Appendix A

Draft Technical Memorandum Geotechnical Assessment

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Geotechnical Assessment

Fremont Weir Adult Fish Passage Modification Project

Yolo County, CA
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FREMONT WEIR ADULT FISH PASSAGE MODIFICATION PROJECT

GEOTECHNICAL ASSESSMENT

OCTOBER 31, 2016

1 INTRODUCTION

This technical memorandum is being provided to summarize geotechnical conclusions and recommendations in support of the 30 percent design level proposed adult fish passage improvements at the Fremont Weir in Yolo County, California. The Fremont Weir is located on the west bank of the Sacramento River and forms the entrance to the Yolo Bypass. The 1.8 mile long weir is located approximately 2 miles southwest of the City of Verona, California and is owned and operated by State of California Department of Water Resources (DWR). The Freemont Weir was constructed in 1924 to provide an overflow into the Yolo Basins during periods of high water in the Sacramento River and consists of a linear "L shaped" concrete structure setback from the riverbank. There is an existing fish passage structure located approximately 0.6 miles west of the weir and east bypass levee tie-in. The fish passage structure was constructed in 1966 and penetrates the vertical portion of the weir. The existing fish passage structure will be removed and replaced with the new facility. The new structure will penetrate the Freemont Weir.

The fish passage design is being advanced to a 30 percent design level by DWR. In coordinating with the structural and civil design team members, it is our understanding that the proposed improvements will consist of the following:

- An improved approach channel
- A short box culvert entrance channel including a sheet pile wingwall/cutoff installed along the face of the weir between the box culvert and the gate structure.
- A gate structure with a separate elevated structure located landward of the weir that houses the mechanical/electrical equipment to operate the gate.
- A transition structure
- An armored channel that extends from the concrete apron to an existing scour pond.

The flow line of the new structure will be located 4.25 feet lower than the weir splash pad. The combined length of the Box and the U-shape structures is currently proposed to be 32 feet (16 feet each) and a width of 19 feet (parallel to the existing Fremont Weir). The Box structure roof will be 1.5 feet thick with the intent to support maintenance vehicle traffic loads and the top surface will match the existing top of weir elevation of 32 feet[1]. The foundations for both the box and U-shaped concrete structures have been designed to be 3-foot thick concrete with deepened (down turned) perimeter

[1] Elevations reference North American Vertical Datum 1988 (NAVD88)

footings at the upstream and downstream ends. The deepened footings are proposed to extend 2 feet below the structure (elevation of 17 feet). Downstream of the U-shape gate housing will be an open channel reinforced concrete transition that will be 19 feet in length and ties into the downstream armored channel. A radial gate is proposed to control flow through the structure with electrical and mechanical controls located on an elevated structure located on the downstream side and 100 feet laterally offset from the weir. The elevated structure is proposed to be supported by four columns founded on spread footings.

1.1 CRITERIA

This geotechnical assessment primarily focused on seepage and settlement. USACE levee design criteria was used to assess geotechnical performance of the fish passage structure. Table 1 summarizes the criteria adopted for the current geotechnical assessments. The criteria is based on Engineer Manual (EM) 1110-2-1913 (USACE, 2000), Engineer Technical Letter (ETL) 1110-2-569 (USACE, 2005), and USACE Sacramento District Levee Practice Group Geotechnical Guidance (2008).

The fish passage structure and much of the Fremont Weir is underlain by potentially liquefiable soils that could lose strength and settle when subjected to strong ground shaking. Evaluation of the project site seismic response and seismic design of the elevation of structure and the Fremont Weir is beyond the scope of these studies. For 30 percent design the existing weir was not evaluated for through-seepage or stability with the inclusion of the sheetpile wingwall/cutoff along the riverside face of the weir.

Table 1. Summary of Evaluation Criteria

Analysis	Applicable Evaluation Criteria	Required Value
Seepage Analysis	Exit Gradient at Toe of Levee	$i_{ave} \leq 0.5$ (with FS[1] ≥ 1.6 for $\gamma_{blanket} \leq 112$ pcf)
	Through-Seepage	Phreatic surface should not exit landside slope for cohesionless soils
Uplift	Steady-State	FS ≥ 1.4 (DWSE)
Settlement	Maintain Top of Box/Weir Elevation	Minimum 2-inches above current weir elevation be maintained.

1. FS - Factor of Safety

2 Site Characterization

2.1 SUBSURFACE CONDITIONS

Limited historical information is available for developing geotechnical site characterization for the fish passage project. Historical information is limited to explorations conducted by USACE along the east and west Yolo Bypass levee. For the purpose of this feasibility design phase, DWR conducted two explorations to a depth of up to 60 feet to better characterize near surface foundation conditions. Borings FW-DH-2 and FW-DH-3 were advanced using hollow stem augers between April 19 and April 20,

2016. These borings form the basis for these geotechnical evaluations. The DWR borings are included as an attachment to this memorandum.

The two explorations were advanced between 51 ½ and 60 feet below the existing ground surface. Samples were taken continuously to depths using Standard Penetration Test (SPT), soil cores, and Shelby Tubes. SPT samples were advanced 18-inches using a 140 pound hammer with a free-fall drop of 30-inches. Corrected blows per foot were presented on the DWR boring logs along with the percent recovery of each sample. Shelby tubes were advanced hydraulically using the weight of the drill rig as a reaction force.

Exploration logs indicate that the subsurface conditions generally consist of the following:

- Lean Clay (CL) was recorded near the ground surface and to depths between 12 and 14.5 feet below the existing ground surface (bgs);
- Silty Sand (SM) and Clayey Sand (SC) was recorded below the CL materials to depths between 17.5 and 19 feet bgs;
- Poorly graded Sand (SP) was recorded below the SM and SC materials to depths between 41 and 42.5 feet bgs;
- Layers of interbedded CL, SC, Silt (MH), and SP materials to the depths explored.
- Groundwater readings were recorded during drilling between 9 and 11.4 feet bgs.

2.2 SELECTED SOIL PARAMETERS

For preliminary design purposes, HDR developed a representative soil profile for the foundation soils and assigned soil seepage and strength parameters using the information provided with the two DWR explorations and previous experience with similar Sacramento River deposits in the region. The material properties assigned for each of the materials are provided in Table 2.

Table 2. Soil and Rock Seepage and Strength Parameters

Layer Number	Layer Name	k_v [1]	k_h [1]	Total Unit Weight	Drained Parameters		Undrained Parameters	
					c’	φ’	c	φ
		(CM/SEC)	(CM/SEC)	(PCF)[3]	(PSF)[3]	(DEG)	(PSF)	(DEG)
1	Weir Lean Clay	1.0E-6	4.0E-6	125	0	28	400	0
2	Foundation Silty Sand	4.0E-5	1.6E-4	125	0	28	0	28
3	Foundation Poorly Graded Sand	2.0E-2	2.0E-2	125	0	32	0	32
4	Foundation Fine Grain Materials	1.0E-6	4.0E-6	130	0	30	1,000	0

1 Hydraulic conductivities based upon Goodbye, Hazen; Hello, Kozeny-Carman, W. David Carrier III, Journal of Geotechnical and Environmental Engineering, ASCE, 2003

2 Based on Duncan, Wright, and Brandon (2014) the Soil Strength and Slope Stability 2nd Ed and Laboratory testing

3 PCF –pounds per cubic foot; PSF – pounds per square foot

Ranges of total unit weights were assigned based on SPT blow counts and laboratory results. Furthermore, the strength parameters are based on the laboratory testing program, SPT blow counts, and performance of the current system. HDR used the lower bounds of the plotted case histories to develop the parameters in Table 2.

3 Preliminary Analysis

3.1 SEEPAGE & UPLIFT

Hydraulic modeling by DWR set the Design Water Surface Elevation (DWSE) at elevation 33 feet as documented in HDRs Technical Memorandum entitled Fremont Weir Adult Fish Passage Modification Project, Gate Operations Scenarios. This is 1-foot above the top of weir (elevation of 32 feet) and was used for underseepage analyses described below.

The weir is underlain by a permeable aquifer that is assumed to have direct connectivity to the Sacramento River. The proposed fish passage invert will be set at elevation 22 feet, which is 10 feet below the existing top of the weir and will penetrate the underlying fine grained blanket soils and bear on the underlying sand aquifer. Therefore, elevated river levels will induce elevated seepage gradients along the downstream edge of the U-shaped structure. High unbalanced uplift forces will also develop along the base of the foundation and will be greatest when the fish ladder gate is closed and the water surface just reaches the top of the weir. In these analyses it is also assumed that the bottom slab-on-grade will be a minimum of 3 feet thick to provide mass to resist uplift.

Two scenarios were analyzed for underseepage and associated uplift pressures.

1. No Sheet Pile Cutoff
2. A hanging sheet pile cutoff that extended 15-feet below the fish passage invert elevation of 22 feet.

The results of the uplift analyses are presented in terms of factor of safety against uplift. Uplift pressures associated with no seepage cutoff (scenario 1) results in uplift pressures along the foundation of the box culvert with a factor of safety less than 1.4 against uplift, not meeting the project uplift criteria. The inclusion of a hanging sheet pile (scenario 2) resulted in unbalanced uplift pressures on the order of 312 psf along the riverside of the foundations and 78 psf along the downstream edge. Given the proposed structural plans and these uplift pressures, the calculated factor of safety is 1.5, meeting the project design criteria.

3.2 SETTLEMENT

Settlement of the structures was assessed using the limited information presented on the DWR explorations. Specifically, correct SPT blow counts were used in calculations following methods presented by Terzaghi and Peck (1967) to determine bearing capacities associated with 1-inch total settlement and 2-inch differential settlement in 40 feet. The calculations resulted in an allowable bearing capacity of 2,300 pounds per square foot (psf) within the sand foundation materials.

4 RECOMMENDATIONS

Based on the results from analyses summarized above, the proposed project is feasible from a geotechnical standpoint so long as the recommendations presented herein are incorporated into the project preliminary design.

4.1 SEEPAGE & UPLIFT

The structures will be subjected to elevated uplift pressures when the river elevation reaches the top of the weir and the gate is closed.

To reduce the potential for structure uplift, the following measures should be incorporated into the project:

- Sheet pile cutoff extending to a depth of 15 feet below the passage invert elevation.
- Slab-on-grade foundation should be a minimum of 3-feet thick for the gate structure and 1.5-feet thick for the splash pad.

4.2 SEEPAGE

Underseepage analyses indicate that with a project DSWE of 33 feet, the concrete structures will likely experience uplift. Based on the calculated uplift pressures, a hanging sheet pile wall is recommended at the upstream face of the weir to meet the project uplift design criteria. The sheet pile tip elevations should extend 15 feet below the passage invert elevation (invert elevation of 22 feet). The Box structure should be structurally connected to the sheet piles to prevent end around seepage occurring between the structures.

Underseepage is anticipated to result in high seepage flows through and into the bottom of channel in the outlet channel. Channel armoring has been sized by the HDR hydraulic team to be a uniform 12-inch rip rap. A weighted filter section on the order of 12-inches thick is recommended to be placed between the rip rap and the native foundation materials to protect the native foundation materials from piping losses due to underseepage. The weighted filter should consist of two filter layer: a 6 inches graded filter materials overlain by a 6-inch protective layer of drain rock. For the purposes for 30 percent design the graded filter materials should consist of clean-well-graded sand free of clay materials, organic material, or other deleterious substances, and shall be of such size that 90 to 100 percent will pass a No. 4 sieve and not more than 5 percent will pass a No. 200 sieve, such as am ASTM C-33 concrete Sand. Drain rock should be in conformance with Caltrans Section 19. Section 19 outlines pervious Class 2 rock (drain rock) materials as crushed gravel with a minimum of two broken faces; durable; as well as free from clay lumps, friable particles, organic matter and foreign materials.

4.3 FOUNDATIONS

As previously discussed, bearing capacities for spread footing foundations were assessed assuming 1-inch total settlement and 2-inch differential settlement in 40 feet. Bearing pressures were developed based on the conditions presented in the DWR explorations assuming that the bottom of footings will be founded at elevations between 22 and 26 feet and foundations bear on undisturbed sand in the underlying aquifer. To limit settlement allowable bearing pressure of 2,300 psf for dead plus live loads is recommended. Bearing pressures may be increased by one-third for total loads, including wind and/or seismic loads. The bottom of footings should be at least 3-feet of embedment relative to the adjacent grades and into medium dense native sand or stiff clay.

Lateral loads on footings can be resisted by a combination of passive resistance acting against the vertical faces of the footings and friction along the bases of the footings. Passive resistance may be calculated using lateral pressures described in the following section. A coefficient of friction of 0.35 should be used in the determination of friction along the base of footings. This coefficient applies to footings constructed of reinforced concrete placed neat against cleaned excavation into native materials.

4.4 LATERAL PRESSURES

Active and passive lateral pressures should be applied to the design of the sheet pile cross section and to the lateral resistance of spread footings. As discussed in Section 4.2, Sheet piles are proposed to be installed along the riverside of the proposed fish ladder. The elevated power structure will be supported by spread footings foundations.

The following equivalent active pressure should be determined by $Pa = 0.5\gamma_{ef}H^2$

Where active $\gamma_{ef} = 45$ pcf

The following equivalent passive pressure should be determined by $Pp = 0.5\gamma_{ef}H^2$

Where passive $\gamma_{ef} = 340$ pcf

4.5 CONSTRUCTION RECOMMENDATIONS

The construction of the proposed improvements are anticipated to require demolition of a portion of the existing weir and general earthwork. Demolition of the existing weir section would include the cutting of the unreinforced concrete weir armoring and excavation of the soils within the weir within the limits of the Box and U shaped gate structure excavations. Also, earthwork is anticipated to be required for the splash pad, the channel construction and the elevated power structure.

Excavations through the weir may be performed by laying back the weir soils at 2:1 (horizontal:vertical) or maybe shored in conformance with CalOSHA requirements. Backfill of the weir with soils similar to the existing weir soils is required. Excavated weir materials are considered suitable reuse as backfill so long as the material is kept free of debris. Backfill materials should be moisture conditioned to be between 1 and 3 percent of optimum moisture content prior to placement where optimum moisture content is determined by ASTM D 1557. Backfills should be placed in 4 to 6 inch loose lifts for compaction effort and compacted to 90 percent relative to ASTM D 1557. Light weight equipment should be used in the compaction effort in order to reduce lateral pressures against the existing unreinforced concrete armoring and the proposed structures.

Materials associated with the open channel armoring filter should be placed in controlled lifts and compacted using vibrating equipment.

5 LIMITATIONS

This memorandum is based on 30 percent design plans and structural information and limited geotechnical information provided by DWR. Supplemental geotechnical information is anticipated to be acquired as the project design progresses. Characterization of subsurface conditions was carried out only at the locations where the explorations or tests are performed; actual subsurface conditions between explorations or tests may be different than those described in this report. Variations of subsurface conditions from those analyzed or characterized in this report are not uncommon and may become evident during any future construction. In addition, changes in the condition of the site can occur over time as a result of either natural processes (such as earthquakes, flooding, or changes in ground water levels) or human activity (such as construction adjacent to the site, dumping of fill, or excavating). If changes to the site's surface or subsurface conditions occur since the performance of the field work described in this report, or if differing subsurface conditions are encountered, HDR should be contacted immediately to evaluate the differing conditions to assess if the opinions, conclusions, and recommendations provided in this report are still applicable or should be amended.

This report has been prepared in accordance with generally accepted geotechnical engineering practices for the exclusive use of DWR and their consultants for the 30 percent design of the Fremont Weir Adult Fish Passage improvements. The conclusions and recommendations contained in this report are solely professional opinions.

In the event that there are any changes in the nature, design or location of the project, as described in this report, or if any future additions or expansions are planned, the conclusions and recommendations contained in this report shall not be considered valid unless we are contacted in writing, the project changes are reviewed by us, and the conclusions and recommendations presented in this report are modified or verified in writing.