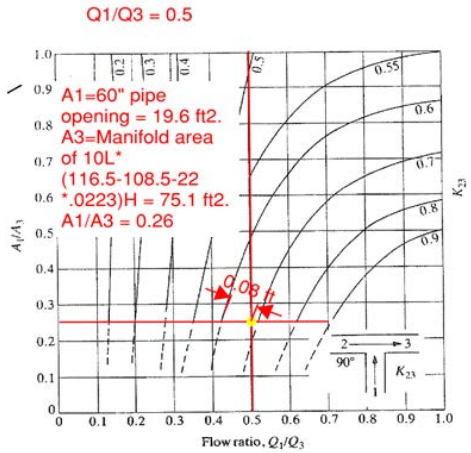


Fig. 13.11. Combining flow: branch angle 90°, loss coefficient K_{23}

element b
Combining flow within tee screen
0.45

element g
combining flow; Scenario 1-3a, 1-3b: Screen 2 flow combining with flow in manifold
0.68

Pipe n (steel)	0.013	CERM A-36
Pipe n (PVC/HDPE)	0.009	CERM A-36
Pipe n (concrete)	0.013	CERM A-36
Check valve	2.3	CERM 17-12
30-degree bend	0.1	Larry Pauls reference (see addl tab)
45-degree bend	0.18	Larry Pauls reference (see addl tab)
90-degree bend	0.72	Larry Pauls reference (see addl tab)
45-degree elbow	0.19	Larry Pauls reference (see addl tab)
entrance	0.5	CERM 17-13
exit	1	CERM 17-13
Gate valve	0.19	CERM 17-12

Bellota Weir Modifications
System Hydraulic Profile
Scenario 2a

Color Key	Manual Entry
	Formula

Assumptions:
- Do not use Hazen-Williams for surface water pipe because Manning's equation provides more conservative headloss for long lengths of pipe.
- 6" air vacuum/air release valves on 36" pipe create negligible headloss.

flow meter	0.2	Matt assumed this but no reference?
combining flow; within 60" tee screen	0.45	Figure below, DS Miller Internal Flow Systems
combining flow; Scenario 1-3a, 1-4b: Screen 2 flow combining with flow in manifold	0.68	Miller Internal Flow Systems
combining flow; Scenario 4a: Screen 3 flow combining with flow in manifold.	0.86	
combining flow; Scenario 4b: Screen 3 flow combining with flow in manifold	0.65	
combining flow; Scenario 4b: Screen 4 flow combining with flow in manifold	0.55	
through screen	0.5	Matt assumed this but no reference?
loss for 60" screen, max (through screen; suction pipe; and inlet)*	1	Per conversation with ISI, 12/9/21
loss for 60" screen, min (through screen; suction pipe; and inlet)*	0.5	Per conversation with ISI, 12/9/21
loss for 90" screen, max (through screen; suction pipe; and inlet)*	1.17	Per conversation with ISI, 12/9/21
loss for 90" screen, min (through screen; suction pipe; and inlet)*	0.67	Per conversation with ISI, 12/9/21
*Screen sizes were updated, values left as is to be conservative		

Bellota Weir Modifications
System Hydraulic Profile
Scenario 4a

Color Key	Manual Entry
	Formula

Assumptions:
- Do not use Hazen-Williams for surface water pipe because Manning's equation provides more conservative headloss for long lengths of pipe.
- 6" air vacuum/air release valves on 36" pipe create negligible headloss.

T-Screens to D-Box

										Steel Pipe								Old Pipe			
			Invert EL	Upstream	Downstream	Pipe	Slope	Flow	Flow	Nominal	Inside	Area	Vel	V^2	Loss Coef	Minor	Pipe	Hydraulic Grade Line	Energy Grade Line	
	Element	Component	(US; ft)	Pipe STA	Pipe STA	Length		(cfs)	(gpm)	Size (in)	Dia. (in)	(SF)	(ft/s)	2g	C or k	Loss (ft)	Loss	(Head) (ft)	(Head)(ft)	psi
Scenario 2a Current Non-irrigation: T-screens to wet well																		121.44	121.44	
	a	T-screen losses	109.44	0+00.0	-	0	0.0000	28.09	12605	60	60	19.63	1.43	0.03	-	1.00	0.00	120.41	120.44	2.59
	d	gate valve full open	109.44	0+10.0	-	0	0.0000	56.17	25209	60	60	19.63	2.86	0.13	0.19	0.02	0.00	120.29	120.42	2.53
	e	exit loss	109.44	0+10.0	-	0	0.0000	56.17	25209	60	60	19.63	2.86	0.13	1	0.13	0.00	120.16	120.29	2.48
	f	pipe length	108.05	0+10.0	0+50.0	40	0.0223	56.17	25209	9.335	9.335	79	0.71	0.01	0.013	0.00	0.01	120.27	120.28	4.96
	g	combining flow	108.12	0+33.0	-	0	0.0223	220.00	98736	15	15	107	2.06	0.07	0.68	0.04	0.00	120.16	120.23	4.68
	h	pipe length	106.85	0+50.0	1+02.3	52.25	0.0223	220.00	98736	19.835	19.835	260.78	0.84	0.01	0.013	0.00	0.01	120.21	120.22	5.07
	i	exit loss	106.27	1+02.3	-	0	0.0000	220.00	98736	9.335	9.335	233.38	0.94	0.01	1	0.01	0.00	120.19	120.21	5.70
	j	D-Box HGL & EGL	100.50	1+02.3	-	0	0.0000	220.00	98736	0	0	0.00	0.00	0.00	0	0.00	0.00	120.19	120.21	5.70

Top of Pipe @ End of Section:	100.50	ft
Total Section Loss:	1.23	ft
Max Internal Pressure:	5.70	psi
Static Internal Pressure:	9.07	psi

Invert in structure	112.6	ft
Invert in channel	112	ft

See OCRheadloss tab.

Top of Pipe @ End of Section:	116.50	ft
HGL in OCR channel:	115.90	ft
Channel IE @ End of Section:	113.00	ft
Total Section Loss:	4.29	ft
Max Internal Pressure:	1.60	psi
Static Internal Pressure:	3.55	psi

High flow case A: T-screens to wet well

										Steel Pipe								Old Pipe			
			Invert EL	Upstream	Downstream	Pipe	Slope	Flow	Flow	Nominal	Inside	Area	Vel	V^2	Loss Coef	Minor	Pipe	Hydraulic Grade Line	Energy Grade Line	
	Element	Component	(US; ft)	Pipe STA	Pipe STA	Length		(cfs)	(gpm)	Size (in)	Dia. (in)	(SF)	(ft/s)	2g	C or k	Loss (ft)	Loss	(Head) (ft)	(Head)(ft)	psi
																		120.19	120.21	
	ba	ent loss	101.00	100+04.7	-	0	0.0000	70.00	31416	54	54	15.90	4.40	0.30	0.5	0.15	0.00	119.76	120.06	6.18
D-	bb	chem injection port loss	101.00	100+04.7	-	0	0.0000	70.00	31416	54	54	15.90	4.40	0.30	0.2	0.06	0.00	119.70	120.00	6.15
	bc	pipe length	101.00	100+04.7	100+22.2	17.5	0.0000	70.00	31416	54	54	15.90	4.40	0.30	0.013	0.00	0.02	119.67	119.97	6.14
	bd	45 deg bend (horz)	101.00	100+22.2	-	0	0.0000	70.00	31416	54	54	15.90	4.40	0.30	0.18	0.05	0.00	119.62	119.92	6.12
	be	pipe length	101.00	100+22.2	100+38.4	16.2	0.0000	70.00	31416	54	54	15.90	4.40	0.30	0.013	0.00	0.02	119.60	119.90	6.11
	bf	45 deg bend (horz)	101.00	100+38.4	-	0	0.0000	70.00	31416	54	54	15.90	4.40	0.30	0.18	0.05	0.00	119.54	119.85	6.09
	bg	pipe length	101.00	100+38.4	102+93.4	255	0.0163	70.00	31416	54	54	15.90	4.40	0.30	0.013	0.00	0.32	119.22	119.52	5.95
	bh	30 deg bend (vert)	101.00	102+60.7	-	0	0.0163	70.00	31416	54	54	15.90	4.40	0.30	0.1	0.03	0.00	119.19	119.49	5.93

Bellota Weir Modifications
System Hydraulic Profile
Scenario 4a

Color Key	Manual Entry
	Formula

Assumptions:
- Do not use Hazen-Williams for surface water pipe because Manning's equation provides more conservative headloss for long lengths of pipe.
- 6" air vacuum/air release valves on 36" pipe create negligible headloss.

bj	45 deg bend (horz)	106.00	102+93.4	-	0	0.0171	70.00	31416	54	54	15.90	4.40	0.30	0.18	0.05	0.00	119.14	119.44	3.74
bk	pipe length	106.00	102+93.4	103+88.4	95	0.0171	70.00	31416	54	54	15.90	4.40	0.30	0.013	0.00	0.12	119.02	119.32	3.69
bl	flow meter	106.00	103+88.4	-	0	0.0171	70.00	31416	54	54	15.90	4.40	0.30	0.2	0.06	0.00	118.96	119.26	3.67
bm	exit loss	106.32	103+88.4	-	0	0.0171	70.00	31416	54	54	15.90	4.40	0.30	1	0.30	0.00	118.66	118.96	3.40
bn	existing conveyance pipe	106.32	103+88.4	-	0	0.0000	70.00	31416	54	54	15.90	4.40	0.30	0	0.00	0.00	118.66	118.96	3.40

Top of Pipe @ End of Section:	110.82	ft
Total Section Loss:	1.25	ft
Max Internal Pressure:	6.18	psi
Static Internal Pressure:	6.01	psi
HGL at MH:	118.66	ft

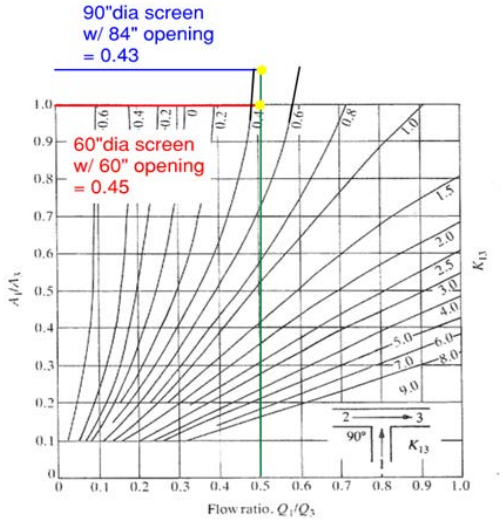


Fig. 13.10†. Combining flow: branch angle 90°, loss coefficient K_{13}

element b
Combining flow within tee screen
0.45

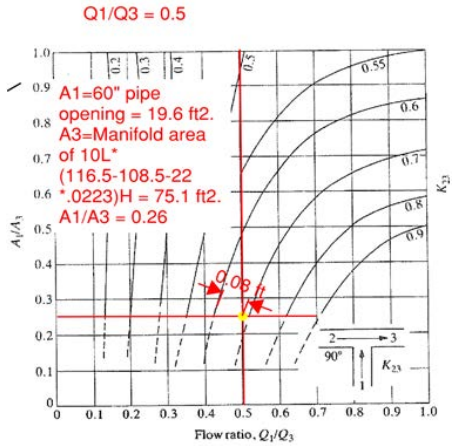


Fig. 13.11. Combining flow: branch angle 90°, loss coefficient K_{23}

element g
combining flow; Scenario 1-3a, 1-3b: Screen 2 flow combining with flow in manifold
0.68

Pipe n (steel)	0.013	CERM A-36
Pipe n (PVC/HDPE)	0.009	CERM A-36
Pipe n (concrete)	0.013	CERM A-36
Check valve	2.3	CERM 17-12
30-degree bend	0.1	Larry Pauls reference (see addl tab)
45-degree bend	0.18	Larry Pauls reference (see addl tab)
90-degree bend	0.72	Larry Pauls reference (see addl tab)
45-degree elbow	0.19	Larry Pauls reference (see addl tab)
entrance	0.5	CERM 17-13
exit	1	CERM 17-13
Gate valve	0.19	CERM 17-12

Bellota Weir Modifications
System Hydraulic Profile
Scenario 4a

Color Key	Manual Entry
	Formula

Assumptions:
- Do not use Hazen-Williams for surface water pipe because Manning's equation provides more conservative headloss for long lengths of pipe.
- 6" air vacuum/air release valves on 36" pipe create negligible headloss.

flow meter	0.2	Matt assumed this but no reference?
combining flow; within 60" tee screen	0.45	Figure below, DS Miller Internal Flow Systems
combining flow; Scenario 1-3a, 1-4b: Screen 2 flow combining with flow in manifold	0.68	Figure below, DS Miller Internal Flow Systems
combining flow; Scenario 4a: Screen 3 flow combining with flow in manifold.	0.86	
combining flow; Scenario 4b: Screen 3 flow combining with flow in manifold	0.65	
combining flow; Scenario 4b: Screen 4 flow combining with flow in manifold	0.55	
through screen	0.5	Matt assumed this but no reference?
loss for 60" screen, max (through screen; suction pipe; and inlet)*	1	Per conversation with ISI, 12/9/21
loss for 60" screen, min (through screen; suction pipe; and inlet)*	0.5	Per conversation with ISI, 12/9/21
loss for 90" screen, max (through screen; suction pipe; and inlet)*	1.17	Per conversation with ISI, 12/9/21
loss for 90" screen, min (through screen; suction pipe; and inlet)*	0.67	Per conversation with ISI, 12/9/21
*Screen sizes were updated, values left as is to be conservative		

Bellota Weir Modifications
System Hydraulic Profile
Scenario 2b

Color Key	Manual Entry
	Formula

Assumptions:
- Do not use Hazen-Williams for surface water pipe because Manning's equation provides more conservative headloss for long lengths of pipe.
- 6" air vacuum/air release valves on 36" pipe create negligible headloss.

T-Screens to D-Box

										Steel Pipe								Old Pipe			
			Invert EL	Upstream	Downstream	Pipe	Slope	Flow	Flow	Nominal	Inside	Area	Vel	V^2	Loss Coef	Minor	Pipe	Hydraulic Grade Line	Energy Grade Line	
	Element	Component	(US; ft)	Pipe STA	Pipe STA	Length		(cfs)	(gpm)	Size (in)	Dia. (in)	(SF)	(ft/s)	2g	C or k	Loss (ft)	Loss	(Head) (ft)	(Head)(ft)	psi
Scenario 2a Current Non-irrigation: T-screens to wet well																		115.44	115.44	
	a	T-screen losses	109.44	0+00.0	-	0	0.0000	18.75	8415	60	60	19.63	0.95	0.01	-	1.00	0.00	114.43	114.44	-0.01
	d	gate valve full open	109.44	0+10.0	-	0	0.0000	37.50	16830	60	60	19.63	1.91	0.06	0.19	0.01	0.00	114.37	114.43	-0.03
	e	exit loss	109.44	0+10.0	-	0	0.0000	37.50	16830	60	60	19.63	1.91	0.06	1	0.06	0.00	114.32	114.37	-0.05
	f	pipe length	108.05	0+10.0	0+50.0	40	0.0223	37.50	16830	9.335	9.335	79	0.48	0.00	0.013	0.00	0.01	114.36	114.37	2.40
	g	combining flow	108.12	0+33.0	-	0	0.0223	75.00	33660	15	15	107	0.70	0.01	0.68	0.01	0.00	114.35	114.36	2.16
	h	pipe length	106.85	0+50.0	1+02.3	52.25	0.0223	75.00	33660	19.835	19.835	260.78	0.29	0.00	0.013	0.00	0.00	114.36	114.36	2.54
	i	exit loss	106.27	1+02.3	-	0	0.0000	75.00	33660	9.335	9.335	233.38	0.32	0.00	1	0.00	0.00	114.36	114.36	3.17
	j	D-Box HGL & EGL	100.50	1+02.3	-	0	0.0000	75.00	33660	0	0	0.00	0.00	0.00	0	0.00	0.00	114.36	114.36	3.17

Top of Pipe @ End of Section: 100.50 ft
Total Section Loss: 1.08 ft
Max Internal Pressure: 3.17 psi
Static Internal Pressure: 6.47 psi

Invert in structure 112.6 ft
Invert in channel 112 ft

See OCRheadloss tab.

Top of Pipe @ End of Section: 116.50 ft
HGL in OCR channel: 114.00 ft
Channel IE @ End of Section: 113.00 ft
Total Section Loss: 0.36 ft
Max Internal Pressure: -0.93 psi
Static Internal Pressure: 1.02 psi

High flow case A: T-screens to wet well

										Steel Pipe								Old Pipe			
			Invert EL	Upstream	Downstream	Pipe	Slope	Flow	Flow	Nominal	Inside	Area	Vel	V^2	Loss Coef	Minor	Pipe	Hydraulic Grade Line	Energy Grade Line	
	Element	Component	(US; ft)	Pipe STA	Pipe STA	Length		(cfs)	(gpm)	Size (in)	Dia. (in)	(SF)	(ft/s)	2g	C or k	Loss (ft)	Loss	(Head) (ft)	(Head)(ft)	psi
																		114.36	114.36	
	ba	ent loss	101.00	100+04.7	-	0	0.0000	60.00	26928	54	54	15.90	3.77	0.22	0.5	0.11	0.00	114.03	114.25	3.70
D-	bb	chem injection port loss	101.00	100+04.7	-	0	0.0000	60.00	26928	54	54	15.90	3.77	0.22	0.2	0.04	0.00	113.98	114.20	3.68
	bc	pipe length	101.00	100+04.7	100+22.2	17.5	0.0000	60.00	26928	54	54	15.90	3.77	0.22	0.013	0.00	0.02	113.97	114.19	3.67
	bd	45 deg bend (horz)	101.00	100+22.2	-	0	0.0000	60.00	26928	54	54	15.90	3.77	0.22	0.18	0.04	0.00	113.93	114.15	3.65
	be	pipe length	101.00	100+22.2	100+38.4	16.2	0.0000	60.00	26928	54	54	15.90	3.77	0.22	0.013	0.00	0.02	113.91	114.13	3.65
	bf	45 deg bend (horz)	101.00	100+38.4	-	0	0.0000	60.00	26928	54	54	15.90	3.77	0.22	0.18	0.04	0.00	113.87	114.09	3.63
	bg	pipe length	101.00	100+38.4	102+93.4	255	0.0163	60.00	26928	54	54	15.90	3.77	0.22	0.013	0.00	0.24	113.64	113.86	3.53
	bh	30 deg bend (vert)	101.00	102+60.7	-	0	0.0163	60.00	26928	54	54	15.90	3.77	0.22	0.1	0.02	0.00	113.61	113.83	3.52

Bellota Weir Modifications
System Hydraulic Profile
Scenario 2b

Color Key	Manual Entry
	Formula

Assumptions:
- Do not use Hazen-Williams for surface water pipe because Manning's equation provides more conservative headloss for long lengths of pipe.
- 6" air vacuum/air release valves on 36" pipe create negligible headloss.

bj	45 deg bend (horz)	106.00	102+93.4	-	0	0.0171	60.00	26928	54	54	15.90	3.77	0.22	0.18	0.04	0.00	113.57	113.80	1.33
bk	pipe length	106.00	102+93.4	103+88.4	95	0.0171	60.00	26928	54	54	15.90	3.77	0.22	0.013	0.00	0.09	113.49	113.71	1.29
bl	flow meter	106.00	103+88.4	-	0	0.0171	60.00	26928	54	54	15.90	3.77	0.22	0.2	0.04	0.00	113.44	113.66	1.27
bm	exit loss	106.32	103+88.4	-	0	0.0171	60.00	26928	54	54	15.90	3.77	0.22	1	0.22	0.00	113.22	113.44	1.04
bn	existing conveyance pipe	106.32	103+88.4	-	0	0.0000	60.00	26928	54	54	15.90	3.77	0.22	0	0.00	0.00	113.22	113.44	1.04

Top of Pipe @ End of Section:	110.82	ft
Total Section Loss:	0.92	ft
Max Internal Pressure:	3.70	psi
Static Internal Pressure:	3.48	psi
HGL at MH:	113.22	ft

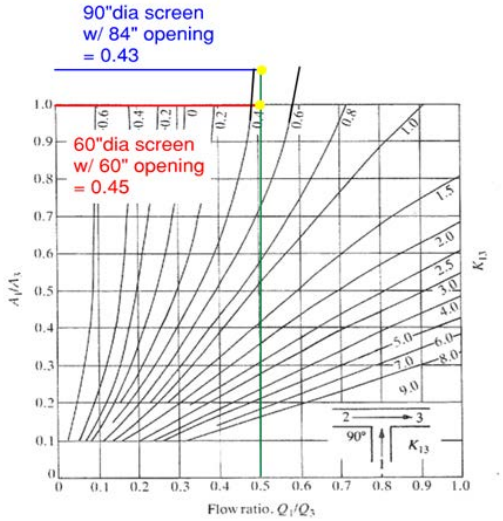


Fig. 13.10†. Combining flow: branch angle 90°, loss coefficient K_{13}

element b
Combining flow within tee screen
0.45

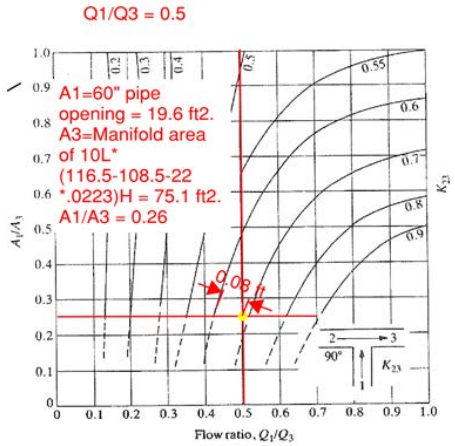


Fig. 13.11. Combining flow: branch angle 90°, loss coefficient K_{23}

element g
combining flow; Scenario 1-3a, 1-3b: Screen 2 flow combining with flow in manifold
0.68

Pipe n (steel)	0.013	CERM A-36
Pipe n (PVC/HDPE)	0.009	CERM A-36
Pipe n (concrete)	0.013	CERM A-36
Check valve	2.3	CERM 17-12
30-degree bend	0.1	Larry Pauls reference (see addl tab)
45-degree bend	0.18	Larry Pauls reference (see addl tab)
90-degree bend	0.72	Larry Pauls reference (see addl tab)
45-degree elbow	0.19	Larry Pauls reference (see addl tab)
entrance	0.5	CERM 17-13
exit	1	CERM 17-13
Gate valve	0.19	CERM 17-12
flow meter	0.2	Matt assumed this but no reference?

Bellota Weir Modifications
System Hydraulic Profile
Scenario 2b

Color Key	Manual Entry
	Formula

Assumptions:
- Do not use Hazen-Williams for surface water pipe because Manning's equation provides more conservative headloss for long lengths of pipe.
- 6" air vacuum/air release valves on 36" pipe create negligible headloss.

combining flow; within 60" tee screen	0.45	Figure below, DS Miller Internal Flow Systems
combining flow; Scenario 1-3a, 1-4b: Screen 2 flow combining with flow in manifold	0.68	Figure below, DS Miller Internal Flow Systems
combining flow; Scenario 4a: Screen 3 flow combining with flow in manifold.	0.86	
combining flow; Scenario 4b: Screen 3 flow combining with flow in manifold	0.65	
combining flow; Scenario 4b: Screen 4 flow combining with flow in manifold	0.55	
through screen	0.5	Matt assumed this but no reference?
loss for 60" screen, max (through screen; suction pipe; and inlet)*	1	Per conversation with ISI, 12/9/21
loss for 60" screen, min (through screen; suction pipe; and inlet)*	0.5	Per conversation with ISI, 12/9/21
loss for 90" screen, max (through screen; suction pipe; and inlet)*	1.17	Per conversation with ISI, 12/9/21
loss for 90" screen, min (through screen; suction pipe; and inlet)*	0.67	Per conversation with ISI, 12/9/21
*Screen sizes were updated, values left as is to be conservative		

Bellota Weir Modifications
System Hydraulic Profile
Scenario 4b

Color Key	Manual Entry
	Formula

Assumptions:
- Do not use Hazen-Williams for surface water pipe because Manning's equation provides more conservative headloss for long lengths of pipe.
- 6" air vacuum/air release valves on 36" pipe create negligible headloss.

T-Screens to D-Box

										Steel Pipe								Old Pipe			
	Element	Component	Invert EL (US; ft)	Upstream Pipe STA	Downstream Pipe STA	Pipe Length	Slope	Flow (cfs)	Flow (gpm)	Nominal Size (in)	Inside Dia. (in)	Area (SF)	Vel (ft/s)	V^2 2g	Loss Coef C or k	Minor Loss (ft)	Pipe Loss	Hydraulic Grade Line (Head) (ft)	Energy Grade Line (Head)(ft)	psi
Scenario 2a Current Non-irrigation: T-screens to wet well																		121.44	121.44	
	a	T-screen losses	109.44	0+00.0	-	0	0.0000	22.34	10026	60	60	19.63	1.14	0.02	-	1.00	0.00	120.42	120.44	2.59
	d	gate valve full open	109.44	0+10.0	-	0	0.0000	44.68	20053	60	60	19.63	2.28	0.08	0.19	0.02	0.00	120.34	120.42	2.56
	e	exit loss	109.44	0+10.0	-	0	0.0000	44.68	20053	60	60	19.63	2.28	0.08	1	0.08	0.00	120.26	120.34	2.52
	f	pipe length	108.05	0+10.0	0+50.0	40	0.0223	44.68	20053	9.335	9.335	79	0.57	0.00	0.013	0.00	0.01	120.33	120.34	4.98
	g	combining flow	108.12	0+33.0	-	0	0.0223	350.00	157080	15	15	107	3.28	0.17	0.68	0.11	0.00	120.05	120.22	4.63
	h	pipe length	106.85	0+50.0	1+02.3	52.25	0.0223	350.00	157080	19.835	19.835	260.78	1.34	0.03	0.013	0.00	0.02	120.17	120.20	5.05
	i	exit loss	106.27	1+02.3	-	0	0.0000	350.00	157080	9.335	9.335	233.38	1.50	0.03	1	0.03	0.00	120.13	120.16	5.67
	j	D-Box HGL & EGL	100.50	1+02.3	-	0	0.0000	350.00	157080	0	0	0.00	0.00	0.00	0	0.00	0.00	120.13	120.16	5.67

Top of Pipe @ End of Section: 100.50 ft
Total Section Loss: 1.28 ft
Max Internal Pressure: 5.67 psi
Static Internal Pressure: 9.07 psi

Invert in structure 112.6 ft
Invert in channel 112 ft

See OCRheadloss tab.

Top of Pipe @ End of Section: 116.50 ft
HGL in OCR channel: 115.90 ft
Channel IE @ End of Section: 113.00 ft
Total Section Loss: 4.23 ft
Max Internal Pressure: 1.57 psi
Static Internal Pressure: 3.52 psi

High flow case A: T-screens to wet well

										Steel Pipe								Old Pipe			
	Element	Component	Invert EL (US; ft)	Upstream Pipe STA	Downstream Pipe STA	Pipe Length	Slope	Flow (cfs)	Flow (gpm)	Nominal Size (in)	Inside Dia. (in)	Area (SF)	Vel (ft/s)	V^2 2g	Loss Coef C or k	Minor Loss (ft)	Pipe Loss	Hydraulic Grade Line (Head) (ft)	Energy Grade Line (Head)(ft)	psi
																		120.13	120.16	
	ba	ent loss	101.00	100+04.7	-	0	0.0000	66.67	29920	54	54	15.90	4.19	0.27	0.5	0.14	0.00	119.75	120.03	6.18
D-	bb	chem injection port loss	101.00	100+04.7	-	0	0.0000	66.67	29920	54	54	15.90	4.19	0.27	0.2	0.05	0.00	119.70	119.97	6.15
	bc	pipe length	101.00	100+04.7	100+22.2	17.5	0.0000	66.67	29920	54	54	15.90	4.19	0.27	0.013	0.00	0.02	119.68	119.95	6.14
	bd	45 deg bend (horz)	101.00	100+22.2	-	0	0.0000	66.67	29920	54	54	15.90	4.19	0.27	0.18	0.05	0.00	119.63	119.90	6.12
	be	pipe length	101.00	100+22.2	100+38.4	16.2	0.0000	66.67	29920	54	54	15.90	4.19	0.27	0.013	0.00	0.02	119.61	119.89	6.12
	bf	45 deg bend (horz)	101.00	100+38.4	-	0	0.0000	66.67	29920	54	54	15.90	4.19	0.27	0.18	0.05	0.00	119.56	119.84	6.09
	bg	pipe length	101.00	100+38.4	102+93.4	255	0.0163	66.67	29920	54	54	15.90	4.19	0.27	0.013	0.00	0.29	119.27	119.54	5.97
	bh	30 deg bend (vert)	101.00	102+60.7	-	0	0.0163	66.67	29920	54	54	15.90	4.19	0.27	0.1	0.03	0.00	119.24	119.52	5.96

Bellota Weir Modifications
System Hydraulic Profile
Scenario 4b

Color Key	Manual Entry
	Formula

Assumptions:
- Do not use Hazen-Williams for surface water pipe because Manning's equation provides more conservative headloss for long lengths of pipe.
- 6" air vacuum/air release valves on 36" pipe create negligible headloss.

bj	45 deg bend (horz)	106.00	102+93.4	-	0	0.0171	66.67	29920	54	54	15.90	4.19	0.27	0.18	0.05	0.00	119.20	119.47	3.77
bk	pipe length	106.00	102+93.4	103+88.4	95	0.0171	66.67	29920	54	54	15.90	4.19	0.27	0.013	0.00	0.11	119.09	119.36	3.72
bl	flow meter	106.00	103+88.4	-	0	0.0171	66.67	29920	54	54	15.90	4.19	0.27	0.2	0.05	0.00	119.03	119.30	3.70
bm	exit loss	106.32	103+88.4	-	0	0.0171	66.67	29920	54	54	15.90	4.19	0.27	1	0.27	0.00	118.76	119.03	3.44
bn	existing conveyance pipe	106.32	103+88.4	-	0	0.0000	66.67	29920	54	54	15.90	4.19	0.27	0	0.00	0.00	118.76	119.03	3.44

Top of Pipe @ End of Section:	110.82	ft
Total Section Loss:	1.13	ft
Max Internal Pressure:	6.18	psi
Static Internal Pressure:	5.98	psi
HGL at MH:	118.76	ft

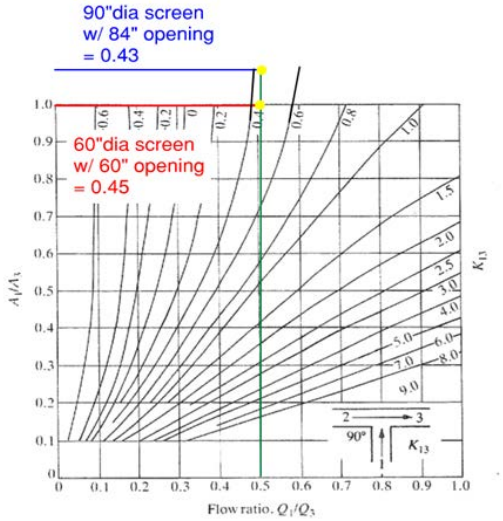


Fig. 13.10†. Combining flow: branch angle 90°, loss coefficient K_{13}

element b
Combining flow within tee screen
0.45

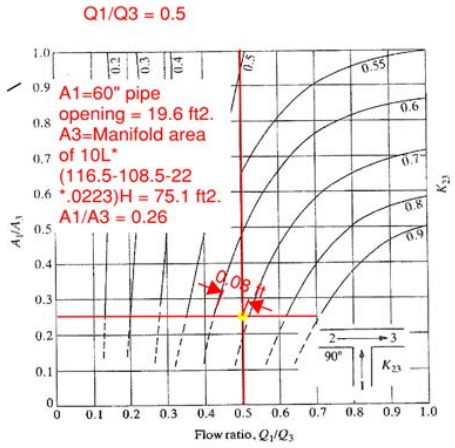


Fig. 13.11. Combining flow: branch angle 90°, loss coefficient K_{23}

element g
combining flow; Scenario 1-3a, 1-3b: Screen 2 flow combining with flow in manifold
0.68

Pipe n (steel)	0.013	CERM A-36
Pipe n (PVC/HDPE)	0.009	CERM A-36
Pipe n (concrete)	0.013	CERM A-36
Check valve	2.3	CERM 17-12
30-degree bend	0.1	Larry Pauls reference (see addl tab)
45-degree bend	0.18	Larry Pauls reference (see addl tab)
90-degree bend	0.72	Larry Pauls reference (see addl tab)
45-degree elbow	0.19	Larry Pauls reference (see addl tab)
entrance	0.5	CERM 17-13
exit	1	CERM 17-13
Gate valve	0.19	CERM 17-12
flow meter	0.2	Matt assumed this but no reference?

Bellota Weir Modifications
System Hydraulic Profile
Scenario 4b

Color Key	Manual Entry
	Formula

Assumptions:
- Do not use Hazen-Williams for surface water pipe because Manning's equation provides more conservative headloss for long lengths of pipe.
- 6" air vacuum/air release valves on 36" pipe create negligible headloss.

combining flow; within 60" tee screen	0.45	Figure below, DS Miller Internal Flow Systems
combining flow; Scenario 1-3a, 1-4b: Screen 2 flow combining with flow in manifold	0.68	Figure below, DS Miller Internal Flow Systems
combining flow; Scenario 4a: Screen 3 flow combining with flow in manifold.	0.86	
combining flow; Scenario 4b: Screen 3 flow combining with flow in manifold	0.65	
combining flow; Scenario 4b: Screen 4 flow combining with flow in manifold	0.55	
through screen	0.5	Matt assumed this but no reference?
loss for 60" screen, max (through screen; suction pipe; and inlet)*	1	Per conversation with ISI, 12/9/21
loss for 60" screen, min (through screen; suction pipe; and inlet)*	0.5	Per conversation with ISI, 12/9/21
loss for 90" screen, max (through screen; suction pipe; and inlet)*	1.17	Per conversation with ISI, 12/9/21
loss for 90" screen, min (through screen; suction pipe; and inlet)*	0.67	Per conversation with ISI, 12/9/21
*Screen sizes were updated, values left as is to be conservative		

Bellota Weir Modifications
System Hydraulic Profile
Scenario 5

Color Key	Manual Entry
	Formula

Assumptions:
- Do not use Hazen-Williams for surface water pipe because Manning's equation provides more conservative headloss for long lengths of pipe.
- 6" air vacuum/air release valves on 36" pipe create negligible headloss.

T-Screens to D-Box

										Steel Pipe								Old Pipe			
																		Hydraulic	Energy		
	Element	Component	Invert EL (US; ft)	Upstream Pipe STA	Downstream Pipe STA	Pipe Length	Slope	Flow (cfs)	Flow (gpm)	Nominal Size (in)	Inside Dia. (in)	Area (SF)	Vel (ft/s)	V^2 2g	Loss Coef C or k	Minor Loss (ft)	Pipe Loss	Grade Line (Head) (ft)	Grade Line (Head)(ft)	psi
Scenario 2a Current Non-irrigation: T-screens to wet well																		121.44	121.44	
	a	T-screen losses	109.44	0+00.0	-	0	0.0000	28.09	12605	60	60	19.63	1.43	0.03	-	0.50	0.00	120.91	120.94	2.80
	b	combining flow	109.44	0+00.0	-	0	0.0000	56.17	25209	60	60	19.63	2.86	0.13	0.45	0.06	0.00	120.76	120.88	2.74
	c	pipe length	109.44	0+00.0	0+10.0	10	0.0000	56.17	25209	60	60	19.63	2.86	0.13	0.013	0.00	0.00	120.75	120.88	2.73
	d	gate valve full open	109.44	0+10.0	-	0	0.0000	56.17	25209	60	60	19.63	2.86	0.13	0.19	0.02	0.00	120.73	120.85	2.72
	e	exit loss	109.44	0+10.0	-	0	0.0000	56.17	25209	60	60	19.63	2.86	0.13	1	0.13	0.00	120.60	120.73	2.67
	f	pipe length	108.05	0+10.0	0+50.0	40	0.0223	56.17	25209	9.335	9.335	79	0.71	0.01	0.013	0.00	0.01	120.71	120.71	5.15
	g	combining flow	108.12	0+33.0	-	0	0.0223	220.00	98736	15	15	107	2.06	0.07	0.68	0.04	0.00	120.60	120.67	4.87
	h	pipe length	106.85	0+50.0	1+02.3	52.25	0.0223	220.00	98736	19.835	19.835	260.78	0.84	0.01	0.013	0.00	0.01	120.65	120.66	5.26
	i	exit loss	106.27	1+02.3	-	0	0.0000	220.00	98736	9.335	9.335	233.38	0.94	0.01	1	0.01	0.00	120.63	120.65	5.89
	j	D-Box HGL & EGL	100.50	1+02.3	-	0	0.0000	220.00	98736	0	0	0.00	0.00	0.00	0	0.00	0.00	120.63	120.65	5.89

Top of Pipe @ End of Section: 100.50 ft
Total Section Loss: 0.79 ft
Max Internal Pressure: 5.89 psi
Static Internal Pressure: 9.07 psi

Invert in structure 112.6 ft
Invert in channel 112 ft

See OCRheadloss tab.

Top of Pipe @ End of Section: 116.50 ft
HGL in OCR channel: 115.90 ft
Channel IE @ End of Section: 113.00 ft
Total Section Loss: 4.73 ft
Max Internal Pressure: 1.79 psi
Static Internal Pressure: 3.74 psi

High flow case A: T-screens to wet well

										Steel Pipe								Old Pipe			
																		Hydraulic	Energy		
	Element	Component	Invert EL (US; ft)	Upstream Pipe STA	Downstream Pipe STA	Pipe Length	Slope	Flow (cfs)	Flow (gpm)	Nominal Size (in)	Inside Dia. (in)	Area (SF)	Vel (ft/s)	V^2 2g	Loss Coef C or k	Minor Loss (ft)	Pipe Loss	Grade Line (Head) (ft)	Grade Line (Head)(ft)	psi
D-Box to Existing 54" Conveyance Pipe																		120.63	120.65	
	ba	ent loss	101.00	100+04.7	-	0	0.0000	23.33	10472	54	54	15.90	1.47	0.03	0.5	0.02	0.00	120.60	120.63	6.54
	bb	chem injection port loss	101.00	100+04.7	-	0	0.0000	23.33	10472	54	54	15.90	1.47	0.03	0.2	0.01	0.00	120.59	120.62	6.54
	bc	pipe length	101.00	100+04.7	100+22.2	17.5	0.0000	23.33	10472	54	54	15.90	1.47	0.03	0.013	0.00	0.00	120.59	120.62	6.54
	bd	45 deg bend (horz)	101.00	100+22.2	-	0	0.0000	23.33	10472	54	54	15.90	1.47	0.03	0.18	0.01	0.00	120.58	120.61	6.53
	be	pipe length	101.00	100+22.2	100+38.4	16.2	0.0000	23.33	10472	54	54	15.90	1.47	0.03	0.013	0.00	0.00	120.58	120.61	6.53
	bf	30 deg bend (vert)	101.00	100+30.0	-	0	0.0000	23.33	10472	54	54	15.90	1.47	0.03	0.1	0.00	0.00	120.57	120.61	6.53

Bellota Weir Modifications
System Hydraulic Profile
Scenario 5

Color Key	Manual Entry
	Formula

Assumptions:
- Do not use Hazen-Williams for surface water pipe because Manning's equation provides more conservative headloss for long lengths of pipe.
- 6" air vacuum/air release valves on 36" pipe create negligible headloss.

bg	45 deg bend (horz)	101.00	100+38.4	-	0	0.0000	23.33	10472	54	54	15.90	1.47	0.03	0.18	0.01	0.00	120.57	120.60	6.53
bh	pipe length	101.00	100+38.4	102+86.4	248	0.0225	23.33	10472	54	54	15.90	1.47	0.03	0.013	0.00	0.03	120.53	120.57	6.51
bi	30 deg bend (vert)	101.00	102+60.7	-	0	0.0000	23.33	10472	54	54	15.90	1.47	0.03	0.1	0.00	0.00	120.53	120.56	6.51
bj	45 deg bend (horz)	106.00	102+86.4	-	0	0.0000	23.33	10472	54	54	15.90	1.47	0.03	0.18	0.01	0.00	120.52	120.56	4.34
bk	pipe length	106.00	102+86.4	103+80.9	94.5	0.0000	23.33	10472	54	54	15.90	1.47	0.03	0.013	0.00	0.01	120.51	120.54	4.34
bl	flow meter	106.00	103+80.9	-	0	0.0000	23.33	10472	54	54	15.90	1.47	0.03	0.2	0.01	0.00	120.50	120.54	4.34
bm	exit loss	106.32	103+80.9	-	0	#REF!	23.33	10472	54	54	15.90	1.47	0.03	1	0.03	0.00	120.47	120.50	4.18
bn	existing conveyance pipe	106.32	103+80.9	-	0	0.0000	23.33	10472	54	54	15.90	1.47	0.03	0	0.00	0.00	120.47	120.50	4.18

Top of Pipe @ End of Section:	110.82	ft
Total Section Loss:	0.14	ft
Max Internal Pressure:	6.54	psi
Static Internal Pressure:	6.20	psi
HGL at MH:	120.47	ft

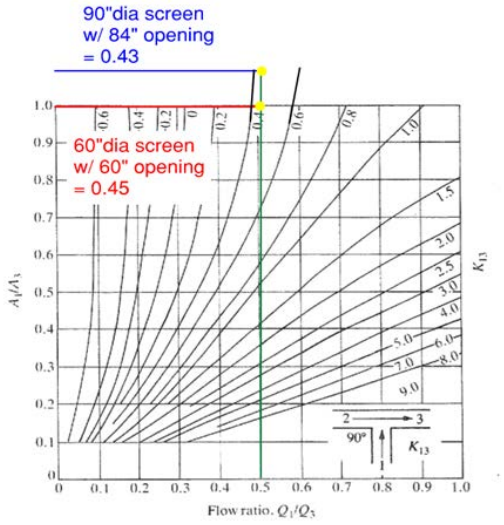


Fig. 13.10†. Combining flow: branch angle 90°, loss coefficient K_{13}

element b
Combining flow within tee screen
0.45

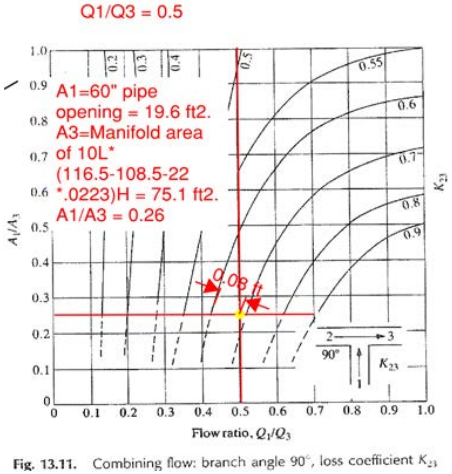


Fig. 13.11. Combining flow: branch angle 90°, loss coefficient K_{23}

element g
combining flow; Scenario 1-3a, 1-3b: Screen 2 flow combining with flow in manifold
0.68

Pipe n (steel)	0.013	CERM A-36
Pipe n (PVC/HDPE)	0.009	CERM A-36
Pipe n (concrete)	0.013	CERM A-36
Check valve	2.3	CERM 17-12
30-degree bend	0.1	Larry Pauls reference (see addl tab)
45-degree bend	0.18	Larry Pauls reference (see addl tab)
90-degree bend	0.72	Larry Pauls reference (see addl tab)
45-degree elbow	0.19	Larry Pauls reference (see addl tab)
entrance	0.5	CERM 17-13

Bellota Weir Modifications
System Hydraulic Profile
Scenario 5

Color Key	Manual Entry
	Formula

Assumptions:
- Do not use Hazen-Williams for surface water pipe because Manning's equation provides more conservative headloss for long lengths of pipe.
- 6" air vacuum/air release valves on 36" pipe create negligible headloss.

exit	1	CERM 17-13
Gate valve	0.19	CERM 17-12
flow meter	0.2	Matt assumed this but no reference?
combining flow; within 60" tee screen	0.45	Figure below, DS Miller Internal Flow Systems
combining flow; Scenario 1-3a, 1-4b: Screen 2 flow combining with flow in manifold	0.68	Figure below, DS Miller Internal Flow Systems
combining flow; Scenario 4a: Screen 3 flow combining with flow in manifold.	0.86	
combining flow; Scenario 4b: Screen 3 flow combining with flow in manifold	0.65	
combining flow; Scenario 4b: Screen 4 flow combining with flow in manifold	0.55	
through screen	0.5	Matt assumed this but no reference?
loss for 60" screen, max (through screen; suction pipe; and inlet)*	1	Per conversation with ISI, 12/9/21
loss for 60" screen, min (through screen; suction pipe; and inlet)*	0.5	Per conversation with ISI, 12/9/21
loss for 90" screen, max (through screen; suction pipe; and inlet)*	1.17	Per conversation with ISI, 12/9/21
loss for 90" screen, min (through screen; suction pipe; and inlet)*	0.67	Per conversation with ISI, 12/9/21
*Screen sizes were updated, values left as is to be conservative		

Friction Losses in Pipe Fittings
Resistance Coefficient K (use in formula $h_f = Kv^2/2g$)

Fitting		LD	Nominal Pipe Size											
			½	¾	1	1¼	1½	2	2½-3	4	6	8-10	12-16	18-24
			K Value											
Angle Valve		55	1.48	1.38	1.27	1.21	1.16	1.05	0.99	0.94	0.83	0.77	0.72	0.66
Angle Valve		150	4.05	3.75	3.45	3.30	3.15	2.85	2.70	2.55	2.25	2.10	1.95	1.80
Ball Valve		3	0.08	0.08	0.07	0.07	0.06	0.06	0.05	0.05	0.05	0.04	0.04	0.04
Butterfly Valve								0.86	0.81	0.77	0.68	0.63	0.35	0.30
Gate Valve		8	0.22	0.20	0.18	0.18	0.15	0.15	0.14	0.14	0.12	0.11	0.10	0.10
Globe Valve		340	9.2	8.5	7.8	7.5	7.1	6.5	6.1	5.8	5.1	4.8	4.4	4.1
Plug Valve Branch Flow		90	2.43	2.25	2.07	1.98	1.89	1.71	1.62	1.53	1.35	1.26	1.17	1.08
Plug Valve Straightaway		18	0.48	0.45	0.41	0.40	0.38	0.34	0.32	0.31	0.27	0.25	0.23	0.22
Plug Valve 3-Way Thru-Flow		30	0.81	0.75	0.69	0.66	0.63	0.57	0.54	0.51	0.45	0.42	0.39	0.36
Standard Elbow	90°	30	0.81	0.75	0.69	0.66	0.63	0.57	0.54	0.51	0.45	0.42	0.39	0.36
	45°	16	0.43	0.40	0.37	0.35	0.34	0.30	0.29	0.27	0.24	0.22	0.21	0.19
	long radius 90°	16	0.43	0.40	0.37	0.35	0.34	0.30	0.29	0.27	0.24	0.22	0.21	0.19
Close Return Bend		50	1.35	1.25	1.15	1.10	1.05	0.95	0.90	0.85	0.75	0.70	0.65	0.60
Standard Tee	Thru-Flow	20	0.54	0.50	0.46	0.44	0.42	0.38	0.36	0.34	0.30	0.28	0.26	0.24
	Thru-	60	1.62	1.50	1.38	1.32	1.26	1.14	1.08	1.02	0.90	0.84	0.78	0.72
90 Bends, Pipe Bends, Flanged Elbows, Butt-Welded Elbows	r/d=1	20	0.54	0.50	0.46	0.44	0.42	0.38	0.36	0.34	0.30	0.28	0.26	0.24
	r/d=2	12	0.32	0.30	0.28	0.26	0.25	0.23	0.22	0.20	0.18	0.17	0.16	0.14
	r/d=3	12	0.32	0.30	0.28	0.26	0.25	0.23	0.22	0.20	0.18	0.17	0.16	0.14
	r/d=4	14	0.38	0.35	0.32	0.31	0.29	0.27	0.25	0.24	0.21	0.20	0.18	0.17
	r/d=6	17	0.46	0.43	0.39	0.37	0.36	0.32	0.31	0.29	0.26	0.24	0.22	0.20
	r/d=8	24	0.65	0.60	0.55	0.53	0.50	0.46	0.43	0.41	0.36	0.34	0.31	0.29
	r/d=10	30	0.81	0.75	0.69	0.66	0.63	0.57	0.54	0.51	0.45	0.42	0.39	0.36
	r/d=12	34	0.92	0.85	0.78	0.75	0.71	0.65	0.61	0.58	0.51	0.48	0.44	0.41
	r/d=14	38	1.03	0.95	0.87	0.84	0.80	0.72	0.68	0.65	0.57	0.53	0.49	0.46
	r/d=16	42	1.13	1.05	0.97	0.92	0.88	0.80	0.76	0.71	0.63	0.59	0.55	0.50
	r/d=18	45	1.24	1.15	1.06	1.01	0.97	0.87	0.83	0.78	0.69	0.64	0.60	0.55
Mitre Bends	a=0°	2	0.05	0.05	0.05	0.04	0.04	0.04	0.04	0.03	0.03	0.03	0.03	0.02
	a=15°	4	0.11	0.10	0.09	0.09	0.08	0.08	0.07	0.07	0.06	0.06	0.05	0.05
	a=30°	8	0.22	0.20	0.18	0.18	0.17	0.15	0.14	0.14	0.12	0.11	0.10	0.10
	a=45°	15	0.41	0.38	0.35	0.33	0.32	0.29	0.27	0.26	0.23	0.21	0.20	0.18
	a=60°	25	0.68	0.63	0.58	0.55	0.53	0.48	0.45	0.43	0.38	0.35	0.33	0.30
	a=75°	40	1.09	1.00	0.92	0.88	0.84	0.76	0.72	0.68	0.60	0.56	0.52	0.48
	a=90°	60	1.62	1.50	1.38	1.32	1.26	1.14	1.08	1.02	0.90	0.84	0.78	0.72

Note: Fittings are standard with full openings.

H&H CALCULATIONS

FISH LADDER VERTICAL SLOT

FOR THE

BELLOTA WEIR MODIFICATIONS PROJECT

PREPARED FOR

STOCKTON EAST WATER DISTRICT

CALCULATIONS PREPARED BY

HDR ENGINEERING, INC.

RACHEL AINSLIE, E.I.T.

DATE PREPARED: 7/28/21

QC REVIEWER: SHAUN BEVAN, P.E.

QC REVIEW DATE: 9/28/21

DATE SUBMITTED: 10/06/21

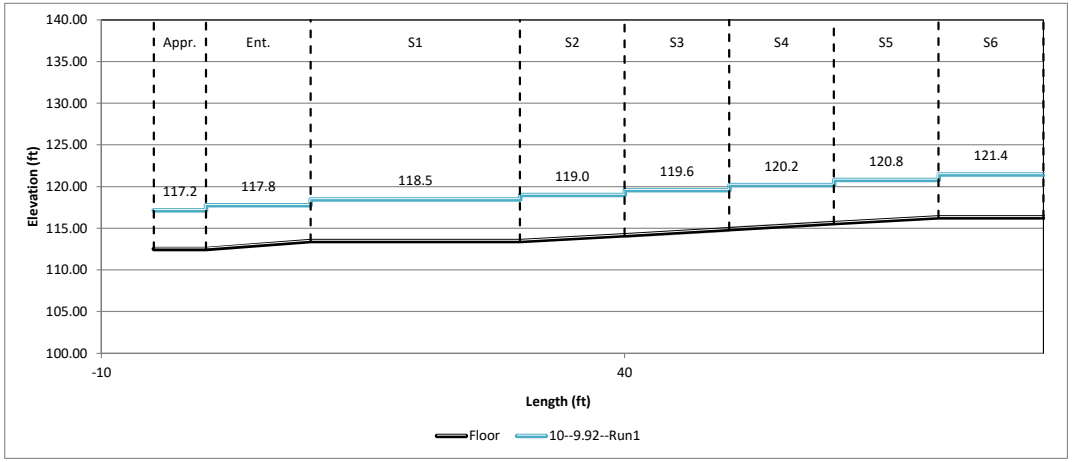
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Exp. MM-DD-YYYY

SUMMARY OF INITIAL HYDRAULIC CONDITIONS

ID #	Stream Discharge (cfs)	Headwater WSE (ft)	Headwater Depth (ft)	Tailwater WSE (ft)	Tailwater Depth (ft)
1	10.00	119.60	3.30	117.40	4.87
2	20.00	121.44	5.14	117.16	4.62
3	35.00	121.44	5.14	117.53	5.00
4	60.00	121.44	5.14	117.58	5.05
5	70.00	121.44	5.14	117.60	5.07
6	85.00	121.44	5.14	117.62	5.09
7	100.00	121.44	5.14	117.64	5.11
8	150.90	121.44	5.14	118.12	5.59
9	202.30	121.44	5.14	118.17	5.64
10					
11					
12					
13					
14					
15					

Notes: 1. WSE = Water Surface Elevation
2. Data displayed in United States Customary Units.



ANALYSIS TABLE

Stream Discharge = 20 cfs | Fishway Flow (avg.) = 20 cfs | Aux. Flow = None | Baffle Wall Thickness = 12 in.

Description	Fishway Q (cfs)	Auxiliary Flow (cfs)	WSE Downstream of Slot (ft)	Floor Elevation at Slot (ft)	Slope of Floor Upstream of Slot (%)	Pool Length Upstream of Slot (ft)	Sill Height at Slot (ft)	Pool Depth Downstream of Slot D (ft)	Depth d/s over sill d (ft)	Head on Slot h (ft)	Slot Width w (ft)	Orifice Coefficient C_d	Pool ID	Pool Width (ft)	EDF (ft-lbs/cfs)	Adjusted Solver Fishway Q w/ Aux. (cfs)
Entrance	20.02	0.00	117.16	112.53	9.260%	10.00	1.00	4.62	3.62	0.62	1.00	0.75		8.00	2.012	20.0
Slot 1	20.02	0.00	117.77	113.46	0.000%	20.00	1.00	4.31	3.31	0.69	1.00	0.75		8.00	1.077	20.0
Slot 2	20.02	0.00	118.46	113.46	6.460%	10.00	1.00	5.00	4.00	0.54	1.00	0.75		8.00	1.606	20.0
Slot 3	20.02	0.00	119.00	114.17	6.460%	10.00	1.00	4.83	3.83	0.57	1.00	0.75		8.00	1.756	20.0
Slot 4	20.02	0.00	119.57	114.88	6.460%	10.00	1.00	4.69	3.69	0.60	1.00	0.75		8.00	1.888	20.0
Slot 5	20.02	0.00	120.17	115.59	6.460%	10.00	1.00	4.58	3.58	0.63	1.00	0.75		8.00	2.000	20.0
Slot 6	20.02	0.00	120.79	116.30	0.000%	10.00	1.00	4.49	3.49	0.65	1.00	0.75		8.00	1.960	20.0
Headwater			121.44	116.30												
Minimum									3.3	0.5					1.1	
Maximum									4.0	0.7					2.0	
Mean									3.6	0.6					1.8	
Headwater			121.44		Initial Condition WSE Diff--> 4.28				Sum of h (Calculated)--> 4.28				WSE Off by --> 0.00			

Notes:

FISH PASSAGE CALCULATIONS

ENERGY DISSIPATION FACTOR IN POOLS

FOR THE

BELLOTA WEIR MODIFICATIONS PROJECT

PREPARED FOR

STOCKTON EAST WATER DISTRICT

CALCULATIONS PREPARED BY

HDR ENGINEERING, INC.

RACHEL AINSLIE, E.I.T.

DATE PREPARED: 01/05/22

QC REVIEWER: MATT PROCIV, P.E.

QC REVIEW DATE: 01/12/22

DATE SUBMITTED: 2/04/22

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Exp. MM-DD-YYYY

Project:	**Bellota Weir Widening**	Prepared: RA		Date: 2021.12.17
Subject:	**Fish Passage Calculations**	Checked: MP		Date:
Task:	**Pool EDF Calculations**			
Job#:			No:	

Objective: Calculate the Energy Dissipation Factor in each ladder pool (both in the pool and weir and in vertical slot portions) and determine that it is less than 4 to meet criteria (NMFS 2011).

4.5.3.5 Pool Volume: The *fishway* pools must be a minimum water volume of:

$$V = \frac{(\gamma)(Q)(H)}{(4\ ft - lbs/s)/\ ft^3}$$

where: V = pool volume, in ft^3
γ = unit weight of water, 62.4 pounds (lb) per ft^3
Q = *fish ladder* flow, in ft^3/s
H = energy head of pool-to-pool flow, in feet

This pool volume must be provided under every expected design flow condition, with the entire pool volume having active flow and contributing to energy dissipation.

rearranging gives

$$\frac{\gamma Q H}{v} \leq 4$$

NMFS, 2011. Anadromous Salmonid Passage Facility Design. P. 43

Q in ladder	20	cfs
Unit weight of water	62.4	lb/ft^3
Q in river, low flow	20	cfs

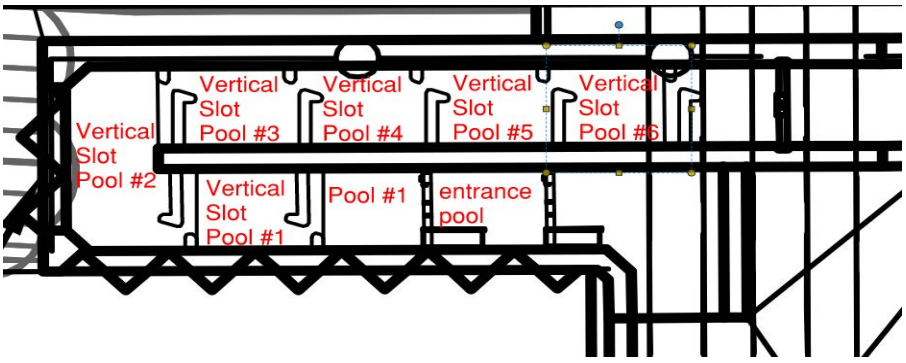
Low flow to allow passage in roughened channel

From d/s to u/s	Length (ft)	Width (ft)	Depth (ft)*	H (hydraulic drop) (ft)	EDF (ft-lbs/cfs)	CHECK
Entrance Pool	10	8	4.17	0.49	1.83	OK
Pool #1	10	8	4.72	0.60	1.98	OK

Q in river, high flow	202.3	cfs	5% exceedance flow per Pool and Weir spreadsheet

From d/s to u/s	Length (ft)	Width (ft)	Depth (ft)*	H (hydraulic drop) (ft)	EDF (ft-lbs/cfs)	
Entrance Pool Weir	10	8	5.5	0.18	0.51	OK
Pool #1 Weir	10	8	5.73	0.37	1.02	OK

*Depth is taken through the worst case scenario point, at slot face where floor is at higher elevation
Depth and hydraulic drop values are from Vertical Slot and Pool and Weir spreadsheets



H&H CALCULATIONS

FISH LADDER POOL AND WEIR

FOR THE

BELLOTA WEIR MODIFICATIONS PROJECT

PREPARED FOR

STOCKTON EAST WATER DISTRICT

CALCULATIONS PREPARED BY

HDR ENGINEERING, INC.

RACHEL AINSLIE, E.I.T.

DATE PREPARED: 7/28/21

QC REVIEWER: SHAUN BEVAN, P.E.

QC REVIEW DATE: 9/28/21

DATE SUBMITTED: 10/06/21

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Exp. MM-DD-YYYY

Project:	**Bellota Weir Widening**	Prepared:	RA	Date:	2021.07.28
Subject:	**Fish Passage Calculations**	Checked:	SPB	Date:	2021.09.21
Task:	**Cipoletti Weir Sizing**				
Job#:				No:	

DESCRIPTION:

The purpose of this calculation is to
1) Calculate the size of the Cipoletti notch for the Entrance Weir and Pool #1 Weir

REFERENCES:

Provide any applicable references
Civil Engineering Review Manual, Lindeburg, pages 19-11 to 19-13

INPUTS:

Include any assumptions used in the calculation

Table 1 is copied from the MDF and Exceedance calculation.
Table 1:

		Low Design Flow (cfs)	**High Design Flow (Cfs)**
Scenario	**Time period**	**NMFS Adults (95% Exceedance)**	**NMFS Adults (5% Exceedance)**
Annual	All year	**3***	545.3
Irrigation Season	Mid-April to Mid Oct	7.2	**150.9**
Fall Chinook	Nov 1 to Dec 31	3*	420.6
Steelhead	Nov 1 to March 31	3*	1,728.6
Late fall Chinook	Nov 1 to Feb 28	3*	**1,735.2**
Spring Chinook	March 1 to May 31	3*	908.1
Winter Chinook	Jan 1 to May 31	3*	1,710

95% and 5% Exceedance flow guidelines per NMFS Anadromous Salmonid Passage Facility Design, p. 24-25.
**CDFG guidelines recommend that 3 cfs be selected for an alternative minimum design flow if the low flow is calculated to be less than 3 cfs.*
Table 4, Culvert Criteria for Fish Passage, CDFG 2002 and Table IX-5, California Salmonid Habitat Restoration Manual, Part IX: Fish Passage Evaluation at Stream Crossings, CDFG 2004

Based on the results, fish passage must be provided from a minimum of 3 cfs to a maximum of 1,735 cfs year round through the use of both the fish ladder and roughened channel.

The fish ladder will only be utilized during irrigation season, from mid-April to mid-October, so it need not pass fish up to 1,735 cfs. The 5% exceedance is 150.9 cfs during irrigation season, and the 2% exceedance is 202.3 cfs during irrigation season. The fish ladder will be designed to provide passage up to the 2% irrigation season exceedance of 202.3 cfs river flow, while the roughened channel will provide passage up to to 1,735.2 cfs river flow. In order for the roughened channel to be fish passable based on flow depths, 20 cfs is the minimum flow. As a result, the fish ladder will be designed to provide passage at a low flow of 20 cfs. The Cipoletti notch will be remain, designed to 3 cfs, so attraction flow is more focused and so that if operational criteria is ever adjusted in the future the ladder has the flexibility to meet this criteria.
The fish ladder design flow is 20 cfs.

Input	
Result	
Constant	

If changes are to be made to inputs, make them on this tab and they will be reflected throughout the spreadsheet.

Table 2:

Entrance Pool Weir Cipoletti Notch Width	1.0	ft	$b_{cipoletti}$
Entrance Pool Sharp Crested Weir Width	8.0	ft	$b_{sharp-crest}$
Pool #1 Weir Cipoletti Notch Width	1.0	ft	$b_{cipoletti}$
Pool #1 Weir Pool Sharp Crested Weir Width	8.0	ft	$b_{sharp-crest}$
Low Design Flow for Fish Ladder	3.0	cfs	Q_{ladder}
High Design Flow for Fish Ladder	20.0	cfs	Q_{ladder}
Design Flow for Cipoletti Notch	3.0	cfs	
High Design Flow for Rectangular Sharp Crested Weir	17.0	cfs	
High River Flow for Fish Ladder Passage	202.3	cfs	
Coefficient, Sharp Crested Weir	0.62		C_1
Coefficient, Cipoletti Weir	0.63		C_d
Gravitational constant	32.2	ft/s^2	g
Entrance Weir and Pool #1 Weir Cipoletti Notch Height	0.93	ft	H_{LDF}

$$Q = \frac{2}{3} * C_d * b * \sqrt{2g} * H^{3/2}$$

$$Q_{sub} = Q * (1 - \left(\frac{H_{downstream}}{H_{upstream}}\right)^{\frac{3}{2}})^{0.385}$$

METHODS
Cipoletti Weir Equation, Lindeburg 19-11

CALCULATIONS:
The following calculations are based on the assumptions and references provided above.

Evaluate H_{LDF} over the Entrance Weir Cipoletti Notch at Low Design Flow:

Entrance Pool Notch Weir Width ($b_{cipoletti}$) = 1.00 ft

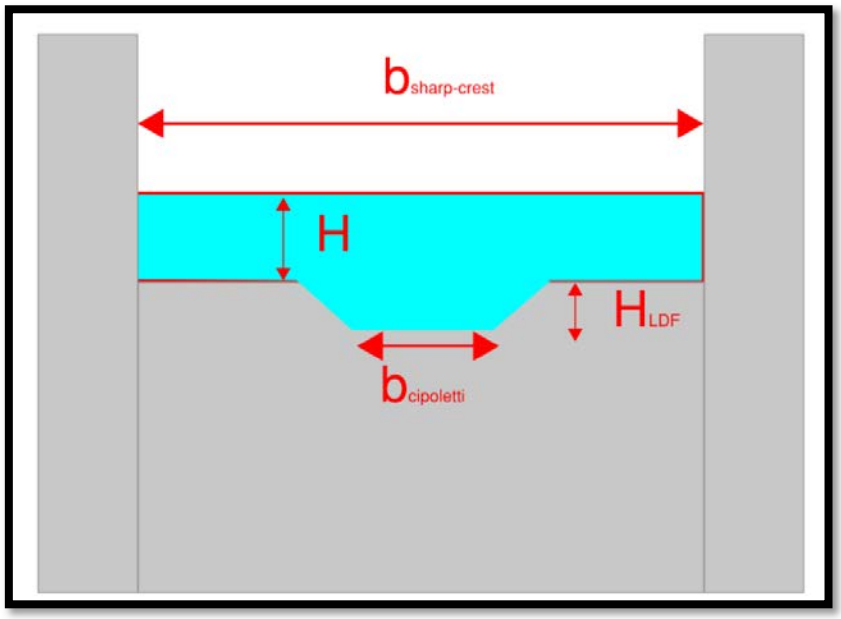
Q_{ladder} = 3.0 cfs

$$H_{LDF} = \left(\frac{Q}{\frac{2}{3} * C_d * b * \sqrt{2g}}\right)^{2/3}$$

Cipoletti Weir Equation

H_{LDF}= 0.93 ft

Size of notch over both Entrance Weir and Pool #1 Weir should be 0.93', or 11" to accommodate low design flow of 3 cfs.

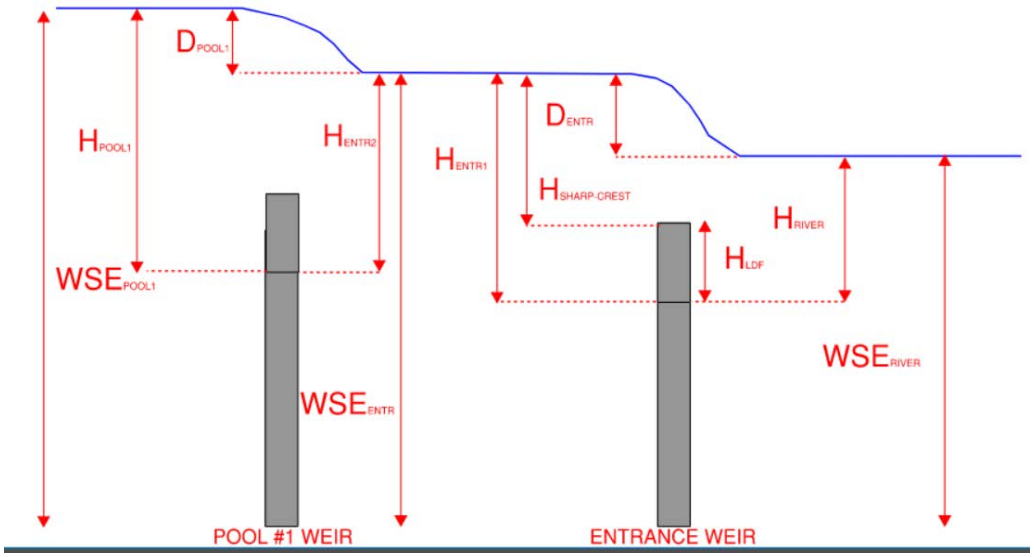


Period 1 Duration Analysis

Time Period: 15Apr - 15Oct

Number Valid Values:	4329
Number Missing Values:	0
Minimum Value:	0.0
Maximum Value:	2,128.8

Percent of Time Exceeded	Flow CFS
99.0	0.0
95.0	7.2
90.0	17.7
80.0	27.1
50.0	40.7
25.0	58.6
15.0	92.3
10.0	118.4
5.0	150.9
2.0	202.3
1.0	484.7
0.1	1,356.6



Project:	**Bellota Weir Widening**	Prepared:	RA	Date:	2021.07.28
Subject:	**Fish Passage Calculations**	Checked:	SPB	Date:	2021.09.28
Task:	**Pool and Weir Fish Ladder Elevations**				
Job#:		No:			

DESCRIPTION:

The purpose of this calculation is to
1) Calculate the hydraulic differential for the fish ladder Entrance Weir and determine the weir notch elevation
2) Calculate the hydraulic differential for the fish ladder Pool #1 Weir and determine the weir notch elevation
3) Calculate tailwater conditions for use in the vertical slot ladder calculations

REFERENCES:

Provide any applicable references

Civil Engineering Review Manual, Lindeburg, pages 19-11 to 19-13

INPUTS:

Include any assumptions used in the calculation

Input	
Result	
Constant	

Entrance Pool Weir Cipoletti Notch Width	1.00	ft	$b_{trapezoidal}$
Entrance Pool Sharp Crested Weir Width	8.00	ft	$b_{sharp-crest}$
Pool #1 Weir Cipoletti Notch Width	1.00	ft	$b_{cipoletti}$
Pool #1 Weir Sharp Crested Weir Width	8.00	ft	$b_{sharp-crest}$
Low Design Flow for Fish Ladder	3.00	cfs	Q_{ladder}
High Design Flow for Fish Ladder	20.00	cfs	Q_{ladder}
High River Design Flow for Fish Ladder Passage	202.30	cfs	
High Design Flow for Rectangular Sharp Crested Weir	17.00	cfs	
Coefficient, Sharp-Crested Weir	0.62		C_1
Coefficient, Cipoletti Weir	0.63		C_d
Gravitational constant	32.20	ft/s^2	g
Cipoletti Side Slope	4:1	V:H	
Entrance Weir and Pool #1 Weir Cipoletti Notch Height	0.93	ft	H_{LDF}

METHODS

Cipoletti Weir Equation, Lindeburg 19-13 $$Q = \frac{2}{3} * C_d * b * \sqrt{2g} * H^{3/2}$$

Sharp-Crested Weir Equation, Lindeburg 19-11 $$Q = \frac{2}{3} * C_1 * b * \sqrt{2g} * H^{3/2}$$

Submerged Weir Equation, Lindeburg 19-12 $$Q_{sub} = Q * (1 - \left(\frac{H_{downstream}}{H_{upstream}}\right)^{\frac{3}{2}})^{0.385}$$

WSE_{river} Equation, see Rating Curve tab $$WSE_{river} = 113.88 * Q^{0.0045}$$

Goal seek was used on columns D and Q in order to find the appropriate H value that would allow 3 cfs or 20 cfs of flow through the fish ladder.
The entrance weir is designed to have between 1 and 1.5 feet of head differential, and the Pool #1 Weir is designed to have under 1 foot of head differential.

6-inch stoplogs will be used for the entrance weir so that it can be easily adjusted to provide 1-1.5 ft of hydraulic differential at all flow events.

									Entrance Weir Notch	115	115.5	116
ENTRANCE WEIR CALCULATIONS												
Q_{river}	Q_{ladder}	Q_{ladder}	Submerged?	H_{entr}	H_{river}/H_{entr1}	H_{entr1}	H_{river}	WSE_{river}	Entrance Weir Notch El	WSE_{entr}	D_{entr} (Head Diff.)	
cfs	cfs	cfs		ft		ft	ft	ft	ft	ft	ft	
10	10	10.00	NO	1.34	-0.32	1.34	-0.43	115.07	115.00	116.34	1.27	
20	20	20.00	NO	1.67	-0.04	1.67	-0.07	115.43	115.00	116.67	1.24	
35	20.00001	20.00	YES	1.72	0.13	1.72	0.22	115.72	115.50	117.22	1.50	
60	20.00001	20.00	YES	1.79	0.28	1.79	0.50	116.00	115.50	117.29	1.29	
70	20.00001	20.00	YES	1.81	0.32	1.81	0.58	116.08	115.50	117.31	1.23	
85	20.00001	20.00	YES	1.84	0.37	1.84	0.68	116.18	115.50	117.34	1.16	
100	20.00002	20.00	YES	1.86	0.41	1.86	0.76	116.26	115.50	117.36	1.10	
150.9	20	20.00	YES	1.94	0.51	1.94	0.98	116.48	116.00	117.94	1.46	
202.3	20	20.00	YES	2.00	0.57	2.00	1.13	116.63	116.00	118.00	1.36	

Pool #1 Weir Notch	116

POOL #1 WEIR CALCULATIONS

Q_{ladder}	Q_{ladder}	Submerged?	H_{pool1}	H_{entr2}/H_{pool1}	H_{pool1}	H_{entr2}	WSE_{pool1}	D_{pool1} (Head Diff.)
cfs	cfs		ft		ft	ft	ft	ft
10	10.00	YES	1.40	0.24	1.40	0.34	117.40	1.06
20	20.00	YES	1.16	0.58	1.16	0.67	117.16	0.49
20	20.00	YES	1.53	0.80	1.53	1.22	117.53	0.31
20	20.00	YES	1.58	0.81	1.58	1.29	117.58	0.30
20	20.00	YES	1.60	0.82	1.60	1.31	117.60	0.29
20	20.00	YES	1.62	0.83	1.62	1.34	117.62	0.28
20	20.00	YES	1.64	0.83	1.64	1.36	117.64	0.28
20	20.00	YES	2.12	0.91	2.12	1.94	118.12	0.18
20	20.00	YES	2.17	0.92	2.17	2.00	118.17	0.18

Check on if using 115.5 would be acceptable for all flow events.

Entrance Weir Notch	115.5

ENTRANCE WEIR CALCULATIONS

Q_{river}	Q_{ladder}	Q_{ladder}	Submerged?	H_{entr}	H_{river}/H_{entr1}	H_{entr1}	H_{river}	WSE_{river}	Entrance Weir Notch El	WSE_{entr}	D_{entr} (Head Diff.)
cfs	cfs	cfs		ft		ft	ft	ft	ft	ft	ft
15	15	15.00	NO	1.51	-0.15	1.51	-0.22	115.28	115.50	117.01	1.74
20	20	20.00	NO	1.67	-0.04	1.67	-0.07	115.43	115.50	117.17	1.74
35	20.00001	20.00	YES	1.72	0.13	1.72	0.22	115.72	115.50	117.22	1.50
60	20.00001	20.00	YES	1.79	0.28	1.79	0.50	116.00	115.50	117.29	1.29
70	20.00001	20.00	YES	1.81	0.32	1.81	0.58	116.08	115.50	117.31	1.23
85	20.00001	20.00	YES	1.84	0.37	1.84	0.68	116.18	115.50	117.34	1.16
100	20.00002	20.00	YES	1.86	0.41	1.86	0.76	116.26	115.50	117.36	1.10
150.9	20	20.00	YES	1.94	0.51	1.94	0.98	116.48	115.50	117.44	0.96
202.3	20	20.00	YES	2.00	0.57	2.00	1.13	116.63	115.50	117.50	0.86

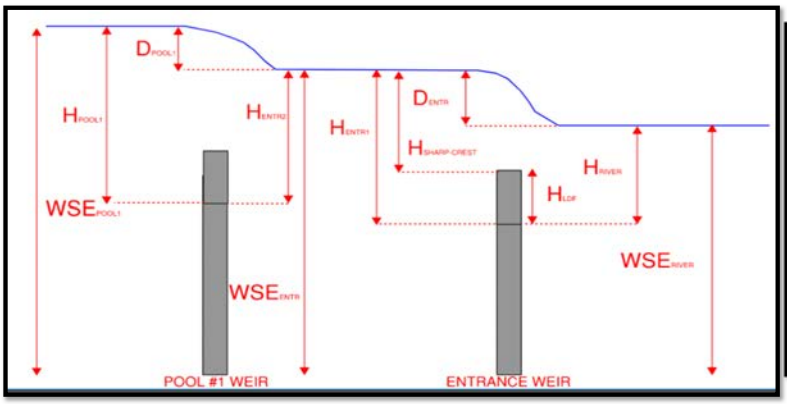
If weir remains at elevation 115.5, head differential will range from 1.7' to 0.9'.

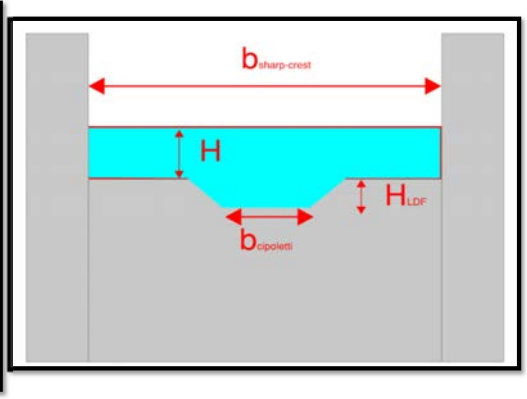
Pool #1 Weir Notch	116

POOL #1 WEIR CALCULATIONS

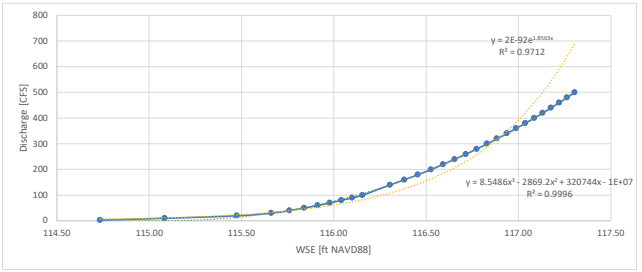
Q_{ladder}	Q_{ladder}	Submerged?	H_{pool1}	H_{entr2}/H_{pool1}	H_{pool1}	H_{entr2}	WSE_{pool1}	D_{pool1} (Head Diff.)
cfs	cfs		ft		ft	ft	ft	ft
15	15.00	YES	1.79	0.57	1.79	1.01	117.79	0.78
20	20.00	YES	1.49	0.78	1.49	1.17	117.49	0.32
20	20.00	YES	1.53	0.80	1.53	1.22	117.53	0.31
20	20.00	YES	1.58	0.81	1.58	1.29	117.58	0.30
20	20.00	YES	1.60	0.82	1.60	1.31	117.60	0.29
20	20.00	YES	1.62	0.83	1.62	1.34	117.62	0.28
20	20.00	YES	1.64	0.83	1.64	1.36	117.64	0.28
20	20.00	YES	1.70	0.85	1.70	1.44	117.70	0.26
20	20.00	YES	1.75	0.86	1.75	1.50	117.75	0.25

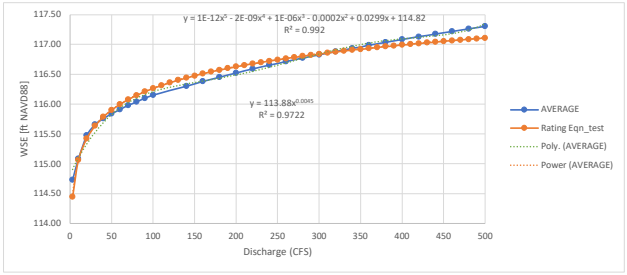
If weir remains at elevation 115.5, head differential will range from 0.3' to 0.8'.





Total Flow	Average_WSE1	Average_WSE2	Average_WSE3	Average_WSE4	Average_WSE5	Average_WSE6	AVERAGE	LINEAR SLOPE (CFS/ FT WSE)
3	114.73	114.73	114.73	114.73	114.73	114.73	114.73	
10	115.08	115.08	115.08	115.08	115.08	115.08	115.08	25.57
20	115.46	115.46	115.47	115.47	115.49	115.50	115.47	53.75
30	115.64	115.65	115.65	115.66	115.68	115.69	115.66	101.41
40	115.74	115.74	115.75	115.75	115.78	115.80	115.76	123.67
50	115.83	115.83	115.84	115.84	115.85	115.85	115.84	137.84
60	115.90	115.91	115.91	115.91	115.92	115.93	115.91	150.54
70	115.97	115.97	115.98	115.98	115.99	116.00	115.98	161.79
80	116.03	116.03	116.04	116.04	116.05	116.06	116.04	169.50
90	116.08	116.09	116.10	116.10	116.11	116.12	116.10	183.46
100	116.14	116.15	116.15	116.16	116.16	116.17	116.15	268.32
140	116.30	116.30	116.30	116.30	116.30	116.30	116.30	254.25
160	116.38	116.38	116.38	116.38	116.38	116.38	116.38	272.10
180	116.46	116.46	116.46	116.46	116.46	116.46	116.46	286.39
200	116.53	116.53	116.53	116.53	116.53	116.53	116.53	302.42
220	116.59	116.59	116.59	116.59	116.59	116.59	116.59	316.56
240	116.65	116.65	116.66	116.65	116.65	116.66	116.65	332.31
260	116.71	116.71	116.72	116.72	116.71	116.72	116.71	341.01
280	116.77	116.77	116.77	116.77	116.77	116.77	116.77	356.12
300	116.83	116.83	116.83	116.83	116.83	116.83	116.83	370.34
320	116.88	116.88	116.88	116.88	116.88	116.88	116.88	378.58
340	116.94	116.94	116.94	116.94	116.94	116.94	116.94	394.65
360	116.99	116.99	116.99	116.99	116.99	116.99	116.99	405.53
380	117.04	117.04	117.04	117.04	117.04	117.04	117.04	409.51
400	117.08	117.09	117.09	117.09	117.08	117.09	117.09	433.16
420	117.13	117.13	117.13	117.13	117.13	117.13	117.13	442.72
440	117.18	117.18	117.18	117.18	117.18	117.18	117.18	456.39
460	117.22	117.22	117.22	117.22	117.22	117.22	117.22	472.34
480	117.26	117.26	117.26	117.26	117.26	117.26	117.26	484.08
500	117.30	117.30	117.31	117.30	117.30	117.31	117.30	4.26





0.423295

0.322348

Calculated WSEL based on Rating Equation, for use in weir and vertical slot calculations:

River (cfs)	WSEL at fish entrance
3	114.4443888
10	115.0661172
20	115.4255874
30	115.6363844
40	115.7861807
50	115.9025053
60	115.9976362
70	116.0781291
80	116.1479004
90	116.2094778
100	116.2645884
110	116.3144645
120	116.3600165
130	116.401936
140	116.4407609
150	116.4769176
160	116.5107501
170	116.5425399
180	116.57252
190	116.6008858
200	116.6278027
210	116.6534118
220	116.6778346
230	116.7011763
240	116.7235289
250	116.7449729
260	116.7655794
270	116.7854115
280	116.8045255
290	116.8229717
300	116.8407952
310	116.8580368
320	116.8747334
330	116.8909185
340	116.9066225
350	116.9218732
360	116.9366962
370	116.9511149
380	116.9651507
390	116.9788235
400	116.9921517
410	117.0051522
420	117.0178408
430	117.0302321
440	117.0423399
450	117.0541767
460	117.0657545
470	117.0770845
480	117.0881769
490	117.0990417
500	117.1096879

FISH PASSAGE CALCULATIONS

OLD CALAVERAS RIVER HEAD LOSS

FOR THE

BELLOTA WEIR MODIFICATIONS PROJECT

PREPARED FOR

STOCKTON EAST WATER DISTRICT

CALCULATIONS PREPARED BY

HDR ENGINEERING, INC.

RACHEL AINSLIE, E.I.T.

DATE PREPARED: 12/10/21

QC REVIEWER: SHAUN BEVAN, P.E.

QC REVIEW DATE: 12/21/21

DATE SUBMITTED: 2/04/21

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Exp. MM-DD-YYYY

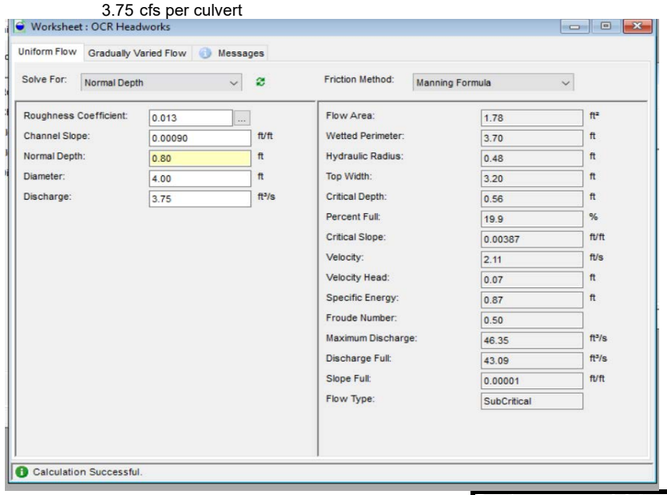
Project:	**Bellota**	Computed:	**RA**	Date:
Subject:	**Old Calaveras River**	Checked:	**SB**	Date:
Task:	**Pipe Headloss, Velocities**	Page:		of:
Job #:		No:		

Objective: 1) Determine the range of normal depth scenarios in the Old Calaveras River Channel for hydraulic profile calculations
2) Determine whether or the Old Calaveras River Pipelines have sufficient capacity for all scenarios
3) Find expected pipe velocities and depths for design of OCR Structure

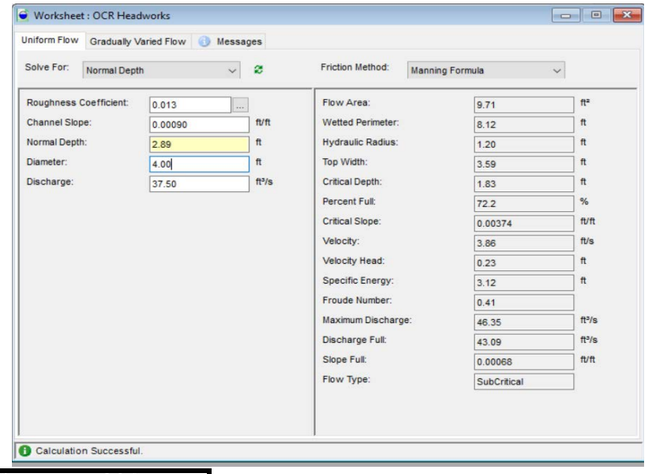
Objective (1)
Determine the range of normal depth scenarios in the Old Calaveras River Channel for hydraulic profile calculations

Old Calaveras Headworks
FlowMaster normal depth calculation

Low Flow Investigation
15 cfs
3.75 cfs per culvert

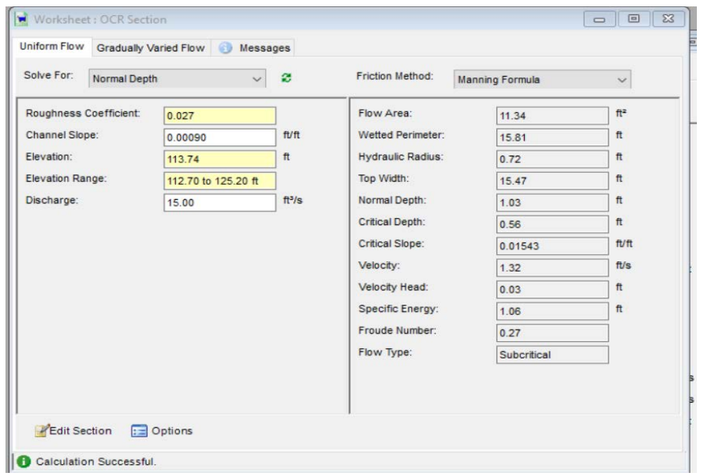


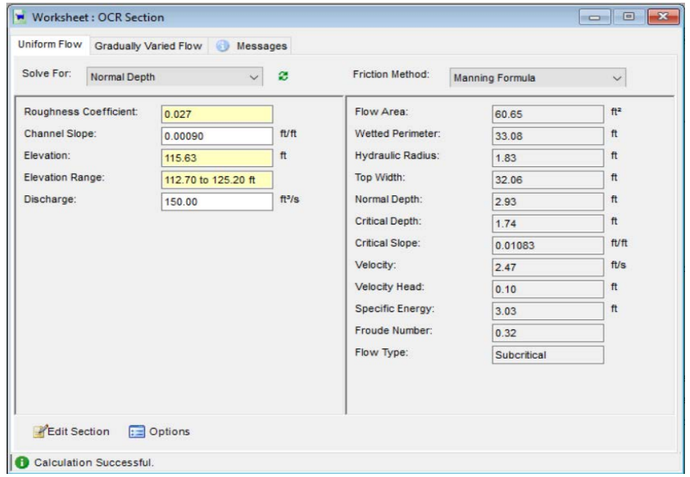
High Flow Investigation
150 cfs
37.5 cfs per culvert



Depth:	0.8 ft	Depth:	2.9 ft
Velocity:	2.11 fps	Velocity:	3.86 fps

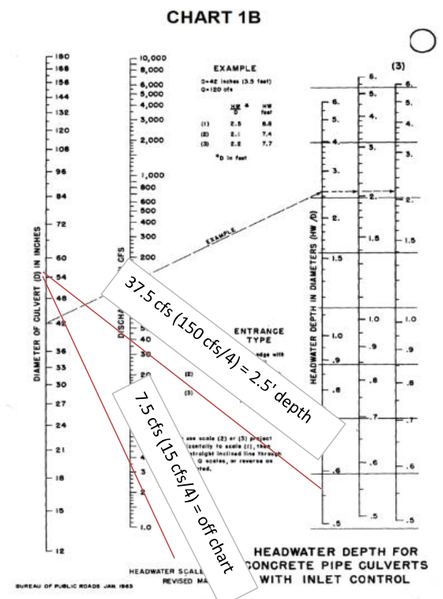
Old Calaveras Channel
FlowMaster normal depth calculation





Depth:	1 ft	Depth:	2.9 ft
Velocity:	1.32 fps	Velocity:	2.47 fps

Old Calaveras Headworks
Nomograph normal depth calculation



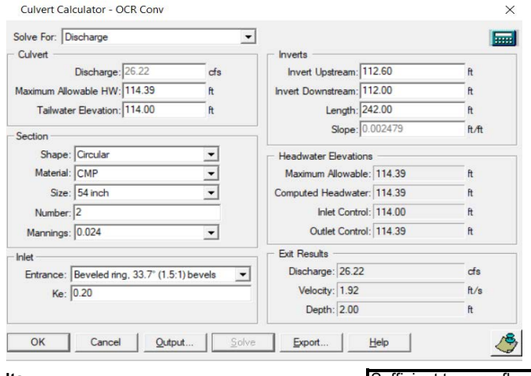
Result:	N/A ft	Result:	2.5 ft

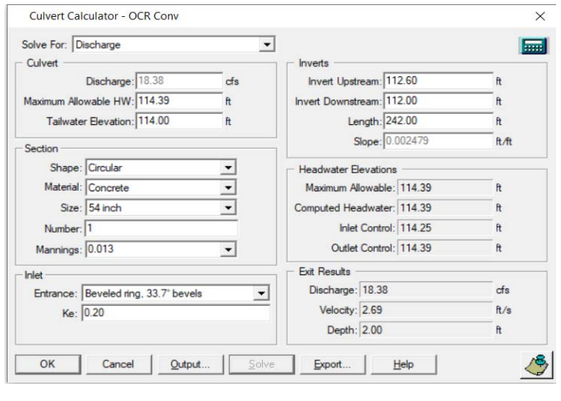
Objective (1): Final normal depth results

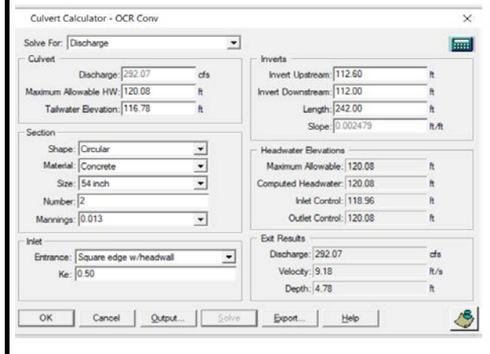
Max normal depth result:	1 ft	Max normal depth result:	2.9 ft
Min normal depth result:	0.8 ft	Min normal depth result:	2.5 ft
Max velocity result:	2.11 fps	Max velocity result:	3.86 fps
Min velocity result:	1.32 fps	Min velocity result:	2.47 fps

Objective (2)
Determine whether or the Old Calaveras River Pipelines have sufficient capacity for all scenarios
Exclude weir at Old Calaveras Outlet Structure
Check discharge capabilities

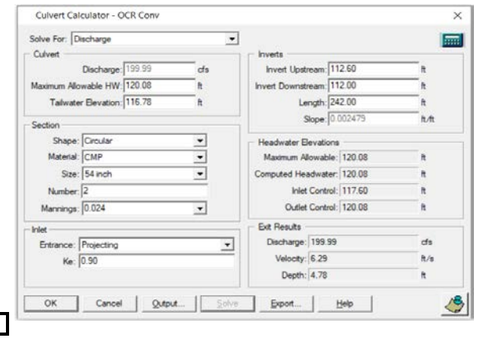
HW	114.39
TW max	114
TW min	113.8
Channel inv	113







HW	120.13
TW max	115.9
TW min	115.5
Channel in	113

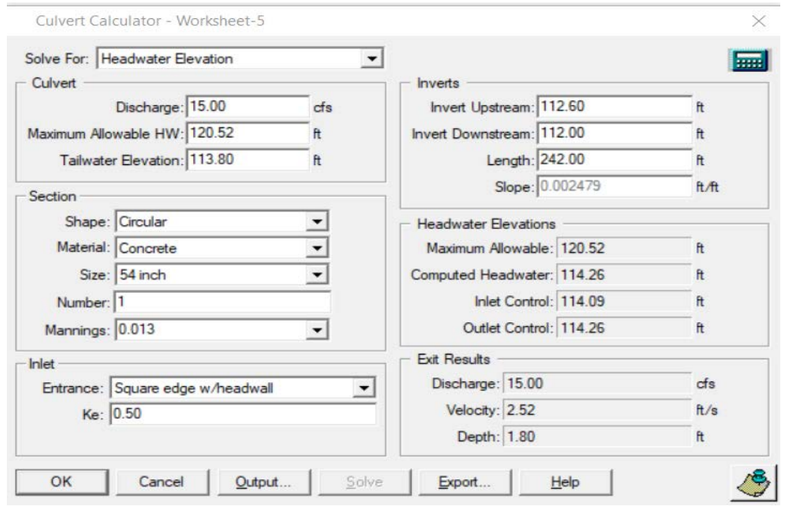


Objective (2): Final capacity results

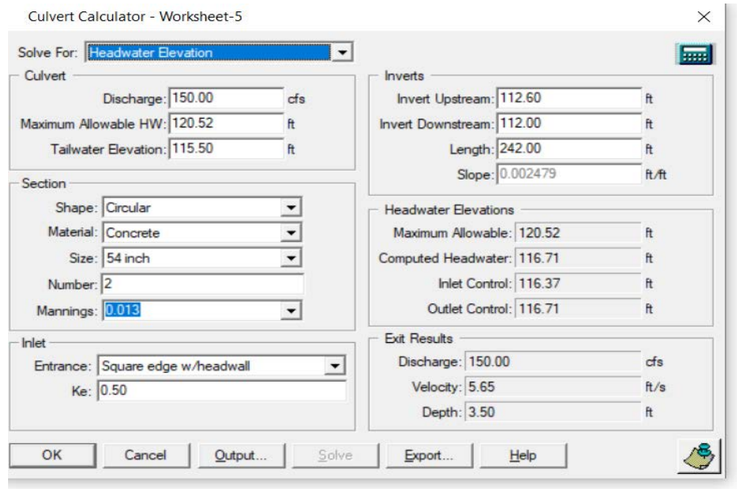
Sufficient to carry flows in one 54" pipe in concrete pipe, but insufficient for CMP. Can convey 15 cfs in CMP if two pipes are used.	Sufficient to carry flows in two 54" pipes with concrete or CMP.

Objective (3):
Find expected pipe velocities and depths for design of OCR Structure

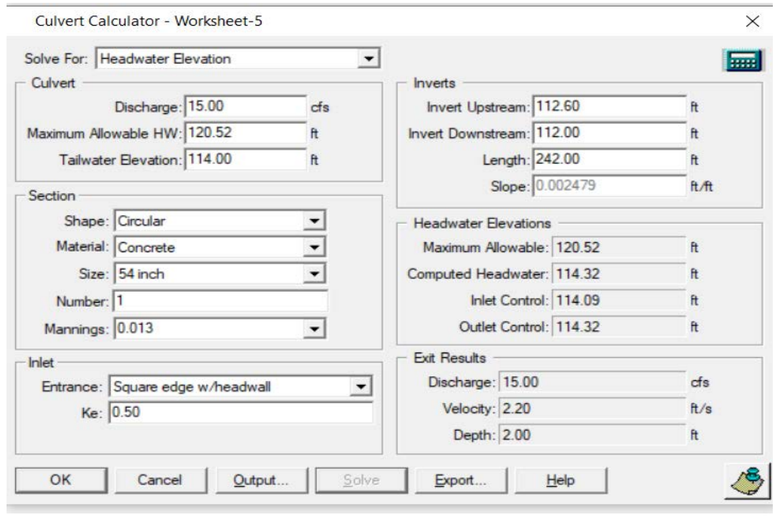
Using minimum tailwater elevation of 0.8 feet



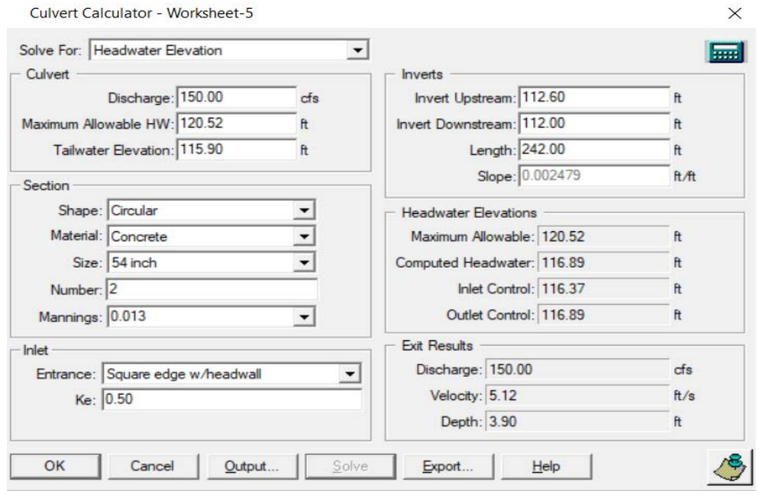
Using minimum tailwater elevation of 2.5 feet



Using maximum tailwater elevation of 1.0 feet



Using maximum tailwater elevation of 2.9 feet



Objective (3): Final pipe velocity and depth results

Max normal depth result:	2 ft	Max normal depth result:	3.9 ft
Min normal depth result:	1.8 ft	Min normal depth result:	3.5 ft
Max velocity result:	2.52 fps	Max velocity result:	5.65 fps
Min velocity result:	2.2 fps	Min velocity result:	5.1 fps

FISH PASSAGE CALCULATIONS

OLD CALAVERAS RIVER STRUCTURE

FOR THE

BELLOTA WEIR MODIFICATIONS PROJECT

PREPARED FOR

STOCKTON EAST WATER DISTRICT

CALCULATIONS PREPARED BY

HDR ENGINEERING, INC.

RACHEL AINSLIE, E.I.T.

DATE PREPARED: 01/05/22

QC REVIEWER: MATT PROCIV, P.E.

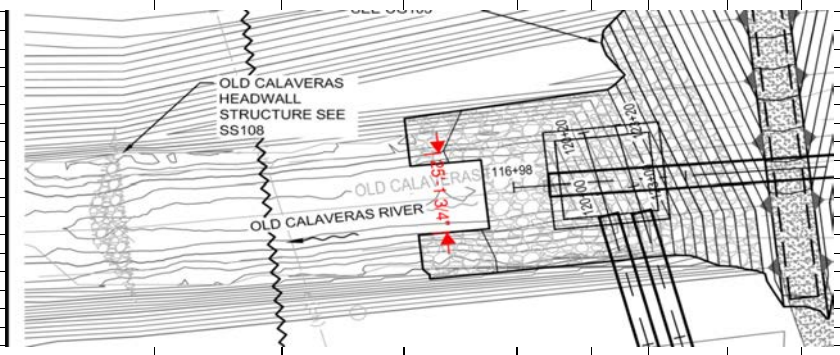
QC REVIEW DATE: 01/12/22

DATE SUBMITTED: 2/04/22

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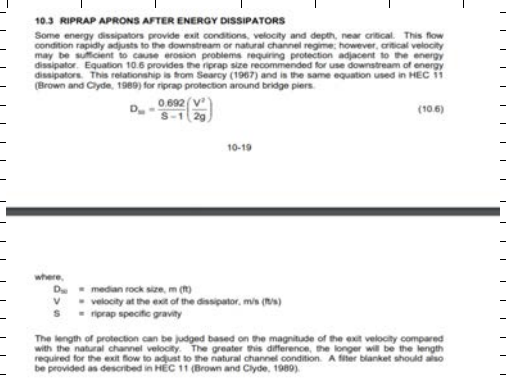
		Project:	**Bellota**			Computed:	**RA**	Date:							
		Subject:	**Old Calaveras River**			Checked:	**SB / MP**	Date:							
		Task:	**Structure Design**			Page:		of:							
		Job #:				No:									
Objective:	1) Determine whether or not an energy dissipation structure is required at the outlet of the OCR pipelines														
	2) Design a structure for the outlet of the OCR pipelines														
	3) Design energy dissipation apron for outlet of structure														
Objective (1)															
Determine whether or not an energy dissipation structure is required at the outlet of the OCR pipelines															
Reference:	USBR 1978. Design of Small Canal Structures.														
	https://www.usbr.gov/tsc/techreferences/mands/mands-pdfs/SmallCanals.pdf														
	Concrete transition (p. 337) cannot be used because the flow is turning 90 degrees into a 20-25 foot wide channel; there isn't sufficient room to transition into the canal.														
	Baffled outlet structure (p. 308) cannot be used because it is only used for Froude numbers ranging from 1 to 9.														
	Velocity at outlet:		5.65	fps											
	Depth at outlet:		3.5	ft		$F_r = \frac{v_0}{\sqrt{g * y_0}}$									
	Gravity:		32.2	ft/s^2											
	Fr:		0.53												
Objective (1) Result:															
Energy dissipation structure is not required at outlet; Froude number is under 1, which means energy excess is minimal and this application would be impractical (p. 309).															
Objective (2)															
Design a structure for the outlet of the OCR pipelines															
Determine structure width															
Structure is width of channel, approx 25 feet.															



Max flow		150	cfs	
Normal Depth in OCR (min)		2.5	ft	
Normal Depth in OCR (max)		2.9	ft	
Invert of Structure		111.5	ft	
Invert of Curb		113	ft	
Normal depth in structure (min)		4	ft	
Normal depth in structure (max)		4.4	ft	
Normal depth over curb (min)		2.5	ft	
Normal depth over curb (max)		2.9	ft	
Width of structure		25	ft	
Velocity calculations as check for energy dissipation				
Velocity at outlet of pipe under Scenario 4b:		5.65	fps	
Velocity in OCR channel:		2.47	fps	
Velocity outletting from structure:		2.4	fps	
Velocity outletting from half of structure:		4.8	fps	
Confirmation of Objective (1) results:				
The velocity outletting from the structure, 2.4 fps, is nearly equal to that in the channel, showing that there is no need for energy dissipation.				
Even if flow bounces against the far wall and only flows through half the structure to enter the channel, velocity is 4.8 fps, which is low enough to not require energy dissipation.				
Estimate length of bend scour to determine length of the structure.				
Reference:	NRCS. 2007. National Engineering Handbook Part 654. TS 14B. Scour Calculations.			
	https://directives.sc.egov.usda.gov/OpenNonWebContent.aspx?content=17811.wba			
$\frac{L_P}{R} = 0.0604\left(\frac{R^{\frac{1}{6}}}{n}\right)$				
R	ft	Hydraulic radius = flow area/wetted perimeter		
A	sq-ft	Flow area		
P	ft	Wetted perimeter		
n		Manning n value for the bend		
L_p	ft	Length of scour zone, measured downstream from the bend apex		
	Assuming full structure is used for flow (not half of structure)			
A	110	ft^2		
P	33.8	ft		
n	0.013		Chow 1959	
R	3.25	ft		
L_p	18.41	ft		
Based on calculations, there will be a length of 18.4 feet of bend scour.				

c. Concrete			
1. trowel finish	0.011	0.013	0.015

Calculate total length of box:				
Length from pipe exterior to inside of box		2	ft	
Pipe thickness		0.46	ft	
Pipe diameter		4.5	ft	
Pipe thickness		0.46	ft	
Clearance between pipes		3	ft	
Pipe thickness		0.46	ft	
Pipe diameter		4.5	ft	
Pipe thickness		0.46	ft	
Length from box u/s side to end of pipes on LB (y)		15.83		
Angle of pipe hitting opposite side of structure		78.82	degrees	
Jet expansion angle		7.00	degrees	
Length from u/s side of box to pipe's jet hitting opp wall (x)		24.04		
Length of bend scour		18.41	ft	
Total length of scour protection needed:		42.45	ft	
Length of structure		35.00	ft	
Length of riprap needed		7.45		
Calculate height of structure.				
Height of structure will be at elevation 123'; if Old Calaveras Headworks gates are ever closed downstream while the forebay is at elevation 121.44,				
this will allow 1.5 feet of freeboard within the structure.				
Invert of structure is at EL 112', so an 11-foot tall structure.				
Objective (2) Results				
OCR Structure will be 25' width; 35' length; 11' height (interior dimensions); and will involve 7.5' of a riprap apron.				
Objective (3)				
Design energy dissipation apron for outlet of structure				
Riprap aprons after energy dissipators				
Reference:	FHWA 2006. Hydraulic Design of Energy Dissipators for Culverts and Channels. Hydraulic Engineering Circular Number 14.			
	https://www.fhwa.dot.gov/engineering/hydraulics/pubs/06086/hec14.pdf			
Equation for "RIPRAP APRONS AFTER ENERGY DISSIPATORS"				
$D_{50} = \frac{0.692}{s-1}\left(\frac{v^2}{2g}\right)$	p. 10-19			
V	4.8	fps	Use V from half of structure to be conservative	
S	2.65			
g	32.2	ft/s^2		
D_{50}	0.15	ft		
	1.80	in		
Riprap class	Class I	Caltrans		
D_{50}	6.00	in		



10.3 RIPRAP APRONS AFTER ENERGY DISSIPATORS

Some energy dissipators provide exit conditions, velocity and depth, near critical. This flow condition rapidly adjusts to the downstream or natural channel regime; however, critical velocity may be sufficient to cause erosion problems requiring protection adjacent to the energy dissipator. Equation 10.6 provides the riprap size recommended for use downstream of energy dissipators. This relationship is from Searcy (1967) and is the same equation used in HEC 11 (Brown and Clyde, 1989) for riprap protection around bridge piers.

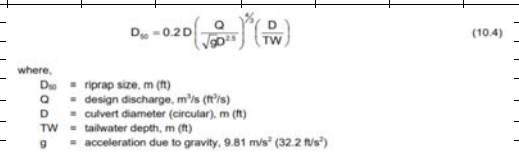
$$D_{50} = \frac{0.692}{S-1}\left(\frac{V^2}{2g}\right) \qquad (10.6)$$

10-19

where,
D_{50} = median rock size, m (ft)
V = velocity at the exit of the dissipator, m/s (ft/s)
S = riprap specific gravity

The length of protection can be judged based on the magnitude of the exit velocity compared with the natural channel velocity. The greater this difference, the longer will be the length required for the exit flow to adjust to the natural channel condition. A filter blanket should also be provided as described in HEC 11 (Brown and Clyde, 1989).

Equation for "RIPRAP APRONS"				
$D_{50} = 0.2D\left(\frac{Q}{\sqrt{g}D^{2.5}}\right)^{4/3}\left(\frac{D}{Tw}\right)$			p. 10-17	
Q	150	cfs		
D	4.5	ft		
TW	2.5	ft		
g	32.2	ft/s^2		
D_{50}	0.85	ft		
	10.18	in		
Riprap class	Class III	Caltrans		
D_{50}	12	in		
Recommend we use Class III, 12" diameter instead of the Class I rock.				
Our structure isn't technically an energy dissipation structure, since we didn't use a transition or baffle blocks or anything.				
Additionally, KSN is already specifying Class III riprap on their earthen embankment. To allow us to use fewer different riprap gradations, using Class III would make more sense.				



$$D_{50} = 0.2\,D\left(\frac{Q}{\sqrt{g}D^{2.5}}\right)^{4/3}\left(\frac{D}{TW}\right) \qquad (10.4)$$

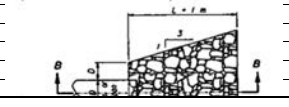
where,
D_{50} = riprap size, m (ft)
Q = design discharge, m^3/s (ft^3/s)
D = culvert diameter (circular), m (ft)
TW = tailwater depth, m (ft)
g = acceleration due to gravity, 9.81 m/s^2 (32.2 ft/s^2)

Riprap apron dimensions		p. 10-18		
Class	Apron Length (ft)	Apron Depth (in)	Apron Depth (ft)	
I	18	21	1.75	
III	22.5	28.8	2.4	
			Round 2.4' depth to 2.5' for ease.	
Apron width is to expand at a 3:1 rate until it conforms to the width of channel per p. 10-18.				
In this case the apron begins already at the width of the channel / structure (25'). Apron width should accommodate the expected WS in channel.				
Expected channel WS:		115.9	ft	
Freeboard:		2	ft	
Contour apron should reach:		118	ft	
Objective (3) Results				
OCR Structure apron will be to 118' bank contour in width; 22.5' in length; 2.5' in depth.				

Table 10.1. Example Riprap Classes and Apron Dimensions

Class	D_{50} (mm)	D_{50} (in)	Apron Length[1]	Apron Depth
1	125	5	4D	$3.5D_{50}$
2	150	6	4D	$3.3D_{50}$
3	250	10	5D	$2.4D_{50}$
4	350	14	6D	$2.2D_{50}$
5	500	20	7D	$2.0D_{50}$
6	550	22	8D	$2.0D_{50}$

[1]D is the culvert rise.

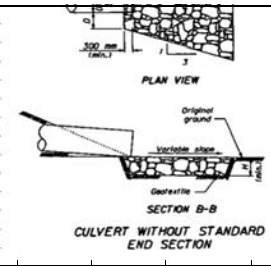


Rock Gradation

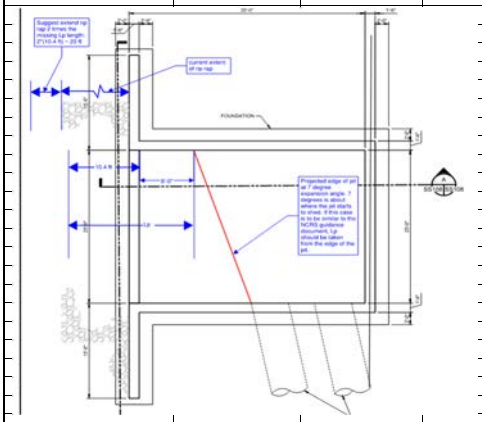
Nominal RSP class	Nominal

by median particle diameter[b]		median particle weight W_{50}[c,d]	d_{15}[c] (inches)		d_{50}[c] (inches)		d_{100}[c] (inches)	Placement
Class[a]	Diameter (inches)		Min	Max	Min	Max	Max	Method
I	6	20 lb	3.7	5.2	5.7	6.9	12.0	B
II	9	60 lb	5.5	7.8	8.5	10.5	18.0	B
III	12	150 lb	7.3	10.5	11.5	14.0	24.0	B
IV	15	300 lb	9.2	13.0	14.5	17.5	30.0	B
V	18	1/4 ton	11.0	15.5	17.0	20.5	36.0	B
VI	21	3/8 ton	13.0	18.5	20.0	24.0	42.0	A or B
VII	24	1/2 ton	14.5	21.0	23.0	27.5	48.0	A or B
VIII	30	1 ton	18.5	26.0	28.5	34.5	48.0	A or B
IX	36	2 ton	22.0	31.5	34.0	41.5	52.8	A
X	42	3 ton	25.5	36.5	40.0	48.5	60.5	A
XI	46	4 ton	28.0	39.4	43.7	53.1	66.6	A

[a]For RSP Classes I–VIII, use Class 8 RSP fabric. For RSP Classes IX–XI, use Class 10 RSP fabric.
[b]Intermediate or B dimension (i.e., width) where A dimension is length and C dimension is thickness.
[c]d%, where % denotes the percentage of the total weight of the graded material.
[d]Values shown are based on the minimum and maximum particle diameters shown and an average specific gravity of 2.65. Weight will vary based on specific gravity of rock available for the project.



Total:	
Structure:	**OCR Structure will be 25' width; 35' length; 11' height (interior dimensions).**
Riprap:	**Riprap will be a total of 30' in length; to 118' bank contour in width; 2.5' depth.**



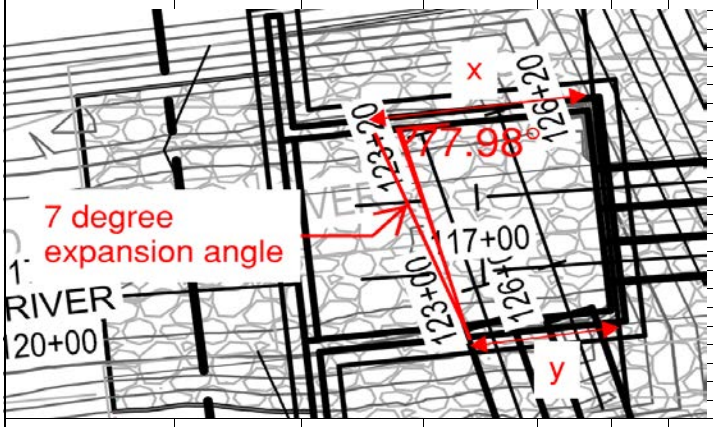


Figure for calculation of length of box

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Appendix F. Mechanical Calculations

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MECHANICAL CALCULATIONS

48” FISH SCREEN GATE ACTUATOR SIZING

FOR THE

BELLOTA WEIR MODIFICATIONS PROJECT

PREPARED FOR

STOCKTON EAST WATER DISTRICT

CALCULATIONS PREPARED BY

HDR ENGINEERING, INC.

EMMA PAZOFF, E.I.T.

DATE PREPARED: 7/15/21

QC REVIEWER: LARRY PAULS, P.E.

QC REVIEW DATE: 8/2/21

DATE SUBMITTED: 10/06/21

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Exp. MM-DD-YYYY

DESCRIPTION:

This calculation is used to gain preliminary sizing of the 48" fish screen stop gate actuator located in the intake. The gates will be lowered to the closed position when the tee screens are in the raised position for maintenance. The gates will be raised to the open position when the tee screens are actively taking water. Reference BLVD_MOGx_2101 and BLVD_MOGx_2102 in the gate schedule.

Key:

Input Value	Calculated

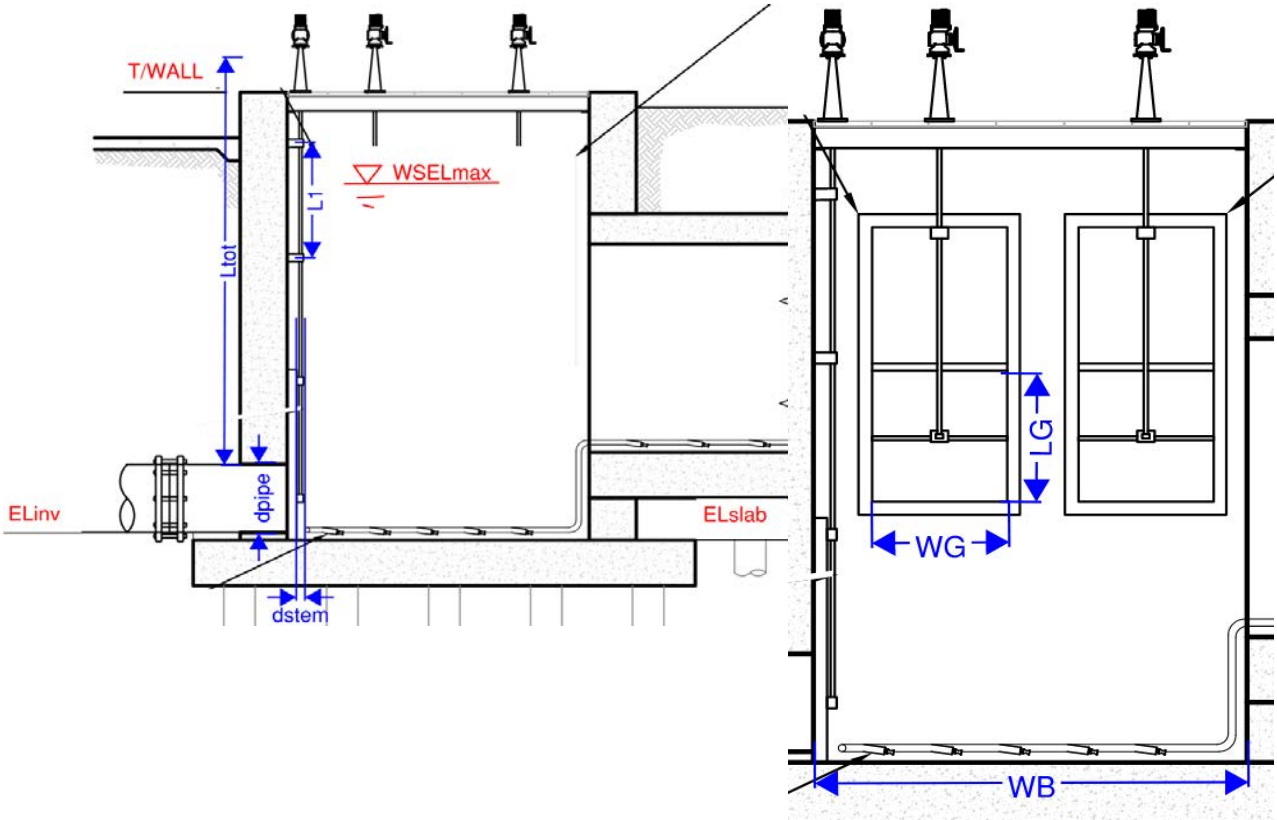
Unit Change

REFERENCES:

The following are referenced throughout the calculation:

1) 90% Design Package, Bellota Weir Modifications
2) Machinery's Handbook, 26th Ed
3) Mechanical Engineering Reference Manual, 12th Ed
4) Shigley's Machine Design

SCHEMATIC:



ASSUMPTIONS:

Assumptions made during this calculation are as follows:

Description	Variable	Value and Unit	Reference
Loads			
Weight of gate leaf	W_{leaf}	161 lbs	estimated
Service Factor	SF	1.15	EM 1110-2-2703, 5.2.3.b
Volumetric flow	Q	63 cfs	Ref 1
Geometry			
Gate			
Gate width	W_G	48 in	Ref 1
Gate height	L_G	48 in	Ref 1
Stem diameter	d_{stem}	1.25 in	
Threads per inch	TPI_{stem}	4 threads/in	Ref 2, pg 1795-1798
Thread type		ACME	
Length to first support	L_1	54 in	length of gate + 6 in
Total stem length	L_{tot}	217 in	Ref 1, $=(T_{wall} + 2') - EL_{inv} - L_G$
Intake			
Culvert height	$H_{culvert}$	48 in	
Top of wall	T_{wall}	130 ft	
Maximum WSEL	$WSEL_{max}$	127.0 ft	Ref 1, DCR Table 3-6
Wall penetration invert EL	EL_{inv}	109.9 ft	
Wall penetration CL EL	EL_{CL}	111.9 ft	$= EL_{inv} + (H_{culvert}/2)$
Slab elevation	EL_{slab}	108.2 ft	Ref 1
Width of bay	W_{AB}	5 ft	Ref 1
Machinery/Parts			
Rotational speed	RPM_{output}	52 rpm	Limitorque MX-20
Rated torque	T	225 ft-lbs	Limitorque MX-20
Efficiency	η_{motor}	80%	
Material Properties			
Density of water	rho_w	1.94 slugs/ft^3	
Gravity	g	32.2 ft/s^2	
Seal friction coeff	mu	0.3	
Stem material		A276 T316	
	rho_{stem}	0.28 lbs/in^3	
	$F_{y,stem}$	30 ksi	
	$F_{u,stem}$	75 ksi	
	E_{stem}	28000 ksi	
Thrust/Lift Nut		B584 CA863	
Material	$F_{y,nut}$	60 ksi	
	$F_{u,nut}$	110 ksi	
Stem/nut friction	f	0.38	

Allowable Stresses

	Tens/Comp	Bearing	Shear	
Stem	13.5	27.0	12.0	ksi
Thrust Nut	27.0	54.0	24.0	ksi

showing allowable stresses for tension, shear, bending and bearing. Note, allowable bearing is for when no movement is intended between the two components

TENSION	$0.45S_y \le \sigma_{all} \le 0.60S_y$
SHEAR	$\tau_{all} = 0.40S_y$
BENDING	$0.60S_y \le \sigma_{all} \le 0.75S_y$
BEARING	$\sigma_{all} = 0.90S_y$

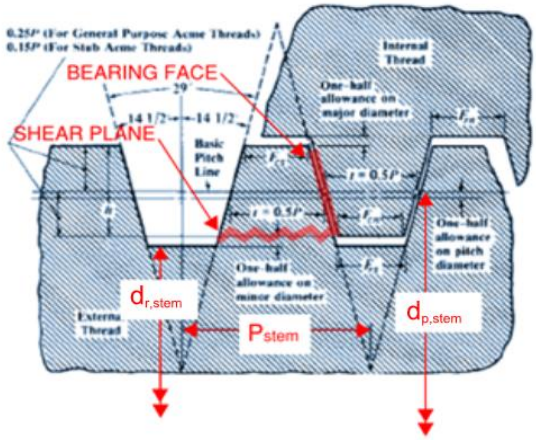
CALCULATIONS:

The following calculations are based on the assumptions and references provided above.

Max operating head on the gate	$h_G = WSEL_{max} - EL_{CL} =$	15.1 ft
Hydrostatic pressure on gate	$P_{hys} = h_G/2.31 =$	6.5 psi
Area of water acting on gate	$A_G = W_G * L_G =$	2304 in^2
Force acting on gate due to hydrostatic pressure	$F_{hys} = P_{hys} * A_G =$	15061 lbs
Frictional resistance force	$F_{FR} = F_{hys} * mu =$	4518 lbs
Weight of stem	$W_{stem} = pi*(d_{stem}/2)^2*L_{tot}*rho_{stem} =$	74.6 lbs
Cross sectional area of channel	$A_{channel} = W_{AB}*(WSEL_{max}-EL_{slab}) =$	94 ft^2
Upstream channel velocity	$v_{US} = Q/A_{channel} =$	0.7 ft/s
Dynamic force on gate	$F_{DG} = rho_w * A_G * (v_{US})^2 =$	14 lbs
Vertical force required to overcome F_{DG}	$F_{DGV} = F_{DG} * mu =$	3.90 lbs
Weight actuator needs to be sized for		
Opening (tension)	$F_{act} = SF*(W_{leaf} + F_{FR} + W_{stem} + F_{DGV}) =$	5472 lbs
Closing (compression)	$F_{act} = SF*(F_{FR} + F_{DGV}) =$	5200 lbs

Stem[Ref 2, ACME threads]

Pitch	$P_{stem} = 1/TPI_{stem}$ =	0.25 in
Lead	$L_{stem} = 1/TPI_{stem}$ =	0.25 in
Pitch diameter	$d_{p,stem} = d_{stem} - (0.5*P_{stem})$ =	1.125 in
Root diameter	$d_{r,stem} = d_{stem} - P_{stem}$ =	1.00 in
Nut length	$L_{nut} = 2*d_{stem}$ =	2.5 in
No. threads engaged	$N_{thr} = L_{nut}*TPI_{stem}$ =	10 threads
Thread half-angle	Θ =	14.5 degrees



Stresses on Stem

Tension

	$\sigma_{T,stem} = F_{act}/(pi*(d_{r,stem}/2)^2)$ =	6967 psi
	$\sigma_{T,stem}$ =	*6.97* ksi
	Check: Is $\sigma_{T,stem} < \sigma_{T,allowable}$?	OK

Bearing

Bearing area of single thread

	$A_{stem,B1} = d_{p,stem}*pi*0.5*P_{stem}*\cos(14.5)$ =	0.428 in^2

Total bearing area of engaged threads[Ref 4]

	$A_{stem,B} = 3\ threads*A_{stem,B1}$ =	1.283 in^2
Bearing stress	$\sigma_{B,stem} = F_{act}/A_{stem,B}$ =	4264 psi
	$\sigma_{B,stem}$ =	*4.26* ksi
	Check: Is $\sigma_{B,stem} < \sigma_{B,allowable}$?	OK

Shear

Shear area per thread

$A_{stem,S1} = d_{p,stem}*pi*0.5*P_{stem}$ = 0.442 in^2

Total shear area[Ref 4]

$A_{stem,S} = 3\ threads*A_{stem,S1}$ = 1.325 in^2

Shear stress

$\tau_{stem} = F_{act}/A_{stem,S}$ = 4129 psi

τ_{stem} = *4.13* ksi

Check: Is $\tau_{stem} < \tau_{stem,allowable}$? OK

Buckling[Ref 3, Ch 51, Sec 1]

Radius of gyration $k = d_{p,stem}/4$ = 0.281 in

Min area of stem $A = pi*(d_{p,stem}/2)^2$ = 0.994 in^2

Slenderness criterion $SR_{crit} = (2*pi^2*E_{stem}/F_{y,stem})^{1/2}$ = 135.7

Slenderness ratio $SR = L_1/k$ = 192.0

Check: SR_{crit} < SR, so Long Column

Euler stress, max allowable compressive stress

$\sigma_e = pi^2*E_{stem}/(SR^2)$ = 7.4964 ksi

Max allowable compressive force on stem

$F_{max} = \sigma_e*A$ = 7451.6 lbs

Check: Is $F_C < F_{max}$? OK

Time for one operation (fully closed to fully open, or vice versa)

Height traveled per one revolution $L_{one\ rev} = P_{stem}$ = 0.25 in/rev

No. revolutions per one operation $N_{rev} = L_1/L_{one\ rev}$ = 216 rev/op

Time for one operation $t_{op} = N_{rev}/RPM_{motor}$ = 4.2 minutes

Torque required to raise the load[Ref 3, Eq'n 52.121]

$$T_{req,raise} = \frac{F_{act} * d_{p,stem}}{2} * \frac{\tan(\theta) + f}{1 - f * \tan(\theta)} =$$ 2180 in-lbs

$T_{design,raise}$ = *181.6 ft-lbs*

Check: $T_{design,raise} < T_{MX20}$? OK

Required actuator horsepower[Ref 3, Eq'n 51.50]

$HP_{motor} = (T_{design,raise}*RPM_{motor}/63025)/\eta_{motor})$ = 2.25 HP

Standard motor size $HP_{motor,std}$ = 3 HP

Additional motor capacity $R_{extra\ cap} = (HP_{motor,std}/HP_{motor}) - 1$ = 0.33

Snippet from Limitorque Cutsheet

Output Speed (RPM)		MX-05		MX-10		MX-20		MX-40		MX-85		MX-140		MX-150	
		Rated Output Torque													
60 Hz	50 Hz	ft-lb	N m	ft-lb	N m	ft-lb	N m	ft-lb	N m	ft-lb	N m	ft-lb	N m	ft-lb	N m
18	15	55	75	125	170	225	305	440	597	n/a	n/a	n/a	n/a	n/a	n/a
26	22	55	75	125	170	225	305	440	597	850	1,153	1,500	2,036	n/a	n/a
40	33	55	75	125	170	225	305	440	597	1,225	1,662	1,700	2,307	n/a	n/a
52	43	55	75	125	170	225	305	440	597	1,150	1,561	1,600	2,171	n/a	n/a
77	65	48	65	107	145	178	241	345	468	850	1,153	1,200	1,628	n/a	n/a
100 131[1]	84 110[1]	39	53	89	121	148	201	286	388	600	814	815	1,105	1,500	2,036
155 170[1]	127 143[1]	41	56	89	121	140	190	260	353	450	611	650	882	1,150	1,561
200	165	34	46	73	99	114	155	210	285	n/a	n/a	n/a	n/a	n/a	n/a

Note 1: MX-85, MX-140 and MX-150

MECHANICAL CALCULATIONS

84” FISH SCREEN GATE ACTUATOR SIZING

FOR THE

BELLOTA WEIR MODIFICATIONS PROJECT

PREPARED FOR

STOCKTON EAST WATER DISTRICT

CALCULATIONS PREPARED BY

HDR ENGINEERING, INC.

EMMA PAZOFF, E.I.T.

DATE PREPARED: 7/15/21

QC REVIEWER: LARRY PAULS, P.E.

QC REVIEW DATE: 8/2/21

DATE SUBMITTED: 10/06/21

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Exp. MM-DD-YYYY

DESCRIPTION:

This calculation is used to gain preliminary sizing of the 84" fish screen stop gate actuator located in the intake. The gates will be lowered to the closed position when the tee screens are in the raised position for maintenance. The gates will be raised to the open position when the tee screens are actively taking water. Reference BLVD_MOGx_2103 and BLVD_MOGx_2104 in the gate schedule.

Key:

Input Value	Calculated

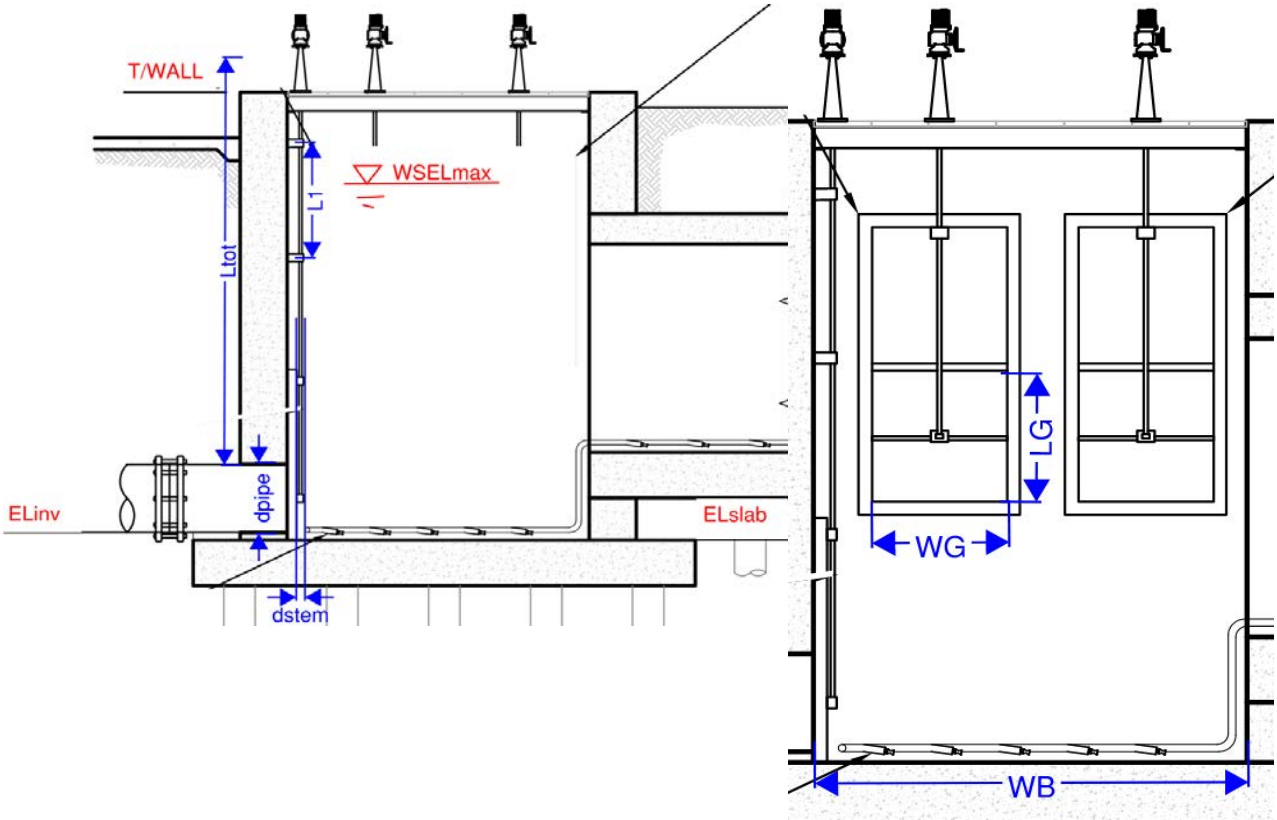
Unit Change

REFERENCES:

The following are referenced throughout the calculation:

1) 90% Design Package, Bellota Weir Modifications
2) Machinery's Handbook, 26th Ed
3) Mechanical Engineering Reference Manual, 12th Ed
4) Shigley's Machine Design

SCHEMATIC:



ASSUMPTIONS:

Assumptions made during this calculation are as follows:

Description	Variable	Value and Unit	Reference
Loads			
Weight of gate leaf	W_{leaf}	567 lbs	estimated
Service Factor	SF	1.15	EM 1110-2-2703, 5.2.3.b
Volumetric flow	Q	174 cfs	Ref 1
Geometry			
Gate			
Gate width	W_G	84 in	Ref 1
Gate height	L_G	84 in	Ref 1
Stem diameter	d_{stem}	2.00 in	
Threads per inch	TPI_{stem}	4 threads/in	Ref 2, pg 1795-1798
Thread type		ACME	
Length to first support	L_1	90 in	length of gate + 6 in
Total stem length	L_{tot}	169 in	Ref 1, $=(T_{wall} + 2') - EL_{inv} - L_G$
Intake			
Culvert height	$H_{culvert}$	90 in	
Top of wall	T_{wall}	130 ft	
Maximum WSEL	$WSEL_{max}$	127.0 ft	Ref 1, DCR Table 3-6
Wall penetration invert EL	EL_{inv}	110.9 ft	
Wall penetration CL EL	EL_{CL}	114.7 ft	$= EL_{inv} + (H_{culvert}/2)$
Slab elevation	EL_{slab}	106.5 ft	Ref 1
Width of bay	W_{AB}	7.5 ft	Ref 1
Machinery/Parts			
Rotational speed	RPM_{output}	52 rpm	Limitorque MX-85
Rated torque	T	1150 ft-lbs	Limitorque MX-85
Efficiency	η_{motor}	80%	
Material Properties			
Density of water	rho_w	1.94 $slugs/ft^3$	
Gravity	g	32.2 ft/s^2	
Seal friction coeff	mu	0.3	
Stem material		A276 T316	
	rho_{stem}	0.28 lbs/in^3	
	$F_{y,stem}$	30 ksi	
	$F_{u,stem}$	75 ksi	
	E_{stem}	28000 ksi	
Thrust/Lift Nut		B584 CA863	
Material	$F_{y,nut}$	60 ksi	
	$F_{u,nut}$	110 ksi	
Stem/nut friction	f	0.38	

Allowable Stresses

	Tens/Comp	Bearing	Shear	
Stem	13.5	27.0	12.0	ksi
Thrust Nut	27.0	54.0	24.0	ksi

showing allowable stresses for tension, shear, bending and bearing. Note, allowable bearing is for when no movement is intended between the two components

TENSION	$0.45S_y \le \sigma_{all} \le 0.60S_y$
SHEAR	$\tau_{all} = 0.40S_y$
BENDING	$0.60S_y \le \sigma_{all} \le 0.75S_y$
BEARING	$\sigma_{all} = 0.90S_y$

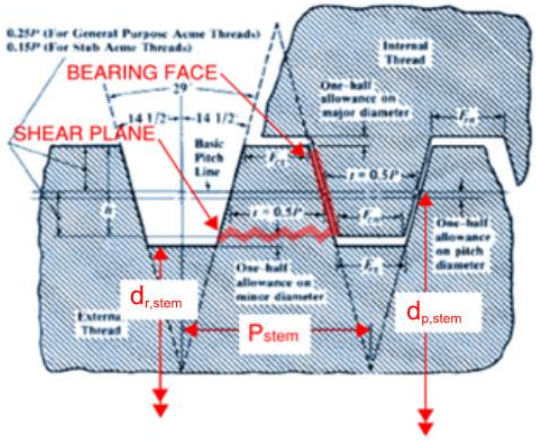
CALCULATIONS:

The following calculations are based on the assumptions and references provided above.

Max operating head on the gate	$h_G = WSEL_{max} - EL_{CL} =$	12.4 ft
Hydrostatic pressure on gate	$P_{hys} = h_G/2.31 =$	5.3 psi
Area of water acting on gate	$A_G = W_G * L_G =$	7056 in^2
Force acting on gate due to hydrostatic pressure	$F_{hys} = P_{hys} * A_G =$	37724 lbs
Frictional resistance force	$F_{FR} = F_{hys} * mu =$	11317 lbs
Weight of stem	$W_{stem} = pi*(d_{stem}/2)^2*L_{tot}*rho_{stem} =$	148.8 lbs
Cross sectional area of channel	$A_{channel} = W_{AB}*(WSEL_{max}-EL_{slab}) =$	154 ft^2
Upstream channel velocity	$v_{US} = Q/A_{channel} =$	1.1 ft/s
Dynamic force on gate	$F_{DG} = rho_w * A_G * (v_{US})^2 =$	122 lbs
Vertical force required to overcome F_{DG}	$F_{DGV} = F_{DG} * mu =$	34.05 lbs
Weight actuator needs to be sized for		
Opening (tension)	$F_{act} = SF*(W_{leaf} + F_{FR} + W_{stem} + F_{DGV}) =$	13877 lbs
Closing (compression)	$F_{act} = SF*(F_{FR} + F_{DGV}) =$	13054 lbs

Stem[Ref 2, ACME threads]

Pitch	$P_{stem} = 1/TPI_{stem}$ =	0.25 in
Lead	$L_{stem} = 1/TPI_{stem}$ =	0.25 in
Pitch diameter	$d_{p,stem} = d_{stem} - (0.5*P_{stem})$ =	1.875 in
Root diameter	$d_{r,stem} = d_{stem} - P_{stem}$ =	1.75 in
Nut length	$L_{nut} = 2*d_{stem}$ =	4 in
No. threads engaged	$N_{thr} = L_{nut}*TPI_{stem}$ =	16 threads
Thread half-angle	Θ =	14.5 degrees



Stresses on Stem

Tension

$\sigma_{T,stem} = F_{act}/(pi*(d_{r,stem}/2)^2)$ =	5769 psi
$\sigma_{T,stem}$ =	*5.77* ksi
Check: Is $\sigma_{T,stem} < \sigma_{T,allowable}$?	OK

Bearing

Bearing area of single thread

$A_{stem,B1} = d_{p,stem}*pi*0.5*P_{stem}*\cos(14.5)$ =	0.713 in^2

Total bearing area of engaged threads[Ref 4]

$A_{stem,B} = 3\ threads*A_{stem,B1}$ =	2.139 in^2
Bearing stress $\sigma_{B,stem} = F_{act}/A_{stem,B}$ =	6489 psi
$\sigma_{B,stem}$ =	*6.49* ksi
Check: Is $\sigma_{B,stem} < \sigma_{B,allowable}$?	OK

Shear

Shear area per thread

$A_{stem,S1} = d_{p,stem}*pi*0.5*P_{stem}$ = 0.736 in^2

Total shear area[Ref 4]

$A_{stem,S} = 3\ threads*A_{stem,S1}$ = 2.209 in^2

Shear stress

$\tau_{stem} = F_{act}/A_{stem,S}$ = 6282 psi

τ_{stem} = *6.28* ksi

Check: Is $\tau_{stem} < \tau_{stem,allowable}$? OK

Buckling[Ref 3, Ch 51, Sec 1]

Radius of gyration $k = d_{p,stem}/4$ = 0.469 in

Min area of stem $A = pi*(d_{p,stem}/2)^2$ = 2.761 in^2

Slenderness criterion $SR_{crit} = (2*pi^2*E_{stem}/F_{y,stem})^{1/2}$ = 135.7

Slenderness ratio $SR = L_1/k$ = 192.0

Check: SR_{crit} < SR, so Long Column

Euler stress, max allowable compressive stress

$\sigma_e = pi^2*E_{stem}/(SR^2)$ = 7.4964 ksi

Max allowable compressive force on stem

$F_{max} = \sigma_e*A$ = 20698.9 lbs

Check: Is $F_C < F_{max}$? OK

Time for one operation (fully closed to fully open, or vice versa)

Height traveled per one revolution $L_{one\ rev} = P_{stem}$ = 0.25 in/rev

No. revolutions per one operation $N_{rev} = L_1/L_{one\ rev}$ = 360 rev/op

Time for one operation $t_{op} = N_{rev}/RPM_{motor}$ = 6.9 minutes

Torque required to raise the load[Ref 3, Eq'n 52.121]

$$T_{req,raise} = \frac{F_{act} * d_{p,stem}}{2} * \frac{\tan(\theta) + f}{1 - f * \tan(\theta)} =$$ 9214 in-lbs

$T_{design,raise}$ = *767.8 ft-lbs*

Check: $T_{design,raise} < T_{MX85}$? OK

Required actuator horsepower[Ref 3, Eq'n 51.50]

$HP_{motor} = (T_{design,raise}*RPM_{motor}/63025)/\eta_{motor})$ = 9.50 HP

Standard motor size $HP_{motor,std}$ = 10 HP

Additional motor capacity $R_{extra\ cap} = (HP_{motor,std}/HP_{motor}) - 1$ = 0.05

Snippet from Limitorque Cutsheet

Output Speed (RPM)		MX-05		MX-10		MX-20		MX-40		MX-85		MX-140		MX-150	
		Rated Output Torque													
60 Hz	50 Hz	ft-lb	N m	ft-lb	N m	ft-lb	N m	ft-lb	N m	ft-lb	N m	ft-lb	N m	ft-lb	N m
18	15	55	75	125	170	225	305	440	597	n/a	n/a	n/a	n/a	n/a	n/a
26	22	55	75	125	170	225	305	440	597	850	1,153	1,500	2,036	n/a	n/a
40	33	55	75	125	170	225	305	440	597	1,225	1,662	1,700	2,307	n/a	n/a
52	43	55	75	125	170	225	305	440	597	1,150	1,561	1,600	2,171	n/a	n/a
77	65	48	65	107	145	178	241	345	468	850	1,153	1,200	1,628	n/a	n/a
100 131[1]	84 110[1]	39	53	89	121	148	201	286	388	600	814	815	1,105	1,500	2,036
155 170[1]	127 143[1]	41	56	89	121	140	190	260	353	450	611	650	882	1,150	1,561
200	165	34	46	73	99	114	155	210	285	n/a	n/a	n/a	n/a	n/a	n/a

Note 1: MX-85, MX-140 and MX-150

MECHANICAL CALCULATIONS

BYPASS FLOW GATE ACTUATOR SIZING

FOR THE

BELLOTA WEIR MODIFICATIONS PROJECT

PREPARED FOR

STOCKTON EAST WATER DISTRICT

CALCULATIONS PREPARED BY

HDR ENGINEERING, INC.

EMMA PAZOFF, E.I.T.

DATE PREPARED: 7/15/21

QC REVIEWER: LARRY PAULS, P.E.

QC REVIEW DATE: 8/2/21

DATE SUBMITTED: 10/06/21

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Exp. MM-DD-YYYY

DESCRIPTION:

This calculation is used to gain preliminary sizing of the bypass flow gate actuator located in the intake manifold. This gate provides additional flow to the afterbay when needed. Reference BLVD_MOGx_2106 in the gate schedule.

Key:

Input Value	Calculated

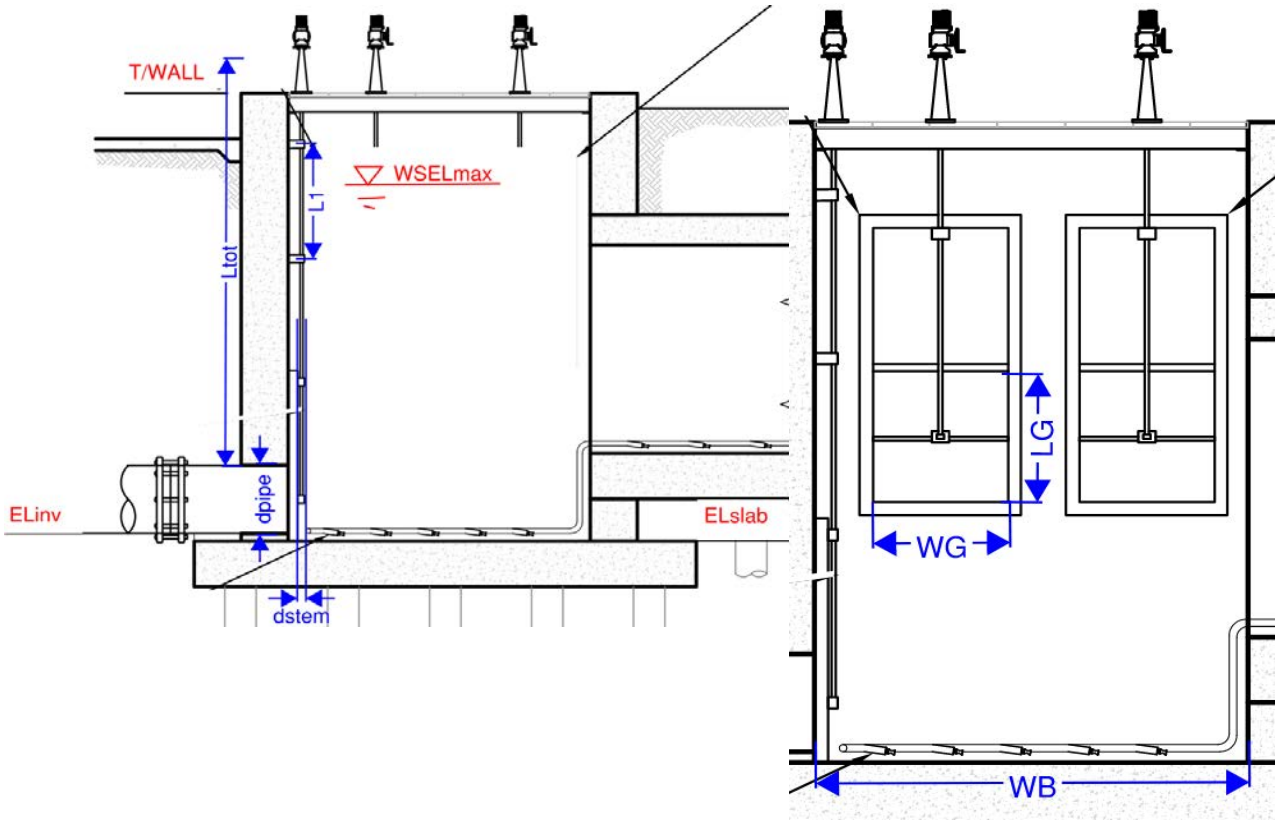
Unit Change

REFERENCES:

The following are referenced throughout the calculation:

1) 90% Design Package, Bellota Weir Modifications
2) Machinery's Handbook, 26th Ed
3) Mechanical Engineering Reference Manual, 12th Ed
4) Shigley's Machine Design

SCHEMATIC:



ASSUMPTIONS:

Assumptions made during this calculation are as follows:

Description	Variable	Value and Unit	Reference
Loads			
Weight of gate leaf	W_{leaf}	600 lbs	assumed
Service Factor	SF	1.15	EM 1110-2-2703, 5.2.3.b
Volumetric flow	Q	100 cfs	Ref 1, minimum river flow
Geometry			
Gate			
Gate width	W_G	72 in	Ref 1
Gate height	L_G	72 in	Ref 1
Stem diameter	d_{stem}	1.75 in	assumed
Threads per inch	TPI_{stem}	4 threads/in	Ref 2, pg 1795-1798
Thread type		ACME	
Length to first support	L_1	78 in	length of gate + 6 in
Total stem length	L_{tot}	102 in	Ref 1, $=(T_{wall} + 2') - EL_{inv} - L_G$
Intake			
Culvert height	d_{pipe}	72 in	
Top of wall	T_{wall}	127 ft	
Maximum WSEL	$WSEL_{max}$	127.0 ft	Ref 1, DCR Table 3-6
Wall penetration invert EL	EL_{inv}	114.5 ft	
Wall penetration CL EL	EL_{CL}	117.5 ft	$= EL_{inv} + (d_{pipe}/2)$
Slab elevation	EL_{slab}	106.0 ft	Ref 1
Width of bay	W_{AB}	8.00 ft	Ref 1
Machinery/Parts			
Rotational speed	RPM_{motor}	52 rpm	Limitorque MX-40
Rated torque, T	T	440 ft-lbs	Limitorque MX-40
Efficiency	η_{motor}	80%	
Material Properties			
Density of water	rho_w	1.94 slugs/ft^3	
Gravity	g	32.2 ft/s^2	
Seal friction coeff	mu	0.3	
Stem material		A276 T316	
	rho_{stem}	0.28 lbs/in^3	
	$F_{y,stem}$	30 ksi	
	$F_{u,stem}$	75 ksi	
	E_{stem}	28000 ksi	
Thrust/Lift Nut		B584 CA863	
Material	$F_{y,nut}$	60 ksi	
	$F_{u,nut}$	110 ksi	
Stem/nut friction	f	0.38	

Allowable Stresses

	Tens/Comp	Bearing	Shear	
Stem	13.5	27.0	12.0	ksi
Thrust Nut	27.0	54.0	24.0	ksi

showing allowable stresses for tension, shear, bending and bearing. Note, allowable bearing is for when no movement is intended between the two components

TENSION	$0.45S_y \le \sigma_{all} \le 0.60S_y$
SHEAR	$\tau_{all} = 0.40S_y$
BENDING	$0.60S_y \le \sigma_{all} \le 0.75S_y$
BEARING	$\sigma_{all} = 0.90S_y$

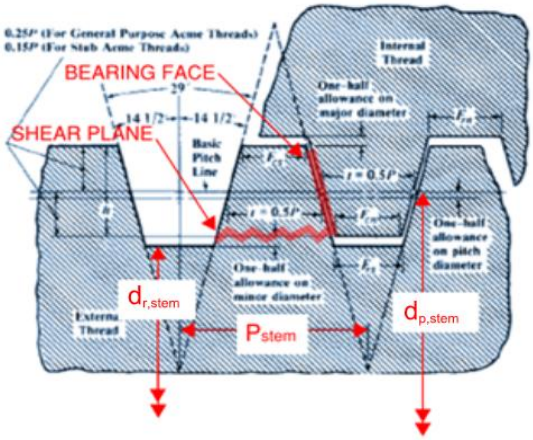
CALCULATIONS:

The following calculations are based on the assumptions and references provided above.

Max operating head on the gate	$h_G = WSEL_{max} - EL_{CL} =$	9.5 ft
Hydrostatic pressure on gate	$P_{hys} = h_G/2.31 =$	4.1 psi
Area of water acting on gate	$A_G = W_G * L_G =$	5184 in^2
Force acting on gate due to hydrostatic pressure	$F_{hys} = P_{hys} * A_G =$	21319.48 lbs
Frictional resistance force	$F_{FR} = F_{hys} * mu =$	6395.844 lbs
Weight of stem	$W_{stem} = pi*(d_{stem}/2)^2*L_{tot}*rho_{stem} =$	68.7 lbs
Cross sectional area of channel	$A_{channel} = W_{AB}*(WSEL_{max}-EL_{slab}) =$	168 ft^2
Upstream channel velocity	$v_{US} = Q/A_{channel} =$	0.6 ft/s
Horizontal dynamic force on gate	$F_{DG} = rho_w*A_G*(v_{US})^2 =$	25 lbs
Vertical force required to overcome F_{DG}	$F_{DGV} = F_{DG}*mu =$	7 lbs
Weight actuator needs to be sized for		
Opening (tension)	$F_{act} = SF*(W_{leaf} + F_{FR} + W_{stem} + F_{DGV}) =$	8132 lbs
Closing (compression)	$F_{act} = SF*(F_{FR} + F_{DGV}) =$	7363 lbs

Stem[Ref 2, ACME threads]

Pitch	$P_{stem} = 1/TPI_{stem}$ =	0.25 in
Lead	$L_{stem} = 1/TPI_{stem}$ =	0.25 in
Pitch diameter	$d_{p,stem} = d_{stem} - (0.5*P_{stem})$ =	1.625 in
Root diameter	$d_{r,stem} = d_{stem} - P_{stem}$ =	1.50 in
Nut length	$L_{nut} = 2*d_{stem}$ =	3.5 in
No. threads engaged	$N_{thr} = L_{nut}*TPI_{stem}$ =	14 threads
Thread half-angle	Θ =	14.5 degrees



Stresses on Stem

Tension

	$\sigma_{T,stem} = F_{act}/(pi*(d_{r,stem}/2)^2)$ =	4602 psi
	$\sigma_{T,stem}$ =	*4.60* ksi
	Check: Is $\sigma_{T,stem} < \sigma_{T,allowable}$?	OK

Bearing

Bearing area of single thread

	$A_{stem,B1} = d_{p,stem}*pi*0.5*P_{stem}*\cos(14.5)$ =	0.618 in^2

Total bearing area of engaged threads[Ref 4]

	$A_{stem,B} = 3\ threads*A_{stem,B1}$ =	1.853 in^2
Bearing stress	$\sigma_{B,stem} = F_{act}/A_{stem,B}$ =	4388 psi
	$\sigma_{B,stem}$ =	*4.39* ksi
	Check: Is $\sigma_{B,stem} < \sigma_{B,allowable}$?	OK

Shear

Shear area per thread

$A_{stem,S1} = d_{p,stem}*pi*0.5*P_{stem}$ = 0.638 in^2

Total shear area[Ref 4]

$A_{stem,S} = 3\ threads*A_{stem,S1}$ = 1.914 in^2

Shear stress

$\tau_{stem} = F_{act}/A_{stem,S}$ = 4248 psi

τ_{stem} = *4.25* ksi

Check: Is $\tau_{stem} < \tau_{stem,allowable}$? OK

Buckling[Ref 3, Ch 51, Sec 1]

Radius of gyration $k = d_{p,stem}/4$ = 0.406 in

Min area of stem $A = pi*(d_{p,stem}/2)^2$ = 2.074 in^2

Slenderness criterion $SR_{crit} = (2*pi^2*E_{stem}/F_{y,stem})^{1/2}$ = 135.7

Slenderness ratio $SR = L_1/k$ = 192.0

Check: SR_{crit} < SR, so Long Column

Euler stress, max allowable compressive stress

$\sigma_e = pi^2*E_{stem}/(SR^2)$ = 7.50 ksi

Max allowable compressive force on stem

$F_{max} = \sigma_e*A$ = 15547.2 lbs

Check: Is $F_C < F_{max}$? OK

Time for one operation (fully closed to fully open, or vice versa)

Height traveled per one revolution $L_{one\ rev} = P_{stem}$ = 0.25 in/rev

No. revolutions per one operation $N_{rev} = L_1/L_{one\ rev}$ = 312 rev/op

Time for one operation $t_{op} = N_{rev}/RPM_{motor}$ = 6.0 minutes

Torque required to raise the load[Ref 3, Eq'n 52.121]

$$T_{req,raise} = \frac{F_{act} * d_{p,stem}}{2} * \frac{\tan(\theta) + f}{1 - f * \tan(\theta)} =$$ 4679 in-lbs

$T_{design,raise}$ = *390.0 ft-lbs*

Check: $T_{design,raise} < T_{MX85}$? OK

Required actuator horsepower[Ref 3, Eq'n 51.50]

$HP_{motor} = (T_{design,raise}*RPM_{motor}/63025)/\eta_{motor})$ = 4.83 HP

Standard motor size $HP_{motor,std}$ = 5 HP

Additional motor capacity $R_{extra\ cap} = (HP_{motor,std}/HP_{motor}) - 1$ = 0.04

Snippet from Limitorque Cutsheet

Output Speed (RPM)		MX-05		MX-10		MX-20		MX-40		MX-85		MX-140		MX-150	
		Rated Output Torque													
60 Hz	50 Hz	ft-lb	N m	ft-lb	N m	ft-lb	N m	ft-lb	N m	ft-lb	N m	ft-lb	N m	ft-lb	N m
18	15	55	75	125	170	225	305	440	597	n/a	n/a	n/a	n/a	n/a	n/a
26	22	55	75	125	170	225	305	440	597	850	1,153	1,500	2,036	n/a	n/a
40	33	55	75	125	170	225	305	440	597	1,225	1,662	1,700	2,307	n/a	n/a
52	43	55	75	125	170	225	305	440	597	1,150	1,561	1,600	2,171	n/a	n/a
77	65	48	65	107	145	178	241	345	468	850	1,153	1,200	1,628	n/a	n/a
100 131[1]	84 110[1]	39	53	89	121	148	201	286	388	600	814	815	1,105	1,500	2,036
155 170[1]	127 143[1]	41	56	89	121	140	190	260	353	450	611	650	882	1,150	1,561
200	165	34	46	73	99	114	155	210	285	n/a	n/a	n/a	n/a	n/a	n/a

Note 1: MX-85, MX-140 and MX-150

MECHANICAL CALCULATIONS

CALAVERAS INTAKE GATE ACTUATOR SIZING

FOR THE

BELLOTA WEIR MODIFICATIONS PROJECT

PREPARED FOR

STOCKTON EAST WATER DISTRICT

CALCULATIONS PREPARED BY

HDR ENGINEERING, INC.

EMMA PAZOFF, E.I.T.

DATE PREPARED: 7/15/21

QC REVIEWER: LARRY PAULS, P.E.

QC REVIEW DATE: 8/2/21

DATE SUBMITTED: 10/06/21

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Exp. MM-DD-YYYY

DESCRIPTION:

This calculation is used to gain preliminary sizing of the slide gate actuators located at the Calaveras River intake. These gates control the flow to the Old Calaveras River. Reference BLVD_MOGx_4011 and BLVD_MOGx_4012 on the gate schedule.

Key:

Input Value	Calculated

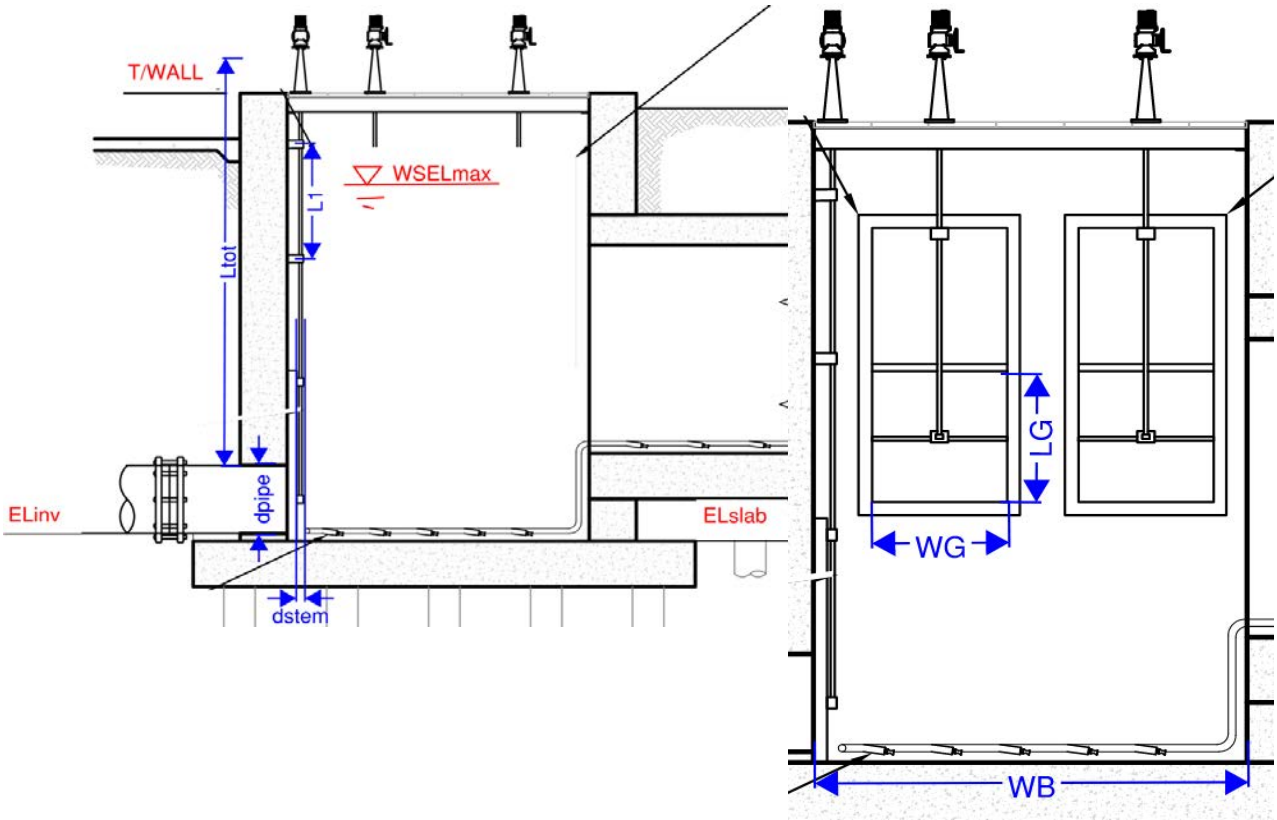
Unit Change

REFERENCES:

The following are referenced throughout the calculation:

1) 90% Design Package, Bellota Weir Modifications
2) Machinery's Handbook, 26th Ed
3) Mechanical Engineering Reference Manual, 12th Ed
4) Shigley's Machine Design

SCHEMATIC:



ASSUMPTIONS:

Assumptions made during this calculation are as follows:

Description	Variable	Value and Unit	Reference
Loads			
Weight of gate leaf	W_{leaf}	950 lbs	
Service Factor	SF	1.15	EM 1110-2-2703, 5.2.3.b
Volumetric flow	Q	150 cfs	Ref 1
Geometry			
Gate			
Gate width	W_G	54 in	Ref 1
Gate height	L_G	54 in	Ref 1
Stem diameter	d_{stem}	1.50 in	
Threads per inch	TPI_{stem}	4 threads/in	Ref 2, pg 1795-1798
Thread type		ACME	
Length to first support	L_1	60 in	length of gate + 6 in
Total stem length	L_{tot}	179 in	Ref 1, $=(T_{wall} + 2') - EL_{inv} - L_G$
Intake			
Culvert height	$H_{culvert}$	54 in	
Top of wall	T_{wall}	130 ft	
Maximum WSEL	$WSEL_{max}$	127.0 ft	Ref 1, DCR Table 3-6
Wall penetration invert EL	EL_{inv}	112.6 ft	
Wall penetration CL EL	EL_{CL}	114.9 ft	$= EL_{inv} +(H_{culvert} /2)$
Slab elevation	EL_{slab}	100.5 ft	Ref 1
Width of bay	W_{AB}	19.00 ft	Ref 1
Machinery/Parts			
Rotational speed	RPM_{motor}	52 rpm	Limitorque MX-40
Rated torque, T	T	440 ft-lbs	Limitorque MX-40
Efficiency	η_{motor}	80%	
Material Properties			
Density of water	rho_w	1.94 slugs/ft^3	
Gravity	g	32.2 ft/s^2	
Seal friction coeff	mu	0.3	
Stem material		A276 T316	
	rho_{stem}	0.28 lbs/in^3	
	$F_{y,stem}$	30 ksi	
	$F_{u,stem}$	75 ksi	
	E_{stem}	28000 ksi	
Thrust/Lift Nut		B584 CA863	
Material	$F_{y,nut}$	60 ksi	
	$F_{u,nut}$	110 ksi	
Stem/nut friction	f	0.38	

Allowable Stresses

	Tens/Comp	Bearing	Shear	
Stem	13.5	27.0	12.0	ksi
Thrust Nut	27.0	54.0	24.0	ksi

showing allowable stresses for tension, shear, bending and bearing. Note, allowable bearing is for when no movement is intended between the two components

TENSION	$0.45S_y \le \sigma_{all} \le 0.60S_y$
SHEAR	$\tau_{all} = 0.40S_y$
BENDING	$0.60S_y \le \sigma_{all} \le 0.75S_y$
BEARING	$\sigma_{all} = 0.90S_y$

CALCULATIONS:

The following calculations are based on the assumptions and references provided above.

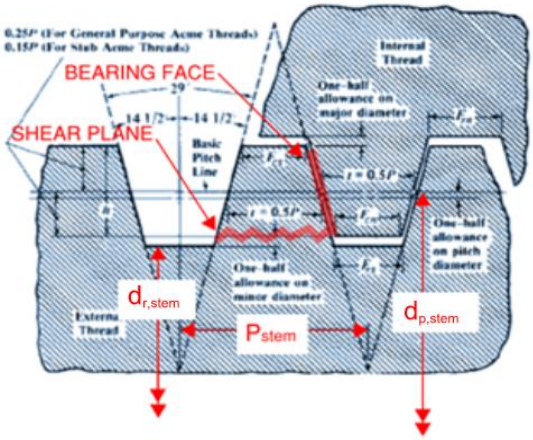
Max operating head on the gate	$h_G = WSEL_{max} - EL_{CL}$ =	12.2 ft
Hydrostatic pressure on gate	$P_{hys} = h_G/2.31$ =	5.3 psi
Area of water acting on gate	$A_G = W_G * L_G$ =	2916 in^2
Force acting on gate due to hydrostatic pressure	$F_{hys} = P_{hys} * A_G$ =	15337.4 lbs
Frictional resistance force	$F_{FR} = F_{hys} * mu$ =	4601.221 lbs
Weight of stem	$W_{stem} = pi*(d_{stem}/2)^2*L_{tot}*rho_{stem}$ =	88.5 lbs
Cross sectional area of channel	$A_{channel} = W_{AB}*(WSEL_{max}-EL_{slab})$ =	504 ft^2
Upstream channel velocity	$v_{US} = Q/A_{channel}$ =	0.3 ft/s
Horizontal dynamic force on gate	$F_{DG} = rho_w*A_G*(v_{US})^2$ =	3 lbs
Vertical force required to overcome F_{DG}	$F_{DGV} = F_{DG}*mu$ =	1 lbs

Weight actuator needs to be sized for

Opening (tension)	$F_{act} = SF*(W_{leaf} + F_{FR} + W_{stem} + F_{DGV})$ =	6487 lbs
Closing (compression)	$F_{act} = SF*(F_{FR} + F_{DGV})$ =	5293 lbs

Stem[Ref 2, ACME threads]

Pitch	$P_{stem} = 1/TPI_{stem}$ =	0.25 in
Lead	$L_{stem} = 1/TPI_{stem}$ =	0.25 in
Pitch diameter	$d_{p,stem} = d_{stem} - (0.5*P_{stem})$ =	1.375 in
Root diameter	$d_{r,stem} = d_{stem} - P_{stem}$ =	1.25 in
Nut length	$L_{nut} = 2*d_{stem}$ =	3 in
No. threads engaged	$N_{thr} = L_{nut}*TPI_{stem}$ =	12 threads
Thread half-angle	Θ =	14.5 degrees



Stresses on Stem

Tension	$\sigma_{T,stem} = F_{act}/(pi*(d_{r,stem}/2)^2)$ =	5286 psi
	$\sigma_{T,stem}$ =	*5.29* ksi
	Check: Is $\sigma_{T,stem} < \sigma_{T,allowable}$?	OK

Bearing

Bearing area of single thread

	$A_{stem,B1} = d_{p,stem}*pi*0.5*P_{stem}*\cos(14.5)$ =	0.523 in^2

Total bearing area of engaged threads[Ref 4]

	$A_{stem,B} = 3\ threads*A_{stem,B1}$ =	1.568 in^2
Bearing stress	$\sigma_{B,stem} = F_{act}/A_{stem,B}$ =	4136 psi
	$\sigma_{B,stem}$ =	*4.14* ksi
	Check: Is $\sigma_{B,stem} < \sigma_{B,allowable}$?	OK

Shear

Shear area per thread

$A_{stem,S1} = d_{p,stem}*pi*0.5*P_{stem}$ = 0.540 in^2

Total shear area[Ref 4]

$A_{stem,S} = 3\ threads*A_{stem,S1}$ = 1.620 in^2

Shear stress

$\tau_{stem} = F_{act}/A_{stem,S}$ = 4004 psi

τ_{stem} = *4.00* ksi

Check: Is $\tau_{stem} < \tau_{stem,allowable}$? OK

Buckling[Ref 3, Ch 51, Sec 1]

Radius of gyration $k = d_{p,stem}/4$ = 0.344 in

Min area of stem $A = pi*(d_{p,stem}/2)^2$ = 1.485 in^2

Slenderness criterion $SR_{crit} = (2*pi^2*E_{stem}/F_{y,stem})^{1/2}$ = 135.7

Slenderness ratio $SR = L_1/k$ = 174.5

Check: SR_{crit} < SR, so Long Column

Euler stress, max allowable compressive stress

$\sigma_e = pi^2*E_{stem}/(SR^2)$ = 9.07 ksi

Max allowable compressive force on stem

$F_{max} = \sigma_e*A$ = 13469.0 lbs

Check: Is $F_C < F_{max}$? OK

Time for one operation (fully closed to fully open, or vice versa)

Height traveled per one revolution $L_{one\ rev} = P_{stem}$ = 0.25 in/rev

No. revolutions per one operation $N_{rev} = L_1/L_{one\ rev}$ = 240 rev/op

Time for one operation $t_{op} = N_{rev}/RPM_{motor}$ = 4.6 minutes

Torque required to raise the load[Ref 3, Eq'n 52.121]

$$T_{req,raise} = \frac{F_{act} * d_{p,stem}}{2} * \frac{\tan(\theta) + f}{1 - f * \tan(\theta)} =$$ 3158 in-lbs

$T_{design,raise}$ = *263.2 ft-lbs*

Check: $T_{design,raise} < T_{MX40}$? OK

Required actuator horsepower[Ref 3, Eq'n 51.50]

$HP_{motor} = (T_{design,raise}*RPM_{motor}/63025)/\eta_{motor})$ = 3.26 HP

Standard motor size $HP_{motor,std}$ = 5 HP

Additional motor capacity $R_{extra\ cap} = (HP_{motor,std}/HP_{motor}) - 1$ = 0.53

Snippet from Limitorque Cutsheet

Output Speed (RPM)		MX-05		MX-10		MX-20		MX-40		MX-85		MX-140		MX-150	
		Rated Output Torque													
60 Hz	50 Hz	ft-lb	N m	ft-lb	N m	ft-lb	N m	ft-lb	N m	ft-lb	N m	ft-lb	N m	ft-lb	N m
18	15	55	75	125	170	225	305	440	597	n/a	n/a	n/a	n/a	n/a	n/a
26	22	55	75	125	170	225	305	440	597	850	1,153	1,500	2,036	n/a	n/a
40	33	55	75	125	170	225	305	440	597	1,225	1,662	1,700	2,307	n/a	n/a
52	43	55	75	125	170	225	305	440	597	1,150	1,561	1,600	2,171	n/a	n/a
77	65	48	65	107	145	178	241	345	468	850	1,153	1,200	1,628	n/a	n/a
100 131[1]	84 110[1]	39	53	89	121	148	201	286	388	600	814	815	1,105	1,500	2,036
155 170[1]	127 143[1]	41	56	89	121	140	190	260	353	450	611	650	882	1,150	1,561
200	165	34	46	73	99	114	155	210	285	n/a	n/a	n/a	n/a	n/a	n/a

Note 1: MX-85, MX-140 and MX-150

MECHANICAL CALCULATIONS

FLOATING DEBRIS BARRIER LOAD

FOR THE

BELLOTA WEIR MODIFICATIONS PROJECT

PREPARED FOR

STOCKTON EAST WATER DISTRICT

CALCULATIONS PREPARED BY

HDR ENGINEERING, INC.

EMMA PAZOFF, E.I.T.

DATE PREPARED: 7/15/21

QC REVIEWER: LARRY PAULS, P.E.

QC REVIEW DATE: 8/2/21

DATE SUBMITTED: 10/06/21

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DESCRIPTION:

Calculate the resultant load on each of the four debris boom piles

REFERENCES:

References used in this calculation are as follows:

1) Bellota Weir Modifications, 90% design drawings
2) Bellota Design Criteria Report
3) ASCE 7-16
4) USACE CRTD No. 96-01
5) EM 1110-2-1100 (Part II)
6) Wave Load on Cylinders, P Frigaard and H.F. Burcharth

ASSUMPTIONS:

Assumptions used in this calcuation are as follows:

Description	Variable	Value	Reference
Geometry			
Diameter of debris boom	D_{boom}	2.5 ft	Ref 1
Height of screen	H_{screen}	3 ft	Ref 1
Width of screen	W_{screen}	0.083 ft	Ref 1
Debris boom chord length	L_c	145 ft	Ref 1
Length from A1 to A2	L_{1-2}	45 ft	Ref 1
Length from A2 to A3	L_{2-3}	55 ft	Ref 1
Boom line sag ratio	s	0.1	Estimate
Material			
Density of water	rho_w	1.94 slugs/ft^3	known constant
Density of air	rho_a	0.0023 slugs/ft^3	known constant
Gravitational acceleration	g	32.2 ft/s^2	known constant
Critera			
Boom drag coefficient	C_d	1.25	assumed
Boom wind drag coefficien	C_w	1	assumed
Maximum water current velocity	$V_{c,max}$	9.4 fps	Ref 2, design flood = 12,690 cfs / (~150 ft long x (127 ft WSEL - 110 ft ground))
Maximum wind velocity	$V_{a,max}$	154 fps	Ref 3, Fig. 26.5-1B, assuming class IV building

SCHEMATICS:

Fig 1 - Plan view of floating debris boom

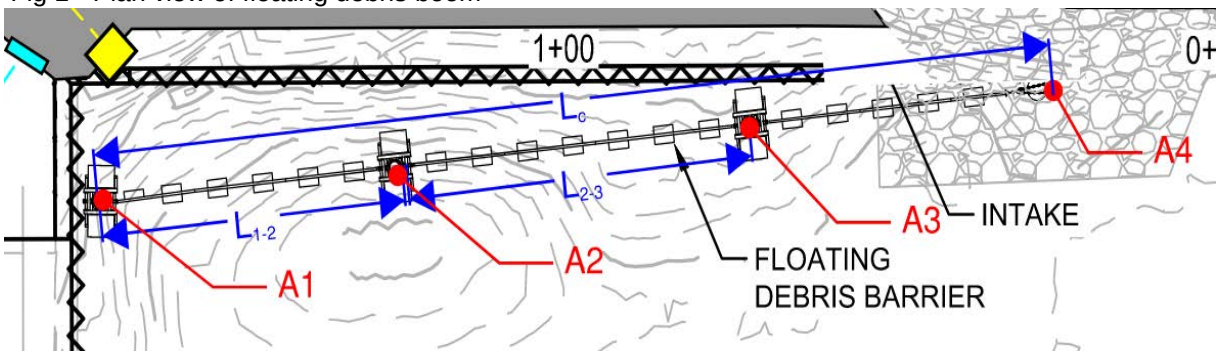


Fig 2 - Section view of floating debris boom

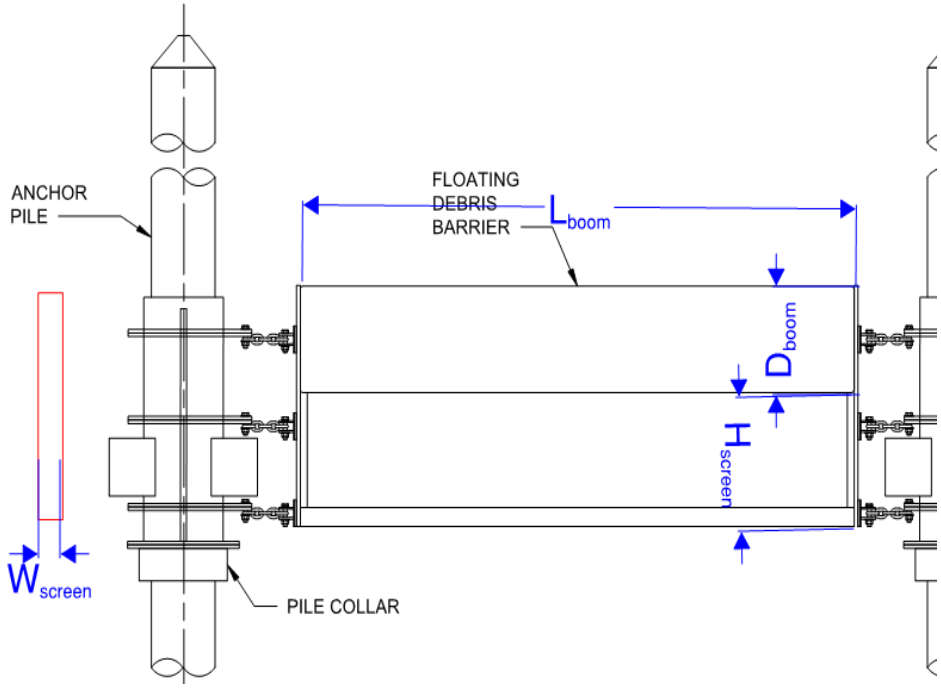


Fig 3 - Keulegan-Carpenter number vs Drag Coefficient

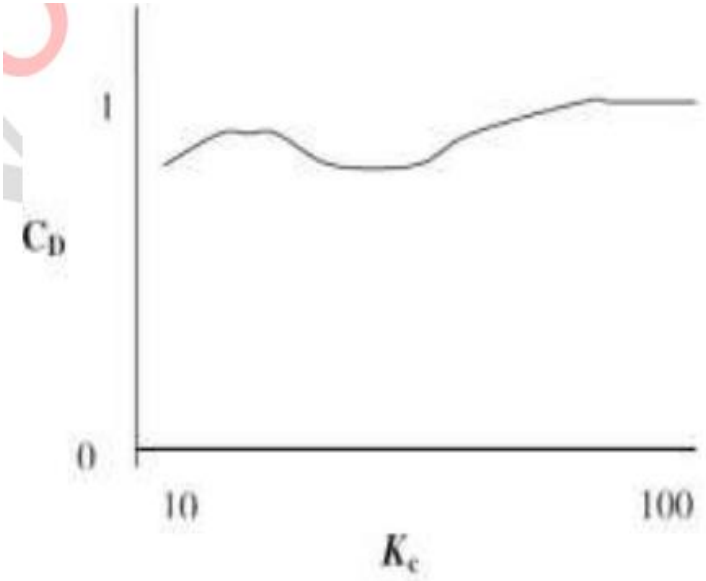


Fig 4 - Keulegan-Carpenter number vs. Wave inertia coefficient

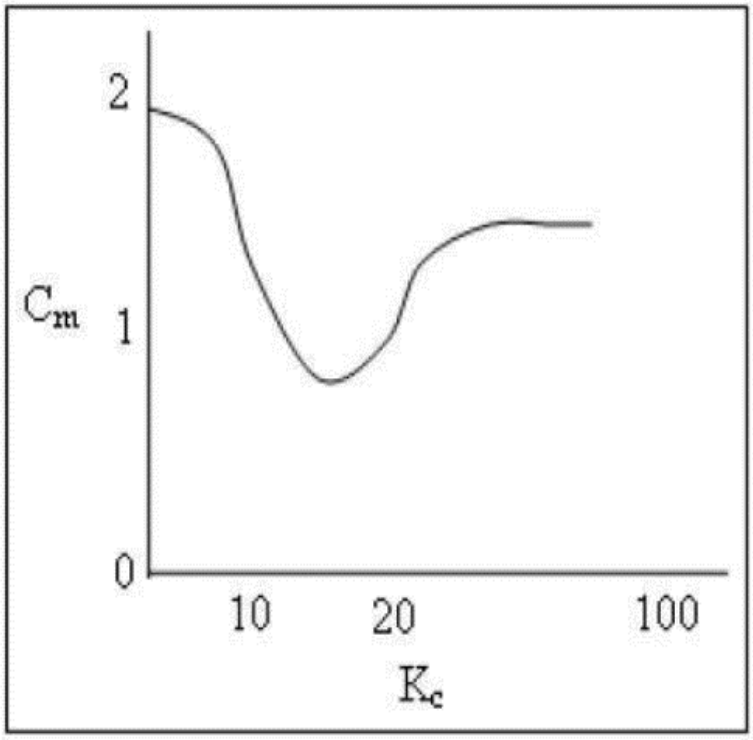
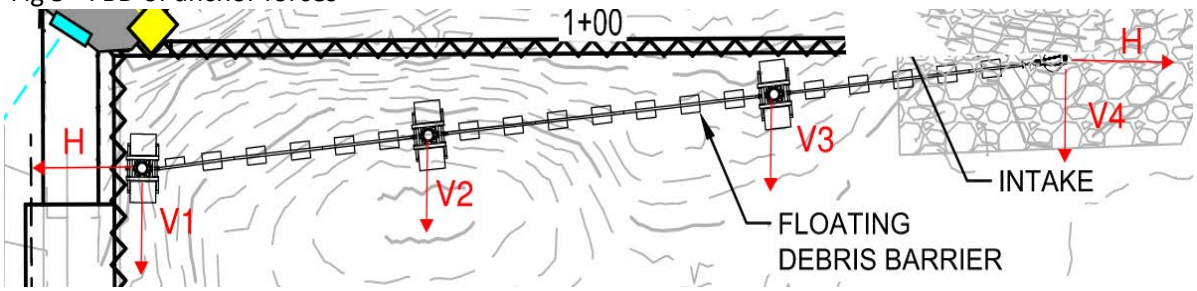


Fig 5 - FBD of anchor forces



CALCULATIONS:
The following calculations are based on the assumptions and references provided above.

Water Current Velocity Drag[Ref 4, eqn 5]

Debris boom submerged area per linear foot

$A_s = H_{screen} + D_{boom} / 2 =$ 4.25 ft

Water Current Velocity Drag Force

$F_C = 0.5 * rho_w * C_d * A_s * (V_{c,max})^2 =$ 455.33 plf

Wind Velocity Drag[Ref 4, eqn 5]

Debris boom exposed area per linear foot $A_e = D_{boom} / 2 =$ 1.25 ft

Wind Velocity Drag Force

$F_A = 0.5 * rho_a * C_w * A_e * (V_{a,max})^2 =$ 34.09 plf

Wave Force

Maximum wave height[Ref 5, Fig II-2-23] $H_w =$ 3 ft

Maximum wave period[Ref 5, Fig II-2-24] $T_w =$ 1.6 s

Maximum wavelength[Ref 5, Fig II-1-9, eqn 3] $L_w = g * T_w^2 / (2 * pi) =$ 13.1 ft

Submergence depth of boom $z = H_{screen} + D_{boom} / 2 =$ 4.25 ft

Wave particle velocity[Ref 5, Fig II-1-9, eqn 5a]

$U = (pi * H_w / T_w) * e^{2*pi*-z/Lw} =$ 0.77 fps

Wave particle acceleration[Ref 5, Fig II-1-9, eqn 6a]

$U_{dot} = (2 * H_w * (pi / T_w)^2) * e^{2*pi*-z/Lw} =$ 3.02 ft/s^2

Keulegan-Carpenter coefficient[Ref 6] $K_c = U * T_w / z =$ 0.29

Drag coefficient[Ref 6, see Fig 3] $C_{d,KC} =$ 0.8

Wave inertia coefficient[Ref 6, see Fig 4] $C_m =$ 2

Wave Force[Ref 6, eqn 1]

$F_W = (0.5 * rho_w * C_{d,KC} * z * U^2) + (rho_w * C_m * (pi * z^2 / 4) * U_{dot}) =$ 168.3 plf

Total unfactored load

Total unfactored load $F_T = F_C + F_A + F_W =$ 657.69 plf

Load at each pile[Fig 5]

Load perpendicular to current[Ref 4, eqn 8a] $H = F_T * L_c / (8 * s) =$ 119.21 kips

Col	1	2	3
Anchor	x *[ft]*	V *[kips]*	T *[kips]*
A1	0	47.68	128.39
A2	45	18.09	18.09
A3	100	-18.09	-18.09
A4	145	-47.68	128.39

Where
Col 1 = Position of anchor along chord length in feet
Col 2 = Load parallel to current at each anchor = $(F_T * L_C / 2) * (1 - 2 * [Col 1] / L_c)$[Ref 4, eqn 8a]
Col 3 = Tension in anchor, H only applies to A1 and A4 = $(H^2 + [Col 2]^2)^{1/2}$

Loads after breakaway

End area of debris boom (now parallel to load)

$A_{end} = (pi * D_{boom}^2 / 4) + (W_{screen} * H_{screen}) =$ 5.16 ft^2

Water current drag force on end of debris boom[Ref 4, eqn 5]

$F_{C,end} = 0.5 * rho_w * C_d * A_{end} * (V_{c,max})^2 =$ 552.69 lbf

Wave drag force/sq.ft on end of debris boom

$F_{W,endSQ} = F_W / z =$ 39.59 psf

Wave drag force on end of debris boom

$F_{W,end} = F_{W,endSQ} * A_{end} =$ 204.25 lbf

Total force on end of debris boom

$F_{T,end} = F_{C,end} + F_{W,end} =$ 756.94 lbf

MECHANICAL CALCULATIONS

FISH LADDER GATE ACTUATOR SIZING

FOR THE

BELLOTA WEIR MODIFICATIONS PROJECT

PREPARED FOR

STOCKTON EAST WATER DISTRICT

CALCULATIONS PREPARED BY

HDR ENGINEERING, INC.

EMMA PAZOFF, E.I.T.

DATE PREPARED: 7/15/21

QC REVIEWER: LARRY PAULS, P.E.

QC REVIEW DATE: 8/2/21

DATE SUBMITTED: 10/06/21

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Exp. MM-DD-YYYY

DESCRIPTION:

This calculation is used to gain preliminary sizing of the fish ladder gate actuator. This gate controls flow through the fish ladder when needed. Reference BLVD_MOGx_3021 in the gate schedule.

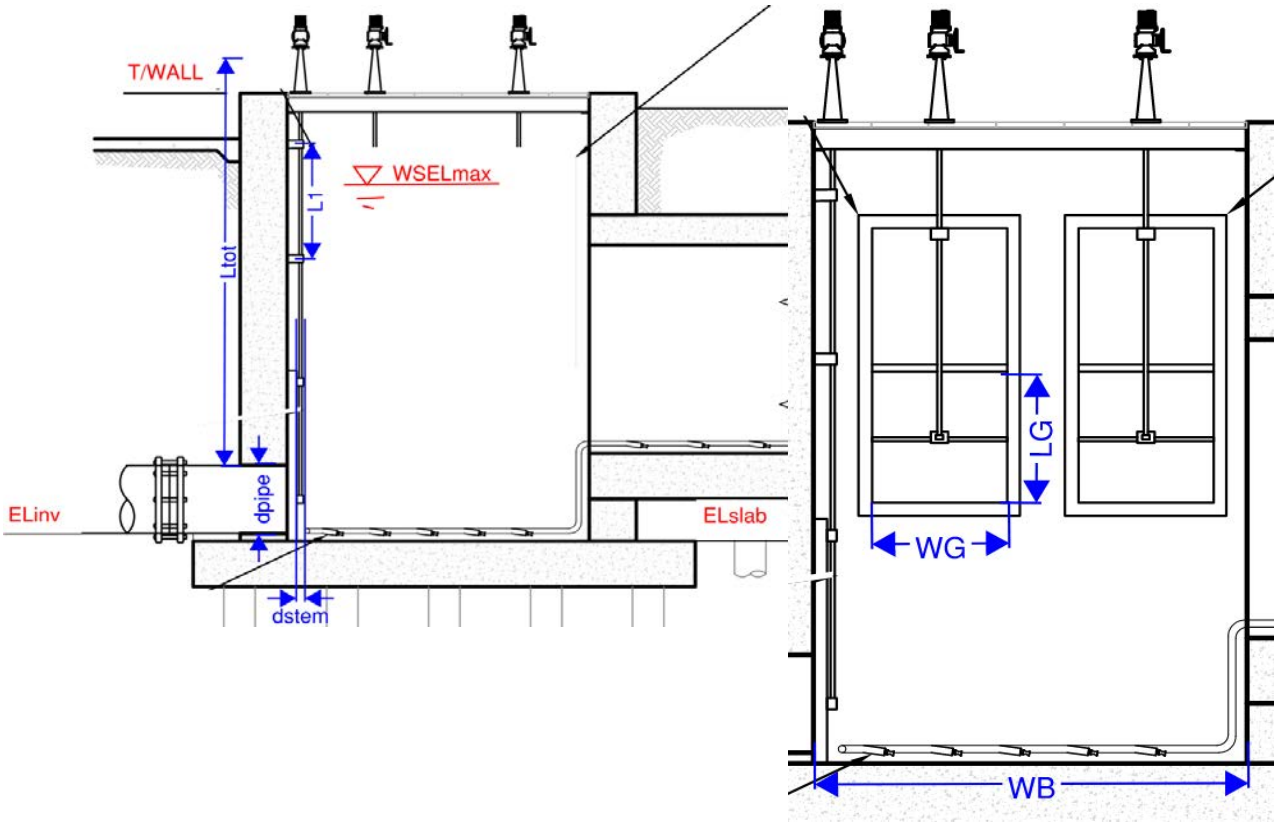
Key: | Input Value | Calculated | *Unit Change*

REFERENCES:

The following are referenced throughout the calculation:

1) 90% Design Package, Bellota Weir Modifications
2) Machinery's Handbook, 26th Ed
3) Mechanical Engineering Reference Manual, 12th Ed
4) Shigley's Machine Design

SCHEMATIC:



ASSUMPTIONS:

Assumptions made during this calculation are as follows:

Description	Variable	Value and Unit	Reference
Loads			
Weight of gate leaf	W_{leaf}	1600 lbs	
Service Factor	SF	1.15	EM 1110-2-2703, 5.2.3.b
Volumetric flow	Q	30 cfs	Ref 1, from fish ladder hydraulics
Geometry			
Gate			
Gate width	W_G	96 in	Ref 1
Gate height	L_G	62 in	Ref 1
Stem diameter	d_{stem}	1.50 in	
Threads per inch	TPI_{stem}	4 threads/in	Ref 2, pg 1795-1798
Thread type		ACME	
Length to first support	L_1	68 in	length of gate + 6 in
Total stem length	L_{tot}	126 in	Ref 1, $=(T_{wall} + 2') - EL_{inv} - L_G$
Intake			
Pipe diameter	$H_{culvert}$	62 in	
Top of wall	T_{wall}	130 ft	
Maximum WSEL	$WSEL_{max}$	127.0 ft	Ref 1, DCR Table 3-6
Wall penetration invert EL	EL_{inv}	116.3 ft	
Wall penetration CL EL	EL_{CL}	118.9 ft	$= EL_{inv} + (H_{culvert}/2)$
Slab elevation	EL_{slab}	116.3 ft	Ref 1
Width of bay	W_{AB}	96 ft	Ref 1
Machinery/Parts			
Rotational speed	RPM_{output}	52 rpm	Limitorque MX-40
Rated torque	T	440 ft-lbs	Limitorque MX-40
Efficiency	η_{motor}	80%	
Material Properties			
Density of water	rho_w	1.94 slugs/ft^3	
Gravity	g	32.2 ft/s^2	
Seal friction coeff	mu	0.3	
Stem material		A276 T316	
	rho_{stem}	0.28 lbs/in^3	
	$F_{y,stem}$	30 ksi	
	$F_{u,stem}$	75 ksi	
	E_{stem}	28000 ksi	
Thrust/Lift Nut		B584 CA863	
Material	$F_{y,nut}$	60 ksi	
	$F_{u,nut}$	110 ksi	
Stem/nut friction	f	0.38	

Allowable Stresses

	Tens/Comp	Bearing	Shear	
Stem	13.5	27.0	12.0	ksi
Thrust Nut	27.0	54.0	24.0	ksi

showing allowable stresses for tension, shear, bending and bearing. Note, allowable bearing is for when no movement is intended between the two components

TENSION	$0.45S_y \le \sigma_{all} \le 0.60S_y$
SHEAR	$\tau_{all} = 0.40S_y$
BENDING	$0.60S_y \le \sigma_{all} \le 0.75S_y$
BEARING	$\sigma_{all} = 0.90S_y$

CALCULATIONS:

The following calculations are based on the assumptions and references provided above.

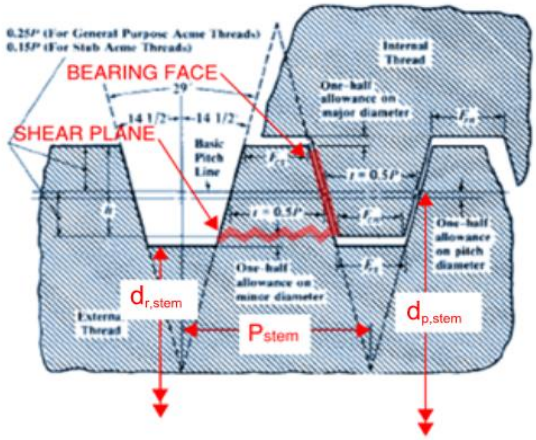
Max operating head on the gate	$h_G = WSEL_{max} - EL_{CL}$ =	8.1 ft
Hydrostatic pressure on gate	$P_{hys} = h_G/2.31$ =	3.5 psi
Area of water acting on gate	$A_G = W_G * L_G$ =	5952 in^2
Force acting on gate due to hydrostatic pressure	$F_{hys} = P_{hys} * A_G$ =	20914 lbs
Frictional resistance force	$F_{FR} = F_{hys} * mu$ =	6274 lbs
Weight of stem	$W_{stem} = pi*(d_{stem}/2)^2*L_{tot}*rho_{stem}$ =	62.5 lbs
Cross sectional area of channel	$A_{channel} = W_{AB}*(WSEL_{max}-EL_{slab})$ =	1027 ft^2
Upstream channel velocity	$v_{US} = Q/A_{channel}$ =	0.0 ft/s
Dynamic force on gate	$F_{DG} = rho_w * A_G * (v_{US})^2$ =	0 lbs
Vertical force required to overcome F_{DG}	$F_{DGV} = F_{DG} * mu$ =	0.02 lbs

Weight actuator needs to be sized for

Opening (tension)	$F_{act} = SF*(W_{leaf} + F_{FR} + W_{stem} + F_{DGV})$ =	9127 lbs
Closing (compression)	$F_{act} = SF*(F_{FR} + F_{DGV})$ =	7215 lbs

Stem[Ref 2, ACME threads]

Pitch	$P_{stem} = 1/TPI_{stem}$ =	0.25 in
Lead	$L_{stem} = 1/TPI_{stem}$ =	0.25 in
Pitch diameter	$d_{p,stem} = d_{stem} - (0.5*P_{stem})$ =	1.375 in
Root diameter	$d_{r,stem} = d_{stem} - P_{stem}$ =	1.25 in
Nut length	$L_{nut} = 2*d_{stem}$ =	3 in
No. threads engaged	$N_{thr} = L_{nut}*TPI_{stem}$ =	12 threads
Thread half-angle	Θ =	14.5 degrees



Stresses on Stem

Tension

$\sigma_{T,stem} = F_{act}/(pi*(d_{r,stem}/2)^2)$ =	7437 psi
$\sigma_{T,stem}$ =	*7.44* ksi
Check: Is $\sigma_{T,stem} < \sigma_{T,allowable}$?	OK

Bearing

Bearing area of single thread

$A_{stem,B1} = d_{p,stem}*pi*0.5*P_{stem}*\cos(14.5)$ =	0.523 in^2

Total bearing area of engaged threads[Ref 4]

$A_{stem,B} = 3\ threads*A_{stem,B1}$ =	1.568 in^2
Bearing stress $\sigma_{B,stem} = F_{act}/A_{stem,B}$ =	5820 psi
$\sigma_{B,stem}$ =	*5.82* ksi
Check: Is $\sigma_{B,stem} < \sigma_{B,allowable}$?	OK

Shear

Shear area per thread

$A_{stem,S1} = d_{p,stem}*pi*0.5*P_{stem}$ = 0.540 in^2

Total shear area[Ref 4]

$A_{stem,S} = 3\ threads*A_{stem,S1}$ = 1.620 in^2

Shear stress

$\tau_{stem} = F_{act}/A_{stem,S}$ = 5634 psi

τ_{stem} = *5.63* ksi

Check: Is $\tau_{stem} < \tau_{stem,allowable}$? OK

Buckling[Ref 3, Ch 51, Sec 1]

Radius of gyration $k = d_{p,stem}/4$ = 0.344 in

Min area of stem $A = pi*(d_{p,stem}/2)^2$ = 1.485 in^2

Slenderness criterion $SR_{crit} = (2*pi^2*E_{stem}/F_{y,stem})^{1/2}$ = 135.7

Slenderness ratio $SR = L_1/k$ = 197.8

Check: SR_{crit} < SR, so Long Column

Euler stress, max allowable compressive stress

$\sigma_e = pi^2*E_{stem}/(SR^2)$ = 7.0620 ksi

Max allowable compressive force on stem

$F_{max} = \sigma_e*A$ = 10486.3 lbs

Check: Is $F_C < F_{max}$? OK

Time for one operation (fully closed to fully open, or vice versa)

Height traveled per one revolution $L_{one\ rev} = P_{stem}$ = 0.25 in/rev

No. revolutions per one operation $N_{rev} = L_1/L_{one\ rev}$ = 272 rev/op

Time for one operation $t_{op} = N_{rev}/RPM_{motor}$ = 5.2 minutes

Torque required to raise the load[Ref 3, Eq'n 52.121]

$$T_{req,raise} = \frac{F_{act} * d_{p,stem}}{2} * \frac{\tan(\theta) + f}{1 - f * \tan(\theta)} =$$ 4444 in-lbs

$T_{design,raise}$ = *370.3 ft-lbs*

Check: $T_{design,raise} < T_{MX40}$? OK

Required actuator horsepower[Ref 3, Eq'n 51.50]

$HP_{motor} = (T_{design,raise}*RPM_{motor}/63025)/\eta_{motor})$ = 4.58 HP

Standard motor size $HP_{motor,std}$ = 5 HP

Additional motor capacity $R_{extra\ cap} = (HP_{motor,std}/HP_{motor}) - 1$ = 0.09

Snippet from Limitorque Cutsheet

Output Speed (RPM)		MX-05		MX-10		MX-20		MX-40		MX-85		MX-140		MX-150	
		Rated Output Torque													
60 Hz	50 Hz	ft-lb	N m	ft-lb	N m	ft-lb	N m	ft-lb	N m	ft-lb	N m	ft-lb	N m	ft-lb	N m
18	15	55	75	125	170	225	305	440	597	n/a	n/a	n/a	n/a	n/a	n/a
26	22	55	75	125	170	225	305	440	597	850	1,153	1,500	2,036	n/a	n/a
40	33	55	75	125	170	225	305	440	597	1,225	1,662	1,700	2,307	n/a	n/a
52	43	55	75	125	170	225	305	440	597	1,150	1,561	1,600	2,171	n/a	n/a
77	65	48	65	107	145	178	241	345	468	850	1,153	1,200	1,628	n/a	n/a
100 131[1]	84 110[1]	39	53	89	121	148	201	286	388	600	814	815	1,105	1,500	2,036
155 170[1]	127 143[1]	41	56	89	121	140	190	260	353	450	611	650	882	1,150	1,561
200	165	34	46	73	99	114	155	210	285	n/a	n/a	n/a	n/a	n/a	n/a

Note 1: MX-85, MX-140 and MX-150

MECHANICAL CALCULATIONS

FOREBAY SLUICE GATE ACTUATOR SIZING

FOR THE

BELLOTA WEIR MODIFICATIONS PROJECT

PREPARED FOR

STOCKTON EAST WATER DISTRICT

CALCULATIONS PREPARED BY

HDR ENGINEERING, INC.

EMMA PAZOFF, E.I.T.

DATE PREPARED: 7/15/21

QC REVIEWER: LARRY PAULS, P.E.

QC REVIEW DATE: 8/2/21

DATE SUBMITTED: 10/06/21

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Exp. MM-DD-YYYY

DESCRIPTION:

This calculation is used to gain preliminary equipment sizing of the sediment sluice gate actuator located in the forebay. This gate controls the flow from the forebay into the downstream end of the intake channel. Reference BLVD_MOGx_1414 in gate schedule.

Key:

Input Value	Calculated

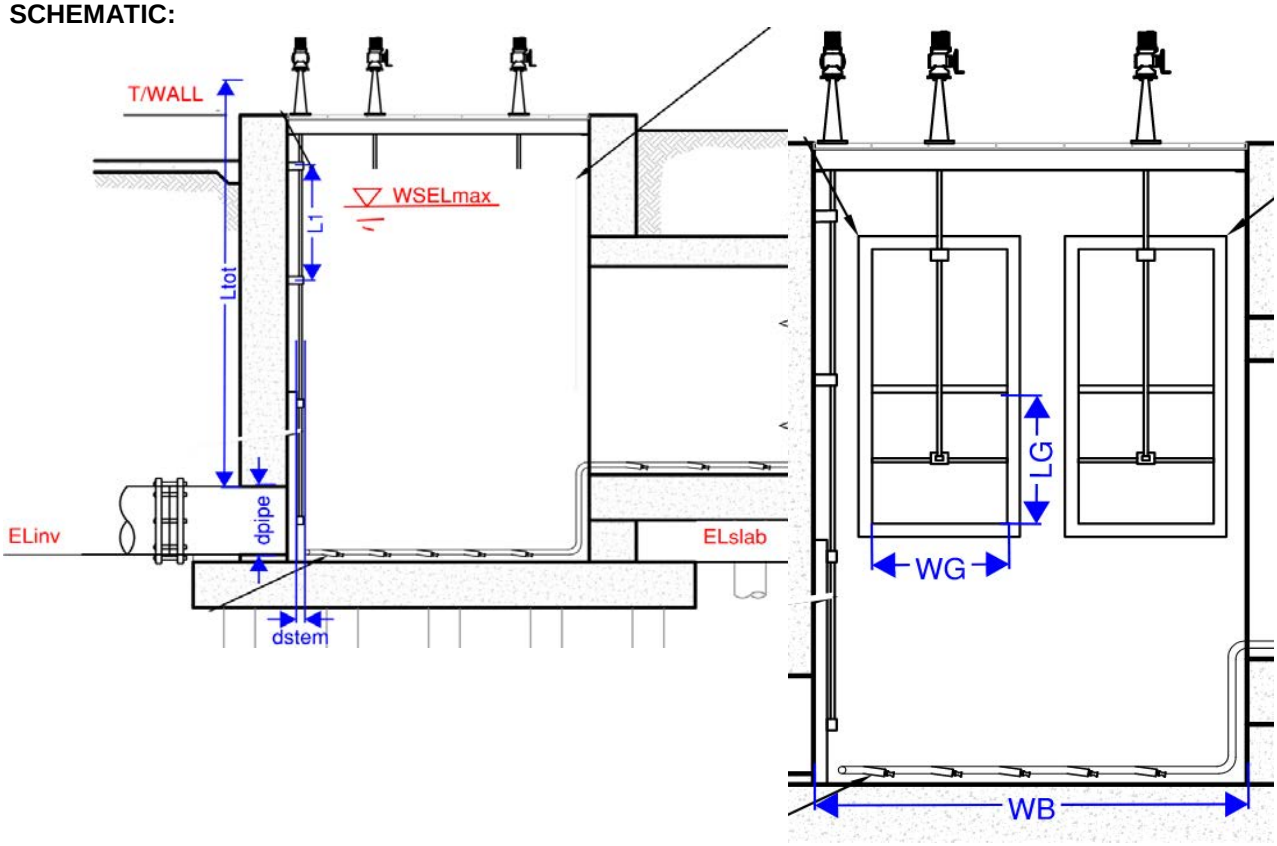
Unit Change

REFERENCES:

The following are referenced throughout the calculation:

1) 90% Design Package, Bellota Weir Modifications
2) Machinery's Handbook, 26th Ed
3) Mechanical Engineering Reference Manual, 12th Ed
4) Shigley's Machine Design

SCHEMATIC:



ASSUMPTIONS:

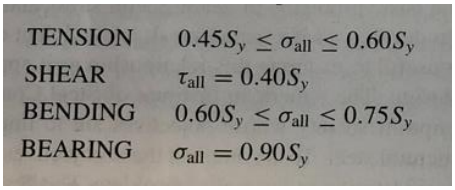
Assumptions made during this calculation are as follows:

Description	Variable	Value and Unit	Reference
Loads			
Weight of gate leaf	W_{leaf}	750 lbs	assume ~30 lb/sq.ft.
Service Factor	SF	1.15	EM 1110-2-2703, 5.2.3.b
Volumetric flow	Q	420 cfs	Ref 1, Intake Sluice Pipe Sizing Calc
Geometry			
Gate			
Gate width	W_G	60 in	Ref 1
Gate height	L_G	60 in	Ref 1
Stem diameter	d_{stem}	1.50 in	
Threads per inch	TPI_{stem}	4 threads/in	Ref 2, pg 1795-1798
Thread type		ACME	
Length to first support	L_1	66 in	length of gate + 6 in
Total stem length	L_{tot}	168 in	Ref 1, $=(T_{wall} + 2') - EL_{inv} - L_G$
Intake			
Culvert height	$H_{culvert}$	60 in	
Top of wall	T_{wall}	127 ft	
Maximum WSEL	$WSEL_{max}$	127.0 ft	Ref 1, DCR Table 3-6
Wall penetration invert EL	EL_{inv}	110.0 ft	
Wall penetration CL EL	EL_{CL}	112.5 ft	$= EL_{inv} + (H_{culvert}/2)$
Slab elevation	EL_{slab}	106.0 ft	Ref 1
Width of bay	W_{AB}	11 ft	Ref 1
Machinery/Parts			
Rotational speed	RPM_{output}	52 rpm	Limitorque MX-40
Rated torque	T	440 ft-lbs	Limitorque MX-40
Efficiency	η_{motor}	80%	
Material Properties			
Density of water	rho_w	1.94 $slugs/ft^3$	
Gravity	g	32.2 ft/s^2	
Seal friction coeff	mu	0.3	
Stem material		A276 T316	
	rho_{stem}	0.28 lbs/in^3	
	$F_{y,stem}$	30 ksi	
	$F_{u,stem}$	75 ksi	
	E_{stem}	28000 ksi	
Thrust/Lift Nut		B584 CA863	
Material	$F_{y,nut}$	60 ksi	
	$F_{u,nut}$	110 ksi	
Stem/nut friction	f	0.38	

Allowable Stresses

	Tens/Comp	Bearing	Shear	
Stem	13.5	27.0	12.0	ksi
Thrust Nut	27.0	54.0	24.0	ksi

Values from allowable stresses above come from Ref 4. Snippet from Ref 4 showing allowable stresses for tension, shear, bending and bearing. Note, allowable bearing is for when no movement is intended between the two components



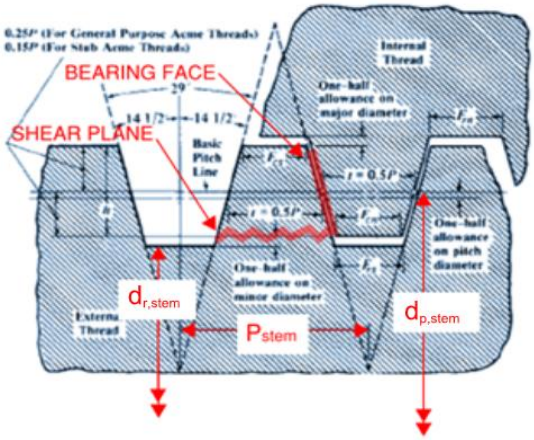
TENSION	$0.45S_y \le \sigma_{all} \le 0.60S_y$
SHEAR	$\tau_{all} = 0.40S_y$
BENDING	$0.60S_y \le \sigma_{all} \le 0.75S_y$
BEARING	$\sigma_{all} = 0.90S_y$

CALCULATIONS:
The following calculations are based on the assumptions and references provided above.

Max operating head on the gate	$h_G = WSEL_{max} - EL_{CL}$ =	14.5 ft
Hydrostatic pressure on gate	$P_{hys} = h_G/2.31$ =	6.3 psi
Area of water acting on gate	$A_G = W_G * L_G$ =	3600 in^2
Force acting on gate due to hydrostatic pressure	$F_{hys} = P_{hys} * A_G$ =	22597 lbs
Frictional resistance force	$F_{FR} = F_{hys} * mu$ =	6779 lbs
Weight of stem	$W_{stem} = pi*(d_{stem}/2)^2*L_{tot}*rho_{stem}$ =	83.1 lbs
Cross sectional area of channel	$A_{channel} = W_{AB}*(WSEL_{max}-EL_{slab})$ =	231 ft^2
Upstream channel velocity	$v_{US} = Q/A_{channel}$ =	1.8 ft/s
Dynamic force on gate	$F_{DG} = rho_w * A_G * (v_{US})^2$ =	160 lbs
Vertical force required to overcome F_{DG}	$F_{DGV} = F_{DG} * mu$ =	44.94 lbs
Weight actuator needs to be sized for		
Opening (tension)	$F_{act} = SF*(W_{leaf} + F_{FR} + W_{stem} + F_{DGV})$ =	8806 lbs
Closing (compression)	$F_{act} = SF*(F_{FR} + F_{DGV})$ =	7848 lbs

Stem [Ref 2, ACME threads]

Pitch	$P_{stem} = 1/TPI_{stem}$ =	0.25 in
Lead	$L_{stem} = 1/TPI_{stem}$ =	0.25 in
Pitch diameter	$d_{p,stem} = d_{stem} - (0.5*P_{stem})$ =	1.375 in
Root diameter	$d_{r,stem} = d_{stem} - P_{stem}$ =	1.25 in
Nut length	$L_{nut} = 2*d_{stem}$ =	3 in
No. threads engaged	$N_{thr} = L_{nut}*TPI_{stem}$ =	12 threads
Thread half-angle	Θ =	14.5 degrees



Stresses on Stem

Tension

	$\sigma_{T,stem} = F_{act}/(pi*(d_{r,stem}/2)^2)$ =	7176 psi
	$\sigma_{T,stem}$ =	*7.18* ksi
	Check: Is $\sigma_{T,stem} < \sigma_{T,allowable}$?	OK

Bearing

Bearing area of single thread

	$A_{stem,B1} = d_{p,stem}*pi*0.5*P_{stem}*\cos(14.5)$ =	0.523 in^2

Total bearing area of engaged threads [Ref 4]

	$A_{stem,B} = 3\ threads*A_{stem,B1}$ =	1.568 in^2
Bearing stress	$\sigma_{B,stem} = F_{act}/A_{stem,B}$ =	5615 psi
	$\sigma_{B,stem}$ =	*5.61* ksi
	Check: Is $\sigma_{B,stem} < \sigma_{B,allowable}$?	OK

Shear

Shear area per thread

$A_{stem,S1} = d_{p,stem}*pi*0.5*P_{stem}$ = 0.540 in^2

Total shear area[Ref 4]

$A_{stem,S} = 3\ threads*A_{stem,S1}$ = 1.620 in^2

Shear stress

$\tau_{stem} = F_{act}/A_{stem,S}$ = 5436 psi

τ_{stem} = *5.44* ksi

Check: Is $\tau_{stem} < \tau_{stem,allowable}$? OK

Buckling[Ref 3, Ch 51, Sec 1]

Radius of gyration $k = d_{p,stem}/4$ = 0.344 in

Min area of stem $A = pi*(d_{p,stem}/2)^2$ = 1.485 in^2

Slenderness criterion $SR_{crit} = (2*pi^2*E_{stem}/F_{y,stem})^{1/2}$ = 135.7

Slenderness ratio $SR = L_1/k$ = 192.0

Check: SR_{crit} < SR, so Long Column

Euler stress, max allowable compressive stress

$\sigma_e = pi^2*E_{stem}/(SR^2)$ = 7.4964 ksi

Max allowable compressive force on stem

$F_{max} = \sigma_e*A$ = 11131.4 lbs

Check: Is $F_C < F_{max}$? OK

Time for one operation (fully closed to fully open, or vice versa)

Height traveled per one revolution $L_{one\ rev} = P_{stem}$ = 0.25 in/rev

No. revolutions per one operation $N_{rev} = L_1/L_{one\ rev}$ = 264 rev/op

Time for one operation $t_{op} = N_{rev}/RPM_{motor}$ = 5.1 minutes

Torque required to raise the load[Ref 3, Eq'n 52.121]

$$T_{req,raise} = \frac{F_{act} * d_{p,stem}}{2} * \frac{\tan(\theta) + f}{1 - f * \tan(\theta)} =$$ 4288 in-lbs

$T_{design,raise}$ = *357.3 ft-lbs*

Check: $T_{design,raise} < T_{MX40}$? OK

Required actuator horsepower[Ref 3, Eq'n 51.50]

$HP_{motor} = (T_{design,raise}*RPM_{motor}/63025)/\eta_{motor})$ = 4.42 HP

Standard motor size $HP_{motor,std}$ = 5 HP

Additional motor capacity $R_{extra\ cap} = (HP_{motor,std}/HP_{motor}) - 1$ = 0.13

Snippet from Limitorque Cutsheet

Output Speed (RPM)		MX-05		MX-10		MX-20		MX-40		MX-85		MX-140		MX-150	
		Rated Output Torque													
60 Hz	50 Hz	ft-lb	N m	ft-lb	N m	ft-lb	N m	ft-lb	N m	ft-lb	N m	ft-lb	N m	ft-lb	N m
18	15	55	75	125	170	225	305	440	597	n/a	n/a	n/a	n/a	n/a	n/a
26	22	55	75	125	170	225	305	440	597	850	1,153	1,500	2,036	n/a	n/a
40	33	55	75	125	170	225	305	440	597	1,225	1,662	1,700	2,307	n/a	n/a
52	43	55	75	125	170	225	305	440	597	1,150	1,561	1,600	2,171	n/a	n/a
77	65	48	65	107	145	178	241	345	468	850	1,153	1,200	1,628	n/a	n/a
100 131[1]	84 110[1]	39	53	89	121	148	201	286	388	600	814	815	1,105	1,500	2,036
155 170[1]	127 143[1]	41	56	89	121	140	190	260	353	450	611	650	882	1,150	1,561
200	165	34	46	73	99	114	155	210	285	n/a	n/a	n/a	n/a	n/a	n/a

Note 1: MX-85, MX-140 and MX-150

MECHANICAL CALCULATIONS

INTAKE MANIFOLD SLUICE GATE ACTUATOR SIZING

FOR THE

BELLOTA WEIR MODIFICATIONS PROJECT

PREPARED FOR

STOCKTON EAST WATER DISTRICT

CALCULATIONS PREPARED BY

HDR ENGINEERING, INC.

EMMA PAZOFF, E.I.T.

DATE PREPARED: 7/15/21

QC REVIEWER: LARRY PAULS, P.E.

QC REVIEW DATE: 8/2/21

DATE SUBMITTED: 10/06/21

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Exp. MM-DD-YYYY

DESCRIPTION:

This calculation is used to gain preliminary sizing of the sediment sluice gate actuator located at the end of the intake channel. This gate controls the flow of sediment rich water into the sediment sluice pipe that discharges that water several hundred feet downstream. Reference BLVD_MOGx_2105 in the gate schedule.

Key:

Input Value	Calculated

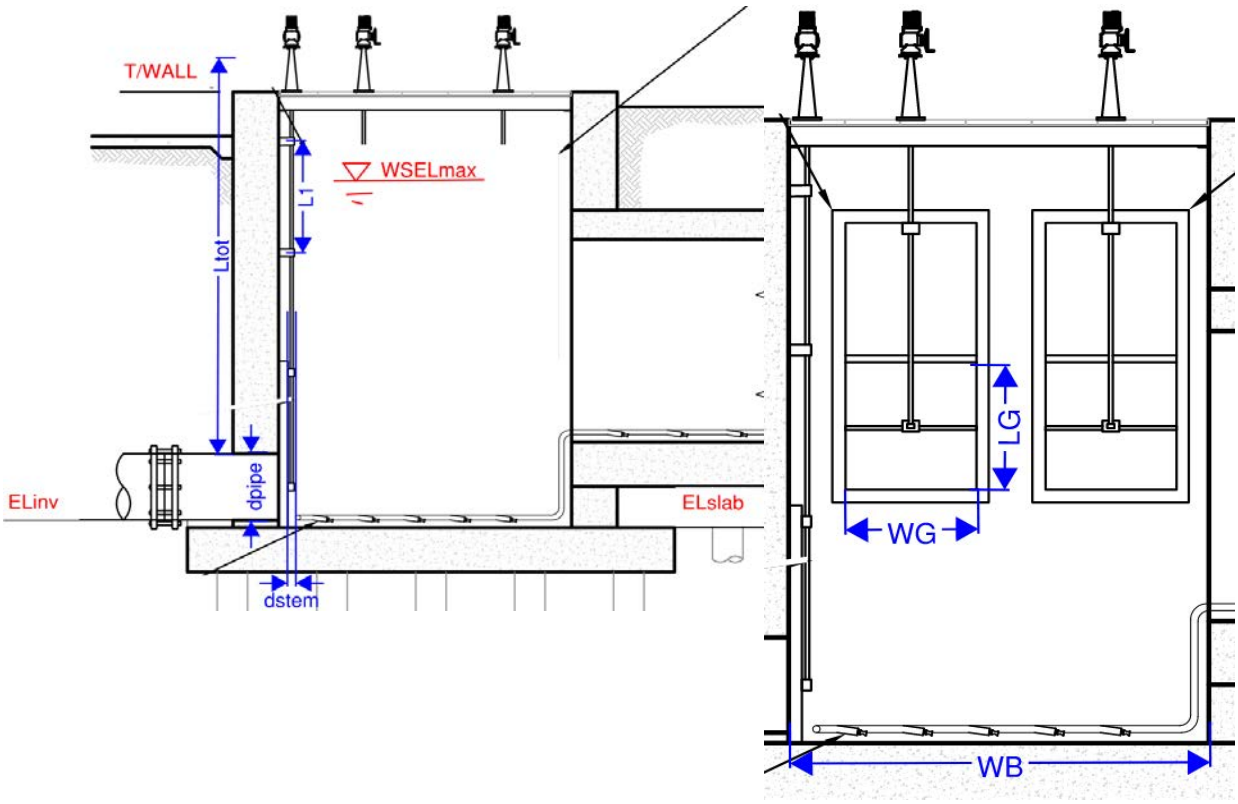
Unit Change

REFERENCES:

The following are referenced throughout the calculation:

1) 90% Design Package, Bellota Weir Modifications
2) Machinery's Handbook, 26th Ed
3) Mechanical Engineering Reference Manual, 12th Ed
4) Shigley's Machine Design

SCHEMATIC:



ASSUMPTIONS:

Assumptions made during this calculation are as follows:

Description	Variable	Value and Unit	Reference
Loads			
Weight of gate leaf	W_{leaf}	600 lbs	Estimated
Service Factor	SF	1.15	EM 1110-2-2703, 5.2.3.b
Volumetric flow	Q	420 cfs	Ref 1, Intake Sluice Pipe Sizing Calc
Geometry			
Gate			
Gate width	W_G	54 in	Ref 1
Gate height	L_G	54 in	Ref 1
Stem diameter	d_{stem}	1.50 in	
Threads per inch	TPI_{stem}	4 threads/in	Ref 2, pg 1795-1798
Thread type		ACME	
Length to first support	L_1	60 in	length of gate + 6 in
Total stem length	L_{tot}	252 in	Ref 1, $=(T_{wall} + 2') - EL_{inv} - L_G$
Intake			
Culvert height	d_{pipe}	54 in	
Top of wall	T_{wall}	130 ft	
Maximum WSEL	$WSEL_{max}$	127.0 ft	Ref 1, DCR Table 3-6
Wall penetration invert EL	EL_{inv}	106.5 ft	
Wall penetration CL EL	EL_{CL}	108.8 ft	$= EL_{inv} + (d_{pipe}/2)$
Slab elevation	EL_{slab}	106.0 ft	Ref 1
Width of bay	W_{AB}	13.00 ft	Ref 1
Machinery/Parts			
Rotational speed	RPM_{motor}	77 rpm	Limitorque MX-40
Rated torque, T	T	440 ft-lbs	Limitorque MX-40
Efficiency	η_{motor}	80%	
Material Properties			
Density of water	rho_w	1.94 slugs/ft^3	
Gravity	g	32.2 ft/s^2	
Seal friction coeff	mu	0.3	
Stem material		A276 T316	
	rho_{stem}	0.28 lbs/in^3	
	$F_{y,stem}$	30 ksi	
	$F_{u,stem}$	75 ksi	
	E_{stem}	28000 ksi	
Thrust/Lift Nut		B584 CA863	
Material	$F_{y,nut}$	60 ksi	
	$F_{u,nut}$	110 ksi	
Stem/nut friction	f	0.38	

Allowable Stresses

	Tens/Comp	Bearing	Shear	
Stem	13.5	27.0	12.0	ksi
Thrust Nut	27.0	54.0	24.0	ksi

showing allowable stresses for tension, shear, bending and bearing. Note, allowable bearing is for when no movement is intended between the two components

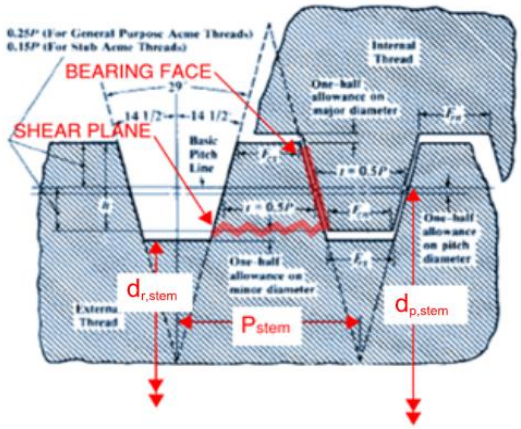
TENSION	$0.45S_y \le \sigma_{all} \le 0.60S_y$
SHEAR	$\tau_{all} = 0.40S_y$
BENDING	$0.60S_y \le \sigma_{all} \le 0.75S_y$
BEARING	$\sigma_{all} = 0.90S_y$

CALCULATIONS:
The following calculations are based on the assumptions and references provided above.

Max operating head on the gate	$h_G = WSEL_{max} - EL_{CL} =$	18.3 ft
Hydrostatic pressure on gate	$P_{hys} = h_G/2.31 =$	7.9 psi
Area of water acting on gate	$A_G = W_G*L_G =$	2916 in^2
Force acting on gate due to hydrostatic pressure	$F_{hys} = P_{hys}*A_G =$	23037.66 lbs
Frictional resistance force	$F_{FR} = F_{hys}*mu =$	6911.299 lbs
Weight of stem	$W_{stem} = pi*(d_{stem}/2)^2*L_{tot}*rho_{stem} =$	124.7 lbs
Cross sectional area of channel	$A_{channel} = W_{AB}*(WSEL_{max}-EL_{slab}) =$	273 ft^2
Upstream channel velocity	$v_{US} = Q/A_{channel} =$	1.5 ft/s
Horizontal dynamic force on gate	$F_{DG} = rho_w*A_G*(v_{US})^2 =$	93 lbs
Vertical force required to overcome F_{DG}	$F_{DGV} = F_{DG}*mu =$	26 lbs
Weight actuator needs to be sized for		
Opening (tension)	$F_{act} = SF*(W_{leaf} + F_{FR} + W_{stem} + F_{DGV}) =$	8811 lbs
Closing (compression)	$F_{act} = SF*(F_{FR} + F_{DGV}) =$	7978 lbs

Stem[Ref 2, ACME threads]

Pitch	$P_{stem} = 1/TPI_{stem}$ =	0.25 in
Lead	$L_{stem} = 1/TPI_{stem}$ =	0.25 in
Pitch diameter	$d_{p,stem} = d_{stem} - (0.5*P_{stem})$ =	1.375 in
Root diameter	$d_{r,stem} = d_{stem} - P_{stem}$ =	1.25 in
Nut length	$L_{nut} = 2*d_{stem}$ =	3 in
No. threads engaged	$N_{thr} = L_{nut}*TPI_{stem}$ =	12 threads
Thread half-angle	Θ =	14.5 degrees



Stresses on Stem

Tension

$\sigma_{T,stem} = F_{act}/(pi*(d_{r,stem}/2)^2)$ =	7180 psi
$\sigma_{T,stem}$ =	*7.18* ksi
Check: Is $\sigma_{T,stem} < \sigma_{T,allowable}$?	OK

Bearing

Bearing area of single thread

$A_{stem,B1} = d_{p,stem}*pi*0.5*P_{stem}*\cos(14.5)$ =	0.523 in^2

Total bearing area of engaged threads[Ref 4]

$A_{stem,B} = 3\ threads*A_{stem,B1}$ =	1.568 in^2
Bearing stress $\sigma_{B,stem} = F_{act}/A_{stem,B}$ =	5618 psi
$\sigma_{B,stem}$ =	*5.62* ksi
Check: Is $\sigma_{B,stem} < \sigma_{B,allowable}$?	OK

Shear

Shear area per thread

$A_{stem,S1} = d_{p,stem}*pi*0.5*P_{stem}$ = 0.540 in^2

Total shear area[Ref 4] $A_{stem,S} = 3\ threads*A_{stem,S1}$ = 1.620 in^2

Shear stress $\tau_{stem} = F_{act}/A_{stem,S}$ = 5439 psi

τ_{stem} = *5.44* ksi

Check: Is $\tau_{stem} < \tau_{stem,allowable}$? OK

Buckling[Ref 3, Ch 51, Sec 1]

Radius of gyration $k = d_{p,stem}/4$ = 0.344 in

Min area of stem $A = pi*(d_{p,stem}/2)^2$ = 1.485 in^2

Slenderness criterion $SR_{crit} = (2*pi^2*E_{stem}/F_{y,stem})^{1/2}$ = 135.7

Slenderness ratio $SR = L_1/k$ = 174.5

Check: SR_{crit} < SR, so Long Column

Euler stress, max allowable compressive stress

$\sigma_e = pi^2*E_{stem}/(SR^2)$ = 9.07 ksi

Max allowable compressive force on stem

$F_{max} = \sigma_e*A$ = 13469.0 lbs

Check: Is $F_C < F_{max}$? OK

Time for one operation (fully closed to fully open, or vice versa)

Height traveled per one revolution $L_{one\ rev} = P_{stem}$ = 0.25 in/rev

No. revolutions per one operation $N_{rev} = L_1/L_{one\ rev}$ = 240 rev/op

Time for one operation $t_{op} = N_{rev}/RPM_{motor}$ = 3.1 minutes

Torque required to raise the load[Ref 3, Eq'n 52.121]

$$T_{req,raise} = \frac{F_{act} * d_{p,stem}}{2} * \frac{\tan(\theta) + f}{1 - f * \tan(\theta)} =$$ 4290 in-lbs

$T_{design,raise}$ = *357.5 ft-lbs*

Check: $T_{design,raise} < T_{MX40}$? OK

Required actuator horsepower[Ref 3, Eq'n 51.50]

$HP_{motor} = (T_{design,raise}*RPM_{motor}/63025)/\eta_{motor})$ = 6.55 HP

Standard motor size $HP_{motor,std}$ = 7.5 HP

Additional motor capacity $R_{extra\ cap} = (HP_{motor,std}/HP_{motor}) - 1$ = 0.14

Snippet from Limitorque Cutsheet

Output Speed (RPM)		MX-05		MX-10		MX-20		MX-40		MX-85		MX-140		MX-150	
		Rated Output Torque													
60 Hz	50 Hz	ft-lb	N m	ft-lb	N m	ft-lb	N m	ft-lb	N m	ft-lb	N m	ft-lb	N m	ft-lb	N m
18	15	55	75	125	170	225	305	440	597	n/a	n/a	n/a	n/a	n/a	n/a
26	22	55	75	125	170	225	305	440	597	850	1,153	1,500	2,036	n/a	n/a
40	33	55	75	125	170	225	305	440	597	1,225	1,662	1,700	2,307	n/a	n/a
52	43	55	75	125	170	225	305	440	597	1,150	1,561	1,600	2,171	n/a	n/a
77	65	48	65	107	145	178	241	345	468	850	1,153	1,200	1,628	n/a	n/a
100 131[1]	84 110[1]	39	53	89	121	148	201	286	388	600	814	815	1,105	1,500	2,036
155 170[1]	127 143[1]	41	56	89	121	140	190	260	353	450	611	650	882	1,150	1,561
200	165	34	46	73	99	114	155	210	285	n/a	n/a	n/a	n/a	n/a	n/a

Note 1: MX-85, MX-140 and MX-150

MECHANICAL CALCULATIONS

INTAKE WEIR

FOR THE

BELLOTA WEIR MODIFICATIONS PROJECT

PREPARED FOR

STOCKTON EAST WATER DISTRICT

CALCULATIONS PREPARED BY

HDR ENGINEERING, INC.

EMMA PAZOFF, E.I.T.

DATE PREPARED: 7/15/21

QC REVIEWER: LARRY PAULS, P.E.

QC REVIEW DATE: 8/2/21

DATE SUBMITTED: 10/06/21

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DESCRIPTION:

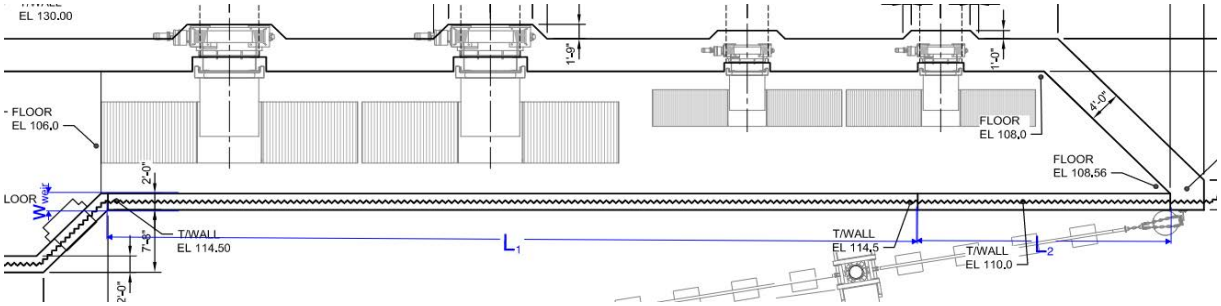
This calculation is used to determine the weir length leading into the intake structure.

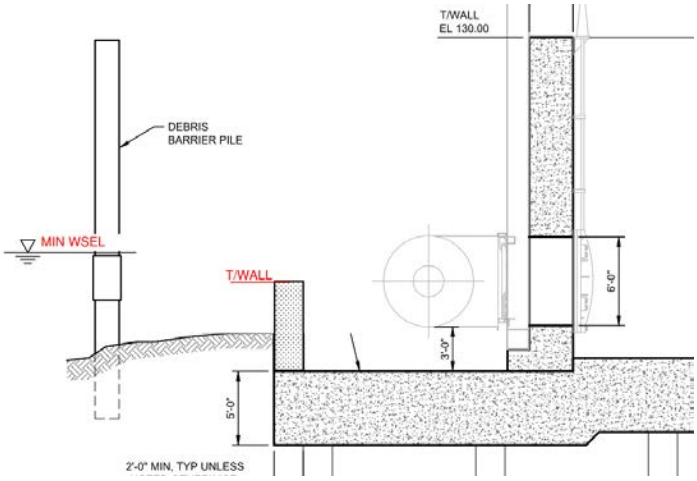
REFERENCES:

The following are referenced throughout the calculation:

1) 90% Plan Set, Bellota Weir Modifications
2) Civil Engineering Reference Manual

SCHEMATICS:





ASSUMPTIONS:

Assumptions made during this calculation are as follows:

Description	Variable	Value and Unit	Reference
Geometry			
Top of wall at L1	$T/wall_1$	114.50 ft	Ref 1
Top of wall at L2	$T/wall_2$	110.00 ft	Ref 1
Minimum WSEL	$WSEL_{min}$	115.44 ft	Ref 1
Length of higher weir	L_1	115 ft	Ref 1
Length of notch	L_2	20 ft	Ref 1
Width of weir	w_{weir}	2 ft	Ref 1
Loads			
Maximum flow thru distribution box	$Q_{max,DB}$	350 cfs	Ref 1
Maximum flow thru Mormon slough	$Q_{max,MS}$	100 cfs	Ref 1
Maximum flow to pump system	$Q_{max,PS}$	5 cfs	Ref 1

CALCULATIONS:

The following calculations are based on the assumptions and references provided above.

Minimum head over L1 $h_{US,1} = WSEL_{min} - T/wall_1 =$ 0.94 ft

Minimum head over L2 $h_{US,2} = WSEL_{min} - T/wall_2 =$ 5.44 ft

Weir coefficient[Ref 2] $C_W =$ 3.33

Free flow through L1[Ref 2] $Q_1 = C_W * L_1 * h_{US,1}^{3/2} =$ 349.01 cfs

Free flow through L2[Ref 2] $Q_2 = C_W * L_2 * h_{US,2}^{3/2} =$ 845.0 cfs

Total flow through intake $Q_{max,tot} = Q_{max,DB} + Q_{max,MS} + Q_{max,PS} =$ 455 cfs

Headloss through weir at $Q_{max,tot}$ $H_{max} =$ 0.13 ft

MECHANICAL CALCULATIONS

BYPASS FLOW GATE SIZE

FOR THE

BELLOTA WEIR MODIFICATIONS PROJECT

PREPARED FOR

STOCKTON EAST WATER DISTRICT

CALCULATIONS PREPARED BY

HDR ENGINEERING, INC.

EMMA PAZOFF, E.I.T.

DATE PREPARED: 7/15/21

QC REVIEWER: LARRY PAULS, P.E.

QC REVIEW DATE: 8/2/21

DATE SUBMITTED: 10/06/21

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Exp. MM-DD-YYYY

DESCRIPTION:

This calculation is used to verify the bypass flow gate is sufficiently sized to pass the required 100 cfs of flow needed to go through the Mormon Slough at all times.

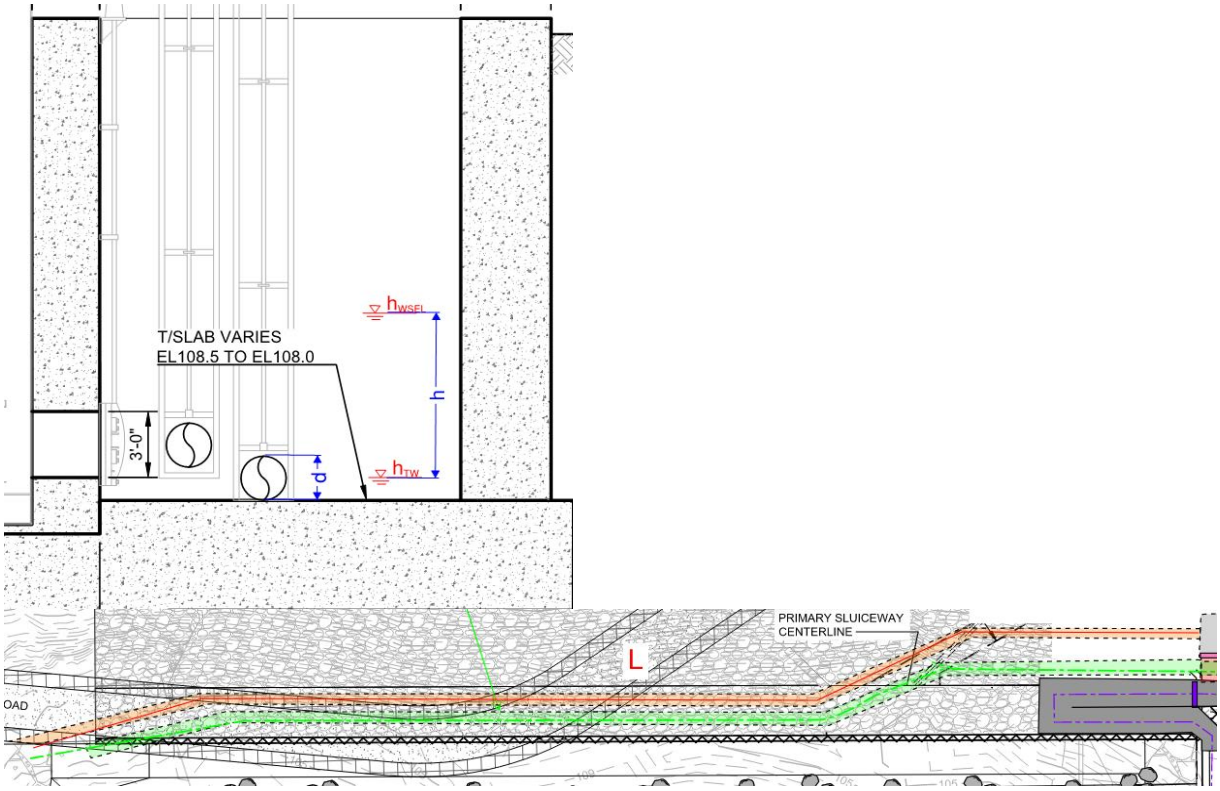
Key: | Input Value | Calculated | *Unit Change*

REFERENCES:

The following are referenced throughout the calculation:

1) 90% Plan Set, Bellota Weir Modifications
2) Mechanical Engineering Reference Manual, 13th Ed

SCHEMATIC:



Note: wet well sluice pipe is shown in orange.

ASSUMPTIONS:

Assumptions made during this calculation are as follows:

Description	Variable	Value and Unit	Reference
Loads			
Min. head on pipe	h_{min}	2 ft	
Geometry			
Minimum WSEL	$h_{WSEL,min}$	115.5 ft	Ref 1
Sluice pipe tailwater WSEL	h_{TW}	114.0 ft	Assumed tailwater WSEL, provided by HDR hydraulics
Pipe diameter	d	72 in	Ref 1
Length of pipe	L	4 ft	Ref 1
Invery el. of gate	EL_{inv}	113 ft	Ref 1
Material Properties			
Gravity	g	32.2 ft/s^2	

CALCULATIONS:

The following calculations are based on the assumptions and references provided above.

Coefficient of velocity[Ref 2, Table 17.5]

If $L < 3*d$, $C_v = 0.61$, else $C_v = 0.82$

Check: L > 3*d so C_v = 0.61

Min. velocity of flow through pipe[Ref 2, Eqn 17.68]

$v_{0,min} = C_v * (2 * g * h_{min})^{1/2}$ = 6.0 ft/s

Cross sectional area of pipe

$A_{pipe} = (pi / 4) * d^2$ = 28.27 ft^2

Discharge coefficient[Ref 2, Table 17.5]

C_d = 0.82

Minimum flow through pipe[Ref 2, Eqn 17.85]

$Q_{min} = C_d * v_{0,min} * A_{pipe}$ = 139.0 cfs

MECHANICAL CALCULATIONS

INTAKE SLUICE PIPE SIZE

FOR THE

BELLOTA WEIR MODIFICATIONS PROJECT

PREPARED FOR

STOCKTON EAST WATER DISTRICT

CALCULATIONS PREPARED BY

HDR ENGINEERING, INC.

EMMA PAZOFF, E.I.T.

DATE PREPARED: 7/15/21

QC REVIEWER: LARRY PAULS, P.E.

QC REVIEW DATE: 8/2/21

DATE SUBMITTED: 10/06/21

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Exp. MM-DD-YYYY

DESCRIPTION:

This calculation is used to verify the wet well sediment sluice pipe is sized such that it will keep sediment entrained in its flow.

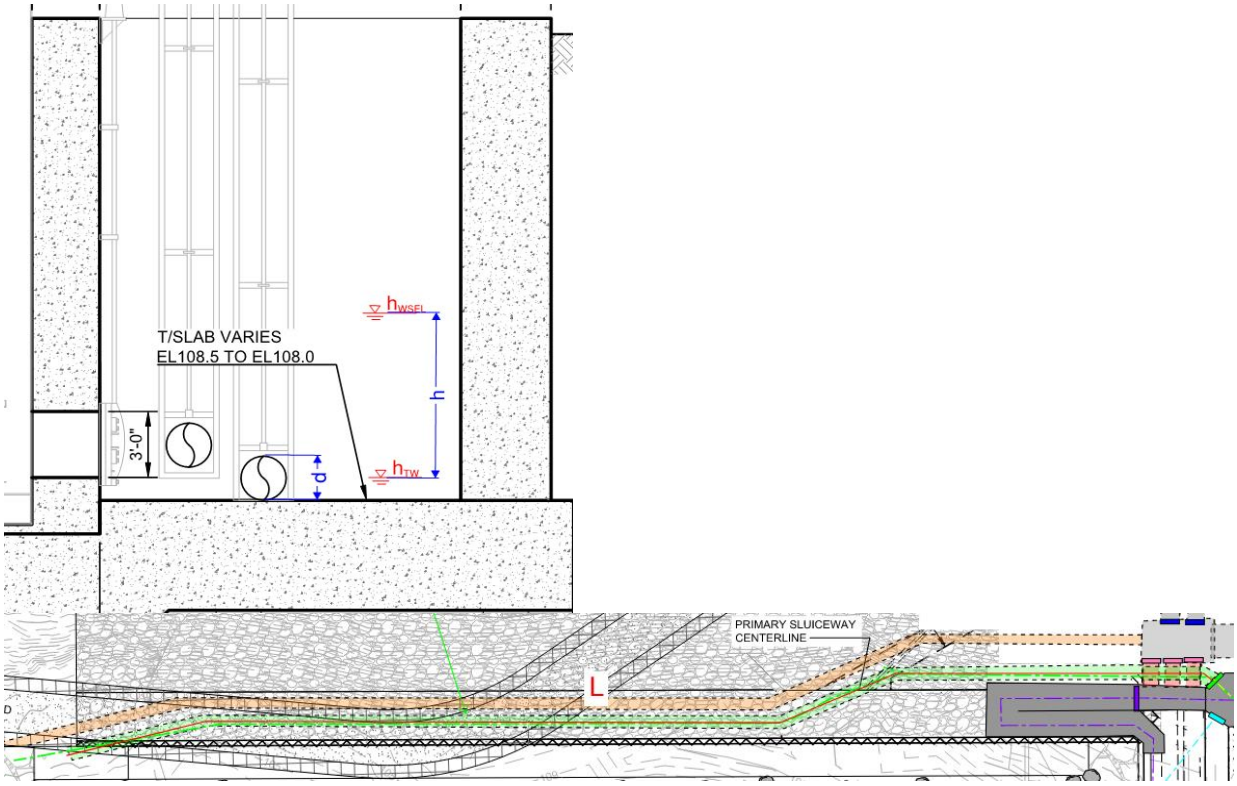
Key:	Input Value	Calculated	*Unit Change*

REFERENCES:

The following are referenced throughout the calculation:

1) 90% Plan Set, Bellota Weir Modifications
2) Mechanical Engineering Reference Manual, 13th Ed

SCHEMATIC:



ASSUMPTIONS:

Assumptions made during this calculation are as follows:

Description	Variable	Value and Unit	Reference
Loads			
Min. head on pipe	h_{min}	12.5 ft	
Max. head on pipe	h_{max}	24.0 ft	
Geometry			
Minimum WSEL	$h_{WSEL,min}$	115.5 ft	Ref 1
Maximum WSEL	$h_{WSEL,max}$	127 ft	Ref 1, 100 yr flood
Sluice pipe tailwater WSEL	h_{TW}	103.0 ft	Assumed tailwater WSEL, provided by HDR hydraulics
Pipe diameter	d	54 in	Ref 1
Length of pipe	L	430 ft	Ref 1
Width of wet well	W_{well}	13 ft	Ref 1
Floor el. of wet well	EL_{well}	108 ft	Ref 1
Material Properties			
Gravity	g	32.2 ft/s^2	
Minimum velocity	v_{min}	10.0 ft/s	REF
Misc			
Assuming free discharge to atmosphere			

CALCULATIONS:

The following calculations are based on the assumptions and references provided above.

Coefficient of velocity[Ref 2, Table 17.5]: If $L < 3*d$, $C_v = 0.61$, else $C_v = 0.82$

Check: L > 3*d so C_v = 0.82

Velocity of flow through pipe[Ref 2, Eqn 17.68]: $v_{0,min} = C_v * (2 * g * h_{min})^{1/2}$ = 23.3 ft/s

Will sediment remain entrained in the flow? Check: Is $v_{0,min} > v_{min}$? OK

Cross sectional area of pipe: $A_{pipe} = (pi / 4) * d^2$ = 15.90 ft^2

Discharge coefficient[Ref 2, Table 17.5]: C_d = 0.82

Minimum flow through pipe[Ref 2, Eqn 17.85]: $Q_{min} = C_d * v_{0,min} * A_{pipe}$ = 303.4 cfs

Min cross sectional area of wet well: $A_{well,min} = W_{well} * (h_{WSEL,min} - EL_{well})$ = 97.5 ft^2

Time to drain wet well, min WSEL [Ref 2, Eqn 17.83]

$t_{min} = (2 * A_{well} * ((h_{WSEL,min})^{1/2} - (EL_{well})^{1/2}))) / (C_d * A_{pipe} * (2 * g)^{1/2}) =$ 1 seconds

Max. velocity of flow through pipe [Ref 2, Eqn 17.68]

$v_{0,max} = C_v * (2 * g * h_{max})^{1/2} =$ 32.2 ft/s

Maximum flow through pipe [Ref 2, Eqn 17.85]

$Q_{max} = C_d * v_{0,max} * A_{pipe} =$ 420.4 cfs

Max cross sectional area of wet well

$A_{well,max} = W_{well} * (h_{WSEL,max} - EL_{well}) =$ 247 ft^2

Time to drain wet well, max WSEL [Ref 2, Eqn 17.83]

$t_{max} = (2 * A_{well} * ((h_{WSEL,max})^{1/2} - (EL_{well})^{1/2}))) / (C_d * A_{pipe} * (2 * g)^{1/2}) =$ 4 seconds

MECHANICAL CALCULATIONS

SEDIMENT PUMP SYSTEM CALCULATIONS

FOR THE

BELLOTA WEIR MODIFICATIONS PROJECT

PREPARED FOR

STOCKTON EAST WATER DISTRICT

CALCULATIONS PREPARED BY

HDR ENGINEERING, INC.

EMMA PAZOFF, E.I.T.

DATE PREPARED: 10/14/21

QC REVIEWER: LARRY PAULS, P.E.

QC REVIEW DATE: 12/27/21

DATE SUBMITTED: 2/4/22

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Exp. MM-DD-YYYY

DESCRIPTION:

State the intent and justification surrounding the sedimentation pump system at the distribution box.

REFERENCES:

References used in this calculation are as follows:

1) Bellota Weir Modifications, 90% design drawings
2) Barmesa pumps, 8BSE-HLDS

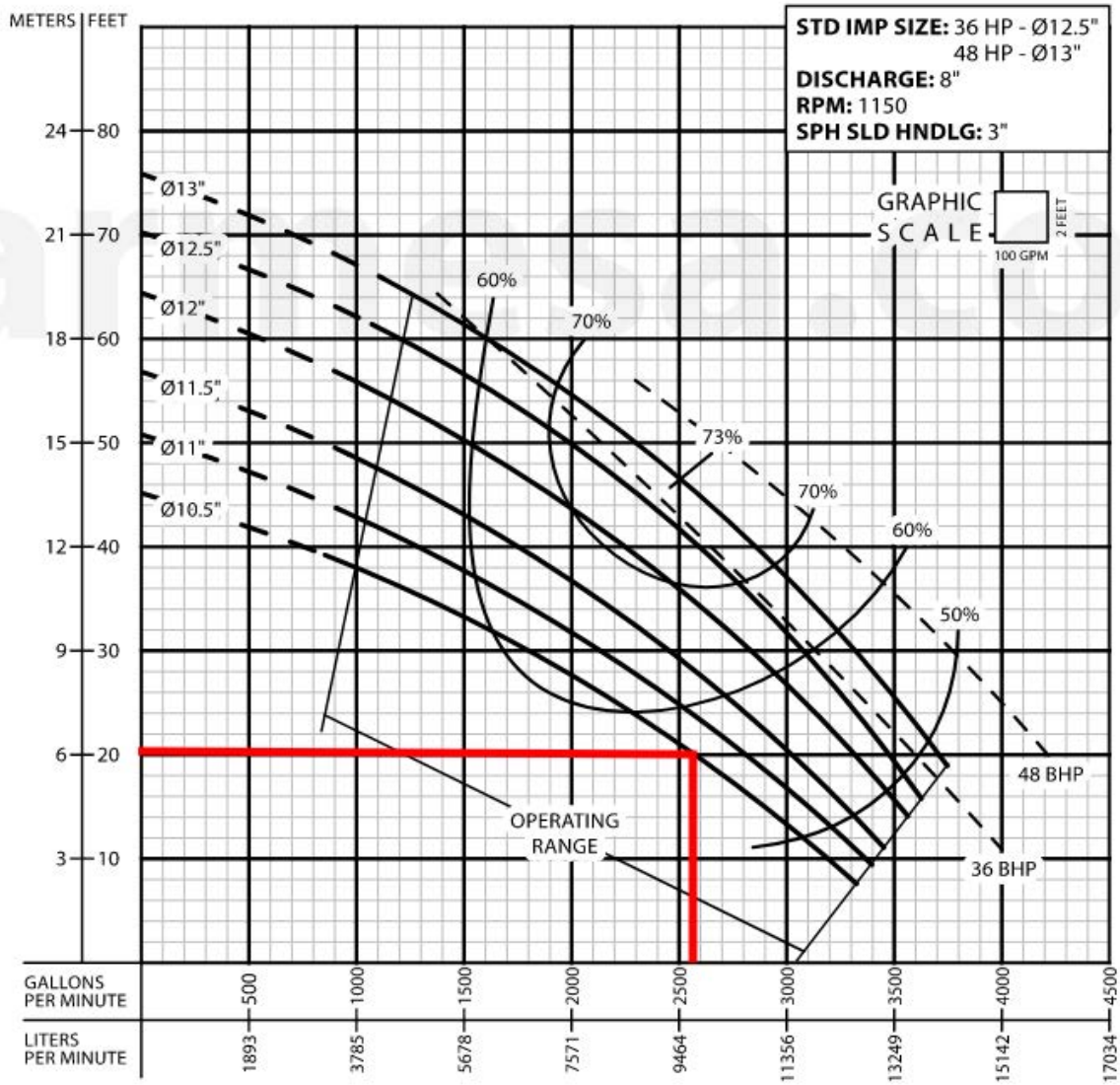
ASSUMPTIONS:

Assumptions are as follows:

- The intent of the sediment pump system is to transport fine (less than 0.069" dia.) sediment particles out of the distribution box and to drain the distribution box as needed.
- The sump area in the distribution box will be 5'-6" long by 3'-6" wide, and 2'-0" deep.
- The pump will be a slide rail pump with the ability to clip into the pipe via a slide rail coupling.
- The pipe will lead to the 54" sluice pipe to be discharged downstream of the roughened channel
- The pipe will include a check valve to prevent backwash from the 54" sluice pipe and a ball valve to isolate the pump as needed.
- The total head in the distribution box is approximately 20 ft
- The approximate volume of the distribution box is 50,000 ft^3. To drain the box in about 3 hours, the flow rate will have to be about 5 cfs (with a factor of safety)
- The pump has an 8" discharge, the piping leading to the 54" pipe will be 8" throughout.

SCHEMATICS:

Fig 1 - Pump cut sheet



MECHANICAL CALCULATIONS

SEWD INTAKE GATE ACTUATOR SIZING

FOR THE

BELLOTA WEIR MODIFICATIONS PROJECT

PREPARED FOR

STOCKTON EAST WATER DISTRICT

CALCULATIONS PREPARED BY

HDR ENGINEERING, INC.

EMMA PAZOFF, E.I.T.

DATE PREPARED: 7/15/21

QC REVIEWER: LARRY PAULS, P.E.

QC REVIEW DATE: 8/2/21

DATE SUBMITTED: 10/06/21

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Exp. MM-DD-YYYY

DESCRIPTION:

This calculation is used to gain preliminary sizing of the slide gate actuators located at the SEWD intake. These gates control the flow to the existing SEWD conveyance line. Reference BLVD_MOGx_4013, BLVD_MOGx_4014, and BLVD_MOGx_4015 in the gate schedule.

Key:

Input Value	Calculated

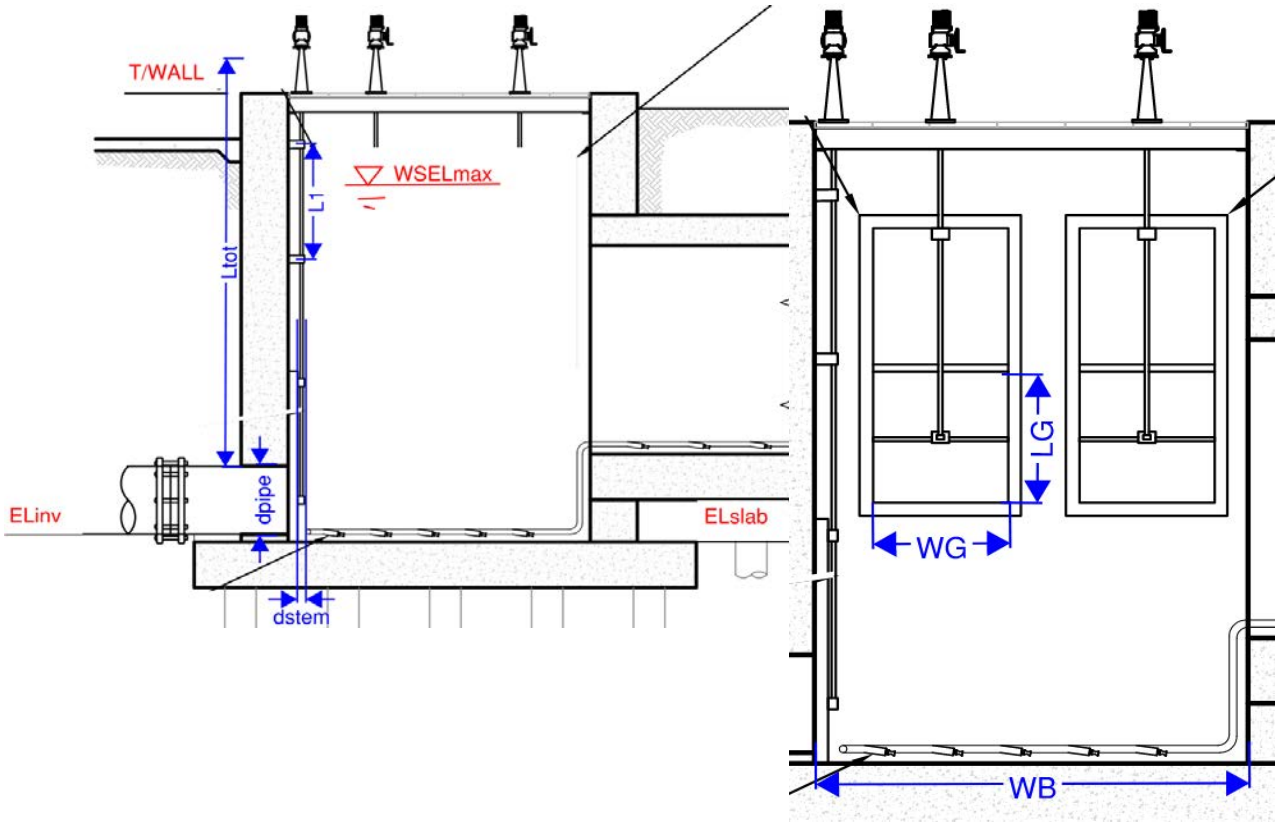
Unit Change

REFERENCES:

The following are referenced throughout the calculation:

1) 90% Design Package, Bellota Weir Modifications
2) Machinery's Handbook, 26th Ed
3) Mechanical Engineering Reference Manual, 12th Ed
4) Shigley's Machine Design

SCHEMATIC:



ASSUMPTIONS:

Assumptions made during this calculation are as follows:

Description	Variable	Value and Unit	Reference
Loads			
Weight of gate leaf	W_{leaf}	600 lbs	
Service Factor	SF	1.15	EM 1110-2-2703, 5.2.3.b
Volumetric flow	Q	58 cfs	Ref 1
Geometry			
Gate			
Gate width	W_G	54 in	Ref 1
Gate height	L_G	54 in	Ref 1
Stem diameter	d_{stem}	1.75 in	
Threads per inch	TPI_{stem}	4 threads/in	Ref 2, pg 1795-1798
Thread type		ACME	
Length to first support	L_1	60 in	length of gate + 6 in
Total stem length	L_{tot}	318 in	Ref 1, $=(T_{wall} + 2') - EL_{inv} - L_G$
Intake			
Pipe diameter	d_{pipe}	54.0 in	
Top of wall	T_{wall}	130 ft	
Maximum WSEL	$WSEL_{max}$	127.0 ft	Ref 1, DCR Table 3-6
Wall penetration invert EL	EL_{inv}	101.0 ft	
Wall penetration CL EL	EL_{CL}	103.3 ft	$= EL_{inv} + (d_{pipe}/2)$
Slab elevation	EL_{slab}	100.5 ft	Ref 1
Width of bay	W_{AB}	8.17 ft	Ref 1
Machinery/Parts			
Rotational speed	RPM_{motor}	77 rpm	Limitorque MX-85
Rated torque	T	850 ft-lbs	Limitorque MX-85
Efficiency	η_{motor}	80%	
Material Properties			
Density of water	rho_w	1.94 $slugs/ft^3$	
Gravity	g	32.2 ft/s^2	
Seal friction coeff	mu	0.3	
Stem material		A276 T316	
	rho_{stem}	0.28 lbs/in^3	
	$F_{y,stem}$	30 ksi	
	$F_{u,stem}$	75 ksi	
	E_{stem}	28000 ksi	
Thrust/Lift Nut		B584 CA863	
Material	$F_{y,nut}$	60 ksi	
	$F_{u,nut}$	110 ksi	
Stem/nut friction	f	0.38	

Allowable Stresses

	Tens/Comp	Bearing	Shear	
Stem	13.5	27.0	12.0	ksi
Thrust Nut	27.0	54.0	24.0	ksi

showing allowable stresses for tension, shear, bending and bearing. Note, allowable bearing is for when no movement is intended between the two components

TENSION	$0.45S_y \leq \sigma_{all} \leq 0.60S_y$
SHEAR	$\tau_{all} = 0.40S_y$
BENDING	$0.60S_y \leq \sigma_{all} \leq 0.75S_y$
BEARING	$\sigma_{all} = 0.90S_y$

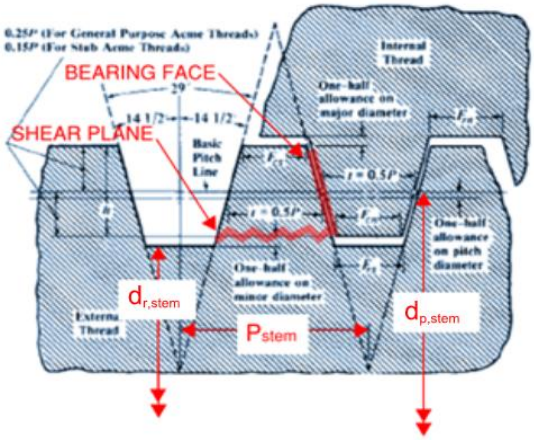
CALCULATIONS:

The following calculations are based on the assumptions and references provided above.

Max operating head on the gate	$h_G = WSEL_{max} - EL_{CL}$ =	23.8 ft
Hydrostatic pressure on gate	$P_{hys} = h_G/2.31$ =	10.3 psi
Area of gate the water is acting on	$A_G = W_G*L_G$ =	2916 in^2
Force acting on gate due to hydrostatic pressure	$F_{hys} = P_{hys}*A_G$ =	29981 lbs
Frictional resistance force	$F_{FR} = F_{hys}*mu$ =	8994 lbs
Weight of stem	$W_{stem} = pi*(d_{stem}/2)^2*L_{tot}*rho_{stem}$ =	214 lbs
Cross sectional area of channel	$A_{channel} = W_{AB}*(WSEL_{max}-EL_{slab})$ =	216 ft^2
Upstream channel velocity	$v_{US} = Q/A_{channel}$ =	0.3 ft/s
Horizontal dynamic force on gate	$F_{DG} = rho_w*A_G*(v_{US})^2$ =	3 lbs
Vertical force required to overcome F_{DG}	$F_{DGV} = F_{DG}*mu$ =	1 lbs
Weight actuator needs to be sized for		
Opening (tension)	$F_{act} = SF*(W_{leaf} + F_{FR} + W_{stem} + F_{DGV})$ =	11281 lbs
Closing (compression)	$F_{act} = SF*(F_{FR} + F_{DGV})$ =	10344 lbs

Stem[Ref 2, ACME threads]

Pitch	$P_{stem} = 1/TPI_{stem}$ =	0.25 in
Lead	$L_{stem} = 1/TPI_{stem}$ =	0.25 in
Pitch diameter	$d_{p,stem} = d_{stem} - (0.5*P_{stem})$ =	1.625 in
Root diameter	$d_{r,stem} = d_{stem} - P_{stem}$ =	1.50 in
Nut length	$L_{nut} = 2*d_{stem}$ =	3.5 in
No. threads engaged	$N_{thr} = L_{nut}*TPI_{stem}$ =	14 threads
Thread half-angle	Θ =	14.5 degrees



Stresses on Stem

Tension

$\sigma_{T,stem} = F_{act}/(pi*(d_{r,stem}/2)^2)$ =	6383 psi
$\sigma_{T,stem}$ =	*6.38* ksi
Check: Is $\sigma_{T,stem} < \sigma_{T,allowable}$?	OK

Bearing

Bearing area of single thread

$A_{stem,B1} = d_{p,stem}*pi*0.5*P_{stem}*\cos(14.5)$ =	0.618 in^2

Total bearing area of engaged threads[Ref 4]

$A_{stem,B} = 3\ threads*A_{stem,B1}$ =	1.853 in^2
Bearing stress $\sigma_{B,stem} = F_{act}/A_{stem,B}$ =	6086 psi
$\sigma_{B,stem}$ =	*6.09* ksi
Check: Is $\sigma_{B,stem} < \sigma_{B,allowable}$?	OK

Shear

Shear area per thread

$A_{stem,S1} = d_{p,stem}*pi*0.5*P_{stem}$ = 0.638 in^2

Total shear area[Ref 4]

$A_{stem,S} = 3\ threads*A_{stem,S1}$ = 1.914 in^2

Shear stress

$\tau_{stem} = F_{act}/A_{stem,S}$ = 5892 psi

τ_{stem} = *5.89* ksi

Check: Is $\tau_{stem} < \tau_{stem,allowable}$? OK

Buckling[Ref 3, Ch 51, Sec 1]

Radius of gyration $k = d_{p,stem}/4$ = 0.406 in

Min area of stem $A = pi*(d_{p,stem}/2)^2$ = 2.074 in^2

Slenderness criterion $SR_{crit} = (2*pi^2*E_{stem}/F_{y,stem})^{1/2}$ = 135.7

Slenderness ratio $SR = L_1/k$ = 147.7

Check: SR_{crit} < SR, so Long Column

Euler stress, max allowable compressive stress

$\sigma_e = pi^2*E_{stem}/(SR^2)$ = 12.6690 ksi

Max allowable compressive force on stem

$F_{max} = \sigma_e*A$ = 26274.8 lbs

Check: Is $F_C < F_{max}$? OK

Time for one operation (fully closed to fully open, or vice versa)

Height traveled per one revolution $L_{one\ rev} = P_{stem}$ = 0.25 in/rev

No. revolutions per one operation $N_{rev} = L_1/L_{one\ rev}$ = 240 rev/op

Time for one operation $t_{op} = N_{rev}/RPM_{motor}$ = 3.1 minutes

Torque required to raise the load[Ref 3, Eq'n 52.121]

$$T_{req,raise} = \frac{F_{act} * d_{p,stem}}{2} * \frac{\tan(\theta) + f}{1 - f * \tan(\theta)} =$$ 6491 in-lbs

$T_{design,raise}$ = *541 ft-lbs*

Check: $T_{design,raise} < T_{MX85}$? OK

Required actuator horsepower[Ref 3, Eq'n 51.50]

$HP_{motor} = (T_{design,raise}*RPM_{motor}/63025)/\eta_{motor})$ = 9.91 HP

Standard motor size $HP_{motor,std}$ = 10 HP

Additional motor capacity $R_{extra\ cap} = (HP_{motor,std}/HP_{motor}) - 1$ = 0.01

Snippet from Limitorque Cutsheet

Output Speed (RPM)		MX-05		MX-10		MX-20		MX-40		MX-85		MX-140		MX-150	
		Rated Output Torque													
60 Hz	50 Hz	ft-lb	N m	ft-lb	N m	ft-lb	N m	ft-lb	N m	ft-lb	N m	ft-lb	N m	ft-lb	N m
18	15	55	75	125	170	225	305	440	597	n/a	n/a	n/a	n/a	n/a	n/a
26	22	55	75	125	170	225	305	440	597	850	1,153	1,500	2,036	n/a	n/a
40	33	55	75	125	170	225	305	440	597	1,225	1,662	1,700	2,307	n/a	n/a
52	43	55	75	125	170	225	305	440	597	1,150	1,561	1,600	2,171	n/a	n/a
77	65	48	65	107	145	178	241	345	468	850	1,153	1,200	1,628	n/a	n/a
100 131[1]	84 110[1]	39	53	89	121	148	201	286	388	600	814	815	1,105	1,500	2,036
155 170[1]	127 143[1]	41	56	89	121	140	190	260	353	450	611	650	882	1,150	1,561
200	165	34	46	73	99	114	155	210	285	n/a	n/a	n/a	n/a	n/a	n/a

Note 1: MX-85, MX-140 and MX-150

MECHANICAL CALCULATIONS

SEWD WET WELL GATE ACTUATOR SIZING

FOR THE

BELLOTA WEIR MODIFICATIONS PROJECT

PREPARED FOR

STOCKTON EAST WATER DISTRICT

CALCULATIONS PREPARED BY

HDR ENGINEERING, INC.

EMMA PAZOFF, E.I.T.

DATE PREPARED: 12/3/21

QC REVIEWER: LARRY PAULS, P.E.

QC REVIEW DATE: 12/27/21

DATE SUBMITTED: 2/4/22

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Exp. MM-DD-YYYY

DESCRIPTION:

This calculation is used to gain preliminary sizing of the slide gate actuators located in the SEWD wet well.

Key:

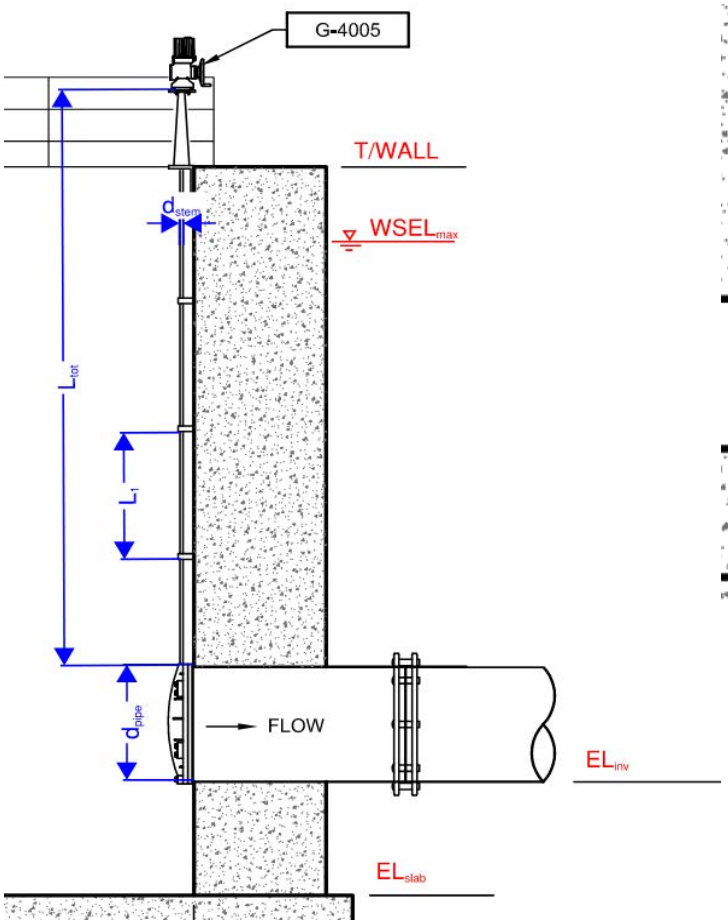
Input Value	Calculated	*Unit Change*

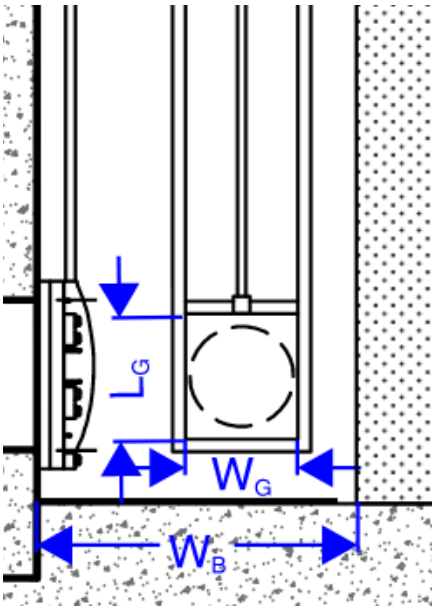
REFERENCES:

The following are referenced throughout the calculation:

1) 90% Design Package, Bellota Weir Modifications
2) Machinery's Handbook, 26th Ed
3) Mechanical Engineering Reference Manual, 12th Ed
4) Shigley's Machine Design

SCHEMATIC:





ASSUMPTIONS:

Assumptions made during this calculation are as follows:

Description	Variable	Value and Unit	Reference
Loads			
Weight of gate leaf	W_{leaf}	600 lbs	
Service Factor	SF	1.15	EM 1110-2-2703, 5.2.3.b
Volumetric flow	Q	173 cfs	Ref 1
Geometry			
Gate			
Gate width	W_G	54 in	Ref 1
Gate height	L_G	54 in	Ref 1
Stem diameter	d_{stem}	1.50 in	
Threads per inch	TPI_{stem}	4 threads/in	Ref 2, pg 1795-1798
Thread type		ACME	
Length to first support	L_1	60 in	length of gate + 6 in
Total stem length	L_{tot}	265 in	Ref 1, $=(T_{wall} + 2') - EL_{inv} - L_G$
Intake			
Pipe diameter	d_{pipe}	54 in	
Top of wall	T_{wall}	131.0 ft	
Maximum WSEL	$WSEL_{max}$	127.0 ft	Ref 1, DCR Table 3-6
Wall penetration invert EL	EL_{inv}	106.5 ft	
Wall penetration CL EL	EL_{CL}	108.7 ft	$= EL_{inv} + (d_{pipe}/2)$
Slab elevation	EL_{slab}	102.0 ft	Ref 1
Width of bay	W_{AB}	20.0 ft	Ref 1
Machinery/Parts			
Rotational speed	RPM_{motor}	77 rpm	Limitorque MX-40
Rated torque	T	440 ft-lbs	Limitorque MX-40
Efficiency	η_{motor}	80%	
Material Properties			
Density of water	rho_w	1.94 slugs/ft^3	
Gravity	g	32.2 ft/s^2	
Seal friction coeff	mu	0.3	
Stem material		A276 T316	
	rho_{stem}	0.28 lbs/in^3	
	$F_{y,stem}$	30 ksi	
	$F_{u,stem}$	75 ksi	
	E_{stem}	28000 ksi	
Thrust/Lift Nut		B584 CA863	
Material	$F_{y,nut}$	60 ksi	
	$F_{u,nut}$	110 ksi	
Stem/nut friction	mu_{SN}	0.38	

Allowable Stresses

	Tens/Comp	Bearing	Shear	
Stem	13.5	18.0	12.0	ksi
Thrust Nut	27.0	36.0	24.0	ksi

showing allowable stresses for tension, shear, bending and bearing. Note, allowable bearing is for when no movement is intended between the two components

TENSION	$0.45S_y \le \sigma_{all} \le 0.60S_y$
SHEAR	$\tau_{all} = 0.40S_y$
BENDING	$0.60S_y \le \sigma_{all} \le 0.75S_y$
BEARING	$\sigma_{all} = 0.90S_y$

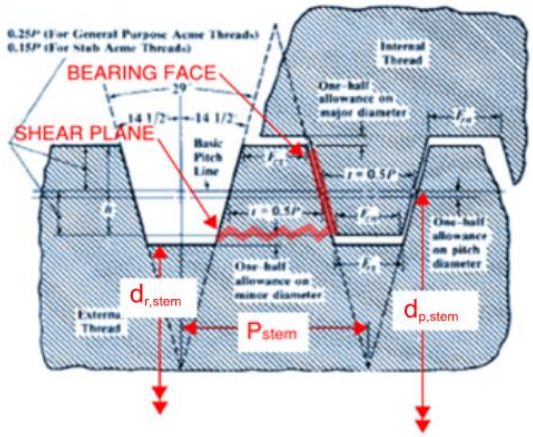
CALCULATIONS:

The following calculations are based on the assumptions and references provided above.

Max operating head on the gate	$h_G = WSEL_{max} - EL_{CL}$ =	18.3 ft
Hydrostatic pressure on gate	$P_{hys} = h_G/2.31$ =	7.9 psi
Area of water acting on gate	$A_G = W_G * L_G$ =	2916 in^2
Force acting on gate due to hydrostatic pressure	$F_{hys} = P_{hys} * A_G$ =	23101 lbs
Frictional resistance force	$F_{FR} = F_{hys} * mu$ =	6930 lbs
Weight of stem	$W_{stem} = pi*(d_{stem}/2)^2*L_{tot}*rho_{stem}$ =	130.9 lbs
Cross sectional area of channel	$A_{channel} = W_{AB}*(WSEL_{max}-EL_{slab})$ =	500 ft^2
Upstream channel velocity	$v_{US} = Q/A_{channel}$ =	0.3 ft/s
Horizontal dynamic force on gate	$F_{DG} = rho_w*A_G*(v_{US})^2$ =	5 lbs
Vertical force required to overcome F_{DG}	$F_{DGV} = F_{DG}*mu$ =	1 lbs
Weight actuator needs to be sized for		
Opening (tension)	$F_{act} = SF*(W_{leaf} + F_{FR} + W_{stem} + F_{DGV})$ =	8812 lbs
Closing (compression)	$F_{act} = SF*(F_{FR} + F_{DGV})$ =	7971 lbs

Stem[Ref 2, ACME threads]

Pitch	$P_{stem} = 1/TPI_{stem}$ =	0.25 in
Lead	$L_{stem} = 1/TPI_{stem}$ =	0.25 in
Pitch diameter	$d_{p,stem} = d_{stem} - (0.5*P_{stem})$ =	1.375 in
Root diameter	$d_{r,stem} = d_{stem} - P_{stem}$ =	1.25 in
Nut length	$L_{nut} = 2*d_{stem}$ =	3 in
No. threads engaged	$N_{thr} = L_{nut}*TPI_{stem}$ =	12 threads



Stresses on Stem

Tension

	$\sigma_{T,stem} = F_{act}/(pi*(d_{r,stem}/2)^2)$ =	7181 psi
	$\sigma_{T,stem}$ =	*7.18* ksi
	Check: Is $\sigma_{T,stem} < \sigma_{T,allowable}$?	OK

Bearing

Bearing area of single thread

	$A_{stem,B1} = d_{p,stem}*pi*0.5*P_{stem}*\cos(14.5)$ =	0.523 in^2

Total bearing area of engaged threads[Ref 4]

	$A_{stem,B} = 3\ threads*A_{stem,B1}$ =	1.568 in^2
Bearing stress	$\sigma_{B,stem} = F_{act}/A_{stem,B}$ =	5619 psi
	$\sigma_{B,stem}$ =	*5.62* ksi
	Check: Is $\sigma_{B,stem} < \sigma_{B,allowable}$?	OK

Shear

Shear area per thread

	$A_{stem,S1} = d_{p,stem}*pi*0.5*P_{stem}$ =	0.540 in^2
Total shear area[Ref 4]	$A_{stem,S} = 3\ threads*A_{stem,S1}$ =	1.620 in^2
Shear stress	$\tau_{stem} = F_{act}/A_{stem,S}$ =	5440 psi
	τ_{stem} =	*5.44* ksi
	Check: Is $\tau_{stem} < \tau_{stem,allowable}$?	OK

Buckling[Ref 3, Ch 51, Sec 1]

Radius of gyration	$k = d_{p,stem}/4 =$	0.344 in
Min area of stem	$A = pi*(d_{p,stem}/2)^2 =$	1.485 in^2
Slenderness criterion	$SR_{crit} = (2*pi^2*E_{stem}/F_{y,stem})^{1/2} =$	135.7
Slenderness ratio	$SR = L_1/k =$	174.5
Check: SR_{crit}	< SR, so	Long Column

Euler stress, max allowable compressive stress

$\sigma_e = pi^2*E_{stem}/(SR^2) =$ 9.07 ksi

Max allowable compressive force on stem

$F_{max} = \sigma_e*A =$ 13469.0 lbs

Check: Is $F_C < F_{max}$? OK

Time for one operation (fully closed to fully open, or vice versa)

Height traveled per one revolution	$L_{one\ rev} = P_{stem} =$	0.25 in/rev
No. revolutions per one operation	$N_{rev} = L_1/L_{one\ rev} =$	240 rev/op
Time for one operation	$t_{op} = N_{rev}/RPM_{motor} =$	3.1 minutes

Torque required to raise the load[Ref 3, Eq'n 52.121]

$$T_{req,raise} = \frac{F_{act} * d_{p,stem}}{2} * \frac{\tan(\theta) + f}{1 - f * \tan(\theta)} =$$ 4291 in-lbs

$T_{design,raise} =$ *357.5 ft-lbs*

Check: $T_{design,raise} < T_{MX85}$? OK

Required actuator horsepower[Ref 3, Eq'n 51.50]

$HP_{motor} = (T_{design,raise}*RPM_{motor}/63025)/\eta_{motor}) =$ 6.55 HP

Standard motor size $HP_{motor,std} =$ 7.5 HP

Additional motor capacity $R_{extra\ cap} = (HP_{motor,std}/HP_{motor}) - 1 =$ 0.14

Snippet from Limitorque Cutsheet

Output Speed (RPM)		MX-05		MX-10		MX-20		MX-40		MX-85		MX-140		MX-150	
		Rated Output Torque													
60 Hz	50 Hz	ft-lb	N m	ft-lb	N m	ft-lb	N m	ft-lb	N m	ft-lb	N m	ft-lb	N m	ft-lb	N m
18	15	55	75	125	170	225	305	440	597	n/a	n/a	n/a	n/a	n/a	n/a
26	22	55	75	125	170	225	305	440	597	850	1,153	1,500	2,036	n/a	n/a
40	33	55	75	125	170	225	305	440	597	1,225	1,662	1,700	2,307	n/a	n/a
52	43	55	75	125	170	225	305	440	597	1,150	1,561	1,600	2,171	n/a	n/a
77	65	48	65	107	145	178	241	345	468	850	1,153	1,200	1,628	n/a	n/a
100 131[1]	84 110[1]	39	53	89	121	148	201	286	388	600	814	815	1,105	1,500	2,036

155 170[1]	127 143[1]	41	56	89	121	140	190	260	353	450	611	650	882	1,150	1,561
200	165	34	46	73	99	114	155	210	285	n/a	n/a	n/a	n/a	n/a	n/a

Note 1: MX-85, MX-140 and MX-150

MECHANICAL CALCULATIONS

WATER JET SYSTEM CALCULATIONS

FOR THE

BELLOTA WEIR MODIFICATIONS PROJECT

PREPARED FOR

STOCKTON EAST WATER DISTRICT

CALCULATIONS PREPARED BY

HDR ENGINEERING, INC.

EMMA PAZOFF, E.I.T.

DATE PREPARED: 10/14/21

QC REVIEWER: LARRY PAULS, P.E.

QC REVIEW DATE: 12/27/21

DATE SUBMITTED: 2/4/22

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Exp. MM-DD-YYYY

DESCRIPTION:

The goal of this calculation is to calculate headloss from the pump to the point of use items so that the pump can be correctly sized for the system and the system needs. The goal of the system is to have pressurized water agitate any settled sediment on the intake manifold floor. Once agitated/suspended in the water, the flows will be high enough so entrained settlement can be carried downstream through the sediment sluice pipe at the distribution structure.

There are two types of point of use items, they are: 1) The eductor nozzles for the sediment suspension inside the distribution box and 2) Quick Disconnect locations for washdown (3 total)

For operations, it is assumed that if the the eductor nozzles are active, the quick disconnect locations are inactive. Also, if the quick disconnect locations are active, it is assume the loop with eductor nozzles are inactive.

Key:

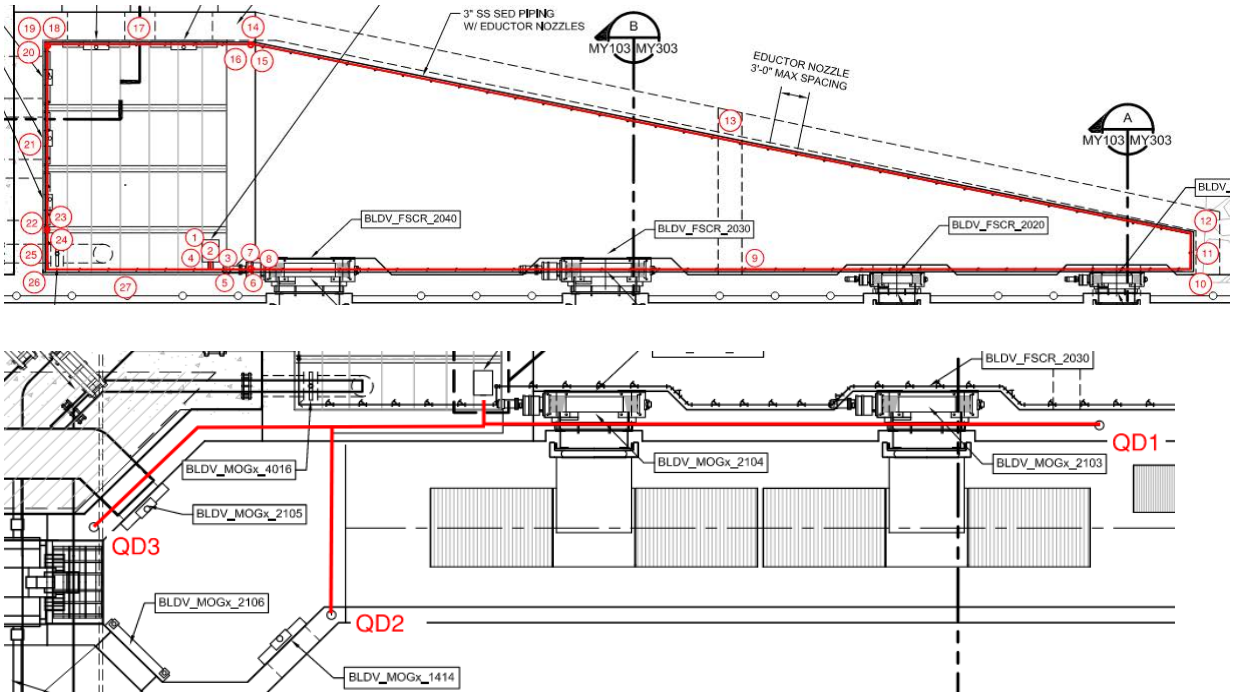
Input Value	Calculated	*Unit Change*

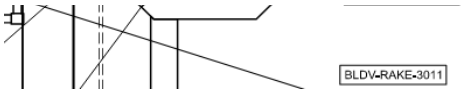
REFERENCES:

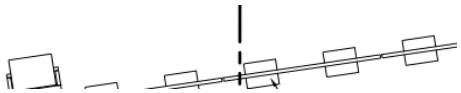
The following are referenced throughout the calculation:

1) Bellota Weir Modifications, 60% design drawings
2) Mechanical Engineering Reference Manual for the PE Exam, 13th Edition
3) Crane, Friction Water Pipe fittings
4) The Force Required to Move Particles on a Stream Bed, Rubey, 1937
5) King, "Handbook of Hydraulics"

SCHEMATIC:




BLDV-RAKE-3011



ASSUMPTIONS:

Assumptions made during this calculation are as follows:

Description	Variable	Value and Unit	Reference
Pipe flow diameter	d	8 in	
Eductor nozzle size	d_e	0.75 in	
No. of e.nozzles	N	55	One per every 3'
Max intake WSEL	EL_{intake}	121.44 ft	Ref 1, 100 yr flood
Pump elevation	EL_{pump}	101 ft	Ref 1
Slab elevation	EL_{slab}	100 ft	Ref 1
Minimum flow	Q	0.05 cfs	$=pi*(d_e/2)^2 * v_{min,e}$
Fluid velocity	$v_{min,e}$	15.5 ft/s	per manufacturer (21.2 gpm / A)
Force of water jet	F_{jet}	10 lbs	
Pump efficiency	ζ	0.63	Assumed value
gravity	g	32.2 ft/s^2	Known constant
Water specific gravity	SG	1	Known constant

CALCULATIONS:

The following calculations are based on the assumptions and references provided above.

Area from single eductor nozzle

$A_H = pi*(d_e/2)^2 =$ 0.44 in^2 / hole

Total hole area at manifold

$A_{tot} = A_H*N =$ 24.30 in^2

$A_{tot} =$ *0.169* ft^2

Min required flow to meet min exit velocity

$Q_{min} = V_{min}*A_{tot} =$ 2.6 cfs

$Q_{min} =$ *1173.0 gpm*

Area of sediment suspension loop

$A_{loop} = pi*(d/2)^2 =$ 0.349 ft^2

Velocity through sediment suspension loop

$v_{loop} = Q_{min} / A_{loop} =$ 7.49 ft/s

Reynolds number, sediment suspension loop

$Re_{loop} = d * v_{loop} / 1.004E-06 =$ 5.0E+06

Frictional factor, sediment suspension loop[Ref 2, Moody's diagram]

$f_{loop} =$ 0.009

Water jet force table

Pump Pressure	Pressure - losses	Force per nozzle		
@ 50 psi	18.6 psi	8.2 lbf	> *min ?*	*No*
@ 60 psi	28.6 psi	12.6 lbf	> *min ?*	*Yes*
@ 70 psi	38.6 psi	17.1 lbf	> *min ?*	*Yes*
@ 80 psi	48.6 psi	21.5 lbf	> *min ?*	*Yes*

@ 90 psi	58.6 psi	25.9 lbf	*> min ?*	*Yes*
@ 100 psi	68.6 psi	30.3 lbf	*> min ?*	*Yes*

Where, Pressure = [psi] pressure pump produces
Pressure minus losses = [psi] pressure at point of use item, based on piping losses outlined below
Per hole = [Pressure minus losses] * A_H = Force of water jet at each hole
Note: Sufficient if force per hole is greater than min force defined in the assumption section

Piping losses

Table below, where
Column 1 = Type of device
Column 2 = Minor loss coefficient = K[Ref 2, Table 17.4; Ref 5]
Column 3 = Quantity of device = N
Column 4[Ref 2, eq'n 17.28] = Minor head loss of component = N * (K * v_{loop}^2 / (2g))

Eductor Nozzle System

Column

1	2	3	4
Device	K value	Quantity	Headloss (ft)
Tee, run	0.6	55	28.77
Tee, bend	1.8	1	1.57
45 elbow	0.2	1	0.17
90 elbow	0.9	14	10.98
Butterfly valve	0.405	2	0.71
Pipe length	0.027	333	7.84

h_T = total frictional head loss = 50.0 ft
h_S = total static head loss = (EL_{intake} - EL_{slab} + 1) = 22.4 ft
Pressure loss = (h_T + h_S)/2.31 = 31.4 psi

Quick Disconnect System

Column

1	2	3	4
Device	K value	Quantity	Headloss (ft)
Tee, run	0.6	0	0.00
Tee, bend	1.8	4	6.28
45 elbow	0.2	2	0.35
90 elbow	0.9	9	7.06
Butterfly valve	0.405	4	1.41
Pipe length	0.027	100	2.35

h_T = total frictional head loss = 17.5 ft

h_S = total static head loss = (EL_{intake} - EL_{slab} + 1) = 22.4 ft

Pressure loss = (h_T + h_S)/2.31 = 17.3 psi

Note: The eductor nozzle system and the quick disconnect system will not be operating at the same time. The system shall be sized based on the larger of the two demands.

Total pressure req'd at pump = fluid pressure + total losses = 60 psi

HP req'd per pump[Ref 2, Table 18.4] $HP_p = h_T*Q_{tot}*SG/3956$ = 41.1 HP

Actual HP req'd per pump accounting for losses $HP = HP_p/\zeta$ = 65.2 HP

Traditional motor size = 75.0 HP

The manufacturer recommends that there is at least a 20 psi differential between the pressure in the piping system and the distribution structure. Since the pressure - losses > 20 psi, that requirement is satisfied.

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Appendix G. Structural Calculations

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Intake Design

Descriptions:

The following calculations provide the design of Intake Walls, Slabs and Pile foundations for 90% design phase.

The lateral soil pressure and soil seismic loads were used according to the recommendation given in Geotechnical Report for Bellota Project. According to geotech report the seismic load parameters were obtained based on ASCE7-16 with PGA equal to 0.18g. The load combination used for this structure is from following table per EM 1110-2-2104.

Load Case	Load Description	Load Category	Factored Loads
R1	Normal Operating	Usual	$2.2(D + EH + EV + Hs_U)$
R2A	Normal Operating + Surcharge	Unusual	$1.6(D + EH + EV + Hs_U + ES_N)$
R2B	Normal Operating + Short Duration Water Load	Unusual	$1.6(D + EH + EV + Hs_N)$
R3A	Normal Operating + OBE	Unusual	$1.5(D + EH + EV + Hs_U + EQ)$
R3B	Normal Operating + MDE	Extreme (Strength)	$(1.2 \text{ or } 0.9)D + (1.5 \text{ or } 0.5)EH + (1.35 \text{ or } 1.0)EV + 1.0\ Hs_U + 1.25EQ$
R4	Normal Operating + Maximum Water	Extreme (Strength)	$(1.2 \text{ or } 0.9)D + (1.5 \text{ or } 0.5)EH + (1.35 \text{ or } 1.0)EV + 1.3Hs_X$

Source: EM 1110-2-2104

Where: D: Dead Load; EH: Lateral Earth Pressure; EV: Vertical Earth Load; ES: Surcharge Load; HS_N: Normal Hydroststaic Load; HS_U: Hydroststaic Load during Flood; EQ: earthquake Load;

References:

- EM 1110-2-2104, Strength Design for Reinforced Concrete Hydraulic Structures
- ASCE/SEI 7-16 Code
- International Building Code IBC 2018
- ACI318-19
- Geotechnical Investigation Report-Bellota Weir Modification Project

Assumptions:

The intake structure is modeled in SAP2000 program including all the pressures such as active, at-rest and seismic soil pressure, hydrostatic and hydrodynamic pressure; and response spectrum analysis was performed for seismic analysis. Per EM 1110-2-2104 once response spectrum analysis is used, the soil load factor can be reduced to 1.0, which would be the case for this structure. For the design both 100-year flood as unusual and 500 year flood as extreme load case were also used.
For 65% design phase the maximum shear on piles were used for design the piles. Therefore one group effect value was used for critical pile and equivalent spring stiffness was calibrated based on critical pile using the iteration process between SAP2000 and LPile programs. For 90% design the effect of each pile would be evaluated individually and equivalent stiffness will be calibrated for each pile based on location and associated shear forces.
4" concrete cover is provided for both faces of walls and slabs for corrosion protection purposes. For all structural concrete, 5000 psi concrete will be used.
The following reported demands from SAP2000 is representing the maximum absolute value among all the defined load combinations

Material Properties:

$f_c := 5000psi$ — Assumed concrete compressive strength

$\gamma_c := 150pcf$ — Concrete unit weight

$E_c := 57000 \cdot \sqrt{\frac{f_c \cdot in^2}{lbf}} \cdot \frac{lbf}{in^2} = 4.031 \times 10^6 \cdot \frac{lbf}{in^2}$ — Concrete elastic modulus (ACI 318)

$$\beta_1 := \begin{cases} 0.85 & \text{if } f_c \le 4000psi \\ \max\left[0.85 - 0.05 \cdot \left(\frac{f_c - 4000psi}{1000psi}\right), 0.65\right] & \text{otherwise} \end{cases} = 0.8$$

Compression block factor (ACI 318)

$\phi_v := 0.75$ — Shear strength reduction factor

$\lambda := 1$ — Normal weight concrete factor

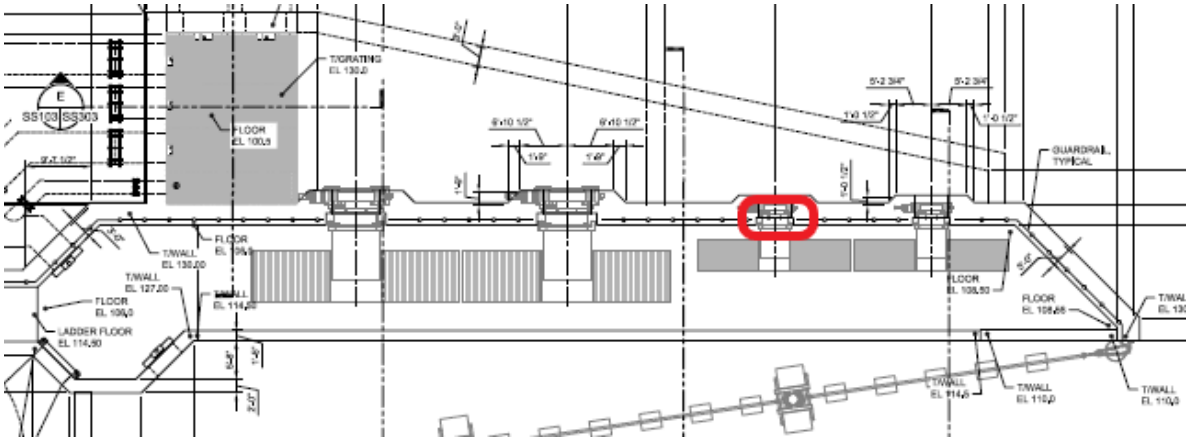
$f_y := 60ksi$ — Assumed yield strength of rebar

$E_s := 29000ksi$ — Steel elastic modulus

Check Strength Capacity of Concrete Components:

For Strength design the Vault box was modeled in SAP2000 and loads are applied on model to get the shear and bending demands on walls

NW Wall Design:



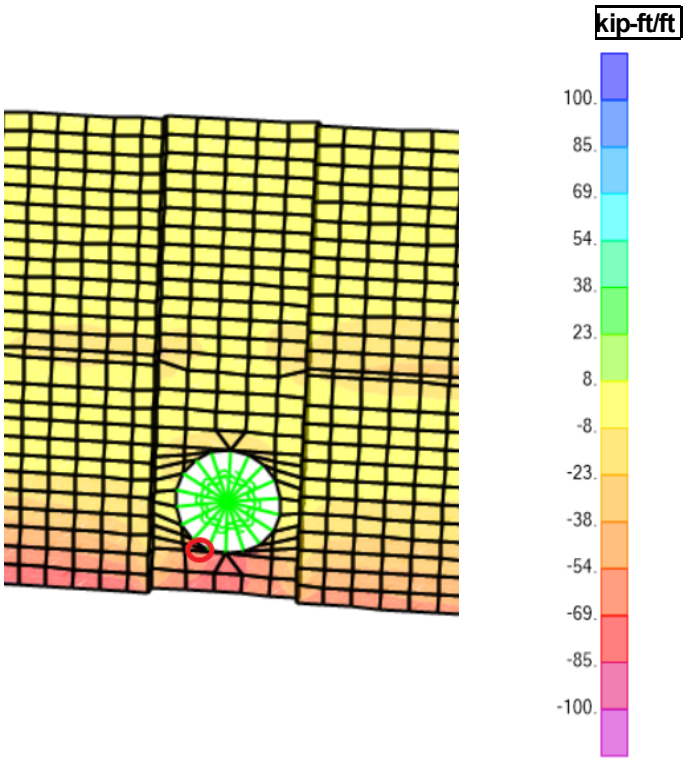


Figure 1: NW Wall with region of maximum moment circled in red

Positive Moment (tension at outer face of wall)

$M_u := 123kip \cdot \frac{ft}{ft}$ *Moment from SAP2000*

$V_u := 35 \frac{kip}{ft}$ *Shear from SAP2000*

$c_{bs} := 4in$ *Cover for tension rebar*

$c_{ts} := 4in$ *Cover for compression rebar*

$b := 1ft$ *Assumed width of section*

$d_{bar\_t} := 1in$ *Diameter of Tensile Reinforcement (#8 @8in bar)*

$A_{ts} := \frac{12}{8} \frac{\pi}{4} \cdot d_{bar\_t}^2 = 1.178 \cdot in^2$ *Cross Sectional Area of Tensile Reinforcement*

$t_{wall} := 3ft$ *Thickness of wall*

$d_{ts} := t_{wall} - c_{bs} - 0.5 d_{bar\_t} = 31.5 \cdot in$ *Depth of Tensile Reinforcement in the Slab*

$d_{bar\_c} := 1in$ *Diameter of Compression Reinforcement (1 #8@12in bar)*

$A_{cs} := \frac{12}{12} \frac{\pi}{4} \cdot d_{bar\_c}^2 = 0.785 \cdot in^2$ *Cross Sec Area of Compression Reinforcement*

$d_{cs} := c_{ts} + 0.5 \cdot d_{bar\_c} = 4.5 \cdot in$ *Depth of Compression Reinforcement in the Slab*

$A_{bs} := \begin{pmatrix} A_{ts} \\ A_{cs} \end{pmatrix} = \begin{pmatrix} 1.178 \\ 0.785 \end{pmatrix} \cdot in^2$ *Cross Sectional Areas Reinforcement*

$d_{bs} := \begin{pmatrix} d_{ts} \\ d_{cs} \end{pmatrix} = \begin{pmatrix} 31.5 \\ 4.5 \end{pmatrix} \cdot in$ *Depths of Reinforcement*

$C_{TS.Slab} :=$ "OK" if $A_{ts} \geq 1in^2 \wedge A_{cs} \geq 1in^2$ = "OK"
otherwise
  "OK" if $\sum A_{bs} > 0.003 \cdot b \cdot t_{wall}$
  "Min Reinf. Not Met" otherwise

Minimum reinforcement (ACI 318-19-8-6-1)

$T_a := 113kip$ *Assumed Tension Force on Section*

$a := \frac{T_a}{0.85 \cdot f_c \cdot b} = 2.216 \cdot in$ *Width of Whitney Stress Block*

$$\varepsilon_s := \left| \begin{array}{l} \text{for } i \in 0..\text{rows}(d_{bs}) - 1 \\ \quad \varepsilon_{s_i} \leftarrow \dfrac{0.003}{\dfrac{a}{\beta_1}} \cdot \left(d_{bs_i} - \dfrac{a}{\beta_1} \right) \\ \varepsilon_s \end{array} \right. = \begin{pmatrix} 0.031 \\ 1.874 \times 10^{-3} \end{pmatrix}$$ *Strain in Reinforcement*

$$\text{Check}_{\text{Tensile\_Strain}} := \left| \begin{array}{l} \text{"OK"} \quad \text{if } \max(\varepsilon_s) \geq \max\left(\dfrac{f_y}{E_s}, 0.005 \right) \\ \text{"NG - Too much reinforcement"} \quad \text{otherwise} \end{array} \right. = \text{"OK"}$$

$$f_s := \left| \begin{array}{l} \text{for } i \in 0..\text{rows}(d_{bs}) - 1 \\ \quad \left| \begin{array}{l} f_{s_i} \leftarrow f_y \quad \text{if } \varepsilon_{s_i} \geq \dfrac{f_y}{E_s} \\ f_{s_i} \leftarrow -f_y \quad \text{if } \varepsilon_{s_i} \leq \dfrac{-f_y}{E_s} \\ f_{s_i} \leftarrow E_s \cdot \varepsilon_{s_i} \quad \text{if } \varepsilon_{s_i} < \dfrac{f_y}{E_s} \wedge \varepsilon_{s_i} \geq 0 \\ f_{s_i} \leftarrow E_s \cdot \varepsilon_{s_i} + 0.85 \cdot f_c \quad \text{otherwise} \end{array} \right. \\ f_s \end{array} \right. = \begin{pmatrix} 60 \\ 54.356 \end{pmatrix} \cdot \text{ksi}$$ *Stress in Reinforcement*

$$P_s := \left| \begin{array}{l} \text{for } i \in 0..\text{rows}(d_{bs}) - 1 \\ \quad P_{s_i} \leftarrow f_{s_i} \cdot A_{bs_i} \\ P_s \end{array} \right. = \begin{pmatrix} 70.686 \\ 42.691 \end{pmatrix} \cdot \text{kip}$$ *Force in Reinforcement*

$$P_{\text{check}} := \begin{pmatrix} \dfrac{\sum P_s}{T_a} \\ \left| \begin{array}{l} \text{"OK"} \quad \text{if } \dfrac{\sum P_s}{T_a} \geq 0.95 \wedge \dfrac{\sum P_s}{T_a} \leq 1.05 \\ \text{"Reiterate"} \quad \text{otherwise} \end{array} \right. \end{pmatrix} = \begin{pmatrix} 1.003 \\ \text{"OK"} \end{pmatrix}$$ *Check for Equilibrium*

$$M_s := \left| \begin{array}{l} \text{for } i \in 0..\text{rows}(d_{bs}) - 1 \\ \quad M_{s_i} \leftarrow P_{s_i} \cdot \left(d_{bs_i} - \frac{a}{2} \right) \\ M_s \end{array} \right. = \begin{pmatrix} 179.025 \\ 12.068 \end{pmatrix} \cdot \text{ft} \cdot \text{kip}$$ *Moment in Reinforcement*

$$\phi_b := \left| \begin{array}{ll} 0.65 & \text{if } \max(\varepsilon_s) \le 0.002 \\ 0.90 & \text{if } \max(\varepsilon_s) \ge 0.005 \\ 0.65 + (\max(\varepsilon_s) - 0.002) \cdot \left(\frac{250}{3} \right) & \text{otherwise} \end{array} \right. = 0.9$$ *Load Reduction Factor*

$$\phi M_{n\_wall\_pos} := \frac{\phi_b \cdot \sum M_s}{b} = 171.98 \cdot \frac{\text{kip} \cdot \text{ft}}{\text{ft}}$$ *Nominal Moment Capacity of Slab*

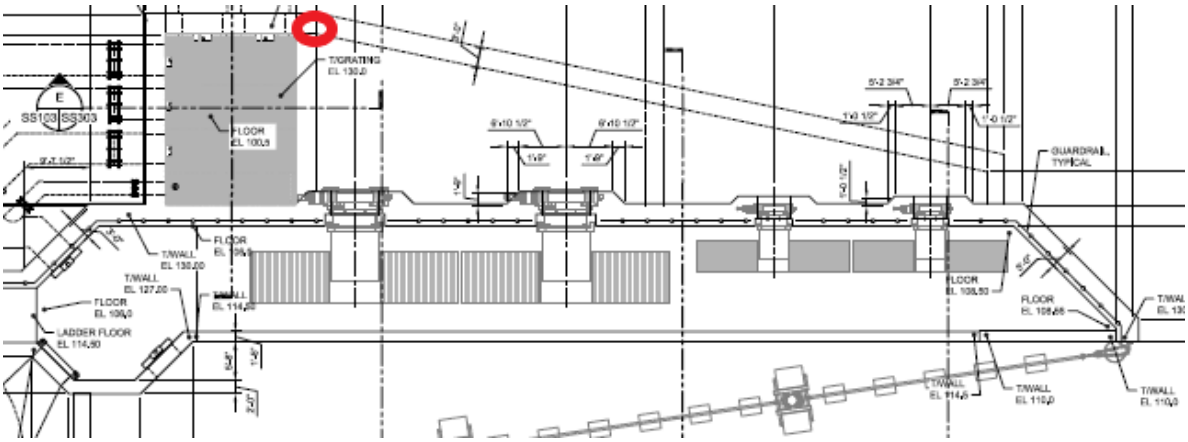
$$\phi V_{n\_wall} := \phi_v \cdot 2 \cdot \lambda \cdot \sqrt{\frac{f_c}{\text{psi}}} \cdot \text{psi} \cdot d_{ts} = 40.093 \cdot \frac{\text{kip}}{\text{ft}}$$ *Nominal Shear Capacity of Slab (ACI 318 Section 22.5.5.1)*

$$DCR_M := \frac{M_u}{\phi M_{n\_wall\_pos}} = 0.715$$

$$DCR_V := \frac{V_u}{\phi V_{n\_wall}} = 0.873$$

OK use #8@8 at outer face and #8@12" at inner face

Culvert Wall Design:



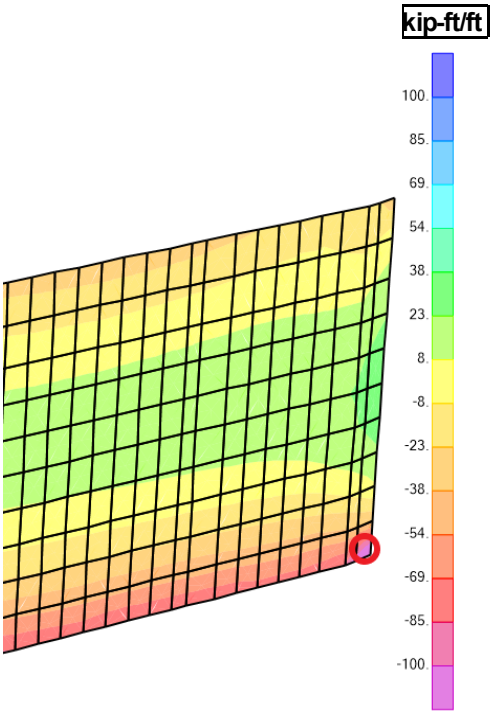


Figure 2: Culvert wall with region of maximum moment circled in red

Positive Moment (tension at outer face of wall)

$M_u := 113kip \cdot \frac{ft}{ft}$ *Maximum moment from SAP2000*

$V_u := 32.11 \frac{kip}{ft}$ *Maximum shear from SAP2000*

$c_{bs} := 4in$ Cover for tension rebar

$c_{ts} := 4in$ Cover for compression rebar

$b := 1ft$ *Assumed width of section*

$d_{bar\_t} := 1in$ *Diameter of Tensile Reinforcement (1 #8 @8in bar)*

$A_{ts} := \frac{12}{8} \frac{\pi}{4} \cdot d_{bar\_t}^{2} = 1.178 \cdot in^2$ *Cross Sectional Area of Tensile Reinforcement*

$t_{wall} := 2.5ft$ *Thickness of wall*

$d_{ts} := t_{wall} - c_{bs} - 0.5 d_{bar\_t} = 25.5 \cdot in$ *Depth of Tensile Reinforcement in the Slab*

$d_{bar\_c} := 1in$ *Diameter of Compression Reinforcement (1 #8@12in bar)*

$A_{cs} := \frac{\pi}{4} \cdot d_{bar\_c}^{2} = 0.785 \cdot in^2$ *Cross Sec Area of Compression Reinforcement*

$d_{cs} := c_{ts} + 0.5 \cdot d_{bar\_c} = 4.5 \cdot in$ *Depth of Compression Reinforcement in the Slab*

$A_{bs} := \begin{pmatrix} A_{ts} \\ A_{cs} \end{pmatrix} = \begin{pmatrix} 1.178 \\ 0.785 \end{pmatrix} \cdot in^2$ *Cross Sectional Areas Reinforcement*

$d_{bs} := \begin{pmatrix} d_{ts} \\ d_{cs} \end{pmatrix} = \begin{pmatrix} 25.5 \\ 4.5 \end{pmatrix} \cdot in$ *Depths of Reinforcement*

$C_{TS.Slab} := \begin{cases} \text{"OK"} & \text{if } A_{ts} \geq 1in^2 \wedge A_{cs} \geq 1in^2 \\ \text{"OK"} & \text{if } \sum A_{bs} > 0.003 \cdot b \cdot t_{wall} \\ \text{"Min Reinf. Not Met"} & \text{otherwise} \end{cases} = \text{"OK"}$

Minimum reinforcement (ACI 318-19-8-6-1)

$T_a := 113kip$ *Assumed Tension Force on Section*

$a := \frac{T_a}{0.85 \cdot f_c \cdot b} = 2.216 \cdot in$ *Width of Whitney Stress Block*

$$\varepsilon_s := \left| \begin{array}{l} \text{for } i \in 0..\text{rows}(d_{bs}) - 1 \\ \quad \varepsilon_{s_i} \leftarrow \dfrac{0.003}{\dfrac{a}{\beta_1}} \cdot \left(d_{bs_i} - \dfrac{a}{\beta_1} \right) \\ \varepsilon_s \end{array} \right. = \begin{pmatrix} 0.025 \\ 1.874 \times 10^{-3} \end{pmatrix}$$ *Strain in Reinforcement*

$$\text{Check}_{\text{Tensile\_Strain}} := \left| \begin{array}{l} \text{"OK"} \quad \text{if } \max(\varepsilon_s) \geq \max\left(\dfrac{f_y}{E_s}, 0.005 \right) \\ \text{"NG - Too much reinforcement"} \quad \text{otherwise} \end{array} \right. = \text{"OK"}$$

$$f_s := \left| \begin{array}{l} \text{for } i \in 0..\text{rows}(d_{bs}) - 1 \\ \quad \left| \begin{array}{l} f_{s_i} \leftarrow f_y \quad \text{if } \varepsilon_{s_i} \geq \dfrac{f_y}{E_s} \\ f_{s_i} \leftarrow -f_y \quad \text{if } \varepsilon_{s_i} \leq \dfrac{-f_y}{E_s} \\ f_{s_i} \leftarrow E_s \cdot \varepsilon_{s_i} \quad \text{if } \varepsilon_{s_i} < \dfrac{f_y}{E_s} \wedge \varepsilon_{s_i} \geq 0 \\ f_{s_i} \leftarrow E_s \cdot \varepsilon_{s_i} + 0.85 \cdot f_c \quad \text{otherwise} \end{array} \right. \\ f_s \end{array} \right. = \begin{pmatrix} 60 \\ 54.356 \end{pmatrix} \cdot \text{ksi}$$ *Stress in Reinforcement*

$$P_s := \left| \begin{array}{l} \text{for } i \in 0..\text{rows}(d_{bs}) - 1 \\ \quad P_{s_i} \leftarrow f_{s_i} \cdot A_{bs_i} \\ P_s \end{array} \right. = \begin{pmatrix} 70.686 \\ 42.691 \end{pmatrix} \cdot \text{kip}$$ *Force in Reinforcement*

$$P_{\text{check}} := \left(\begin{array}{l} \dfrac{\sum P_s}{T_a} \\ \left| \begin{array}{l} \text{"OK"} \quad \text{if } \dfrac{\sum P_s}{T_a} \geq 0.95 \wedge \dfrac{\sum P_s}{T_a} \leq 1.05 \\ \text{"Reiterate"} \quad \text{otherwise} \end{array} \right. \end{array} \right) = \begin{pmatrix} 1.003 \\ \text{"OK"} \end{pmatrix}$$ *Check for Equilibrium*

$$M_s := \left| \begin{array}{l} \text{for } i \in 0..\text{rows}(d_{bs}) - 1 \\ \quad M_{s_i} \leftarrow P_{s_i} \cdot \left(d_{bs_i} - \frac{a}{2} \right) \\ M_s \end{array} \right. = \begin{pmatrix} 143.682 \\ 12.068 \end{pmatrix} \cdot \text{ft} \cdot \text{kip}$$ *Moment in Reinforcement*

$$\phi_b := \left| \begin{array}{ll} 0.65 & \text{if } \max(\varepsilon_s) \le 0.002 \\ 0.90 & \text{if } \max(\varepsilon_s) \ge 0.005 \\ 0.65 + (\max(\varepsilon_s) - 0.002) \cdot \left(\frac{250}{3} \right) & \text{otherwise} \end{array} \right. = 0.9$$ *Load Reduction Factor*

$$\phi M_{n\_wall\_pos} := \frac{\phi_b \cdot \sum M_s}{b} = 140.17 \cdot \frac{\text{kip} \cdot \text{ft}}{\text{ft}}$$ *Nominal Moment Capacity of Slab*

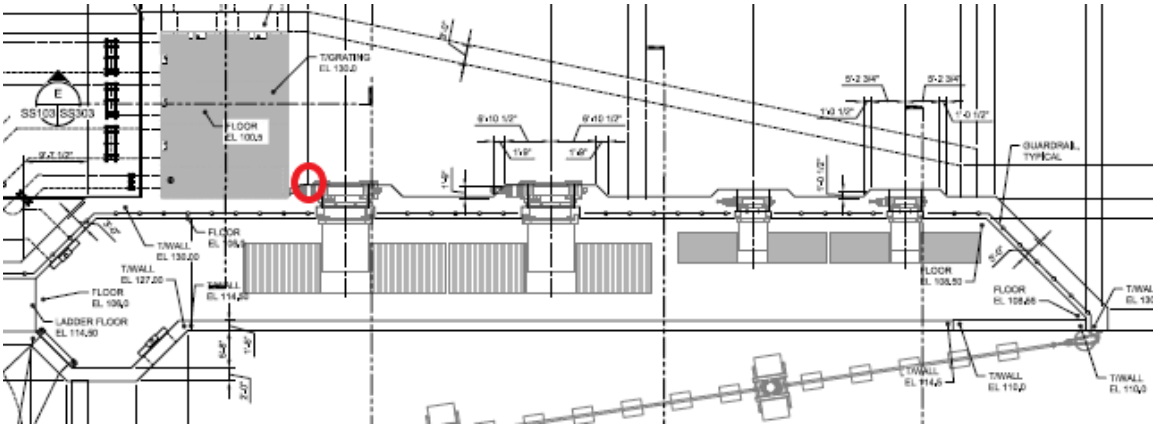
$$\phi V_{n\_wall} := \phi_v \cdot 2 \cdot \lambda \cdot \sqrt{\frac{f_c}{\text{psi}}} \cdot \text{psi} \cdot d_{ts} = 32.456 \cdot \frac{\text{kip}}{\text{ft}}$$ *Nominal Shear Capacity of Slab (ACI 318 Section 22.5.5.1)*

$$DCR_M := \frac{M_u}{\phi M_{n\_wall\_pos}} = 0.806$$

$$DCR_V := \frac{V_u}{\phi V_{n\_wall}} = 0.989$$

OK use #8@8 at outer face and #8@12" at inner face

Culvert Roof Design:



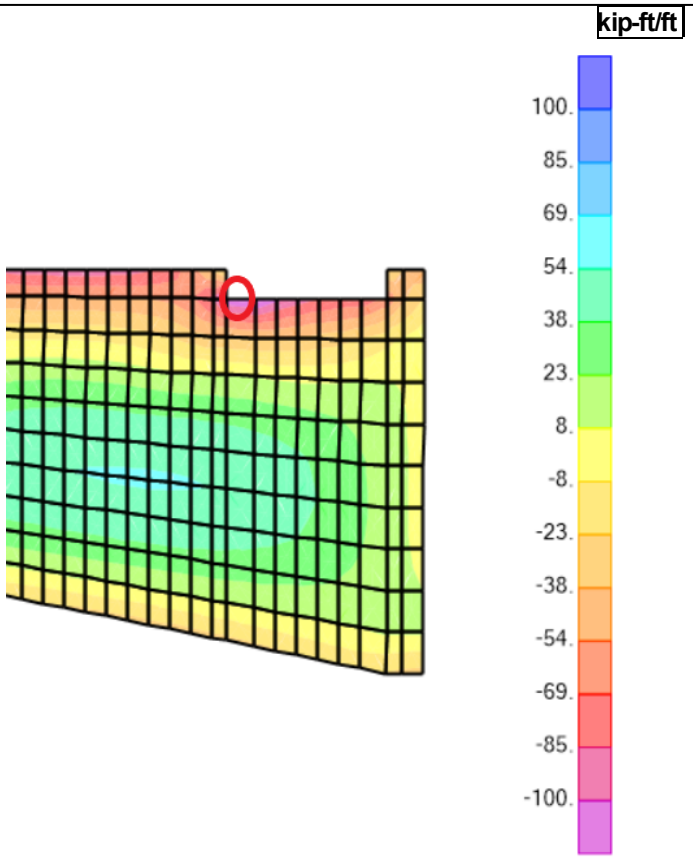


Figure 3: Culvert Roof (upper half) with region of maximum moment circled in red

Negative Moment (tension at top)

$M_u := 135\text{kip}\cdot\frac{\text{ft}}{\text{ft}}$ *Maximum moment from SAP2000*

$V_u := 30\frac{\text{kip}}{\text{ft}}$ *Maximum shear from SAP2000*

$c_{bs} := 4\text{in}$ Cover for tension rebar

$c_{ts} := 4\text{in}$ Cover for compression rebar

$b := 1\text{ft}$ *Assumed width of section*

$d_{bar\_t} := 1\text{in}$ *Diameter of Tensile Reinforcement (1 #8 @8in bar)*

$A_{ts} := \frac{12}{8}\frac{\pi}{4}\cdot d_{bar\_t}^{\ 2} = 1.178\cdot\text{in}^2$ *Cross Sectional Area of Tensile Reinforcement*

$t_{roof} := 2.5\text{ft}$ *Thickness of roof*

$d_{ts} := t_{roof} - c_{bs} - 0.5d_{bar\_t} = 25.5\cdot\text{in}$ *Depth of Tensile Reinforcement in the Slab*

$d_{bar\_c} := 1in$ *Diameter of Compression Reinforcement (1 #8@12in bar)*

$A_{cs} := \frac{\pi}{4} \cdot d_{bar\_c}^{2} = 0.785 \cdot in^2$ *Cross Sec Area of Compression Reinforcement*

$d_{cs} := c_{ts} + 0.5 \cdot d_{bar\_c} = 4.5 \cdot in$ *Depth of Compression Reinforcement in the Slab*

$A_{bs} := \begin{pmatrix} A_{ts} \\ A_{cs} \end{pmatrix} = \begin{pmatrix} 1.178 \\ 0.785 \end{pmatrix} \cdot in^2$ *Cross Sectional Areas Reinforcement*

$d_{bs} := \begin{pmatrix} d_{ts} \\ d_{cs} \end{pmatrix} = \begin{pmatrix} 25.5 \\ 4.5 \end{pmatrix} \cdot in$ *Depths of Reinforcement*

$C_{TS.Slab} :=$ "OK" if $A_{ts} \geq 1in^2 \wedge A_{cs} \geq 1in^2$ = "OK"
otherwise
  "OK" if $\sum A_{bs} > 0.003 \cdot b \cdot t_{roof}$
  "Min Reinf. Not Met" otherwise

Minimum reinforcement (ACI 318-19-8-6-1)

$T_a := 113kip$ *Assumed Tension Force on Section*

$a := \frac{T_a}{0.85 \cdot f_c \cdot b} = 2.216 \cdot in$ *Width of Whitney Stress Block*

$\varepsilon_s :=$ for $i \in 0 .. rows(d_{bs}) - 1$ $= \begin{pmatrix} 0.025 \\ 1.874 \times 10^{-3} \end{pmatrix}$ *Strain in Reinforcement*
  $\varepsilon_{s_i} \leftarrow \frac{0.003}{\frac{a}{\beta_1}} \cdot \left(d_{bs_i} - \frac{a}{\beta_1} \right)$
  ε_s

$Check_{Tensile\_Strain} :=$ "OK" if $\max(\varepsilon_s) \geq \max\left(\frac{f_y}{E_s}, 0.005 \right)$ = "OK"
"NG - Too much reinforcement" otherwise

$$f_s := \left| \begin{array}{l} \text{for } i \in 0..\text{rows}(d_{bs}) - 1 \\ \quad \left| \begin{array}{l} f_{s_i} \leftarrow f_y \text{ if } \varepsilon_{s_i} \geq \frac{f_y}{E_s} \\ f_{s_i} \leftarrow -f_y \text{ if } \varepsilon_{s_i} \leq \frac{-f_y}{E_s} \\ f_{s_i} \leftarrow E_s \cdot \varepsilon_{s_i} \text{ if } \varepsilon_{s_i} < \frac{f_y}{E_s} \wedge \varepsilon_{s_i} \geq 0 \\ f_{s_i} \leftarrow E_s \cdot \varepsilon_{s_i} + 0.85 \cdot f_c \text{ otherwise} \end{array} \right. \\ f_s \end{array} \right. = \begin{pmatrix} 60 \\ 54.356 \end{pmatrix} \cdot \text{ksi}$$ *Stress in Reinforcement*

$$P_s := \left| \begin{array}{l} \text{for } i \in 0..\text{rows}(d_{bs}) - 1 \\ \quad P_{s_i} \leftarrow f_{s_i} \cdot A_{bs_i} \\ P_s \end{array} \right. = \begin{pmatrix} 70.686 \\ 42.691 \end{pmatrix} \cdot \text{kip}$$ *Force in Reinforcement*

$$P_{check} := \begin{pmatrix} \frac{\sum P_s}{T_a} \\ \left| \begin{array}{l} \text{"OK" if } \frac{\sum P_s}{T_a} \geq 0.95 \wedge \frac{\sum P_s}{T_a} \leq 1.05 \\ \text{"Reiterate" otherwise} \end{array} \right. \end{pmatrix} = \begin{pmatrix} 1.003 \\ \text{"OK"} \end{pmatrix}$$ *Check for Equilibrium*

$$M_s := \left| \begin{array}{l} \text{for } i \in 0..\text{rows}(d_{bs}) - 1 \\ \quad M_{s_i} \leftarrow P_{s_i} \cdot \left(d_{bs_i} - \frac{a}{2} \right) \\ M_s \end{array} \right. = \begin{pmatrix} 143.682 \\ 12.068 \end{pmatrix} \cdot \text{ft} \cdot \text{kip}$$ *Moment in Reinforcement*

$$\phi_b := \left| \begin{array}{l} 0.65 \text{ if } \max(\varepsilon_s) \leq 0.002 \\ 0.90 \text{ if } \max(\varepsilon_s) \geq 0.005 \\ 0.65 + \left(\max(\varepsilon_s) - 0.002 \right) \cdot \left(\frac{250}{3} \right) \text{ otherwise} \end{array} \right. = 0.9$$ *Load Reduction Factor*

$$\phi M_{n\_slab\_pos} := \frac{\phi_b \cdot \sum M_s}{b} = 140.17 \cdot \frac{\text{kip} \cdot \text{ft}}{\text{ft}}$$ *Nominal Moment Capacity of Slab*

$$\phi V_{n\_slab} := \phi_v \cdot 2 \cdot \lambda \cdot \sqrt{\frac{f_c}{psi}} \cdot psi \cdot d_{ts} = 32.456 \cdot \frac{kip}{ft}$$

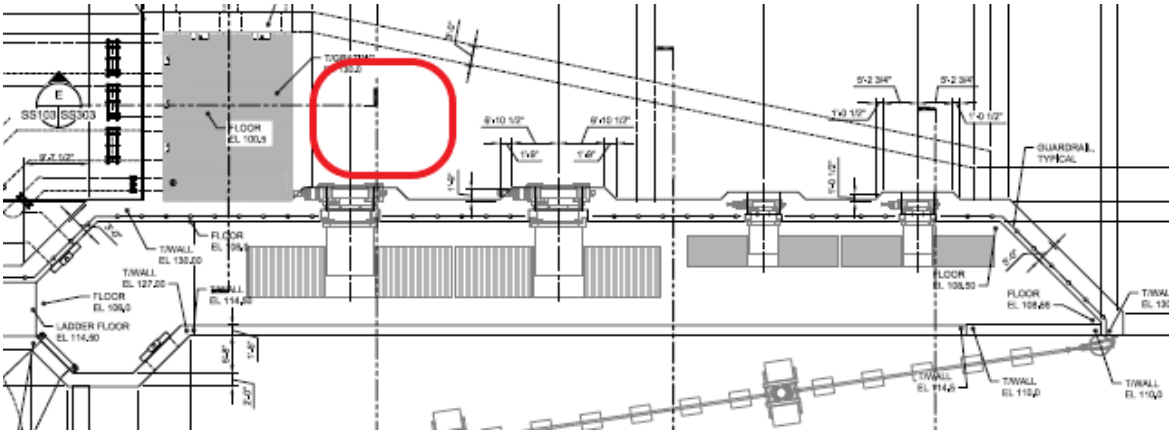
Nominal Shear Capacity of Slab (ACI 318 Section 22.5.5.1)

$$DCR_M := \frac{M_u}{\phi M_{n\_slab\_pos}} = 0.963$$

$$DCR_V := \frac{V_u}{\phi V_{n\_slab}} = 0.924$$

OK use #8@8 at upper layer of roof and #8@12" at lower layer of roof

Culvert Roof Design (lower half):



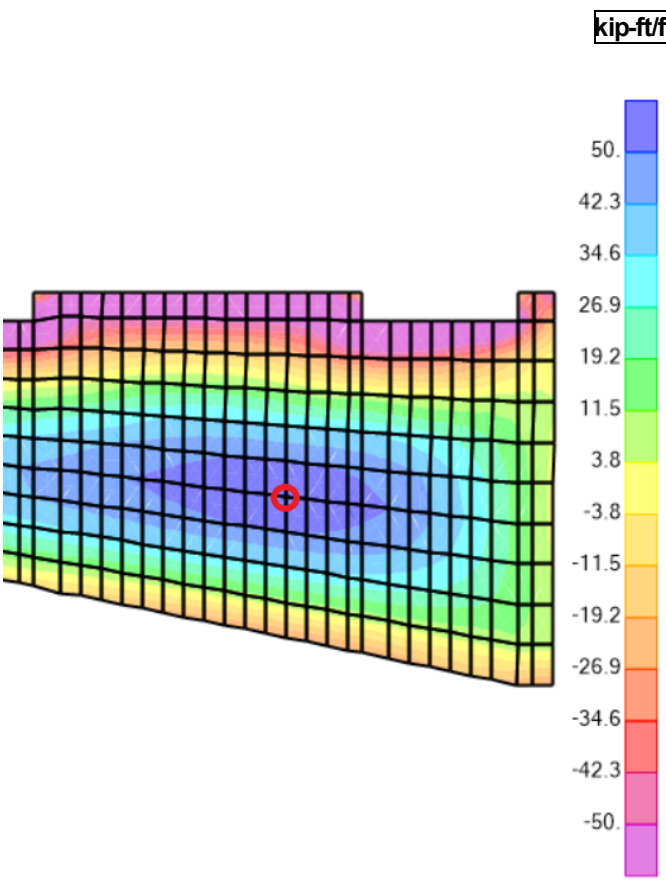


Figure 4: Culvert Roof (lower half) with region of maximum moment circled in red

Positive Moment (tension at top)

$M_u := 62kip \cdot \frac{ft}{ft}$ *Maximum moment from SAP2000*

$V_u := 14\frac{kip}{ft}$ *Maximum shear from SAP2000*

$c_{bs} := 4in$ Cover for tension rebar

$c_{ts} := 4in$ Cover for compression rebar

$b := 1ft$ *Assumed width of section*

$d_{bar\_t} := 1in$ *Diameter of Tensile Reinforcement (1 #8 @12in bar)*

$A_{ts} := \frac{12}{12}\frac{\pi}{4} \cdot d_{bar\_t}^{\ 2} = 0.785 \cdot in^2$ *Cross Sectional Area of Tensile Reinforcement*

$t_{roof} := 2.5ft$ *Thickness of roof*

$d_{ts} := t_{roof} - c_{bs} - 0.5d_{bar\_t} = 25.5 \cdot in$ *Depth of Tensile Reinforcement in the Slab*

$d_{bar\_c} := 1in$ *Diameter of Compression Reinforcement (1 #8@12in bar)*

$A_{cs} := \frac{12}{8}\frac{\pi}{4} \cdot d_{bar\_c}^{\ 2} = 1.178 \cdot in^2$ *Cross Sec Area of Compression Reinforcement #8@8"*

$d_{cs} := c_{ts} + 0.5 \cdot d_{bar\_c} = 4.5 \cdot in$ *Depth of Compression Reinforcement in the Slab*

$A_{bs} := \begin{pmatrix} A_{ts} \\ A_{cs} \end{pmatrix} = \begin{pmatrix} 0.785 \\ 1.178 \end{pmatrix} \cdot in^2$ *Cross Sectional Areas Reinforcement*

$d_{bs} := \begin{pmatrix} d_{ts} \\ d_{cs} \end{pmatrix} = \begin{pmatrix} 25.5 \\ 4.5 \end{pmatrix} \cdot in$ *Depths of Reinforcement*

$C_{TS.Slab} :=$ "OK" if $A_{ts} \geq 1in^2 \wedge A_{cs} \geq 1in^2$ = "OK"
otherwise
  "OK" if $\sum A_{bs} > 0.003 \cdot b \cdot t_{roof}$
  "Min Reinf. Not Met" otherwise

Minimum reinforcement (ACI 318-19-8-6-1)

$T_a := 112kip$ *Assumed Tension Force on Section*

$a := \frac{T_a}{0.85 \cdot f_c \cdot b} = 2.196 \cdot in$ *Width of Whitney Stress Block*

$\varepsilon_s :=$ for $i \in 0..rows(d_{bs}) - 1$ $= \begin{pmatrix} 0.025 \\ 1.918 \times 10^{-3} \end{pmatrix}$ *Strain in Reinforcement*
  $\varepsilon_{s_i} \leftarrow \frac{0.003}{\frac{a}{\beta_1}} \cdot \left(d_{bs_i} - \frac{a}{\beta_1}\right)$
  ε_s

$$\text{Check}_{\text{Tensile\_Strain}} := \begin{cases} \text{"OK"} & \text{if } \max(\varepsilon_s) \geq \max\left(\frac{f_y}{E_s}, 0.005\right) \\ \text{"NG - Too much reinforcement"} & \text{otherwise} \end{cases} = \text{"OK"}$$

$$f_s := \left| \begin{array}{l} \text{for } i \in 0..\text{rows}(d_{bs}) - 1 \\ \quad \left| \begin{array}{l} f_{s_i} \leftarrow f_y \text{ if } \varepsilon_{s_i} \geq \frac{f_y}{E_s} \\ f_{s_i} \leftarrow -f_y \text{ if } \varepsilon_{s_i} \leq \frac{-f_y}{E_s} \\ f_{s_i} \leftarrow E_s \cdot \varepsilon_{s_i} \text{ if } \varepsilon_{s_i} < \frac{f_y}{E_s} \wedge \varepsilon_{s_i} \geq 0 \\ f_{s_i} \leftarrow E_s \cdot \varepsilon_{s_i} + 0.85 \cdot f_c \text{ otherwise} \end{array} \right. \\ f_s \end{array} \right. = \begin{pmatrix} 60 \\ 55.618 \end{pmatrix} \cdot \text{ksi}$$

Stress in Reinforcement

$$P_s := \left| \begin{array}{l} \text{for } i \in 0..\text{rows}(d_{bs}) - 1 \\ \quad P_{s_i} \leftarrow f_{s_i} \cdot A_{bs_i} \\ P_s \end{array} \right. = \begin{pmatrix} 47.124 \\ 65.523 \end{pmatrix} \cdot \text{kip}$$

Force in Reinforcement

$$P_{\text{check}} := \begin{pmatrix} \frac{\sum P_s}{T_a} \\ \begin{cases} \text{"OK"} & \text{if } \frac{\sum P_s}{T_a} \geq 0.95 \wedge \frac{\sum P_s}{T_a} \leq 1.05 \\ \text{"Reiterate"} & \text{otherwise} \end{cases} \end{pmatrix} = \begin{pmatrix} 1.006 \\ \text{"OK"} \end{pmatrix}$$

Check for Equilibrium

$$M_s := \left| \begin{array}{l} \text{for } i \in 0..\text{rows}(d_{bs}) - 1 \\ \quad M_{s_i} \leftarrow P_{s_i} \cdot \left(d_{bs_i} - \frac{a}{2}\right) \\ M_s \end{array} \right. = \begin{pmatrix} 95.826 \\ 18.576 \end{pmatrix} \cdot \text{ft} \cdot \text{kip}$$

Moment in Reinforcement

$$\phi_b := \begin{cases} 0.65 & \text{if } \max(\varepsilon_s) \le 0.002 \\ 0.90 & \text{if } \max(\varepsilon_s) \ge 0.005 \\ 0.65 + \left(\max(\varepsilon_s) - 0.002\right)\cdot\left(\frac{250}{3}\right) & \text{otherwise} \end{cases} = 0.9$$ *Load Reduction Factor*

$$\phi M_{n\_roof\_pos} := \frac{\phi_b \cdot \sum M_s}{b} = 102.96 \cdot \frac{kip \cdot ft}{ft}$$ *Nominal Moment Capacity of Slab*

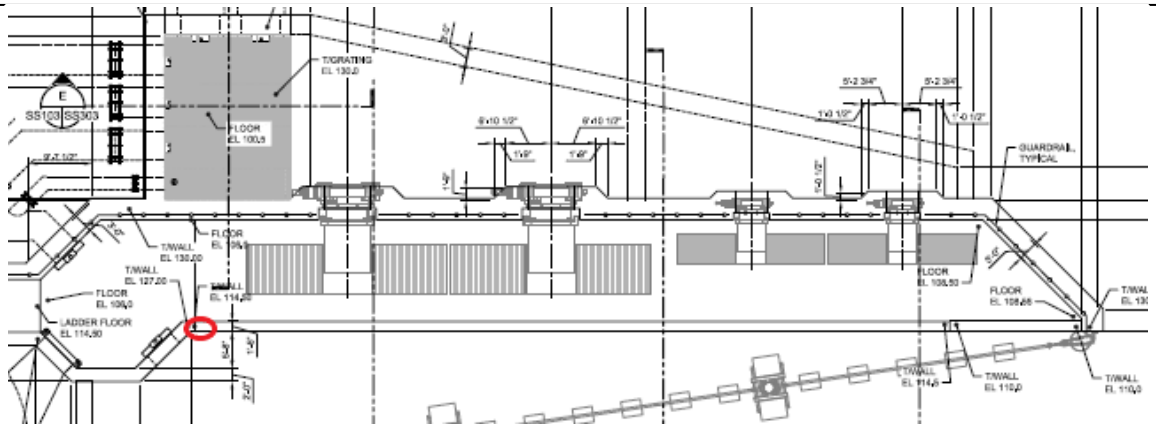
$$\phi V_{n\_roof} := \phi_v \cdot 2 \cdot \lambda \cdot \sqrt{\frac{f_c}{psi}} \cdot psi \cdot d_{ts} = 32.456 \cdot \frac{kip}{ft}$$ *Nominal Shear Capacity of Slab (ACI 318 Section 22.5.5.1)*

$$DCR_M := \frac{M_u}{\phi M_{n\_roof\_pos}} = 0.602$$

$$DCR_V := \frac{V_u}{\phi V_{n\_roof}} = 0.431$$

OK use #8@12 at bottom layer of roof and #8@8" at upper layer

SW Wall Design:



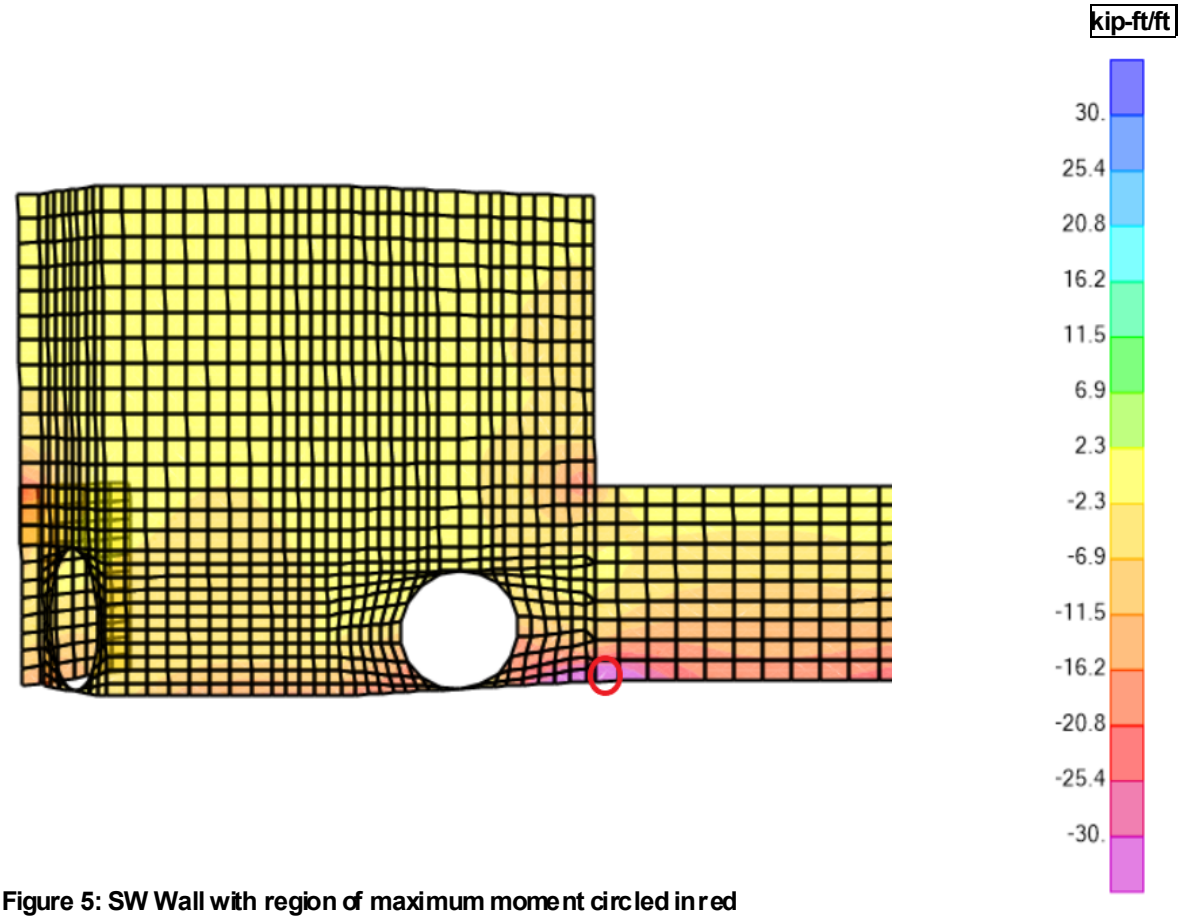


Figure 5: SW Wall with region of maximum moment circled in red

Positive Moment (tension at outer face of wall)

$M_u := 35.6\text{kip}\cdot\frac{\text{ft}}{\text{ft}}$ *Maximum moment from SAP2000*

$V_u := 12.1\frac{\text{kip}}{\text{ft}}$ *Maximum shear from SAP2000*

$c_{bs} := 4in$ Cover for tension rebar

$c_{ts} := 4in$ Cover for compression rebar

$b := 1ft$ *Assumed width of section*

$d_{bar\_t} := 1in$ *Diameter of Tensile Reinforcement (1 #8 @12in bar)*

$A_{ts} := \frac{12}{12}\frac{\pi}{4}\cdot d_{bar\_t}^{2} = 0.785\cdot in^2$ *Cross Sectional Area of Tensile Reinforcement*

$t_{wall} := 2ft$ *Thickness of wall*

$d_{ts} := t_{wall} - c_{bs} - 0.5d_{bar\_t} = 19.5\cdot in$ *Depth of Tensile Reinforcement in the Slab*

$d_{bar\_c} := 1in$ *Diameter of Compression Reinforcement (1 #8@12in bar)*

$A_{cs} := \frac{\pi}{4}\cdot d_{bar\_c}^{2} = 0.785\cdot in^2$ *Cross Sec Area of Compression Reinforcement*

$d_{cs} := c_{ts} + 0.5\cdot d_{bar\_c} = 4.5\cdot in$ *Depth of Compression Reinforcement in the Slab*

$A_{bs} := \begin{pmatrix} A_{ts} \\ A_{cs} \end{pmatrix} = \begin{pmatrix} 0.785 \\ 0.785 \end{pmatrix}\cdot in^2$ *Cross Sectional Areas Reinforcement*

$d_{bs} := \begin{pmatrix} d_{ts} \\ d_{cs} \end{pmatrix} = \begin{pmatrix} 19.5 \\ 4.5 \end{pmatrix}\cdot in$ *Depths of Reinforcement*

$C_{TS.Slab} :=$ "OK" if $A_{ts} \geq 1in^2 \wedge A_{cs} \geq 1in^2$ = "OK"
otherwise
  "OK" if $\sum A_{bs} > 0.003\cdot b\cdot t_{wall}$
  "Min Reinf. Not Met" otherwise

Minimum reinforcement (ACI 318-19-8-6-1)

$T_a := 94kip$ *Assumed Tension Force on Section*

$a := \frac{T_a}{0.85\cdot f_{c}\cdot b} = 1.843\cdot in$ *Width of Whitney Stress Block*

$\varepsilon_s :=$ for $i \in 0..rows(d_{bs}) - 1$ $= \begin{pmatrix} 0.022 \\ 2.86\times 10^{-3} \end{pmatrix}$ *Strain in Reinforcement*
  $\varepsilon_{s_i} \leftarrow \frac{0.003}{\frac{a}{\beta_1}}\cdot\left(d_{bs_i} - \frac{a}{\beta_1}\right)$
  ε_s

$Check_{Tensile\_Strain} :=$ "OK" if $\max(\varepsilon_s) \geq \max\left(\frac{f_y}{E_s}, 0.005\right)$ = "OK"
"NG - Too much reinforcement" otherwise

$$f_s := \left| \begin{array}{l} \text{for } i \in 0..\text{rows}(d_{bs}) - 1 \\ \quad \left| \begin{array}{l} f_{s_i} \leftarrow f_y \text{ if } \varepsilon_{s_i} \geq \frac{f_y}{E_s} \\ f_{s_i} \leftarrow -f_y \text{ if } \varepsilon_{s_i} \leq \frac{-f_y}{E_s} \\ f_{s_i} \leftarrow E_s \cdot \varepsilon_{s_i} \text{ if } \varepsilon_{s_i} < \frac{f_y}{E_s} \wedge \varepsilon_{s_i} \geq 0 \\ f_{s_i} \leftarrow E_s \cdot \varepsilon_{s_i} + 0.85 \cdot f_c \text{ otherwise} \end{array} \right. \\ f_s \end{array} \right. = \begin{pmatrix} 60 \\ 60 \end{pmatrix} \cdot \text{ksi}$$

Stress in Reinforcement

$$P_s := \left| \begin{array}{l} \text{for } i \in 0..\text{rows}(d_{bs}) - 1 \\ \quad P_{s_i} \leftarrow f_{s_i} \cdot A_{bs_i} \\ P_s \end{array} \right. = \begin{pmatrix} 47.124 \\ 47.124 \end{pmatrix} \cdot \text{kip}$$

Force in Reinforcement

$$P_{check} := \left(\begin{array}{l} \frac{\sum P_s}{T_a} \\ \left| \begin{array}{l} \text{"OK" if } \frac{\sum P_s}{T_a} \geq 0.95 \wedge \frac{\sum P_s}{T_a} \leq 1.05 \\ \text{"Reiterate" otherwise} \end{array} \right. \end{array} \right) = \begin{pmatrix} 1.003 \\ \text{"OK"} \end{pmatrix}$$

Check for Equilibrium

$$M_s := \left| \begin{array}{l} \text{for } i \in 0..\text{rows}(d_{bs}) - 1 \\ \quad M_{s_i} \leftarrow P_{s_i} \cdot \left(d_{bs_i} - \frac{a}{2} \right) \\ M_s \end{array} \right. = \begin{pmatrix} 72.957 \\ 14.052 \end{pmatrix} \cdot \text{ft} \cdot \text{kip}$$

Moment in Reinforcement

$$\phi_b := \left| \begin{array}{l} 0.65 \text{ if } \max(\varepsilon_s) \leq 0.002 \\ 0.90 \text{ if } \max(\varepsilon_s) \geq 0.005 \\ 0.65 + (\max(\varepsilon_s) - 0.002) \cdot \left(\frac{250}{3} \right) \text{ otherwise} \end{array} \right. = 0.9$$

Load Reduction Factor

$$\phi M_{n\_wall\_pos} := \frac{\phi_b \cdot \sum M_s}{b} = 78.31 \cdot \frac{\text{kip} \cdot \text{ft}}{\text{ft}}$$

Nominal Moment Capacity of Slab

$$\phi V_{n\_wall} := \phi_v \cdot 2 \cdot \lambda \cdot \sqrt{\frac{f_c}{psi}} \cdot psi \cdot d_{ts} = 24.819 \cdot \frac{kip}{ft}$$

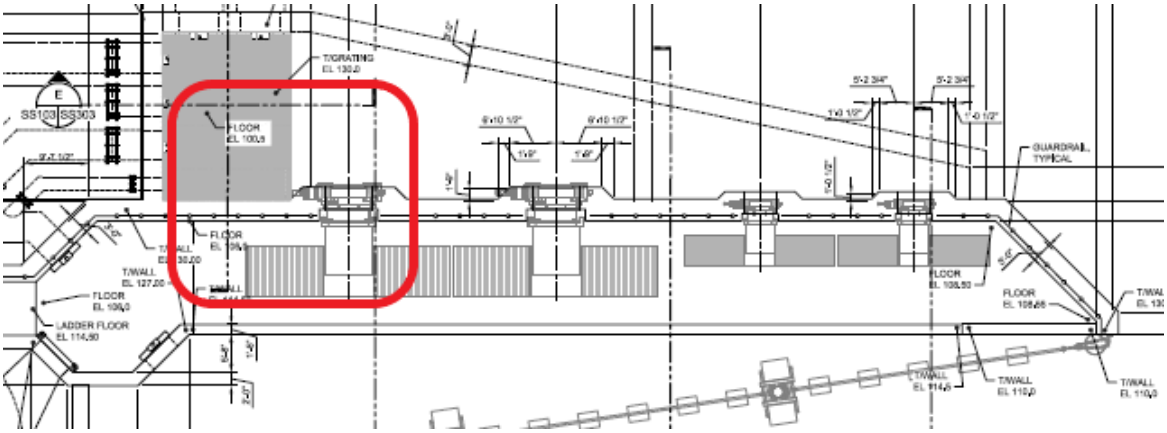
Nominal Shear Capacity of Slab (ACI 318 Section 22.5.5.1)

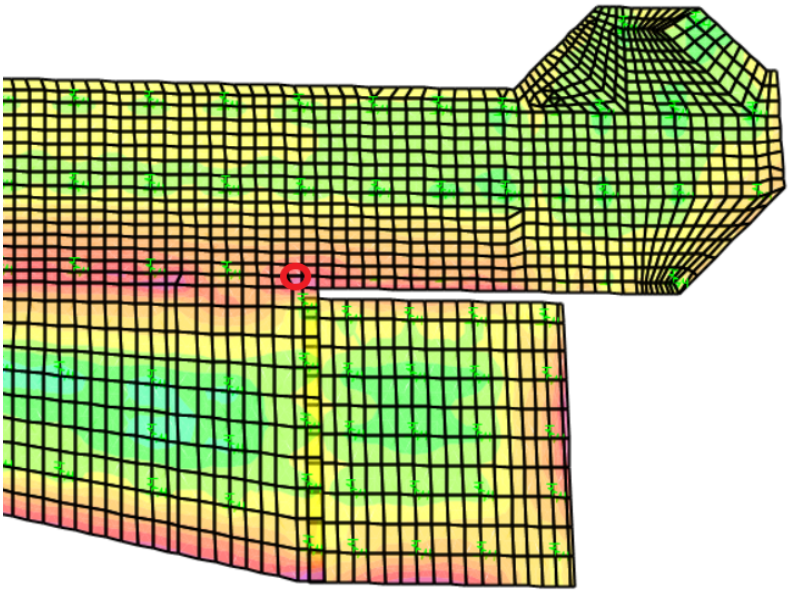
$$DCR_M := \frac{M_u}{\phi M_{n\_wall\_pos}} = 0.455$$

$$DCR_V := \frac{V_u}{\phi V_{n\_wall}} = 0.488$$

Use #8@12in for both faces

Slab Design:





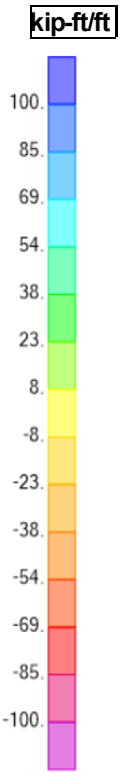


Figure 6: Slab with region of maximum moment circled in red

Positive Moment (tension at bottom)

$c_{bs} := 6in$ Cover for tension rebar

$c_{ts} := 6in$ Cover for compression rebar

$b := 1ft$ *Assumed width of section*

$d_{bar\_t} := 1.128in$ *Diameter of Tensile Reinforcement (1 #9 @8in bar)*

$A_{ts} := \frac{12}{8} 1in^2 = 1.5 \cdot in^2$ *Cross Sectional Area of Tensile Reinforcement*

$t_{slab} := 3ft$ *Thickness of slab*

$d_{ts} := t_{slab} - c_{bs} - 0.5 d_{bar\_t} = 29.436 \cdot in$ *Depth of Tensile Reinforcement in the Slab*

$d_{bar\_c} := 1.128in$ *Diameter of Compression Reinforcement (1 #9@8" bar)*

$A_{cs} := \frac{12}{8} 1in^2 = 1.5 \cdot in^2$ *Cross Sec Area of Compression Reinforcement*

$d_{cs} := c_{ts} + 0.5 \cdot d_{bar\_c} = 6.564 \cdot in$ *Depth of Compression Reinforcement in the Slab*

$A_{bs} := \begin{pmatrix} A_{ts} \\ A_{cs} \end{pmatrix} = \begin{pmatrix} 1.5 \\ 1.5 \end{pmatrix} \cdot in^2$ *Cross Sectional Areas Reinforcement*

$d_{bs} := \begin{pmatrix} d_{ts} \\ d_{cs} \end{pmatrix} = \begin{pmatrix} 29.436 \\ 6.564 \end{pmatrix} \cdot in$ *Depths of Reinforcement*

$C_{TS.Slab} :=$ "OK" if $A_{ts} \geq 1in^2 \wedge A_{cs} \geq 1in^2$ = "OK"
otherwise
  "OK" if $\sum A_{bs} > 0.003 \cdot b \cdot t_{slab}$
  "Min Reinf. Not Met" otherwise

Minimum reinforcement (ACI 318-19-8-6-1)

$T_a := 167kip$ *Assumed Tension Force on Section*

$a := \frac{T_a}{0.85 \cdot f_c \cdot b} = 3.275 \cdot in$ *Width of Whitney Stress Block*

$\varepsilon_s :=$ for $i \in 0 .. rows(d_{bs}) - 1$ $= \begin{pmatrix} 0.019 \\ 1.811 \times 10^{-3} \end{pmatrix}$ *Strain in Reinforcement*
  $\varepsilon_{s_i} \leftarrow \frac{0.003}{\frac{a}{\beta_1}} \cdot \left(d_{bs_i} - \frac{a}{\beta_1} \right)$
  ε_s

$$Check_{Tensile\_Strain} := \begin{cases} \text{"OK"} & \text{if } \max(\varepsilon_s) \geq \max\left(\frac{f_y}{E_s}, 0.005\right) \\ \text{"NG - Too much reinforcement"} & \text{otherwise} \end{cases} = \text{"OK"}$$

$$f_s := \left| \begin{array}{l} \text{for } i \in 0..\text{rows}(d_{bs}) - 1 \\ \quad \left| \begin{array}{l} f_{s_i} \leftarrow f_y \text{ if } \varepsilon_{s_i} \geq \frac{f_y}{E_s} \\ f_{s_i} \leftarrow -f_y \text{ if } \varepsilon_{s_i} \leq \frac{-f_y}{E_s} \\ f_{s_i} \leftarrow E_s \cdot \varepsilon_{s_i} \text{ if } \varepsilon_{s_i} < \frac{f_y}{E_s} \wedge \varepsilon_{s_i} \geq 0 \\ f_{s_i} \leftarrow E_s \cdot \varepsilon_{s_i} + 0.85 \cdot f_c \text{ otherwise} \end{array} \right. \\ f_s \end{array} \right. = \begin{pmatrix} 60 \\ 52.518 \end{pmatrix} \cdot \text{ksi}$$

Stress in Reinforcement

$$P_s := \left| \begin{array}{l} \text{for } i \in 0..\text{rows}(d_{bs}) - 1 \\ \quad P_{s_i} \leftarrow f_{s_i} \cdot A_{bs_i} \\ P_s \end{array} \right. = \begin{pmatrix} 90.000 \\ 78.778 \end{pmatrix} \cdot \text{kip}$$

Force in Reinforcement

$$P_{check} := \begin{pmatrix} \frac{\sum P_s}{T_a} \\ \begin{cases} \text{"OK"} & \text{if } \frac{\sum P_s}{T_a} \geq 0.95 \wedge \frac{\sum P_s}{T_a} \leq 1.05 \\ \text{"Reiterate"} & \text{otherwise} \end{cases} \end{pmatrix} = \begin{pmatrix} 1.011 \\ \text{"OK"} \end{pmatrix}$$

Check for Equilibrium

$$M_s := \left| \begin{array}{l} \text{for } i \in 0..\text{rows}(d_{bs}) - 1 \\ \quad M_{s_i} \leftarrow P_{s_i} \cdot \left(d_{bs_i} - \frac{a}{2}\right) \\ M_s \end{array} \right. = \begin{pmatrix} 208.491 \\ 32.343 \end{pmatrix} \cdot \text{ft} \cdot \text{kip}$$

Moment in Reinforcement

$$\phi_b := \begin{cases} 0.65 & \text{if } \max(\varepsilon_s) \leq 0.002 \\ 0.90 & \text{if } \max(\varepsilon_s) \geq 0.005 \\ 0.65 + \left(\max(\varepsilon_s) - 0.002\right) \cdot \left(\frac{250}{3}\right) & \text{otherwise} \end{cases} = 0.9$$

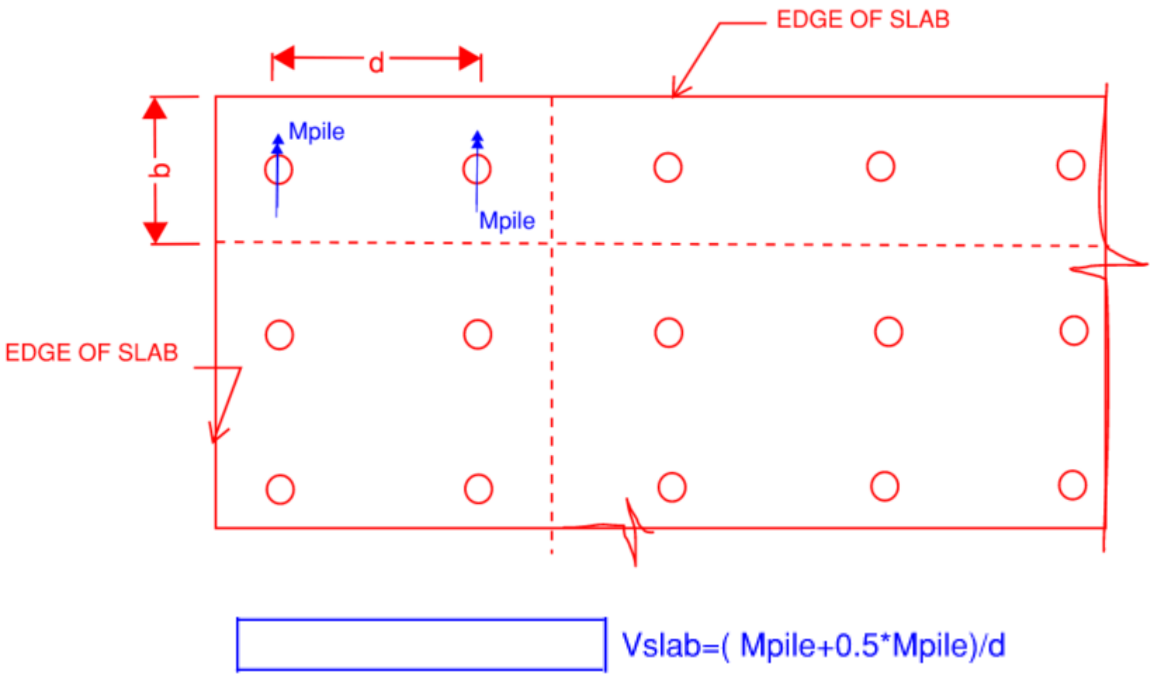
Load Reduction Factor

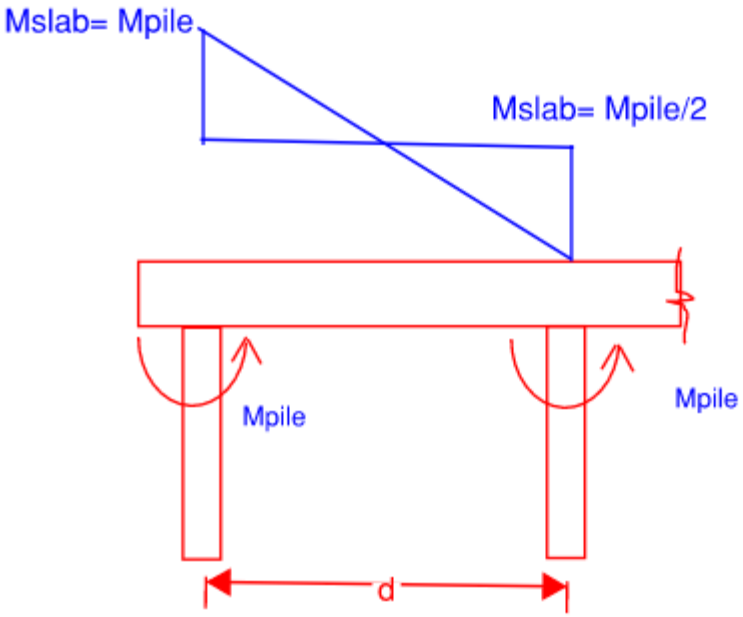
$$\phi M_{n\_slab\_pos} := \frac{\phi_b \cdot \sum M_s}{b} = 216.75 \cdot \frac{kip \cdot ft}{ft}$$

Nominal Moment Capacity of Slab

$$\phi V_{n\_slab} := \phi_v \cdot 2 \cdot \lambda \cdot \sqrt{\frac{f_c}{psi}} \cdot psi \cdot d_{ts} = 37.466 \cdot \frac{kip}{ft}$$

Nominal Shear Capacity of Slab (ACI 318 Section 22.5.5.1)





$M_{pile} := 12700\,kip \cdot in$ Plastic moment capacity of 24" steel pile-See next Section

$b := 7\,ft + 5\,in = 7.417\,ft$ Tributary width from middle of slab edge

$d := 6\,ft + 9\,in = 6.75\,ft$ Distance between two piles

$M_{slab} := \frac{M_{pile}}{b} = 142.697 \frac{1}{ft} \cdot kip \cdot ft$ *Ultimate moment on pile*

$V_{slab} := \frac{\frac{M_{pile} + 0.5 M_{pile}}{d}}{b} = 31.71 \frac{1}{ft} \cdot kip$ *Ultimate shear on pile*

$DCR_M := \frac{M_{slab}}{\phi M_{n\_slab\_pos}} = 0.658$

$DCR_V := \frac{V_{slab}}{\phi V_{n\_slab}} = 0.846$

Use #9@8in with 3 ft thick slab for top and bottom layers each way

Pile design

$D_p := 24in$ *Outer diameter of steel pile*

$t_p := 0.5in$ *Thickness of steel pile*

$Z_p := \frac{\left[\frac{4}{3} \cdot (0.5 \cdot D_p)^3 - \frac{4}{3}(0.5D_p - t_p)^3\right]}{\pi\left[(0.5D_p)^2 - (0.5D_p - t_p)^2\right]} \cdot \left[\pi\left[(0.5D_p)^2 - (0.5D_p - t_p)^2\right]\right] = 276.167 \cdot in^3$ *Plastic modulus*

$S_y := \pi \cdot \frac{\left[D_p{}^4 - (D_p - t_p)^4\right]}{32 D_p} = 109.612 \cdot in^3$ *Section modulus*

$\frac{D_p}{t_p} = 48$ *Slenderness ratio of pile*

$f_{yp} := 46ksi$ *Yield strength of pile*

$E_p := 29000ksi$ *Bulk modulus of pile*

$\frac{0.45 \cdot E_p}{f_{yp}} = 283.696$ **> Dp/tp** *Section F8, AISC Steel Manual*

Hence we can use Section F8 of AISC-360

For 65% the maximum shear on piles was considered to calibrate the equivalent lateral spring between SAP2000 and LPile. The following shows the iteration results.

First iteration

$V_{pmax} := 80.592kip$ *Maximum lateral shear from SAP2000*

$u_{lateral} := 1.436in$ *Maximum lateral deflection from LPile*

$K_p := \frac{V_{pmax}}{u_{lateral}} = 673.471 \cdot klf$ *Lateral stiffness of pile*

Second iteration (take Kp=660 klf)

$V_{pmax} := 80.844kip$ *Maximum lateral shear from SAP2000*

$u_{lateral} := 1.435in$ *Maximum lateral deflection from LPile*

$K_p := \frac{V_{pmax}}{u_{lateral}} = 676.047 \cdot klf$ *Lateral stiffness of pile*

Converges, hence accepted

1. Yielding

$M_{n1} := f_{yp} \cdot Z_p = 1.27 \times 10^4 \cdot in \cdot kip$ *Nominal flexural strength due to yielding*

2. Local Buckling

$\lambda_r := 0.31 \frac{E_p}{f_{yp}} = 195.435$ > Dp/tp *Table B4.1b, AISC 360. Hence section is slender*

$M_{n2} := \left[\frac{0.33E_p}{\left(\frac{D_p}{t_p}\right)}\right] \cdot S_y = 2.185 \times 10^4 \cdot in \cdot kip$ *Nominal flexural strength due to local buckling*

Therefore the minimum of the above two values gives us

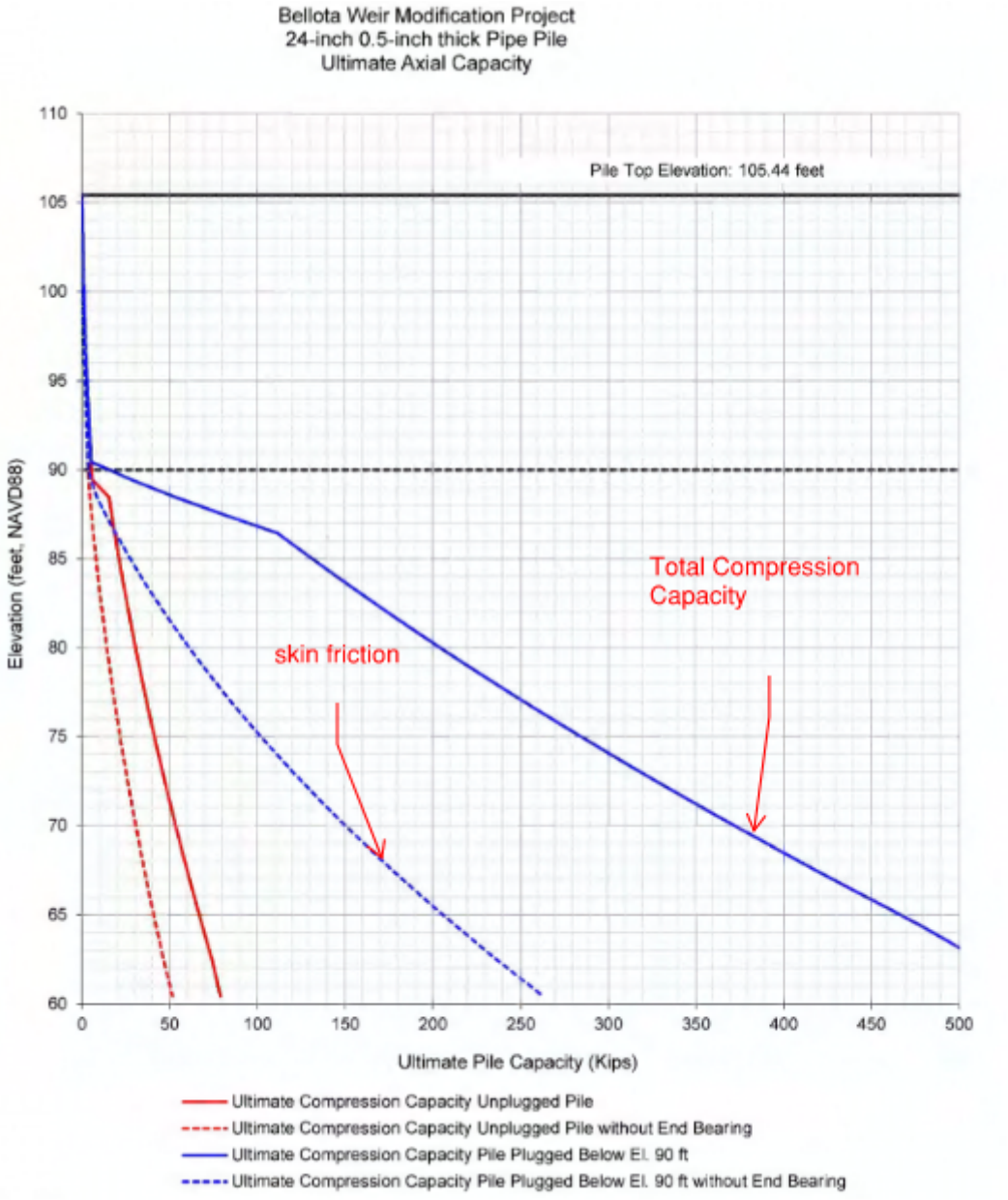
$M_n := \min(M_{n1}, M_{n2}) = 1.27 \times 10^4 \cdot in \cdot kip$

From LPile, the value of maximum moment on the pile is as follows

$M_y := 10058kip \cdot in$ which is lesser than the pile capacity

$DCR_{pile} := \frac{M_y}{0.9M_n} = 0.88$ **Hence OK**

Axial Check



$P_{tmax} := 185.433\,kip$ *Maximum tensile force on pile due to seismic*

$P_{cmax} := 219.935\,kip$ *Maximum compressive force on pile due to normal operation*

The top graph is from geotech report. The 80% of skin friction can be used as tensile capacity in pile

If we look at the above graph, for a pile 70ft long, the elevation would be at around 35 ft for which the corresponding value of skin friction and ultimate pile capacity would be much higher than both the maximum tensile and compressive forces on the pile respectively.

For intake structure the governing factor for pile length would be lateral capacity of piles under seismic loads. In 90% design phase more accurate iteration procedure would follow to elaborate the pile design and optimize the pile length.

Fish Passage Design

Descriptions:

The following calculations provide the design of Intake Walls, Slabs and Pile foundations for 65% design phase.

The lateral soil pressure and soil seismic loads were used according to the recommendation given in Geotechnical Report for Bellota Project. According to geotech report the seismic load parameters were obtained based on ASCE7-16 with PGA equal to 0.18g. The load combination used for this structure is from following table per EM 1110-2-2104.

Load Case	Load Description	Load Category	Factored Loads
R1	Normal Operating	Usual	$2.2(D + EH + EV + Hs_U)$
R2A	Normal Operating + Surcharge	Unusual	$1.6(D + EH + EV + Hs_U + ES_N)$
R2B	Normal Operating + Short Duration Water Load	Unusual	$1.6(D + EH + EV + Hs_N)$
R3A	Normal Operating + OBE	Unusual	$1.5(D + EH + EV + Hs_U + EQ)$
R3B	Normal Operating + MDE	Extreme (Strength)	$(1.2 \text{ or } 0.9)D + (1.5 \text{ or } 0.5)EH + (1.35 \text{ or } 1.0)EV + 1.0\ Hs_U + 1.25EQ$
R4	Normal Operating + Maximum Water	Extreme (Strength)	$(1.2 \text{ or } 0.9)D + (1.5 \text{ or } 0.5)EH + (1.35 \text{ or } 1.0)EV + 1.3Hs_X$

Source: EM 1110-2-2104

Where: D: Dead Load; EH: Lateral Earth Pressure; EV: Vertical Earth Load; ES: Surcharge Load; HS_N: Normal Hydroststaic Load; HS_U: Hydroststaic Load during Flood; EQ: earthquake Load;

References:

- EM 1110-2-2104, Strength Design for Reinforced Concrete Hydraulic Structures
- ASCE/SEI 7-16 Code
- International Building Code IBC 2018
- ACI318-19
- Geotechnical Investigation Report-Bellota Weir Modification Project

Assumptions:

The intake structure is modeled in SAP2000 program including all the pressures such as active, at-rest and seismic soil pressure, hydrostatic and hydrodynamic pressure; and response spectrum analysis was performed for seismic analysis. Per EM 1110-2-2104 once response spectrum analysis is used, the soil load factor can be reduced to 1.0, which would be the case for this structure. For the design both 100-year flood as unusual and 500 year flood as extreme load case were also used.
For 65% design phase the maximum shear on piles were used for design the piles. Therefore one group effect value was used for critical pile and equivalent spring stiffness was calibrated based on critical pile using the iteration process between SAP2000 and LPile programs. For 90% design the effect of each pile would be evaluated individually and equivalent stiffness will be calibrated for each pile based on location and associated shear forces.
4" concrete cover is provided for both faces of walls and slabs for corrosion protection purposes. For all structural concrete, 5000 psi concrete will be used.
The following reported demands from SAP2000 is representing the maximum absolute value among all the defined load combinations

Material Properties:

$f_c := 5000psi$ — Assumed concrete compressive strength

$\gamma_c := 150pcf$ — Concrete unit weight

$E_c := 57000 \cdot \sqrt{\frac{f_c \cdot in^2}{lbf}} \cdot \frac{lbf}{in^2} = 4.031 \times 10^6 \cdot \frac{lbf}{in^2}$ — Concrete elastic modulus (ACI 318)

$$\beta_1 := \begin{cases} 0.85 & \text{if } f_c \le 4000psi \\ \max\left[0.85 - 0.05 \cdot \left(\frac{f_c - 4000psi}{1000psi}\right), 0.65\right] & \text{otherwise} \end{cases} = 0.8$$

Compression block factor (ACI 318)

$\phi_v := 0.75$ — Shear strength reduction factor

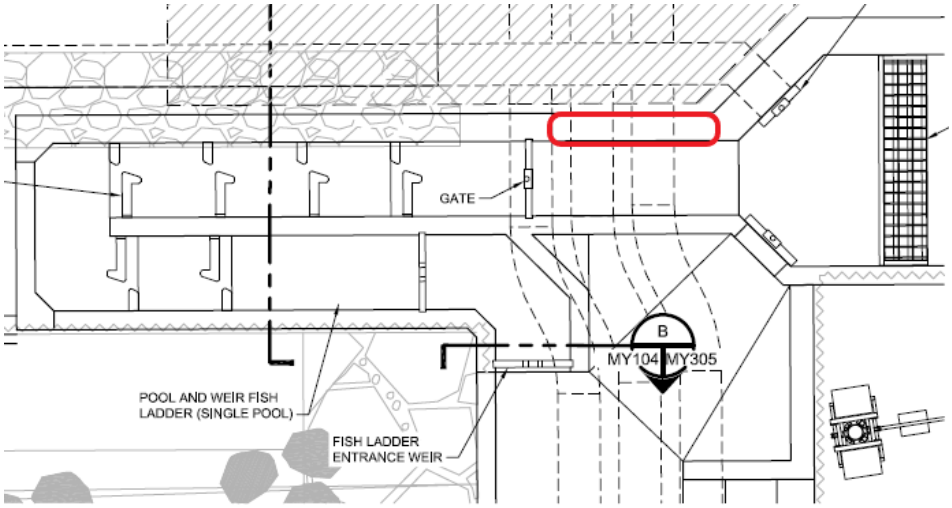
$\lambda := 1$ — Normal weight concrete factor

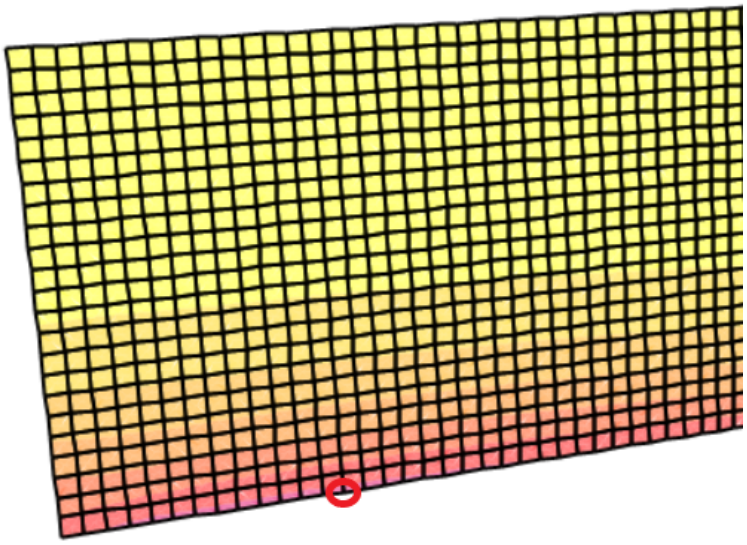
$f_y := 60ksi$ — Assumed yield strength of rebar

$E_s := 29000ksi$ — Steel elastic modulus

Check Strength Capacity of Concrete Components:

2.5 ft thick wall (outer wall) design:





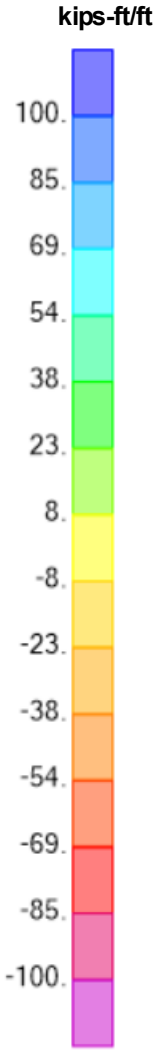


Figure 1: 2.5 ft thick wall with region of maximum moment circled in red

Positive Moment (tension at outer face)

$M_u := 127kip \cdot \frac{ft}{ft}$ *Maximum Moment from SAP2000*

$V_u := 25 \frac{kip}{ft}$ *Maximum Shear from SAP2000*

$c_{bs} := 4in$ *Cover for tension rebar*

$c_{ts} := 4in$ *Cover for compression rebar*

$b := 1ft$ *Assumed width of section*

$d_{bar\_t} := 1in$ *Diameter of Tensile Reinforcement (1 #8 @8in bar)*

$A_{ts} := \frac{12}{8} \frac{\pi}{4} \cdot d_{bar\_t}^2 = 1.178 \cdot in^2$ *Cross Sectional Area of Tensile Reinforcement*

$t_{wall} := 2.5ft$ *Thickness of wall*

$d_{ts} := t_{wall} - c_{bs} - 0.5 d_{bar\_t} = 25.5 \cdot in$ *Depth of Tensile Reinforcement in the Slab*

$d_{bar\_c} := 1in$ *Diameter of Compression Reinforcement (1 #8@12in bar)*

$A_{cs} := \frac{\pi}{4} \cdot d_{bar\_c}^2 = 0.785 \cdot in^2$ *Cross Sec Area of Compression Reinforcement*

$d_{cs} := c_{ts} + 0.5 \cdot d_{bar\_c} = 4.5 \cdot in$ *Depth of Compression Reinforcement in the Slab*

$A_{bs} := \begin{pmatrix} A_{ts} \\ A_{cs} \end{pmatrix} = \begin{pmatrix} 1.178 \\ 0.785 \end{pmatrix} \cdot in^2$ *Cross Sectional Areas Reinforcement*

$d_{bs} := \begin{pmatrix} d_{ts} \\ d_{cs} \end{pmatrix} = \begin{pmatrix} 25.5 \\ 4.5 \end{pmatrix} \cdot in$ *Depths of Reinforcement*

$C_{TS.Slab} := \begin{cases} \text{"OK"} & \text{if } A_{ts} \geq 1in^2 \wedge A_{cs} \geq 1in^2 \\ \text{otherwise} & \\ \quad \begin{cases} \text{"OK"} & \text{if } \sum A_{bs} > 0.003 \cdot b \cdot t_{wall} \\ \text{"Min Reinf. Not Met"} & \text{otherwise} \end{cases} & \end{cases} = \text{"OK"}$

Minimum reinforcement (ACI 318-19-8-6-1)

$T_a := 113kip$ *Assumed Tension Force on Section*

$a := \frac{T_a}{0.85 \cdot f_c \cdot b} = 2.216 \cdot in$ *Width of Whitney Stress Block*

$$\varepsilon_s := \left| \begin{array}{l} \text{for } i \in 0..\text{rows}(d_{bs}) - 1 \\ \quad \varepsilon_{s_i} \leftarrow \dfrac{0.003}{\dfrac{a}{\beta_1}} \cdot \left(d_{bs_i} - \dfrac{a}{\beta_1} \right) \\ \varepsilon_s \end{array} \right. = \begin{pmatrix} 0.025 \\ 1.874 \times 10^{-3} \end{pmatrix}$$ *Strain in Reinforcement*

$$\text{Check}_{\text{Tensile\_Strain}} := \left| \begin{array}{l} \text{"OK"} \quad \text{if } \max(\varepsilon_s) \geq \max\left(\dfrac{f_y}{E_s}, 0.005 \right) \\ \text{"NG - Too much reinforcement"} \quad \text{otherwise} \end{array} \right. = \text{"OK"}$$

$$f_s := \left| \begin{array}{l} \text{for } i \in 0..\text{rows}(d_{bs}) - 1 \\ \quad \left| \begin{array}{l} f_{s_i} \leftarrow f_y \quad \text{if } \varepsilon_{s_i} \geq \dfrac{f_y}{E_s} \\ f_{s_i} \leftarrow -f_y \quad \text{if } \varepsilon_{s_i} \leq \dfrac{-f_y}{E_s} \\ f_{s_i} \leftarrow E_s \cdot \varepsilon_{s_i} \quad \text{if } \varepsilon_{s_i} < \dfrac{f_y}{E_s} \wedge \varepsilon_{s_i} \geq 0 \\ f_{s_i} \leftarrow E_s \cdot \varepsilon_{s_i} + 0.85 \cdot f_c \quad \text{otherwise} \end{array} \right. \\ f_s \end{array} \right. = \begin{pmatrix} 60 \\ 54.356 \end{pmatrix} \cdot \text{ksi}$$ *Stress in Reinforcement*

$$P_s := \left| \begin{array}{l} \text{for } i \in 0..\text{rows}(d_{bs}) - 1 \\ \quad P_{s_i} \leftarrow f_{s_i} \cdot A_{bs_i} \\ P_s \end{array} \right. = \begin{pmatrix} 70.686 \\ 42.691 \end{pmatrix} \cdot \text{kip}$$ *Force in Reinforcement*

$$P_{\text{check}} := \begin{pmatrix} \dfrac{\sum P_s}{T_a} \\ \left| \begin{array}{l} \text{"OK"} \quad \text{if } \dfrac{\sum P_s}{T_a} \geq 0.95 \wedge \dfrac{\sum P_s}{T_a} \leq 1.05 \\ \text{"Reiterate"} \quad \text{otherwise} \end{array} \right. \end{pmatrix} = \begin{pmatrix} 1.003 \\ \text{"OK"} \end{pmatrix}$$ *Check for Equilibrium*

$$M_s := \left| \begin{array}{l} \text{for } i \in 0..\text{rows}(d_{bs}) - 1 \\ \quad M_{s_i} \leftarrow P_{s_i} \cdot \left(d_{bs_i} - \frac{a}{2} \right) \\ M_s \end{array} \right. = \begin{pmatrix} 143.682 \\ 12.068 \end{pmatrix} \cdot \text{ft} \cdot \text{kip}$$ *Moment in Reinforcement*

$$\phi_b := \left| \begin{array}{ll} 0.65 & \text{if } \max(\varepsilon_s) \le 0.002 \\ 0.90 & \text{if } \max(\varepsilon_s) \ge 0.005 \\ 0.65 + (\max(\varepsilon_s) - 0.002) \cdot \left(\frac{250}{3} \right) & \text{otherwise} \end{array} \right. = 0.9$$ *Load Reduction Factor*

$$\phi M_{n\_wall\_pos} := \frac{\phi_b \cdot \sum M_s}{b} = 140.17 \cdot \frac{\text{kip} \cdot \text{ft}}{\text{ft}}$$ *Nominal Moment Capacity of Slab*

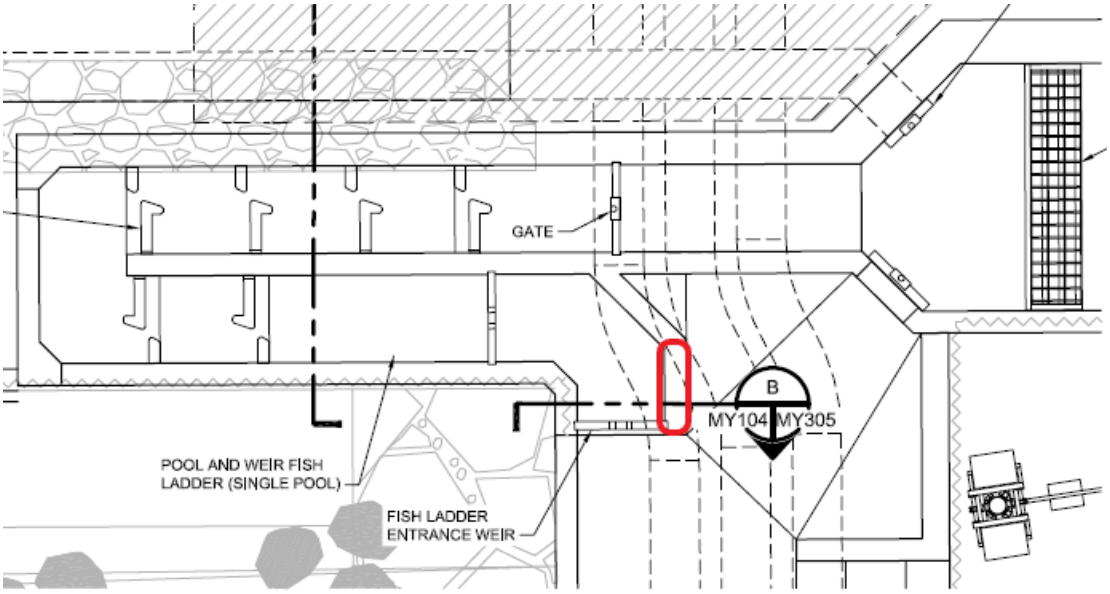
$$\phi V_{n\_wall} := \phi_v \cdot 2 \cdot \lambda \cdot \sqrt{\frac{f_c}{\text{psi}}} \cdot \text{psi} \cdot d_{ts} = 32.456 \cdot \frac{\text{kip}}{\text{ft}}$$ *Nominal Shear Capacity of Slab (ACI 318 Section 22.5.5.1)*

$$DCR_M := \frac{M_u}{\phi M_{n\_wall\_pos}} = 0.906$$

$$DCR_V := \frac{V_u}{\phi V_{n\_wall}} = 0.77$$

OK use #8@8 at outer face and #8@12 at inner face

2ft thick Wall Design:



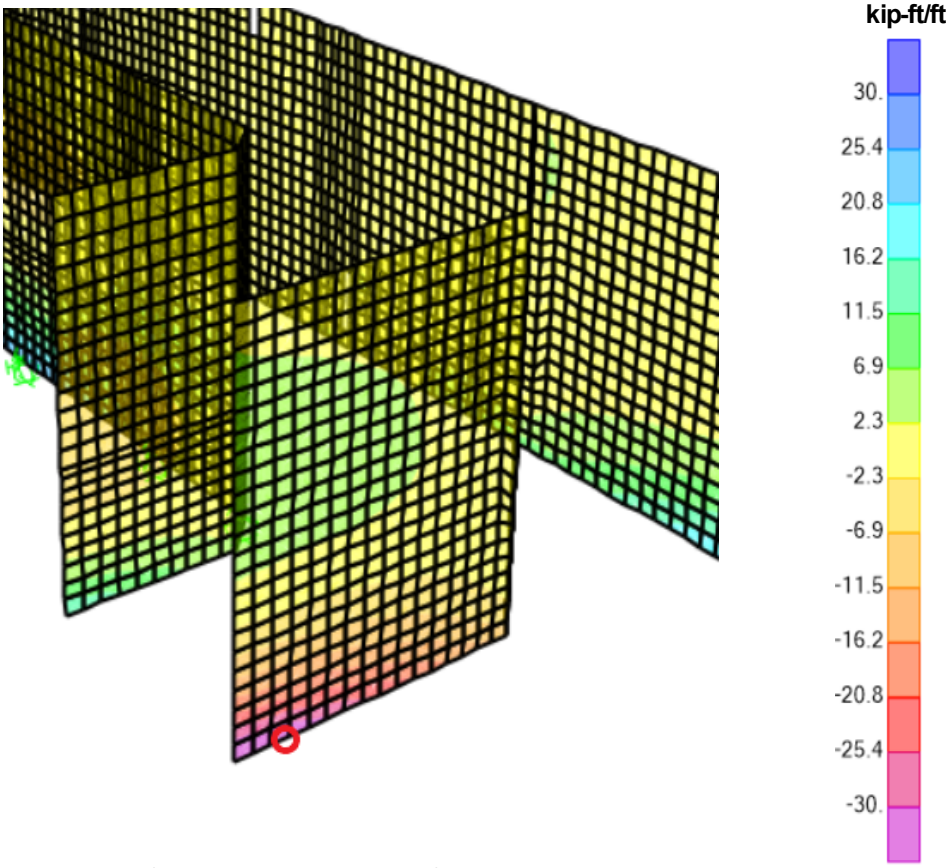


Figure 2: 2.5 ft thick wall with region of maximum moment circled in red

Positive Moment (tension at outer face); Moment about horizontal axis is higher than about vertical

$M_u := 38.09\text{kip}\cdot\frac{\text{ft}}{\text{ft}}$ *Maximum Moment from SAP2000*

$V_u := 12.4\frac{\text{kip}}{\text{ft}}$ *Maximum Shear from SAP2000*

$c_{bs} := 4\text{in}$ Cover for tension rebar

$c_{ts} := 4\text{in}$ Cover for compression rebar

$b := 1\text{ft}$ *Assumed width of section*

$d_{bar\_t} := 1\text{in}$ *Diameter of Tensile Reinforcement (1 #8 @12in bar)*

$A_{ts} := \frac{\pi}{4}\cdot d_{bar\_t}^{\,2} = 0.785\cdot\text{in}^2$ *Cross Sectional Area of Tensile Reinforcement*

$t_{wall} := 2\text{ft}$ *Thickness of wall*

$d_{ts} := t_{wall} - c_{bs} - 0.5d_{bar\_t} = 19.5\cdot\text{in}$ *Depth of Tensile Reinforcement in the Slab*

$d_{bar\_c} := 1\text{in}$ *Diameter of Compression Reinforcement (1 #8@12in bar)*

$A_{cs} := \frac{\pi}{4}\cdot d_{bar\_c}^{\,2} = 0.785\cdot\text{in}^2$ *Cross Sec Area of Compression Reinforcement*

$d_{cs} := c_{ts} + 0.5\cdot d_{bar\_c} = 4.5\cdot\text{in}$ *Depth of Compression Reinforcement in the Slab*

$A_{bs} := \begin{pmatrix} A_{ts} \\ A_{cs} \end{pmatrix} = \begin{pmatrix} 0.785 \\ 0.785 \end{pmatrix}\cdot\text{in}^2$ *Cross Sectional Areas Reinforcement*

$d_{bs} := \begin{pmatrix} d_{ts} \\ d_{cs} \end{pmatrix} = \begin{pmatrix} 19.5 \\ 4.5 \end{pmatrix}\cdot\text{in}$ *Depths of Reinforcement*

$C_{TS.Slab} :=$ "OK" if $A_{ts} \geq 1\text{in}^2 \wedge A_{cs} \geq 1\text{in}^2$ = "OK"
otherwise
  "OK" if $\sum A_{bs} > 0.003\cdot b\cdot t_{wall}$
  "Min Reinf. Not Met" otherwise

Minimum reinforcement (ACI 318-19-8-6-1)

$T_a := 94\text{kip}$ *Assumed Tension Force on Section*

$a := \frac{T_a}{0.85\cdot f_c\cdot b} = 1.843\cdot\text{in}$ *Width of Whitney Stress Block*

$$\varepsilon_s := \left| \begin{array}{l} \text{for } i \in 0..\text{rows}(d_{bs}) - 1 \\ \quad \varepsilon_{s_i} \leftarrow \dfrac{0.003}{\dfrac{a}{\beta_1}} \cdot \left(d_{bs_i} - \dfrac{a}{\beta_1} \right) \\ \varepsilon_s \end{array} \right. = \begin{pmatrix} 0.022 \\ 2.86 \times 10^{-3} \end{pmatrix}$$ *Strain in Reinforcement*

$$\text{Check}_{\text{Tensile\_Strain}} := \left| \begin{array}{l} \text{"OK"} \quad \text{if } \max(\varepsilon_s) \geq \max\left(\dfrac{f_y}{E_s}, 0.005 \right) \\ \text{"NG - Too much reinforcement"} \quad \text{otherwise} \end{array} \right. = \text{"OK"}$$

$$f_s := \left| \begin{array}{l} \text{for } i \in 0..\text{rows}(d_{bs}) - 1 \\ \quad \left| \begin{array}{l} f_{s_i} \leftarrow f_y \quad \text{if } \varepsilon_{s_i} \geq \dfrac{f_y}{E_s} \\ f_{s_i} \leftarrow -f_y \quad \text{if } \varepsilon_{s_i} \leq \dfrac{-f_y}{E_s} \\ f_{s_i} \leftarrow E_s \cdot \varepsilon_{s_i} \quad \text{if } \varepsilon_{s_i} < \dfrac{f_y}{E_s} \wedge \varepsilon_{s_i} \geq 0 \\ f_{s_i} \leftarrow E_s \cdot \varepsilon_{s_i} + 0.85 \cdot f_c \quad \text{otherwise} \end{array} \right. \\ f_s \end{array} \right. = \begin{pmatrix} 60 \\ 60 \end{pmatrix} \cdot \text{ksi}$$ *Stress in Reinforcement*

$$P_s := \left| \begin{array}{l} \text{for } i \in 0..\text{rows}(d_{bs}) - 1 \\ \quad P_{s_i} \leftarrow f_{s_i} \cdot A_{bs_i} \\ P_s \end{array} \right. = \begin{pmatrix} 47.124 \\ 47.124 \end{pmatrix} \cdot \text{kip}$$ *Force in Reinforcement*

$$P_{\text{check}} := \begin{pmatrix} \dfrac{\sum P_s}{T_a} \\ \left| \begin{array}{l} \text{"OK"} \quad \text{if } \dfrac{\sum P_s}{T_a} \geq 0.95 \wedge \dfrac{\sum P_s}{T_a} \leq 1.05 \\ \text{"Reiterate"} \quad \text{otherwise} \end{array} \right. \end{pmatrix} = \begin{pmatrix} 1.003 \\ \text{"OK"} \end{pmatrix}$$ *Check for Equilibrium*

$$M_s := \left| \begin{array}{l} \text{for } i \in 0..\text{rows}(d_{bs}) - 1 \\ \quad M_{s_i} \leftarrow P_{s_i} \cdot \left(d_{bs_i} - \frac{a}{2} \right) \\ M_s \end{array} \right. = \begin{pmatrix} 72.957 \\ 14.052 \end{pmatrix} \cdot \text{ft} \cdot \text{kip}$$

Moment in Reinforcement

$$\phi_b := \left| \begin{array}{ll} 0.65 & \text{if } \max(\varepsilon_s) \le 0.002 \\ 0.90 & \text{if } \max(\varepsilon_s) \ge 0.005 \\ 0.65 + (\max(\varepsilon_s) - 0.002) \cdot \left(\frac{250}{3} \right) & \text{otherwise} \end{array} \right. = 0.9$$

Load Reduction Factor

$$\phi M_{n\_wall\_pos} := \frac{\phi_b \cdot \sum M_s}{b} = 78.31 \cdot \frac{\text{kip} \cdot \text{ft}}{\text{ft}}$$

Nominal Moment Capacity of Slab

$$\phi V_{n\_wall} := \phi_v \cdot 2 \cdot \lambda \cdot \sqrt{\frac{f_c}{\text{psi}}} \cdot \text{psi} \cdot d_{ts} = 24.819 \cdot \frac{\text{kip}}{\text{ft}}$$

Nominal Shear Capacity of Slab (ACI 318 Section 22.5.5.1)

$$DCR_M := \frac{M_u}{\phi M_{n\_wall\_pos}} = 0.486$$

$$DCR_V := \frac{V_u}{\phi V_{n\_wall}} = 0.5$$

Use #8@12in with 2 ft thick wall at both faces of wall

1ft thick Baffle Wall Design:

Figure 2: 2.5 ft thick wall with region of maximum moment circled in red

Positive Moment (tension at outer face); Moment about horizontal axis is higher than about vertical

$M_u := 5kip \cdot \frac{ft}{ft}$ *Maximum Moment from SAP2000*

$V_u := 4 \frac{kip}{ft}$ *Maximum Shear from SAP2000*

$c_{bs} := 4in$ Cover for tension rebar

$c_{ts} := 4in$ Cover for compression rebar

$b := 1ft$ *Assumed width of section*

$d_{bar\_t} := \frac{6}{8}in$ *Diameter of Tensile Reinforcement (1 #6 @10in bar)*

$A_{ts} := \frac{12}{10} \cdot \frac{\pi}{4} \cdot d_{bar\_t}^2 = 0.53 \cdot in^2$ *Cross Sectional Area of Tensile Reinforcement*

$t_{wall} := 1ft$ *Thickness of wall*

$d_{ts} := t_{wall} - c_{bs} - 0.5 d_{bar\_t} = 7.625 \cdot in$ *Depth of Tensile Reinforcement in the Slab*

$d_{bar\_c} := \frac{6}{8} in$ *Diameter of Compression Reinforcement (1 #8@12in bar)*

$A_{cs} := \frac{\pi}{4} \cdot d_{bar\_c}^{2} = 0.442 \cdot in^2$ *Cross Sec Area of Compression Reinforcement*

$d_{cs} := c_{ts} + 0.5 \cdot d_{bar\_c} = 4.375 \cdot in$ *Depth of Compression Reinforcement in the Slab*

$A_{bs} := \begin{pmatrix} A_{ts} \\ A_{cs} \end{pmatrix} = \begin{pmatrix} 0.53 \\ 0.442 \end{pmatrix} \cdot in^2$ *Cross Sectional Areas Reinforcement*

$d_{bs} := \begin{pmatrix} d_{ts} \\ d_{cs} \end{pmatrix} = \begin{pmatrix} 7.625 \\ 4.375 \end{pmatrix} \cdot in$ *Depths of Reinforcement*

$C_{TS.Slab} :=$ "OK" if $A_{ts} \geq 1 in^2 \wedge A_{cs} \geq 1 in^2$ = "OK"
otherwise
  "OK" if $\sum A_{bs} > 0.003 \cdot b \cdot t_{wall}$
  "Min Reinf. Not Met" otherwise

Minimum reinforcement (ACI 318-19-8-6-1)

$T_a := 58 kip$ *Assumed Tension Force on Section*

$a := \frac{T_a}{0.85 \cdot f_c \cdot b} = 1.137 \cdot in$ *Width of Whitney Stress Block*

$\varepsilon_s :=$ for $i \in 0 .. rows(d_{bs}) - 1$ $= \begin{pmatrix} 0.013 \\ 6.233 \times 10^{-3} \end{pmatrix}$ *Strain in Reinforcement*
  $\varepsilon_{s_i} \leftarrow \frac{0.003}{\frac{a}{\beta_1}} \cdot \left(d_{bs_i} - \frac{a}{\beta_1} \right)$
  ε_s

$Check_{Tensile\_Strain} :=$ "OK" if $\max(\varepsilon_s) \geq \max\left(\frac{f_y}{E_s}, 0.005 \right)$ = "OK"
"NG - Too much reinforcement" otherwise

$$f_s := \left| \begin{array}{l} \text{for } i \in 0..\text{rows}(d_{bs}) - 1 \\ \quad f_{s_i} \leftarrow f_y \text{ if } \varepsilon_{s_i} \geq \frac{f_y}{E_s} \\ \quad f_{s_i} \leftarrow -f_y \text{ if } \varepsilon_{s_i} \leq \frac{-f_y}{E_s} \\ \quad f_{s_i} \leftarrow E_s \cdot \varepsilon_{s_i} \text{ if } \varepsilon_{s_i} < \frac{f_y}{E_s} \wedge \varepsilon_{s_i} \geq 0 \\ \quad f_{s_i} \leftarrow E_s \cdot \varepsilon_{s_i} + 0.85 \cdot f_c \text{ otherwise} \\ f_s \end{array} \right. = \begin{pmatrix} 60 \\ 60 \end{pmatrix} \cdot \text{ksi}$$ *Stress in Reinforcement*

$$P_s := \left| \begin{array}{l} \text{for } i \in 0..\text{rows}(d_{bs}) - 1 \\ \quad P_{s_i} \leftarrow f_{s_i} \cdot A_{bs_i} \\ P_s \end{array} \right. = \begin{pmatrix} 31.809 \\ 26.507 \end{pmatrix} \cdot \text{kip}$$ *Force in Reinforcement*

$$P_{check} := \begin{pmatrix} \frac{\sum P_s}{T_a} \\ \text{"OK" if } \frac{\sum P_s}{T_a} \geq 0.95 \wedge \frac{\sum P_s}{T_a} \leq 1.05 \\ \text{"Reiterate" otherwise} \end{pmatrix} = \begin{pmatrix} 1.005 \\ \text{"OK"} \end{pmatrix}$$ *Check for Equilibrium*

$$M_s := \left| \begin{array}{l} \text{for } i \in 0..\text{rows}(d_{bs}) - 1 \\ \quad M_{s_i} \leftarrow P_{s_i} \cdot \left(d_{bs_i} - \frac{a}{2} \right) \\ M_s \end{array} \right. = \begin{pmatrix} 18.704 \\ 8.408 \end{pmatrix} \cdot \text{ft} \cdot \text{kip}$$ *Moment in Reinforcement*

$$\phi_b := \left| \begin{array}{l} 0.65 \text{ if } \max(\varepsilon_s) \leq 0.002 \\ 0.90 \text{ if } \max(\varepsilon_s) \geq 0.005 \\ 0.65 + (\max(\varepsilon_s) - 0.002) \cdot \left(\frac{250}{3} \right) \text{ otherwise} \end{array} \right. = 0.9$$ *Load Reduction Factor*

$$\phi M_{n\_wall\_pos} := \frac{\phi_b \cdot \sum M_s}{b} = 24.4 \cdot \frac{kip \cdot ft}{ft}$$

Nominal Moment Capacity of Slab

$$\phi V_{n\_wall} := \phi_v \cdot 2 \cdot \lambda \cdot \sqrt{\frac{f_c}{psi}} \cdot psi \cdot d_{ts} = 9.705 \cdot \frac{kip}{ft}$$

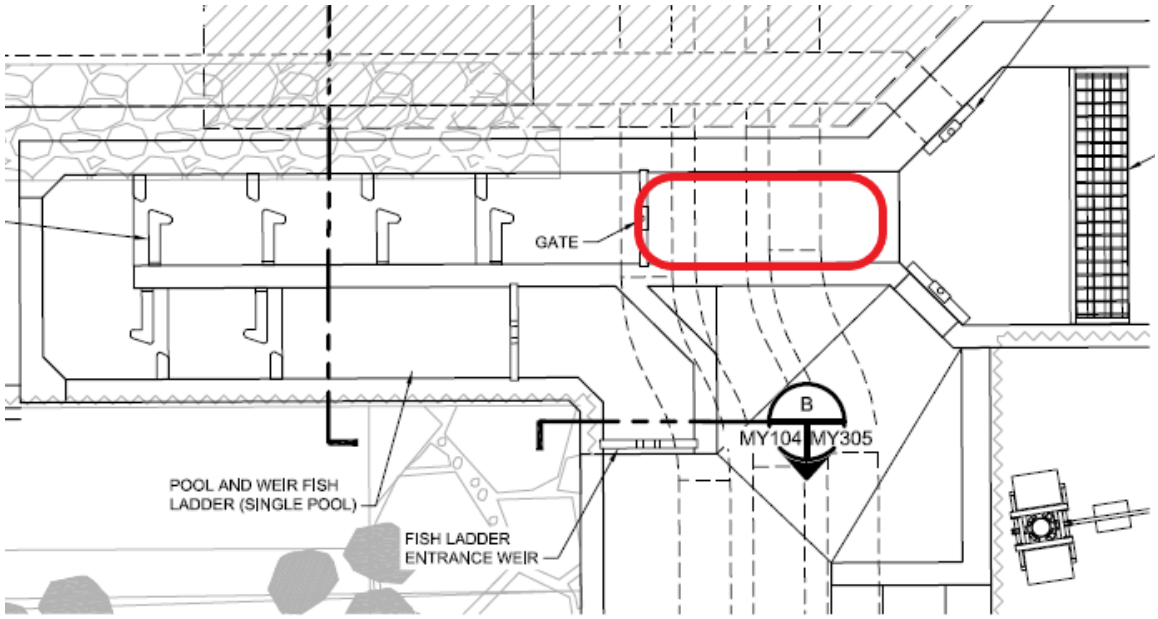
Nominal Shear Capacity of Slab (ACI 318 Section 22.5.5.1)

$$DCR_M := \frac{M_u}{\phi M_{n\_wall\_pos}} = 0.205$$

$$DCR_V := \frac{V_u}{\phi V_{n\_wall}} = 0.412$$

Use #8@12in with 1 ft thick wall at both faces of wall

Slab Design:



kip-ft/ft

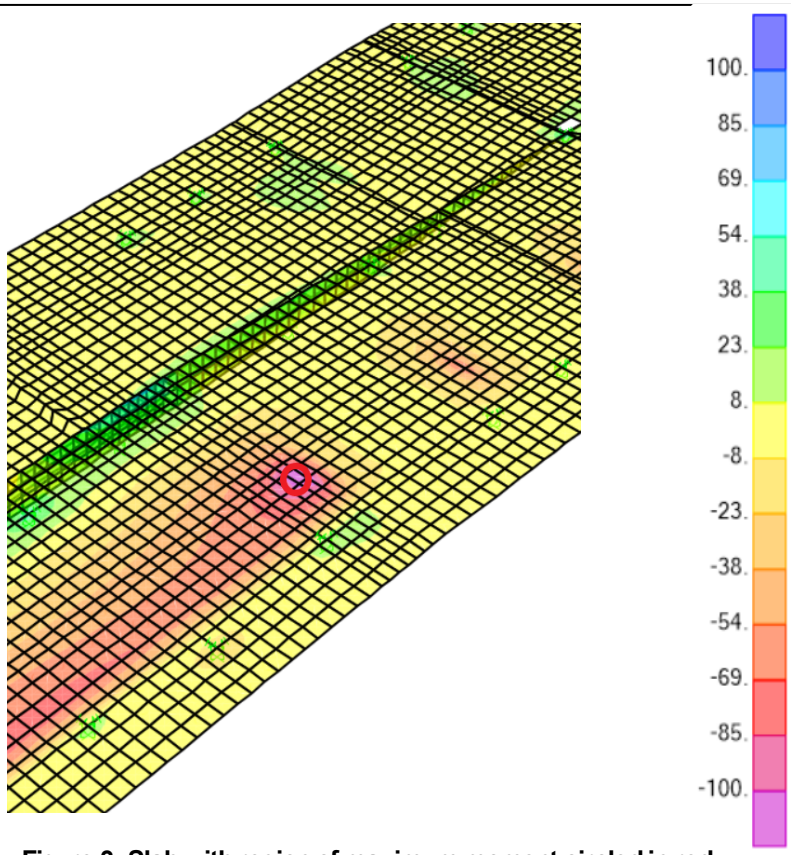


Figure 3: Slab with region of maximum moment circled in red

Positive Moment (tension at bottom)

$c_{bs} := 4in$ Cover for tension rebar

$c_{ts} := 4in$ Cover for compression rebar

$b := 1ft$ *Assumed width of section*

$d_{bar\_t} := 1.128in$ *Diameter of Tensile Reinforcement (1 #9 @8in bar DWG 12-37-1)*

$A_{ts} := \frac{12}{8} 1in^2 = 1.5 \cdot in^2$ *Cross Sectional Area of Tensile Reinforcement*

$t_{slab} := 3ft$ *Thickness of slab*

$d_{ts} := t_{slab} - c_{bs} - 0.5 d_{bar\_t} = 31.436 \cdot in$ *Depth of Tensile Reinforcement in the Slab*

$d_{bar\_c} := 1.128in$ *Diameter of Compression Reinforcement (1 #9@8in bar)*

$A_{cs} := \frac{12}{8} 1in^2 = 1.5 \cdot in^2$ *Cross Sec Area of Compression Reinforcement*

$d_{cs} := c_{ts} + 0.5 \cdot d_{bar\_c} = 4.564 \cdot in$ *Depth of Compression Reinforcement in the Slab*

$$A_{bs} := \begin{pmatrix} A_{ts} \\ A_{cs} \end{pmatrix} = \begin{pmatrix} 1.5 \\ 1.5 \end{pmatrix} \cdot in^2$$

Cross Sectional Areas Reinforcement

$$d_{bs} := \begin{pmatrix} d_{ts} \\ d_{cs} \end{pmatrix} = \begin{pmatrix} 31.436 \\ 4.564 \end{pmatrix} \cdot in$$

Depths of Reinforcement

$$C_{TS.Slab} := \begin{cases} \text{"OK"} & \text{if } A_{ts} \geq 1in^2 \wedge A_{cs} \geq 1in^2 \\ & = \text{"OK"} \\ \text{otherwise} & \\ \quad \begin{cases} \text{"OK"} & \text{if } \sum A_{bs} > 0.003 \cdot b \cdot t_{slab} \\ \text{"Min Reinf. Not Met"} & \text{otherwise} \end{cases} & \end{cases}$$

Minimum reinforcement for two way slab (ACI 318-19-8-6-1)

$$T_a := 136.5kip$$

Assumed Tension Force on Section

$$a := \frac{T_a}{0.85 \cdot f_c \cdot b} = 2.676 \cdot in$$

Width of Whitney Stress Block

$$\varepsilon_s := \left| \begin{array}{l} \text{for } i \in 0 .. rows(d_{bs}) - 1 \\ \quad \varepsilon_{s_i} \leftarrow \dfrac{0.003}{\frac{a}{\beta_1}} \cdot \left(d_{bs_i} - \dfrac{a}{\beta_1} \right) \\ \varepsilon_s \end{array} \right. = \begin{pmatrix} 0.025 \\ 1.093 \times 10^{-3} \end{pmatrix}$$

Strain in Reinforcement

$$Check_{Tensile\_Strain} := \begin{cases} \text{"OK"} & \text{if } \max(\varepsilon_s) \geq \max\left(\dfrac{f_y}{E_s}, 0.005 \right) \\ & = \text{"OK"} \\ \text{"NG - Too much reinforcement"} & \text{otherwise} \end{cases}$$

$$f_s := \left| \begin{array}{l} \text{for } i \in 0..\text{rows}(d_{bs}) - 1 \\ \quad \left| \begin{array}{l} f_{s_i} \leftarrow f_y \text{ if } \varepsilon_{s_i} \geq \frac{f_y}{E_s} \\ f_{s_i} \leftarrow -f_y \text{ if } \varepsilon_{s_i} \leq \frac{-f_y}{E_s} \\ f_{s_i} \leftarrow E_s \cdot \varepsilon_{s_i} \text{ if } \varepsilon_{s_i} < \frac{f_y}{E_s} \wedge \varepsilon_{s_i} \geq 0 \\ f_{s_i} \leftarrow E_s \cdot \varepsilon_{s_i} + 0.85 \cdot f_c \text{ otherwise} \end{array} \right. \\ f_s \end{array} \right. = \begin{pmatrix} 60 \\ 31.684 \end{pmatrix} \cdot \text{ksi}$$

Stress in Reinforcement

$$P_s := \left| \begin{array}{l} \text{for } i \in 0..\text{rows}(d_{bs}) - 1 \\ \quad P_{s_i} \leftarrow f_{s_i} \cdot A_{bs_i} \\ P_s \end{array} \right. = \begin{pmatrix} 90.000 \\ 47.526 \end{pmatrix} \cdot \text{kip}$$

Force in Reinforcement

$$P_{check} := \begin{pmatrix} \frac{\sum P_s}{T_a} \\ \text{"OK" if } \frac{\sum P_s}{T_a} \geq 0.95 \wedge \frac{\sum P_s}{T_a} \leq 1.05 \\ \text{"Reiterate" otherwise} \end{pmatrix} = \begin{pmatrix} 1.008 \\ \text{"OK"} \end{pmatrix}$$

Check for Equilibrium

$$M_s := \left| \begin{array}{l} \text{for } i \in 0..\text{rows}(d_{bs}) - 1 \\ \quad M_{s_i} \leftarrow P_{s_i} \cdot \left(d_{bs_i} - \frac{a}{2} \right) \\ M_s \end{array} \right. = \begin{pmatrix} 225.733 \\ 12.776 \end{pmatrix} \cdot \text{ft} \cdot \text{kip}$$

Moment in Reinforcement

$$\phi_b := \left| \begin{array}{l} 0.65 \text{ if } \max(\varepsilon_s) \leq 0.002 \\ 0.90 \text{ if } \max(\varepsilon_s) \geq 0.005 \\ 0.65 + \left(\max(\varepsilon_s) - 0.002 \right) \cdot \left(\frac{250}{3} \right) \text{ otherwise} \end{array} \right. = 0.9$$

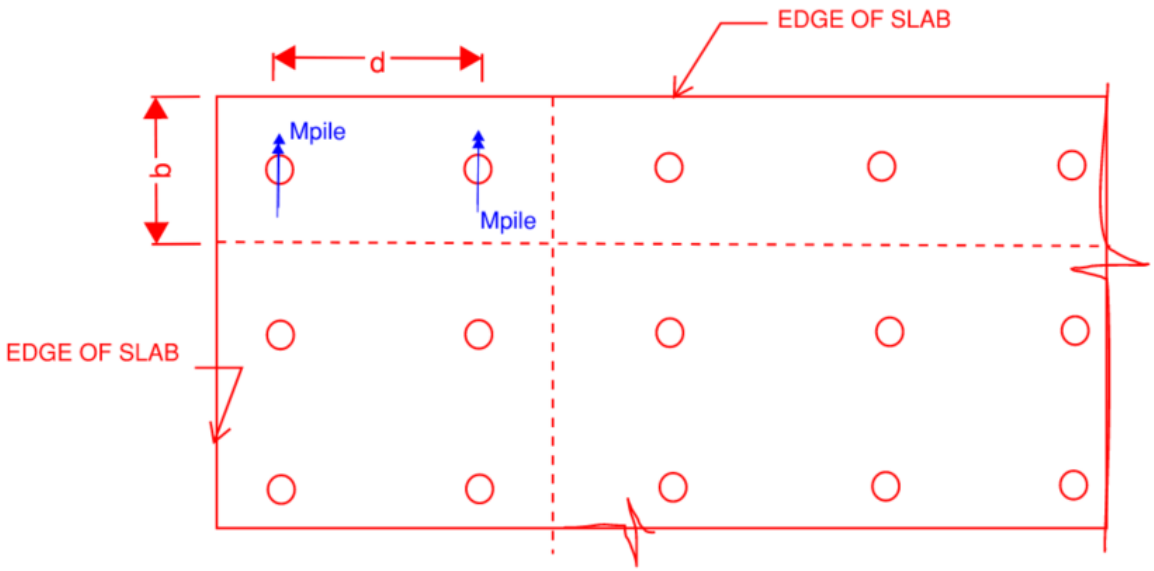
Load Reduction Factor

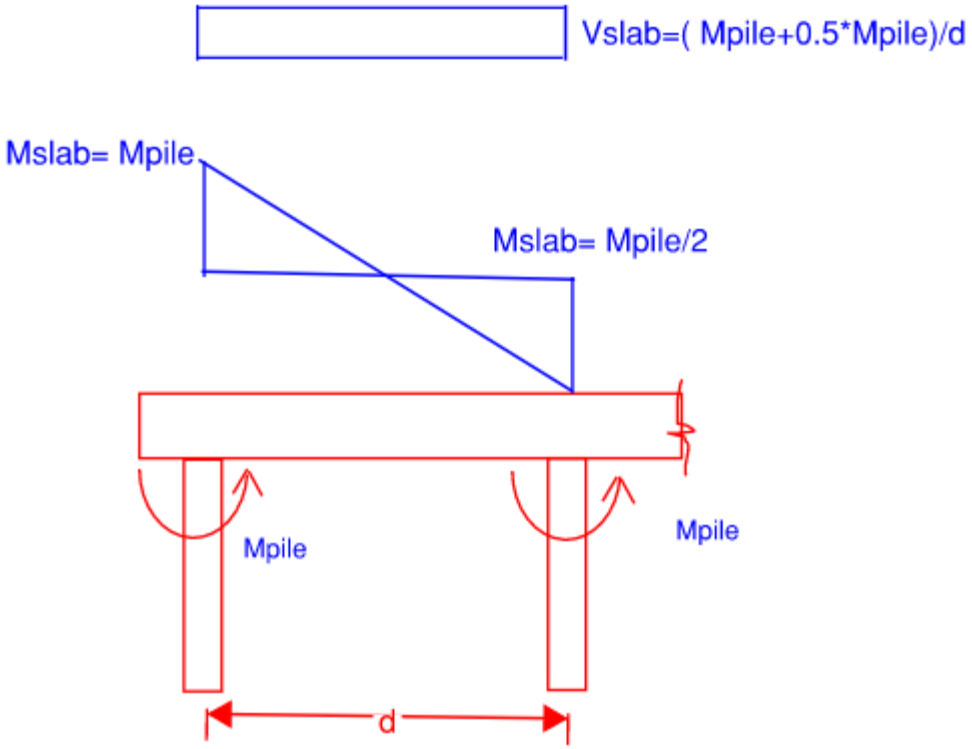
$$\phi M_{n\_slab\_pos} := \frac{\phi_b \cdot \sum M_s}{b} = 214.66 \cdot \frac{kip \cdot ft}{ft}$$

Nominal Moment Capacity of Slab

$$\phi V_{n\_slab} := \phi_v \cdot 2 \cdot \lambda \cdot \sqrt{\frac{f_c}{psi}} \cdot psi \cdot d_{ts} = 40.011 \cdot \frac{kip}{ft}$$

Nominal Shear Capacity of Slab (ACI 318 Section 22.5.5.1)





$M_{pile} := 12700\,kip\cdot in$

Plastic moment capacity of 24" steel pile-See next Section

$b := 6\,ft + 9\,in = 6.75\,ft$

Tributary width from middle of slab edge

$d := 8\,ft + 6\,in = 8.5\,ft$

Distance between two piles

$$M_{slab} := \frac{M_{pile}}{b} = 156.79\,\frac{1}{ft}\cdot kip\cdot ft$$

Ultimate moment on pile

$$V_{slab} := \frac{\frac{M_{pile}+0.5M_{pile}}{d}}{b} = 27.669\,\frac{1}{ft}\cdot kip$$

Ultimate shear on pile

$$DCR_M := \frac{M_{slab}}{\phi M_{n\_slab\_pos}} = 0.73$$

$$DCR_V := \frac{V_{slab}}{\phi V_{n\_slab}} = 0.692$$

Use #9@8in with 3 ft thick slab at top and bottom layers each way

Pile design:

$D_p := 24in$ *Outer diameter of steel pile*

$t_p := 0.5in$ *Thickness of steel pile*

$$Z_p := \frac{\left[\frac{4}{3}\cdot(0.5\cdot D_p)^3 - \frac{4}{3}(0.5D_p - t_p)^3\right]}{\pi\left[(0.5D_p)^2 - (0.5D_p - t_p)^2\right]}\cdot\left[\pi\left[(0.5D_p)^2 - (0.5D_p - t_p)^2\right]\right] = 276.167\cdot in^3$$

Plastic modulus of pile

$$S_y := \pi\cdot\frac{\left[D_p{}^4 - (D_p - t_p)^4\right]}{32D_p} = 109.612\cdot in^3$$

Section modulus of pile

$$\frac{D_p}{t_p} = 48$$

Slenderness ratio of pile

$f_{yp} := 46ksi$ *Yield strength of pile*

$E_p := 29000ksi$ *Bulk modulus of pile*

$$\frac{0.45\cdot E_p}{f_{yp}} = 283.696$$ **> Dp/tp**

Section F8, AISC Steel Manual

Hence we can use Section F8 of AISC-360

Now we assume a value of the stiffnesses in the SAP2000 model and iterate using value of lateral shear from SAP2000 and lateral deflection from LPile till the value of the stiffness converges

First iteration

$V_{pmax} := 85.445kip$ *Maximum lateral shear from SAP*

$u_{lateral} := 1.66in$ *Maximum lateral deflection from LPile*

$$K_p := \frac{V_{pmax}}{u_{lateral}} = 617.675\cdot klf$$

Lateral stiffness of pile

Second iteration (take Kp=600 klf)

$V_{pmax} := 86.084kip$ *Maximum lateral shear on pile from SAP*

$u_{lateral} := 1.69in$ *Maximum lateral deflection on pile from LPile*

$K_p := \frac{V_{pmax}}{u_{lateral}} = 611.247 \cdot klf$ *Lateral stiffness of pile*

Converges, hence accepted

1. Yielding

$M_{n1} := f_{yp} \cdot Z_p = 1.27 \times 10^4 \cdot in \cdot kip$ *Nominal flexural strength due to yielding*

2. Local Buckling

$\lambda_r := 0.31 \frac{E_p}{f_{yp}} = 195.435$ > Dp/tp *Table B4.1b, AISC 360. Hence section is slender*

$M_{n2} := \left[\frac{0.33E_p}{\left(\frac{D_p}{t_p}\right)}\right] \cdot S_y = 2.185 \times 10^4 \cdot in \cdot kip$ *Nominal flexural strength due to local buckling*

Therefore the minimum of the above two values gives us

$M_n := \min(M_{n1}, M_{n2}) = 1.27 \times 10^4 \cdot in \cdot kip$

From LPile, the value of maximum moment on the pile is as follows

$M_y := 9068kip \cdot in$ which is lesser than the pile capacity

$DCR_{pile} := \frac{M_y}{0.9 \cdot M_n} = 0.793$ **Hence OK**

Axial Check

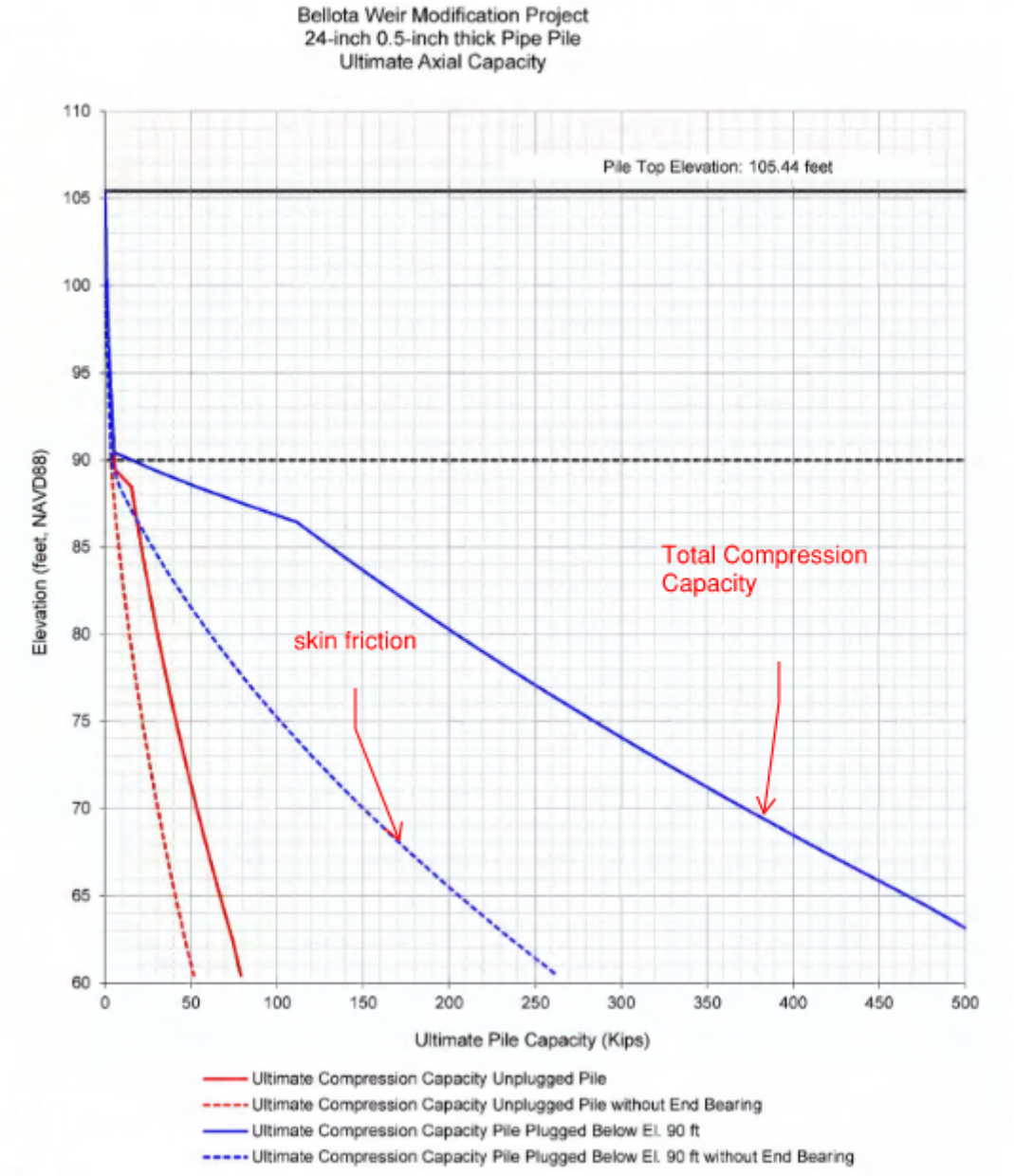


Figure 4: Ultimate Axial Pile Capacity, 24 inch pipe pile, 0.5 inch thick

$P_{tmax} := 14.99kip$ *Maximum tensile force on pile due to seismic (obtained from SAP2000 tables)*

$P_{cmax} := 220.834kip$ *Maximum compressive force on pile (obtained from SAP2000 tables)*

The top graph is from the Geotechnical Investigation Report (App. A, Fig. 15). The 80% of skin friction can

be used as tensile capacity in pile
If we look at the above graph, for a 70ft long pile, the elevation would be at around 35 ft for which the corresponding value of skin friction and ultimate pile capacity would be much higher than both the maximum tensile and compressive forces on the pile respectively.

For fish passage structure the governing factor for pile length would be lateral capacity of piles under seismic loads. In 90% design phase more accurate iteration procedure would follow to elaborate the pile design and optimize the pile length.

Check Slab Joint Shear for Pile Moment (AASHTO Guide Spec 2011-Section 8)

$t_{slab} := 3\,ft$ Slab thickness

$M_{pile} := 12700\,kip\cdot in$ Moment at top of pile

$P_{pile} := 289.4\,kip$ Maximum axial load on pile

$$L_{ac} := \frac{0.79\cdot\frac{9}{8}in\cdot\frac{68ksi}{ksi}}{\sqrt{\frac{f_c}{ksi}}} = 2.252\,ft$$

Lenght of rebar inside cap (AASHTO Guide Spec-8.8.4-1)

$$T_c := \frac{M_{pile}}{2\cdot\left(\frac{2\cdot D_p}{2\cdot\pi}\right)} = 831.213\cdot kip$$

Tensile force inside joint- Lever arm is tnesion centroid to compression centroid of pile

$B_{eff} := \sqrt{2}\cdot D_p = 2.828\,ft$ Effective width of joint

$B_{cap} := \min\left(D_p + 2t_{slab}, 5ft\right) = 5\,ft$

$f_h := 0$

$$f_v := \frac{P_{pile}}{\left(D_p + t_{slab}\right)\cdot B_{cap}} = 0.08\cdot ksi$$

$$\upsilon_{jv} := \frac{T_c}{(t_{slab} - 4in)\cdot B_{eff}} = 0.765\cdot ksi$$

$$p_t := \left| \frac{f_h + f_v}{2} - \sqrt{\left(\frac{f_h - f_v}{2}\right)^2 + \upsilon_{jv}^2} \right| = 0.726\cdot ksi$$

$$p_c := \left| \frac{f_h + f_v}{2} + \sqrt{\left(\frac{f_h - f_v}{2}\right)^2 + \upsilon_{jv}^2} \right| = 0.807\cdot ksi$$

$$\frac{p_t}{0.38\cdot\sqrt{\frac{f_c}{ksi}}\cdot ksi} = 0.855$$ 8.13.2-1 LESS THAN 1 ok

$$\frac{p_c}{0.25\cdot f_c} = 0.645$$ 8.13.2-2

Weir Design

Descriptions:

The following calculations provide the design of Intake Walls, Slabs and Pile foundations for 90% design phase.

The lateral soil pressure and soil seismic loads were used according to the recommendation given in Geotechnical Report for Bellota Project. According to geotech report the seismic load parameters were obtained based on ASCE7-16 with PGA equal to 0.18g. The load combination used for this structure is from following table per EM 1110-2-2104.

Load Case	Load Description	Load Category	Factored Loads
R1	Normal Operating	Usual	$2.2(D + EH + EV + Hs_U)$
R2A	Normal Operating + Surcharge	Unusual	$1.6(D + EH + EV + Hs_U + ES_N)$
R2B	Normal Operating + Short Duration Water Load	Unusual	$1.6(D + EH + EV + Hs_N)$
R3A	Normal Operating + OBE	Unusual	$1.5(D + EH + EV + Hs_U + EQ)$
R3B	Normal Operating + MDE	Extreme (Strength)	$(1.2 \text{ or } 0.9)D + (1.5 \text{ or } 0.5)EH + (1.35 \text{ or } 1.0)EV + 1.0\ Hs_U + 1.25EQ$
R4	Normal Operating + Maximum Water	Extreme (Strength)	$(1.2 \text{ or } 0.9)D + (1.5 \text{ or } 0.5)EH + (1.35 \text{ or } 1.0)EV + 1.3Hs_X$

Source: EM 1110-2-2104

Where: D: Dead Load; EH: Lateral Earth Pressure; EV: Vertical Earth Load; ES: Surcharge Load; HS_N: Normal Hydroststaic Load; HS_U: Hydroststaic Load during Flood; EQ: earthquake Load;

References:

- EM 1110-2-2104, Strength Design for Reinforced Concrete Hydraulic Structures
- ASCE/SEI 7-16 Code
- International Building Code IBC 2018
- ACI318-19
- Geotechnical Investigation Report-Bellota Weir Modification Project

Assumptions:

The intake structure is modeled in SAP2000 program including all the pressures such as active, at-rest and seismic soil pressure, hydrostatic and hydrodynamic pressure; and response spectrum analysis was performed for seismic analysis. Per EM 1110-2-2104 once response spectrum analysis is used, the soil load factor can be reduced to 1.0, which would be the case for this structure. For the design both 100-year flood as unusual and 500 year flood as extreme load case were also used.
For 65% design phase the maximum shear on piles were used for design the piles. Therefore one group effect value was used for critical pile and equivalent spring stiffness was calibrated based on critical pile using the iteration process between SAP2000 and LPile programs. For 90% design the effect of each pile would be evaluated individually and equivalent stiffness will be calibrated for each pile based on location and associated shear forces.
4" concrete cover is provided for both faces of walls and slabs for corrosion protection purposes. For all structural concrete, 5000 psi concrete will be used.
The following reported demands from SAP2000 is representing the maximum absolute value among all the defined load combinations

Material Properties:

$f_c := 5000psi$ — Assumed concrete compressive strength

$\gamma_c := 150pcf$ — Concrete unit weight

$E_c := 57000 \cdot \sqrt{\frac{f_c \cdot in^2}{lbf}} \cdot \frac{lbf}{in^2} = 4.031 \times 10^6 \cdot \frac{lbf}{in^2}$ — Concrete elastic modulus (ACI 318)

$$\beta_1 := \begin{cases} 0.85 & \text{if } f_c \le 4000psi \\ \max\left[0.85 - 0.05 \cdot \left(\frac{f_c - 4000psi}{1000psi}\right), 0.65\right] & \text{otherwise} \end{cases} = 0.8$$

Compression block factor (ACI 318)

$\phi_v := 0.75$ — Shear strength reduction factor

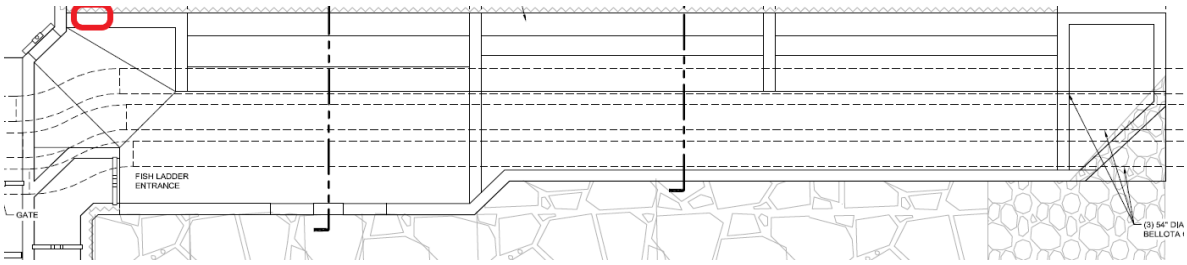
$\lambda := 1$ — Normal weight concrete factor

$f_y := 60ksi$ — Assumed yield strength of rebar

$E_s := 29000ksi$ — Steel elastic modulus

Check Strength Capacity of Concrete Components:

Weir Wall Design:



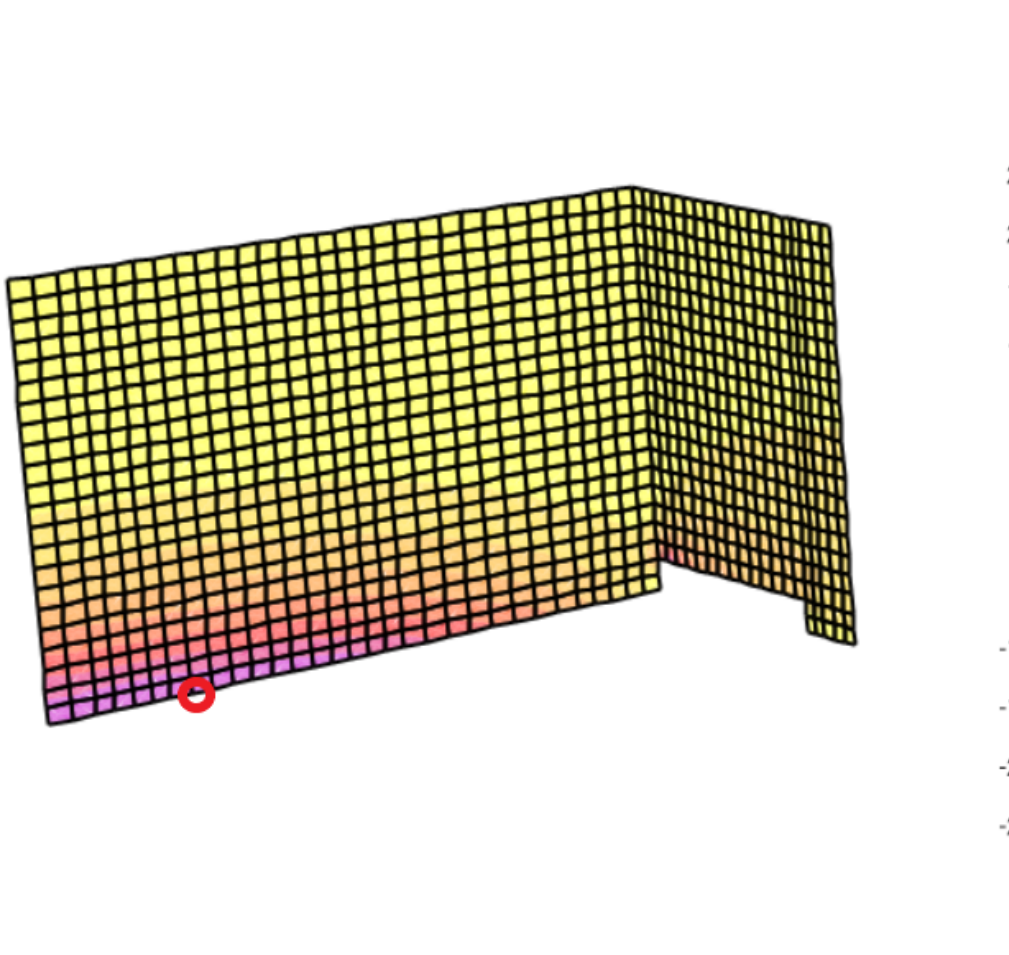


Figure 1: Weir Wall with region of maximum moment circled in red

Positive Moment (tension at outer face of wall)

$M_u := 35kip \cdot \frac{ft}{ft}$ *Maximum moment from SAP2000*

$V_u := 7.3 \frac{kip}{ft}$ *Maximum shear from SAP2000*

$c_{bs} := 4in$ Cover for tension rebar

$c_{ts} := 4in$ Cover for compression rebar

$b := 1ft$ *Assumed width of section*

$d_{bar\_t} := 1in$ *Diameter of Tensile Reinforcement (1 #8 @12in bar DWG 12-37-1)*

$A_{ts} := \frac{\pi}{4} \cdot d_{bar\_t}^{2} = 0.785 \cdot in^2$ *Cross Sectional Area of Tensile Reinforcement*

$t_{wall} := 2ft$ *Thickness of wall*

$d_{ts} := t_{wall} - c_{bs} - 0.5 d_{bar\_t} = 19.5 \cdot in$ *Depth of Tensile Reinforcement in the Slab*

$d_{bar\_c} := 1in$ *Diameter of Compression Reinforcement (1 #8@12in bar)*

$A_{cs} := \frac{\pi}{4} \cdot d_{bar\_c}^{2} = 0.785 \cdot in^2$ *Cross Sec Area of Compression Reinforcement*

$d_{cs} := c_{ts} + 0.5 \cdot d_{bar\_c} = 4.5 \cdot in$ *Depth of Compression Reinforcement in the Slab*

$A_{bs} := \begin{pmatrix} A_{ts} \\ A_{cs} \end{pmatrix} = \begin{pmatrix} 0.785 \\ 0.785 \end{pmatrix} \cdot in^2$ *Cross Sectional Areas Reinforcement*

$d_{bs} := \begin{pmatrix} d_{ts} \\ d_{cs} \end{pmatrix} = \begin{pmatrix} 19.5 \\ 4.5 \end{pmatrix} \cdot in$ *Depths of Reinforcement*

$C_{TS.Slab} :=$ "OK" if $A_{ts} \geq 1in^2 \wedge A_{cs} \geq 1in^2$ = "OK"; otherwise: "OK" if $\sum A_{bs} > 0.003 \cdot b \cdot t_{wall}$; "Min Reinf. Not Met" otherwise

Minimum reinforcement for two way slab (ACI 318-19-8-6-1)

$T_a := 94kip$ *Assumed Tension Force on Section*

$a := \frac{T_a}{0.85 \cdot f_c \cdot b} = 1.843 \cdot in$ *Width of Whitney Stress Block*

$\varepsilon_s :=$ for $i \in 0 .. rows(d_{bs}) - 1$: $\varepsilon_{s_i} \leftarrow \frac{0.003}{\frac{a}{\beta_1}} \cdot \left(d_{bs_i} - \frac{a}{\beta_1} \right)$; $\varepsilon_s = \begin{pmatrix} 0.022 \\ 2.86 \times 10^{-3} \end{pmatrix}$ *Strain in Reinforcement*

$$Check_{Tensile\_Strain} := \begin{cases} \text{"OK"} & \text{if } \max(\varepsilon_s) \geq \max\left(\frac{f_y}{E_s}, 0.005\right) \\ \text{"NG - Too much reinforcement"} & \text{otherwise} \end{cases} = \text{"OK"}$$

$$f_s := \left| \begin{array}{l} \text{for } i \in 0..\text{rows}(d_{bs}) - 1 \\ \quad \left| \begin{array}{l} f_{s_i} \leftarrow f_y \text{ if } \varepsilon_{s_i} \geq \frac{f_y}{E_s} \\ f_{s_i} \leftarrow -f_y \text{ if } \varepsilon_{s_i} \leq \frac{-f_y}{E_s} \\ f_{s_i} \leftarrow E_s \cdot \varepsilon_{s_i} \text{ if } \varepsilon_{s_i} < \frac{f_y}{E_s} \wedge \varepsilon_{s_i} \geq 0 \\ f_{s_i} \leftarrow E_s \cdot \varepsilon_{s_i} + 0.85 \cdot f_c \text{ otherwise} \end{array} \right. \\ f_s \end{array} \right. = \begin{pmatrix} 60 \\ 60 \end{pmatrix} \cdot \text{ksi}$$ *Stress in Reinforcement*

$$P_s := \left| \begin{array}{l} \text{for } i \in 0..\text{rows}(d_{bs}) - 1 \\ \quad P_{s_i} \leftarrow f_{s_i} \cdot A_{bs_i} \\ P_s \end{array} \right. = \begin{pmatrix} 47.124 \\ 47.124 \end{pmatrix} \cdot \text{kip}$$ *Force in Reinforcement*

$$P_{check} := \begin{pmatrix} \frac{\sum P_s}{T_a} \\ \begin{cases} \text{"OK"} & \text{if } \frac{\sum P_s}{T_a} \geq 0.95 \wedge \frac{\sum P_s}{T_a} \leq 1.05 \\ \text{"Reiterate"} & \text{otherwise} \end{cases} \end{pmatrix} = \begin{pmatrix} 1.003 \\ \text{"OK"} \end{pmatrix}$$ *Check for Equilibrium*

$$M_s := \left| \begin{array}{l} \text{for } i \in 0..\text{rows}(d_{bs}) - 1 \\ \quad M_{s_i} \leftarrow P_{s_i} \cdot \left(d_{bs_i} - \frac{a}{2}\right) \\ M_s \end{array} \right. = \begin{pmatrix} 72.957 \\ 14.052 \end{pmatrix} \cdot \text{ft} \cdot \text{kip}$$ *Moment in Reinforcement*

$$\phi_b := \begin{cases} 0.65 & \text{if } \max(\varepsilon_s) \leq 0.002 \\ 0.90 & \text{if } \max(\varepsilon_s) \geq 0.005 \\ 0.65 + \left(\max(\varepsilon_s) - 0.002\right) \cdot \left(\frac{250}{3}\right) & \text{otherwise} \end{cases} = 0.9$$ *Load Reduction Factor*

$$\phi M_{n\_wall\_pos} := \frac{\phi_b \cdot \sum M_s}{b} = 78.31 \cdot \frac{kip \cdot ft}{ft}$$

Nominal Moment Capacity of Slab

$$\phi V_{n\_wall} := \phi_v \cdot 2 \cdot \lambda \cdot \sqrt{\frac{f_c}{psi}} \cdot psi \cdot d_{ts} = 24.819 \cdot \frac{kip}{ft}$$

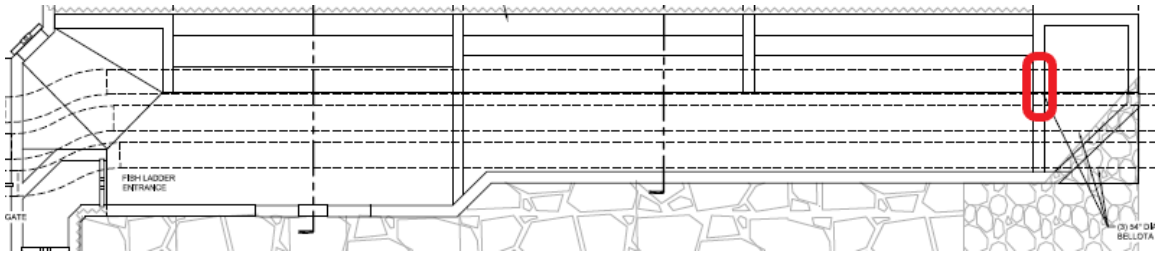
Nominal Shear Capacity of Slab (ACI 318 Section 22.5.5.1)

$$DCR_M := \frac{M_u}{\phi M_{n\_wall\_pos}} = 0.447$$

$$DCR_V := \frac{V_u}{\phi V_{n\_wall}} = 0.294$$

OK use #8@12" at outer face and #8@12" at inner face

End Wall Design:



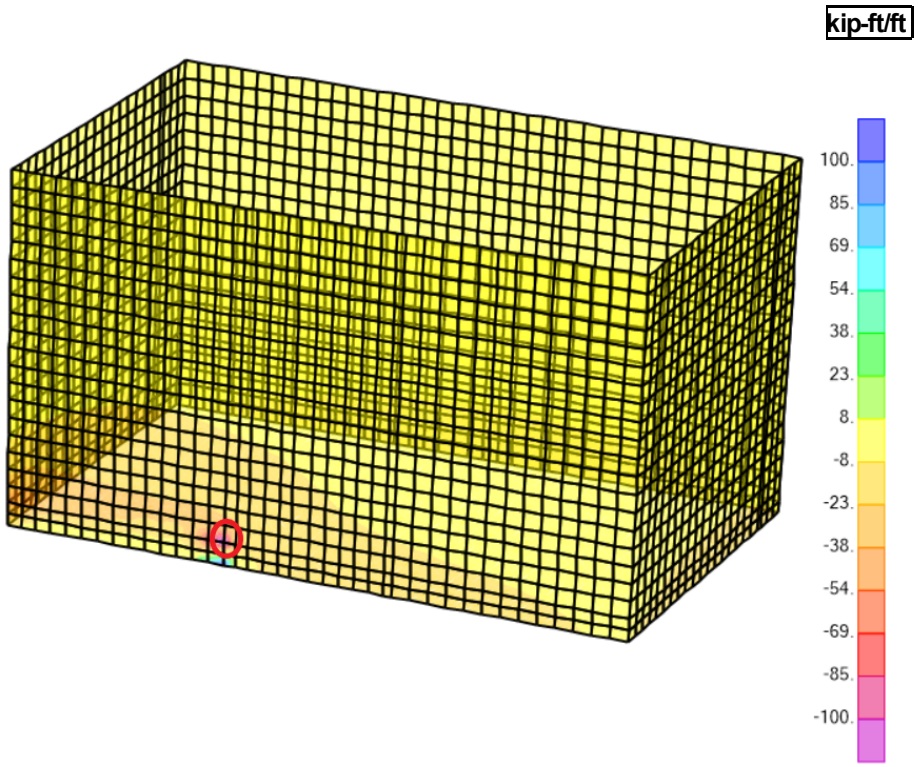


Figure 2: End Wall with region of maximum moment circled in red

Positive Moment (tension at outer face); Moment about horizontal axis is higher than about vertical

$M_u := 68kip \cdot \frac{ft}{ft}$ Maximum moment from SAP2000

$V_u := 4.1 \frac{kip}{ft}$ The value of shear at a distance of d from the step up region is taken from the SAP2000 model

$c_{bs} := 4in$ Cover for tension rebar

$c_{ts} := 4in$ Cover for compression rebar

$b := 1ft$ *Assumed width of section*

$d_{bar\_t} := 1in$ *Diameter of Tensile Reinforcement (1 #8 @8in bar)*

$A_{ts} := \frac{12}{8} \cdot \frac{\pi}{4} \cdot d_{bar\_t}^{2} = 1.178 \cdot in^2$ *Cross Sectional Area of Tensile Reinforcement*

$t_{wall} := 2ft$ *Thickness of wall*

$d_{ts} := t_{wall} - c_{bs} - 0.5 d_{bar\_t} = 19.5 \cdot in$ *Depth of Tensile Reinforcement in the Slab*

$d_{bar\_c} := 1in$ *Diameter of Compression Reinforcement (1 #8@12in bar)*

$A_{cs} := \frac{\pi}{4} \cdot d_{bar\_c}^{2} = 0.785 \cdot in^2$ *Cross Sec Area of Compression Reinforcement*

$d_{cs} := c_{ts} + 0.5 \cdot d_{bar\_c} = 4.5 \cdot in$ *Depth of Compression Reinforcement in the Slab*

$A_{bs} := \begin{pmatrix} A_{ts} \\ A_{cs} \end{pmatrix} = \begin{pmatrix} 1.178 \\ 0.785 \end{pmatrix} \cdot in^2$ *Cross Sectional Areas Reinforcement*

$d_{bs} := \begin{pmatrix} d_{ts} \\ d_{cs} \end{pmatrix} = \begin{pmatrix} 19.5 \\ 4.5 \end{pmatrix} \cdot in$ *Depths of Reinforcement*

$C_{TS.Slab} :=$ "OK" if $A_{ts} \geq 1in^2 \wedge A_{cs} \geq 1in^2$ = "OK"; otherwise: "OK" if $\sum A_{bs} > 0.003 \cdot b \cdot t_{wall}$; "Min Reinf. Not Met" otherwise

Minimum reinforcement (ACI 318-19-8-6-1)

$T_a := 113kip$ *Assumed Tension Force on Section*

$a := \frac{T_a}{0.85 \cdot f_c \cdot b} = 2.216 \cdot in$ *Width of Whitney Stress Block*

$\varepsilon_s :=$ for $i \in 0 .. rows(d_{bs}) - 1$: $\varepsilon_{s_i} \leftarrow \frac{0.003}{\frac{a}{\beta_1}} \cdot \left(d_{bs_i} - \frac{a}{\beta_1} \right)$; $\varepsilon_s = \begin{pmatrix} 0.018 \\ 1.874 \times 10^{-3} \end{pmatrix}$ *Strain in Reinforcement*

$$Check_{Tensile\_Strain} := \begin{cases} \text{"OK"} & \text{if } \max(\varepsilon_s) \geq \max\left(\frac{f_y}{E_s}, 0.005\right) \\ \text{"NG - Too much reinforcement"} & \text{otherwise} \end{cases} = \text{"OK"}$$

$$f_s := \left| \begin{array}{l} \text{for } i \in 0..\text{rows}(d_{bs}) - 1 \\ \quad \left| \begin{array}{l} f_{s_i} \leftarrow f_y \text{ if } \varepsilon_{s_i} \geq \frac{f_y}{E_s} \\ f_{s_i} \leftarrow -f_y \text{ if } \varepsilon_{s_i} \leq \frac{-f_y}{E_s} \\ f_{s_i} \leftarrow E_s \cdot \varepsilon_{s_i} \text{ if } \varepsilon_{s_i} < \frac{f_y}{E_s} \wedge \varepsilon_{s_i} \geq 0 \\ f_{s_i} \leftarrow E_s \cdot \varepsilon_{s_i} + 0.85 \cdot f_c \text{ otherwise} \end{array} \right. \\ f_s \end{array} \right. = \begin{pmatrix} 60 \\ 54.356 \end{pmatrix} \cdot \text{ksi}$$ *Stress in Reinforcement*

$$P_s := \left| \begin{array}{l} \text{for } i \in 0..\text{rows}(d_{bs}) - 1 \\ \quad P_{s_i} \leftarrow f_{s_i} \cdot A_{bs_i} \\ P_s \end{array} \right. = \begin{pmatrix} 70.686 \\ 42.691 \end{pmatrix} \cdot \text{kip}$$ *Force in Reinforcement*

$$P_{check} := \begin{pmatrix} \frac{\sum P_s}{T_a} \\ \begin{cases} \text{"OK"} & \text{if } \frac{\sum P_s}{T_a} \geq 0.95 \wedge \frac{\sum P_s}{T_a} \leq 1.05 \\ \text{"Reiterate"} & \text{otherwise} \end{cases} \end{pmatrix} = \begin{pmatrix} 1.003 \\ \text{"OK"} \end{pmatrix}$$ *Check for Equilibrium*

$$M_s := \left| \begin{array}{l} \text{for } i \in 0..\text{rows}(d_{bs}) - 1 \\ \quad M_{s_i} \leftarrow P_{s_i} \cdot \left(d_{bs_i} - \frac{a}{2}\right) \\ M_s \end{array} \right. = \begin{pmatrix} 108.339 \\ 12.068 \end{pmatrix} \cdot \text{ft} \cdot \text{kip}$$ *Moment in Reinforcement*

$$\phi_b := \begin{cases} 0.65 & \text{if } \max(\varepsilon_s) \leq 0.002 \\ 0.90 & \text{if } \max(\varepsilon_s) \geq 0.005 \\ 0.65 + \left(\max(\varepsilon_s) - 0.002\right) \cdot \left(\frac{250}{3}\right) & \text{otherwise} \end{cases} = 0.9$$ *Load Reduction Factor*

$$\phi M_{n\_wall\_pos} := \frac{\phi_b \cdot \sum M_s}{b} = 108.37 \cdot \frac{kip \cdot ft}{ft}$$

Nominal Moment Capacity of Slab

$$\phi V_{n\_wall} := \phi_v \cdot 2 \cdot \lambda \cdot \sqrt{\frac{f_c}{psi}} \cdot psi \cdot d_{ts} = 24.819 \cdot \frac{kip}{ft}$$

Nominal Shear Capacity of Slab (ACI 318 Section 22.5.5.1)

$$DCR_M := \frac{M_u}{\phi M_{n\_wall\_pos}} = 0.628$$

Use #8@8" at outer face and #8@12" at inner face

$$DCR_V := \frac{V_u}{\phi V_{n\_wall}} = 0.165$$

Slab Design:

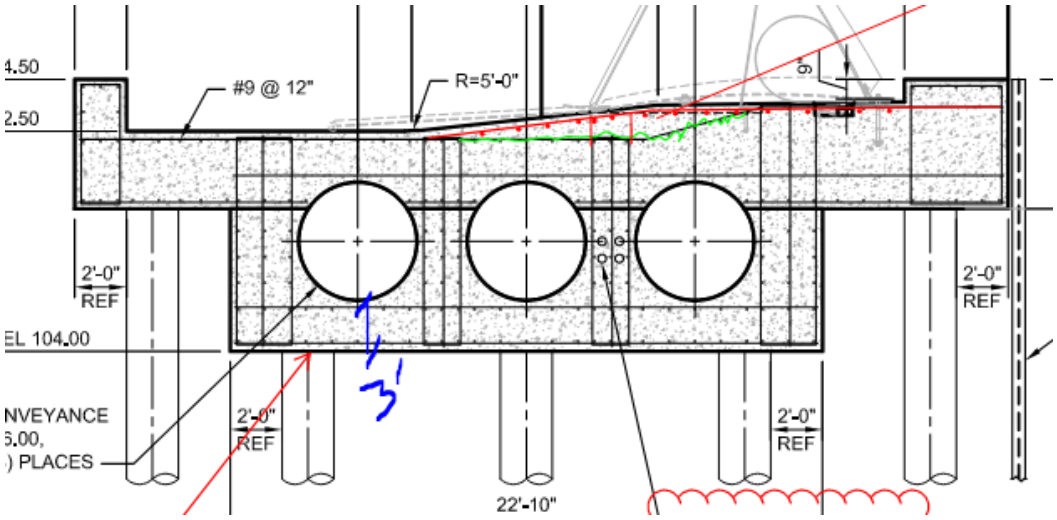


Figure 3: Cross section of slab

Positive Moment (tension at bottom)

$c_{bs} := 4in$ Cover for tension rebar

$c_{ts} := 4in$ Cover for compression rebar

$b := 1ft$ *Assumed width of section*

$d_{bar\_t} := 1.128in$ *Diameter of Tensile Reinforcement (1 #9 @8in)*

$A_{ts} := \frac{12}{8} 1in^2 = 1.5 \cdot in^2$ *Cross Sectional Area of Tensile Reinforcement*

$t_{slab} := 3ft$ *Thickness of slab*

$d_{ts} := t_{slab} - c_{bs} - 0.5 d_{bar\_t} = 31.436 \cdot in$ *Depth of Tensile Reinforcement in the Slab*

$d_{bar\_c} := 1.128in$ *Diameter of Compression Reinforcement (1 #9@8in bar)*

$A_{cs} := \frac{12}{8} 1in^2 = 1.5 \cdot in^2$ *Cross Sec Area of Compression Reinforcement*

$d_{cs} := c_{ts} + 0.5 \cdot d_{bar\_c} = 4.564 \cdot in$ *Depth of Compression Reinforcement in the Slab*

$A_{bs} := \begin{pmatrix} A_{ts} \\ A_{cs} \end{pmatrix} = \begin{pmatrix} 1.5 \\ 1.5 \end{pmatrix} \cdot in^2$ *Cross Sectional Areas Reinforcement*

$d_{bs} := \begin{pmatrix} d_{ts} \\ d_{cs} \end{pmatrix} = \begin{pmatrix} 31.436 \\ 4.564 \end{pmatrix} \cdot in$ *Depths of Reinforcement*

$$C_{TS.Slab} := \begin{cases} \text{"OK"} & \text{if } A_{ts} \geq 1\text{in}^2 \wedge A_{cs} \geq 1\text{in}^2 \\ & = \text{"OK"} \\ \text{otherwise} & \\ \quad \text{"OK"} & \text{if } \sum A_{bs} > 0.003 \cdot b \cdot t_{slab} \\ \quad \text{"Min Reinf. Not Met"} & \text{otherwise} \end{cases}$$

Minimum reinforcement (ACI 318-19-8-6-1)

$T_a := 136.5\text{kip}$ *Assumed Tension Force on Section*

$$a := \frac{T_a}{0.85 \cdot f_c \cdot b} = 2.676 \cdot \text{in}$$ *Width of Whitney Stress Block*

$$\varepsilon_s := \begin{cases} \text{for } i \in 0..\text{rows}(d_{bs}) - 1 & = \begin{pmatrix} 0.025 \\ 1.093 \times 10^{-3} \end{pmatrix} \\ \quad \varepsilon_{s_i} \leftarrow \dfrac{0.003}{\frac{a}{\beta_1}} \cdot \left(d_{bs_i} - \dfrac{a}{\beta_1} \right) & \\ \varepsilon_s & \end{cases}$$ *Strain in Reinforcement*

$$\text{Check}_{\text{Tensile\_Strain}} := \begin{cases} \text{"OK"} & \text{if } \max(\varepsilon_s) \geq \max\left(\dfrac{f_y}{E_s}, 0.005 \right) \\ & = \text{"OK"} \\ \text{"NG - Too much reinforcement"} & \text{otherwise} \end{cases}$$

$$f_s := \begin{cases} \text{for } i \in 0..\text{rows}(d_{bs}) - 1 & = \begin{pmatrix} 60 \\ 31.684 \end{pmatrix} \cdot \text{ksi} \\ \quad f_{s_i} \leftarrow f_y & \text{if } \varepsilon_{s_i} \geq \dfrac{f_y}{E_s} \\ \quad f_{s_i} \leftarrow -f_y & \text{if } \varepsilon_{s_i} \leq \dfrac{-f_y}{E_s} \\ \quad f_{s_i} \leftarrow E_s \cdot \varepsilon_{s_i} & \text{if } \varepsilon_{s_i} < \dfrac{f_y}{E_s} \wedge \varepsilon_{s_i} \geq 0 \\ \quad f_{s_i} \leftarrow E_s \cdot \varepsilon_{s_i} + 0.85 \cdot f_c & \text{otherwise} \\ f_s & \end{cases}$$ *Stress in Reinforcement*

$$P_s := \begin{cases} \text{for } i \in 0..\text{rows}(d_{bs}) - 1 & = \begin{pmatrix} 90.000 \\ 47.526 \end{pmatrix} \cdot \text{kip} \\ \quad P_{s_i} \leftarrow f_{s_i} \cdot A_{bs_i} & \\ P_s & \end{cases}$$ *Force in Reinforcement*

$$P_{check} := \begin{pmatrix} \dfrac{\sum P_s}{T_a} \\ \left| \begin{array}{ll} \text{"OK"} & \text{if } \dfrac{\sum P_s}{T_a} \geq 0.95 \wedge \dfrac{\sum P_s}{T_a} \leq 1.05 \\ \text{"Reiterate"} & \text{otherwise} \end{array} \right. \end{pmatrix} = \begin{pmatrix} 1.008 \\ \text{"OK"} \end{pmatrix}$$

Check for Equilibrium

$$M_s := \left| \begin{array}{l} \text{for } i \in 0..\text{rows}(d_{bs}) - 1 \\ \quad M_{s_i} \leftarrow P_{s_i} \cdot \left(d_{bs_i} - \dfrac{a}{2} \right) \\ M_s \end{array} \right. = \begin{pmatrix} 225.733 \\ 12.776 \end{pmatrix} \cdot \text{ft} \cdot \text{kip}$$

Moment in Reinforcement

$$\phi_b := \left| \begin{array}{ll} 0.65 & \text{if } \max(\varepsilon_s) \leq 0.002 \\ 0.90 & \text{if } \max(\varepsilon_s) \geq 0.005 \\ 0.65 + (\max(\varepsilon_s) - 0.002) \cdot \left(\dfrac{250}{3} \right) & \text{otherwise} \end{array} \right. = 0.9$$

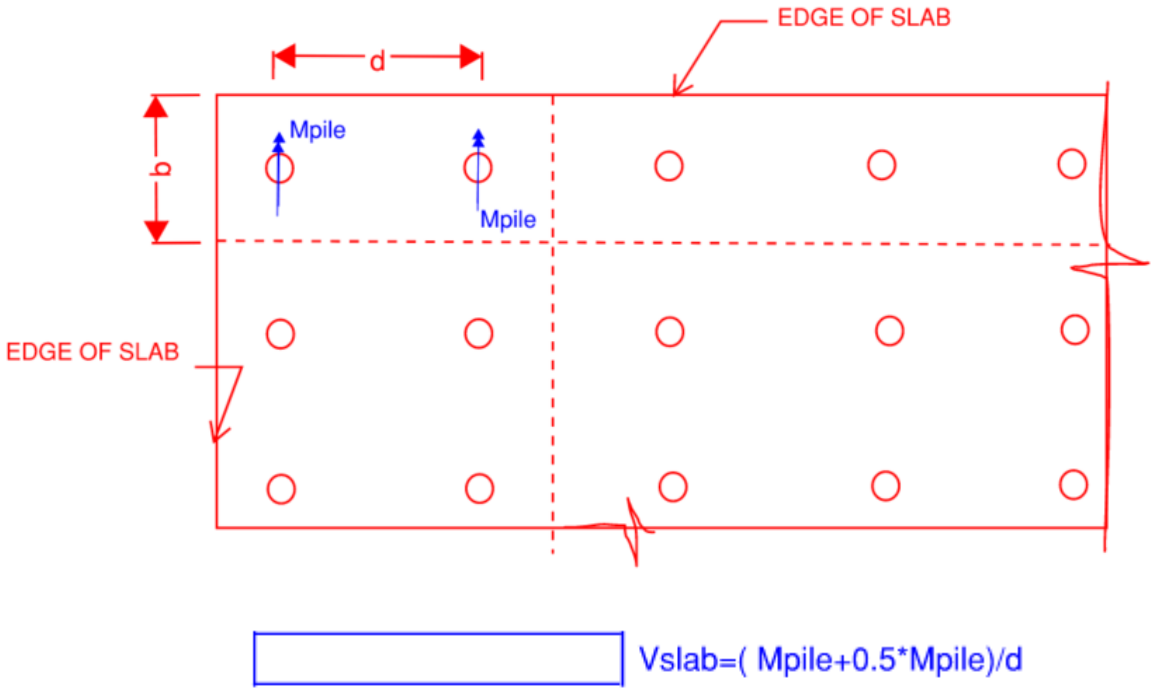
Load Reduction Factor

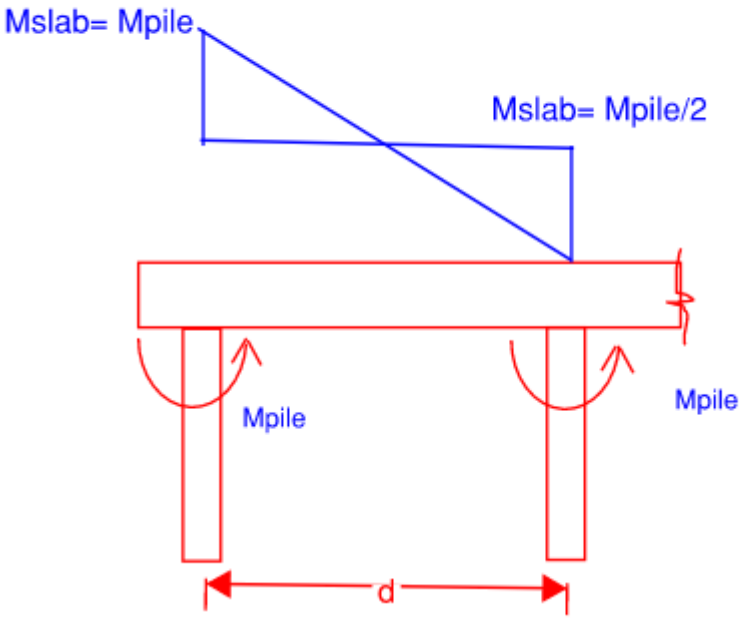
$$\phi M_{n\_slab\_pos} := \frac{\phi_b \cdot \sum M_s}{b} = 214.66 \cdot \frac{\text{kip} \cdot \text{ft}}{\text{ft}}$$

Nominal Moment Capacity of Slab

$$\phi V_{n\_slab} := \phi_v \cdot 2 \cdot \lambda \cdot \sqrt{\frac{f_c}{\text{psi}}} \cdot \text{psi} \cdot d_{ts} = 40.011 \cdot \frac{\text{kip}}{\text{ft}}$$

Nominal Shear Capacity of Slab (ACI 318 Section 22.5.5.1)





$M_{pile} := 12700kip \cdot in$ Plastic moment capacity of 24" steel pile-See next Section

$b := 7ft$ Tributary width from middle of slab edge

$d := 10ft$ Distance between two piles

$$M_{slab} := \frac{M_{pile}}{b} = 151.19 \frac{1}{ft} \cdot kip \cdot ft$$

Ultimate moment on pile

$$V_{slab} := \frac{\frac{M_{pile}+0.5M_{pile}}{d}}{b} = 22.679 \frac{1}{ft} \cdot kip$$

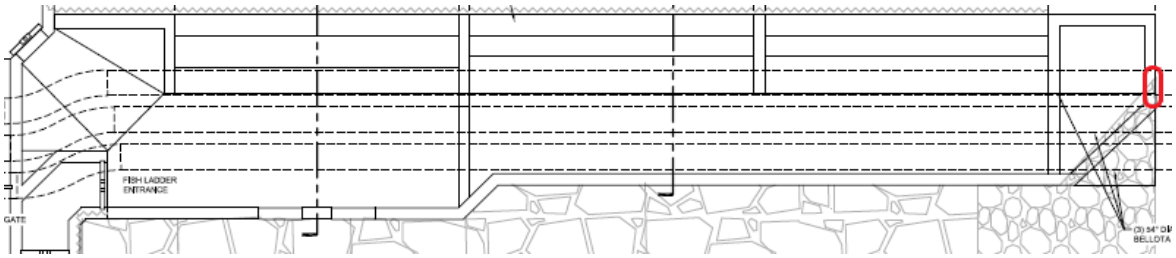
Ultimate shear on pile

$$DCR_M := \frac{M_{slab}}{\phi M_{n\_slab\_pos}} = 0.704$$

$$DCR_V := \frac{V_{slab}}{\phi V_{n\_slab}} = 0.567$$

Use #9@8in with 3 ft thick slab top and bottom each way

End Structure Roof Design:



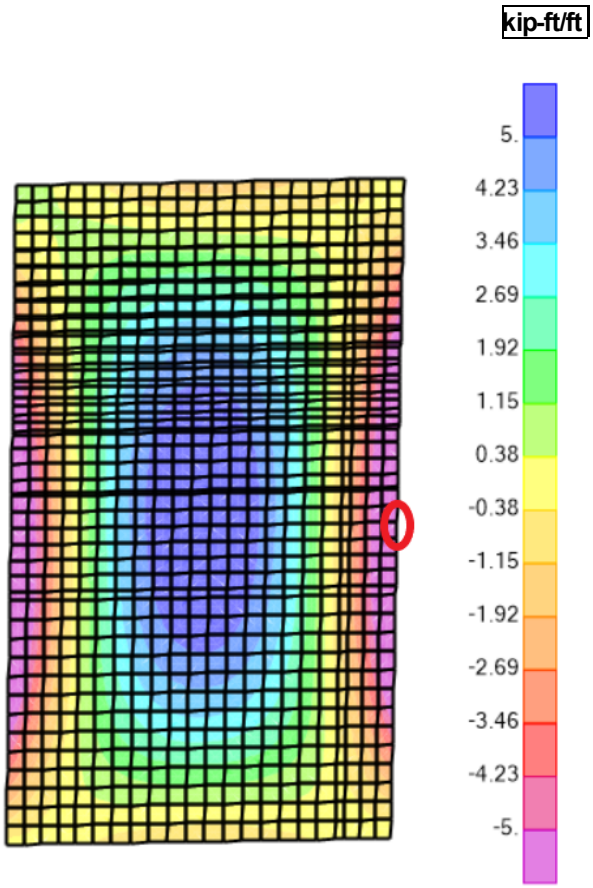


Figure 4: End Structure Roof with region of maximum moment circled in red

Positive Moment (tension at top)

$M_u := 10.22kip \cdot \frac{ft}{ft}$ *Maximum moment from SAP2000*

$V_{u.} := 4.16 \frac{kip}{ft}$ *Maximum shear from SAP2000*

$c_{bs} := 4in$ Cover for tension rebar

$c_{ts} := 4in$ Cover for compression rebar

$b := 1ft$ *Assumed width of section*

$d_{bar\_t} := 1in$ *Diameter of Tensile Reinforcement (1 #8 @12in bar)*

$A_{ts} := \frac{12}{12} \frac{\pi}{4} \cdot d_{bar\_t}^{2} = 0.785 \cdot in^2$ *Cross Sectional Area of Tensile Reinforcement*

$t_{roof} := 1.5ft$ *Thickness of roof*

$d_{ts} := t_{roof} - c_{bs} - 0.5 d_{bar\_t} = 13.5 \cdot in$ *Depth of Tensile Reinforcement in the Slab*

$d_{bar\_c} := 1in$ *Diameter of Compression Reinforcement (1 #8@12in bar)*

$A_{cs} := \frac{12}{12} \frac{\pi}{4} \cdot d_{bar\_c}^{2} = 0.785 \cdot in^2$ *Cross Sec Area of Compression Reinforcement*

$d_{cs} := c_{ts} + 0.5 \cdot d_{bar\_c} = 4.5 \cdot in$ *Depth of Compression Reinforcement in the Slab*

$A_{bs} := \begin{pmatrix} A_{ts} \\ A_{cs} \end{pmatrix} = \begin{pmatrix} 0.785 \\ 0.785 \end{pmatrix} \cdot in^2$ *Cross Sectional Areas Reinforcement*

$d_{bs} := \begin{pmatrix} d_{ts} \\ d_{cs} \end{pmatrix} = \begin{pmatrix} 13.5 \\ 4.5 \end{pmatrix} \cdot in$ *Depths of Reinforcement*

$C_{TS.Slab} := \begin{cases} \text{"OK"} & \text{if } A_{ts} \geq 1in^2 \wedge A_{cs} \geq 1in^2 \\ & = \text{"OK"} \\ \text{otherwise} & \\ \quad \begin{cases} \text{"OK"} & \text{if } \sum A_{bs} > 0.003 \cdot b \cdot t_{roof} \\ \text{"Min Reinf. Not Met"} & \text{otherwise} \end{cases} & \end{cases}$

$T_a := 94kip$ *Assumed Tension Force on Section*

$a := \frac{T_a}{0.85 \cdot f_c \cdot b} = 1.843 \cdot in$ *Width of Whitney Stress Block*

$$\varepsilon_s := \left| \begin{array}{l} \text{for } i \in 0..\text{rows}(d_{bs}) - 1 \\ \quad \varepsilon_{s_i} \leftarrow \dfrac{0.003}{\frac{a}{\beta_1}} \cdot \left(d_{bs_i} - \dfrac{a}{\beta_1} \right) \\ \varepsilon_s \end{array} \right. = \begin{pmatrix} 0.015 \\ 2.86 \times 10^{-3} \end{pmatrix}$$

Strain in Reinforcement

$$\text{Check}_{\text{Tensile\_Strain}} := \left| \begin{array}{l} \text{"OK"} \quad \text{if } \max(\varepsilon_s) \geq \max\left(\dfrac{f_y}{E_s}, 0.005 \right) \\ \text{"NG - Too much reinforcement"} \quad \text{otherwise} \end{array} \right. = \text{"OK"}$$

$$f_s := \left| \begin{array}{l} \text{for } i \in 0..\text{rows}(d_{bs}) - 1 \\ \quad \left| \begin{array}{l} f_{s_i} \leftarrow f_y \quad \text{if } \varepsilon_{s_i} \geq \dfrac{f_y}{E_s} \\ f_{s_i} \leftarrow -f_y \quad \text{if } \varepsilon_{s_i} \leq \dfrac{-f_y}{E_s} \\ f_{s_i} \leftarrow E_s \cdot \varepsilon_{s_i} \quad \text{if } \varepsilon_{s_i} < \dfrac{f_y}{E_s} \wedge \varepsilon_{s_i} \geq 0 \\ f_{s_i} \leftarrow E_s \cdot \varepsilon_{s_i} + 0.85 \cdot f_c \quad \text{otherwise} \end{array} \right. \\ f_s \end{array} \right. = \begin{pmatrix} 60 \\ 60 \end{pmatrix} \cdot \text{ksi}$$

Stress in Reinforcement

$$P_s := \left| \begin{array}{l} \text{for } i \in 0..\text{rows}(d_{bs}) - 1 \\ \quad P_{s_i} \leftarrow f_{s_i} \cdot A_{bs_i} \\ P_s \end{array} \right. = \begin{pmatrix} 47.124 \\ 47.124 \end{pmatrix} \cdot \text{kip}$$

Force in Reinforcement

$$P_{\text{check}} := \left(\begin{array}{l} \dfrac{\sum P_s}{T_a} \\ \left| \begin{array}{l} \text{"OK"} \quad \text{if } \dfrac{\sum P_s}{T_a} \geq 0.95 \wedge \dfrac{\sum P_s}{T_a} \leq 1.05 \\ \text{"Reiterate"} \quad \text{otherwise} \end{array} \right. \end{array} \right) = \begin{pmatrix} 1.003 \\ \text{"OK"} \end{pmatrix}$$

Check for Equilibrium

$$M_s := \left| \begin{array}{l} \text{for } i \in 0..\text{rows}(d_{bs}) - 1 \\ \quad M_{s_i} \leftarrow P_{s_i} \cdot \left(d_{bs_i} - \frac{a}{2} \right) \\ M_s \end{array} \right. = \begin{pmatrix} 49.395 \\ 14.052 \end{pmatrix} \cdot \text{ft} \cdot \text{kip}$$

Moment in Reinforcement

$$\phi_b := \left| \begin{array}{ll} 0.65 & \text{if } \max(\varepsilon_s) \le 0.002 \\ 0.90 & \text{if } \max(\varepsilon_s) \ge 0.005 \\ 0.65 + (\max(\varepsilon_s) - 0.002) \cdot \left(\frac{250}{3} \right) & \text{otherwise} \end{array} \right. = 0.9$$

Load Reduction Factor

$$\phi M_{n\_slab\_pos} := \frac{\phi_b \cdot \sum M_s}{b} = 57.1 \cdot \frac{\text{kip} \cdot \text{ft}}{\text{ft}}$$

Nominal Moment Capacity of Slab

$$\phi V_{n\_slab} := \phi_v \cdot 2 \cdot \lambda \cdot \sqrt{\frac{f_c}{\text{psi}}} \cdot \text{psi} \cdot d_{ts} = 17.183 \cdot \frac{\text{kip}}{\text{ft}}$$

Nominal Shear Capacity of Slab (ACI 318 Section 22.5.5.1)

$$DCR_M := \frac{M_u}{\phi M_{n\_slab\_pos}} = 0.179$$

$$DCR_V := \frac{V_u}{\phi V_{n\_slab}} = 0.239$$

Use #8@12in with 1.5 ft thick roof slab top and bottom

Pile design

$D_p := 24in$ *Outer diameter of steel pile*

$t_p := 0.5in$ *Thickness of steel pile*

$$Z_p := \frac{\left[\frac{4}{3}\cdot\left(0.5\cdot D_p\right)^3 - \frac{4}{3}\left(0.5D_p - t_p\right)^3\right]}{\pi\left[\left(0.5D_p\right)^2 - \left(0.5D_p - t_p\right)^2\right]}\cdot\left[\pi\left[\left(0.5D_p\right)^2 - \left(0.5D_p - t_p\right)^2\right]\right] = 276.167\cdot in^3$$ *Plastic modulus*

$$S_y := \pi\cdot\frac{\left[D_p{}^4 - \left(D_p - t_p\right)^4\right]}{32D_p} = 109.612\cdot in^3$$ *Section modulus*

$\frac{D_p}{t_p} = 48$ *Slenderness ratio of pile*

$f_{yp} := 46ksi$ *Yield strength of pile*

$E_p := 29000ksi$ *Bulk modulus of pile*

$\frac{0.45\cdot E_p}{f_{yp}} = 283.696$ **> Dp/tp** *Section F8, AISC Steel Manual*

Hence we can use Section F8 of AISC-360

For 65% the maximum shear on piles was considered to calibrate the equivalent lateral spring between SAP and LPile. The following shows the iteration results.

First iteration

$V_{pmax} := 53.83kip$ *Maximum lateral shear from SAP*

$u_{lateral} := 0.63in$ *Maximum lateral deflection from LPile*

$K_p := \frac{V_{pmax}}{u_{lateral}} = 1.025\times 10^3\cdot klf$ *Lateral stiffness of pile*

Second iteration (take Kp=990 klf)

$V_{pmax} := 53.844kip$ *Maximum lateral shear from SAP*

$u_{lateral} := 0.63in$ *Maximum lateral deflection from LPile*

$K_p := \frac{V_{pmax}}{u_{lateral}} = 1.026\times 10^3\cdot klf$ *Lateral stiffness of pile*

Converges, hence accepted

1. Yielding

$M_{n1} := f_{yp} \cdot Z_p = 1.27 \times 10^4 \cdot in \cdot kip$ *Nominal flexural strength due to yielding*

2. Local Buckling

$\lambda_r := 0.31 \frac{E_p}{f_{yp}} = 195.435$ > Dp/tp *Table B4.1b, AISC 360. Hence section is slender*

$M_{n2} := \left[\frac{0.33 E_p}{\left(\frac{D_p}{t_p}\right)}\right] \cdot S_y = 2.185 \times 10^4 \cdot in \cdot kip$ *Nominal flexural strength due to local buckling*

Therefore the minimum of the above two values gives us

$M_n := \min\left(M_{n1}, M_{n2}\right) = 1.27 \times 10^4 \cdot in \cdot kip$

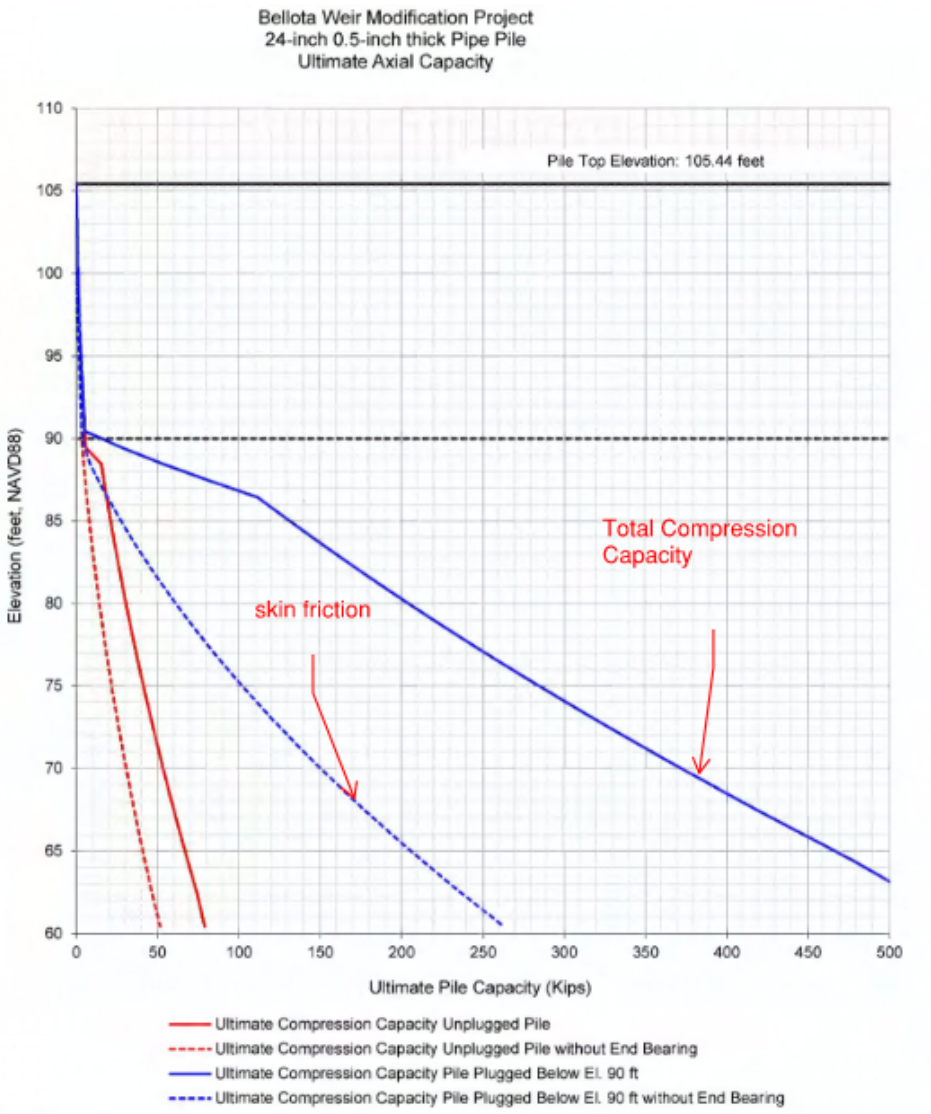
From LPile, the value of maximum moment on the pile is as follows

$M_y := 4850 kip \cdot in$ which is lesser than the pile capacity

$DCR_{pile} := \frac{M_y}{0.9 \cdot M_n} = 0.424$ Use 40 ft long pile

Hence OK

Axial Check



$P_{tmax} := 26.856kip$ *Maximum tensile force on pile*

$P_{cmax} := 180.105kip$ *Maximum compressive force on pile*

The top graph is from geotech report. The 80% of skin friction can be used as tensile capacity in pile

If we look at the above graph, for a pile 40ft long, the elevation would be at around 65 ft for which the corresponding value of skin friction and ultimate pile capacity would be much higher than both the maximum tensile and compressive forces on the pile respectively.

For Weir structure the governing factor for pile length would be lateral capacity of piles under seismic loads. In 90% design phase more accurate iteration procedure would follow to elaborate the pile design and optimize the pile length.

South Diversion Design

Descriptions:

The following calculations provide the design of Walls, Slabs and Pile foundations for 90% design phase.

The lateral soil pressure and soil seismic loads were used according to the recommendation given in Geotechnical Report for Bellota Project. According to geotech report the seismic load parameters were obtained based on ASCE7-16 with PGA equal to 0.18g. The load combination used for this structure is from following table per EM 1110-2-2104.

Load Case	Load Description	Load Category	Factored Loads
R1	Normal Operating	Usual	$2.2(D + EH + EV + Hs_U)$
R2A	Normal Operating + Surcharge	Unusual	$1.6(D + EH + EV + Hs_U + ES_N)$
R2B	Normal Operating + Short Duration Water Load	Unusual	$1.6(D + EH + EV + Hs_N)$
R3A	Normal Operating + OBE	Unusual	$1.5(D + EH + EV + Hs_U + EQ)$
R3B	Normal Operating + MDE	Extreme (Strength)	$(1.2 \text{ or } 0.9)D + (1.5 \text{ or } 0.5)EH + (1.35 \text{ or } 1.0)EV + 1.0\ Hs_U + 1.25EQ$
R4	Normal Operating + Maximum Water	Extreme (Strength)	$(1.2 \text{ or } 0.9)D + (1.5 \text{ or } 0.5)EH + (1.35 \text{ or } 1.0)EV + 1.3Hs_X$

Source: EM 1110-2-2104

Where: D: Dead Load; EH: Lateral Earth Pressure; EV: Vertical Earth Load; ES: Surcharge Load; HS_N: Normal Hydroststaic Load; HS_U: Hydroststaic Load during Flood; EQ: earthquake Load;

References:

- EM 1110-2-2104, Strength Design for Reinforced Concrete Hydraulic Structures
- ASCE/SEI 7-16 Code
- International Building Code IBC 2018
- ACI318-19
- Geotechnical Investigation Report-Bellota Weir Modification Project

Assumptions:

The intake structure is modeled in SAP2000 program including all the pressures such as active, at-rest and seismic soil pressure, hydrostatic and hydrodynamic pressure; and response spectrum analysis was performed for seismic analysis. Per EM 1110-2-2104 once response spectrum analysis is used, the soil load factor can be reduced to 1.0, which would be the case for this structure. For the design both 100-year flood as unusual and 500 year flood as extreme load case were also used.
For 65% design phase the maximum shear on piles were used for design the piles. Therefore one group effect value was used for critical pile and equivalent spring stiffness was calibrated based on critical pile using the iteration process between SAP2000 and LPile programs. For 90% design the effect of each pile would be evaluated individually and equivalent stiffness will be calibrated for each pile based on location and associated shear forces.
4" concrete cover is provided for both faces of walls and slabs for corrosion protection purposes. For all structural concrete, 5000 psi concrete will be used.
The following reported demands from SAP2000 is representing the maximum absolute value among all the defined load combinations

Material Properties:

$f_c := 5000psi$ — Assumed concrete compressive strength

$\gamma_c := 150pcf$ — Concrete unit weight

$E_c := 57000 \cdot \sqrt{\frac{f_c \cdot in^2}{lbf}} \cdot \frac{lbf}{in^2} = 4.031 \times 10^6 \cdot \frac{lbf}{in^2}$ — Concrete elastic modulus (ACI 318)

$$\beta_1 := \begin{cases} 0.85 & \text{if } f_c \le 4000psi \\ \max\left[0.85 - 0.05 \cdot \left(\frac{f_c - 4000psi}{1000psi}\right), 0.65\right] & \text{otherwise} \end{cases} = 0.8$$

Compression block factor (ACI 318)

$\phi_v := 0.75$ — Shear strength reduction factor

$\lambda := 1$ — Normal weight concrete factor

$f_y := 60ksi$ — Assumed yield strength of rebar

$E_s := 29000ksi$ — Steel elastic modulus

Check Strength Capacity of Concrete Components:

For Strength design the Vault box was modeled in SAP2000 and loads are applied on model to get the shear and bending demands on walls

Wall Design:

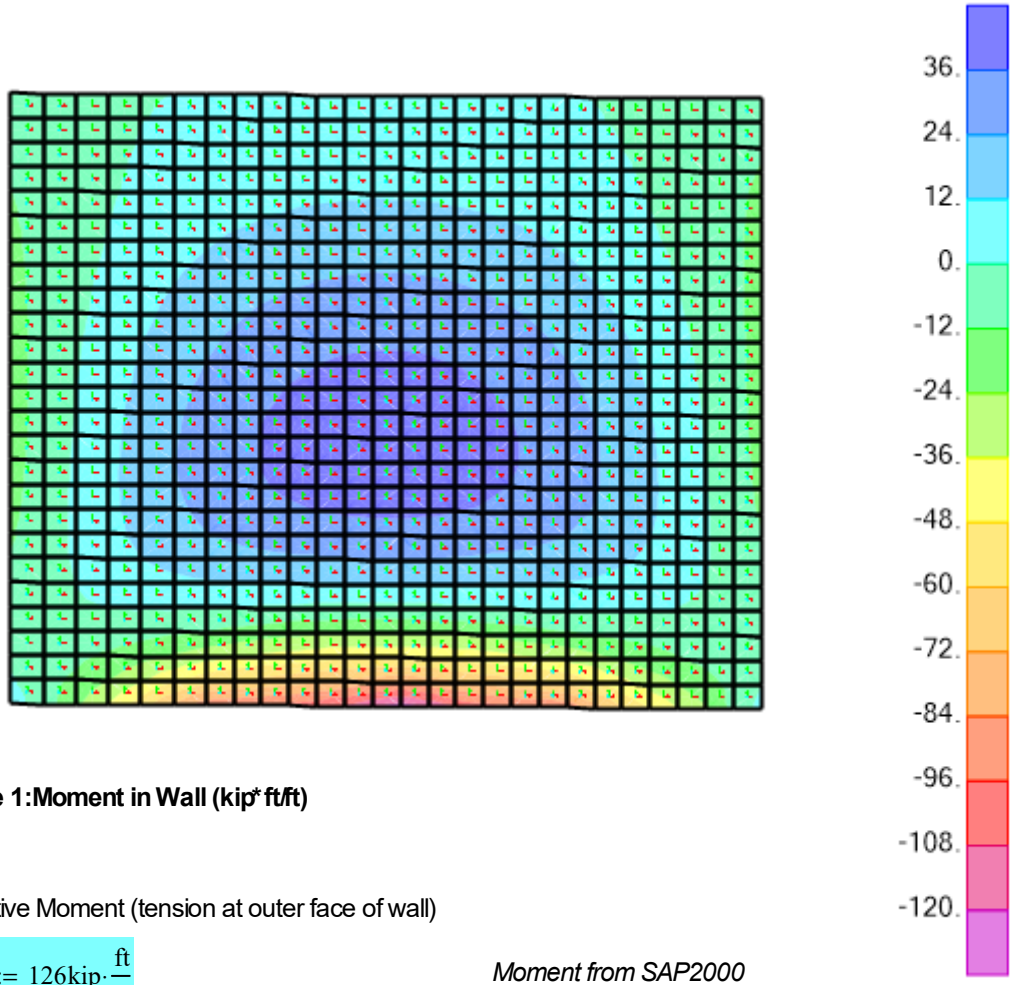


Figure 1:Moment in Wall (kip* ft/ft)

Positive Moment (tension at outer face of wall)

$M_u := 126\text{kip}\cdot\frac{\text{ft}}{\text{ft}}$ *Moment from SAP2000*

$V_u := 26\frac{\text{kip}}{\text{ft}}$ *Shear from SAP2000*

$c_{bs} := 4\text{in}$ *Cover for tension rebar*

$c_{ts} := 4\text{in}$ *Cover for compression rebar*

$b := 1\text{ft}$ *Assumed width of section*

$d_{bar\_t} := 1\text{in}$ *Diameter of Tensile Reinforcement (#8 @8in bar)*

$A_{ts} := \frac{12}{8}\frac{\pi}{4}\cdot d_{bar\_t}^2 = 1.178\cdot\text{in}^2$ *Cross Sectional Area of Tensile Reinforcement*

$t_{wall} := 2.5ft$ *Thickness of wall*

$d_{ts} := t_{wall} - c_{bs} - 0.5d_{bar\_t} = 25.5 \cdot in$ *Depth of Tensile Reinforcement in the Slab*

$d_{bar\_c} := 1in$ *Diameter of Compression Reinforcement (1 #8@12in bar)*

$A_{cs} := \frac{12}{12}\frac{\pi}{4} \cdot d_{bar\_c}^{2} = 0.785 \cdot in^2$ *Cross Sec Area of Compression Reinforcement*

$d_{cs} := c_{ts} + 0.5 \cdot d_{bar\_c} = 4.5 \cdot in$ *Depth of Compression Reinforcement in the Slab*

$A_{bs} := \begin{pmatrix} A_{ts} \\ A_{cs} \end{pmatrix} = \begin{pmatrix} 1.178 \\ 0.785 \end{pmatrix} \cdot in^2$ *Cross Sectional Areas Reinforcement*

$d_{bs} := \begin{pmatrix} d_{ts} \\ d_{cs} \end{pmatrix} = \begin{pmatrix} 25.5 \\ 4.5 \end{pmatrix} \cdot in$ *Depths of Reinforcement*

$C_{TS.Slab} :=$ "OK" if $A_{ts} \geq 1in^2 \wedge A_{cs} \geq 1in^2$ = "OK"
otherwise
  "OK" if $\sum A_{bs} > 0.003 \cdot b \cdot t_{wall}$
  "Min Reinf. Not Met" otherwise

Minimum reinforcement (ACI 318-19-8-6-1)

$T_a := 113kip$ *Assumed Tension Force on Section*

$a := \frac{T_a}{0.85 \cdot f_c \cdot b} = 2.216 \cdot in$ *Width of Whitney Stress Block*

$\varepsilon_s :=$ for $i \in 0..rows(d_{bs}) - 1$ $= \begin{pmatrix} 0.025 \\ 1.874 \times 10^{-3} \end{pmatrix}$ *Strain in Reinforcement*
  $\varepsilon_{s_i} \leftarrow \frac{0.003}{\frac{a}{\beta_1}} \cdot \left(d_{bs_i} - \frac{a}{\beta_1} \right)$
  ε_s

$Check_{Tensile\_Strain} :=$ "OK" if $\max(\varepsilon_s) \geq \max\left(\frac{f_y}{E_s}, 0.005\right)$ = "OK"
"NG - Too much reinforcement" otherwise

$$f_s := \left| \begin{array}{l} \text{for } i \in 0..\text{rows}(d_{bs}) - 1 \\ \quad \left| \begin{array}{l} f_{s_i} \leftarrow f_y \text{ if } \varepsilon_{s_i} \geq \frac{f_y}{E_s} \\ f_{s_i} \leftarrow -f_y \text{ if } \varepsilon_{s_i} \leq \frac{-f_y}{E_s} \\ f_{s_i} \leftarrow E_s \cdot \varepsilon_{s_i} \text{ if } \varepsilon_{s_i} < \frac{f_y}{E_s} \wedge \varepsilon_{s_i} \geq 0 \\ f_{s_i} \leftarrow E_s \cdot \varepsilon_{s_i} + 0.85 \cdot f_c \text{ otherwise} \end{array} \right. \\ f_s \end{array} \right. = \begin{pmatrix} 60 \\ 54.356 \end{pmatrix} \cdot \text{ksi}$$ *Stress in Reinforcement*

$$P_s := \left| \begin{array}{l} \text{for } i \in 0..\text{rows}(d_{bs}) - 1 \\ \quad P_{s_i} \leftarrow f_{s_i} \cdot A_{bs_i} \\ P_s \end{array} \right. = \begin{pmatrix} 70.686 \\ 42.691 \end{pmatrix} \cdot \text{kip}$$ *Force in Reinforcement*

$$P_{check} := \begin{pmatrix} \frac{\sum P_s}{T_a} \\ \text{"OK" if } \frac{\sum P_s}{T_a} \geq 0.95 \wedge \frac{\sum P_s}{T_a} \leq 1.05 \\ \text{"Reiterate" otherwise} \end{pmatrix} = \begin{pmatrix} 1.003 \\ \text{"OK"} \end{pmatrix}$$ *Check for Equilibrium*

$$M_s := \left| \begin{array}{l} \text{for } i \in 0..\text{rows}(d_{bs}) - 1 \\ \quad M_{s_i} \leftarrow P_{s_i} \cdot \left(d_{bs_i} - \frac{a}{2} \right) \\ M_s \end{array} \right. = \begin{pmatrix} 143.682 \\ 12.068 \end{pmatrix} \cdot \text{ft} \cdot \text{kip}$$ *Moment in Reinforcement*

$$\phi_b := \left| \begin{array}{l} 0.65 \text{ if } \max(\varepsilon_s) \leq 0.002 \\ 0.90 \text{ if } \max(\varepsilon_s) \geq 0.005 \\ 0.65 + \left(\max(\varepsilon_s) - 0.002 \right) \cdot \left(\frac{250}{3} \right) \text{ otherwise} \end{array} \right. = 0.9$$ *Load Reduction Factor*

$$\phi M_{n\_wall\_pos} := \frac{\phi_b \cdot \sum M_s}{b} = 140.17 \cdot \frac{kip \cdot ft}{ft}$$

Nominal Moment Capacity of Slab

$$\phi V_{n\_wall} := \phi_v \cdot 2 \cdot \lambda \cdot \sqrt{\frac{f_c}{psi}} \cdot psi \cdot d_{ts} = 32.456 \cdot \frac{kip}{ft}$$

Nominal Shear Capacity of Slab (ACI 318 Section 22.5.5.1)

$$DCR_M := \frac{M_u}{\phi M_{n\_wall\_pos}} = 0.899$$

$$DCR_V := \frac{V_u}{\phi V_{n\_wall}} = 0.801$$

OK use #8@8 at outer face and #8@12" at inner face

Slab Design:

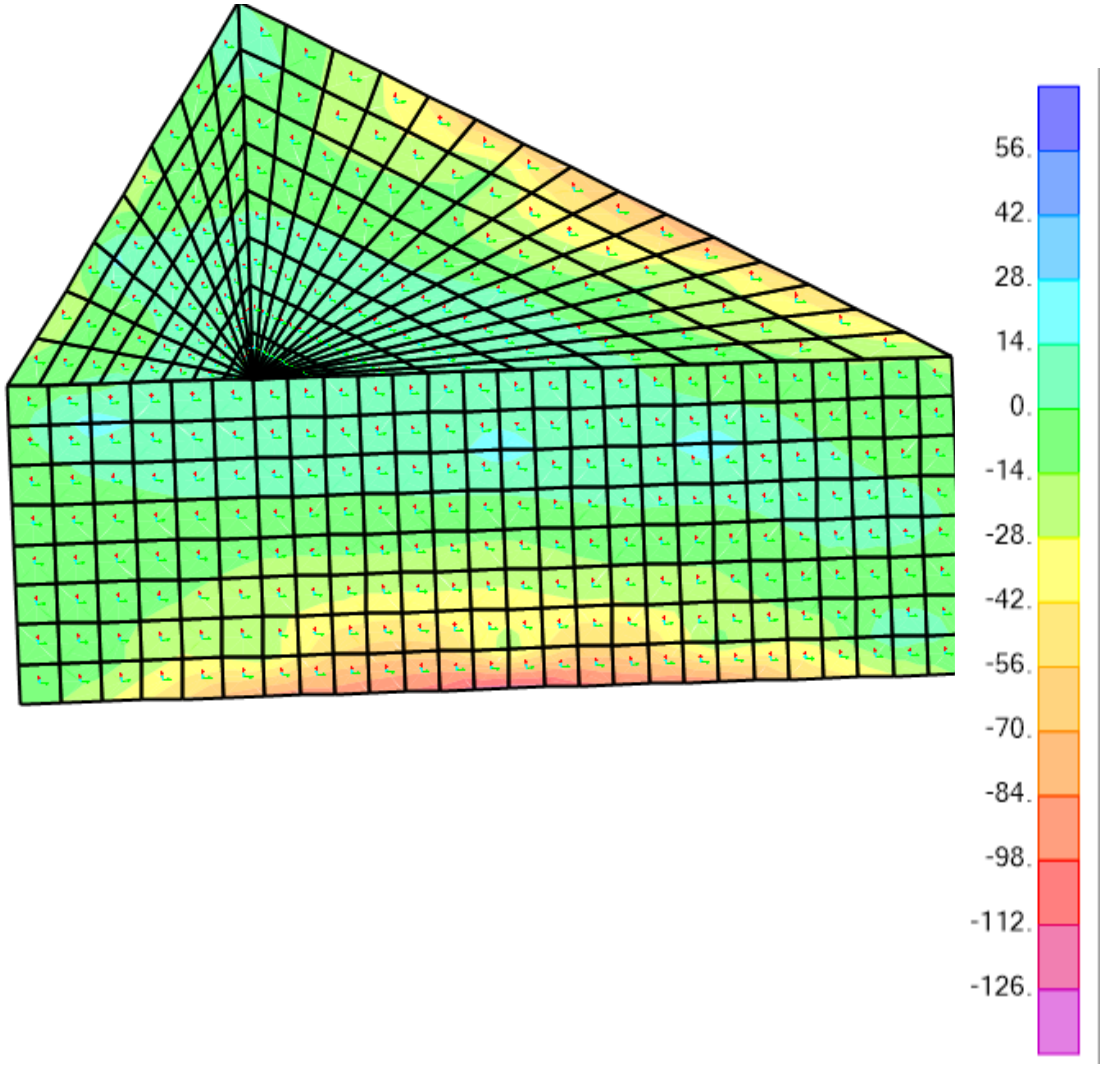


Figure 2: Slab Moment (kip*ft/ft)

$M_u := 126kip \cdot \frac{ft}{ft}$ *Moment from SAP2000*

$V_u := 26 \frac{kip}{ft}$ *Shear from SAP2000*

Positive Moment (tension at bottom)

$c_{bs} := 6in$ Cover for tension rebar

$c_{ts} := 6in$ Cover for compression rebar

$b := 1ft$ *Assumed width of section*

$d_{bar\_t} := 1.128in$ *Diameter of Tensile Reinforcement (1 #9 @8in bar)*

$A_{ts} := \frac{12}{8} 1in^2 = 1.5 \cdot in^2$ *Cross Sectional Area of Tensile Reinforcement*

$t_{slab} := 3ft$ *Thickness of slab*

$d_{ts} := t_{slab} - c_{bs} - 0.5 d_{bar\_t} = 29.436 \cdot in$ *Depth of Tensile Reinforcement in the Slab*

$d_{bar\_c} := 1.128in$ *Diameter of Compression Reinforcement (1 #9@8" bar)*

$A_{cs} := \frac{12}{8} 1in^2 = 1.5 \cdot in^2$ *Cross Sec Area of Compression Reinforcement*

$d_{cs} := c_{ts} + 0.5 \cdot d_{bar\_c} = 6.564 \cdot in$ *Depth of Compression Reinforcement in the Slab*

$A_{bs} := \begin{pmatrix} A_{ts} \\ A_{cs} \end{pmatrix} = \begin{pmatrix} 1.5 \\ 1.5 \end{pmatrix} \cdot in^2$ *Cross Sectional Areas Reinforcement*

$d_{bs} := \begin{pmatrix} d_{ts} \\ d_{cs} \end{pmatrix} = \begin{pmatrix} 29.436 \\ 6.564 \end{pmatrix} \cdot in$ *Depths of Reinforcement*

$C_{TS.Slab} :=$ "OK" if $A_{ts} \geq 1in^2 \wedge A_{cs} \geq 1in^2$ = "OK"
otherwise
"OK" if $\sum A_{bs} > 0.003 \cdot b \cdot t_{slab}$
"Min Reinf. Not Met" otherwise

Minimum reinforcement (ACI 318-19-8-6-1)

$T_a := 167kip$ *Assumed Tension Force on Section*

$$a := \frac{T_a}{0.85 \cdot f_c \cdot b} = 3.275 \cdot in$$ *Width of Whitney Stress Block*

$$\varepsilon_s := \left| \begin{array}{l} \text{for } i \in 0..\text{rows}(d_{bs}) - 1 \\ \quad \varepsilon_{s_i} \leftarrow \dfrac{0.003}{\dfrac{a}{\beta_1}} \cdot \left(d_{bs_i} - \dfrac{a}{\beta_1} \right) \\ \varepsilon_s \end{array} \right. = \begin{pmatrix} 0.019 \\ 1.811 \times 10^{-3} \end{pmatrix}$$ *Strain in Reinforcement*

$$\text{Check}_{\text{Tensile\_Strain}} := \left| \begin{array}{l} \text{"OK"} \quad \text{if } \max(\varepsilon_s) \geq \max\left(\dfrac{f_y}{E_s}, 0.005 \right) \\ \text{"NG - Too much reinforcement"} \quad \text{otherwise} \end{array} \right. = \text{"OK"}$$

$$f_s := \left| \begin{array}{l} \text{for } i \in 0..\text{rows}(d_{bs}) - 1 \\ \quad \left| \begin{array}{l} f_{s_i} \leftarrow f_y \quad \text{if } \varepsilon_{s_i} \geq \dfrac{f_y}{E_s} \\ f_{s_i} \leftarrow -f_y \quad \text{if } \varepsilon_{s_i} \leq \dfrac{-f_y}{E_s} \\ f_{s_i} \leftarrow E_s \cdot \varepsilon_{s_i} \quad \text{if } \varepsilon_{s_i} < \dfrac{f_y}{E_s} \wedge \varepsilon_{s_i} \geq 0 \\ f_{s_i} \leftarrow E_s \cdot \varepsilon_{s_i} + 0.85 \cdot f_c \quad \text{otherwise} \end{array} \right. \\ f_s \end{array} \right. = \begin{pmatrix} 60 \\ 52.518 \end{pmatrix} \cdot ksi$$ *Stress in Reinforcement*

$$P_s := \left| \begin{array}{l} \text{for } i \in 0..\text{rows}(d_{bs}) - 1 \\ \quad P_{s_i} \leftarrow f_{s_i} \cdot A_{bs_i} \\ P_s \end{array} \right. = \begin{pmatrix} 90.000 \\ 78.778 \end{pmatrix} \cdot kip$$ *Force in Reinforcement*

$$P_{check} := \left(\begin{array}{l} \dfrac{\sum P_s}{T_a} \\ \left| \begin{array}{l} \text{"OK"} \quad \text{if } \dfrac{\sum P_s}{T_a} \geq 0.95 \wedge \dfrac{\sum P_s}{T_a} \leq 1.05 \\ \text{"Reiterate"} \quad \text{otherwise} \end{array} \right. \end{array} \right) = \begin{pmatrix} 1.011 \\ \text{"OK"} \end{pmatrix}$$ *Check for Equilibrium*

$$M_s := \left| \begin{array}{l} \text{for } i \in 0..\text{rows}(d_{bs}) - 1 \\ \quad M_{s_i} \leftarrow P_{s_i} \cdot \left(d_{bs_i} - \frac{a}{2} \right) \\ M_s \end{array} \right. = \begin{pmatrix} 208.491 \\ 32.343 \end{pmatrix} \cdot ft \cdot kip$$

Moment in Reinforcement

$$\phi_b := \left| \begin{array}{ll} 0.65 & \text{if } \max(\varepsilon_s) \le 0.002 \\ 0.90 & \text{if } \max(\varepsilon_s) \ge 0.005 \\ 0.65 + (\max(\varepsilon_s) - 0.002) \cdot \left(\frac{250}{3} \right) & \text{otherwise} \end{array} \right. = 0.9$$

Load Reduction Factor

$$\phi M_{n\_slab\_pos} := \frac{\phi_b \cdot \sum M_s}{b} = 216.75 \cdot \frac{kip \cdot ft}{ft}$$

Nominal Moment Capacity of Slab

$$\phi V_{n\_slab} := \phi_v \cdot 2 \cdot \lambda \cdot \sqrt{\frac{f_c}{psi}} \cdot psi \cdot d_{ts} = 37.466 \cdot \frac{kip}{ft}$$

Nominal Shear Capacity of Slab (ACI 318 Section 22.5.5.1)

$$\frac{M_u}{\phi M_{n\_slab\_pos}} = 0.581$$

$$\frac{V_u}{\phi V_{n\_slab}} = 0.694$$

1. Loading

1.1 Dead Loads

$DL_{roof} := 40psf$ Dead load of roof

$t_{wall} := 8in$ Thickness of wall

$DL_{wall} := 84psf$ Dead load of wall

1.2 Snow Load

$SL_{roof} := 21psf$ Snow load of roof

1.3 Seismic loads

In-plane

Special reinforced masonry shear wall,

$R_{seis} := 5$

$I_e := 1.25$

Vw=0.115W

Out-of-plane

$S_{DS} := 0.46g$

$F_p := 0.4 \cdot \frac{S_{DS}}{g} \cdot I_e \cdot DL_{wall} = 19.32 \cdot psf$ Shear load on wall (out of plane)

Seismic Force-Resisting System => Table 12.2-1
BEARING WALL SYSTEMS

Seismic Force-Resisting System Type => Table 12.2-1
Special reinforced masonry shear walls

Building Structure Type => Table 12.8-2
Steel moment-resisting frames

Seismic Design Parameters

Latitude =	38.052 N	
Longitude =	121.011 W	
Soils Report?	Yes	
Site Class =	D	=> Table 20.3-1
Risk Category =	III	=> Table 1.5-1
S_S =	0.490 g	=> USGS Ground Motion Parameter Calculator
S_1 =	0.287 g	
S_{DS} =	0.460 g	
S_{D1} =	0.320 g	
Seismic Design Category =	C	=> Table 11.6-1
Seismic Design Category =	D	=> Table 11.6-2
Seismic Design Category =	D	=> Section 11.6
Importance Factor, I_e =	1.25	=> Table 1.5-2
R =	5	=> Table 12.2-1
Ω_o =	2.5	
C_d =	3.5	
Height Limit =	160 ft	
Structure Height, h_n =	30.0 ft	
C_t =	0.028	=> Table 12.8-2
x =	0.80	=> Table 12.8-2
Approximate Period, T_a =	0.425 s	=> EQ 12.8-7
C_u =	1.4	=> Table 12.8-1
Calc'd Period Upper Limit =	0.596 s	=> Section 12.8-2
Actual Period Known?	No	

$V = C_s W$ => EQ 12.8-1

$$C_s = \frac{S_{DS}}{(R/I_e)} = 0.115$$ => EQ 12.8-2 (Governs)

$$C_{s\text{-}Max} = \frac{S_{D1}}{T(R/I_e)} = 0.188$$ => EQ 12.8-3

$C_{s\text{-}Min} = \max(0.044 S_{DS} I_e, 0.01) = 0.025$ => EQ 12.8-5

for $S_1 > 0.6$ g: $C_{s\text{-}Min} = 0.5 S_1/(R/I_e) =$ N/A => EQ 12.8-6

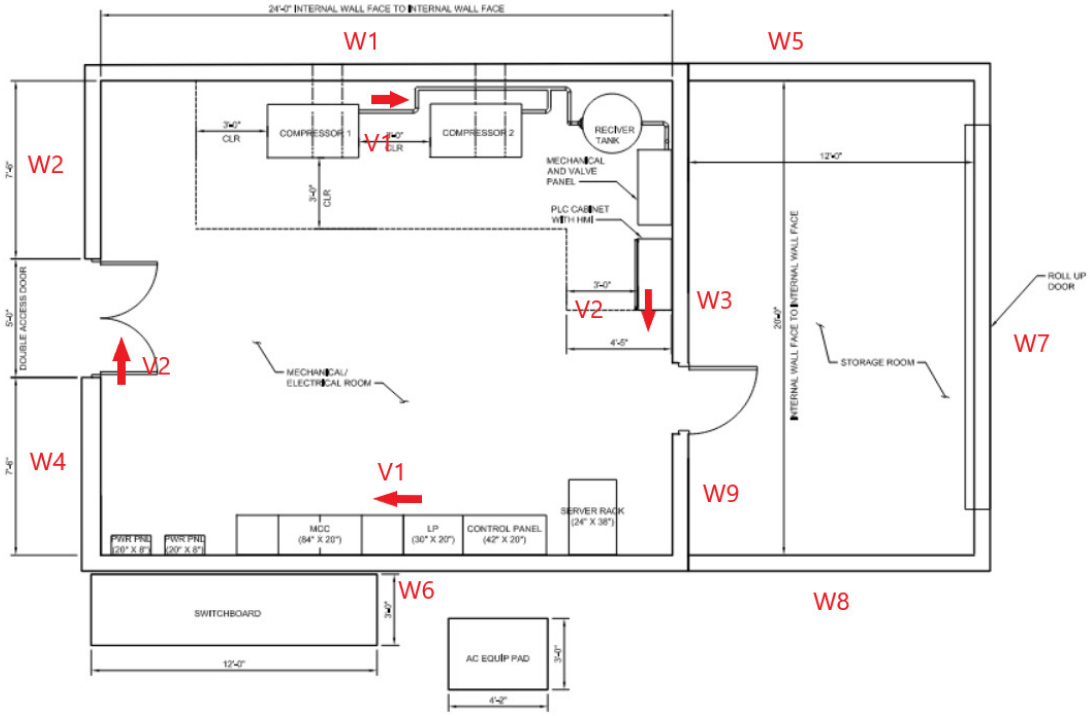
$V = 0.115\ W$

$E_v = 0.2 S_{DS} D$ => EQ 12.4-4

$E_v = 0.092\ D$

Belotta Utlity Structure
Design of CMU walls

Calculated by: UB
Checked by: IG
Date: 12/15/2021



$h_{wall} := 9.5ft$ Height of wall

$L_{roof} := 28ft + 4in$ Length of roof

$B_{roof} := 20ft + 4in$ Width of roof

$W_{roof} := DL_{roof} \cdot L_{roof} \cdot B_{roof} = 2.304 \times 10^{4} \cdot lbf$ Weight of roof

$W_1 := DL_{wall} \cdot 24ft \cdot h_{wall} = 1.915 \times 10^{4} \cdot lbf$ Weight of W1

$W_2 := DL_{wall} \cdot (7ft + 6in) \cdot h_{wall} = 5.985 \times 10^{3} \cdot lbf$ Weight of W2

$W_3 := DL_{wall} \cdot (11ft + 10.75in) \cdot h_{wall} = 9.493 \times 10^{3} \cdot lbf$ Weight of W3

$W_4 := DL_{wall} \cdot (7ft + 6in) \cdot h_{wall} = 5.985 \times 10^{3} \cdot lbf$ Weight of W4

$W_5 := DL_{wall} \cdot (12ft) \cdot h_{wall} = 9.576 \times 10^{3} \cdot lbf$ Weight of W5

$W_6 := DL_{wall} \cdot 24ft \cdot h_{wall} = 1.915 \times 10^{4} \cdot lbf$ Weight of W6

$W_7 := DL_{wall} \cdot (20ft) \cdot h_{wall} = 1.596 \times 10^4 \cdot lbf$ Weight of W7

$W_8 := DL_{wall} \cdot (12ft) \cdot h_{wall} = 9.576 \times 10^3 \cdot lbf$ Weight of W8

$W_9 := DL_{wall} \cdot (5ft + 1.25in) \cdot h_{wall} = 4.073 \times 10^3 \cdot lbf$ Weight of W9

In plane loads on top of walls:

$sV_1 := 0.115 \cdot W_1 = 2.202 \cdot kip$ In plane load of W1

$sV_2 := 0.115 \cdot W_2 = 0.688 \cdot kip$ In plane load of W2

$sV_3 := 0.115 \cdot W_3 = 1.092 \cdot kip$ In plane load of W3

$sV_4 := 0.115 \cdot W_4 = 0.688 \cdot kip$ In plane load of W4

$sV_5 := 0.115 \cdot W_5 = 1.101 \cdot kip$ In plane load of W5

$sV_6 := 0.115 \cdot W_6 = 2.202 \cdot kip$ In plane load of W6

$sV_7 := 0.115 \cdot W_7 = 1.835 \cdot kip$ In plane load of W7

$sV_8 := 0.115 \cdot W_8 = 1.101 \cdot kip$ In plane load of W8

$sV_9 := 0.115 \cdot W_9 = 0.468 \cdot kip$ In plane load of W9

In plane moments

$sM_{i1} := h_{wall} \cdot sV_1 = 20.924 \cdot ft \cdot kip$ In plane moment of W1

$sM_{i2} := h_{wall} \cdot sV_2 = 6.539 \cdot ft \cdot kip$ In plane moment of W2

$sM_{i3} := h_{wall} \cdot sV_3 = 10.371 \cdot ft \cdot kip$ In plane moment of W3

$sM_{i4} := h_{wall} \cdot sV_4 = 6.539 \cdot ft \cdot kip$ In plane moment of W4

$sM_{i5} := h_{wall} \cdot sV_5 = 10.462 \cdot ft \cdot kip$ In plane moment of W5

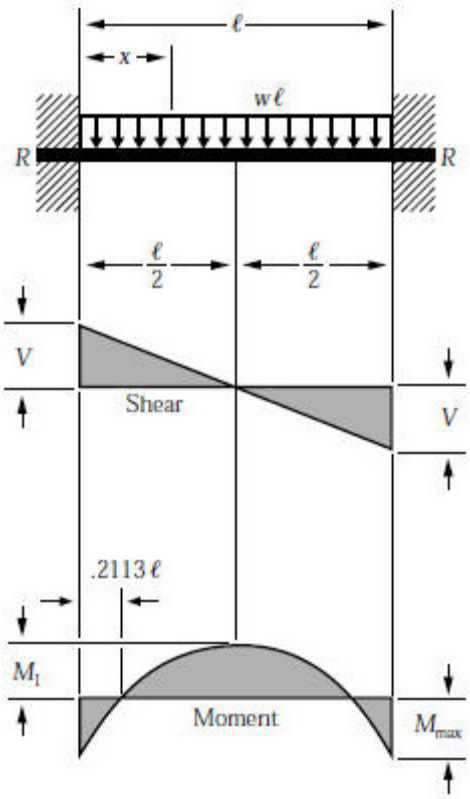
$sM_{i6} := h_{wall} \cdot sV_6 = 20.924 \cdot ft \cdot kip$ In plane moment of W6

$sM_{i7} := h_{wall} \cdot sV_7 = 17.436 \cdot ft \cdot kip$ In plane moment of W7

$sM_{i8} := h_{wall} \cdot sV_8 = 10.462 \cdot ft \cdot kip$ In plane moment of W8

$sM_{i9} := h_{wall} \cdot sV_9 = 4.45 \cdot ft \cdot kip$ In plane moment of W9

Assume the wall to have fixed connections at the corners. Therefore, shear and moment distribution diagrams will be as follows:



$$R = V \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad = \frac{w\ell}{2}$$

$$V_x \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad = w\left(\frac{\ell}{2} - x\right)$$

$$M_{max} \text{ (at ends)} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad = \frac{w\ell^2}{12}$$

$$M_1 \text{ (at center)} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad = \frac{w\ell^2}{24}$$

Out of plane seismic forces applied on top of walls:

$sw_1 := F_p \cdot \frac{24ft}{2} = 0.232 \cdot \frac{kip}{ft}$ Out of plane force of W1

$sw_2 := F_p \cdot \frac{7.5ft}{2} = 0.072 \cdot \frac{kip}{ft}$ Out of plane force of W2

$sw_3 := F_p \cdot \frac{(11ft + 10.75in)}{2} = 0.115 \cdot \frac{kip}{ft}$ Out of plane force of W3

$sw_4 := F_p \cdot \frac{7.5ft}{2} = 0.072 \cdot \frac{kip}{ft}$ Out of plane force of W4

$sw_5 := F_p \cdot \frac{12ft}{2} = 0.116 \cdot \frac{kip}{ft}$ Out of plane force of W5

$sw_6 := F_p \cdot \frac{24ft}{2} = 0.232 \cdot \frac{kip}{ft}$ Out of plane force of W6

$sw_7 := F_p \cdot \frac{20ft}{2} = 0.193 \cdot \frac{kip}{ft}$ Out of plane force of W7

$sw_8 := F_p \cdot \frac{12ft}{2} = 0.116 \cdot \frac{kip}{ft}$ Out of plane force of W8

$sw_9 := F_p \cdot \frac{(5ft + 1.25in)}{2} = 0.049 \cdot \frac{kip}{ft}$ Out of plane force of W9

Out of plane seismic moments applied on top of walls:

$sM_1 := \frac{F_p \cdot (24ft)^2}{12} = 0.927 \cdot \frac{kip \cdot ft}{ft}$ Out of plane moment of W1

$sM_2 := \frac{F_p \cdot (7.5ft)^2}{12} = 0.091 \cdot \frac{kip \cdot ft}{ft}$ Out of plane moment of W2

$sM_3 := \frac{F_p \cdot (11ft + 10.75in)^2}{12} = 0.228 \cdot \frac{kip \cdot ft}{ft}$ Out of plane moment of W3

$$sM_4 := \frac{F_p \cdot (7.5ft)^2}{12} = 0.091 \cdot \frac{kip \cdot ft}{ft}$$ Out of plane moment of W4

$$sM_5 := \frac{F_p \cdot (12ft)^2}{12} = 0.232 \cdot \frac{kip \cdot ft}{ft}$$ Out of plane moment of W5

$$sM_6 := \frac{F_p \cdot (24ft)^2}{12} = 0.927 \cdot \frac{kip \cdot ft}{ft}$$ Out of plane moment of W6

$$sM_7 := \frac{F_p \cdot (20ft)^2}{12} = 0.644 \cdot \frac{kip \cdot ft}{ft}$$ Out of plane moment of W7

$$sM_8 := \frac{F_p \cdot (12ft)^2}{12} = 0.232 \cdot \frac{kip \cdot ft}{ft}$$ Out of plane moment of W8

$$sM_9 := \frac{F_p \cdot (5ft + 1.25in)^2}{12} = 0.042 \cdot \frac{kip \cdot ft}{ft}$$ Out of plane moment of W9

Out of plane axial load:

The out of plane axial load is due to the vertical seismic load on the structure

$$sA_1 := \frac{0.092 \cdot W_1}{24ft} = 0.073 \cdot \frac{kip}{ft}$$ Out of plane axial force of W1

$$sA_2 := \frac{0.092 \cdot W_2}{7.5ft} = 0.073 \cdot \frac{kip}{ft}$$ Out of plane axial force of W2

$$sA_3 := \frac{0.092 \cdot W_3}{11ft + 10.75in} = 0.073 \cdot \frac{kip}{ft}$$ Out of plane axial force of W3

$$sA_4 := \frac{0.092 \cdot W_4}{7.5ft} = 0.073 \cdot \frac{kip}{ft}$$ Out of plane axial force of W4

$$sA_5 := \frac{0.092 \cdot W_5}{12ft} = 0.073 \cdot \frac{kip}{ft}$$ Out of plane axial force of W5

$$sA_6 := \frac{0.092 \cdot W_6}{24ft} = 0.073 \cdot \frac{kip}{ft}$$ Out of plane axial force of W6

$$sA_7 := \frac{0.092 \cdot W_7}{20ft} = 0.073 \cdot \frac{kip}{ft}$$ Out of plane axial force of W7

$$sA_8 := \frac{0.092 \cdot W_8}{12ft} = 0.073 \cdot \frac{kip}{ft}$$ Out of plane axial force of W8

$$sA_9 := \frac{0.092 \cdot W_9}{5ft + 1.25in} = 0.073 \cdot \frac{kip}{ft}$$ Out of plane axial force of W9

Wind Loads

$I := 1.0$	Risk category	ASCE7-16-Table 1.5.1
$V := 88\frac{mi}{hr}$	Wind speed	ASCE7-16-Fig 26.5.1A
$K_d := 1.0$	Wind directionality factor conservatively set to 1.0	ASCE7-16-Table 26.6-1

Exposure Category B was considered for this design according to ASCE7-16 26.7.3

$K_{zt} := 1.0$	There is no topographic effect	ASCE7-16- 26.8.2
$K_e := 1.0$	Ground elevation factor	ASCE7-16-26.9
$G_f := 0.85$		
$GC_{pi} := -0.18$	Internal Pressure Coefficient for enclosed building	ASCE7-16-Table 26.13-1
$K_z := 0.7$	Velocity pressure exposure coefficient-Assume 20 ft total height	ASCE7-16-Table 26.10-1

$$q_z := 0.00256 \cdot K_z \cdot K_{zt} \cdot K_d \cdot K_e \cdot \left(\frac{V}{\frac{mi}{hr}} \right)^2 \cdot \frac{lbf}{ft^2} = 13.877 \cdot psf$$

$GC_{p\_wall} := \frac{(0.53 + 0.8)}{2} = 0.665$ External pressure on wall for 10 deg roof — ASCE7-16-Table 28.3-1

$GC_{p\_roof} := \frac{(-0.69 - 1.07)}{2} = -0.88$ External pressure for roof — ASCE7-16-Table 28.3-1

$$p_{wind\_wall} := q_z \cdot \left(GC_{p\_wall} - GC_{pi} \right) = 11.726 \cdot psf$$

Wind pressure ASCE7-16-28-3-1

$$p_{wind\_roof} := q_z \cdot \left(GC_{p\_roof} + GC_{pi} \right) = -14.71 \cdot psf$$

This wind load is lower than the seismic load of 19.32 psf

Therefore the seismic loads will govern.

	In-plane				Out-of-plane		
Wall ID	**Height h(ft)**	**Shear V(kips)**	**Moment M (kips-ft)**	**Axial Force P (kips)**	**Shear V(kips/ft)**	**Moment M (kips-ft/ft)**	**Axial Force P (kips/ft)**
1	9.5	2.202	20.919	19.15	0.232	0.927	0.073
2	9.5	0.688	6.536	5.985	0.072	0.091	0.073
3	9.5	1.092	10.374	9.493	0.115	0.228	0.073
4	9.5	0.688	6.536	5.985	0.072	0.091	0.073
5	9.5	1.101	10.4595	9.576	0.116	0.232	0.073
6	9.5	2.202	20.919	19.15	0.232	0.927	0.073
7	9.5	1.835	17.4325	15.96	0.193	0.644	0.073
8	9.5	1.101	10.4595	9.576	0.116	0.232	0.073
9	9.5	0.468	4.446	4.073	0.049	0.042	0.073

HDR Inc.	*Job:* Belotta Utility Structure-W1
2365 Iron Point Road	*Sheet No:* 1 OF 1
Folsom, CA 95630	*Calc. by:* UB
Phone: (916)-817-4700	*Check by:*
Fax: (916)-817-4747	*Date:* 12/15/21

Solid-Grouted CMU Wall Design (Special Reinforced Masonry Shear Wall)

WALL PARAMETERS:

Length L :	24.00	Ft
In-Plane Height Hi :	9.50	Ft
Out-of-Plane Height Ho :	9.50	Ft
Thickness t :	7.63	In
Cover C :	2.00	In
CMU Type :	8 x 8 x 16	
Bond Type :	Running Bond	
Radius of Gyration r :	2.20	In

MATERIAL PROPERTIES

fm :	1.50	KSI
fy :	60.00	KSI
fs :	32.00	KSI
Em :	1350.00	KSI
Ev (G) :	540.00	KSI
Es :	29000.00	KSI
n :	21.48	
Fb :	0.68	KSI

DESIGN SERVICE LOADINGS

In-Plane V i :	2.20	KIPS	Out-of-Plane V o :	0.23	KIPS per Ft
In-Plane M i :	20.92	K-Ft	Out-of-Plane M o :	0.93	K-Ft per Ft
In-Plane Axial Load P i :	19.15	KIPS	Out-of-Plane Axial Load P o :	0.07	KIPS per Ft
Use alternate load combination?	No.		Use alternate load combination?	No.	

REBAR ASSIGNMENT

	# of Layers	Rebar Size	Spacing (In)	Area (In^2)	Diam. (In)	d. (In)
Jamb Bars	2	#5		0.31	0.625	284.00
Horiz. Bars	2	#4	@ 32 In oc	0.20	0.500	
Vert. Bars	1	#5	@ 32 In oc	0.31	0.625	3.81

AXIAL DESIGN

In-Plane Bending			**Out-of-Plane Bending Per Foot**		
H / r :	51.79	<= 99	H / r :	51.79	<= 99
Provided Pa :	710.80	KIPS	Provided Pa :	29.62	KIPS
Shear Wall Axial Design is O.K..			**Bearing Wall Axial Design is O.K..**		

FLEXURAL DESIGN

Max. Row Value: 0.0043675

In-Plane Bending			**Out-of-Plane Bending Per Foot**		
M / Vd :	0.40		Min. Flex. Compression f :	0.096	KSI <=Fb O.K.
Flexural Compression f :	0.01	KSI	Min. Flex. Compression f :	-0.095	KSI
Subject to flexural cracking?	No.		Subject to flexural cracking?	Yes.	
Row :	0.000286		Row :	0.002541	
Ma governed by steel :	655.1	K-ft	Ma governed by steel :	1.09	K-Ft per Ft
Ma governed by masonry :	1738.6	K-ft	Ma governed by masonry :	1.25	K-Ft per Ft
Provided Ma :	**655.10**	**K-ft**	**Provided Ma :**	**1.09**	**K-Ft per Ft**
Design M :	20.92	K-ft	Design M:	0.93	K-Ft per Ft
In-Plane Flexural Design is O.K..			**Out-of-Plane Flexural Design is O.K..**		

SHEAR DESIGN

In-Plane				**Out-of-Plane**			
f v :	2.3	PSI		f v :	5.071038	PSI	
F v (w/o Shear Rebars) :	66.0	PSI		F v (w/o Shear Rebars) :	38.72983	PSI	
F v (with Shear Rebars) :	108.4	PSI		F v (with Shear Rebars) :	116.1895	PSI	
OK. No shear reinforcement is required.				**OK. No shear reinforcement is required.**			

REBAR CHECK

	Max. S Horiz.	Max. S Vert.
Shear Capacity	<= 1101 In	
Special Reinforced Masonry Shear Wall	<= 38 In	<= 38 In
Seismic Design Category D	<= 48 In	<= 48 In
Actual Spacing	32 In	32 In
	O.K.	**O.K.**
Vert. Rebar A v :	0.12	In^2 / Ft
Horiz. Rebar A h :	0.15	In^2 / Ft

Req'd Horiz. Rebar A hr : 0.00 In^2 / Ft

Special Reinforced Masonry Shear Wall

A v >= A hr / 3 --- OK

Seismic Design Category D

A v + A h : 0.27 In^2 / Ft

A v + A h >= 0.002 A g --- OK

A v >= 0.0007 A g --- OK

A h >= 0.0007 A g --- OK

Use standard hooks at shear rebar ends.

HDR Inc.	Job: Belotta Utility Structure-W2
2365 Iron Point Road	Sheet No: 1 OF 1
Folsom, CA 95630	Calc. by: UB
Phone: (916)-817-4700	Check by:
Fax: (916)-817-4747	Date: 12/15/21

Solid-Grouted CMU Wall Design (Special Reinforced Masonry Shear Wall)

WALL PARAMETERS:

Length L :	24.00	Ft
In-Plane Height Hi :	9.50	Ft
Out-of-Plane Height Ho :	9.50	Ft
Thickness t :	7.63	In
Cover C :	2.00	In
CMU Type :	8 x 8 x 16	
Bond Type :	Running Bond	
Radius of Gyration r :	2.20	In

MATERIAL PROPERTIES

fm :	1.50	KSI
fy :	60.00	KSI
fs :	32.00	KSI
Em :	1350.00	KSI
Ev (G) :	540.00	KSI
Es :	29000.00	KSI
n :	21.48	
Fb :	0.68	KSI

DESIGN SERVICE LOADINGS

In-Plane V i :	0.69	KIPS	Out-of-Plane V o :	0.07	KIPS per Ft
In-Plane M i :	6.54	K-Ft	Out-of-Plane M o :	0.09	K-Ft per Ft
In-Plane Axial Load P i :	5.99	KIPS	Out-of-Plane Axial Load P o :	0.07	KIPS per Ft
Use alternate load combination?	No.		Use alternate load combination?	No.	

REBAR ASSIGNMENT

	# of Layers	Rebar Size	Spacing (In)	Area (In^2)	Diam. (In)	d. (In)
Jamb Bars	2	#5		0.31	0.625	284.00
Horiz. Bars	2	#4	@ 32 In oc	0.20	0.500	
Vert. Bars	1	#5	@ 32 In oc	0.31	0.625	3.81

AXIAL DESIGN

In-Plane Bending			Out-of-Plane Bending Per Foot		
H / r :	51.79	<= 99	H / r :	51.79	<= 99
Provided Pa :	710.80	KIPS	Provided Pa :	29.62	KIPS
Shear Wall Axial Design is O.K..			Bearing Wall Axial Design is O.K..		

FLEXURAL DESIGN

Max. Row Value: 0.0043675

In-Plane Bending			Out-of-Plane Bending Per Foot		
M / Vd :	0.40		Min. Flex. Compression f :	0.010	KSI <=Fb O.K.
Flexural Compression f :	0.00	KSI	Min. Flex. Compression f :	-0.009	KSI
Subject to flexural cracking?	No.		Subject to flexural cracking?	Yes.	
Row :	0.000286		Row :	0.002541	
Ma governed by steel :	517.2	K-ft	Ma governed by steel :	1.09	K-Ft per Ft
Ma governed by masonry :	1745.0	K-ft	Ma governed by masonry :	1.25	K-Ft per Ft
Provided Ma :	**517.17**	**K-ft**	**Provided Ma :**	**1.09**	**K-Ft per Ft**
Design M :	6.54	K-ft	Design M:	0.09	K-Ft per Ft
In-Plane Flexural Design is O.K..			Out-of-Plane Flexural Design is O.K..		

SHEAR DESIGN

In-Plane				Out-of-Plane			
f v :	0.7	PSI		f v :	1.57377	PSI	
F v (w/o Shear Rebars) :	64.5	PSI		F v (w/o Shear Rebars) :	38.72983	PSI	
F v (with Shear Rebars) :	108.4	PSI		F v (with Shear Rebars) :	116.1895	PSI	
OK. No shear reinforcement is required.				OK. No shear reinforcement is required.			

REBAR CHECK

	Max. S Horiz.	Max. S Vert.
Shear Capacity	<= 3522 In	
Special Reinforced Masonry Shear Wall	<= 38 In	<= 38 In
Seismic Design Category D	<= 48 In	<= 48 In
Actual Spacing	32 In	32 In
	O.K.	O.K.
Vert. Rebar A v :	0.12	In^2 / Ft
Horiz. Rebar A h :	0.15	In^2 / Ft

Req'd Horiz. Rebar A hr : 0.00 In^2 / Ft

Special Reinforced Masonry Shear Wall

A v >= A hr / 3 --- OK

Seismic Design Category D

A v + A h : 0.27 In^2 / Ft

A v + A h >= 0.002 A g --- OK

A v >= 0.0007 A g --- OK

A h >= 0.0007 A g --- OK

Use standard hooks at shear rebar ends.

HDR Inc.	Job: Belotta Utility Structure-W3
2365 Iron Point Road	Sheet No: 1 OF 1
Folsom, CA 95630	Calc. by: UB
Phone: (916)-817-4700	Check by:
Fax: (916)-817-4747	Date: 12/15/21

Solid-Grouted CMU Wall Design (Special Reinforced Masonry Shear Wall)

WALL PARAMETERS:

Length L :	24.00	Ft
In-Plane Height Hi :	9.50	Ft
Out-of-Plane Height Ho :	9.50	Ft
Thickness t :	7.63	In
Cover C :	2.00	In
CMU Type :	8 x 8 x 16	
Bond Type :	Running Bond	
Radius of Gyration r :	2.20	In

MATERIAL PROPERTIES

fm :	1.50	KSI
fy :	60.00	KSI
fs :	32.00	KSI
Em :	1350.00	KSI
Ev (G) :	540.00	KSI
Es :	29000.00	KSI
n :	21.48	
Fb :	0.68	KSI

DESIGN SERVICE LOADINGS

In-Plane V i :	1.09	KIPS	Out-of-Plane V o :	0.12	KIPS per Ft
In-Plane M i :	10.37	K-Ft	Out-of-Plane M o :	0.23	K-Ft per Ft
In-Plane Axial Load P i :	9.49	KIPS	Out-of-Plane Axial Load P o :	0.07	KIPS per Ft
Use alternate load combination?	No.		Use alternate load combination?	No.	

REBAR ASSIGNMENT

	# of Layers	Rebar Size	Spacing (In)	Area (In^2)	Diam. (In)	d. (In)
Jamb Bars	2	#5		0.31	0.625	284.00
Horiz. Bars	2	#4	@ 32 In oc	0.20	0.500	
Vert. Bars	1	#5	@ 32 In oc	0.31	0.625	3.81

AXIAL DESIGN

In-Plane Bending			Out-of-Plane Bending Per Foot		
H / r :	51.79	<= 99	H / r :	51.79	<= 99
Provided Pa :	710.80	KIPS	Provided Pa :	29.62	KIPS
Shear Wall Axial Design is O.K..			Bearing Wall Axial Design is O.K..		

FLEXURAL DESIGN

Max. Row Value: 0.0043675

In-Plane Bending			Out-of-Plane Bending Per Foot		
M / Vd :	0.40		Min. Flex. Compression f :	0.024	KSI <=Fb O.K.
Flexural Compression f :	0.00	KSI	Min. Flex. Compression f :	-0.023	KSI
Subject to flexural cracking?	No.		Subject to flexural cracking?	Yes.	
Row :	0.000286		Row :	0.002541	
Ma governed by steel :	554.3	K-ft	Ma governed by steel :	1.09	K-Ft per Ft
Ma governed by masonry :	1742.2	K-ft	Ma governed by masonry :	1.25	K-Ft per Ft
Provided Ma :	**554.29**	**K-ft**	**Provided Ma :**	**1.09**	**K-Ft per Ft**
Design M :	10.37	K-ft	Design M:	0.23	K-Ft per Ft
In-Plane Flexural Design is O.K..			Out-of-Plane Flexural Design is O.K..		

SHEAR DESIGN

In-Plane				Out-of-Plane			
f v :	1.1	PSI		f v :	2.513661	PSI	
F v (w/o Shear Rebars) :	64.9	PSI		F v (w/o Shear Rebars) :	38.72983	PSI	
F v (with Shear Rebars) :	108.4	PSI		F v (with Shear Rebars) :	116.1895	PSI	
OK. No shear reinforcement is required.				OK. No shear reinforcement is required.			

REBAR CHECK

	Max. S Horiz.	Max. S Vert.
Shear Capacity	<= 2219 In	
Special Reinforced Masonry Shear Wall	<= 38 In	<= 38 In
Seismic Design Category D	<= 48 In	<= 48 In
Actual Spacing	32 In	32 In
	O.K.	O.K.

Vert. Rebar A v : 0.12 In^2 / Ft

Horiz. Rebar A h : 0.15 In^2 / Ft

Req'd Horiz. Rebar A hr : 0.00 In^2 / Ft

Special Reinforced Masonry Shear Wall

A v >= A hr / 3 --- OK

Seismic Design Category D

A v + A h : 0.27 In^2 / Ft

A v + A h >= 0.002 A g --- OK

A v >= 0.0007 A g --- OK

A h >= 0.0007 A g --- OK

Use standard hooks at shear rebar ends.

HDR Inc.	Job: Belotta Utility Structure-W4
2365 Iron Point Road	Sheet No: 1 OF 1
Folsom, CA 95630	Calc. by: UB
Phone: (916)-817-4700	Check by:
Fax: (916)-817-4747	Date: 12/15/21

Solid-Grouted CMU Wall Design (Special Reinforced Masonry Shear Wall)

WALL PARAMETERS:

Length L :	24.00	Ft
In-Plane Height Hi :	9.50	Ft
Out-of-Plane Height Ho :	9.50	Ft
Thickness t :	7.63	In
Cover C :	2.00	In
CMU Type :	8 x 8 x 16	
Bond Type :	Running Bond	
Radius of Gyration r :	2.20	In

MATERIAL PROPERTIES

fm :	1.50	KSI
fy :	60.00	KSI
fs :	32.00	KSI
Em :	1350.00	KSI
Ev (G) :	540.00	KSI
Es :	29000.00	KSI
n :	21.48	
Fb :	0.68	KSI

DESIGN SERVICE LOADINGS

In-Plane V i :	0.69	KIPS	Out-of-Plane V o :	0.07	KIPS per Ft
In-Plane M i :	6.54	K-Ft	Out-of-Plane M o :	0.09	K-Ft per Ft
In-Plane Axial Load P i :	5.99	KIPS	Out-of-Plane Axial Load P o :	0.07	KIPS per Ft
Use alternate load combination?	No.		Use alternate load combination?	No.	

REBAR ASSIGNMENT

	# of Layers	Rebar Size	Spacing (In)	Area (In^2)	Diam. (In)	d. (In)
Jamb Bars	2	#5		0.31	0.625	284.00
Horiz. Bars	2	#4	@ 32 In oc	0.20	0.500	
Vert. Bars	1	#5	@ 32 In oc	0.31	0.625	3.81

AXIAL DESIGN

In-Plane Bending			Out-of-Plane Bending Per Foot		
H / r :	51.79	<= 99	H / r :	51.79	<= 99
Provided Pa :	710.80	KIPS	Provided Pa :	29.62	KIPS
Shear Wall Axial Design is O.K..			Bearing Wall Axial Design is O.K..		

FLEXURAL DESIGN

Max. Row Value: 0.0043675

In-Plane Bending			Out-of-Plane Bending Per Foot		
M / Vd :	0.40		Min. Flex. Compression f :	0.010	KSI <=Fb O.K.
Flexural Compression f :	0.00	KSI	Min. Flex. Compression f :	-0.009	KSI
Subject to flexural cracking?	No.		Subject to flexural cracking?	Yes.	
Row :	0.000286		Row :	0.002541	
Ma governed by steel :	517.2	K-ft	Ma governed by steel :	1.09	K-Ft per Ft
Ma governed by masonry :	1745.0	K-ft	Ma governed by masonry :	1.25	K-Ft per Ft
Provided Ma :	**517.17**	**K-ft**	**Provided Ma :**	**1.09**	**K-Ft per Ft**
Design M :	6.54	K-ft	Design M:	0.09	K-Ft per Ft
In-Plane Flexural Design is O.K..			Out-of-Plane Flexural Design is O.K..		

SHEAR DESIGN

In-Plane				Out-of-Plane			
f v :	0.7	PSI		f v :	1.57377	PSI	
F v (w/o Shear Rebars) :	64.5	PSI		F v (w/o Shear Rebars) :	38.72983	PSI	
F v (with Shear Rebars) :	108.4	PSI		F v (with Shear Rebars) :	116.1895	PSI	
OK. No shear reinforcement is required.				OK. No shear reinforcement is required.			

REBAR CHECK

	Max. S Horiz.	Max. S Vert.
Shear Capacity	<= 3522 In	
Special Reinforced Masonry Shear Wall	<= 38 In	<= 38 In
Seismic Design Category D	<= 48 In	<= 48 In
Actual Spacing	32 In	32 In
	O.K.	O.K.
Vert. Rebar A v :	0.12	In^2 / Ft
Horiz. Rebar A h :	0.15	In^2 / Ft

Req'd Horiz. Rebar A hr : 0.00 In^2 / Ft

Special Reinforced Masonry Shear Wall

A v >= A hr / 3 --- OK

Seismic Design Category D

A v + A h : 0.27 In^2 / Ft

A v + A h >= 0.002 A g --- OK

A v >= 0.0007 A g --- OK

A h >= 0.0007 A g --- OK

Use standard hooks at shear rebar ends.

HDR Inc.	Job: Belotta Utility Structure-W5
2365 Iron Point Road	Sheet No: 1 OF 1
Folsom, CA 95630	Calc. by: UB
Phone: (916)-817-4700	Check by:
Fax: (916)-817-4747	Date: 12/15/21

Solid-Grouted CMU Wall Design (Special Reinforced Masonry Shear Wall)

WALL PARAMETERS:

Length L :	24.00	Ft
In-Plane Height Hi :	9.50	Ft
Out-of-Plane Height Ho :	9.50	Ft
Thickness t :	7.63	In
Cover C :	2.00	In
CMU Type :	8 x 8 x 16	
Bond Type :	Running Bond	
Radius of Gyration r :	2.20	In

MATERIAL PROPERTIES

fm :	1.50	KSI
fy :	60.00	KSI
fs :	32.00	KSI
Em :	1350.00	KSI
Ev (G) :	540.00	KSI
Es :	29000.00	KSI
n :	21.48	
Fb :	0.68	KSI

DESIGN SERVICE LOADINGS

In-Plane V i :	1.10	KIPS	Out-of-Plane V o :	0.12	KIPS per Ft
In-Plane M i :	10.46	K-Ft	Out-of-Plane M o :	0.23	K-Ft per Ft
In-Plane Axial Load P i :	9.58	KIPS	Out-of-Plane Axial Load P o :	0.07	KIPS per Ft
Use alternate load combination?	No.		Use alternate load combination?	No.	

REBAR ASSIGNMENT

	# of Layers	Rebar Size	Spacing (In)	Area (In^2)	Diam. (In)	d. (In)
Jamb Bars	2	#5		0.31	0.625	284.00
Horiz. Bars	2	#4	@ 32 In oc	0.20	0.500	
Vert. Bars	1	#5	@ 32 In oc	0.31	0.625	3.81

AXIAL DESIGN

In-Plane Bending			Out-of-Plane Bending Per Foot		
H / r :	51.79	<= 99	H / r :	51.79	<= 99
Provided Pa :	710.80	KIPS	Provided Pa :	29.62	KIPS
Shear Wall Axial Design is O.K..			Bearing Wall Axial Design is O.K..		

FLEXURAL DESIGN

Max. Row Value: 0.0043675

In-Plane Bending			Out-of-Plane Bending Per Foot		
M / Vd :	0.40		Min. Flex. Compression f :	0.025	KSI <=Fb O.K.
Flexural Compression f :	0.00	KSI	Min. Flex. Compression f :	-0.023	KSI
Subject to flexural cracking?	No.		Subject to flexural cracking?	Yes.	
Row :	0.000286		Row :	0.002541	
Ma governed by steel :	555.2	K-ft	Ma governed by steel :	1.09	K-Ft per Ft
Ma governed by masonry :	1742.2	K-ft	Ma governed by masonry :	1.25	K-Ft per Ft
Provided Ma :	**555.16**	**K-ft**	**Provided Ma :**	**1.09**	**K-Ft per Ft**
Design M :	10.46	K-ft	Design M:	0.23	K-Ft per Ft
In-Plane Flexural Design is O.K..			Out-of-Plane Flexural Design is O.K..		

SHEAR DESIGN

In-Plane				Out-of-Plane			
f v :	1.1	PSI		f v :	2.535519	PSI	
F v (w/o Shear Rebars) :	64.9	PSI		F v (w/o Shear Rebars) :	38.72983	PSI	
F v (with Shear Rebars) :	108.4	PSI		F v (with Shear Rebars) :	116.1895	PSI	
OK. No shear reinforcement is required.				OK. No shear reinforcement is required.			

REBAR CHECK

	Max. S Horiz.	Max. S Vert.
Shear Capacity	<= 2201 In	
Special Reinforced Masonry Shear Wall	<= 38 In	<= 38 In
Seismic Design Category D	<= 48 In	<= 48 In
Actual Spacing	32 In	32 In
	O.K.	O.K.
Vert. Rebar A v :	0.12	In^2 / Ft
Horiz. Rebar A h :	0.15	In^2 / Ft

Req'd Horiz. Rebar A hr :	0.00	In^2 / Ft
Special Reinforced Masonry Shear Wall		
A v >= A hr / 3 --- OK		
Seismic Design Category D		
A v + A h :	0.27	In^2 / Ft
A v + A h >= 0.002 A g --- OK		
A v >= 0.0007 A g --- OK		
A h >= 0.0007 A g --- OK		
Use standard hooks at shear rebar ends.		

HDR Inc.	*Job:* Belotta Utility Structure-W6
2365 Iron Point Road	*Sheet No:* 1 OF 1
Folsom, CA 95630	*Calc. by:* UB
Phone: (916)-817-4700	*Check by:*
Fax: (916)-817-4747	*Date:* 12/15/21

Solid-Grouted CMU Wall Design (Special Reinforced Masonry Shear Wall)

WALL PARAMETERS:

Length L :	24.00	Ft
In-Plane Height Hi :	9.50	Ft
Out-of-Plane Height Ho :	9.50	Ft
Thickness t :	7.63	In
Cover C :	2.00	In
CMU Type :	8 x 8 x 16	
Bond Type :	Running Bond	
Radius of Gyration r :	2.20	In

MATERIAL PROPERTIES

fm :	1.50	KSI
fy :	60.00	KSI
fs :	32.00	KSI
Em :	1350.00	KSI
Ev (G) :	540.00	KSI
Es :	29000.00	KSI
n :	21.48	
Fb :	0.68	KSI

DESIGN SERVICE LOADINGS

In-Plane V i :	2.20	KIPS	Out-of-Plane V o :	0.23	KIPS per Ft
In-Plane M i :	20.92	K-Ft	Out-of-Plane M o :	0.93	K-Ft per Ft
In-Plane Axial Load P i :	19.15	KIPS	Out-of-Plane Axial Load P o :	0.07	KIPS per Ft
Use alternate load combination?	No.		Use alternate load combination?	No.	

REBAR ASSIGNMENT

	# of Layers	Rebar Size	Spacing (In)	Area (In^2)	Diam. (In)	d. (In)
Jamb Bars	2	#5		0.31	0.625	284.00
Horiz. Bars	2	#4	@ 32 In oc	0.20	0.500	
Vert. Bars	1	#5	@ 32 In oc	0.31	0.625	3.81

AXIAL DESIGN

In-Plane Bending			**Out-of-Plane Bending Per Foot**		
H / r :	51.79	<= 99	H / r :	51.79	<= 99
Provided Pa :	710.80	KIPS	Provided Pa :	29.62	KIPS
Shear Wall Axial Design is O.K..			**Bearing Wall Axial Design is O.K..**		

FLEXURAL DESIGN

Max. Row Value: 0.0043675

In-Plane Bending			**Out-of-Plane Bending Per Foot**		
M / Vd :	0.40		Min. Flex. Compression f :	0.096	KSI <=Fb O.K.
Flexural Compression f :	0.01	KSI	Min. Flex. Compression f :	-0.095	KSI
Subject to flexural cracking?	No.		Subject to flexural cracking?	Yes.	
Row :	0.000286		Row :	0.002541	
Ma governed by steel :	655.1	K-ft	Ma governed by steel :	1.09	K-Ft per Ft
Ma governed by masonry :	1738.6	K-ft	Ma governed by masonry :	1.25	K-Ft per Ft
Provided Ma :	**655.10**	**K-ft**	**Provided Ma :**	**1.09**	**K-Ft per Ft**
Design M :	20.92	K-ft	Design M:	0.93	K-Ft per Ft
In-Plane Flexural Design is O.K..			**Out-of-Plane Flexural Design is O.K..**		

SHEAR DESIGN

In-Plane				**Out-of-Plane**			
f v :	2.3	PSI		f v :	5.071038	PSI	
F v (w/o Shear Rebars) :	66.0	PSI		F v (w/o Shear Rebars) :	38.72983	PSI	
F v (with Shear Rebars) :	108.4	PSI		F v (with Shear Rebars) :	116.1895	PSI	
OK. No shear reinforcement is required.				**OK. No shear reinforcement is required.**			

REBAR CHECK

	Max. S Horiz.	Max. S Vert.
Shear Capacity	<= 1101 In	
Special Reinforced Masonry Shear Wall	<= 38 In	<= 38 In
Seismic Design Category D	<= 48 In	<= 48 In
Actual Spacing	32 In	32 In
	O.K.	**O.K.**
Vert. Rebar A v :	0.12	In^2 / Ft
Horiz. Rebar A h :	0.15	In^2 / Ft

Req'd Horiz. Rebar A hr : 0.00 In^2 / Ft

Special Reinforced Masonry Shear Wall

A v >= A hr / 3 --- OK

Seismic Design Category D

A v + A h : 0.27 In^2 / Ft

A v + A h >= 0.002 A g --- OK

A v >= 0.0007 A g --- OK

A h >= 0.0007 A g --- OK

Use standard hooks at shear rebar ends.

HDR Inc.	*Job:* Belotta Utility Structure-W7
2365 Iron Point Road	*Sheet No:* 1 OF 1
Folsom, CA 95630	*Calc. by:* UB
Phone: (916)-817-4700	*Check by:*
Fax: (916)-817-4747	*Date:* 12/15/21

Solid-Grouted CMU Wall Design (Special Reinforced Masonry Shear Wall)

WALL PARAMETERS:

Length L :	24.00	Ft
In-Plane Height Hi :	9.50	Ft
Out-of-Plane Height Ho :	9.50	Ft
Thickness t :	7.63	In
Cover C :	2.00	In
CMU Type :	8 x 8 x 16	
Bond Type :	Running Bond	
Radius of Gyration r :	2.20	In

MATERIAL PROPERTIES

fm :	1.50	KSI
fy :	60.00	KSI
fs :	32.00	KSI
Em :	1350.00	KSI
Ev (G) :	540.00	KSI
Es :	29000.00	KSI
n :	21.48	
Fb :	0.68	KSI

DESIGN SERVICE LOADINGS

In-Plane V i :	1.84	KIPS	Out-of-Plane V o :	0.19	KIPS per Ft
In-Plane M i :	17.43	K-Ft	Out-of-Plane M o :	0.64	K-Ft per Ft
In-Plane Axial Load P i :	15.96	KIPS	Out-of-Plane Axial Load P o :	0.07	KIPS per Ft
Use alternate load combination?	No.		Use alternate load combination?	No.	

REBAR ASSIGNMENT

	# of Layers	Rebar Size	Spacing (In)	Area (In^2)	Diam. (In)	d. (In)
Jamb Bars	2	#5		0.31	0.625	284.00
Horiz. Bars	2	#4	@ 32 In oc	0.20	0.500	
Vert. Bars	1	#5	@ 32 In oc	0.31	0.625	3.81

AXIAL DESIGN

In-Plane Bending			**Out-of-Plane Bending Per Foot**		
H / r :	51.79	<= 99	H / r :	51.79	<= 99
Provided Pa :	710.80	KIPS	Provided Pa :	29.62	KIPS
Shear Wall Axial Design is O.K..			**Bearing Wall Axial Design is O.K..**		

FLEXURAL DESIGN

Max. Row Value: 0.0043675

In-Plane Bending			**Out-of-Plane Bending Per Foot**		
M / Vd :	0.40		Min. Flex. Compression f :	0.067	KSI <=Fb O.K.
Flexural Compression f :	0.01	KSI	Min. Flex. Compression f :	-0.066	KSI
Subject to flexural cracking?	No.		Subject to flexural cracking?	Yes.	
Row :	0.000286		Row :	0.002541	
Ma governed by steel :	622.0	K-ft	Ma governed by steel :	1.09	K-Ft per Ft
Ma governed by masonry :	1739.1	K-ft	Ma governed by masonry :	1.25	K-Ft per Ft
Provided Ma :	**622.01**	**K-ft**	**Provided Ma :**	**1.09**	**K-Ft per Ft**
Design M :	17.43	K-ft	Design M:	0.64	K-Ft per Ft
In-Plane Flexural Design is O.K..			**Out-of-Plane Flexural Design is O.K..**		

SHEAR DESIGN

In-Plane f v :	1.9	PSI	**Out-of-Plane** f v :	4.218579	PSI
F v (w/o Shear Rebars) :	65.7	PSI	F v (w/o Shear Rebars) :	38.72983	PSI
F v (with Shear Rebars) :	108.4	PSI	F v (with Shear Rebars) :	116.1895	PSI
OK. No shear reinforcement is required.			**OK. No shear reinforcement is required.**		

REBAR CHECK

	Max. S Horiz.	Max. S Vert.
Shear Capacity	<= 1321 In	
Special Reinforced Masonry Shear Wall	<= 38 In	<= 38 In
Seismic Design Category D	<= 48 In	<= 48 In
Actual Spacing	32 In	32 In
	O.K.	**O.K.**
Vert. Rebar A v :	0.12	In^2 / Ft
Horiz. Rebar A h :	0.15	In^2 / Ft

Req'd Horiz. Rebar A hr : 0.00 In^2 / Ft

Special Reinforced Masonry Shear Wall

A v >= A hr / 3 --- OK

Seismic Design Category D

A v + A h : 0.27 In^2 / Ft

A v + A h >= 0.002 A g --- OK

A v >= 0.0007 A g --- OK

A h >= 0.0007 A g --- OK

Use standard hooks at shear rebar ends.

HDR Inc.	Job: Belotta Utility Structure-W8
2365 Iron Point Road	Sheet No: 1 OF 1
Folsom, CA 95630	Calc. by: UB
Phone: (916)-817-4700	Check by:
Fax: (916)-817-4747	Date: 12/15/21

Solid-Grouted CMU Wall Design (Special Reinforced Masonry Shear Wall)

WALL PARAMETERS:

Length L :	24.00	Ft
In-Plane Height Hi :	9.50	Ft
Out-of-Plane Height Ho :	9.50	Ft
Thickness t :	7.63	In
Cover C :	2.00	In
CMU Type :	8 x 8 x 16	
Bond Type :	Running Bond	
Radius of Gyration r :	2.20	In

MATERIAL PROPERTIES

fm :	1.50	KSI
fy :	60.00	KSI
fs :	32.00	KSI
Em :	1350.00	KSI
Ev (G) :	540.00	KSI
Es :	29000.00	KSI
n :	21.48	
Fb :	0.68	KSI

DESIGN SERVICE LOADINGS

In-Plane V i :	1.10	KIPS	Out-of-Plane V o :	0.12	KIPS per Ft
In-Plane M i :	10.46	K-Ft	Out-of-Plane M o :	0.23	K-Ft per Ft
In-Plane Axial Load P i :	9.58	KIPS	Out-of-Plane Axial Load P o :	0.07	KIPS per Ft
Use alternate load combination?	No.		Use alternate load combination?	No.	

REBAR ASSIGNMENT

	# of Layers	Rebar Size	Spacing (In)	Area (In^2)	Diam. (In)	d. (In)
Jamb Bars	2	#5		0.31	0.625	284.00
Horiz. Bars	2	#4	@ 32 In oc	0.20	0.500	
Vert. Bars	1	#5	@ 32 In oc	0.31	0.625	3.81

AXIAL DESIGN

In-Plane Bending			Out-of-Plane Bending Per Foot		
H / r :	51.79	<= 99	H / r :	51.79	<= 99
Provided Pa :	710.80	KIPS	Provided Pa :	29.62	KIPS
Shear Wall Axial Design is O.K..			Bearing Wall Axial Design is O.K..		

FLEXURAL DESIGN

Max. Row Value: 0.0043675

In-Plane Bending			Out-of-Plane Bending Per Foot		
M / Vd :	0.40		Min. Flex. Compression f :	0.025	KSI <=Fb O.K.
Flexural Compression f :	0.00	KSI	Min. Flex. Compression f :	-0.023	KSI
Subject to flexural cracking?	No.		Subject to flexural cracking?	Yes.	
Row :	0.000286		Row :	0.002541	
Ma governed by steel :	555.2	K-ft	Ma governed by steel :	1.09	K-Ft per Ft
Ma governed by masonry :	1742.2	K-ft	Ma governed by masonry :	1.25	K-Ft per Ft
Provided Ma :	555.16	K-ft	Provided Ma :	1.09	K-Ft per Ft
Design M :	10.46	K-ft	Design M:	0.23	K-Ft per Ft
In-Plane Flexural Design is O.K..			Out-of-Plane Flexural Design is O.K..		

SHEAR DESIGN

In-Plane				Out-of-Plane			
f v :	1.1	PSI		f v :	2.535519	PSI	
F v (w/o Shear Rebars) :	64.9	PSI		F v (w/o Shear Rebars) :	38.72983	PSI	
F v (with Shear Rebars) :	108.4	PSI		F v (with Shear Rebars) :	116.1895	PSI	
OK. No shear reinforcement is required.				OK. No shear reinforcement is required.			

REBAR CHECK

	Max. S Horiz.	Max. S Vert.
Shear Capacity	<= 2201 In	
Special Reinforced Masonry Shear Wall	<= 38 In	<= 38 In
Seismic Design Category D	<= 48 In	<= 48 In
Actual Spacing	32 In	32 In
	O.K.	O.K.

Vert. Rebar A v :	0.12	In^2 / Ft
Horiz. Rebar A h :	0.15	In^2 / Ft

Req'd Horiz. Rebar A hr :	0.00	In^2 / Ft

Special Reinforced Masonry Shear Wall

A v >= A hr / 3 --- OK

Seismic Design Category D

A v + A h :	0.27	In^2 / Ft

A v + A h >= 0.002 A g --- OK

A v >= 0.0007 A g --- OK

A h >= 0.0007 A g --- OK

Use standard hooks at shear rebar ends.

HDR Inc.	Job: Belotta Utility Structure-W9
2365 Iron Point Road	Sheet No: 1 OF 1
Folsom, CA 95630	Calc. by: UB
Phone: (916)-817-4700	Check by:
Fax: (916)-817-4747	Date: 12/15/21

Solid-Grouted CMU Wall Design (Special Reinforced Masonry Shear Wall)

WALL PARAMETERS:

Length L :	24.00	Ft
In-Plane Height Hi :	9.50	Ft
Out-of-Plane Height Ho :	9.50	Ft
Thickness t :	7.63	In
Cover C :	2.00	In
CMU Type :	8 x 8 x 16	
Bond Type :	Running Bond	
Radius of Gyration r :	2.20	In

MATERIAL PROPERTIES

fm :	1.50	KSI
fy :	60.00	KSI
fs :	32.00	KSI
Em :	1350.00	KSI
Ev (G) :	540.00	KSI
Es :	29000.00	KSI
n :	21.48	
Fb :	0.68	KSI

DESIGN SERVICE LOADINGS

In-Plane V i :	0.47	KIPS	Out-of-Plane V o :	0.05	KIPS per Ft
In-Plane M i :	4.45	K-Ft	Out-of-Plane M o :	0.04	K-Ft per Ft
In-Plane Axial Load P i :	4.07	KIPS	Out-of-Plane Axial Load P o :	0.07	KIPS per Ft
Use alternate load combination?	No.		Use alternate load combination?	No.	

REBAR ASSIGNMENT

	# of Layers	Rebar Size	Spacing (In)	Area (In^2)	Diam. (In)	d. (In)
Jamb Bars	2	#5		0.31	0.625	284.00
Horiz. Bars	2	#4	@ 32 In oc	0.20	0.500	
Vert. Bars	1	#5	@ 32 In oc	0.31	0.625	3.81

AXIAL DESIGN

In-Plane Bending			Out-of-Plane Bending Per Foot		
H / r :	51.79	<= 99	H / r :	51.79	<= 99
Provided Pa :	710.80	KIPS	Provided Pa :	29.62	KIPS
Shear Wall Axial Design is O.K..			Bearing Wall Axial Design is O.K..		

FLEXURAL DESIGN

Max. Row Value: 0.0043675

In-Plane Bending			Out-of-Plane Bending Per Foot		
M / Vd :	0.40		Min. Flex. Compression f :	0.005	KSI <=Fb O.K.
Flexural Compression f :	0.00	KSI	Min. Flex. Compression f :	-0.004	KSI
Subject to flexural cracking?	No.		Subject to flexural cracking?	Yes.	
Row :	0.000286		Row :	0.002541	
Ma governed by steel :	496.8	K-ft	Ma governed by steel :	1.09	K-Ft per Ft
Ma governed by masonry :	1746.8	K-ft	Ma governed by masonry :	1.25	K-Ft per Ft
Provided Ma :	**496.81**	**K-ft**	**Provided Ma :**	**1.09**	**K-Ft per Ft**
Design M :	4.45	K-ft	Design M:	0.04	K-Ft per Ft
In-Plane Flexural Design is O.K..			Out-of-Plane Flexural Design is O.K..		

SHEAR DESIGN

In-Plane				Out-of-Plane			
f v :	0.5	PSI		f v :	1.071038	PSI	
F v (w/o Shear Rebars) :	64.3	PSI		F v (w/o Shear Rebars) :	38.72983	PSI	
F v (with Shear Rebars) :	108.4	PSI		F v (with Shear Rebars) :	116.1895	PSI	
OK. No shear reinforcement is required.				OK. No shear reinforcement is required.			

REBAR CHECK

	Max. S Horiz.	Max. S Vert.
Shear Capacity	<= 5178 In	
Special Reinforced Masonry Shear Wall	<= 38 In	<= 38 In
Seismic Design Category D	<= 48 In	<= 48 In
Actual Spacing	32 In	32 In
	O.K.	O.K.

Vert. Rebar A v :	0.12	In^2 / Ft
Horiz. Rebar A h :	0.15	In^2 / Ft

Req'd Horiz. Rebar A hr :	0.00	In^2 / Ft

Special Reinforced Masonry Shear Wall

A v >= A hr / 3 --- OK

Seismic Design Category D

A v + A h :	0.27	In^2 / Ft

A v + A h >= 0.002 A g --- OK

A v >= 0.0007 A g --- OK

A h >= 0.0007 A g --- OK

Use standard hooks at shear rebar ends.

Client :	Stockton East Water District
Project :	Bellota Weir Modifications Project
Project No :	10209137
Title :	Concrete Crane Pad Strength Design
Purpose:	This calculation package is for the strength design of the concrete pad that will be installed as a part of the Bellota Weir Modifications Project to facilitate use of a crane for construction and operation of the fish passage and weir structure. The calculations assume that the crane will be a 200-ton HSL Crawler Crane for conservative design estimates. The calculations check the strength design of the pad and do not check the pad for stability. The calculations were performed according to relevant provisions in ACI 318-19.
Originator :	Ryan Bunyan, EIT Date : January 23, 2022
Checked By :	Iman Ghorbani, PE

Design Inputs

Steel Properties

$E_s := 29000ksi$ — Steel modulus of elasticity

$\varepsilon_y := 0.00207$ — Steel yield strain of rebar

$F_{y.rb} := 60 \cdot ksi$ — Steel yield strength of rebar (ASTM A615 Gr 60)

$F_{u.rb} := 90 \cdot ksi$ — Steel ultimate strength of rebar (ASTM A615 Gr 60)

$$rebar := \begin{pmatrix} 0 & 0 & 0 \\ 0 & 0 & 0 \\ 0 & 0 & 0 \\ 3 & 0.375 & 0.11 \\ 4 & 0.500 & 0.20 \\ 5 & 0.625 & 0.31 \\ 6 & 0.750 & 0.44 \\ 7 & 0.875 & 0.60 \\ 8 & 1.000 & 0.79 \\ 9 & 1.128 & 1.00 \\ 10 & 1.270 & 1.27 \\ 11 & 1.410 & 1.56 \end{pmatrix}$$

Steel Rebar Dimensions.

The first column denotes the size designation (imperial), the second column denotes the diameter of each size in inches, and the third column denotes the area of each size in square inches. The first three rows are placeholders to make the coding work in the subsequent calculations.

Concrete Properties

$\gamma_c := 150pcf$ — Unit weight of concrete

$f_c := 4500psi$ — Minimum 28-day Compressive strength of concrete

$E_c := 57000 \cdot \sqrt{\frac{f_c \cdot in^2}{lbf}} \cdot \frac{lbf}{in^2} = 3823676.24 \cdot psi$ — Concrete Elastic Modulus (ACI 318)

$$\beta_1 := \begin{cases} 0.85 & \text{if } f_c \le 4000psi \\ \max\left[0.85 - 0.05 \cdot \left(\frac{f_c - 4000psi}{1000psi}\right), 0.65\right] & \text{otherwise} \end{cases} = 0.83$$

Compression Block Factor (ACI 318)

$\lambda := 1$ — Normalweight Concrete Factor (ACI 318)

$\varepsilon_u := 0.003$ — Concrete crushing strain

$c_c := 3in$ — Concrete clear cover for steel reinforcement

Soil Properties

$\gamma_s := 100pcf$ — Unit weight of soil

$q_u := 2000psf$ — Allowable Bearing Pressure

Crane Pad Geometry & Reinforcement

$d_{pad} := 2ft$ Total depth of Crane Pad

$L_{pad} := 50ft$ Total width of Crane Pad

$n_{bar\_t} := 8$ Size of tensile reinforcement

$s_{bar\_t} := 12in$ Spacing of tensile reinforcement

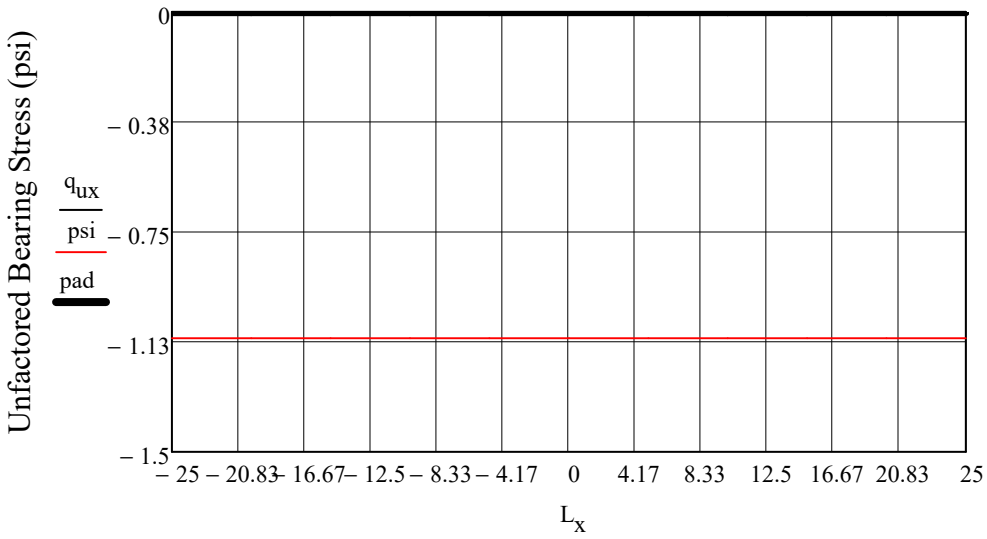
$n_{bar\_c} := 8$ Size of compression reinforcement

$s_{bar\_c} := 12in$ Spacing of compression reinforcement

Design Loading

$W_c := 200ton \cdot g = 400.00 \cdot kip$ Total weight of crane at the center of the foundation

$P_f := 0kip$ Axial force in the Crane Pad



$M_u := -\int_{8.5ft}^{0.5 \cdot L_{pad}} q_{uf}(x) \cdot (x - 8.5ft) \cdot L_{pad}\, dx = 1089.00 \cdot kip \cdot ft$ Total moment demand in the foundation

$M_{u.s} := -\int_{8.5ft}^{0.5 \cdot L_{pad}} q_{uf}(x) \cdot (x - 8.5ft) \cdot 1ft\, dx = 21.78 \cdot kip \cdot ft$ Max moment demand per foot strip in the foundation

$V_{u.s} := -\int_{8.5ft}^{0.5 \cdot L_{pad}} q_{uf}(x) \cdot 1ft\, dx = 2.64 \cdot kip$ Max shear demand per foot strip in the foundation

Foundation Geometry Requirements

Calculate Stresses in Reinforcement

$d_{bar\_t} := rebar_{n_{bar\_t},1} \cdot in = 1.00 \cdot in$ — Diameter of tensile reinforcement

$A_{tw} := rebar_{n_{bar\_t},2} \cdot in^2 \cdot \left(\frac{12in}{s_{bar\_t}}\right) = 0.79 \cdot in^2$ — Cross-Sectional Area of tensile reinforcement

$d_{tw} := d_{pad} - c_c - 0.5 d_{bar\_t} = 20.50 \cdot in$ — Depth of tensile reinforcement

$d_{bar\_c} := rebar_{n_{bar\_c},1} \cdot in = 1.00 \cdot in$ — Diameter of compression reinforcement

$A_{cw} := rebar_{n_{bar\_c},2} \cdot in^2 \cdot \left(\frac{12in}{s_{bar\_c}}\right) = 0.79 \cdot in^2$ — Cross-Sectional Area of compression reinforcement

$d_{cw} := c_c + 0.5 d_{bar\_c} = 3.50 \cdot in$ — Depth of compression reinforcement

$A_{bw} := \begin{pmatrix} A_{tw} \\ A_{cw} \end{pmatrix} = \begin{pmatrix} 0.79 \\ 0.79 \end{pmatrix} \cdot in^2$ — Cross-Section areas of reinforcement

$d_{bw} := \begin{pmatrix} d_{tw} \\ d_{cw} \end{pmatrix} = \begin{pmatrix} 20.50 \\ 3.50 \end{pmatrix} \cdot in$ — Depths of reinforcement measured from top face

$NA = 4.00 \cdot in$ — Depth of Neutral Axis measured from the top face

$\sigma_{srv} := \begin{pmatrix} \sigma_{t.srv} \\ \sigma_{c.srv} \end{pmatrix} = \begin{pmatrix} 16.91 \\ -0.99 \end{pmatrix} \cdot ksi$ — Stress in reinforcement (compression on top, tension on bottom) due to Servicibility Load

Check Geometry Requirements

$\rho_t := \frac{A_{tw}}{d_{pad} \cdot 1ft} = 0.002743$ — Tension Reinforcement Ratio

$\rho_{total} := \frac{A_{tw} + A_{cw}}{d_{pad} \cdot 1ft} = 0.005486$ — Total Reinforcement Ratio

$\rho_b := 0.85 \cdot \beta_1 \cdot \frac{f_c}{F_{y.rb}} \cdot \frac{\varepsilon_u}{\varepsilon_u + \varepsilon_y} = 0.031121$ — Balanced Reinforcement Ratio (Design of Concrete Structures 14th, Nilson 2010, Eq 3.28)

$Check_{TS.min} := \begin{cases} \text{"OK"} & \text{if } \rho_{total} > 0.0018 \\ \text{"NG"} & \text{otherwise} \end{cases} = \text{"OK"}$ — Minimum temperature and shrinkage reinforcement check (ACI 318-19 Section 24.4.3.2)

$Check_{d.min} := \begin{cases} \text{"OK"} & \text{if } d_{pad} \geq \frac{L_{pad}}{27} \\ \text{"NG"} & \text{otherwise} \end{cases} = \text{"OK"}$ — Minimum slab thickness check (ACI 318-19 Table 8.3.1.1)

$$Check_{s.max} := \begin{cases} \text{"OK"} & \text{if } s_{bar\_t} \le \min\left(2 \cdot d_{pad}, 18\text{in}\right) = \text{"OK"} \\ \text{"NG"} & \text{otherwise} \end{cases}$$

Minimum slab thickness check
(ACI 318-19 Table 8.3.1.1)

$$s_{max} := \left| \begin{array}{l} \text{for } i \in 0 .. \text{rows}\left(\sigma_{srv}\right) - 1 \\ \quad s_i \leftarrow \min\left[15 \cdot \left(\frac{40000 \cdot \text{psi} \cdot \text{in}}{\left|\sigma_{srv_i}\right|} \right) - 2.5 \cdot c_c, 12 \cdot \left[\frac{40000 \cdot (\text{psi} \cdot \text{in})}{\left|\sigma_{srv_i}\right|} \right] \right] \\ s \end{array} \right. = \begin{pmatrix} 27.99 \\ 483.92 \end{pmatrix} \cdot \text{in}$$

Max servicibility limit for spacing of reinforcement
(ACI 318-19 Table 24.3.2)

$$Check_{s.t} := \begin{cases} \text{"OK"} & \text{if } s_{bar\_t} \le s_{max_0} = \text{"OK"} \\ \text{"NG"} & \text{otherwise} \end{cases}$$

Check max servicibility limit for spacing of tension reinforcement

$$Check_{s.c} := \begin{cases} \text{"OK"} & \text{if } s_{bar\_c} \le s_{max_1} = \text{"OK"} \\ \text{"NG"} & \text{otherwise} \end{cases}$$

Check max servicibility limit for spacing of compression reinforcement

Foundation Strength Check

Determine Foundation Moment and Shear Capacities

$$
\begin{aligned}
&C_p := \left|\begin{array}{l}
\text{for } T_a \in 1..1000 \\
\quad \left|\begin{array}{l}
a \leftarrow \dfrac{T_a \cdot kip}{0.85 \cdot f_c \cdot b_{st}} \\
\varepsilon_s \leftarrow \left|\begin{array}{l}
\text{for } i \in 0..\text{rows}\left(d_{bw}\right) - 1 \\
\quad \varepsilon_{s_i} \leftarrow \dfrac{0.003}{\frac{a}{\beta_1}} \cdot \left(d_{bw_i} - \dfrac{a}{\beta_1}\right) \\
\varepsilon_s
\end{array}\right. \\
f_s \leftarrow \left|\begin{array}{l}
\text{for } i \in 0..\text{rows}\left(d_{bw}\right) - 1 \\
\quad \left|\begin{array}{l}
f_{s_i} \leftarrow F_{y.rb} \text{ if } \varepsilon_{s_i} \geq \dfrac{F_{y.rb}}{E_s} \\
f_{s_i} \leftarrow -F_{y.rb} \text{ if } \varepsilon_{s_i} \leq \dfrac{-F_{y.rb}}{E_s} \\
f_{s_i} \leftarrow E_s \cdot \varepsilon_{s_i} \text{ if } \varepsilon_{s_i} < \dfrac{F_{y.rb}}{E_s} \wedge \varepsilon_{s_i} \geq 0 \\
f_{s_i} \leftarrow E_s \cdot \varepsilon_{s_i} + 0.85 \cdot f_c \text{ otherwise}
\end{array}\right. \\
f_s
\end{array}\right. \\
P_s \leftarrow \left|\begin{array}{l}
\text{for } i \in 0..\text{rows}\left(A_{bw}\right) - 1 \\
\quad P_{s_i} \leftarrow f_{s_i} \cdot A_{bw_i} \\
P_s
\end{array}\right. \\
\text{if } \dfrac{\sum P_s}{T_a \cdot kip} \geq 0.99 \wedge \dfrac{\sum P_s}{T_a \cdot kip} \leq 1.01 \\
\quad \left|\begin{array}{l}
C_{p_0} \leftarrow T_a \\
C_{p_1} \leftarrow \varepsilon_s \\
C_{p_2} \leftarrow P_s \cdot \dfrac{1}{kip} \\
\text{break}
\end{array}\right.
\end{array}\right. \\
&C_p
\end{aligned}
$$

$$a := \frac{C_{p_0} \cdot kip}{0.85 \cdot f_c \cdot b_{st}} = 1.85 \cdot in$$ Width of Whitney Stress Block

$$\varepsilon_s := C_{p_1} = \begin{pmatrix} 0.02440 \\ 0.00168 \end{pmatrix}$$ Strain in Reinforcement

$$P_s := C_{p_2} \cdot kip = \begin{pmatrix} 47.40 \\ 38.44 \end{pmatrix} \cdot kip$$ Force in Reinforcement

$$Check_{TS} := \begin{cases} \text{"OK"} & \text{if } \max(\varepsilon_s) \geq \max\left(\frac{F_{y.rb}}{E_s}, 0.005\right) \\ \text{"NG"} & \text{otherwise} \end{cases} = \text{"OK"}$$ Check reinforcement strain limit

$$M_s := \left| \begin{array}{l} \text{for } i \in 0..\text{rows}(d_{bw}) - 1 \\ \quad M_{s_i} \leftarrow P_{s_i} \cdot \left(d_{bw_i} - \frac{a}{2}\right) \\ M_s \end{array} \right. = \begin{pmatrix} 77.32 \\ 8.25 \end{pmatrix} \cdot kip \cdot ft$$ Moment in Reinforcement

$$\phi_b := \begin{cases} 0.65 & \text{if } \max(\varepsilon_s) \leq 0.002 \\ 0.90 & \text{if } \max(\varepsilon_s) \geq 0.005 \\ 0.65 + \left(\max(\varepsilon_s) - 0.002\right) \cdot \left(\frac{250}{3}\right) & \text{otherwise} \end{cases} = 0.90$$ Load Reduction Factor

$$\phi M_{n.mat} := 0.9 \cdot \sum M_s = 77.01 \cdot kip \cdot ft$$ Factored Moment Capacity of Crane Pad

$$\phi V_{n.mat} := 0.75 \cdot 2 \cdot \lambda \cdot \sqrt{\frac{f_c}{psi}} \cdot psi \cdot d_{tw} \cdot 1ft = 24.75 \cdot kip$$ Nominal Shear Capacity of Crane Pad
(ACI 318 Section 22.5.5.1)

Check Foundation Strength Design

$$DCR_M := \frac{2.2 M_{u.s}}{\phi M_{n.mat}} = 0.62$$ Moment Capacity DCR

$$Check_M := \begin{cases} \text{"OK"} & \text{if } 1.0 \geq DCR_M \\ \text{"NG"} & \text{otherwise} \end{cases} = \text{"OK"}$$ Check Moment Capacity DCR

$$DCR_V := \frac{2.2 V_{u.s}}{\phi V_{n.mat}} = 0.23$$ Shear Capacity DCR

$$Check_V := \begin{cases} \text{"OK"} & \text{if } 1.0 \geq DCR_V \\ \text{"NG"} & \text{otherwise} \end{cases} = \text{"OK"}$$ Check Shear Capacity DCR

Hanger Design for Conveyance Pipe under Weir Slab

$L_{span} := 13ft + 7in - 2ft = 11.583 \cdot ft$ Maximum span for hanger

$w_{span} := 10ft$ Spacing between pile foundation

$ID := 54in$ Inside diameter of pipe

$OD := 54in + 1in = 55 \cdot in$ Outer diameter

$\gamma_{steel} := 490pcf$ Density of steel

$\gamma_{water} := 62.4pcf$ Density of water

$W_{steel} := \gamma_{steel} \cdot \frac{\pi}{4} \cdot \left(OD^2 - ID^2\right) = 0.291 \cdot klf$ Weight of steel pipe

$W_{water} := \gamma_{water} \cdot \frac{\pi}{4} \cdot \left(ID^2\right) = 0.992 \cdot klf$ Weight of water inside pipe

$P := \left(W_{steel} + W_{water}\right) \cdot w_{span} = 12.837 \cdot kip$ Weight of water and pipe between pile span

$M := \frac{P}{2} \cdot \frac{L_{span}}{2} = 37.175 \cdot kip \cdot ft$ Bending moment on channel

Property of Channel C10x30:

$I_x := 103in^4$ Moment inertia

$I_y := 3.93in^4$

$S_x := 20.7in^3$ Section modulus

$Z_x := 26.7in^3$ Plastic section modulus

$r_y := 0.668in$ Gyration

$C_w := 79.5in^6$

$J := 1.22in^4$

$h_o := 9.56in$

$t_{web} := 0.673in$

$t_{flange} := .436in$

$d_{chanel} := 10in$

$b_{flange} := 3in$

$E := 29000ksi$ Elastic modulus

$F_y := 46ksi$ Yield strength

$L_b := L_{span} - 3.5ft = 8.083 \cdot ft$ Unbraced length

$L_p := 1.76 \cdot r_y \cdot \sqrt{\frac{E}{F_y}} = 2.46 \cdot ft$

$$C_b := \frac{12.5 \cdot 1}{2.5 \cdot 1 + 3 \cdot \frac{1}{4} + 4 \cdot \frac{2}{4} + 3 \cdot \frac{3}{4}} = 1.667$$

$$c := \frac{h_o}{2} \cdot \sqrt{\frac{I_y}{C_w}} = 1.063$$

$$r_{ts} := \left(\frac{\sqrt{I_y \cdot C_w}}{S_x} \right)^{0.5} = 0.924 \cdot in$$

$$L_r := 1.95 \cdot r_{ts} \cdot \frac{E}{0.7 \cdot F_y} \cdot \sqrt{\frac{J \cdot c}{S_x \cdot h_o} + \sqrt{\left(\frac{J \cdot c}{S_x \cdot h_o} \right)^2 + 6.76 \cdot \left(\frac{0.7 \cdot F_y}{E} \right)^2}} = 15.836 \cdot ft$$

$$M_{n1} := F_y \cdot Z_x = 102.35 \cdot kip \cdot ft$$

$$M_{n2} := C_b \cdot \left[F_y \cdot Z_x - \left(F_y \cdot Z_x - 0.7 \cdot S_x \cdot F_y \right) \cdot \left(\frac{L_b - L_p}{L_r - L_p} \right) \right] = 137.788 \cdot kip \cdot ft$$

$$M_n := \min\left(M_{n1}, M_{n2} \right) = 102.35 \cdot kip \cdot ft$$

$$\frac{2.2 \cdot M}{0.9 \cdot M_n} = 0.888$$

Design for Weld

Check flange Minimum weld thickness

Pipe thickness

$t_{BP} := 0.5in$

$t_{ce.f.min} := \min(t_{flange}, t_{BP}) = 0.436 \cdot in$ Minimum thickness of connecting elements

$$t_{w.f.min} := \begin{cases} \frac{1}{8}in & \text{if } t_{ce.f.min} \le \frac{1}{4}in \\ \frac{3}{16}in & \text{if } \frac{1}{4}in < t_{ce.f.min} \le \frac{1}{2}in \\ \frac{1}{4}in & \text{if } \frac{1}{2}in < t_{ce.f.min} \le \frac{3}{4}in \\ \frac{5}{16}in & \text{otherwise} \end{cases} = 0.187 \cdot in$$

Minimum Allowable Weld thickness.
AISC Steel Manual, 15th Edition, Table J2.4

$$t_{w.f.max} := \begin{cases} t_{ce.f.min} & \text{if } t_{ce.f.min} < 0.25in \\ t_{ce.f.min} - \frac{1}{16}in & \text{otherwise} \end{cases} = 0.3735 \cdot in$$

Maximum Allowable Weld thickness.
AISC Steel Manual, 15th Edition, Table J2.2b

$t_{w.f} := \frac{2}{8}in = 0.250 \cdot in$ Proposed weld size

$$Check_{w.f} := \begin{cases} \text{"OK"} & \text{if } t_{w.f.min} \le t_{w.f} \le t_{w.f.max} \\ \text{"NG"} & \text{otherwise} \end{cases} = \text{"OK"}$$

Check proposed weld size

$t_{ce.w.min} := \min(t_{web}, t_{BP}) = 0.500 \cdot in$ Minimum thickness of connecting elements

$$t_{w.w.min} := \begin{cases} \frac{1}{8}in & \text{if } t_{ce.w.min} \le \frac{1}{4}in \\ \frac{3}{16}in & \text{if } \frac{1}{4}in < t_{ce.w.min} \le \frac{1}{2}in \\ \frac{1}{4}in & \text{if } \frac{1}{2}in < t_{ce.w.min} \le \frac{3}{4}in \\ \frac{5}{16}in & \text{otherwise} \end{cases} = 0.187 \cdot in$$

Minimum Allowable Weld thickness.
AISC Steel Manual, 15th Edition, Table J2.4

$$t_{w.w.max} := \begin{cases} t_{ce.w.min} & \text{if } t_{ce.w.min} < 0.25in \\ t_{ce.w.min} - \frac{1}{16}in & \text{otherwise} \end{cases} = 0.4375 \cdot in$$

Maximum Allowable Weld thickness.
AISC Steel Manual, 15th Edition, Table J2.2b

$t_{w.w} := \frac{3}{8}in = 0.375 \cdot in$ Proposed weld size

$Check_{w.w} := \begin{cases} "OK" & \text{if } t_{w.w.min} \le t_{w.w} \le t_{w.w.max} \\ "NG" & \text{otherwise} \end{cases} = "OK"$ Check proposed weld size

Check Base Metal Thickness

$t_{min} := \frac{2 \cdot 0.6 \cdot 70ksi \cdot \frac{\sqrt{2}}{2} \cdot \frac{2}{8}in}{0.6 \cdot 58ksi} = 0.427 \cdot in$ Web is thicker than weld size Eq 9-2 AISC

$\frac{t_{min}}{\min(t_{flange}, t_{web})} = 0.979$ The thickness of channel is more than minimum weld thickness requirement

Check for Strength

Shear

$V := \frac{P \cdot (L_{span})}{L_{span} - 3.5ft} = 18.396 \cdot kip$ Shear at pile connection

$R_{weld} := 2 \cdot 0.6 \cdot 70ksi \cdot \frac{\sqrt{2}}{2} \cdot \frac{3}{8}in \cdot 10in = 222.739 \cdot kip$

$\frac{2.2 \cdot V}{0.75 \cdot R_{weld}} = 0.242$ OK

Moment

$$S_{weld} := \frac{\frac{\sqrt{2}}{2}\cdot\frac{2}{8}in\cdot b_{flange}\cdot\left(0.5d_{chanel} - 0.5\cdot t_{flange}\right)^2\cdot 4}{0.5\cdot d_{chanel}} = 9.702\cdot in^3$$ Section modulus of weld

$$M_{end} := \frac{P\cdot\left(L_{span} - 3.5ft\right)^2\cdot 3.5ft}{L_{span}^{\ 2}} = 21.881\cdot kip\cdot ft$$

$$\sigma_{weld} := \frac{2.2M_{end}}{S_{weld}} = 59.54\cdot ksi$$

$$\frac{\sigma_{weld}}{0.75\cdot 0.6\cdot 70ksi} = 1.89$$ OK

$$\frac{2}{8}\text{in} = 0.25\cdot\text{in}$$

$$109.5\text{ft} - 106\text{ft} = 3.5\cdot\text{ft}$$

$$\frac{(127\text{ft} - 106\text{ft})\cdot 62.4\text{pcf}\cdot(109.5\text{ft} - 106\text{ft})\cdot(7\text{ft})^2}{8}\cdot 2.2 = 61.802\cdot\text{kip}\cdot\text{ft}$$

$$\frac{0.78\text{in}^2\cdot 2}{24\text{in}\cdot 12\text{in}} = 5.417\times 10^{-3}$$

$$\frac{(122.5\text{ft} - 106\text{ft})\cdot 62.4\text{pcf}\cdot(109.5\text{ft} - 106\text{ft})\cdot(7\text{ft})^2}{8}\cdot 2.2 = 48.559\cdot\text{kip}\cdot\text{ft}$$

$$0.9\cdot 0.9\cdot 60\text{ksi}\cdot\frac{3.5\text{ft}}{12\text{in}}\cdot(2\text{ft} - 4\text{in})\cdot 0.78\text{in}^2 = 221.13\cdot\text{kip}\cdot\text{ft}$$

$$\frac{1000\text{lbf}\cdot 2.2}{2} = 1.1\cdot\text{kip}$$

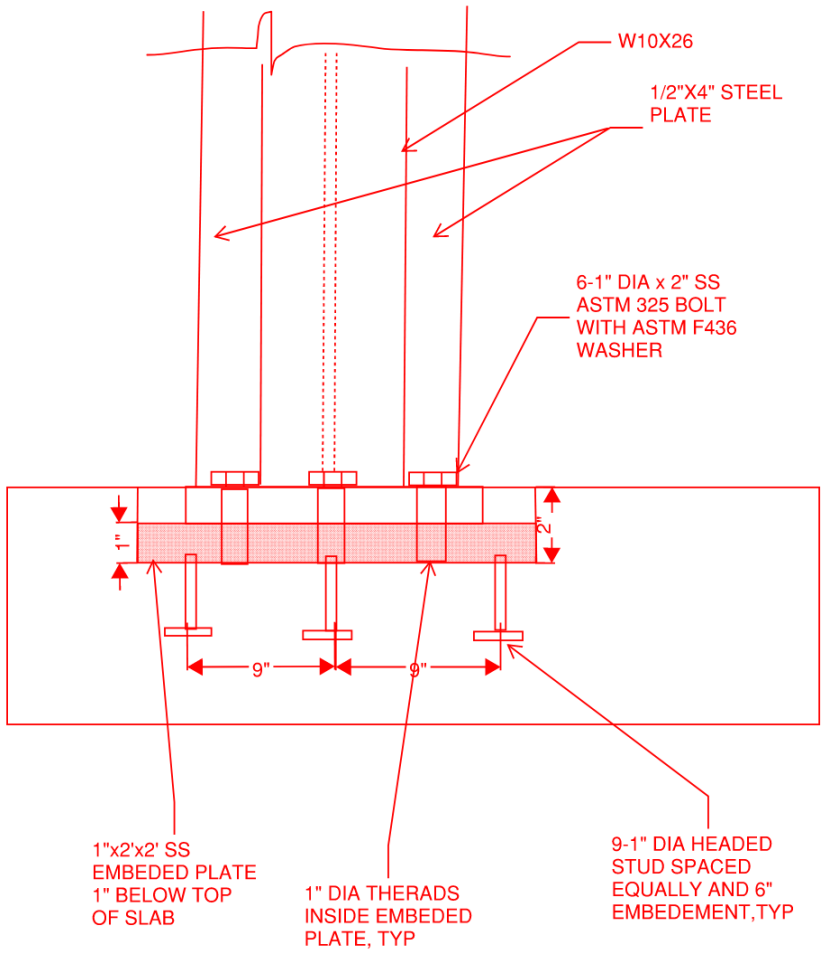
$$0.75\cdot 2\cdot\sqrt{5000}\cdot\text{psi}\cdot(12\text{in} - 4\text{in})\cdot 4\text{ft} = 40.729\cdot\text{kip}$$

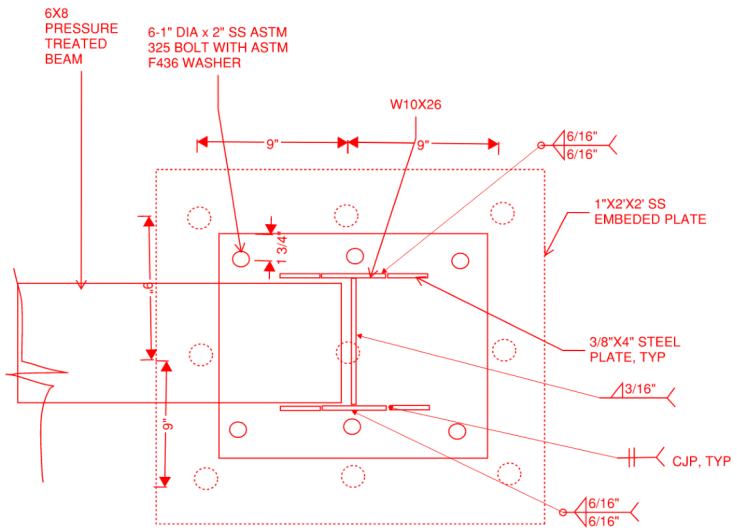
$$\frac{1000\text{lbf}\cdot 2.2\cdot 4\text{ft}}{4} = 2.2\cdot\text{kip}\cdot\text{ft}$$

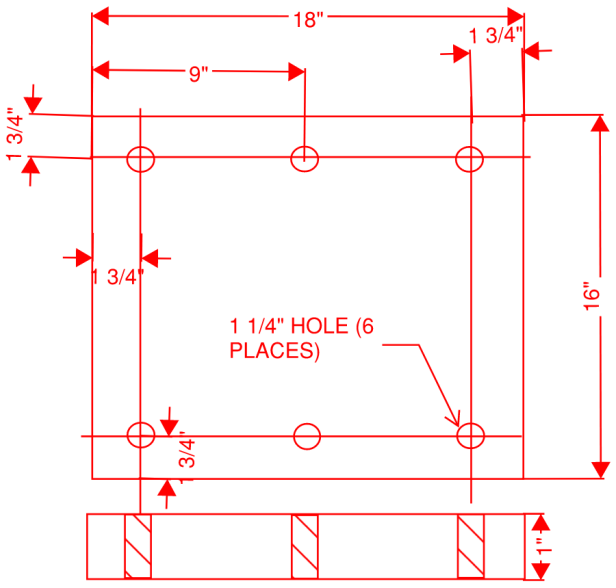
$$0.9\cdot 0.9\cdot 60\text{ksi}\cdot 0.78\text{in}^2\cdot\frac{4\text{ft}}{12\text{in}}\cdot(12\text{in} - 4\text{in}) = 101.088\cdot\text{kip}\cdot\text{ft}$$

$$\frac{0.78\text{in}^2\cdot 2}{2.5\text{ft}\cdot 12\text{in}} = 4.333\times 10^{-3}$$

Client :	Stockton East Water District
Project :	Bellota Weir Modifications Project
Project No :	10209137
Title :	Weir Stop Log Support Design Checks
Purpose:	This calculation package is for the design of the removable bulkheads that will be installed as a part of the Bellota Weir Modifications Project. The calculations check the design of the steel stop log support beams as well as the base plates and anchorage. The anchorage will be checked for pullout, breakout, pryout, and steel yield failure conditions. The calculations were performed according to relevant provisions in ACI 318-19 and AISC 360-16.
Originator :	Ryan Bunyan, EIT Date : January 18, 2022
Checked By :	Iman Ghorbani, PE


W10X26
1/2"X4" STEEL PLATE
6-1" DIA x 2" SS ASTM 325 BOLT WITH ASTM F436 WASHER
1"
2"
9"
9"
1"x2'x2' SS EMBEDED PLATE 1" BELOW TOP OF SLAB
1" DIA THERADS INSIDE EMBEDED PLATE, TYP
9-1" DIA HEADED STUD SPACED EQUALLY AND 6" EMBEDEMENT,TYP


6X8 PRESSURE TREATED BEAM
6-1" DIA x 2" SS ASTM 325 BOLT WITH ASTM F436 WASHER
W10X26
9"
9"
6/16"
6/16"
1"X2'X2' SS EMBEDED PLATE
1 3/4"
3/8"X4" STEEL PLATE, TYP
3/16"
CJP, TYP
6/16"
6/16"
9"


18"
1 3/4"
9"
1 3/4"
1 3/4"
1 1/4" HOLE (6 PLACES)
16"
1 3/4"
1"

Design Inputs

Steel Properties

$E_s := 29000ksi$ Steel modulus of elasticity

$G_s := 11200ksi$ Steel shear modulus of elasticity

$F_{u.A992} := 65ksi$ Ultimate Strength of A992 Steel

$F_{y.A992} := 50 \cdot ksi$ Yield strength of A992 Steel

$F_{u.A36} := 58ksi$ Ultimate Strength of A36 Steel

$F_{y.A36} := 36 \cdot ksi$ Yield strength of A36 Steel

$F_{nt.A325} := 90ksi$ Nominal tensile strength of A325 Group A Bolts

$F_{nv.A325} := 54 \cdot ksi$ Nominal Shear strength of A325 Group A Bolts

$F_{EXX} := 70 \cdot ksi$ Electrode strength of weld metal

Concrete Properties

$\gamma_w := 62.4pcf$ Unit weight of water

$\gamma_c := 150pcf$ Unit weight of concrete

$f_c := 5000psi$ Minimum 28-day Compressive strength of concrete

$$E_c := 57000 \cdot \sqrt{\frac{f_c \cdot in^2}{lbf}} \cdot \frac{lbf}{in^2} = 4030508.65 \cdot psi$$ Concrete Elastic Modulus (ACI 318)

$$\beta_1 := \begin{cases} 0.85 & \text{if } f_c \le 4000psi \\ \max\left[0.85 - 0.05 \cdot \left(\frac{f_c - 4000psi}{1000psi}\right), 0.65\right] & \text{otherwise} \end{cases} = 0.80$$ Compression Block Factor (ACI 318)

$\lambda := 1$ Normalweight Concrete Factor (ACI 318)

$\varepsilon_u := 0.003$ Concrete crushing strain

$c_c := 3in$ Concrete clear cover for steel reinforcement

Timber Properties

$F_{b.u} := 1350psi$ Nominal unfactored allowable bending stress in stop log (NDS-18 Table 4D)

$F_{v.u} := 500psi$ Nominal unfactored allowable shear stress in stop log (NDS-18 Table 4D)

$C_M := 1.0$ Wet Service Factor (NDS-18 Table 4D)

$C_t := 1.0$ Temperature Factor (NDS-18 Table 4D)

$C_L := 1.0$ Beam Stability Factor (NDS-18 Table 4D)

$C_F := 1.0$ Size Factor (NDS-18 Table 4D)

$C_{fu} := 1.0$ Flat Use Factor (NDS-18 Table 4D)

$C_i := 1.0$ Incising Factor (NDS-18 Table 4D)

$C_r := 1.0$ Repetitive Member Factor (NDS-18 Table 4D)

$F_b := C_M \cdot C_t \cdot C_L \cdot C_F \cdot C_{fu} \cdot C_i \cdot C_r \cdot F_{b.u} = 1350.00\ psi$ Unfactored allowable bending stress in stop log (NDS-18 Table 4.3.1)

$F_v := C_M \cdot C_t \cdot C_i \cdot F_{v.u} = 500.00\ psi$ Unfactored allowable shear stress in stop log (NDS-18 Table 4.3.1)

Reduction Factors

Steel Design Load Reduction Factors - AISC 360-16

$\phi_{AISC.b} := 0.9$ AISC Bending LRFD factor (AISC 360-16 F1)

$\phi_{AISC.v\_G1} := 0.9$ AISC Bending LRFD factor (AISC 360-16 G1)

$\phi_{AISC.v\_G2.1} := 1.0$ AISC Bending LRFD factor (AISC 360-16 G2.1)

$\phi_{DG1.b} := 0.9$ AISC Bending LRFD factor (AISC DG 1 Section 3.3)

$\phi_{DG1.v} := 0.9$ AISC Shear LRFD factor (AISC DG 1 Section 3.5)

$\phi_{bolt} := 0.75$ Tensile & Shear reduction factor for bolted connections (AISC 360-16 Sec J3.6)

$\phi_w := 0.75$ Weld reduction factor (AISC 360-16 Sec 8)

Timber Design Load Reduction Factors - AWC NDS-18

$\phi_{NDS.b} := 0.85$ NDS Bending LRFD factor (NDS-18 Table 4.3.1)

$\phi_{NDS.v} := 0.75$ NDS Shear LRFD factor (NDS-18 Table 4.3.1)

Concrete Design Load Reduction Factors - ACI 318-19

$\phi_v := 0.75$ Shear LRFD Factor

$\phi_{BO} := 0.70$ LRFD Factor for Breakout Failure of Cast-in Anchor, Tensile or Shear (ACI 318-19 Table 17.5.3(b)

$\phi_{PT} := 0.70$ LRFD Factor for Pullout Failure of Cast-in Anchor (ACI 318-19 Table 17.5.3(c)

$\phi_{PV} := 0.70$ LRFD Factor for Pryout Failure of Cast-in Anchor (ACI 318-19 Table 17.5.3(c)

Bulkhead Support Beam Design - W10x26 Steel Beam

Bulkhead Support Beam Geometry

$L_b := 7ft$ — Unbraced length of bulkhead support beams

$w_{span} := 8ft + 4in = 8.33\ ft$ — On-center span between bulkhead support beams

$h_s := 10.3in$ — Total depth of beam

$b_f := 5.77in$ — Total width of flanges

$t_w := 0.260in$ — Web thickness

$t_f := 0.440in$ — Flange thickness

$d_w := h_s - 2 \cdot t_f = 9.42 \cdot in$ — Depth of web

$A_s := 2 \cdot t_f \cdot b_f + t_w \cdot d_w = 7.53 \cdot in^2$ — Cross Sectional Area

$I_s := 2 \cdot \left[\frac{b_f \cdot t_f^{\,3}}{12} + b_f \cdot t_f \cdot \left(\frac{h_s}{2} - \frac{t_f}{2} \right)^2 \right] + \frac{t_w \cdot d_w^{\,3}}{12} = 141.60 \cdot in^4$ — Strong Axis Moment of Inertia

$I_w := 2 \cdot \frac{t_f \cdot b_f^{\,3}}{12} + \frac{d_w \cdot t_w^{\,3}}{12} = 14.10 \cdot in^4$ — Weak Axis Moment of Inertia

$S_s := \frac{I_s}{0.5 \cdot h_s} = 27.50 \cdot in^3$ — Elastic section modulus strong axis

$r_w := \sqrt{\frac{I_w}{A_s}} = 1.369 \cdot in$ — Radius of gyration weak axis

$Z_s := 2 \cdot \left[b_f \cdot t_f \cdot \left(\frac{h_s}{2} - \frac{t_f}{2} \right) + \frac{t_w \cdot d_w}{2} \cdot \left(\frac{d_w}{4} \right) \right] = 30.80 \cdot in^3$ — Plastic section modulus strong axis

$J_t := \frac{1}{3} \cdot \left[2 \cdot b_f \cdot t_f^{\,3} + \left(h_s + t_f \right) \cdot t_w^{\,3} \right] = 0.39 \cdot in^4$ — Torsional constant

Design Loading

$V_{SB} := \frac{1}{2} \cdot \gamma_w \cdot L_b^{\,2} \cdot w_{span} = 12.74 \cdot kip$ — Unfactored shear on support beams

$M_{SB} := V_{SB} \cdot \frac{1}{3} \cdot L_b = 29.73 \cdot kip \cdot ft$ — Unfactored moment on support beams

Compactness Limits - AISC B4

$Comp_w := \frac{d_w}{t_w} = 36.23$ — Web slenderness

$$Comp_f := \frac{0.5 \cdot b_f}{t_f} = 6.56$$ Flange Slenderness

$$\lambda_{r.c.w} := 1.49 \cdot \sqrt{\frac{E_s}{F_{y.A992}}} = 35.88$$ Web compression noncompact limit Case 1

$$WC := \begin{cases} \text{"Slender"} & \text{if } Comp_w \geq \lambda_{r.c.w} \\ \text{"Not Slender"} & \text{otherwise} \end{cases} = \text{"Slender"}$$ Beam web is slender

$$\lambda_{r.c.f} := 0.56 \cdot \sqrt{\frac{E_s}{F_{y.A992}}} = 13.49$$ Flange compression noncompact limit Case 5

$$FC := \begin{cases} \text{"Slender"} & \text{if } Comp_f \geq \lambda_{r.c.f} \\ \text{"Not Slender"} & \text{otherwise} \end{cases} = \text{"Not Slender"}$$ Beam flange is not slender

$$\lambda_{p.f.w} := 3.76 \cdot \sqrt{\frac{E_s}{F_{y.A992}}} = 90.55$$ Web flexure compact limit Case 15

$$\lambda_{r.f.w} := 5.70 \cdot \sqrt{\frac{E_s}{F_{y.A992}}} = 137.27$$ Web flexure noncompact limit Case 15

$$WF := \begin{cases} \text{"Compact"} & \text{if } Comp_w < \lambda_{p.f.w} \\ \text{"Slender"} & \text{if } Comp_w \geq \lambda_{r.f.w} \\ \text{"Noncompact"} & \text{otherwise} \end{cases} = \text{"Compact"}$$ Beam web is compact

$$\lambda_{p.f.f} := 0.38 \cdot \sqrt{\frac{E_s}{F_{y.A992}}} = 9.15$$ Flange flexure compact limit Case 10

$$\lambda_{r.f.f} := 1.0 \cdot \sqrt{\frac{E_s}{F_{y.A992}}} = 24.08$$ Flange flexure noncompact limit Case 10

$$FF := \begin{cases} \text{"Compact"} & \text{if } Comp_f < \lambda_{p.f.f} \\ \text{"Slender"} & \text{if } Comp_f \geq \lambda_{r.f.f} \\ \text{"Noncompact"} & \text{otherwise} \end{cases} = \text{"Compact"}$$ Beam flange is compact

Strong Axis Bending Check - AISC F2

Yielding

$M_{n.y.s} := F_{y.A992} \cdot Z_s = 128.34 \cdot ft \cdot kip$ Nominal strong axis moment yield strength

Lateral-Torsional Buckling

$h_o := h_s - t_f = 9.86 \cdot in$ Distance between flange centroids

$r_{ts} := \sqrt{\frac{I_w \cdot h_o}{2 \cdot S_s}} = 1.59 \cdot in$ Approx. radius of gyration of comp. flange + 1/6 of the web

$L_p := 1.76 \cdot r_w \cdot \sqrt{\frac{E_s}{F_{y.A992}}} = 4.83\ ft$ Limiting plastic length

$c_{cs} := 1.0$ Shape modification factor

$L_r := 1.95 \cdot r_{ts} \cdot \frac{E_s}{0.7 \cdot F_{y.A992}} \cdot \sqrt{\frac{J_t \cdot c_{cs}}{S_s \cdot h_o}} \cdot \sqrt{1 + \sqrt{1 + 6.76 \cdot \left(\frac{0.7 \cdot F_{y.A992}}{E_s} \cdot \frac{S_s \cdot h_o}{J_t \cdot c_{cs}} \right)^2}} = 14.98\ ft$ Limiting inelastic buckling length

$C_b := 1.0$ LTB modification factor (1.0 because beam is cantilevered)

$F_{cr.b} := \frac{C_b \cdot \pi^2 \cdot E_s}{\left(\frac{L_b}{r_{ts}} \right)^2} \cdot \sqrt{1 + 0.078 \cdot \frac{J_t \cdot c_{cs}}{S_s \cdot h_o} \cdot \left(\frac{L_b}{r_{ts}} \right)^2} = 117.55 \cdot ksi$ Elastic LTB buclking stress

$$M_{n.ltb} := \begin{cases} M_{n.y.s} & \text{if } L_b \le L_p \\ \min\left(F_{cr.b} \cdot S_s, M_{n.y.s}\right) & \text{if } L_b > L_r \\ \min\left[C_b \cdot \left[M_{n.y.s} - \left(M_{n.y.s} - 0.7 \cdot F_{y.A992} \cdot S_s\right) \cdot \left(\frac{L_b - L_p}{L_r - L_p}\right)\right], M_{n.y.s}\right] & \text{otherwise} \end{cases} = 118.06 \cdot ft \cdot kip$$

Nominal LTB strength

Governing Strong Axis Flexural Strength

$\phi M_{n.s} := \phi_{AISC.b} \cdot \min\left(M_{n.y.s}, M_{n.ltb}\right) = 106.25 \cdot ft \cdot kip$ Design strong axis moment capacity

$DCR_{M.SB} := \frac{2.2 M_{SB}}{\phi M_{n.s}} = 0.62$ Moment Capacity DCR

$$Check_{M.SB} := \begin{cases} \text{"OK"} & \text{if } \phi M_{n.s} \ge 2.2 M_{SB} \\ \text{"NG"} & \text{otherwise} \end{cases} = \text{"OK"}$$

Check Moment Capacity DCR

Strong Axis Shear Check - AISC G2

$A_{w.v} := h_s \cdot t_w = 2.68 \cdot in^2$ Shear area

$$k_v := \begin{cases} 5.34 & \text{if } \frac{L_b}{d_w} > 3.0 \\ 5 + \frac{5}{\left(\frac{L_b}{d_w}\right)^2} & \text{otherwise} \end{cases} = 5.34$$ Web plate buckling coefficient

$$C_{v1} := \begin{cases} 1 & \text{if } \frac{d_w}{t_w} \le 2.24 \cdot \sqrt{\frac{E_s}{F_{y.A992}}} \\ 1.10 \cdot \frac{\sqrt{\frac{k_v \cdot E_s}{F_{y.A992}}}}{\frac{d_w}{t_w}} & \text{if } \frac{d_w}{t_w} > 1.10 \cdot \sqrt{\frac{k_v \cdot E_s}{F_{y.A992}}} \\ 1 & \text{otherwise} \end{cases} = 1.00$$ Web shear coefficient

$\phi V_{n.s} := \phi_{AISC.v\_G2.1} \cdot 0.6 \cdot F_{y.A992} \cdot A_{w.v} \cdot C_{v1} = 80.34 \cdot kip$ Design shear capacity

$DCR_{V.SB} := \frac{2.2 V_{SB}}{\phi V_{n.s}} = 0.35$ Shear Capacity DCR

$$Check_{V.SB} := \begin{cases} \text{"OK"} & \text{if } \phi V_{n.s} \ge 2.2 V_{SB} \\ \text{"NG"} & \text{otherwise} \end{cases} = \text{"OK"}$$ Check Shear Capacity DCR

Stop Log Design - 6"x8" Timber Beam

Stop Log Geometry

$h_{SL} := 7.5in$ — Actual depth of timber stop log (8" nominal)

$w_{SL} := 5.5in$ — Actuall width of timber stop log (6" nominal)

$A_{SL} := h_{SL} \cdot w_{SL} = 41.25 \cdot in^2$ — Cross-Sectional Area of stop log

$I_{y.SL} := \frac{w_{SL} \cdot h_{SL}^3}{12} = 193.36 \cdot in^4$ — Strong-Axis Moment of Inertia of stop log

Strength Design Check

$w_{w.u} := \gamma_w \cdot L_b \cdot w_{SL} = 0.200 \cdot \frac{kip}{ft}$ — Unfactored distributed load on stop logs

$V_{SL} := \frac{w_{w.u} \cdot w_{span}}{2} = 0.83 \cdot kip$ — Unfactored shear on stop logs

$M_{SL} := \frac{w_{w.u} \cdot w_{span}^2}{8} = 1.74 \cdot kip \cdot ft$ — Unfactored moment on stop logs

$\sigma_b := \frac{M_{SL} \cdot (0.5 \cdot h_{SL})}{I_{y.SL}} = 404.44 \cdot psi$ — Unfactored bending stress in stop logs

$\sigma_v := \frac{V_{SL}}{A_{SL}} = 20.22 \cdot psi$ — Unfactored shear stress in stop logs

$DCR_{M.SL} := \frac{2.2 \cdot \sigma_b}{\phi_{NDS.b} \cdot F_b} = 0.78$ — Moment Capacity DCR

$Check_{M.SL} := \begin{cases} \text{"OK"} & \text{if } \phi_{NDS.b} \cdot F_b \geq \sigma_b \\ \text{"NG"} & \text{otherwise} \end{cases} = \text{"OK"}$ — Check Moment Capacity DCR

$DCR_{V.SL} := \frac{2.2\sigma_v}{\phi_{NDS.v} \cdot F_v} = 0.12$ — Shear Capacity DCR

$Check_{V.SL} := \begin{cases} \text{"OK"} & \text{if } \phi_{NDS.v} \cdot F_v \geq \sigma_v \\ \text{"NG"} & \text{otherwise} \end{cases} = \text{"OK"}$ — Check Shear Capacity DCR

Base Plate Design - 18" x 16" Steel Plate

Base Plate Geometry

$t_{BP} := 1.0in$ — Proposed thickness of base plate

$w_s := 18in$ — Proposed width of base plate parallel to strong axis of SB

$w_w := 16in$ — Proposed width of base plate parallel to weak axis of SB

$m_{BP} := 0.5 \cdot (w_s - h_s) = 3.85 \cdot in$ — Proposed width of base plate above flange of SB

$n_{BP} := 0.5 \cdot (w_w - b_f) = 5.12 \cdot in$ — Proposed width of base plate adjacent to flange of SB

$s_{e.bolt} := 1.75in$ — Proposed edge spacing of anchor bolts

$A_1 := w_s \cdot w_w = 288.00 \cdot in^2$ — Area of base plate in plan

$A_2 := 8ft \cdot 4ft = 4608.00 \cdot in^2$ — Maximum tributary area of supporting concrete surface

$A_b := \min\left(2, \sqrt{\frac{A_1}{A_2}}\right) = 0.25$ — Area term for maximum bearing stress calculation

Design Loading

$P_{BP} := 26 \cdot plf \cdot L_b = 0.18 \cdot kip$ — Unfactored axial load on base plate from weight of steel wide flange bulkhead support beam

$V_{BP} := V_{SB} = 12.74 \cdot kip$ — Unfactored shear load on base plate

$M_{BP} := M_{SB} = 29.73 \cdot kip \cdot ft$ — Unfactored moment on base plate

$f_{p.max} := 0.65 \cdot (0.85 \cdot f_c) \cdot A_b = 690.63 \cdot psi$ — Maximum bearing stress under the base plate

$q_{max} := f_{p.max} \cdot w_s = 12.43 \cdot \frac{kip}{in}$ — Maximum bearing stress per unit length of base plate

$e_{crit} := \frac{w_s}{2} - \frac{P_{BP}}{2 \cdot q_{max}} = 8.99 \cdot in$ — Critical eccentricity of demad loading on base plate

$e_{BP} := \frac{M_{BP}}{P_{BP}} = 163.33 \cdot ft$ — Eccentricity of demad loading on base plate

$Check_e :=$ "Small Moment - DG1 Section 3.3" if $e_{BP} < e_{crit}$ = "Large Moment - DG1 Section 3.4"
"Large Moment - DG1 Section 3.4" otherwise

Flexural Design - AISC Design Guide 1 Section 3.4

$f_{BP} := \frac{w_s}{2} - s_{e.bolt} = 7.25 \cdot in$ Dimension from CL of tension anchors to center of BP

$Check_Y := \begin{cases} \text{"OK"} & \text{if } \left(f_{BP} + \frac{w_s}{2}\right)^2 \geq \frac{2 \cdot P_{BP} \cdot (e_{BP} + f_{BP})}{q_{max}} \\ \text{"Increase BP"} & \text{otherwise} \end{cases} = \text{"OK"}$ Check if inequality is satisfied (Eq 3.4.4)

$Y_{BP} := \begin{cases} Y_{BP_0} \leftarrow \left(f_{BP} + \frac{w_s}{2}\right) + \sqrt{\left(f_{BP} + \frac{w_s}{2}\right)^2 - \frac{2 \cdot P_{BP} \cdot (e_{BP} + f_{BP})}{q_{max}}} \\ Y_{BP_1} \leftarrow \left(f_{BP} + \frac{w_s}{2}\right) - \sqrt{\left(f_{BP} + \frac{w_s}{2}\right)^2 - \frac{2 \cdot P_{BP} \cdot (e_{BP} + f_{BP})}{q_{max}}} \end{cases} = \begin{pmatrix} 30.62 \\ 1.88 \end{pmatrix} \cdot in$ Solutions for bearing width under base plate (Eq 3.4.3)

$Y_{BP} := \min(Y_{BP}) = 1.88 \cdot in$ Bearing width under base plate (Eq 3.4.3)

$X_{BP} := 0.5(w_s - h_s) - s_{e.bolt} = 2.10 \cdot in$ Distance between CL of anchor and CL of flange of SB

$T_{u.BP} := q_{max} \cdot Y_{BP} - P_{BP} = 23.20 \cdot kip$ Tension force in anchor rod

$t_{BP.req.m} := \begin{cases} 1.5 \cdot m_{BP} \cdot \sqrt{\frac{f_{p.max}}{F_{y.A36}}} & \text{if } Y_{BP} \geq m_{BP} \\ 2.11 \cdot \sqrt{\frac{f_{p.max} \cdot Y_{BP} \cdot (m_{BP} - 0.5 \cdot Y_{BP})}{F_{y.A36}}} & \text{otherwise} \end{cases} = 0.68 \cdot in$ Required thickness of base plate (AISC Design Guide 1 Eq. 3.3.14a and Eq. 3.3.15a)

$t_{BP.req.x} := 2.11 \cdot \sqrt{\frac{T_{u.BP} \cdot X_{BP}}{w_s \cdot F_{y.A36}}} = 0.58 \cdot in$ Required thickness of base plate (AISC Design Guide 1 Eq. 3.4.7a)

$Check_{t.BP} := \begin{cases} \text{"OK"} & \text{if } t_{BP} \geq \max(t_{BP.req.m}, t_{BP.req.x}) \\ \text{"NG"} & \text{otherwise} \end{cases} = \text{"OK"}$ Check thickness requirement

$M_{n.BP} := \frac{T_{u.BP} \cdot X_{BP}}{w_s} = 2.71 \cdot \frac{kip \cdot ft}{ft}$ Required Flexural Strength of Base Plate per unit width (AISC Design Guide 1 Eq. 3.4.5a)

$\phi R_{n.BP} := \phi_{DG1.b} \cdot F_{y.A36} \cdot \frac{t_{BP}^2}{4} = 8.10 \cdot \frac{kip \cdot ft}{ft}$ Available factored flexural capacity of base plate per unit width (AISC Design Guide 1 Eq. 3.3.13a)

$DCR_{M.BP} := \frac{2.2 M_{n.BP}}{\phi R_{n.BP}} = 0.74$ Moment Capacity DCR

$Check_{M.BP} := \begin{cases} \text{"OK"} & \text{if } \phi R_{n.BP} \geq 2.2 M_{n.BP} \\ \text{"NG"} & \text{otherwise} \end{cases} = \text{"OK"}$ Check Moment Capacity DCR

Support Beam-to-Base Plate Connection Design - PJP Fillet Weld

Determine Flange Weld Size

$t_{ce.f.min} := \min(t_f, t_{BP}) = 0.440 \cdot in$ — Minimum thickness of connecting elements

$$t_{w.f.min} := \begin{cases} \frac{1}{8}in & \text{if } t_{ce.f.min} \le \frac{1}{4}in \\ \frac{3}{16}in & \text{if } \frac{1}{4}in < t_{ce.f.min} \le \frac{1}{2}in \\ \frac{1}{4}in & \text{if } \frac{1}{2}in < t_{ce.f.min} \le \frac{3}{4}in \\ \frac{5}{16}in & \text{otherwise} \end{cases} = 0.187 \cdot in$$

Minimum Allowable Weld thickness.
AISC Steel Manual, 15th Edition, Table J2.4

$$t_{w.f.max} := \begin{cases} t_{ce.f.min} & \text{if } t_{ce.f.min} < 0.25in \\ t_{ce.f.min} - \frac{1}{16}in & \text{otherwise} \end{cases} = 0.3775 \cdot in$$

Maximum Allowable Weld thickness.
AISC Steel Manual, 15th Edition, Table J2.2b

$t_{w.f} := \frac{6}{16}in = 0.375 \cdot in$ — Proposed weld size

$$Check_{w.f} := \begin{cases} \text{"OK"} & \text{if } t_{w.f.min} \le t_{w.f} \le t_{w.f.max} \\ \text{"NG"} & \text{otherwise} \end{cases} = \text{"OK"}$$

Check proposed weld size

Determine Web Weld Size

$t_{ce.w.min} := \min(t_w, t_{BP}) = 0.260 \cdot in$ — Minimum thickness of connecting elements

$$t_{w.w.min} := \begin{cases} \frac{1}{8}in & \text{if } t_{ce.w.min} \le \frac{1}{4}in \\ \frac{3}{16}in & \text{if } \frac{1}{4}in < t_{ce.w.min} \le \frac{1}{2}in \\ \frac{1}{4}in & \text{if } \frac{1}{2}in < t_{ce.w.min} \le \frac{3}{4}in \\ \frac{5}{16}in & \text{otherwise} \end{cases} = 0.187 \cdot in$$

Minimum Allowable Weld thickness.
AISC Steel Manual, 15th Edition, Table J2.4

$$t_{w.w.max} := \begin{cases} t_{ce.w.min} & \text{if } t_{ce.w.min} < 0.25in \\ t_{ce.w.min} - \frac{1}{16}in & \text{otherwise} \end{cases} = 0.1975 \cdot in$$

Maximum Allowable Weld thickness.
AISC Steel Manual, 15th Edition, Table J2.2b

$t_{w.w} := \frac{3}{16}in = 0.187 \cdot in$ — Proposed weld size

$$Check_{w.w} := \begin{cases} \text{"OK"} & \text{if } t_{w.w.min} \le t_{w.w} \le t_{w.w.max} \\ \text{"NG"} & \text{otherwise} \end{cases} = \text{"OK"}$$

Check proposed weld size

Base Metal Checks

$$t_{BP.f.min} := \frac{0.6 \cdot F_{EXX} \cdot \frac{\sqrt{2}}{2} \cdot t_{w.f}}{0.6 F_{u.A36}} = 0.32 \cdot in$$

Minimum required thickness of base metal to resist shear rupture. AISC Steel Manual, 14th Edition, Eq 9-2.

$$Check_{BP.min} := \begin{cases} \text{"OK"} & \text{if } t_{BP.f.min} \le t_{BP} \\ \text{"NG"} & \text{otherwise} \end{cases} = \text{"OK"}$$

Check minimum base metal thickness

Since the base plate is thicker than the minimum required for the flange weld group, it does not govern and does not need to be checked.

$$t_{f.min} := 2\left(\frac{0.6 \cdot F_{EXX} \cdot \frac{\sqrt{2}}{2} \cdot t_{w.f}}{0.6 F_{u.A36}}\right) = 0.64 \cdot in$$

Minimum required thickness of base metal to resist shear rupture. AISC Steel Manual, 14th Edition, Eq 9-2.

$$Check_{f.min} := \begin{cases} \text{"OK"} & \text{if } t_{f.min} \le t_{f} \\ \text{"NG"} & \text{otherwise} \end{cases} = \text{"NG"}$$

Check minimum base metal thickness

The support beam flange is thinner than the result of this equation, however, since this element is subject to tension force and is not in plane with the base plate there is no limit for weld size and a base metal check is not required.

$$t_{BP.w.min} := \frac{0.6 \cdot F_{EXX} \cdot \frac{\sqrt{2}}{2} \cdot t_{w.w}}{0.6 F_{u.A36}} = 0.16 \cdot in$$

Minimum required thickness of base metal to resist shear rupture. AISC Steel Manual, 14th Edition, Eq 9-2.

$$Check_{BP.f.min} := \begin{cases} \text{"OK"} & \text{if } t_{BP.w.min} \le t_{BP} \\ \text{"NG"} & \text{otherwise} \end{cases} = \text{"OK"}$$

Check minimum base metal thickness

Since the base plate is thicker than the minimum required for the web weld group, it does not govern and does not need to be checked.

$$t_{w.min} := \frac{0.6 \cdot F_{EXX} \cdot \frac{\sqrt{2}}{2} \cdot t_{w.w}}{0.6 F_{u.A36}} = 0.16 \cdot in$$

Minimum required thickness of base metal to resist shear rupture. AISC Steel Manual, 14th Edition, Eq 9-2.

$$Check_{w.min} := \begin{cases} \text{"OK"} & \text{if } t_{w.min} \le t_{w} \\ \text{"Check"} & \text{otherwise} \end{cases} = \text{"OK"}$$

Check minimum base metal thickness

Since the support beam web is thicker than the minimum required for the web weld group, it does not govern and does not need to be checked.

Weld Group Geometry

$L_{w.f} := 2 \cdot b_f - t_w = 11.28 \cdot in$ — Length of flange weld (one flange only)

$L_{w.w} := h_s - 2 \cdot t_f = 9.42 \cdot in$ — Length of web welds

$L_w := 2 \cdot L_{w.f} + L_{w.w} = 31.98 \cdot in$ — Total length of weld group

$A_{w.f} := 0.707 \cdot t_{w.f} \cdot L_{w.f} = 2.99 \cdot in^2$ — Total area of flange weld (one flange only)

$A_{w.w} := 0.707 \cdot t_{w.w} \cdot L_{w.w} = 1.25 \cdot in^2$ — Total area of web welds

$A_w := 2 \cdot A_{w.f} + A_{w.w} = 7.23 \cdot in^2$ — Total area of weld group

$S_{w.f} := \frac{A_{w.f} \cdot \left(0.5 h_s - 0.5 t_f\right)^2 2}{0.5 \cdot h_s} = 28.23 \cdot in^3$ — Effective Section Modulus of flange weld group (both flanges)

Weld Capacity Check

$\sigma_P := \frac{P_{BP}}{A_w} = 0.03 \cdot ksi$ — Axial stress on connection

$\sigma_V := \frac{V_{BP}}{A_{w.w}} = 10.20 \cdot ksi$ — Strong-Axis Shear stress on connection (Assumed to be taken by the web)

$\sigma_M := \frac{M_{BP}}{S_{w.f}} = 12.64 \cdot ksi$ — Strong-Axis Bending stress on connection (Assumed to be taken by the flanges)

$\sigma_{w.f} := \sigma_P + \sigma_M = 12.662 \cdot ksi$ — Combined stress on flange weld group

$\sigma_{w.w} := \sqrt{\sigma_P^2 + \sigma_V^2} = 10.202 \cdot ksi$ — Combined stress on web weld group

$\phi R_{nw} := \phi_w \cdot 0.6 \cdot F_{EXX} = 31.50 \cdot ksi$ — Available Strength of the weld metal. AISC Steel Manual, 14th Edition, J2-5. 0.75 from Table 2-5.

$DCR_{w.f} := \frac{2.2 \sigma_{w.f}}{\phi R_{nw}} = 0.88$ — Weld Connection DCR

$Check_{w.f.c} := \begin{cases} \text{"OK"} & \text{if } \phi R_{nw} \geq 2.2 \sigma_{w.f} \\ \text{"NG"} & \text{otherwise} \end{cases} = \text{"OK"}$ — Check Weld Connection Capacity

$DCR_{w.w} := \frac{2.2 \sigma_{w.w}}{\phi R_{nw}} = 0.71$ — Weld Connection DCR

$Check_{w.w.c} := \begin{cases} \text{"OK"} & \text{if } \phi R_{nw} \geq 2.2 \sigma_{w.w} \\ \text{"NG"} & \text{otherwise} \end{cases} = \text{"OK"}$ — Check Weld Connection Capacity

Base Plate-to-Embedded Plate Connection Design - 1" DIA A325 Bolts

Bolted Connection Inputs

$t_{EP} := 1in$ — Proposed thickness of embedded plate

$n_{bolt.row} := 2$ — Proposed number of rows of anchor bolts

$n_{bolt.per} := 3$ — Proposed number of anchor bolts per row

$n_{bolt} := n_{bolt.row} \cdot n_{bolt.per} = 6$ — Proposed number of anchor bolts

$d_{bolt} := 1in$ — Proposed diameter of anchor bolts

$L_{bolt} := 2in$ — Proposed length of anchor bolts

$d_{h.bolt} := 1.25in$ — Proposed diameter of anchor bolt holes

$s_{bolt} := 9in - 1.75in = 7.25 \cdot in$ — Proposed spacing of anchor bolts

$s_{e.bolt} = 1.75 \cdot in$ — Proposed edge spacing of anchor bolts

$A_{bolt} := \frac{\pi}{4} \cdot d_{bolt}^{2} = 0.79 \cdot in^{2}$ — Cross-Sectional Area of anchor bolts

Bolted Connection Geometry Requirements

$t_{min} := \min(t_{BP}, t_{EP}) = 1.0 \cdot in$ — Minimum thickness of connecting elements

$$s_{e.min.std} := \begin{cases} 0.75in & \text{if } d_{bolt} \le 0.5in \\ 0.875in & \text{if } 0.5 \cdot in < d_{bolt} \le 0.625in \\ 1in & \text{if } 0.625 \cdot in < d_{bolt} \le 0.75in \\ 1.125in & \text{if } 0.75 \cdot in < d_{bolt} \le 0.875in \\ 1.25in & \text{if } 0.875 \cdot in < d_{bolt} \le 1in \\ 1.5in & \text{if } 1 \cdot in < d_{bolt} \le 1.125in \\ 1.625in & \text{if } 1.125 \cdot in < d_{bolt} \le 1.25in \\ 1.25 \cdot d_{bolt} & \text{if } 1.25 \cdot in < d_{bolt} \end{cases} = 1.250 \cdot in$$

Minimum edge distance for standard hole (AISC 360-16 Table J3.4) (Center of bolt to edge)

$$C_2 := \begin{cases} 0.0625in & \text{if } d_{bolt} \le 0.875in \\ 0.125in & \text{otherwise} \end{cases} = 0.125 \cdot in$$

Additional edge distance for oversized hole (AISC 360-16 Table J3.5)

$s_{e.min} := s_{e.min.std} + C_2 = 1.375 \cdot in$ — Minimum edge distance

$s_{e.max} := \min(6in, 12 \cdot t_{min}) = 6.000 \cdot in$ — Maximum edge distance (AISC 360-16 Sec J3.5)

$$s_{e.Check} := \begin{cases} \text{"OK"} & \text{if } s_{e.min} \le s_{e.bolt} \le s_{e.max} \\ \text{"NG"} & \text{otherwise} \end{cases} = \text{"OK"}$$

Bolt edge distance check

$s_{b.min} := 2.67 \cdot d_{bolt} = 2.670 \cdot in$ Minimum bolt spacing (AISC 360-16 Sec J3.5)

$s_{b.max} := \min(12in, 24 \cdot t_{BP}) = 12.0 \cdot in$ Maximum bolt spacing (AISC 360-16 Sec J3.5)

$s_{b.Check} := \begin{cases} \text{"OK"} & \text{if } s_{b.min} \le s_{bolt} \le s_{b.max} \\ \text{"NG"} & \text{otherwise} \end{cases} = \text{"OK"}$ Bolt spacing check

Tensile Yield Strength - AISC 360-16 Section J3.6

$\phi R_{nt.b} := \phi_{bolt} \cdot F_{nt.A325} \cdot A_{bolt} = 53.014 \cdot kip$ Tensile strength per bolt (AISC 360-16 Eq J3-1)

$DCR_{bt} := \frac{2.2 T_{u.BP}}{n_{bolt.per} \cdot \phi R_{nt.b}} = 0.321$ Shear capacity DCR of bolted connection

$Check_{bt} := \begin{cases} \text{"OK"} & \text{if } 1.0 \ge DCR_{bt} \\ \text{"NG"} & \text{otherwise} \end{cases} = \text{"OK"}$ Check Bolt Connection Capacity

Shear Yield Strength - AISC 360-16 Section J3.6

$\phi R_{nv.bn} := \phi_{bolt} \cdot F_{nv.A325} \cdot A_{bolt} = 31.809 \cdot kip$ Shear strength per bolt (AISC 360-16 Eq J3-1)

$\phi R_{nb.b} := \phi_{bolt} \cdot 2.4 \cdot d_{bolt} \cdot t_{min} \cdot F_{u.A36} = 104.400 \cdot kip$ Bearing strength per bolt per inch of thickness (AISC 360-16 Eq J3-6a)

$\phi R_{nts.b} := \phi_{bolt} \cdot 1.2 \cdot (s_{b.min} - d_{h.bolt}) \cdot t_{min} \cdot F_{u.A36} = 74.1 \cdot kip$ Tearout strength per bolt per inch of thickness based on bolt spacing (AISC 360-16 Eq J3-6c)

$\phi R_{nte.b} := \phi_{bolt} \cdot 1.2 \cdot (s_{e.bolt} - 0.5 \cdot d_{h.bolt}) \cdot t_{min} \cdot F_{u.A36} = 58.7 \cdot kip$ Tearout strength per bolt per inch of thickness based on edge distance (AISC 360-16 Eq J3-6c)

$\phi R_{nv.b} := \min(\phi R_{nv.bn}, \phi R_{nb.b}, \phi R_{nts.b}, \phi R_{nte.b}) = 31.8 \cdot kip$ Governing shear capacity of each bolt

$DCR_{bv} := \frac{2.2 V_{BP}}{n_{bolt} \cdot \phi R_{nv.b}} = 0.147$ Shear capacity DCR of bolted connection

$Check_{bv} := \begin{cases} \text{"OK"} & \text{if } 1.0 \ge DCR_{bv} \\ \text{"NG"} & \text{otherwise} \end{cases} = \text{"OK"}$ Check Bolt Connection Capacity

Combined Tensile and Shear Yield Strength - AISC 360-16 Section J3.7

$$f_{rv} := \frac{V_{BP}}{n_{bolt} \cdot A_{bolt}} = 2.704 \cdot ksi$$

Requied shear strength per bolt

$$f_{nt'} := 1.3 \cdot F_{nt.A325} - \frac{F_{nt.A325}}{\phi_{bolt} \cdot F_{nv.A325}} \cdot f_{rv} = 110.992 \cdot ksi$$

Combined tensile and shear strength per bolt (AISC 360-16 Eq J3-3a)

$$\phi R_{nc.b} := \phi_{bolt} \cdot f_{nt'} \cdot A_{bolt} = 65.4 \cdot kip$$

Combined capacity of each bolt

$$DCR_{bc} := \frac{2.2 \sqrt{\left(\frac{T_{u.BP}}{n_{bolt.per}}\right)^2 + \left(\frac{V_{BP}}{n_{bolt}}\right)^2}}{\phi R_{nc.b}} = 0.270$$

Combined tensile and shear capacity DCR of bolted connection

$$Check_{bc} := \begin{cases} \text{"OK"} & \text{if } 1.0 \geq DCR_{bc} \\ \text{"NG"} & \text{otherwise} \end{cases} = \text{"OK"}$$

Check Bolt Connection Capacity

Embedded Plate Anchorage Design Check - 1" DIA Headed Studs

Embedded Plate Geometry

$w_{s.EP} := 24in$ — Proposed width of base plate parallel to strong axis of SB

$w_{w.EP} := 24in$ — Proposed width of base plate parallel to weak axis of SB

$n_{anc.row} := 3$ — Proposed number of rows of headed studs

$n_{anc.per} := 3$ — Proposed number of headed studs per row

$n_{anc} := n_{anc.row} \cdot n_{anc.per} = 9$ — Proposed number of headed studs

$d_{anc} := 1in$ — Proposed diameter of shaft of headed studs

$d_{h.anc} := 1in$ — Diameter of head of headed studs

$L_{emb} := 6in$ — Proposed embedment length of headed studs

$s_{anc} := 9in$ — Proposed spacing of headed studs

$s_{e.anc} := 0.5\left(w_{s.EP} - 2s_{anc}\right) = 3.00 \cdot in$ — Proposed edge spacing of headed studs

$s_{e.con} := 8in$ — Proposed spacing of headed studs to concrete edge

$A_{EP.p} := w_{w.EP} \cdot w_{s.EP} = 576.00 \cdot in^2$ — Area of base plate in plan

$I_{EP.zs} := \frac{w_{w.EP} \cdot \left(w_{s.EP}^3\right)}{12} = 27648.00 \cdot in^4$ — Moment of inertia of base plate with respect to plane made by vertical axis and strong axis

$c_{a.min} := s_{e.anc} = 3.00 \cdot in$ — Minimum edge distance from center of anchor shaft to edge of concrete

$A_{anc} := \frac{\pi}{4} \cdot d_{anc}^2 = 0.79 \cdot in^2$ — Cross-Sectional area of shaft of each anchor

$A_{h.anc} := \frac{\pi}{4} \cdot d_{anc}^2 = 0.79 \cdot in^2$ — Cross-Sectional area of head of each anchor

$h_{ef} := L_{emb} = 6.00 \cdot in$ — Effective embedment length of anchors

$c_{a1} := s_{e.con} = 8.00 \cdot in$ — Minimum distance from the edge of anchor shaft to the edge of concrete in one direction

$c_{a2} := s_{e.con} = 8.00 \cdot in$ — Distance from center of an anchor shaft to the edge of concrete in the direction perpendicular to ca1

$A_{1.EP} := w_{s.EP} \cdot w_{w.EP} = 576.00 \cdot in^2$ — Area of base plate in plan

$A_{2.EP} := A_2 = 4608.00 \cdot in^2$ — Maximum tributary area of supporting concrete surface

$A_{b.EP} := \min\left(2, \sqrt{\frac{A_{1.EP}}{A_{2.EP}}}\right) = 0.35$ — Area term for maximum bearing stress calculation

Design Loading

$P_{EP} := P_{BP} = 0.18 \cdot kip$ Unfactored axial load on embedded plate

$V_{EP} := V_{BP} = 12.74 \cdot kip$ Unfactored shear load on embedded plate

$M_{EP} := M_{BP} = 29.73 \cdot kip \cdot ft$ Unfactored moment on embedded plate

$f_{p.EP.max} := 0.65 \cdot \left(0.85 \cdot f_c\right) \cdot A_{b.EP} = 976.69 \cdot psi$ Maximum bearing stress under the base plate

$q_{EP.max} := f_{p.EP.max} \cdot w_{s.EP} = 23.44 \cdot \frac{kip}{in}$ Maximum bearing stress per unit length of base plate

$e_{EP.crit} := \frac{w_{s.EP}}{2} - \frac{P_{EP}}{2 \cdot q_{EP.max}} = 12.00 \cdot in$ Critical eccentricity of demad loading on base plate

$e_{EP} := \frac{M_{EP}}{P_{EP}} = 163.33 \cdot ft$ Eccentricity of demad loading on base plate

$Check_{e.EP} := \begin{cases} \text{"Small Moment - DG1 Section 3.3"} & \text{if } e_{EP} < e_{EP.crit} \\ \text{"Large Moment - DG1 Section 3.4"} & \text{otherwise} \end{cases} = \text{"Large Moment - DG1 Section 3.4"}$

Flexural Design - AISC Design Guide 1 Section 3.4

$f_{EP} := \frac{w_{s.EP}}{2} - s_{e.anc} = 9.00 \cdot in$ Dimension from CL of tension anchors to center of BP

$Check_{Y.EP} := \begin{cases} \text{"OK"} & \text{if } \left(f_{EP} + \frac{w_{s.EP}}{2}\right)^2 \geq \frac{2 \cdot P_{EP} \cdot \left(e_{EP} + f_{EP}\right)}{q_{EP.max}} \\ \text{"Increase BP"} & \text{otherwise} \end{cases} = \text{"OK"}$ Check if inequality is satisfied (Eq 3.4.4)

$Y_{EP} := \begin{cases} Y_{EP_0} \leftarrow \left(f_{EP} + \frac{w_{s.EP}}{2}\right) + \sqrt{\left(f_{EP} + \frac{w_{s.EP}}{2}\right)^2 - \frac{2 \cdot P_{EP} \cdot \left(e_{EP} + f_{EP}\right)}{q_{EP.max}}} \\ Y_{EP_1} \leftarrow \left(f_{EP} + \frac{w_{s.EP}}{2}\right) - \sqrt{\left(f_{EP} + \frac{w_{s.EP}}{2}\right)^2 - \frac{2 \cdot P_{EP} \cdot \left(e_{EP} + f_{EP}\right)}{q_{EP.max}}} \end{cases} = \begin{pmatrix} 41.26 \\ 0.74 \end{pmatrix} \cdot in$ Solutions for bearing width under base plate (Eq 3.4.3)

$Y_{EP} := \min\left(Y_{EP}\right) = 0.74 \cdot in$ Bearing width under base plate (Eq 3.4.3)

$T_{u.EP} := q_{max} \cdot Y_{EP} - P_{EP} = 9.03 \cdot kip$ Tension force in anchor rod

Anchor Geometry Checks - ACI 318-19 Section 17.9

$C_{spacing} := \begin{cases} \text{"OK"} & \text{if } s_{anc} \geq 4 \cdot d_{anc} \\ \text{"Increase Spacing"} & \text{otherwise} \end{cases} = \text{"OK"}$

Check minimum cast-in anchor spacing requirement (ACI 318-19 Table 17.9.2)

$C_{edge} := \begin{cases} \text{"OK"} & \text{if } c_{a.min} \geq 2in \\ \text{"Increase Edge Distance"} & \text{otherwise} \end{cases} = \text{"OK"}$

Check minimum cast-in anchor spacing requirement (ACI 318-19 Table 17.9.2)

Tensile Yield Strength - AISC 360-16 Section J 3.6

$\phi R_{n.TY} := 0.75 \cdot A_{anc} \cdot F_{nt.A325} = 53.01 \cdot kip$

Factored Tensile Yield Strength of each anchor

$DCR_{TY} := \frac{2.2 \cdot T_{u.EP}}{n_{anc.per} \cdot \phi R_{n.TY}} = 0.12$

Tensile Yield Strength DCR

$Check_{TY} := \begin{cases} \text{"OK"} & \text{if } 1.0 \geq DCR_{TY} \\ \text{"NG"} & \text{otherwise} \end{cases} = \text{"OK"}$

Check Tensile Yield Capacity

Shear Yield Strength - AISC 360-16 Section J 3.6

$\phi R_{n.VY} := 0.75 \cdot A_{anc} \cdot F_{nv.A325} = 31.81 \cdot kip$

Factored Shear Yield Strength of each anchor

$DCR_{VY} := \frac{2.2 V_{EP}}{n_{anc} \cdot \phi R_{n.VY}} = 0.10$

Shear Yield Strength DCR

$Check_{VY} := \begin{cases} \text{"OK"} & \text{if } 1.0 \geq DCR_{VY} \\ \text{"NG"} & \text{otherwise} \end{cases} = \text{"OK"}$

Check Shear Yield Capacity

Combined Tensile and Shear Yield Strength - AISC 360-16 Section J3.7

$f_{rv.EP} := \frac{V_{EP}}{n_{bolt} \cdot A_{anc}} = 2.70 \cdot ksi$

Required shear stress on each anchor

$F_{nt.comb} := 1.3 \cdot F_{nt.A325} - \frac{F_{nt.A325}}{0.75 \cdot F_{nv.A325}} \cdot f_{rv.EP} = 110.99 \cdot ksi$

Nominal tensile stress of each anchor modified to include the effects of shear stress

$\phi R_{n.TVY} := 0.9 \cdot A_{anc} \cdot F_{nt.comb} = 78.46 \cdot kip$

Factored Tensile Yield Strength of each anchor

$DCR_{TVY} := \frac{2.2 \sqrt{\left(\frac{T_{u.BP}}{n_{anc.per}}\right)^2 + \left(\frac{V_{BP}}{n_{anc}}\right)^2}}{\phi R_{n.TVY}} = 0.22$

Combined Tensile and Shear Yield Strength DCR

$Check_{TVY} := \begin{cases} \text{"OK"} & \text{if } 1.0 \geq DCR_{TVY} \\ \text{"NG"} & \text{otherwise} \end{cases} = \text{"OK"}$

Check Combined Tensile and Shear Yield Capacity

Concrete Breakout Strength (Tensile) - ACI 318-19 Section 17.6.2

$C_{gbst} := \begin{cases} \text{"Consider Group Effects"} & \text{if } s_{anc} < 3 \cdot h_{ef} \\ \text{"Check as Single Anchor"} & \text{otherwise} \end{cases} = \text{"Consider Group Effects"}$

Check if group effects shall be considered
(ACI 318-19 Table 17.5.1.3.1)

$A_{Nc} := \left(3 \cdot h_{ef} + 2s_{anc}\right) \cdot \left(1.5 \cdot h_{ef} + 2s_{anc} + s_{e.con}\right) = 1260.00 \cdot in^2$

Total projected breakout area of an anchor group

$A_{Nco} := 9 \cdot h_{ef}^{\ 2} = 324.00 \cdot in^2$

Maximum projected breakout area of a single anchor
(ACI 318-19 Eq 17.6.2.1.4)

$N_b := \begin{cases} 16lbf \cdot \sqrt{\frac{f_c}{psi}} \cdot \left(\frac{h_{ef}}{in}\right)^{\frac{5}{3}} & \text{if } 11in \le h_{ef} \le 25in \\ 24lbf \cdot \sqrt{\frac{f_c}{psi}} \cdot \left(\frac{h_{ef}}{in}\right)^{1.5} & \text{otherwise} \end{cases} = 24.94 \cdot kip$

Basic Concrete Breakout Strength of a Single Anchor
(ACI 318-19 Eq 17.6.2.2.1 and 17.6.2.2.3)

$\psi_{ecN} := 1.0$

Modification Factor for Eccentrically Loaded Anchor Groups
(ACI 318-19 Section 17.6.2.3.1)

$\psi_{edN} := \begin{cases} 1.0 & \text{if } c_{a.min} \ge 1.5 \cdot h_{ef} \\ 0.7 + 0.3 \cdot \frac{c_{a.min}}{1.5 \cdot h_{ef}} & \text{otherwise} \end{cases} = 0.80$

Breakout Edge Factor
(ACI 318-19 Eq 17.6.2.4.1)

$\psi_{cN} := 1.25$

Breakout Cracking Factor
(ACI 318-19 Section 17.6.2.5.1)

$\psi_{cpN} := 1.0$

Breakout Splitting Factor
(ACI 318-19 Eq 17.6.2.6.1)

$\phi R_{n.T.BO} := \phi_{BO} \cdot \frac{A_{Nc}}{A_{Nco}} \cdot \psi_{ecN} \cdot \psi_{edN} \cdot \psi_{cN} \cdot \psi_{cpN} \cdot N_b = 67.90 \cdot kip$

Factored Tensile Breakout Strength of Anchor Group

$DCR_{T.BO} := \frac{2.2 \cdot T_{u.EP}}{n_{anc.per} \cdot \phi R_{n.T.BO}} = 0.10$

Tensile Breakout DCR

$Check_{T.BO} := \begin{cases} \text{"OK"} & \text{if } 1.0 \ge DCR_{T.BO} \\ \text{"NG"} & \text{otherwise} \end{cases} = \text{"OK"}$

Check Tensile Breakout Capacity

Concrete Pullout Strength - ACI 318-19 Section 17.6.3

$N_p := 8 \cdot A_{h.anc} \cdot f_c = 31.42 \cdot kip$

Basic Single Anchor Pullout Strength for headed stud
(ACI 318-19 Eq 17.6.3.2.2b)

$\psi_{cp} := 1.0$

Pullout Cracking Factor (ACI 318-19 Eq 17.6.3.3)

$\phi R_{n.PT} := n_{anc} \cdot \phi_{PT} \cdot \psi_{cp} \cdot N_p = 197.92 \cdot kip$

Factored Pullout Strength of Anchor Group

$DCR_{PT} := \frac{2.2 \cdot T_{u.EP}}{n_{anc.per} \cdot \phi R_{n.PT}} = 0.03$

Tensile Pullout DCR

$Check_{PT} := \begin{cases} \text{"OK"} & \text{if } 1.0 \ge DCR_{PT} \\ \text{"NG"} & \text{otherwise} \end{cases} = \text{"OK"}$

Check Pullout Capacity

Concrete Breakout Strength (Shear) - ACI 318-19 Section 17.7.2

$C_{gbsv} := \begin{cases} \text{"Check as Single Anchor"} & \text{if } s_{anc} \le 3 \cdot c_{a1} \\ \text{"Consider Group Effects"} & \text{otherwise} \end{cases} = \text{"Check as Single Anchor"}$

Check if group effects shall be considered (ACI 318-19 Table 17.5.1.3.1)

$A_{Vc} := \left(1.5c_{a1} + 2 \cdot s_{anc}\right) \cdot \left(1.5c_{a1}\right) = 360.00 \cdot in^2$

Total projected breakout area of an anchor group

$A_{Vco} := 4.5 \cdot c_{a1}^{\ 2} = 288.00 \cdot in^2$

Maximum projected breakout area of a single anchor (ACI 318-19 Eq 17.7.2.1.3)

$V_b := \left[8 \cdot \left(\frac{h_{ef}}{d_{anc}} \right)^{0.2} \cdot \sqrt{\frac{d_{anc}}{in}} \right] \cdot \sqrt{\frac{f_c}{psi}} \cdot \left(\frac{c_{a1}}{in} \right)^{1.5} \cdot lbf = 18.32 \cdot kip$

Basic Concrete Breakout Strength of a Single Anchor (ACI 318-19 Eq 17.7.2.2.2)

$\psi_{edV} := \begin{cases} 1.0 & \text{if } c_{a2} \ge 1.5 \cdot c_{a1} \\ 0.7 + 0.3 \cdot \frac{c_{a2}}{1.5 \cdot c_{a1}} & \text{otherwise} \end{cases} = 0.90$

Breakout Edge Factor (ACI 318-19 Eq 17.7.2.4.1)

$\psi_{cV} := 1.0$

Breakout Cracking Factor (ACI 318-19 Table 17.7.2.5.1)

$\psi_{hV} := 1.0$

Breakout Splitting Factor (ACI 318-19 Eq 17.7.2.6.1)

$\phi R_{n.V.BO} := \phi_{BO} \cdot \frac{A_{Vc}}{A_{Vco}} \cdot \psi_{edV} \cdot \psi_{cV} \cdot \psi_{hV} \cdot V_b = 14.42 \cdot kip$

Factored Shear Breakout Strength of an Anchor Group (ACI 318-19 Eq 17.7.2.1b)

$DCR_{V.BO} := \frac{2.2V_{EP}}{n_{anc} \phi R_{n.V.BO}} = 0.22$

Shear Breakout DCR

$Check_{V.BO} := \begin{cases} \text{"OK"} & \text{if } 1.0 \ge DCR_{V.BO} \\ \text{"NG"} & \text{otherwise} \end{cases} = \text{"OK"}$

Check Shear Breakout Capacity

Concrete Pryout Strength - ACI 318-19 Section 17.7.3

$k_{cp} := \begin{cases} 1.0 & \text{if } h_{ef} < 2.5in \\ 2.0 & \text{otherwise} \end{cases} = 2.00$

Effective Embedment Factor (ACI 318-19 Eq 17.7.3.1)

$V_{cpg} := k_{cp} \cdot \left(\frac{A_{Nc}}{A_{Nco}} \cdot \psi_{ecN} \cdot \psi_{edN} \cdot \psi_{cN} \cdot \psi_{cpN} \cdot N_b \right) = 193.99 \cdot kip$

Nominal Single Anchor Pryout Strength (ACI 318-19 Eq 17.7.3.1b)

$\phi R_{n.PV} := \phi_{PV} \cdot V_{cpg} = 135.79 \cdot kip$

Factored Shear Pryout Strength of Anchor Group

$DCR_{PV} := \frac{2.2V_{EP}}{n_{anc} \phi R_{n.PV}} = 0.02$

Shear Pryout DCR

$Check_{PV} := \begin{cases} \text{"OK"} & \text{if } 1.0 \ge DCR_{PV} \\ \text{"NG"} & \text{otherwise} \end{cases} = \text{"OK"}$

Check Shear Pryout Capacity

Client :	Stockton East Water District
Project :	Bellota Weir Modifications Project
Project No :	10209137
Title :	Boom Pile Design Checks
Purpose:	This calculation package is for the design of the boom piles that will be installed as a part of the Bellota Weir Modifications Project. The design will be for driven 36-inch diameter, 0.5-inch thick, open-ended, hollow steel pipe piles. Per the HDR Geotechnical Investigation Report (2021), it is recommended to assume that a plug does not form inside the pile until the pile reaches below an elevation of 90 feet. Additionally, the soil above Elevation 100 feet induces downdrag forces on the pile. End bearing effects are considered for the pile capacities. Group effects will not be considered for the piles in LPile per Table 5 in the HDR Geotechnical Investigation Report (2021).
Originator :	Ryan Bunyan, EIT Date : January 3, 2022
Checked By :	Iman Ghorbani, PE

Design Inputs

Steel Properties

$E_s := 29000ksi$ — Steel modulus of elasticity

$G_s := 11200ksi$ — Steel shear modulus of elasticity

$F_{u.A992} := 65ksi$ — Ultimate Strength of A992 Steel

$F_{y.A992} := 50 \cdot ksi$ — Yield strength of A992 Steel

$\gamma_s := 0.2836 \frac{lbf}{in^3}$ — Unit weight of A992 steel

$F_{y.A500.C} := 46 \cdot ksi$ — Yield strength of A500 Group C Steel

$F_{u.A500.C} := 62 \cdot ksi$ — Ultimate strength of A500 Group C Steel

$\varepsilon_y := 0.00207$ — Steel yield strain of rebar

$F_{nt.A325} := 90ksi$ — Nominal tensile strength of A325 Group A Bolts

$F_{nv.A325} := 54 \cdot ksi$ — Nominal Shear strength of A325 Group A Bolts

Concrete Properties

$\gamma_c := 150pcf$ — Unit weight of concrete

$f_c := 4000psi$ — Minimum 28-day Compressive strength of concrete

Pile Geometry

I cheked LPile we can go with total 70 ft long pile with tip elevation at 57, the result would not change much

$d_{pile} := 36in$ — Outer diameter of piles supporting intake structure

$t_{pile} := 0.5in$ — Outer diameter of piles supporting intake structure

$L_{pile} := 100ft$ — Effective length of pile

$K_s := 2.0$ — Effective Length Factor

$d_{i.pile} := d_{pile} - 2 \cdot t_{pile} = 35.00 \cdot in$ — Inner Diameter

$A_s := \frac{\pi}{4} \cdot \left[d_{pile}^{\,2} - \left(d_{pile} - 2t_{pile} \right)^2 \right] = 55.76 \cdot in^2$ — Cross Sectional Area of Steel Pile

Design Loading

The lateral load on the pile was provided by the HDR Mechanical team and represent a water level at Elev 124.00, which is the 100-year flood elevation. The piles are not bearing any weight from overhead structures. Since the piles are spaced very far apart the calculations will not incorporate any group effect modifiers. The piles will be designed for the maximum shear force Vp, and the anchors will be designed for the combined load of Hp and Vp.

$V_p := 47.68kip$ — Max unfactored lateral load on the piles and pile anchors in the up/downstream direction

$H_p := 119.21kip$ — Max unfactored lateral load on the pile anchors in the cross-channel direction

$T_{u.p} := \sqrt{H_p^2 + V_p^2} = 128391.61 \cdot lbf$ — Resultant unfactored lateral load on the pile anchors

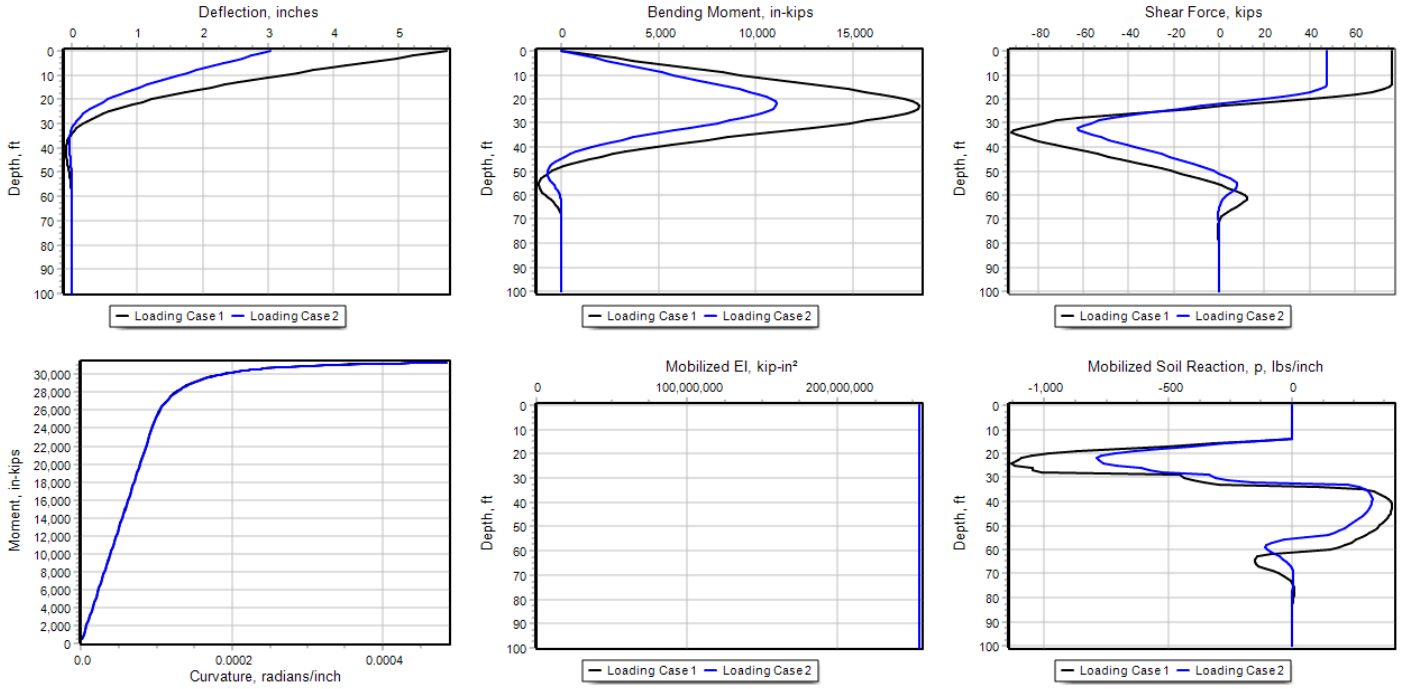
$W_{pile} := A_s \cdot \gamma_s \cdot L_{pile} = 18.98 \cdot kip$ — Weight of pile

$Q_{u.p} := W_{pile} = 18977.36 \cdot lbf$ — Max estimated unfactored bearing force demand on the piles considering skin friction (Servicibility Design)

$V_{u.pile} := 1.6 \cdot V_p = 76288.00 \cdot lbf$ — Resultant factored lateral load on the piles (Strength Design)

$P_{u.pile} := 1.6 \cdot Q_{u.p} = 30363.77 \cdot lbf$ — Resultant factored bearing force on the piles (Strength Design)

$M_{u.pile} := 18385.27 \cdot kip \cdot in$ — Maximum Moment Demand on filled pile from LPile



Hollow Steel Pile Capacity Calculations - AISC SCM Table 6-5

Member Property Calculations

$I_s := \frac{\pi}{64} \cdot \left(d_{pile}^{4} - d_{i.pile}^{4} \right) = 8786.20 \cdot in^4$ Moment of Inertia

$S_s := \frac{\pi}{32 \cdot d_{pile}} \cdot \left(d_{pile}^{4} - d_{i.pile}^{4} \right) = 488.12 \cdot in^3$ Elastic section modulus

$r_s := \sqrt{\frac{I_s}{A_s}} = 12.55 \cdot in$ Radius of gyration strong axis

$Z_s := \frac{1}{6} \cdot \left(d_{pile}^{3} - d_{i.pile}^{3} \right) = 630.17 \cdot in^3$ Plastic section modulus strong axis

$J_t := \frac{\pi}{32} \cdot \left(d_{pile}^{4} - d_{i.pile}^{4} \right) = 17572.40 \cdot in^4$ Torsional constant

Axial Capacity

The axial capacity of the pile can be determined by extrapolating from Figure 15 in the HDR Geotechnical Investigation Report (HDR, 2021). The calculations assume that the trend provided in the figure continues linearly. Similarly, the graph shows capacities for a 24-inch pile so values were increased by 1.5 to estimate the capacities for a 36-inch pile.

$ELEV_{ground} := 105 ft$ Elevation of the ground

$ELEV_{pile.bot} := ELEV_{ground} - \left(L_{pile} - 30ft \right) = 35.00 ft$ Elevation of the bottom of the pile

$P_{n.90ft} := 5kip$ Axial capacity of the pile at Elev. 90ft

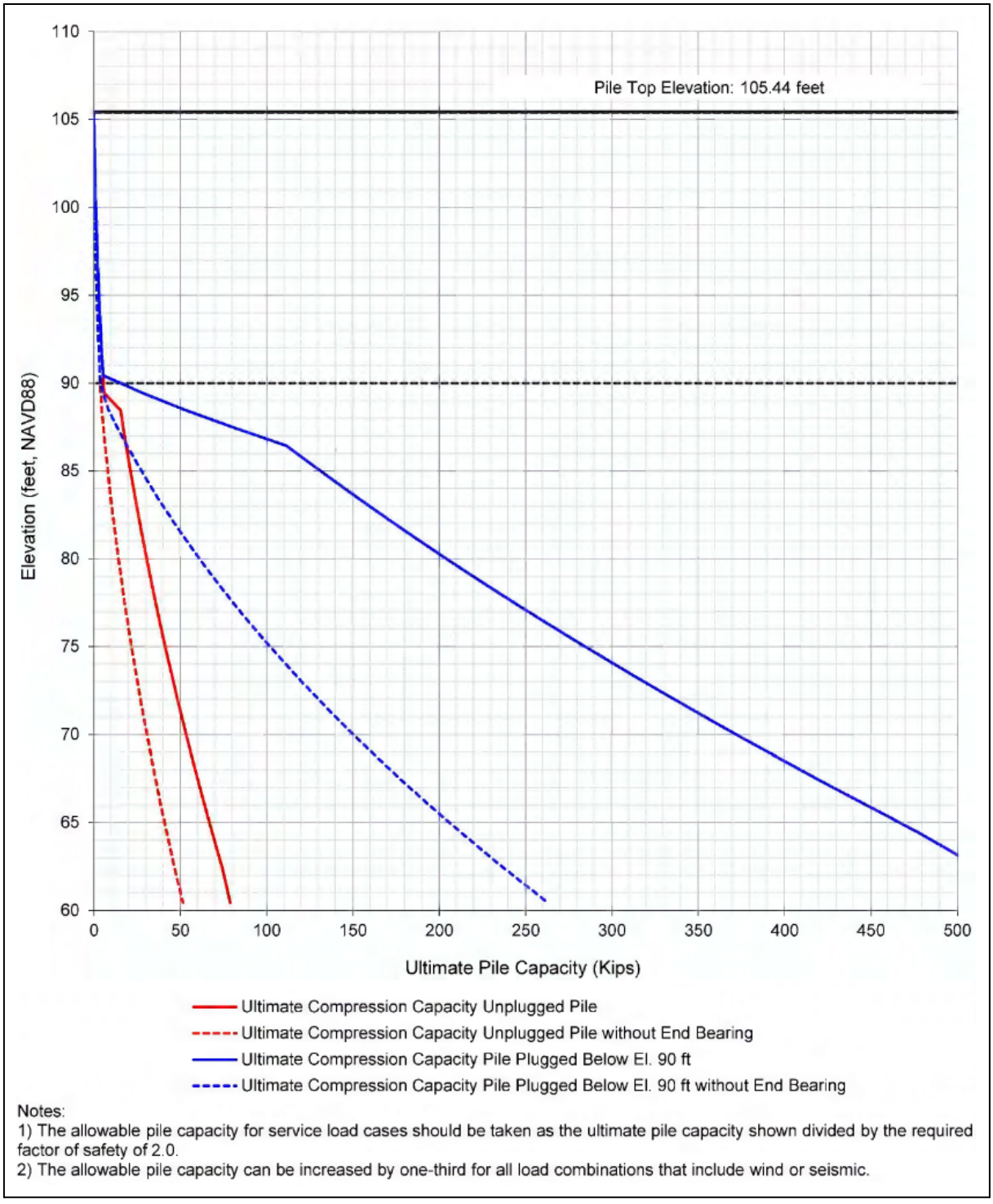
$P_{n.60ft} := 260kip$ Axial capacity of the pile at Elev. 60ft

$P_{n.24} := P_{n.90ft} + \frac{90ft - ELEV_{pile.bot}}{90ft - 60ft} \cdot \left(P_{n.60ft} - P_{n.90ft} \right) = 472.50 \cdot kip$ Extrapolated axial capacity of 24-inch pile

$P_{n.36} := 1.5 \cdot P_{n.24} = 708.75 \cdot kip$ Estimated unfactored axial capacity of 36-inch pile

$\phi P_n := 0.5 P_{n.36} = 354.37 \cdot kip$ Design axial capacity with safety factor of 2 applied

$DCR_A := \frac{P_{u.pile}}{\phi P_n} = 0.09$ DCR for axial forces considering skin friction from the boom



Notes:
1) The allowable pile capacity for service load cases should be taken as the ultimate pile capacity shown divided by the required factor of safety of 2.0.
2) The allowable pile capacity can be increased by one-third for all load combinations that include wind or seismic.

Flexural Strength - AISC F8

Flexural Slenderness Check

$r_{wt} := \frac{d_{pile}}{t_{pile}} = 72.00$ Pile Width-to-Thickness Ratio

$\lambda_{p.f.s} := 0.07 \cdot \sqrt{\frac{E_s}{F_{y.A500.C}}} = 1.76$ Flexural compact limit

$\lambda_{r.f.s} := 0.31 \cdot \sqrt{\frac{E_s}{F_{y.A500.C}}} = 7.78$ Flexural noncompact limit

$$FC := \begin{cases} \text{"Compact"} & \text{if } r_{wt} < \lambda_{p.f.s} \\ \text{"Slender"} & \text{if } r_{wt} \geq \lambda_{r.f.s} \\ \text{"Noncompact"} & \text{otherwise} \end{cases} = \text{"Slender"}$$

Yielding

$M_{n.y} := F_{y.A500.C} \cdot Z_s = 28987.67 \cdot kip \cdot in$ Nominal yield strength (AISC 360-16 F8-1)

Local Buckling

$F_{cr.lb} := \frac{0.33 \cdot E_s}{r_{wt}} = 132.92 \cdot ksi$ Local flexural buckling stress

$$M_{n.lb} := \begin{cases} M_{n.y} & \text{if } FC = \text{"Compact"} \\ \left(\frac{0.21 \cdot E_s}{r_{wt}} + F_{y.A500.C}\right) \cdot S_s & \text{if } FC = \text{"Noncompact"} \\ F_{cr.lb} \cdot S_s & \text{otherwise} \end{cases} = 5406.63 \cdot kip \cdot ft$$ Nominal local buckling strength (AISC 360-16 F8-2, F8-3, F8-4)

$\phi M_n := 0.9 \cdot \min(M_{n.y}, M_{n.lb}) = 26088.90 \cdot kip \cdot in$ Design flexural capacity

$DCR_M := \frac{M_{u.pile}}{\phi M_n} = 0.705$

Shear Strength - AISC G5

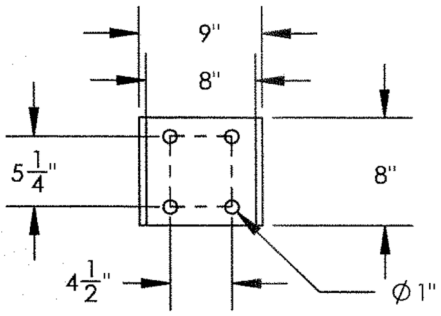
$F_{cr.v} := \max\left(\frac{1.60 \cdot E_s}{\sqrt{\frac{L_{pile}}{d_{pile}}} \cdot r_{wt}^{\frac{5}{4}}}, \frac{0.78 \cdot E_s}{r_{wt}^{\frac{3}{2}}}\right) = 38.32 \cdot ksi$ Critical shear buckling stress (AISC 360-16 G5-2a, G5-2b)

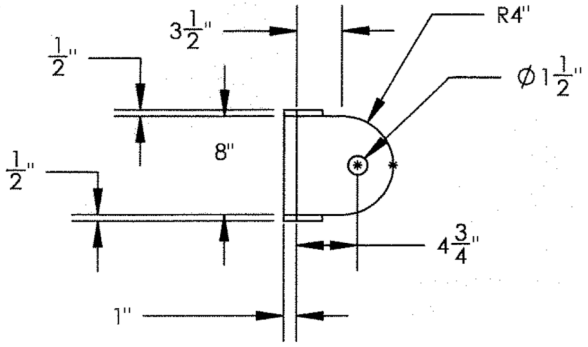
$\phi V_n := 0.5 \cdot F_{cr.v} \cdot A_s = 1068.39 \cdot kip$ Design shear capacity (AISC 360-16 G5-1)

$DCR_V := \frac{V_{u.pile}}{\phi V_n} = 0.071$

Debris Boom Wall-Mount Connection Design Checks

Connection Geometry





$n_{hp} := 4$ — Number of hinge plates

$n_{anc.per} := 4$ — Number of anchors per hinge plate

$n_{anc} := n_{hp} \cdot n_{anc.per} = 16$ — Total number of anchors

$d_{anc} := 1in - \frac{1}{16}in = 0.94 \cdot in$ — Diameter of anchors

$L_{emb} := 12in$ — Embedment Length of anchors

$h_{ef} := L_{emb} = 12.00 \cdot in$ — Effective embedment length of anchors

$s_{anc} := 5.25in$ — Spacing of anchors

$A_{anc} := \frac{\pi}{4} \cdot d_{anc}^{2} = 0.69 \cdot in^2$ — Cross-Sectional area of shaft of each anchor

$t_{wall} := 36in$ — Thickness of the concrete wall that the anchors are embedded into

$c_{a1} := 0.5 \cdot (36in - s_{anc}) = 15.38 \cdot in$ — Distance from the center of an anchor shaft to the edge of concrete in the direction of the shear forces

$c_{a2} := 4in$ — Distance from the center of an anchor shaft to the edge of concrete in the direction perpendicular to c.a1

$c_{a.min} := \min(c_{a1}, c_{a2}) = 4.00 \cdot in$ — Minimum distance from center of an anchor shaft to the edge of concrete

Design Loading

Per design drawings, the anchors shall be designed for a minimum tensile load of 7 kips and a minimum shear load of 500 lbs. The design tensile load will be the maximum of 7 kips and the load provided by the HDR Mechanical team.

$T_u := \max\left(1.6 \cdot 7kip, \frac{1.6 T_{u.p}}{n_{anc}}\right) = 12.84 \cdot kip$ — Total factored tensile load on each anchor

$V_u := 1.6 \cdot 0.5kip = 0.80 \cdot kip$ — Total factored shear load on each anchor

Anchor Geometry Checks - ACI 318-19 Section 17.9

$C_{spacing} := \begin{cases} \text{"OK"} & \text{if } s_{anc} \geq 4 \cdot d_{anc} \\ \text{"Increase Spacing"} & \text{otherwise} \end{cases} = \text{"OK"}$

Check minimum cast-in anchor spacing requirement (ACI 318-19 Table 17.9.2)

Tensile Yield Strength - AISC 360-16 Section J3-1

$\phi R_{n.TY} := 0.9 \cdot A_{anc} \cdot F_{nt.A325} = 55.91 \cdot kip$

Factored Tensile Yield Strength of each anchor

$DCR_{TY} := \frac{T_u}{\phi R_{n.TY}} = 0.23$

Tensile Yield Strength DCR

$Check_{TY} := \begin{cases} \text{"OK"} & \text{if } 1.0 \geq DCR_{TY} \\ \text{"NG"} & \text{otherwise} \end{cases} = \text{"OK"}$

Check Tensile Yield Capacity

Shear Yield Strength - AISC 360-16 Section J3-1

$\phi R_{n.VY} := 0.9 \cdot A_{anc} \cdot F_{nv.A325} = 33.55 \cdot kip$

Factored Shear Yield Strength of each anchor

$DCR_{VY} := \frac{V_u}{\phi R_{n.VY}} = 0.02$

Shear Yield Strength DCR

$Check_{VY} := \begin{cases} \text{"OK"} & \text{if } 1.0 \geq DCR_{VY} \\ \text{"NG"} & \text{otherwise} \end{cases} = \text{"OK"}$

Check Shear Yield Capacity

Combined Tensile and Shear Yield Strength - AISC 360-16 Section J3-2

$f_{rv} := \frac{V_u}{A_{anc}} = 1.16 \cdot ksi$

Required shear stress on each anchor

$F_{nt.comb} := 1.3 \cdot F_{nt.A325} - \frac{F_{nt.A325}}{0.75 \cdot F_{nv.A325}} \cdot f_{rv} = 114.42 \cdot ksi$

Nominal tensile stress of each anchor modified to include the effects of shear stress

$\phi R_{n.TVY} := 0.9 \cdot A_{anc} \cdot F_{nt.comb} = 71.09 \cdot kip$

Factored Tensile Yield Strength of each anchor

$DCR_{TVY} := \frac{\sqrt{T_u^{\,2} + V_u^{\,2}}}{\phi R_{n.TVY}} = 0.18$

Combined Tensile and Shear Yield Strength DCR

$Check_{TVY} := \begin{cases} \text{"OK"} & \text{if } 1.0 \geq DCR_{TVY} \\ \text{"NG"} & \text{otherwise} \end{cases} = \text{"OK"}$

Check Combined Tensile and Shear Yield Capacity

Concrete Breakout Strength (Tensile) - ACI 318-19 Section 17.6.2

$C_{group} := \begin{cases} \text{"Consider Group Effects"} & \text{if } s_{anc} < 3 \cdot h_{ef} \\ \text{"Check as Single Anchor"} & \text{otherwise} \end{cases} = \text{"Consider Group Effects"}$

Check if group effects shall be considered
(ACI 318-19 Table 17.5.1.3.1)

$A_{Nc} := \left(3 \cdot h_{ef} + s_{anc}\right)^2 = 1701.56 \cdot in^2$

Total projected breakout area of an anchor group

$A_{Nco} := 9 \cdot h_{ef}^{\ 2} = 1296.00 \cdot in^2$

Maximum projected breakout area of a single anchor
(ACI 318-19 Eq 17.6.2.1.4)

$N_b := \begin{cases} 16lbf \cdot \sqrt{\frac{f_c}{psi}} \cdot \left(\frac{h_{ef}}{in}\right)^{\frac{5}{3}} & \text{if } 11in \le h_{ef} \le 25in \\ 24lbf \cdot \sqrt{\frac{f_c}{psi}} \cdot \left(\frac{h_{ef}}{in}\right)^{1.5} & \text{otherwise} \end{cases} = 63.65 \cdot kip$

Basic Concrete Breakout Strength of a Single Anchor
(ACI 318-19 Eq 17.6.2.2.1 and 17.6.2.2.3)

$\psi_{ecN} := 1.0$

Modification Factor for Eccentrically Loaded Anchor Groups
(ACI 318-19 Section 17.6.2.3.1)

$\psi_{edN} := \begin{cases} 1.0 & \text{if } c_{a.min} \ge 1.5 \cdot h_{ef} \\ 0.7 + 0.3 \cdot \frac{c_{a.min}}{1.5 \cdot h_{ef}} & \text{otherwise} \end{cases} = 0.77$

Breakout Edge Factor
(ACI 318-19 Eq 17.6.2.4.1)

$\psi_{cN} := 1.25$

Breakout Cracking Factor
(ACI 318-19 Section 17.6.2.5.1)

$\psi_{cpN} := 1.0$

Breakout Splitting Factor
(ACI 318-19 Eq 17.6.2.6.1)

$\phi R_{n.T.BO} := 0.7 \cdot \frac{A_{Nc}}{A_{Nco}} \cdot \psi_{ecN} \cdot \psi_{edN} \cdot \psi_{cN} \cdot \psi_{cpN} \cdot N_b = 56.06 \cdot kip$

Factored Tensile Breakout Strength of Anchor Group

$DCR_{T.BO} := \frac{n_{anc.per} \cdot T_u}{\phi R_{n.T.BO}} = 0.92$

Tensile Breakout DCR

$Check_{T.BO} := \begin{cases} \text{"OK"} & \text{if } 1.0 \ge DCR_{T.BO} \\ \text{"NG"} & \text{otherwise} \end{cases} = \text{"OK"}$

Check Tensile Breakout Capacity

Concrete Pullout Strength - ACI 318-19 Section 17.6.3

$N_p := 8 \cdot A_{anc} \cdot f_c = 22.09 \cdot kip$

Basic Single Anchor Pullout Strength for headed stud
(ACI 318-19 Eq 17.6.3.2.2b)

$\psi_{cp} := 1.4$

Pullout Cracking Factor (ACI 318-19 Eq 17.6.3.3)

$\phi R_{n.PT} := 0.7 \cdot \psi_{cp} \cdot N_p = 21.65 \cdot kip$

Factored Pullout Strength of a Single Anchor

$DCR_{PT} := \frac{T_u}{\phi R_{n.PT}} = 0.59$

Tensile Pullout DCR

$Check_{PT} := \begin{cases} \text{"OK"} & \text{if } 1.0 \ge DCR_{PT} \\ \text{"NG"} & \text{otherwise} \end{cases} = \text{"OK"}$

Check Pullout Capacity

Concrete Breakout Strength (Shear) - ACI 318-19 Section 17.7.2

$C_{gv} := \begin{cases} \text{"Consider Group Effects"} & \text{if } s_{anc} < 3 \cdot c_{a1} \\ \text{"Check as Single Anchor"} & \text{otherwise} \end{cases} = \text{"Consider Group Effects"}$

Check if group effects shall be considered (ACI 318-19 Table 17.5.1.3.1)

$A_{Vc} := t_{wall} \cdot \left[2 \cdot \left(1.5 \cdot c_{a1}\right) + s_{anc}\right] = 1849.50 \cdot in^2$

Total projected breakout area of an anchor group

$A_{Vco} := 4.5 \cdot c_{a1}^{\,2} = 1063.76 \cdot in^2$

Maximum projected breakout area of a single anchor (ACI 318-19 Eq 17.7.2.1.3)

$l_e := \min\left(8 \cdot d_{anc}, h_{ef}\right) = 7.50 \cdot in$

Load-bearing length of the anchor for shear (ACI 318-19 Sec 17.7.2.2.1)

$V_b := \min\left[\left[7 \cdot lbf \cdot \left(\frac{l_e}{d_{anc}}\right)^{0.2} \cdot \sqrt{\frac{d_{anc}}{in}}\right] \cdot \sqrt{\frac{f_c}{psi}} \cdot \left(\frac{c_{a1}}{in}\right)^{1.5}, 9 \cdot lbf \cdot \sqrt{\frac{f_c}{psi}} \cdot \left(\frac{c_{a1}}{in}\right)^{1.5}\right] = 34.32 \cdot kip$

Basic Concrete Breakout Strength of a Single Anchor (ACI 318-19 Eq 17.6.2.2.1(a) and (b))

$\psi_{ecV} := 1.0$

Modification Factor for Eccentrically Loaded Anchor Groups (ACI 318-19 Section 17.7.2.3.1)

$\psi_{edV} := \begin{cases} 1.0 & \text{if } c_{a2} \geq 1.5 \cdot c_{a1} \\ 0.7 + 0.3 \cdot \dfrac{c_{a2}}{1.5 \cdot c_{a1}} & \text{otherwise} \end{cases} = 0.75$

Breakout Edge Factor (ACI 318-19 Eq 17.7.2.4.1)

$\psi_{cV} := 1.2$

Breakout Cracking Factor (ACI 318-19 Table 17.7.2.5.1)

$\psi_{hV} := \begin{cases} 1.0 & \text{if } t_{wall} \geq 1.5 \cdot c_{a1} \\ \max\left(\sqrt{\dfrac{1.5 \cdot c_{a1}}{t_{wall}}}, 1.0\right) & \text{otherwise} \end{cases} = 1.00$

Breakout Splitting Factor (ACI 318-19 Eq 17.7.2.6.1)

$\phi R_{n.V.BO} := 0.7 \cdot \frac{A_{Vc}}{A_{Vco}} \cdot \psi_{ecV} \cdot \psi_{edV} \cdot \psi_{cV} \cdot \psi_{hV} \cdot V_b = 37.69 \cdot kip$

Factored Shear Breakout Strength of Anchor Group (ACI 318-19 Eq. 17.7.2.1b)

$DCR_{V.BO} := \frac{V_u \cdot n_{anc.per}}{\phi R_{n.V.BO}} = 0.08$

Shear Breakout DCR

$Check_{V.BO} := \begin{cases} \text{"OK"} & \text{if } 1.0 \geq DCR_{V.BO} \\ \text{"NG"} & \text{otherwise} \end{cases} = \text{"OK"}$

Check Shear Breakout Capacity

Concrete Pryout Strength - ACI 318-19 Section 17.7.3

$k_{cp} := \begin{cases} 1.0 & \text{if } h_{ef} < 2.5in \\ 2.0 & \text{otherwise} \end{cases} = 2.00$

Effective Embedment Factor (ACI 318-19 Eq 17.7.3.1)

$N_{cpg} := \frac{A_{Nc}}{A_{Nco}} \cdot \psi_{ecN} \cdot \psi_{edN} \cdot \psi_{cN} \cdot \psi_{cpN} \cdot N_b = 80.08 \cdot kip$

Nominal Breakout Strength of an Anchor Group in Tension (ACI 318-19 Section 17.7.3.1.2)

$V_{cpg} := k_{cp} \cdot N_{cpg} = 160.17 \cdot kip$

Nominal Pryout Strength of an Anchor Group in Shear (ACI 318-19 Eq 17.7.3.1b)

$\phi R_{n.PV} := 0.7 \cdot V_{cpg} = 112.12 \cdot kip$

Factored Shear Pryout Strength of the Anchor Group

$$DCR_{PV} := \frac{V_u \cdot n_{anc.per}}{\phi R_{n.PV}} = 0.03$$

Shear Pryout DCR

$$Check_{PV} := \begin{cases} \text{"OK"} & \text{if } 1.0 \geq DCR_{PV} \\ \text{"NG"} & \text{otherwise} \end{cases} = \text{"OK"}$$

Check Shear Pryout Capacity

Designer: Iman Ghorbani

Sheet Pile Design
Option- South Bank

Sheet Pile Property

$E := 29000ksi$ — Modulus of sheet pile

$I_{sheet} := 265.3in^4$ — AZ-17-700 Sheet Pile

$Z_{sheet} := 37.7in^3$ — Plastic modulus

$S_{sheet} := 32.2in^3$ — Elastic modulus

Normal Loading:

Top elevation 127 and tip elevation 100

$\Delta_{ground} := \frac{3.73 \cdot 10^9 lbf \cdot in^3}{E \cdot I_{sheet}} = 0.485 \cdot in$ — Wall deformation at ground surface under flood case

$M_{max} := 1.7 \cdot 10^4 \cdot lbf \cdot ft$

$\sigma := \frac{M_{max} \cdot 2.2}{0.9 \cdot S_{sheet}} = 15.487 \cdot ksi$ — Less than 50 ksi OK-1.4 is unusual load factor

Designer: Iman Ghorbani

Sheet Pile Design for East Side Edge of South Bank

Sheet Pile Property

$E := 29000ksi$ Modulus of sheet pile

$I_{sheet} := 170in^4$ AZ-25-800 Sheet Pile

$Z_{sheet} := 53.8in^3$ Plastic modulus

$S_{sheet} := 46.5in^3$ Elastic modulus

Top elevation 127 and tip elevation 84

Normal Loading

$\Delta_{ground} := \frac{1.7 \cdot 10^{10} lbf \cdot in^3}{E \cdot I_{sheet}} = 3.448 \cdot in$ Wall deformation at ground- Need to use King Pile Design

$M_{max} := 3.14 \cdot 10^4 \cdot lbf \cdot ft$

$\sigma := \frac{M_{max} \cdot 2.2}{0.9 \cdot S_{sheet}} = 19.808 \cdot ksi$ Less than 50 ksi OK-2.2 is usual load factor

Seismic Loading

$\Delta_{ground} := \frac{1.34 \cdot 10^9 lbf \cdot in^3}{E \cdot I_{sheet}} = 0.272 \cdot in$ Wall deformation at ground surface under flood case

$M_{max} := 1.72 \cdot 10^4 \cdot lbf \cdot ft$

$\sigma := \frac{M_{max} \cdot 1.0}{0.9 \cdot S_{sheet}} = 4.932 \cdot ksi$ Less than 50 ksi OK-2.2 is usual load factor

$$\frac{0.44\text{in}^2 \cdot 2}{8\text{in} \cdot 1.5\text{ft}} = 6.111 \times 10^{-3}$$

King Pile Design At East Side of South Bank

$K_a := 0.33$ Active soil pressure coefficient from Geotech for soil above 115 ft

$\gamma_{soil} := 115pcf$ Soil density above 115 ft

$h_{soil} := 11ft$ Height of soil behind wall

$P_{live} := 250psf$

$P_{static} := \frac{1}{2}\cdot K_a\cdot\gamma_{soil}\cdot h_{soil}^{2} + P_{live}\cdot K_a\cdot h_{soil} = 3.203\cdot\frac{kip}{ft}$ Soil pressure force per linear foot

$M_{static} := \frac{1}{2}\cdot K_a\cdot\gamma_{soil}\cdot h_{soil}^{2}\cdot\frac{1}{3}\cdot h_{soil} \ldots + P_{live}\cdot K_a\cdot\frac{h_{soil}^{2}}{2} = 13.41\cdot\frac{kip\cdot ft}{ft}$ Soil bending force per linear foot at ground surface

$P_{EQ} := 6\cdot\left(\frac{h_{soil}}{ft}\right)\cdot psf\cdot h_{soil} = 0.726\cdot\frac{kip}{ft}$ Seismic load

$M_{EQ} := 6\cdot\left(\frac{h_{soil}}{ft}\right)\cdot psf\cdot h_{soil}\cdot\frac{h_{soil}}{2} = 3.993\cdot\frac{kip\cdot ft}{ft}$ Seismic bending load

King Pile Section Property:

Use HZ 630M

$w_{ps} := 83.74in$ Pile width

$A_{ps} := 12.55\frac{in^{2}}{ft}\cdot w_{ps} = 87.578\cdot in^{2}$ Area of pile with sheet pile

$I_{ps} := 1112\frac{in^4}{ft}\cdot w_{ps} = 7.76\times 10^3\cdot in^4$ Moment inertia of pile with sheet pile

$S_{ps} := 83\frac{in^3}{ft}\cdot w_{ps} = 579.202\cdot in^3$ Elastic modulus with sheet pile

$E_{steel} := 29000ksi$

$w_p := 16.54in$ Width of HZ pile

$A_p := 47.83in^2$ Area of HZ pile

$I_p := 5224.5in^4$ Moment inertia of pile

$S_p := 426.2in^3$ Section modulus of pile

$P_{static\_top} := 2.2P_{static}\cdot w_{ps} = 4.918\times 10^4\cdot lbf$

$M_{static\_top} := 2.2M_{static}\cdot w_{ps} = 2.47\times 10^6\cdot lbf\cdot in$

$P_{EQ\_top} := \left(P_{EQ} + P_{static}\right)\cdot w_{ps} = 2.742\times 10^4\cdot lbf$

$M_{EQ\_top} := \left(M_{EQ} + M_{static}\right)\cdot w_{ps} = 1.457\times 10^6\cdot lbf\cdot in$

$M_{demand\_combo} := \frac{10709kip\cdot in}{0.9\cdot S_{ps}} = 20.544\cdot ksi$

$M_{demand\_HZ} := \frac{7427kip\cdot in}{0.9\cdot S_p} = 19.362\cdot ksi$

Use king pile HZ1080M C-12/AZ 18-800
25 ft long sheet pile 55 ft long H Pile

Check deflection at top of ground (124ft)

$$P_{normal\_op} := \frac{1}{2}\cdot K_a\cdot\gamma_{soil}\cdot h_{soil}^{2}\cdot w_{ps} + P_{live}\cdot K_a\cdot h_{soil}\cdot w_{ps} = 2.235\times 10^{4}\cdot lbf$$

$$M_{normal\_op} := \frac{1}{2}\cdot K_a\cdot\gamma_{soil}\cdot h_{soil}^{2}\cdot\frac{1}{3}\cdot h_{soil}\cdot w_{ps} + P_{live}\cdot K_a\cdot\frac{h_{soil}^{2}}{2}\cdot w_{ps} = 1.123\times 10^{6}\cdot lbf\cdot in$$

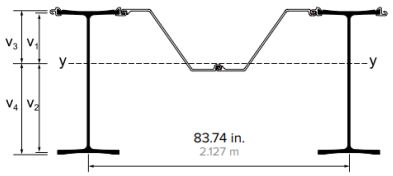
$$\Delta_{113} := 0.2in$$

$$\theta_{113} := 0.00187$$

$$\Delta_{124} := \Delta_{113} + \theta_{113}\cdot h_{soil} + \frac{P_{live}\cdot K_a\cdot w_{ps}}{8\cdot E_{steel}\cdot I_{ps}}\cdot h_{soil}^{4} + \frac{K_a\cdot\gamma_{soil}\cdot h_{soil}\cdot w_{ps}\cdot h_{soil}^{4}}{30\cdot\left(E_{steel}\cdot I_{ps}\right)} = 0.466\cdot in$$

Sheet tip is 97 pile tip is 72

Combination HZ...M-12 / AZ 18-800



SECTION	PROPERTIES PER FOOT OF WALL				MASS OF COMBINATION WITH INTERMEDIARY SECTION				
					*** AZ 18-800			Coating Area	
	Sectional Area	Moment of Inertia	*Elastic Section Modulus	**Elastic Section Modulus	ℓAZ = 60% ℓHZ	ℓAZ = 80% ℓHZ	ℓAZ = ℓHZ	Waterside	Landside
	in²/ft cm²/m	in⁴/ft cm⁴/m	in³/ft cm³/m	in³/ft cm³/m	lb/ft² kg/m²	lb/ft² kg/m²	lb/ft² kg/m²	ft²/ft m²/m	ft²/ft m²/m
HZ 630M	12.55 265.7	1,112.0 151,850	**83.0** **4,465**	93.0 5,000	35.13 172	38.93 190	42.72 209	8.73 2.660	15.11 4.605
HZ 880M A	11.97 253.4	1,641.1 224,100	**93.2** **5,010**	106.7 5,740	33.28 162	37.01 181	40.74 199	8.85 2.699	16.72 5.095
HZ 880M B	12.68 268.3	1,763.9 240,880	**100.7** **5,415**	113.5 6,100	35.69 174	39.42 192	43.14 211	8.86 2.702	16.72 5.097
HZ 880M C	13.00 275.1	1,847.3 252,260	**105.4** **5,665**	118.4 6,365	36.78 180	40.51 198	44.23 216	8.86 2.702	16.72 5.097
	13.69	2,964.2	**131.7**	147.7	39.12	42.86	46.59	8.84	18.15

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Appendix H. Electrical Calculations

Prepared by Control Point Engineering

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SERVICE LOAD CALCULATION

LOAD CENTER: *MSB*	480 VAC, 3 PHASE			4-WIRE
FED FROM: *XFMR*	**MCC**		**UTILITY**	**GENERATOR**
EQUIPMENT DESCRIPTION	**LOAD AMPS**	**LOAD (KVA)**	**LOAD (KVA)**	**LOAD (KVA)**
MCC MOTOR CONTROL CENTER	453.7	377.2	403.8	270.0
SUBTOTAL (KVA)		377.2	403.8	270.0
SUBTOTAL (AMPS)		453.7	485.7	324.7

UTILITY:		**GENERATOR:**	
SERVICE SIZE	454 A	**SERVICE SIZE**	325 A
MAIN BREAKER SIZE	600 A	**GEN SIZE** **450 kW**	541 A
% LOADED	76%	**% LOADED**	60%

LOAD CALCULATION

LOAD CENTER: MCC | **480 VAC, 3 PHASE SERVICE** | **3-WIRE**

FED FROM: MCC

EQUIPMENT NUMBER	HP KVA	MOTOR CONTROL	EQUIPMENT DESCRIPTION	CONNECTED LOAD (AMPS)	CONNECTED QTY	CONNECTED LOAD (KVA)	UTILITY LOAD QTY	UTILITY LOAD (KVA)	GENERATOR LOAD QTY	GENERATOR LOAD (KVA)
BLDV_CPLx_1410	5	CB	WEIR GATE MAIN CONTROL PANEL	7.6	1	6.3	1	6.3	1	6.3
BLDV_CPLx_2000	15	CB	FISH SCREEN MAIN CONTROL PANEL	21.0	1	17.5	1	17.5	1	17.5
BLDV_CPLx_3010	7.5	CB	DEBRIS RAKE MAIN CONTROL PANEL	11.0	1	9.1	1	9.1	1	9.1
BLDV_PUMP_6001	60	FVNR	WATER JET PUMP	77.0	1	64.0	1	64.0	0	0.0
BLDV_PUMP_6002	40	FVNR	SEDIMENT FLUSH PUMP	52.0	1	43.2	1	43.2	0	0.0
		CB	POWER PANEL 1	128.0	1	106.4	1	106.4	1	106.4
		CB	POWER PANEL 2	92.2	1	76.7	1	76.7	1	76.7
	9 kVA	CB	AC UNIT	10.8	1	9.0	1	9.0	1	9.0
	45 kVA	XFMR	LIGHTING PANEL LP9010	54.1	1	45.0	1	45.0	1	45.0
SUBTOTAL						377.2		377.2		270.0
LARGEST MOTOR @ 25%				60	HP			26.6		
TOTAL								403.8		270.0

3 PHASE CURRENT	486	AMPS
	100%	MAXIMUM LOAD CURRENT
CALCULATED SERVICE SIZE	486	AMPS
MAIN BREAKER SIZE	600	AMPS
% MAIN BREAKER LOAD	81%	

LOAD CALCULATION

LOAD CENTER: PP 1 | **480 VAC, 3 PHASE SERVICE** | **3-WIRE**

FED FROM: MCC

EQUIPMENT NUMBER	HP KVA	MOTOR CONTROL	EQUIPMENT DESCRIPTION	CONNECTED LOAD (AMPS)	CONNECTED QTY	CONNECTED LOAD (KVA)	UTILITY LOAD QTY	UTILITY LOAD (KVA)	GENERATOR LOAD QTY	GENERATOR LOAD (KVA)
BLDV_COMP_1111	10	CB	AIR COMPRESSOR	14.0	1	11.6	1	11.6	1	11.6
BLDV_COMP_1211	10	CB	AIR COMPRESSOR	14.0	1	11.6	1	11.6	1	11.6
BLDV_MOGx_ 1414	7.5	CB	FOREBAY SLUICE GATE	11.0	1	9.1	1	9.1	1	9.1
BLDV_MOGx_2101	7.5	CB	FISH SCREEN SHUTOFF GATE	11.0	1	9.1	1	9.1	1	9.1
BLDV_MOGx_2102	7.5	CB	FISH SCREEN SHUTOFF GATE	11.0	1	9.1	1	9.1	1	9.1
BLDV_MOGx_2103	15	CB	FISH SCREEN SHUTOFF GATE	21.0	1	17.5	1	17.5	1	17.5
BLDV_MOGx_2104	15	CB	FISH SCREEN SHUTOFF GATE	21.0	1	17.5	1	17.5	1	17.5
BLDV_MOGx_2105	7.5	CB	INTAKE CHANNEL SLUICE GATE	11.0	1	9.1	1	9.1	1	9.1
BLDV_MOGx_2106	10	CB	BYPASS FLOW GATE	14.0	1	11.6	1	11.6	1	11.6
SUBTOTAL						106.4		106.4		106.4
LARGEST MOTOR @ 25%				15	HP			4.4		
TOTAL								110.8		106.4

3 PHASE CURRENT	133	AMPS
	100%	MAXIMUM LOAD CURRENT
CALCULATED SERVICE SIZE	133	AMPS
MAIN BREAKER SIZE	200	AMPS
% MAIN BREAKER LOAD	67%	

LOAD CALCULATION

LOAD CENTER: PP 2 | **480 VAC, 3 PHASE SERVICE** | **3-WIRE**

FED FROM: MCC

EQUIPMENT NUMBER	HP KVA	MOTOR CONTROL	EQUIPMENT DESCRIPTION	CONNECTED LOAD (AMPS)	CONNECTED QTY	CONNECTED LOAD (KVA)	UTILITY LOAD QTY	UTILITY LOAD (KVA)	GENERATOR LOAD QTY	GENERATOR LOAD (KVA)
BLDV_MOGx_ 3021	15	CB	FISH LADDER GATE	21.0	1	17.5	1	17.5	1	17.5
BLDV_MOGx_ 4011	5	CB	CALAVERAS RIVER SUPPLY GATE	7.6	1	6.3	1	6.3	1	6.3
BLDV_MOGx_ 4012	5	CB	CALAVERAS RIVER SUPPLY GATE	7.6	1	6.3	1	6.3	1	6.3
BLDV_MOGx_ 4013	10	CB	SEWD WATER SUPPLY GATE	14.0	1	11.6	1	11.6	1	11.6
BLDV_MOGx_ 4014	10	CB	SEWD WATER SUPPLY GATE	14.0	1	11.6	1	11.6	1	11.6
BLDV_MOGx_ 4015	10	CB	SEWD WATER SUPPLY GATE	14.0	1	11.6	1	11.6	1	11.6
BLDV_MOGx_ 5011	10	CB	DIVERSION BOX SLUICE GATE	14.0	1	11.6	1	11.6	1	11.6
SUBTOTAL						76.7		76.7		76.7
LARGEST MOTOR @ 25%				15	HP			4.4		
TOTAL								81.0		76.7

3 PHASE CURRENT	97	AMPS
	100%	MAXIMUM LOAD CURRENT
CALCULATED SERVICE SIZE	97	AMPS
MAIN BREAKER SIZE	200	AMPS
% MAIN BREAKER LOAD	49%	

PANEL SCHEDULE

LOCATION CONTROL BLDG
VOLTAGE 120/208Y
TRIM FLUSH

MAIN BKR 200A
SYSTEM 3Φ, 4W
BUS RATING 225A

CKT	LOAD	VA	AMPS	CB	PHASE LOADS-AMPS A	B	C	CB	AMPS	VA	LOAD	CKT
1	LTG - CONTROL BLDG INT	1200	10.0	15/1	19.0			15/1	9.0	1080	CONTROL BLDG INT - RECEPT	2
3	LTG - CONTROL BLDG EXT	1200	10.0	15/1		13.0		15/1	3.0	360	CONTROL BLDG EXT - RECEPT	4
5	LTG - SITE 1	1200	10.0	15/1			13.0	15/1	3.0	360	SPARE	6
7	LTG - SITE 2	480	4.0	15/1	16.0			15/1	12.0	1440	SITE - RECEPT	8
9	LTG - AREA	750	6.3	15/1		18.3		15/1	12.0	1440	AREA - RECEPT	10
11	SPARE	0	0.0	15/1			20.0	30/1	20.0	2400	CONTROL PANEL	12
13	SPARE	0	0.0	15/1	0.0			15/1	0.0	0	SPARE	14
15	GENERATOR BATTERY CHARGER	800	6.7	15/1		6.7		15/1	0.0	0	SPARE	16
17	GENERATOR BLOCK HEATER	1375	11.5	15/2			11.5	30/1	0.0	0	AIR DRYER	18
19		1375	11.5		13.0			15/1	1.5	180	AIR DRYER ISOLATION VALVE NO. 1	20
21	SPARE	0	0.0	15/1		1.5		15/1	1.5	180	AIR DRYER ISOLATION VALVE NO. 2	22
23	SPARE	0	0.0	15/1			1.5	15/1	1.5	180	AIR DRYER BYPASS VALVE	24
25	SPARE	0	0.0	15/1	1.5			15/1	1.5	180	AIR FILTER ISOLATION VALVE NO. 1	26
27	SPARE	0	0.0	15/1		0.9		15/1	0.9	180	AIR FILTER ISOLATION VALVE NO. 2	28
29	SPARE	0	0.0	15/2			0.9	15/1	0.9	180	AIR FILTER BYPASS VALVE	30
31	SPARE	0	0.0	15/3	0.9			15/1	0.9	180	GATE SUPPLY VALVE NO.1	32
33	SPARE	0	0.0	15/4		0.9		15/1	0.9	180	GATE SUPPLY VALVE NO.2	34
35	SPARE	0	0.0	15/5			0.9	15/1	0.9	180	GATE SUPPLY VALVE NO.3	36
37			0.0		0.0				0.0			38
39			0.0			0.0			0.0			40
41			0.0				0.0		0.0			42

Notes		A	B	C
(L)	**Lockout Provision Required**	**50 A**	**41 A**	**48 A**
(H)	**HACR - Rated Breaker**	**6039 VA**	**4938 VA**	**5723 VA**
(GFI)	**Ground Fault Interupt Rated Breaker**	**PHASE LOADS**		
(AFI)	**Arc-Fault Interupt Rated Breaker**	**108%**	**89%**	**103%**

TOTAL	16699 VA	208 V
AMPS	46 A	