APPENDIX M

Hydrodynamic Analyses of 2-Gates Flood Stage Issues

MOFFATT & NICHOL

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MEMORANDUM

To:	**Dennis Majors, Metropolitan Water District**
From:	**Chris Potter, Rick Rhoads, Dilip Trivedi**
Date:	**December 2, 2008**
Subject:	**Flooding Issues Cover-letter**
M&N Job No.:	**2-Gate Barrier Project** **File No: 6097-02**

Based on comments received during the permitting process in regards to the issue of the potential for flooding related to the construction of the 2-gate barrier systems in Old River and Connection Slough, this package presents four technical memorandums summarizing the results of already completed analyses addressing flooding issues.

An initial hydraulic review was completed in July of 2008, which analyzed the flood neutrality of the 2-gate barriers using a simplistic HEC-RAS model of the roughly 1600-ft reach of Old River where the proposed barrier will be located. This preliminary study showed only a negligible impact on flood stage due to the barrier, less than 0.15-ft.

A refined hydraulic analysis was performed in November 2008 to include a sensitivity analysis of the roughness, expansion/contraction, and weir coefficient parameters used in the HEC-RAS model. Based on this additional HEC-RAS modeling, which used the 100-year tide and the 100-year discharge as the downstream and upstream boundary conditions, the worst case increase in flood stage in Old River due to the barrier was still on the order of 0.1-ft to 0.2-ft, the variation being due to the uncertainty in the selection of roughness for the project reach. Both of these HEC-RAS modeling efforts were performed to assist in defining the barrier geometry and to help quantify the potential for impact to flood-stage. However, these studies were rough estimates using an uncalibrated hydraulic model.

The DSM-2, a calibrated hydrodynamic model developed by the Department of Water Resources, was then run by the Contra Costa Water District (CCWD) to verify the hydraulic conditions of Old River with the gate system in place and to assess potential impact. The benefit of using this verified model is that it takes into account the dynamic nature of both the tides and the re-distribution of flood flows within the Delta. Technical memos were created by both the CCWD and Moffatt & Nichol (Supplemental Study of Flood Issues) to summarize the analysis of the DSM-2 modeling results and the comparison of these results with the existing condition, no-gates scenario results.

The DSM-2 modeling confirmed that the 2-gate barriers do not have a significant affect on flood stage within Old River and Connection Slough.

<u>Attachments</u>: Initial Hydraulic Review Memo (M&N)
Refined Hydraulic Review Memo (M&N)
Supplemental Study of Flood Issues Memo (M&N)
DSM2 Flood Analysis for Barrier Project Technical Memo (CCWD)

MOFFATT & NICHOL

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MEMORANDUM

To: **Dennis Majors, Metropolitan Water District**

From: **Chris Potter, Rick Rhoads, Dilip Trivedi**

Date: **November 26, 2008**

Subject: **Initial Hydraulic Review Draft**

M&N Job No.: **2-Gate Barrier Project**
File No: 6097-02

Purpose

This memorandum describes an analysis performed in July 2008 during the conceptual design phase of the project. The objective was to develop the geometry of the gate structure (crest elevation, and width and depth of opening), and to assess at a conceptual level the impacts to flood stage as a result of the structure.

Design Criteria

The Design Criteria for the Bacon Island 2-Gate Barrier System consists of two requirements:

- The system should maintain near Flood Neutrality during the gates-open condition. Flood neutrality was defined as no greater than 0.1-ft increase in flood stage for the 100-yr flow, and flood events less than 100-yr event should not exceed the 100-yr flood stage.
- Recognizing that under a gates-closed scenario a full tidal range could act on 1 side of the structure only, the design operating head differential on the gate was set at the diurnal range in the area, which is about 3.5 feet. This differential is being used to design the structural gate system.

Initial Analyses

The initial analysis is based on a review of bathymetric survey data for the site and available gage data collected by the USGS on the Old River.

During the initial planning phase of the project, the crest elevation of the rock sill/gate structure was set at 7 ft, NAVD. For this analysis, exceedance probabilities for high tides greater than MHHW were analyzed using 21 years of NOAA predicted tide data for the Old River at Orwood station, as presented in **Figure 1**. Based on this analysis, a high tide of 7-ft was determined to have an exceedance probability of 0.003%, while a high tide of 6.6-ft has an exceedance probability of 0.4%. The MHHW elevation of 6.1-ft has an exceedance probability of 3.7%. Based on this analysis, a barrier crest height of 6.6-ft NAVD88 was recommended as being adequate.

Initial HEC-RAS hydraulic simulations were performed to determine a potential range of head differentials resulting from the gates-open condition, using multiple flow-rates of the 100-yr

Flood Stage, MHHW, and MLLW as downstream boundary conditions. Since the objective was to develop structure geometry only, this set of simulations was performed by constricting the cross section at the gate location, rather than using an in-line structure (which is more appropriate for backwater calculations). Tables 1, 2, and 3 present the resultant head differentials and flow velocities for the simulations with downstream boundary conditions of 100-yr Flood Stage, MHHW and MLLW, respectively. It should be noted that there is the potential for high velocities through the open gate when there is a head differential greater than roughly 0.2-ft.

Table 1. Flow Velocities through the Gate with a 100-yr downstream water surface

Discharge (cfs)	Downstream Boundary Condition (ft NAVD88)	Upstream WSEL (ft NAVD88)	Head Differential (ft)	Velocity through Gate (ft/s)	Velocity 30-ft Upstream of Gate (ft/s)	Existing Condition Velocity (ft/s)
500	9.7	9.7	0	0.08	0.03	0.03
1,000	9.7	9.7	0	0.17	0.05	0.05
5,000	9.7	9.71	0.01	0.85	0.26	0.26
10,000	9.7	9.72	0.02	1.71	0.52	0.52
15,000	**9.7**	**9.75**	**0.05**	**2.57**	**0.79**	**0.79**
20,000	**9.7**	**9.79**	**0.09**	**3.46**	**1.05**	**1.05**
25,000	**9.7**	**9.85**	**0.15**	**4.36**	**1.30**	**1.31**

Table 2. Flow Velocities through the Gate with a MHHW downstream water surface

Discharge (cfs)	Downstream Boundary Condition (ft NAVD88)	Upstream WSEL (ft NAVD88)	Head Differential (ft)	Velocity through Gate (ft/s)	Velocity 30-ft Upstream of Gate (ft/s)	Existing Condition Velocity (ft/s)
500	6.1	6.1	0	0.15	0.03	0.03
1,000	**6.1**	**6.1**	**0**	**0.31**	**0.06**	**0.06**
5,000	**6.1**	**6.12**	**0.02**	**1.54**	**0.31**	**0.31**
10,000	**6.1**	**6.16**	**0.06**	**3.10**	**0.62**	**0.62**
15,000	**6.1**	**6.25**	**0.15**	**4.68**	**0.93**	**0.93**
20,000	6.1	6.36	0.26	6.30	1.23	1.24
25,000	6.1	6.52	0.42	7.98	1.52	1.55

Table 3. Flow Velocities through the Gate with a MLLW downstream water surface

Discharge (cfs)	Downstream Boundary Condition (ft NAVD88)	Upstream WSEL (ft NAVD88)	Head Differential (ft)	Velocity through Gate (ft/s)	Velocity 30-ft Upstream of Gate (ft/s)	Existing Condition Velocity (ft/s)
500	**2.4**	**2.4**	**0**	**0.19**	**0.04**	**0.04**
1,000	**2.4**	**2.4**	**0**	**0.38**	**0.08**	**0.08**
5,000	**2.4**	**2.43**	**0.03**	**1.91**	**0.38**	**0.38**
10,000	2.4	2.5	0.10	3.86	0.75	0.76
15,000	2.4	2.64	0.24	5.87	1.12	1.14
20,000	2.4	2.84	0.44	7.98	1.48	1.52
25,000	2.4	3.13	0.73	10.28	1.82	1.89

Tables 1 through 3 present a range of potential flow and stage conditions, however not all of the combinations represent likely conditions. The higher 100-yr flood stage in Table 1 would occur coincident with larger flows (15,000-cfs to 25,000-cfs), whereas the lower MLLW stage in Table 3 would occur coincident with the lower (500-cfs to 5,000-cfs range) range. In other words, a high flow would "mask" the tidal influence and result in a stage higher than a tide-only stage. Conversely, a low stage of MLLW would imply that the flow cannot be very high. Based on these preliminary results, the peak increase in flood stage is on the order of 0.15-ft during a 100-yr flood stage downstream and a 100-yr flow of 25,000-cfs coming down Old River.

Also, it should be noted that the head differential values greater than 0.15 ft in Tables 2 and 3 are for stages lower than 100-yr flood stage. This occurs because the notch influences flows at lower stages.

To check the validity of these results, simultaneous stage and flow data for Old River for an extreme event that occurred on Jan 5, 1997 were obtained. The flow for this event was estimated to be 17,000 cfs, and the stage was recorded to be 7.5 ft, NAVD. A flow of 17,000-cfs represents about a 20-yr return period as shown on **Figure 2** (flow at slack tides). Over the 15-years of USGS gage data, a flow of 17,000-cfs has an exceedance probability of about 0.5%, or roughly 21 hours per year as shown on **Figure 3**.

Based on a HEC-RAS simulation using the above boundary conditions (17,000-cfs flow and 7.5-ft downstream stage), the water surface elevation upstream of the structure was estimated to be 7.68-ft, which constitutes a head difference of 0.18 ft as shown in **Figure 4** below. This agrees well with Table 2 (closest combination is 15,000-cfs and MHHW stage, which yields a head difference of 0.15 ft).

The most likely head-differential for different flows can thus be interpolated using Tables 1 through 3.

This initial study indicates a potential for an increase in flood stage that is slightly greater than 0.1-ft (up to 0.15 ft, per Table 1). Since this analysis used a very simplistic approach of a geometric change in the cross section to represent the structure, additional analysis is needed to verify the flood stage impacts on Old River with the gate system in place.

[The additional analysis is presented as the Refined Hydraulic Review]

Figure 1. Tidal Exceedance Probabilities for Old River at Orwood

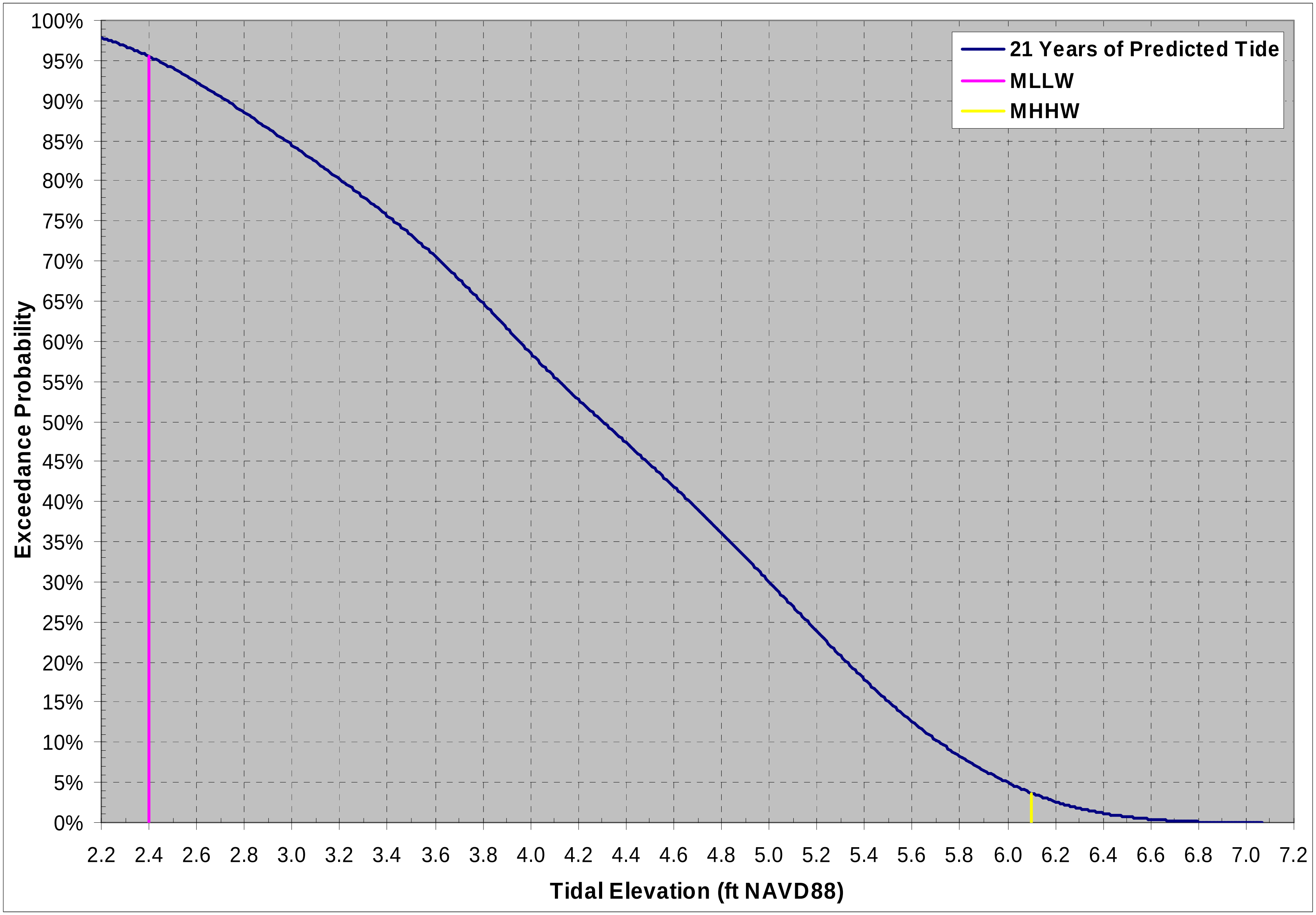


Figure 2. Peak-Flow Recurrence Intervals for Old River at Bacon Island

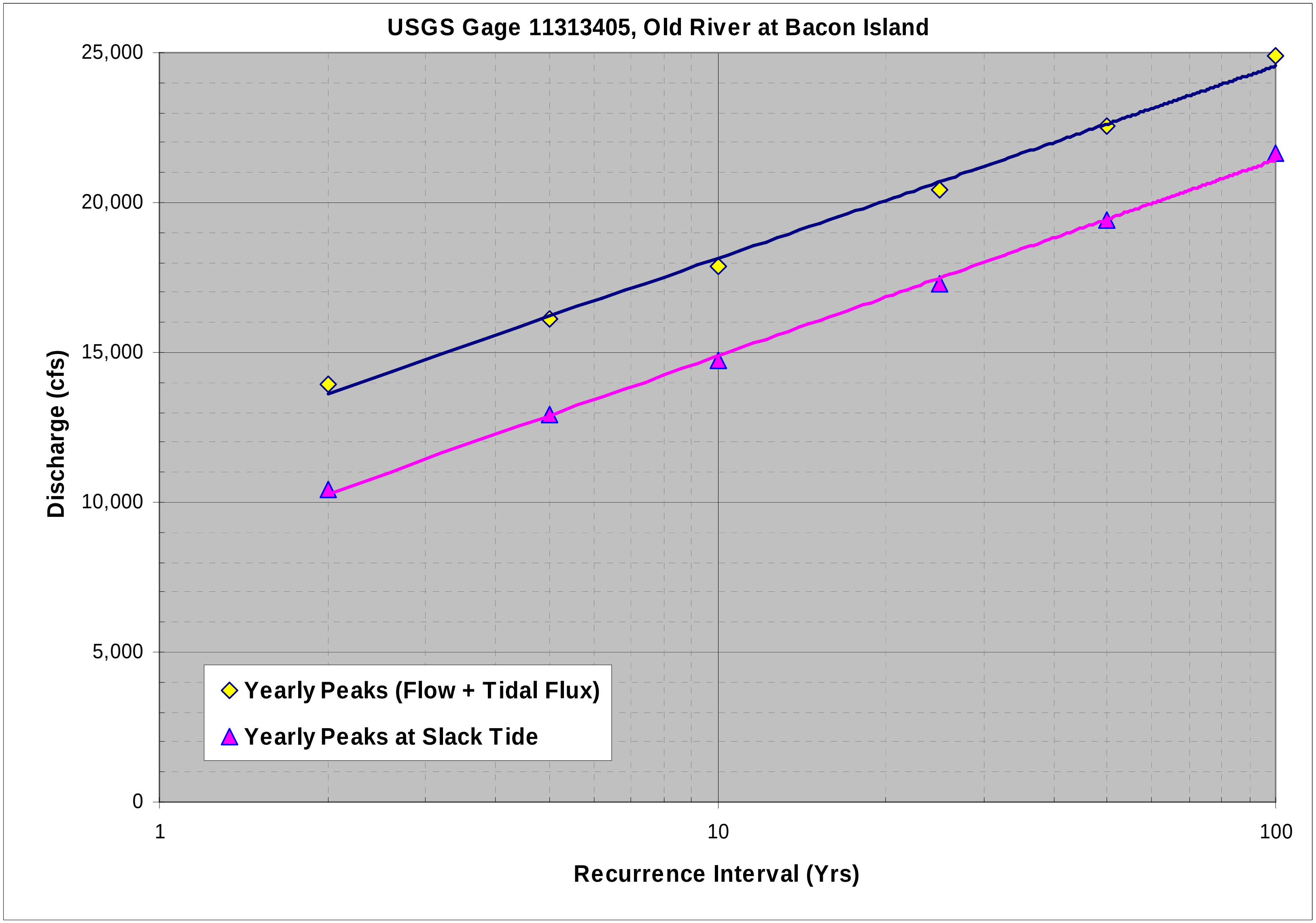


Figure 3. Discharge Exceedance Probabilities for Old River

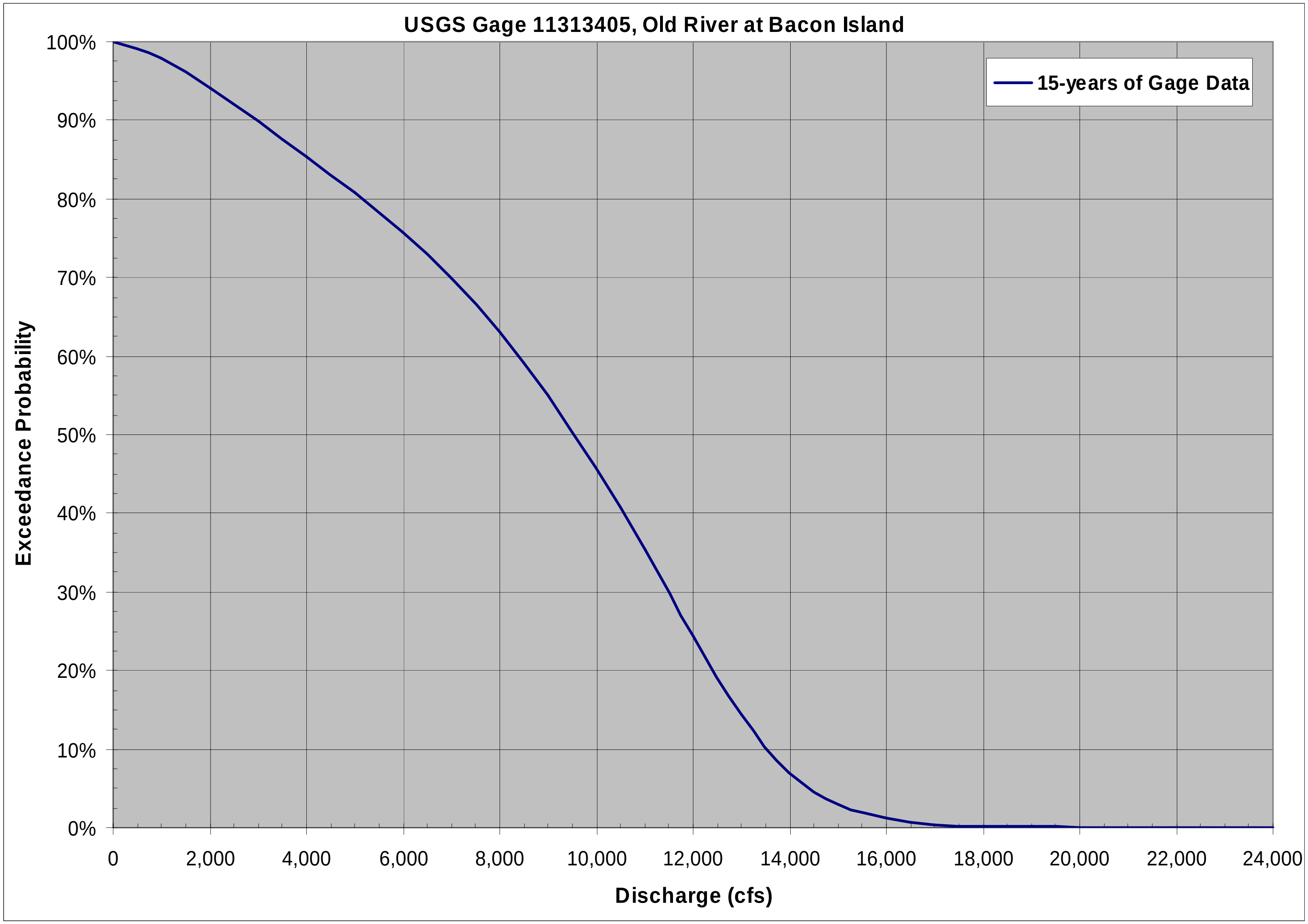
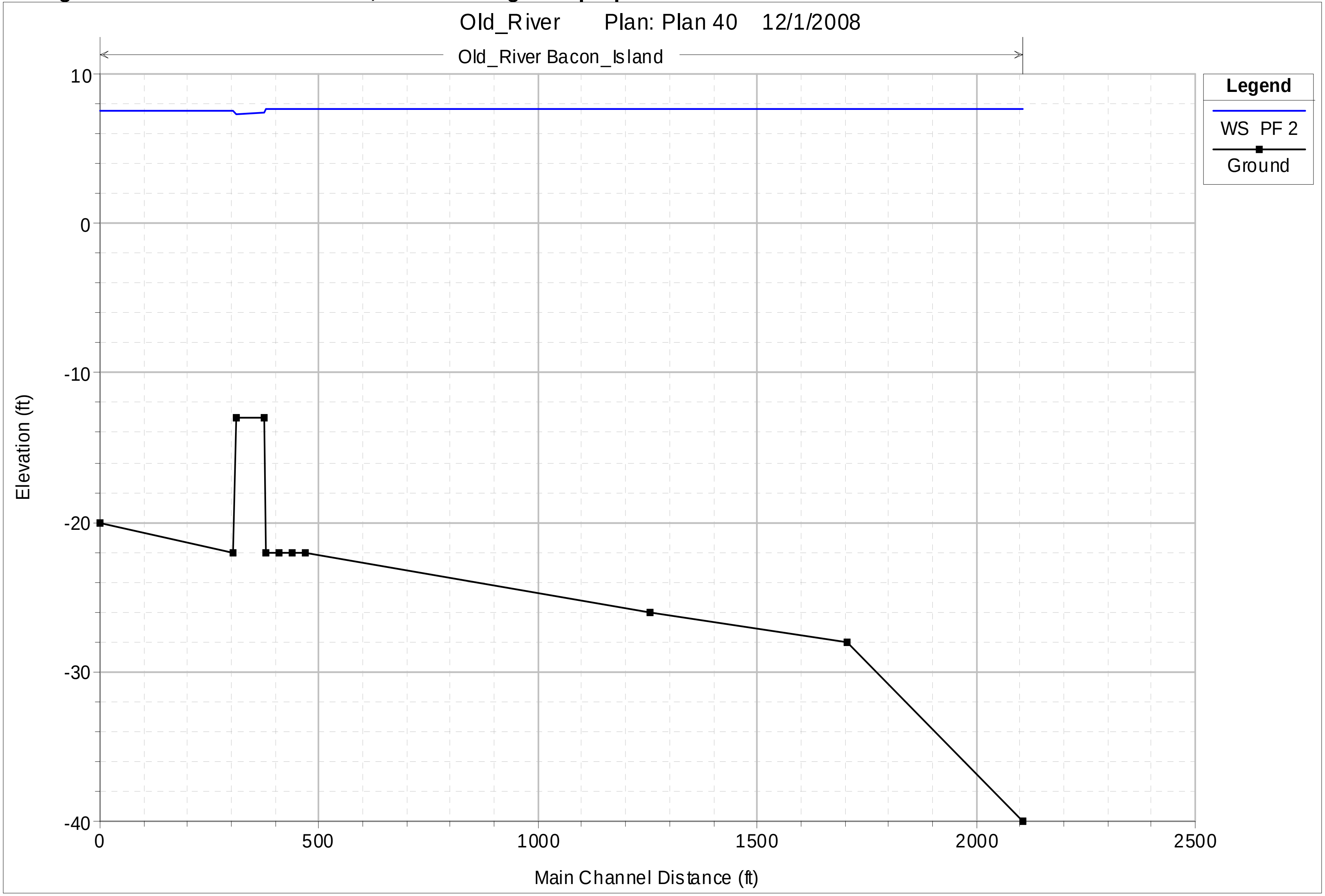


Figure 4 HEC-RAS Profile for 17,000cfs through the proposed Old River Structure with a 7.5-ft downsream water surface



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MEMORANDUM

To: Dennis Majors, Metropolitan Water District

From: Chris Potter, Rick Rhoads, Dilip Trivedi

Date: November 26, 2008

Subject: Refined Hydraulic Review Draft

M&N Job No.: 2-Gate Barrier Project
File No: 6097-02

Purpose

This memorandum and analyses described herein was prepared following the conceptual design phase of the project. The objective was to refine the initial hydraulic study, and to better assess the potential impacts to flood stage as a result of the structure.

Additional HEC RAS Simulations

A second round of HEC-RAS modeling was performed to assess the sensitivity of the model to the selection of manning's roughness, weir coefficients, and expansion and contraction coefficients. In this phase of the analysis, the barrier was modeled using the inline-structure function in HEC-RAS and with the boundary conditions kept constant in a steady-state simulation. The 100-yr tide was applied as the downstream boundary condition (9.7-ft NAVD88) and the 100-year discharge of 25,000-cfs was used as the upstream boundary condition; see Figure 1 which presents a return-frequency analysis of 15-years of USGS peak-flow data on Old River at Bacon Island. This combination of tide level and flow-rate represents a very infrequent, conservative event, representing a return-interval greater than 100 years. Manning's roughness was varied from a value of 0.03 to 0.05, which represent a clean straight channel to a channel vegetated with brush and weeds, respectively. Expansion and contraction coefficients were also varied from 0.1 and 0.3, representing a gradual transition, up to 0.6 and 0.8, representing an abrupt transition. Finally, the weir coefficient was varied from 2.6, a typical broad-crested weir value, up to 3.3, which is a typical sharp-crested weir value.

Based on these simulations, the model showed the most sensitivity to manning's roughness and the weir coefficient, see Tables 1 through 5. Varying the expansion and contraction coefficients did not result in a difference in water surface elevation upstream of the barrier. The greatest increase in water surface elevation during these simulations was 0.28-ft, based on using a manning's roughness of 0.05 and the broad-crested weir coefficient of 2.6. For a flow-rate of 25,000-cfs, the minimum increase in water surface elevation in this analysis was 0.14-ft and occurred using a manning's roughness of 0.03 and the sharp-crested weir coefficient of 3.3.

As the design of the 2-gate barrier structures are now based on the use of sheet-piles instead of the initial broader rock-dyke design, the use of the sharp-crested weir coefficient is more appropriate to the shape of the structure. Therefore the most likely impact to flood stage is on the order of 0.10-ft to 0.23-ft, representing a flow-rate between 20,000-cfs to 25,000-cfs and a manning's n of from 0.03 to 0.05.

Manning's roughness was varied in this sensitivity analysis to assist in making interpretations of the potential range of change to flood-stage. However, the only way to quantify the roughness of the design reach would be to develop a calibrated hydraulic model of the Delta.

An additional modeling study will be performed to verify the hydraulic conditions of Old River with the gate system in place using the calibrated hydrodynamic model DSM-2. The benefit of using this verified model is that it takes into account the dynamic nature of both the tides and the re-distribution of flood flows within the Delta. If the more detailed DSM-2 studies do show an increase in water surface profile greater than 0.1-ft for the 100-yr flood condition, the levees along the reach of Old River could be raised to accommodate this increase.

Table 1. Weir Coefficient = 2.6; Expansion/Contraction Coefficient = 0.1 & 0.3

		Manning's n = 0.03		Manning's n = 0.05	
Discharge (cfs)	**Downstream Boundary Condition** (ft NAVD88)	**Upstream WSEL** (ft NAVD88)	**Head Differential** (ft)	**Upstream WSEL** (ft NAVD88)	**Head Differential** (ft)
500	9.7	9.71	0.01	9.71	0.01
1000	9.7	9.71	0.01	9.71	0.01
5,000	9.7	9.71	0.01	9.72	0.02
10,000	9.7	9.74	0.04	9.75	0.05
15,000	9.7	9.79	0.09	9.82	0.12
20,000	9.7	9.82	0.12	9.88	0.18
25,000	9.7	9.88	0.18	9.97	0.27

Table 2. Weir Coefficient = 2.6; Expansion/Contraction Coefficient = 0.3 & 0.5

		Manning's n = 0.03		Manning's n = 0.05	
Discharge (cfs)	**Downstream Boundary Condition** (ft NAVD88)	**Upstream WSEL** (ft NAVD88)	**Head Differential** (ft)	**Upstream WSEL** (ft NAVD88)	**Head Differential** (ft)
500	9.7	9.71	0.01	9.71	0.01
1000	9.7	9.71	0.01	9.71	0.01
5,000	9.7	9.71	0.01	9.72	0.02
10,000	9.7	9.74	0.04	9.75	0.05
15,000	9.7	9.79	0.09	9.82	0.12
20,000	9.7	9.82	0.12	9.88	0.18
25,000	9.7	9.88	0.18	9.97	0.27

Table 3. Weir Coefficient = 2.6; Expansion/Contraction Coefficient = 0.6 & 0.8

Discharge	Downstream Boundary Condition	Manning's n = 0.03		Manning's n = 0.05	
		Upstream WSEL	Head Differential	Upstream WSEL	Head Differential
(cfs)	(ft NAVD88)	(ft NAVD88)	(ft)	(ft NAVD88)	(ft)
500	9.7	9.71	0.01	9.71	0.01
1000	9.7	9.71	0.01	9.71	0.01
5,000	9.7	9.71	0.01	9.72	0.02
10,000	9.7	9.74	0.04	9.75	0.05
15,000	9.7	9.79	0.09	9.82	0.12
20,000	9.7	9.82	0.12	9.88	0.18
25,000	9.7	9.88	0.18	9.97	0.27

Table 4. Weir Coefficient = 3.0; Expansion/Contraction Coefficient = 0.1 & 0.3

Discharge	Downstream Boundary Condition	Manning's n = 0.03		Manning's n = 0.05	
		Upstream WSEL	Head Differential	Upstream WSEL	Head Differential
(cfs)	(ft NAVD88)	(ft NAVD88)	(ft)	(ft NAVD88)	(ft)
500	9.7	9.71	0.01	9.71	0.01
1000	9.7	9.71	0.01	9.71	0.01
5,000	9.7	9.71	0.01	9.72	0.02
10,000	9.7	9.74	0.04	9.75	0.05
15,000	9.7	9.77	0.07	9.80	0.10
20,000	9.7	9.80	0.10	9.86	0.16
25,000	9.7	9.86	0.16	9.95	0.25

Table 5. Weir Coefficient = 3.3; Expansion/Contraction Coefficient = 0.1 & 0.3

Discharge	Downstream Boundary Condition	Manning's n = 0.03		Manning's n = 0.05	
		Upstream WSEL	Head Differential	Upstream WSEL	Head Differential
(cfs)	(ft NAVD88)	(ft NAVD88)	(ft)	(ft NAVD88)	(ft)
500	9.7	9.71	0.01	9.71	0.01
1000	9.7	9.71	0.01	9.71	0.01
5,000	9.7	9.71	0.01	9.72	0.02
10,000	9.7	9.74	0.04	9.75	0.05
15,000	9.7	9.77	0.07	9.80	0.10
20,000	9.7	9.80	0.10	9.86	0.16
25,000	9.7	9.84	0.14	9.93	0.23

Figure 1. Peak-Flow Recurrence Intervals for Old River at Bacon Island

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MEMORANDUM

To:	**Dennis Majors, Metropolitan Water District**
From:	**Chris Potter, Rick Rhoads, Dilip Trivedi**
Date:	**September 16, 2008**
Subject:	**Supplemental Study of Flood Issues Preliminary Draft**
M&N Job No.:	**2-Gate Barrier Project File No: 6097-02**

Flood Issues - Supplemental Study

1.0 Introduction

The 2-Gate Barrier System will provide a 170' opening for the passage of flood flows when the double butterfly gates are open, which includes the 75' clear center opening for navigation. The gate also provides for additional flood conveyance when the barrier is overtopped at flood stages exceeding 6.6-ft NAVD88.

Since the 2-Gate Barrier is a pilot project that will be deployed seasonally, and removed for the remainder of the year, flood profiles should not be an issue during the months from July through December when the barrier is not deployed. When the barrier is deployed, there will be 2 operating modes: Predominantly open from January through March, and Predominantly closed from April through June. However, for the purpose of this flood study, the gates were assumed to be open from January through June because the gates will be opened during flood events to permit the passage of flood flows.

This memo describes the analysis to assess the potential impacts on flood profiles due to the 2-gate barrier system.

2.0 Methodology

River Stage and discharge data for the 14 year period from 1992 to 2005 is available, and has been used by the CCWD in the development and calibration of its Delta Simulation Model (DSM-2). The model has been used to develop time histories of river stage and discharge for sites immediately upstream and downstream of the proposed barriers. The initial run considered the existing conditions (without the barriers) and produced values at 15 minute intervals over the entire period. It is worth noting that the impacts of astronomical tides propagated upriver from San Francisco Bay, flood flows propagating downstream from the watershed, and the water withdrawals by the State and Federal pumping plants in the South Delta are included in this analysis. The results of the DSM-2 run provided by the CCWD were analyzed by M&N to produce the flood-stage hydrographs and statistical summaries of the percent (of time) occurrence of stage described below.

In order to help identify potential flood and navigation concerns with the barriers in place, M&N requested that CCWD modify the DSM-2 to simulate conditions for both barrier operating modes and rerun the time histories for each; however, only the Gates Open simulation was used in this flood study. The Gates Open barrier simulation neglected overtopping of the barrier during flood flows; therefore, this conservatism in the analysis had the potential to produce higher flood-stages at the gate than would actually occur.

DSM-2 model output was analyzed at points immediately upstream and downstream of the barrier, as well as at two locations further upstream and downstream of the barrier to assess the influence on flood-stage of the barrier within Old River. Gage location ROLD014 is roughly 8000-ft downstream of the barrier, and gage location ROLD024 is roughly 6000-ft upstream of the barrier, see Figure 1.

The peak flood event during the period of DSM-2 model simulation was the February 1998 event. Therefore, stage hydrographs from this event were compared to assess impacts to flood-stage during extreme events. The cumulative frequency of river stage for the January to June period over the entire 14-year simulation for the Gates Open condition and the existing condition were also summarized to provide a statistical comparison.

3.0 Results

The stage hydrographs of the existing and Gates Open conditions for the February 1998 flood event at the 2-gate barrier are compared in Figure 2. As the figure illustrates, the barrier did not increase the flood stage profile at the peak stages immediately upstream of downstream of the barrier.

The stage hydrographs of the existing and Gates Open conditions for the February 1998 flood event at Gage location ROLD014, ~8000-ft downstream of the barrier, are compared in Figure 3. The stage hydrographs of the existing and Gates Open conditions for the February 1998 flood event at Gage location ROLD024, ~6000-ft upstream of the barrier, are compared in Figure 4. Figures 3 and 4 confirm that the barrier did not increase the flood stage profile at the peak stages within a mile upstream of downstream of the barrier.

The exceedance probability expressed as a % for river stage at the sites immediately upstream and downstream of the barrier is presented in Figures 5a and 5b for the Old River Barrier. Lines are shown for the baseline condition, as well as Gates Open on the upstream side of the structure and Gates Open on the downstream side of the structure. The exceedance probability plots support the finding of no impact to flood stage greater than 8.4-ft NAVD88 due to the 2-Gate Barrier. And these results included the inherent conservatism in the analysis due to lack of overtopping of the barrier that would normally occur for flood stages greater than 6.6-ft NAVD88. The 100-yr flood stage within Old River is 9.71-ft NAVD88.

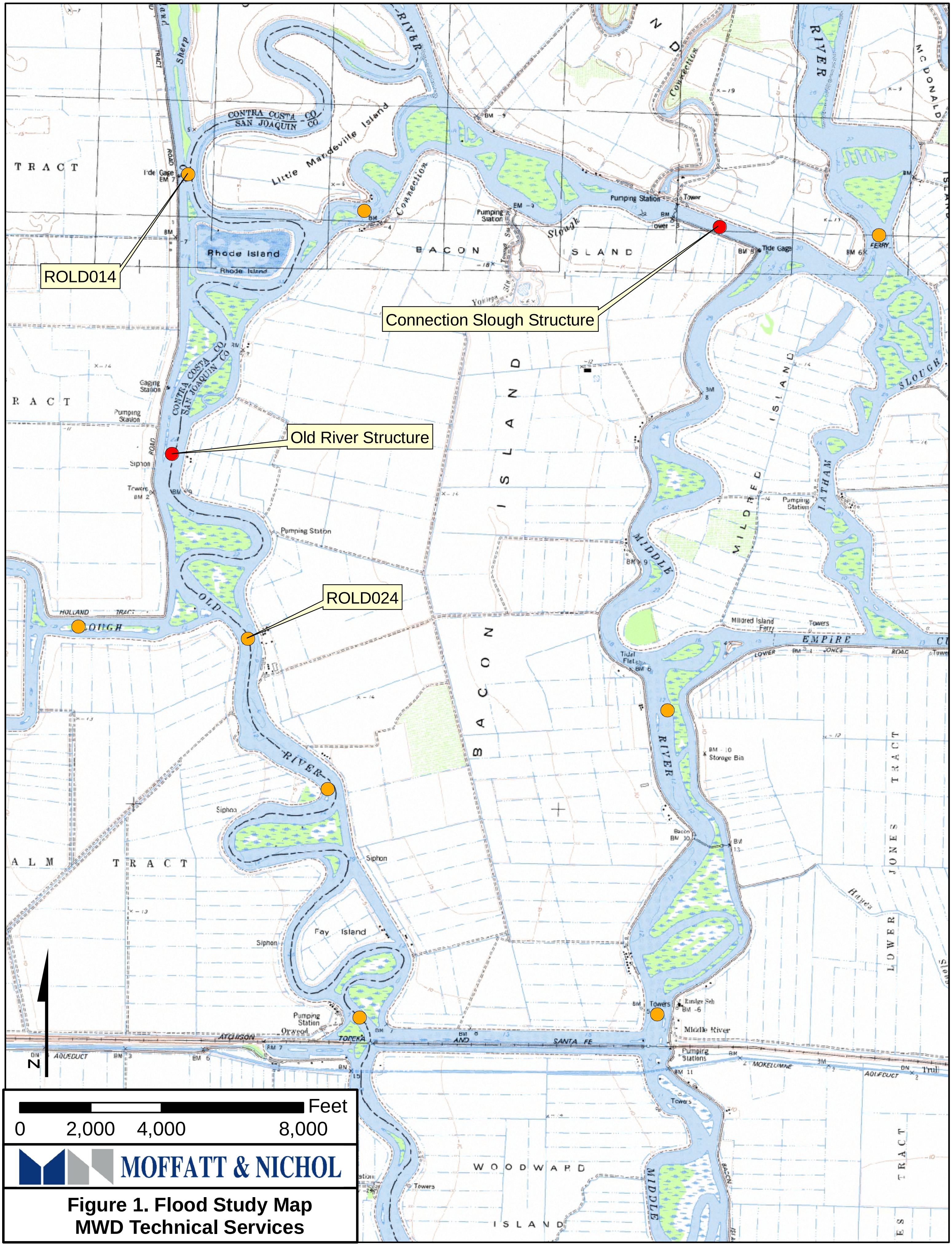


Figure 1. Flood Study Map
MWD Technical Services

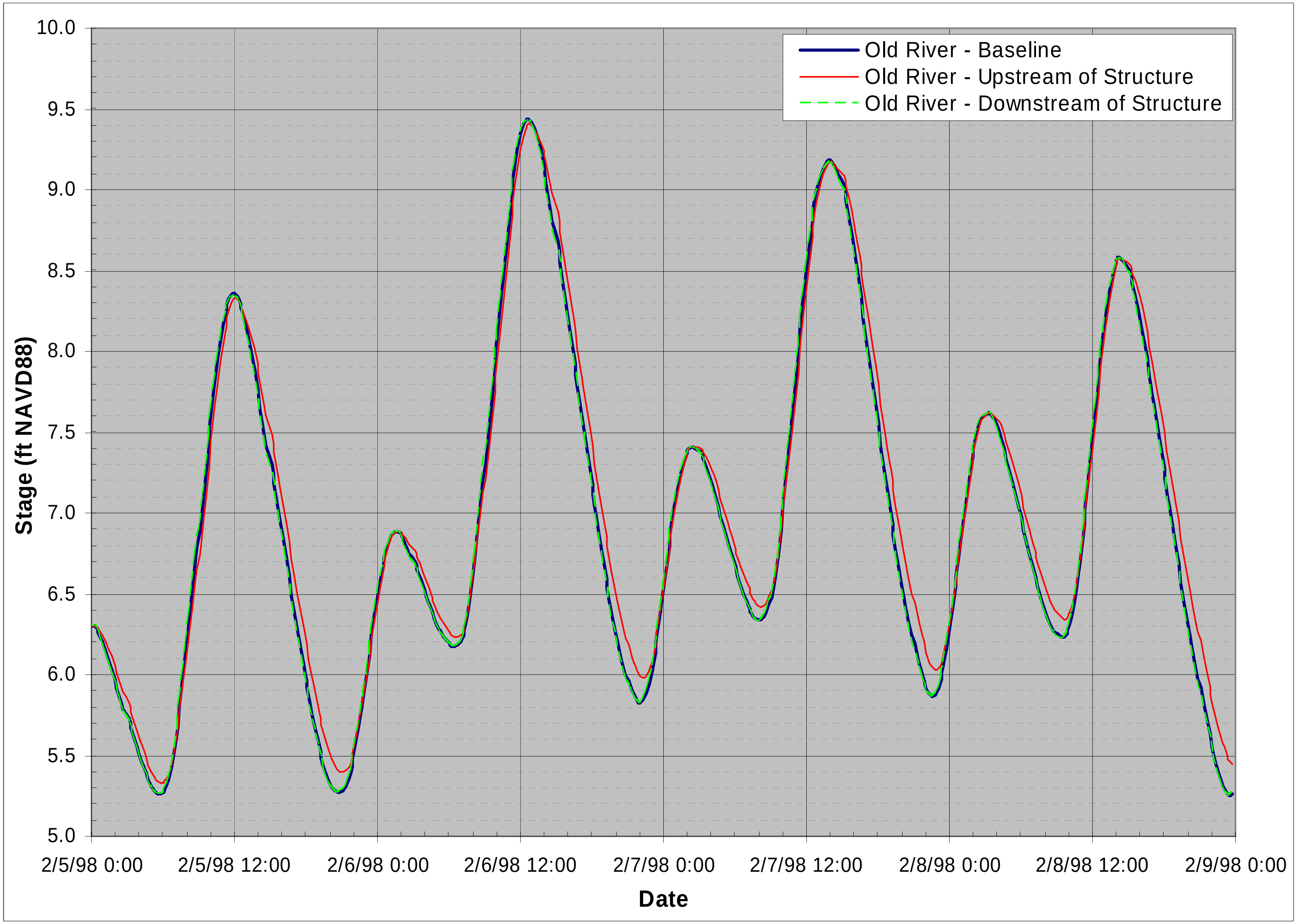


Figure 2. Stage Profiles for February 1998 Flood Event at Old River 2-Gate Barrier

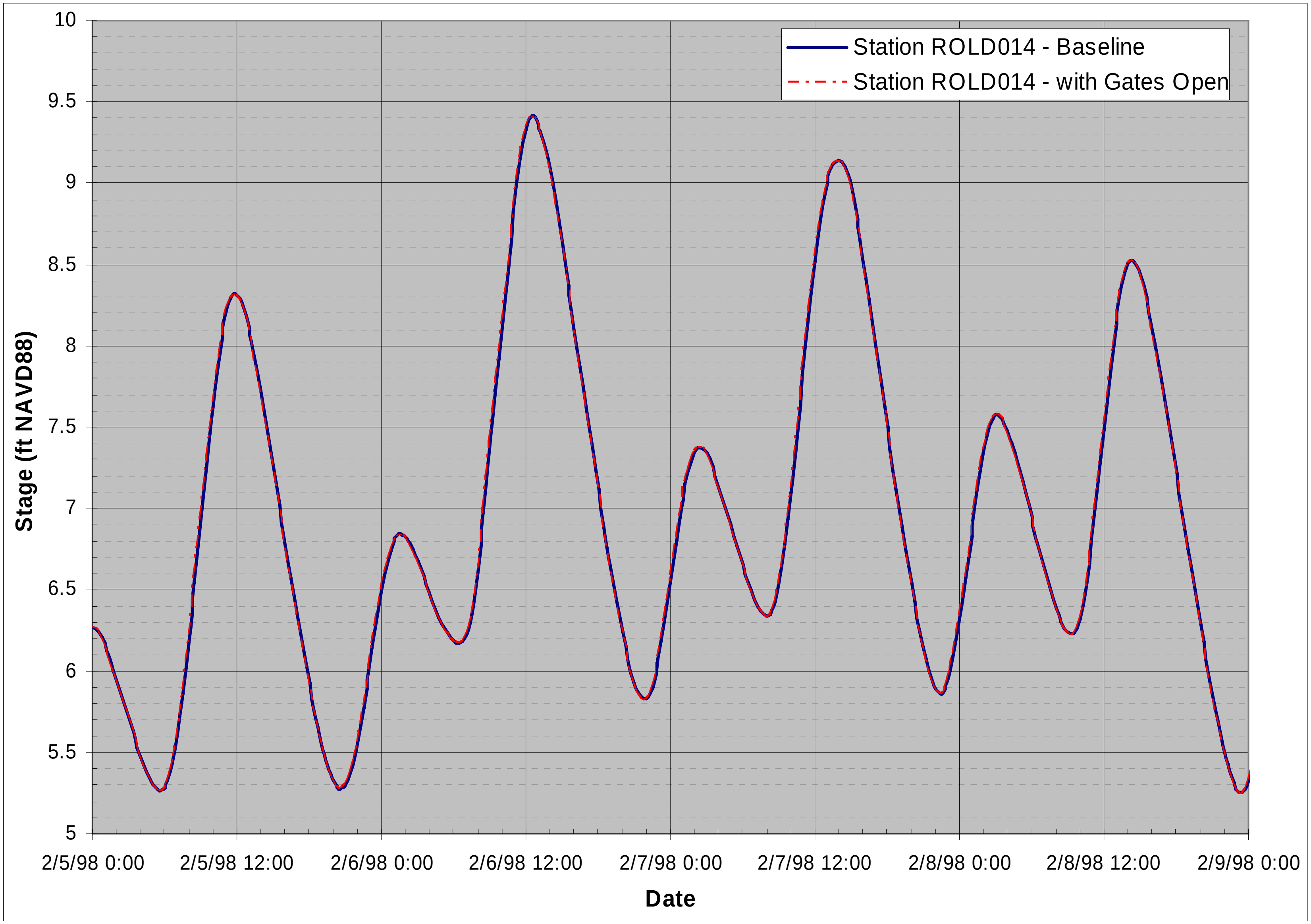


Figure 3. Stage Profiles for February 1998 Flood Event at Old River Gage Station ROLD014

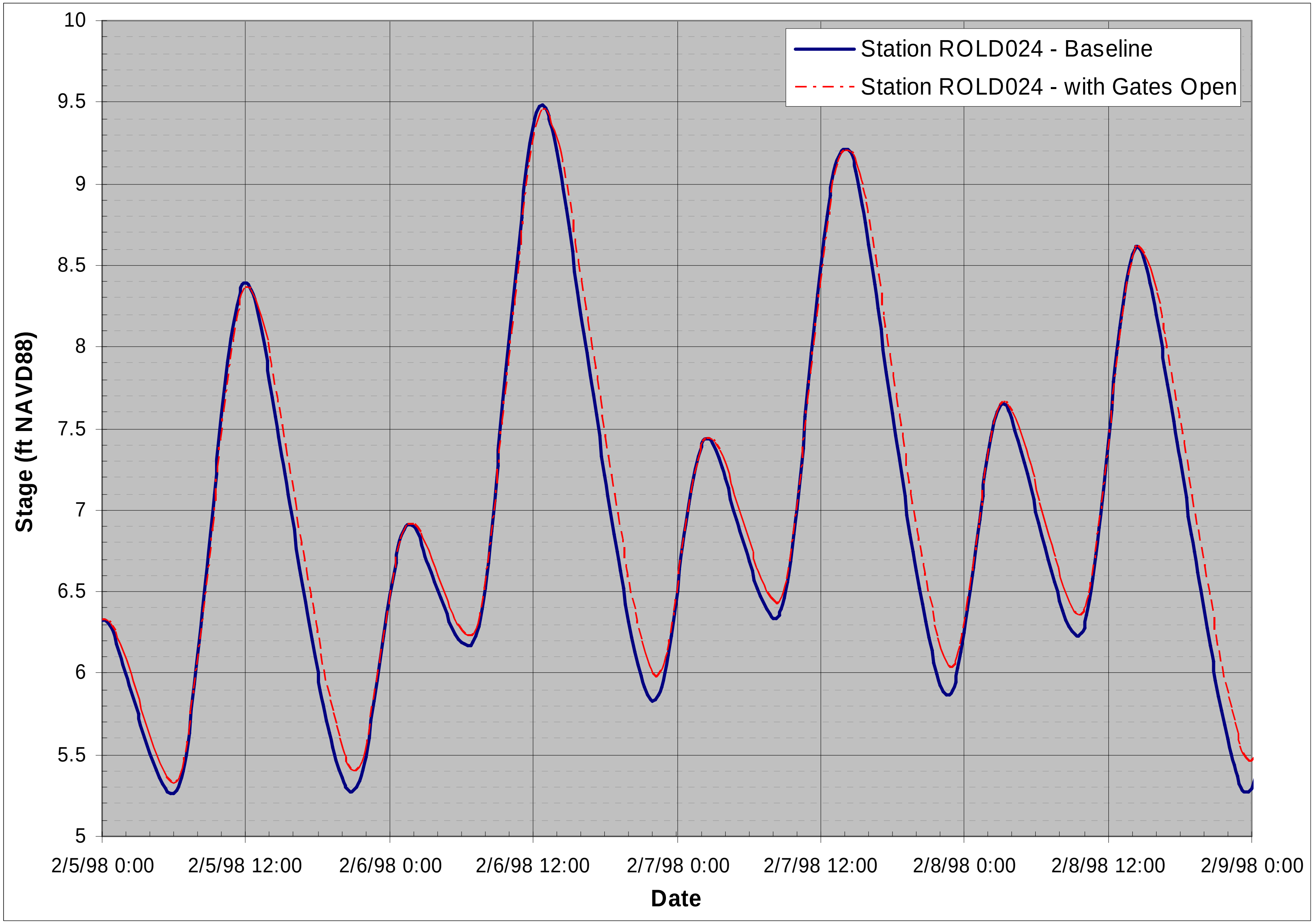


Figure 4. Stage Profiles for February 1998 Flood Event at Old River Gage Station ROLD024

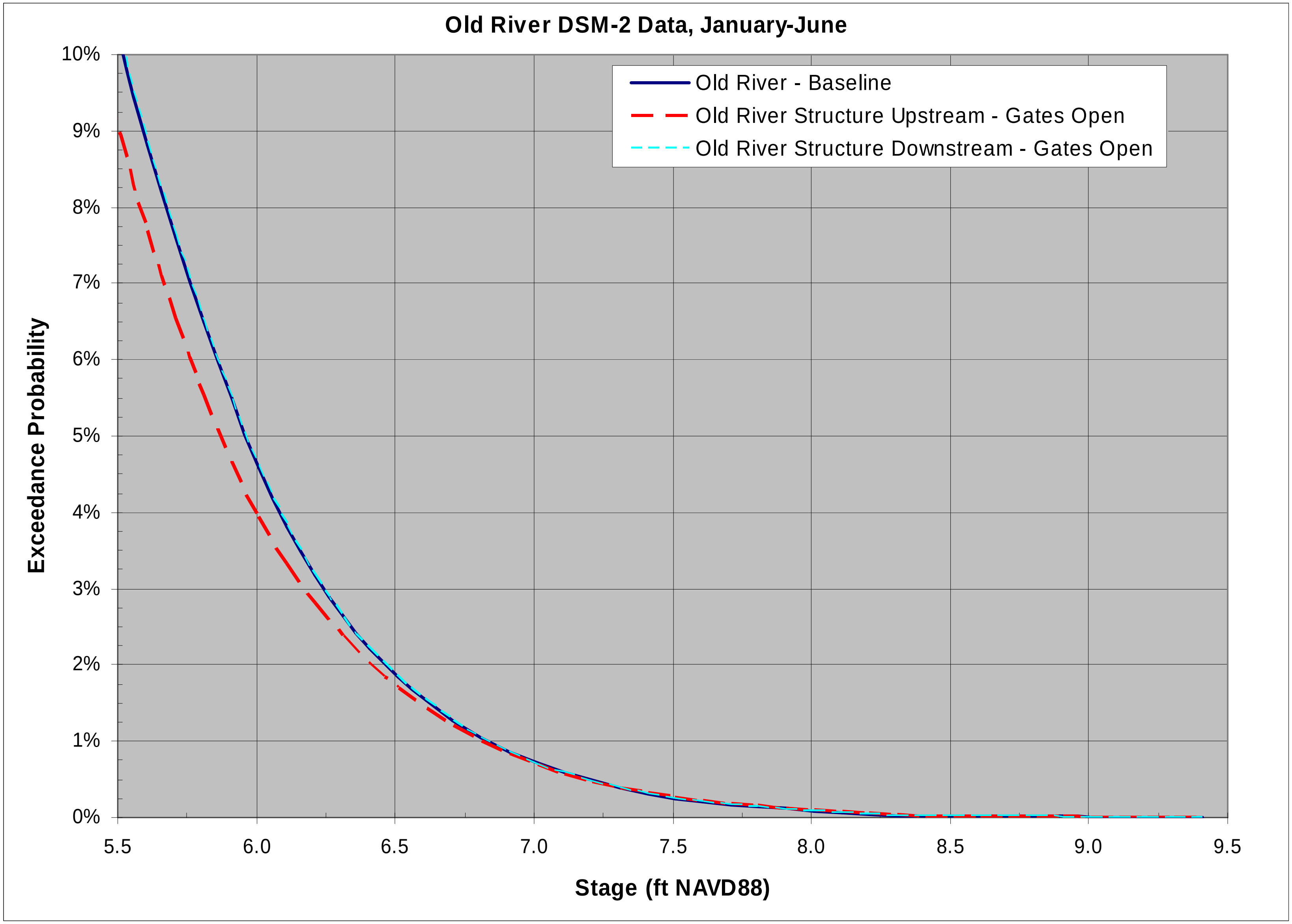


Figure 5a. Exceedance Probabilities for High Stages at Old River 2-Gate Barrier

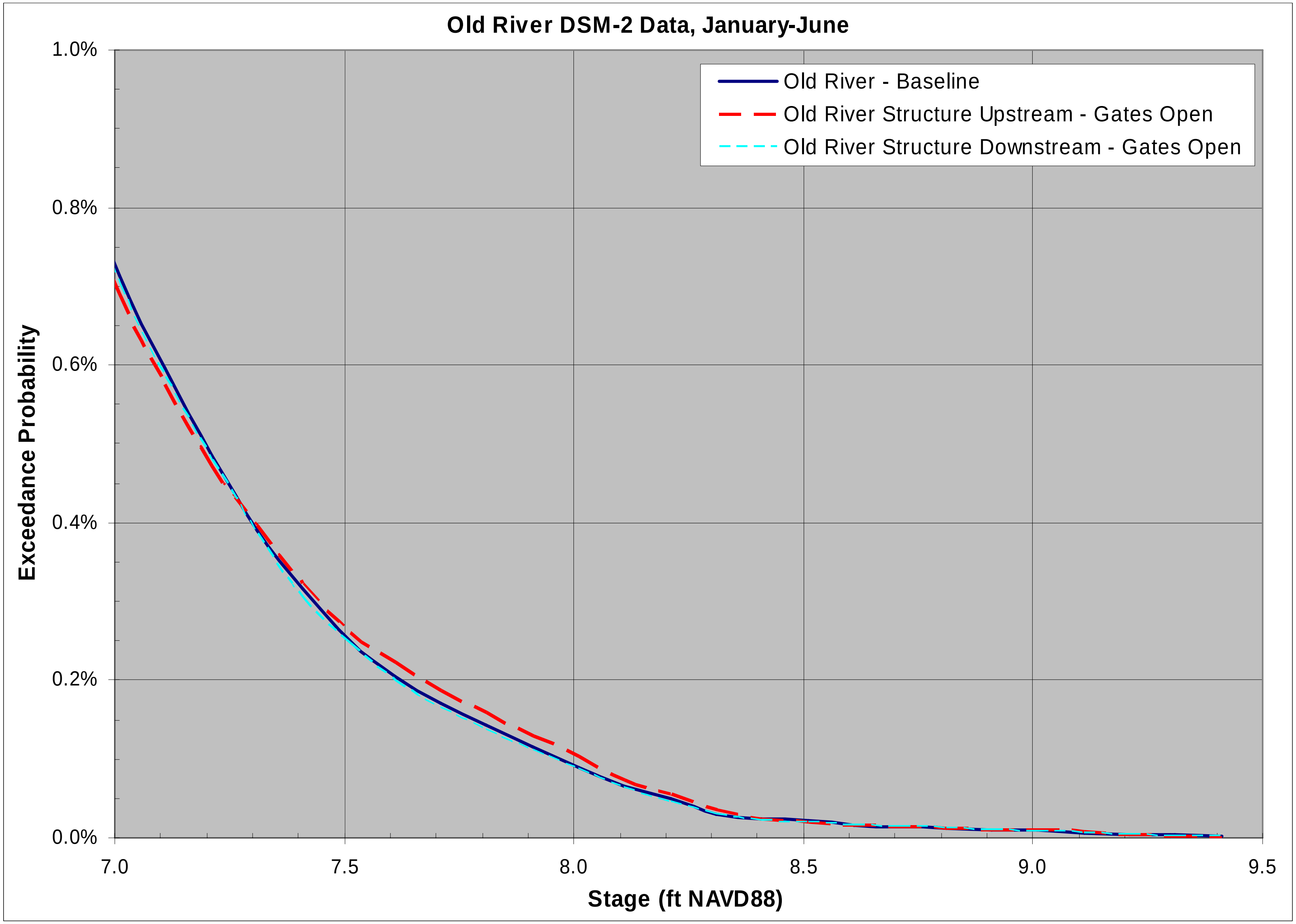


Figure 5b. Exceedance Probabilities for High Stages at Old River 2-Gate Barrier

CONTRA COSTA WATER DISTRICT
Technical Memorandum

DATE: November 26, 2008

PREPARED BY: Brett T. Kawakami, Associate Water Resources Specialist

SUBJECT: CCWD DSM2 Flood Analysis for 2-Barrier Project

PURPOSE: This memorandum describes the hydrodynamic modeling using the Delta Simulation Model, Version 2 (DSM2) that was performed by Contra Costa Water District (CCWD) to determine potential flood effects of the proposed 2-Barrier project. Results of this analysis show no significant flood impacts based on a 16 year historical DSM2 analysis (1991-2005), provided the gates are left open during high water events.

Delta Hydrodynamic Model – DSM2

DSM2 is a one-dimensional model developed by the Department of Water Resources (DWR) for simulating hydrodynamics, water quality, and particle tracking in a network of riverine or estuarine channels (DWR,2000). The model is used by DWR and others to perform operational and planning studies of the Delta. Details of the model, including source codes, model calibration, and model performance, are available from the DWR Bay-Delta Office, Modeling Support Branch. Documentation of model development is discussed in annual reports to the SWRCB which are available at: http://baydeltaoffice.water.ca.gov/modeling/deltamodeling/annualreports.cfm. DSM2 is a widely used model for studying issues pertaining to flow, water elevations, water quality and fisheries issues in the Delta and is well calibrated for flow, stage and water quality (Nader-Tehhrani, 2001;Thein and Nader-Tehrani, 2006).

The Hydro module of DSM2, applied to the Delta, simulates tidal hydrodynamics (channel stage, flow, and water velocity) using a 15-minute time step. For the 2-barrier project, DSM2 Hydro was used to evaluate changes in stage and flow in the vicinity of the barriers. In this analysis, results from use of the Hydro module are used to determine potential flood impacts from implementation of the 2-Barrier Project. A discussion of the DSM2 setup, results and conclusions are provided.

DSM2 Setup

DSM2 (DWR, 2000) was used to simulate the effect of installing temporary barriers in the vicinity south of Franks Tract in Old River and Connection Slough. The simulations were based on the most recent historical DSM2 setup available from DWR and were conducted from 1991-2005.

Gate locations and dimensions

The barriers will consist of sunken barges in Old River and Connection Slough with operable gates placed on top (Moffatt and Nichol, 2008). The barriers with the gates closed are modeled in DSM2 as single gates that extend the width of the channels. The barriers with the gates open are modeled as notched weirs that allow flow through an area defined by the dimensions of the gates (see below). The barriers were placed at DSM2 Channels 111 (Old River) and 248 (Connection Slough). Gate dimensions are as follows:

- Gate width: 170 feet (ft)
- Bottom elevation of gate: -13 ft NAVD88 (-15.4 ft NGVD29)
- Top elevation of gate: 6.6 ft NAVD88 (4.2 ft NGVD29)

Elevations were converted from NAVD88 to NGVD29 for use in DSM2.

Scenario Descriptions

The DSM2 scenarios used in the flood analysis are described in **Table 1**. For the purposes of the analysis, the gates were not operated and were considered either open or closed for the entire simulation. The No Gates scenario represents the base case used for comparison. All scenarios used the same set of unmodified historical boundary flows and operations.

Table 1: DSM2 Scenarios

Scenario	Description
No Gates	Barriers are not installed.
Gates Closed	Barriers are installed and gates closed year round. Flow only occurs when gate is overtopped.
Gates Open	Barriers are installed and gates left open year round. Flow occurs through the gate opening.
Gates Open (0.2 Coefficient)	Same as "Gates Open" scenario with an additional weir friction coefficient of 0.2 applied.

Results

Stage information was output at 15 minute intervals immediately upstream and downstream of both barriers, as well as at other selected locations. The results were provided on CD-ROM to DWR in MATLAB and ASCII format in August, 2008 (CCWD, 2008). All stage results in this discussion are given in the NGVD 1929 datum.

Maximum annual stage impact

For each scenario, the highest stage in each water year of the simulation (maximum annual stage) was identified at four locations immediately upstream and downstream of the Connection Slough and Old River barriers. The changes in maximum annual stage between the with-gate scenarios and the No Gates scenario were determined. Maximum annual stage exceeded the top of the gate elevation (4.2 ft) in all years and scenarios. A comparison between the Gates Closed and No Gates scenario is shown in **Table 2.** Bold numbers indicate the highest increase for a given location over the entire simulation period. The highest increase to the maximum annual stage was 0.23 ft, which occurred in 1997 at the upstream side of Old River. In all other years, the maximum increase was 0.16 ft or lower.

Table 2: Changes in Stage at Maximum Annual Stage for Gates Closed versus No Gates scenario

Year	No Gates Scenario Maximum Annual Stage (feet) NGVD 1929	Change in Maximum Annual Stage (feet)* Compared to No Gates Scenario			
		Connection Slough U/S of Barrier	Connection Slough D/S of Barrier	Old River U/S of Barrier	Old River D/S of Barrier
1992	4.90	-0.01	0.02	-0.48	0.06
1993	5.00	-0.02	0.03	-1.03	0.08
1994	4.83	-0.03	**0.08**	-1.44	**0.16**
1995	5.93	**0.05**	-0.02	0.01	-0.01
1996	5.22	0.04	0.02	-0.41	0.06
1997	5.93	0.04	-0.09	**0.23**	-0.07
1998	7.08	0.01	-0.03	-0.07	-0.02
1999	4.52	0.00	0.03	-0.47	0.11
2000	4.97	-0.03	0.06	-0.84	0.12
2001	4.93	-0.02	0.01	-0.55	0.07
2002	5.10	-0.04	-0.03	-0.50	0.01
2003	5.41	-0.08	-0.06	-0.65	-0.03
2004	5.26	-0.06	0.03	-1.39	0.09
2005	5.49	-0.04	0.01	-0.93	0.07

****Bold indicates maximum increase observed at the location for all years***

In the comparison between the Gates Open and No Gates scenarios shown in **Table 3**, the increase in maximum annual stage in 1997 was reduced to 0.01 ft, and the maximum for all years was 0.02 ft, occurring in 2004. Thus, leaving the gates open (barriers installed) during periods of high flows greatly reduces the impact at the maximum stage.

Table 3: Changes in Stage at Maximum Annual Stage for Gates Open versus No Gates scenario

Year	No Gates Scenario Maximum Annual Stage (feet) NGVD 1929	Change in Maximum Annual Stage (feet)* Compared to No Gates Scenario			
		Connection Slough U/S of Barrier	Connection Slough D/S of Barrier	Old River U/S of Barrier	Old River D/S of Barrier
1992	4.90	**0.00**	0.00	-0.13	-0.01
1993	5.00	**0.00**	0.00	-0.21	0.00
1994	4.83	**0.00**	0.01	-0.31	0.02
1995	5.93	**0.00**	-0.01	-0.02	-0.01
1996	5.22	**0.00**	0.00	-0.14	0.00
1997	5.93	**0.00**	-0.03	**0.01**	-0.02
1998	7.08	**0.00**	0.00	-0.03	-0.01
1999	4.52	**0.00**	0.00	-0.16	0.01
2000	4.97	**0.00**	**0.01**	-0.26	0.01
2001	4.93	**0.00**	0.00	-0.15	0.00
2002	5.10	-0.01	-0.01	-0.13	0.00
2003	5.41	-0.01	-0.01	-0.14	-0.01
2004	5.26	-0.01	0.00	-0.28	**0.02**
2005	5.49	**0.00**	0.00	-0.26	0.01

****Bold indicates maximum increase observed at the location for all years***

We conducted a sensitivity analysis by rerunning the Gates Open scenario with a conservative friction coefficient of 0.2 applied, which serves to constrict the flow allowed through the gates significantly. As shown in **Table 4**, the Gates Open (with 0.2 Coefficient) showed a maximum increase in maximum annual stage of 0.42 ft versus the No Gates scenario. There were also other instances, mostly at the Old River barrier downstream, where changes in maximum annual stage exceed 0.1 ft, although the highest increase is at 0.14 ft. The increases in stage are unexpectedly higher for the Gates Open (with 0.2 Coefficient) than the Gates Closed scenario. This is due to the fact that overtopping is not properly simulated in DSM2 for the Gates Open scenarios. When stage exceeds the top of the gate elevation, flow should be allowed across the entire length of the barrier, which is what does occur in the Closed Gate scenario simulation. However, in the Open Gate scenarios, the gate is essentially modeled as a notched weir, and due to a DSM2 limitation, flow is only allowed through the notched portion no matter how high the stage, leading to artificially higher stages to occur in the simulation. Thus, if overtopping were properly accounted for, the increases in stage would be lower.

Table 4: Changes in Stage at Maximum Annual Stage for Gates Open (with 0.2 Coefficient) versus No Gates scenario

Year	No Gates Scenario Maximum Annual Stage (feet) NGVD29	Change in Maximum Annual Stage (feet) Compared to No Gates Scenario			
		Connection Slough U/S of Barrier	Connection Slough D/S of Barrier	Old River U/S of Barrier	Old River D/S of Barrier
1992	4.90	-0.01	0.01	-0.47	0.04
1993	5.00	-0.01	0.03	-0.90	0.08
1994	4.83	-0.02	**0.07**	-1.28	**0.14**
1995	5.93	**0.02**	-0.03	0.12	-0.03
1996	5.22	**0.02**	0.02	-0.42	0.07
1997	5.93	**0.02**	-0.09	**0.42**	-0.07
1998	7.08	0.00	-0.05	0.08	-0.03
1999	4.52	0.00	0.02	-0.54	0.08
2000	4.97	-0.01	0.05	-0.80	0.11
2001	4.93	-0.01	0.02	-0.57	0.06
2002	5.10	-0.03	-0.01	-0.50	0.03
2003	5.41	-0.04	-0.01	-0.54	0.01
2004	5.26	-0.03	0.05	-1.15	0.11
2005	5.49	-0.02	0.05	-0.86	0.11

Bold indicates maximum increase observed at the location for all years

Changes in maximum annual stages for the Gates Open were also checked for two locations (ROLD014 and ROLD024) at some distance (less than a mile) upstream and downstream of the barriers **(See Figure 1 for locations).** Examination confirmed that the maximum increase was small (0.02 ft) at each location. The stage output for a number of other locations along Old and Middle Rivers were also examined and these showed a maximum increase of less than 0.04 ft when the gates were opened.

Fractional Exceedance Plots

Cumulative distribution function curves for stage output from the simulations were generated for the Gates Open and No Gates scenario at the OR Upstream barrier, ROLD014, and ROLD024 locations. The comparison between the two scenarios at is shown in **Figures 2 thru 4.** The figures illustrate that the there is not a

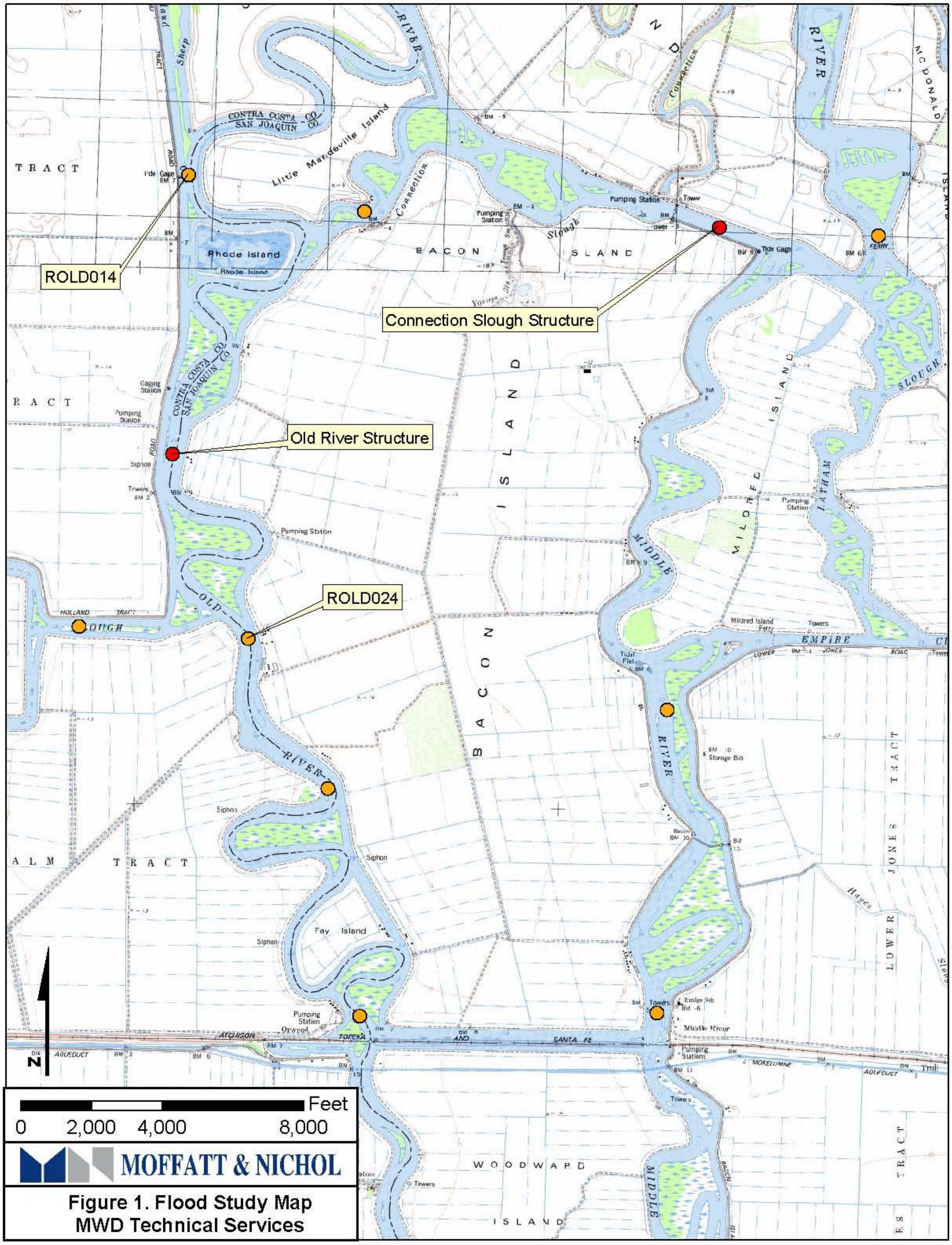


Figure 1: Location of Old River and Connection Slough Barriers and ROLD014 and ROLD024

significant difference in frequency distribution of stage between the Gates Open and No Gates scenarios for the three locations.

Figure 2: Cumulative Distribution Function Plot of Stage for OR Barrier upstream (1992-2005)

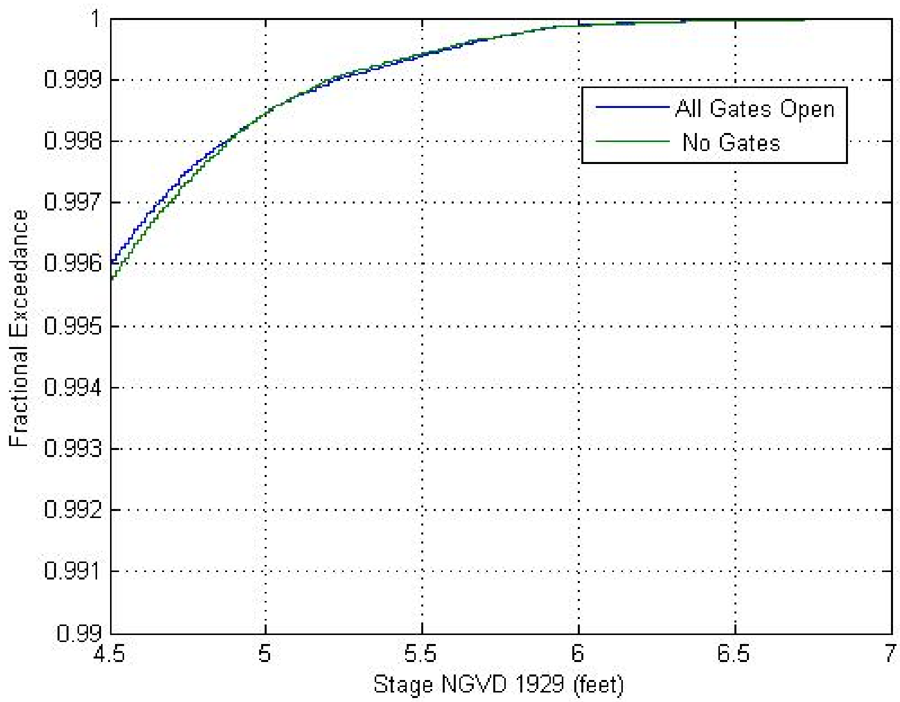


Figure 3: Cumulative Distribution Function Plot of Stage for ROLD014 (1992-2005)

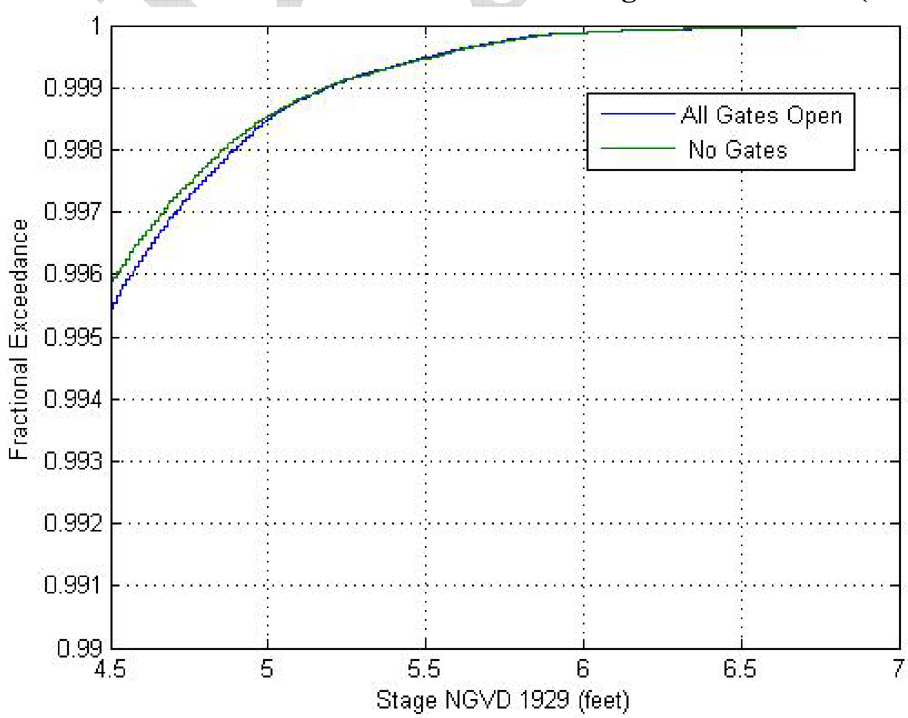
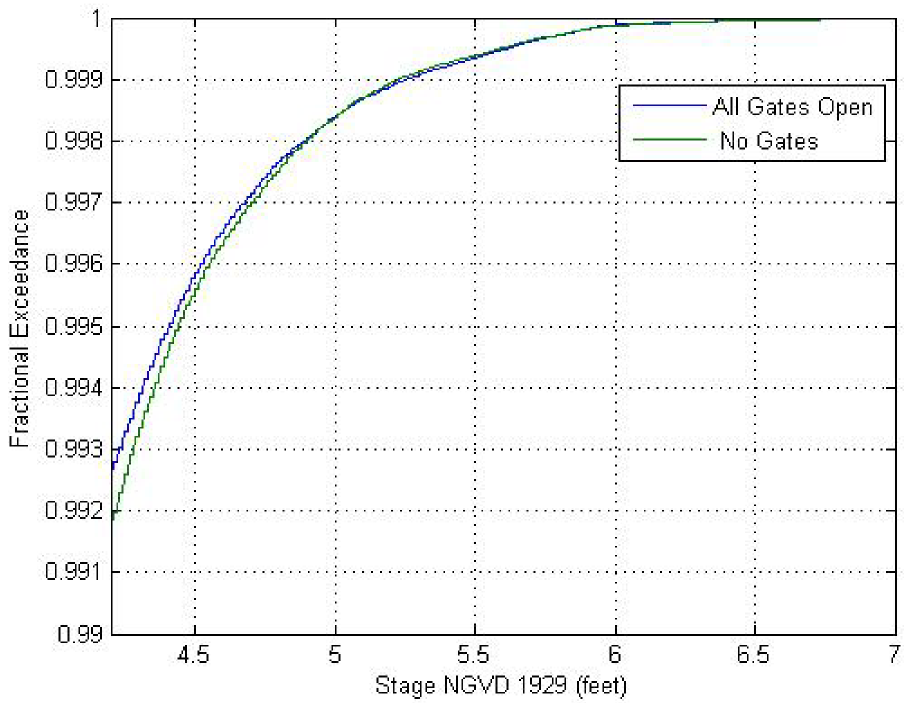


Figure 4: Cumulative Distribution Function Plot of Stage for ROLD024 (1992-2005)



Conclusions

The results of this analysis demonstrate that installation of barriers at Connection Slough and Old River with gates open does not significantly increase stage levels nor result in substantial increases in frequency of higher stages. The analysis confirms that this is true both at the barriers themselves (immediately upstream and downstream) and at locations some distance upstream and downstream from the barriers (ROLD014, ROLD024). The maximum observed increase in maximum annual stage for these locations was small (less than 0.23 feet or about 2.5 inches) when the gates were left closed. When the gates were left open, the maximum increase was reduced to below 0.02 ft. Analysis of stage at other locations along Old and Middle Rivers showed a maximum increase with gates open of 0.04 ft. The cumulative distribution function analysis shows that there is not a significantly higher incidence of high stage levels when the barriers are in with gates open versus when no barriers are present.

This analysis confirms the need to flexibly manage the barriers in response to actual hydrologic conditions such as flood and high water events. Mechanisms for monitoring flow conditions and adjusting gate position are being incorporated into the operational plans for the 2-Barrier Project.

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