Appendix D

# Hydraulic Modeling Report

AYRES ASSOCIATES

# MEMORANDUM

To: Gary Nuss, CH2MHill; cc: Russ Stepp, FRWA

From: Thomas W. Smith, P.E., G.E.

Date: March 28, 2003

Re: Two-Dimensional Hydraulic Analysis at the Proposed Freeport Water Intake Structure

## Introduction

This memorandum summarizes the findings of a two-dimensional hydraulic analysis for the proposed Freeport water intake structure on the Sacramento River. The proposed structure is located upstream from the town of Freeport near River Mile 47.6. The location of this project is shown in **Figure 1**. The purpose of the analysis was to determine if the proposed project would have any detrimental effects on water surface elevation, velocity, sedimentation, or scour on the Sacramento River Flood Control Project (SRFCP) for two high flow scenarios.

This analysis models two flood flow events for an existing river configuration and a proposed structure configuration to identify differences in river hydraulics between the two configurations. The results of this analysis are from an uncalibrated two-dimensional model, however the roughness coefficients used in this model come from a calibrated model in a similar reach of the Sacramento River. Therefore, the results should be interpreted as initial findings. Further modeling may be needed and a calibrated model should be used for final design. Additional runs may also be needed to analyze other project configurations or for the analysis of other flow events, such as those pertinent to fish screen sweep velocity.

This study effort was requested by Freeport Regional Water Authority (FRWA) to support permit applications from the State Reclamation Board for this project. The work was contracted to Ayres Associates through CH2MHill.

## Hydraulic Analysis

The river hydraulics were modeled using the RMA-2V steady state two-dimensional computer simulation with the Surface-Water Modeling System (SMS) graphical interface. The riverbed and bank geometry configuration for the two-dimensional model was developed using existing bathymetric and overbank mapping of the Sacramento River, surveyed by Ayres Associates in 1997 and 2002.

Two flood flow events were simulated for both the existing and proposed configurations. These events included the original design flow and a worst case flood event, with discharges of 111,100 cfs and 150,100 cfs, respectively. The design discharge used for this model came from a modern one-dimensional UNET model of the flood control system and is slightly higher than the 1957 Corps stated design flow of 110,000 cfs. The worst case flood scenario is the hypothetical maximum flow that can reach this point in the river without overtopping the upstream levees.

The hydraulic model for this study extends from River Mile (RM) 51 at the upstream end, to RM 46.5 at the downstream end, as per the U.S. Army Corps of Engineers (Corps) 1997 River Miles. The limits of the model are shown in Figure 2. The boundary conditions were obtained from MBK Engineers' updated version of the U.S. Army Corps of Engineers' Infinite Channel UNET model of the Sacramento River developed for Sacramento Area Flood Control Agency (SAFCA).

At the upstream limit of the model, a discharge of 111,100 cfs was used for the design flow and 150,100 cfs was used for the worst case flood scenario. At the downstream limit of the model, a stage of 22.21 (NGVD) was used for the design flow, and a stage of 28.37 (NGVD) was used for the worst case event. The location and configuration of the proposed structure was provided by CH2MHill and is the preferred option at the time of this report. A portion of the finite element mesh, in the area of the project location, is shown in Figure 2.

The model used the channel roughness parameters from a calibrated two-dimensional model approximately 10 miles upstream on the Sacramento River, which was calibrated to known water surfaces, as well as high water marks from 1997. A summary of the roughness parameters used in both models is provided below in **Table 1**.

**Table 1. Manning's Roughness Coefficients**

| Landscape Description | Manning's Roughness Coefficients |
|---|---|
| Riverbed | 0.030 |
| Marina | 0.042 |
| Levee | 0.033 |
| Grass | 0.035 |
| Sparse Trees | 0.090 |
| Dense Trees | 0.130 |
| Houses/Buildings | 0.200 |

## River Hydraulics Results

The existing conditions water surface elevation at the location of the proposed structure is approximately 23.0 ft for the design flow and 29.3 ft for the worst case flow. The project configuration produces only local disturbances in the water surface, which do not propagate upstream to affect the backwater profile of the river. These local disturbances are shown in **Figure 3** and **Figure 4**, for the two modeled flows and discussed below:

- The design flow of 111,100 cfs (Figure 3) shows a maximum decrease in water surface elevation of 0.3 ft on the face of the structure and a maximum increase in water surface elevation of less than 0.2 ft on the downstream side of the structure.
- The worst case flow of 150,000 cfs (Figure 4) shows a maximum decrease in water surface elevation of 0.4 ft on the face of the structure and a maximum increase in water surface elevation of less than 0.2 ft on the downstream side of the structure.

The design flow channel velocities for existing conditions and for the proposed project conditions are shown in **Figure 5** and **Figure 6**, respectively. The channel velocities for the worst case scenario for existing conditions and with project conditions are shown in **Figure 8** and **Figure 9**, respectively. On the aforementioned figures, the arrows indicate the flow direction, but not the magnitude. The velocity differentials between existing and with project conditions are shown in **Figure 7** for design flow and **Figure 10** for the worst case flow.

The velocity effects from the proposed Freeport Intake Structure are similar for the two modeled flows and are discussed below:

- An increase in velocity occurs along the face of the structure and extends outward toward the middle of the channel for both modeled flows. The velocity increases in this area are up to 1.4 ft/s for the 111,100 cfs flow and 1.7 ft/s for the 150,100 cfs flow.
- The greatest increase in velocity is on the downstream corner of the proposed structure.
- The increased velocity along the face of the structure will likely produce bed scour, especially on the downstream corner where the velocity increase is greatest.
- Increases in velocity occur along the channel bank, downstream of the proposed structure. These increases are up to 0.7 ft/s for the design flow and 0.5 ft/s for worst case flow.
- Velocity decreases occur at the upstream and downstream ends of the proposed structure. The maximum decreases are located on the upstream end of the structure and are –2.3 ft/s for the design flow and –3.0 ft/s for the worst case flow.
- For both modeled flows, the localized flow pattern changed due to the formation of eddies on the upstream and downstream ends of the proposed structure.

## Discussion of Erosion, Scour and Sedimentation

The higher velocities along the face of the proposed structure will likely produce new erosion and scour unless counter measures are provided. Soil boring information is not available at this time and we have assumed that the bed and bank materials are fine grained sands and silts, which are highly susceptible to erosion.

Eddies form at the upstream and downstream ends of the proposed structure. The decrease in velocity caused by these eddies may result in the deposition of sediment immediately upstream and downstream of the structure. Some of these sediments may enter the fish screens and intake conduit.

The effects of the increased velocity along the bank downstream of the proposed structure are more difficult to predict, although there is the potential for erosion to occur. **Figure 11** shows an overall view of the riverbank in the project area. The riverbank and levee have very little vegetation, other than grasses and a few trees. Also, there is no remnant floodplain berm at the toe of the levee slope in the area of increased bank velocity (hatched area in Figure 11). The existing bank protection throughout this reach consists of cobbles installed by the Corps of Engineers in 1953. No further details about the design, such as layer thickness or type of toe trench, are known. Based upon our visual inspection of the above water portion of this reach, we would rate the overall condition of the existing armor layer as fair.

The following two photographs are typical depictions for this reach of the river. **Photograph 1** shows a storm drain outfall, located about two-thirds of the way through the area of increased velocity. **Photograph 2** is just upstream of the outfall structure. Cobbles on a grassed slope are clearly visible as well as deposition and erosion patterns near the waterline. No information is available for the bank condition below the waterline. The water level shown in these photographs (February 17, 2003) is higher than the summer low flow shown in the background aerial (August 1998) of the figures. A review of Figure 11 shows that much of the area of increased velocity is below the summer water level.

**Figure 12** displays river cross sections (looking downstream) through the area of increased velocity. These cross sections show the river immediately adjacent to the levee, with no waterside berm remaining through the area of higher velocities. Any additional erosion of the bank will encroach into the levee cross section. Since much of the bank area where velocities are increased is below the water line, no definitive statement can be made as to whether or not the existing revetment is able to handle the increased velocity without any impact.

## Conclusions

Based upon hydraulic modeling of the proposed water tower structure, we offer the following conclusions:

1. The proposed structure will have only a small and very localized (less than 0.2 ft) increase on water surface for the design flow and worst case flow scenarios.
2. Localized increases in velocity (0.5 ft/s to 0.7 ft/s) may increase the risk of lower bank erosion downstream of the proposed structure.
3. Localized decreases in velocity immediately upstream and downstream of the proposed structure will promote deposition. Some transport of suspended sediment into the pumps should be expected.
4. The increase in velocity along the face of the proposed structure will generate bed scour.

## Recommendations

Based upon our conclusions, we offer the following recommendations:

1. The hydraulic model used for the final design should be calibrated to ensure additional quality control and an additional level of accuracy.
2. Some additional evaluation of the condition of the existing revetment below the water line should be made to determine if this reach can handle the small increase in velocity without an impact.
3. The effects of the local deposition and subsequent mobilization may have an effect on the quality of the pumped water and should be considered in the final design
4. A detailed scour analysis should be performed (based on velocities from the final hydraulic model) to determine the maximum potential scour depth along the face of the proposed structure.