Appendix B
Flood Management and Geomorphic Conditions Technical Appendix

B.1 MBK Engineers. 2013b. Supplemental Memo: Refined Hydraulic Impact Analysis for the Southport Sacramento River Early Implementation Project Environmental Impact Statement/ Environmental Impact Report. September 9, 2013. Sacramento, CA. Prepared by: Mike Archer, P.E. Prepared for: West Sacramento Flood Control Agency, West Sacramento, CA.

B.2 MBK Engineers. 2013a. Hydraulic Impact Analysis for the Southport Sacramento River Early Implementation Project Environmental Impact Statement/ Environmental Impact Report. July 26, 2013. Sacramento, CA. Prepared by: Mike Archer, P.E. Prepared for: West Sacramento Flood Control Agency, West Sacramento, CA.

B.3 MBK Engineers. 2013c. Southport Sacramento River Early Implementation Project Interim Condition Hydraulic Impact Analysis. September 18, 2013. Sacramento, CA. Prepared by: Mike Archer, P.E. Prepared for: West Sacramento Flood Control Agency, West Sacramento, CA.

B.4 MBK Engineers. 2011a. Hydraulic Impact Analysis. Southport Early Implementation Project. June 29, 2011. Sacramento, CA. Prepared by: Mike Archer, P.E. Prepared for: West Sacramento Flood Control Agency, West Sacramento, CA.

B.5 MBK Engineers. 2011b. Preliminary Existing Condition 2-Dimenstional Hydraulic Simulation Model. Southport Early Implementation Project. December 23, 2011. Sacramento, CA. Prepared by: Mike Archer, P.E. Prepared for: West Sacramento Flood Control Agency, West Sacramento, CA.

B.6 cbec inc., eco engineering. 2013. Memorandum—Average annual inundation duration of the offset's lower floodplain (draft). September 6. Prepared by: John Stofleth, Chris Bowles, and Poyom Riles. Prepared for: Michael Vecchio, HDR Engineering, Sergio Jiminez, HDR Engineering, and Carl Jensen, ICF International. Project Number 12-1001. Southport EIP.

B.7 cbec inc., eco engineering. 2011a. West Sacramento / Southport EIP: Task Order 2: Historic and Current Preliminary Geomorphic Assessment. September 12, 2011. Final Report. Project Number 11-1003. Prepared for: West Sacramento Flood Control Agency and HDR Engineering

B.8 cbec inc., eco engineering. 2011b. Technical Memorandum—Existing Conditions Sediment Transport Assessment. November 7, 2011. Prepared by: John Stofleth, Chris Campbell, and Chris Bowles. Prepared for: Michael Vecchio, HDR Engineering. Project Number 11-1013. West Sacramento Southport EIP Task Order 3.

Appendix B.1

Hydraulic Impact Analysis for the Southport Sacramento River Early Implementation Project Environmental Impact Statement/ Environmental Impact Report—MBK Engineers, July 26, 2013

MBK ENGINEERS

SUPPLEMENTAL MEMO: REFINED HYDRAULIC IMPACT ANALYSIS FOR THE SOUTHPORT SACRAMENTO RIVER EARLY IMPLEMENTATION PROJECT ENVIRONMENTAL IMPACT STATEMENT/ ENVIRONMENTAL IMPACT REPORT

September 9, 2013

Prepared by: Mike Archer, P.E.

Reviewed by:

1.0 Background

The hydraulic impacts presented in the MBK Engineers (MBK) technical memorandum "Hydraulic Impact Analysis for the Southport Sacramento River Early Implementation Project Environmental Impact Statement/Environmental Impact Report", dated July 31, 2013, are from hydraulic analyses performed with a HEC-RAS 1-dimensional hydraulic model of the Sacramento River watershed (see Figure 1). Three of the five alternatives evaluated in the Environmental Impact Statement/Environmental Impact Report (EIS/EIS) include levee setbacks: Alternatives 2, 4, and 5 (see Figures 2, 3, and 4).

During the preparation of the EIS/EIR, a 2-dimensional (2-D) hydraulic model of the preferred levee setback alternative, Alternative 5 in the EIS/EIR, was developed by MBK and utilized in the risk based hydraulic impact analysis prepared for the U.S. Army Corps of Engineers (USACE) Section 408 permit request. The 2-D model was prepared to better analyze the localized hydraulic response of the levee setback project, but due to the complexity of 2-D hydraulic analysis does not extend far beyond the project site (see Figure 5). Due to the limited coverage of the 2-D model, the 1-D model of the Sacramento River watershed that was used for the EIS/EIR hydraulic analysis was also used to support the Section 408 permit request for the evaluation of regional impacts of the levee setback project. Results of the 2-D model analysis, however, indicated that the 1-D model did not adequately simulate the two-dimensional nature of the levee setback project, with water entering and leaving the offset area through breaches in remnant levees or over degraded remnant levees. Therefore, the 1-D model was "refined" to reproduce the project site localized hydraulic impacts through the process outlined below.

2.0 Purpose

The purpose of this technical memorandum is to supplement the hydraulic impact analysis previously performed with results from a refined 1-D hydraulic impact analysis.

3.0 Refined 1-D Hydraulic Analysis Method

The general concept of the refined 1-D hydraulic analysis is that the 2-D model would be used to determine changes of the preferred alternative in the project reach, within the 2-D model domain, and that the 1-D model would be used to determine changes outside of the 2-D model domain, or region. This allows for the most accurate representation of conditions for determining results.

The Southport EIP project is located 3.2 miles downstream of the confluence of the Sacramento and American Rivers (see Figure 6). If the project were to affect the stage in the Sacramento River downstream of the American River, hydraulic impacts could extend upstream of the American River and downstream of the project site. A reduction in stage would increase the flow in the Sacramento River downstream of the American River.

Due to the complexity of 2-D hydraulic analysis, the 2-D model does not include the Sacramento and American River confluence. The upstream boundary of the 2-D model is located at the I Street gage, 900 feet upstream of the I Street bridge, approximately 0.6 mile downstream of the American River, and 2.6 miles upstream of the project site. The downstream boundary of the 2-D model is located at the Freeport bridge, approximately 5.3 miles downstream of the project site.

The following steps outline the coordination of the 1-D and 2-D models utilized for the refined 1-D hydraulic impact analysis.

1. The 1-D model was used to develop the relationship between change in stage and change in flow in Sacramento River at the upstream boundary of the 2-D model.

2. The 2-D model was iterated by adjusting the upstream input flow in the project condition model until the change in stage and change in flow at its upstream boundary due to the addition of the levee setback matched the relationship determined in Step 1.

3. Adjustments were made to the setback area in the 1-D model, primarily through the use of ineffective flow area designation, until the results of the 1-D model at the I Street gage matched those of the 2-D model from Step 2. Step 3 is the “refinement” of the levee setback area in the 1-D model.

4.0 Application of Refined 1-D Hydraulic Analysis to EIS/EIR Hydraulic Analysis

The refined 1-D hydraulic analysis evaluated only one levee setback configuration, whereas the EIS/EIR hydraulic analysis evaluated three levee setback configurations. The levee setback evaluated in the refined 1-D analysis was the same as Alternative 5 in the EIS/EIR analysis. Review of the results from the EIS/EIR analysis for the three levee setback alternatives show that the computed effects on the maximum water surface elevation are very similar for all three levee setback alternatives (see Figures 7, 8, and 9). There is very little difference in the computed impacts for Alternatives 2 and 5, therefore the refined 1-D analysis of the impacts of Alternative 5 are also representative of the impacts of Alternative 2. The small (+/- 0.01 ft. to 0.03 ft.) but consistent difference between Alternatives 4 and 5 upstream and downstream of the levee setback are the result of the overestimation of the impacts to the Sacramento River-American River flow split in the EIS/EIR analysis, and therefore would not be applicable to the refined 1-D analysis. The EIS/EIR analysis, however, does show a localized increase in impact in Segment F (between river miles 56 and 57) in Alternatives 2 and 5 that is not present in Alternative 4. This localized impact is also apparent in the 408 analysis (see Figures 10, 11, and 12). It can be concluded that the computed impacts from the 408 analysis are representative of the impacts of Alternative 4 with the exception of the localized impact shown between river miles 56 and 57, where the impact should be closer to zero.

The refined 1-D hydraulic impact analysis assumed that the Folsom Joint Federal Project (JFP) was in place. Modeling results from the EIS/EIR analysis indicate that results are nearly identical for the with- and without- JFP conditions. As such, for the refined 1-D analysis, we can also assume that the results would be nearly identical for the with- and without-JFP conditions.

A summary comparison of the computed impacts of Alternative 5 from the EIS/EIR analysis and the refined 1-D analysis is provided in Table 1.

Table 1. Comparison of Computed Maximum Water Surface Elevations

Location	Comp Study River Mile	NEPA/CEQA (1-D)			408 (1-D & 2-D)		
		Maximum Water Surface Elevation (ft NAVD88)			Maximum Water Surface Elevation (ft NAVD88)		
		Without Setback[1] (with JFP)	With Setback[2] (with JFP)	**Change (ft)**	Without Setback[3] (with JFP)	With Setback[3] (with JFP)	**Change (ft)**
100-yr							
SR4 (I Street)	59.6915	34.39	34.35	**-0.04**	34.41	34.40	**-0.01**
SR5 (u/s end of Project)	57.0026	33.20	33.13	**-0.07**	33.21	33.20	**-0.01**
SR6 (nr Davis Rd.)	54.7464	32.32	32.34	**+0.02**	32.33	32.34	**0**
SR7 (d/s end of Project)	51.7539	30.89	31.03	**+0.14**	30.91	30.92	**+0.01**
SR8 (Babel Slough)	49.9997	30.23	30.36	**+0.13**	30.24	30.26	**+0.01**
SR9 (Freeport Bridge)	46.4268	28.56	28.69	**+0.13**	28.58	28.60	**+0.01**
SR10 (Walnut Grove)	26.7501	17.17	17.22	**+0.05**	17.18	17.19	**0**
Max Change	54.0001	31.83	32.12	**+0.29**	31.32[4]	31.45[4]	**+0.13**[4]
200-yr							
SR4 (I Street)	59.6915	36.24	36.20	**-0.04**	36.27	36.28	**+0.01**
SR5 (u/s end of Project)	57.0026	35.02	34.94	**-0.08**	35.04	35.05	**+0.01**
SR6 (nr Davis Rd.)	54.7464	34.11	34.13	**+0.02**	34.14	34.11	**-0.02**
SR7 (d/s end of Project)	51.7539	32.63	32.78	**+0.15**	32.66	32.64	**-0.02**
SR8 (Babel Slough)	49.9997	31.96	32.10	**+0.14**	31.98	31.96	**-0.02**
SR9 (Freeport Bridge)	46.4268	30.21	30.34	**+0.13**	30.23	30.22	**-0.02**
SR10 (Walnut Grove)	26.7501	18.05	18.10	**+0.05**	18.06	18.05	**-0.01**
Max Change	54.0001	33.59	33.91	**+0.32**	33.21[5]	33.38[5]	**+0.17**[5]
500-yr							
SR4 (I Street)	59.6915	38.51	38.43	**-0.08**	38.39	38.40	**+0.02**
SR5 (u/s end of Project)	57.0026	37.11	36.98	**-0.13**	37.02	37.06	**+0.04**
SR6 (nr Davis Rd.)	54.7464	36.06	36.04	**-0.02**	35.97	35.94	**-0.03**
SR7 (d/s end of Project)	51.7539	34.31	34.42	**+0.11**	34.24	34.20	**-0.04**
SR8 (Babel Slough)	49.9997	33.58	33.67	**+0.09**	33.52	33.48	**-0.04**
SR9 (Freeport Bridge)	46.4268	31.63	31.70	**+0.07**	31.58	31.55	**-0.03**
SR10 (Walnut Grove)	26.7501	18.85	18.88	**+0.03**	18.90	18.90	**-0.01**
Max Change	54.0001	35.43	35.79	**+0.36**	35.49[6]	35.76[6]	**+0.27**[6]

[1] Scenario: "No Action (Future without Setback)". Includes Folsom Joint Federal Project (JFP) and 200-year urban levees.
[2] Scenario: "Future with Alt. 5". Includes Folsom JFP and 200-year urban levees.
[3] Includes Folsom JFP, 200-year urban levees, and 1957 profile levee raises.
[4] Max change from 408 analysis is at RM 53.66.
[5] Max change from 408 analysis is at RM 53.87.
[6] Max change from 408 analysis is at RM 54.07.

5.0 Conclusion

The changes in stage of the Southport EIP preferred alternative (Alternative 5) computed for the refined 1-D hydraulic analysis are representative of the expected impacts of Alternative 2. They are also representative of the expected impacts of Alternative 4 with the exception of the localized impact between river miles 56 and 57. This localized impact, which is due to the levee setback upstream of Bee’s Lake, would not be expected to occur in Alternative 4 since the levee setback upstream of Bee’s Lake is not included in Alternative 4.

FIGURES

LIST OF FIGURES

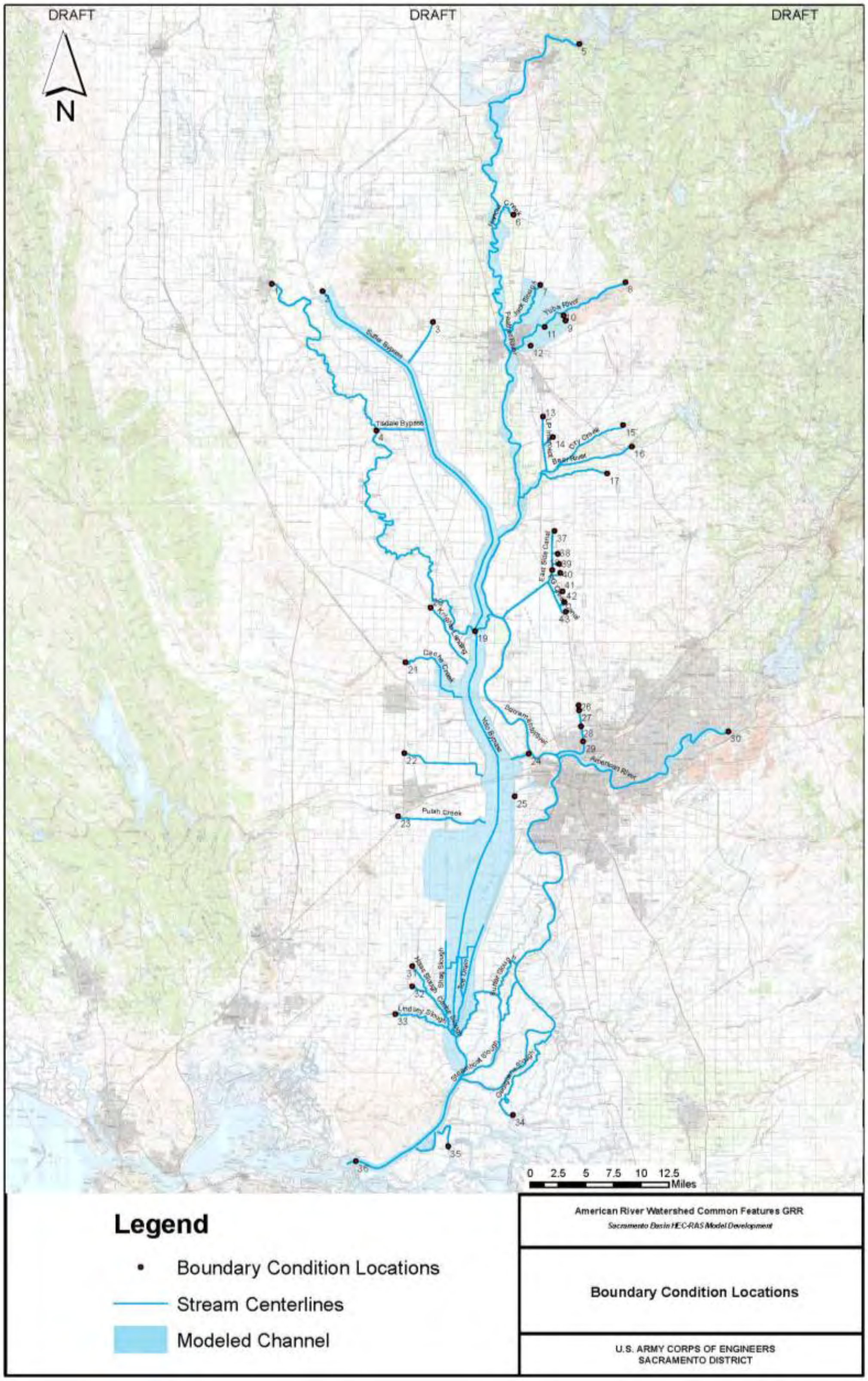

Figure 1. USACE Sacramento River 1-D HEC-RAS Model Extents (Source: USACE)

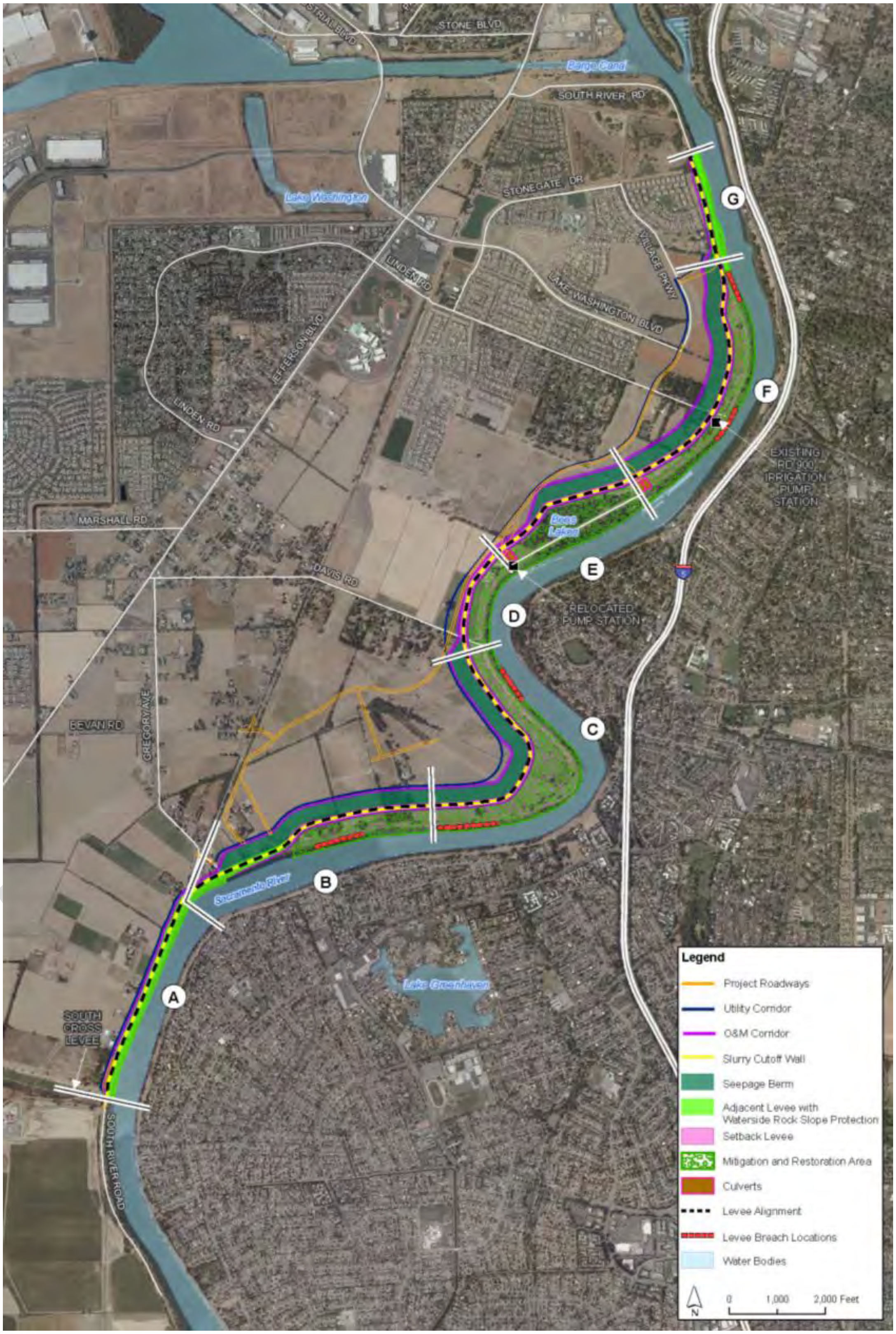

Figure 2. With Project, Alternative 2 (Source: ICF International)

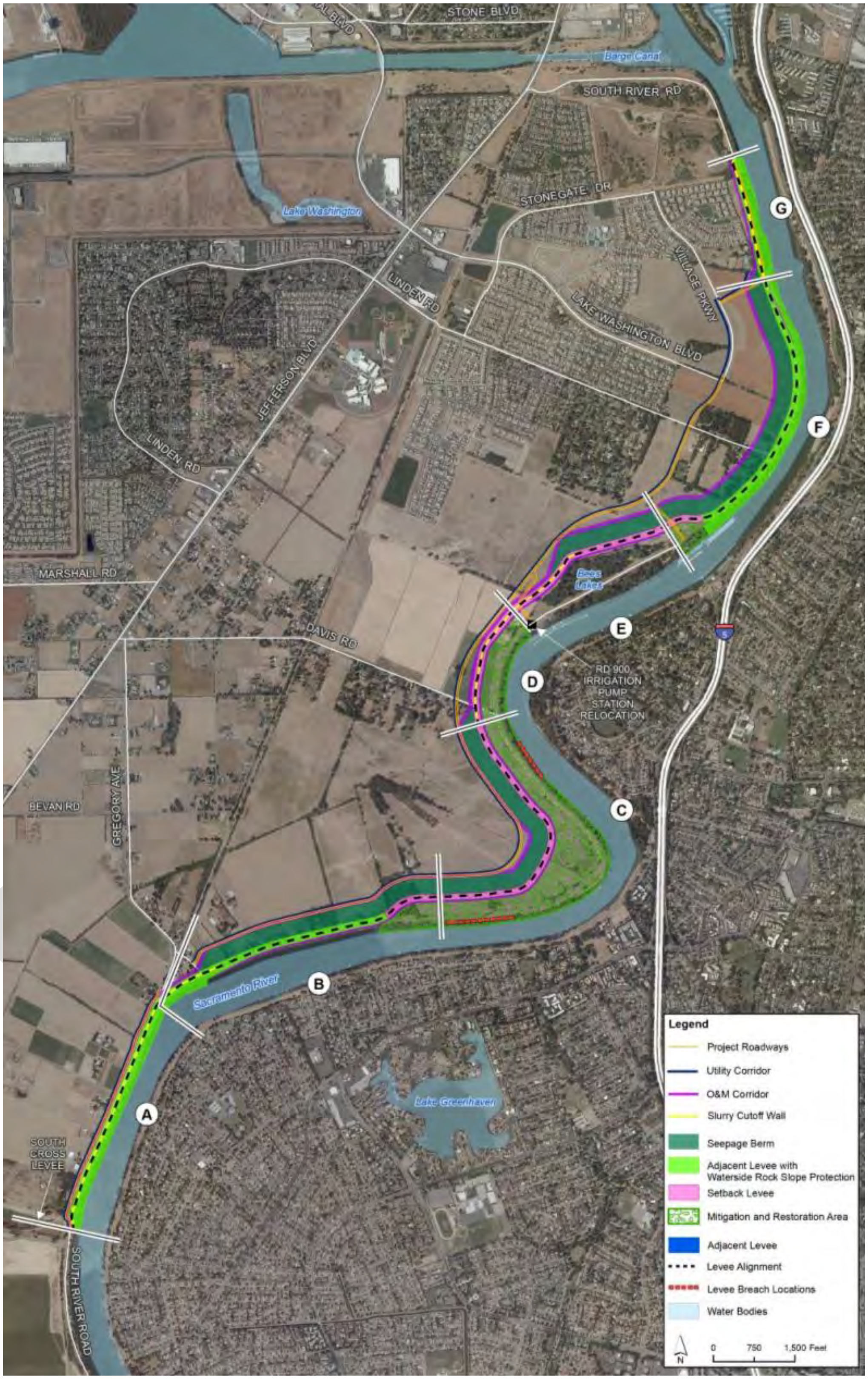

Figure 3. With Project, Alternative 4 (Source: ICF International)

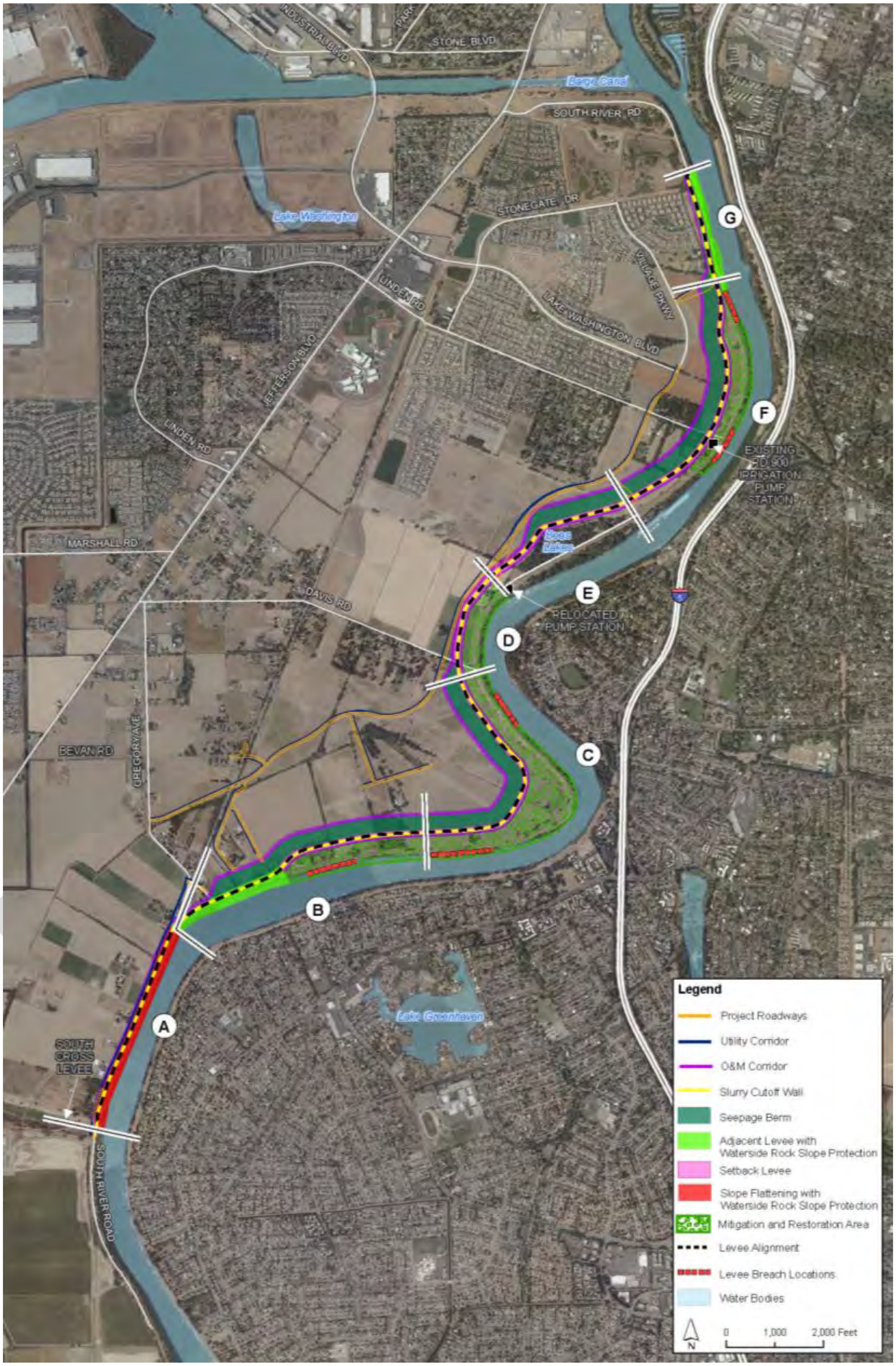

Figure 4. With Project, Alternative 5 (Source: ICF International)

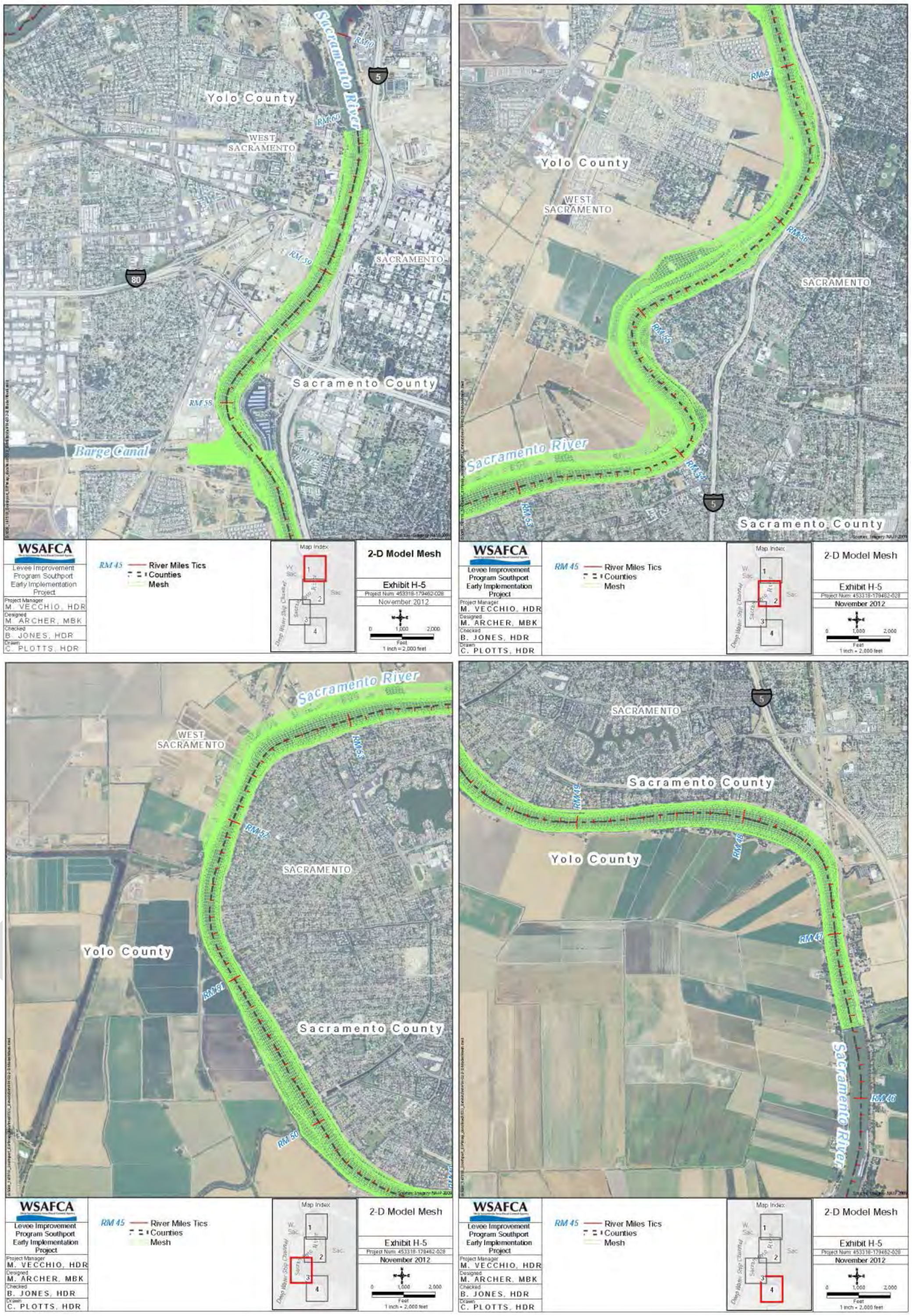

Figure 5. 2-D Hydraulic Model Extents

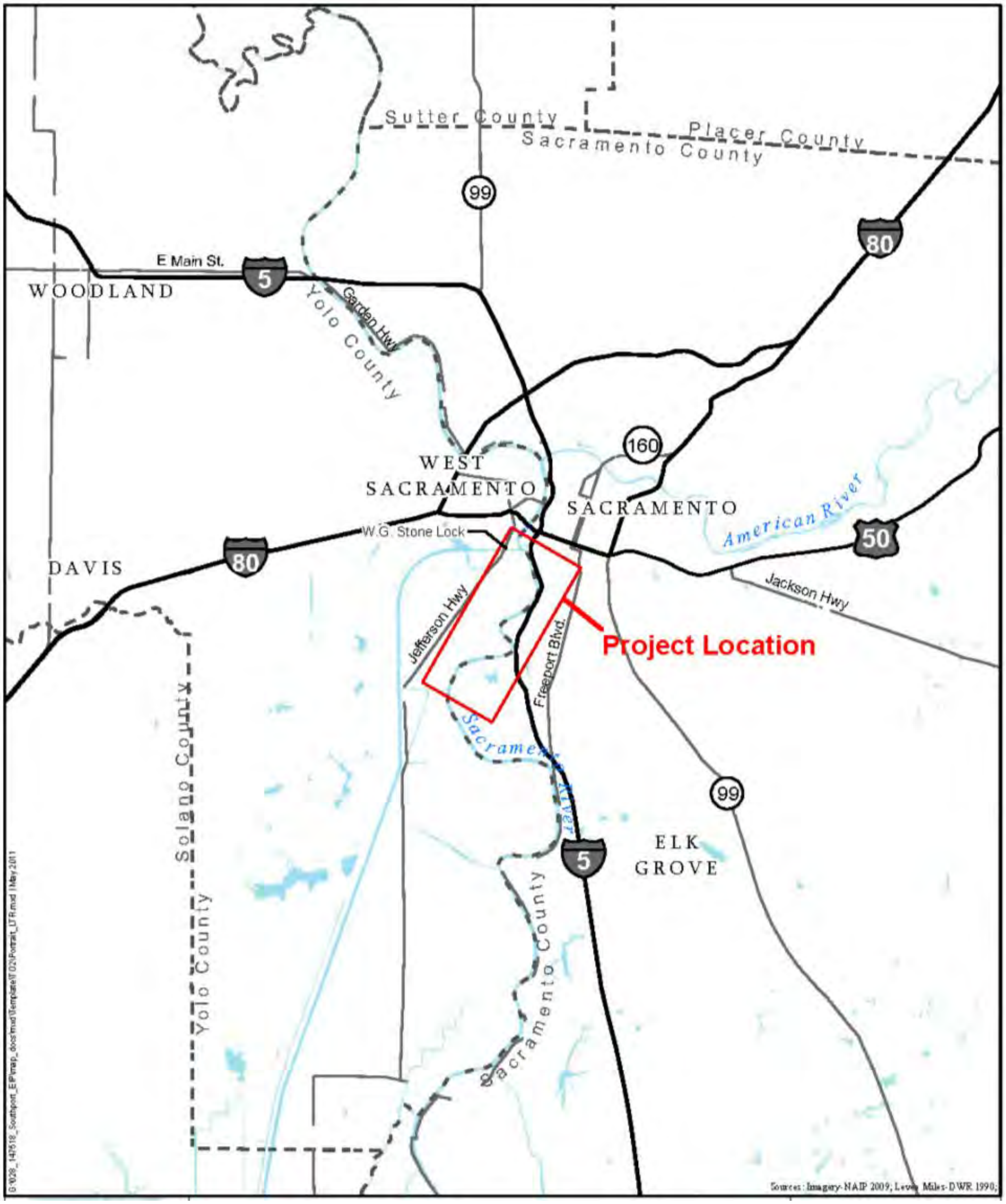

Figure 6. Project Location Map

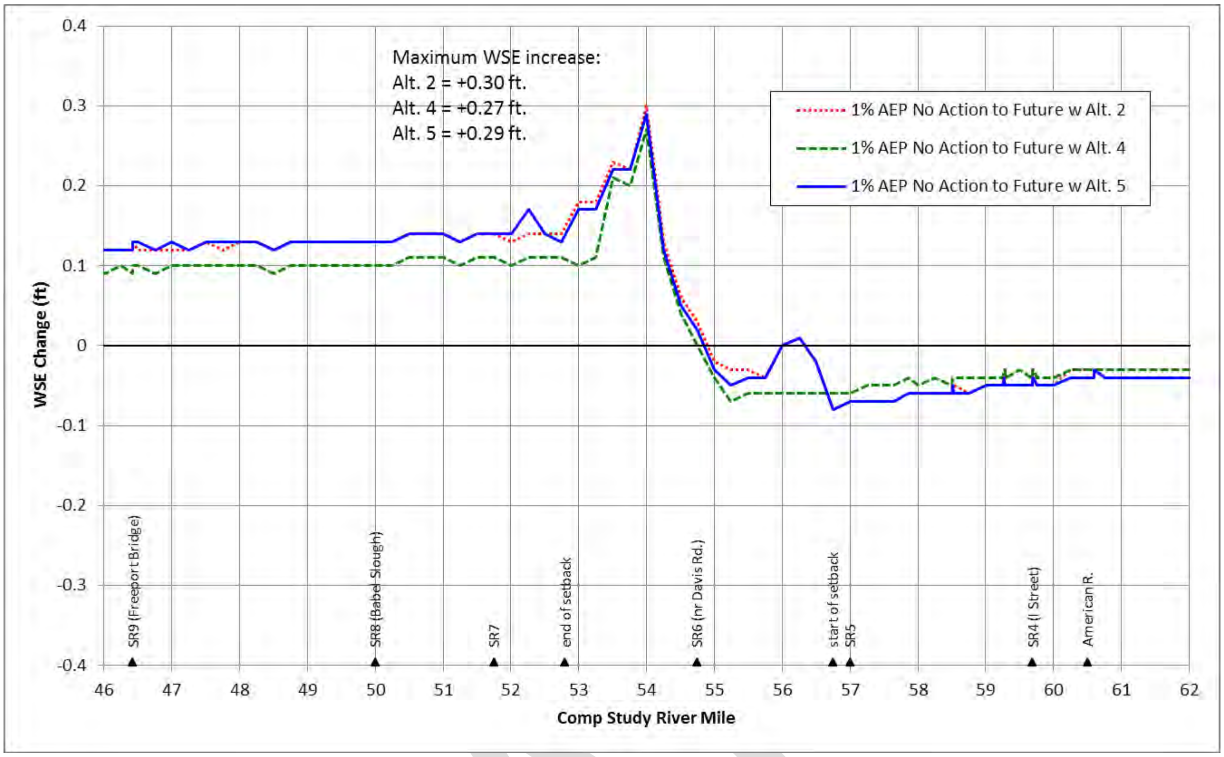

Figure 7. Computed Water Surface Elevation (WSE) change from EIS/EIR hydraulic impact analysis, Alternatives 2, 4, and 5, 1% Annual Exceedance Probability (AEP)

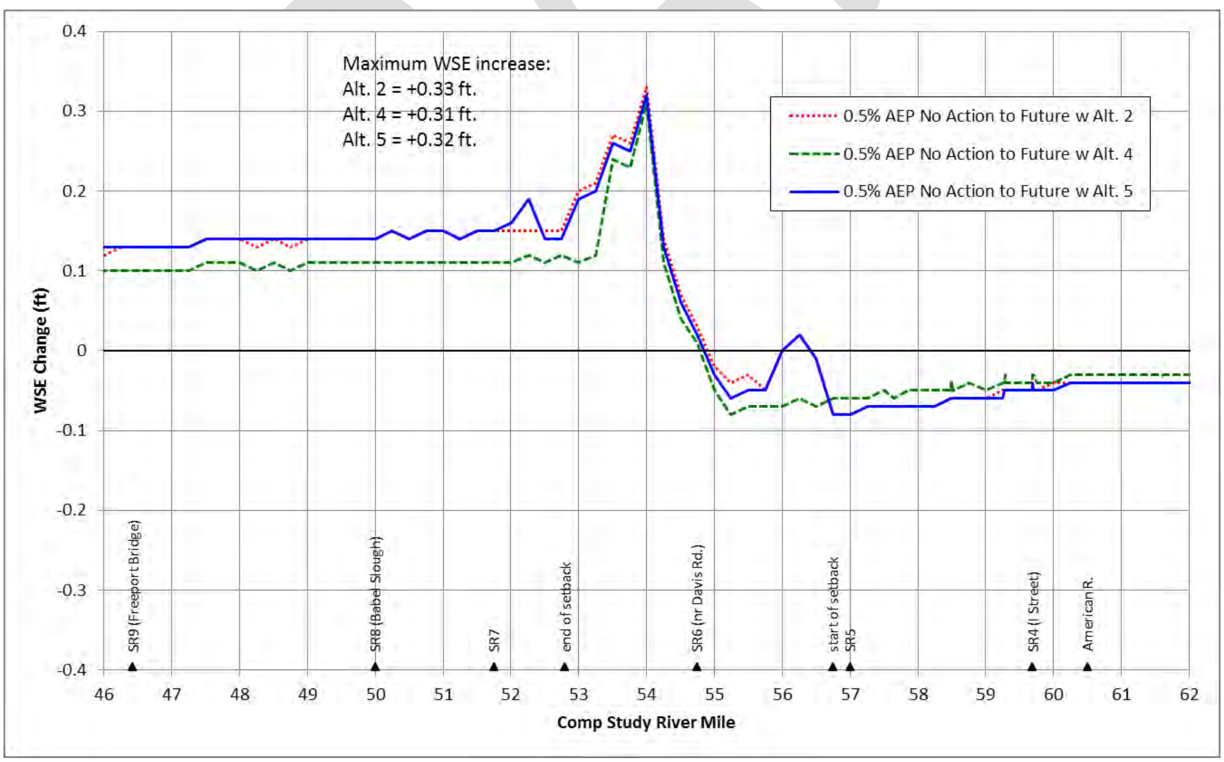

Figure 8. Computed Water Surface Elevation (WSE) change from EIS/EIR hydraulic impact analysis, Alternatives 2, 4, and 5, 0.5% Annual Exceedance Probability (AEP)

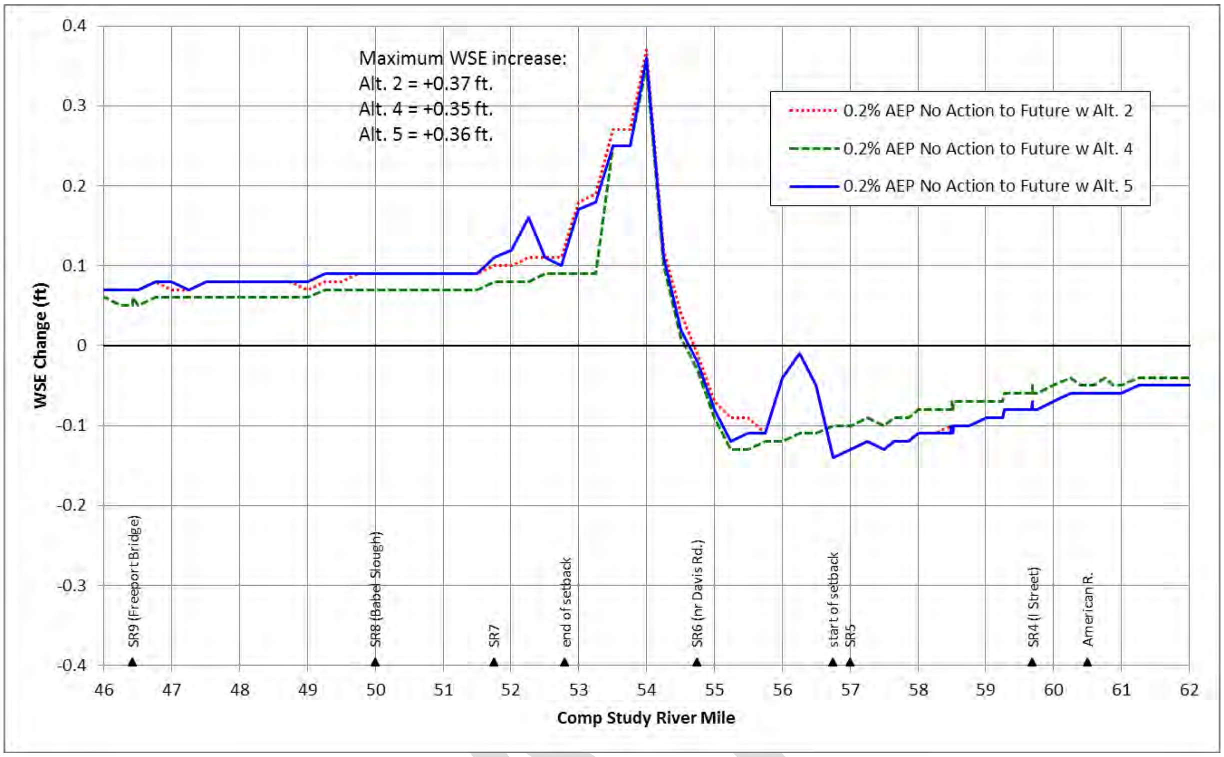

Figure 9. Computed Water Surface Elevation (WSE) change from EIS/EIR hydraulic impact analysis, Alternatives 2, 4, and 5, 0.2% Annual Exceedance Probability (AEP)

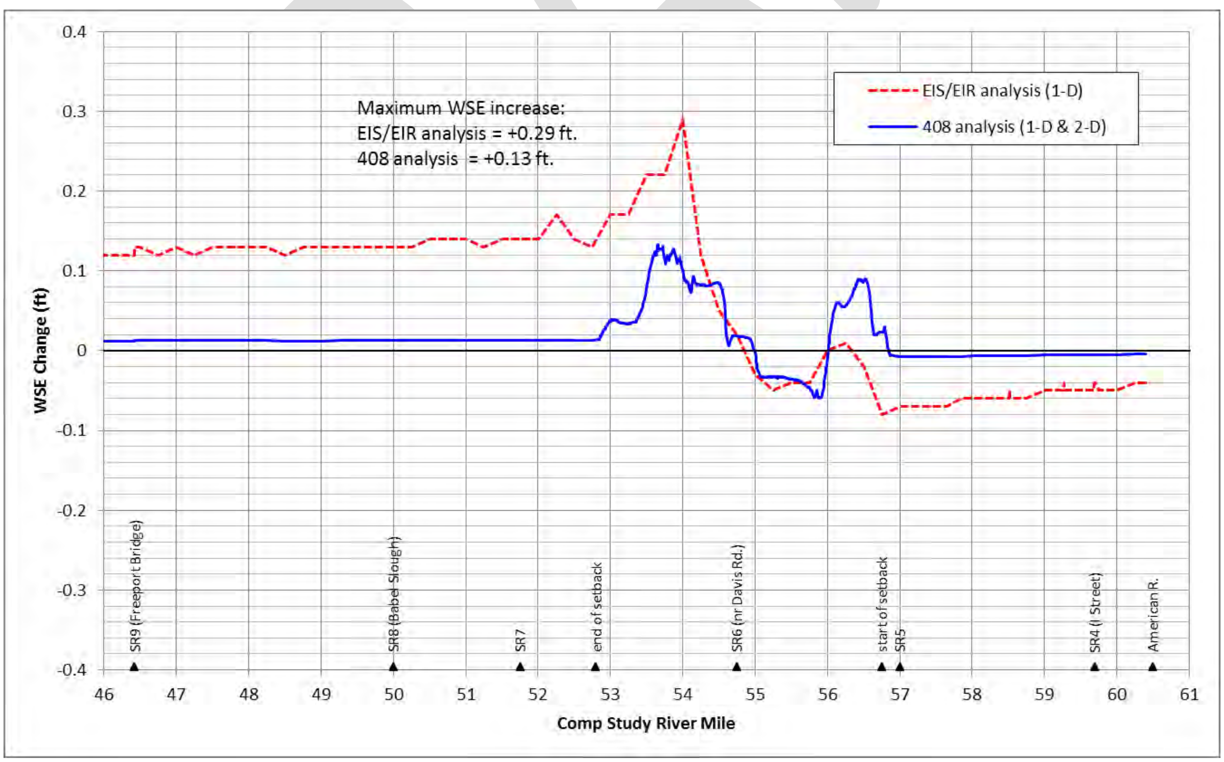

Figure 10. Computed Water Surface Elevation (WSE) change, 408 Analysis and EIS/EIR Analysis; Preferred Alternative (Alternative 5), 1% Annual Exceedance Probability (AEP)

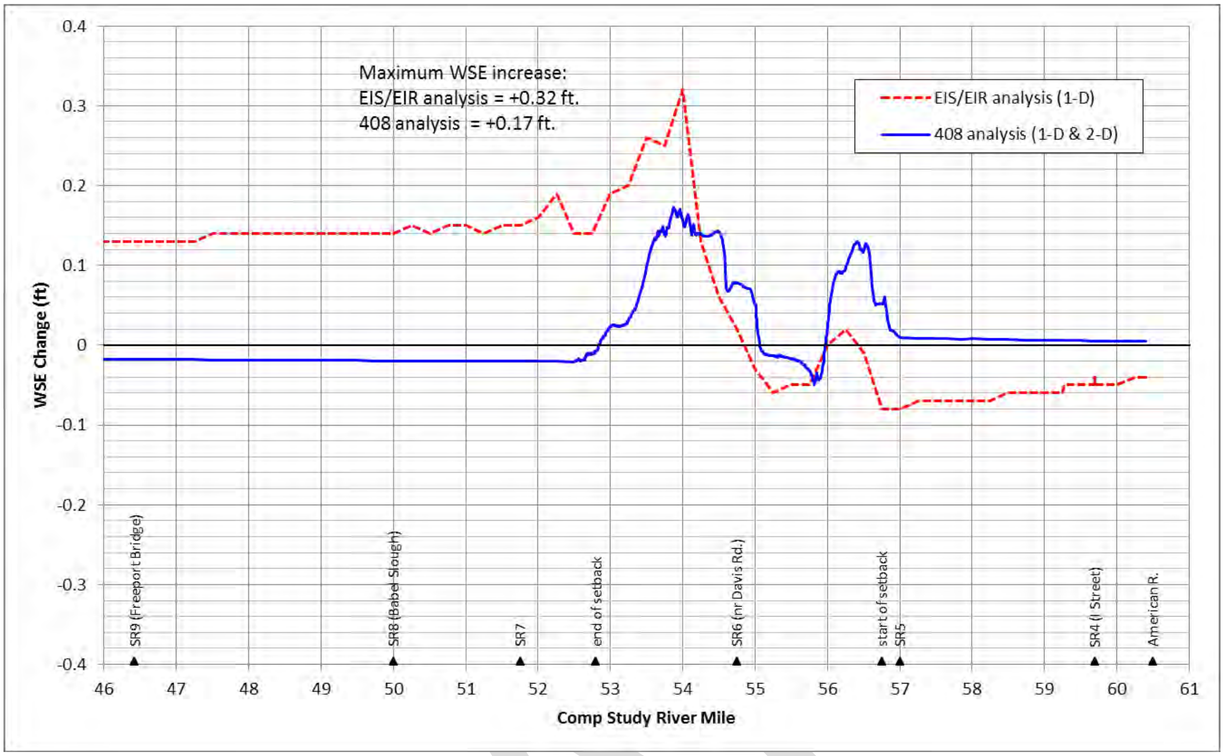

Figure 11. Computed Water Surface Elevation (WSE) change, 408 Analysis and EIS/EIR Analysis; Preferred Alternative (Alternative 5), 0.5% Annual Exceedance Probability (AEP)

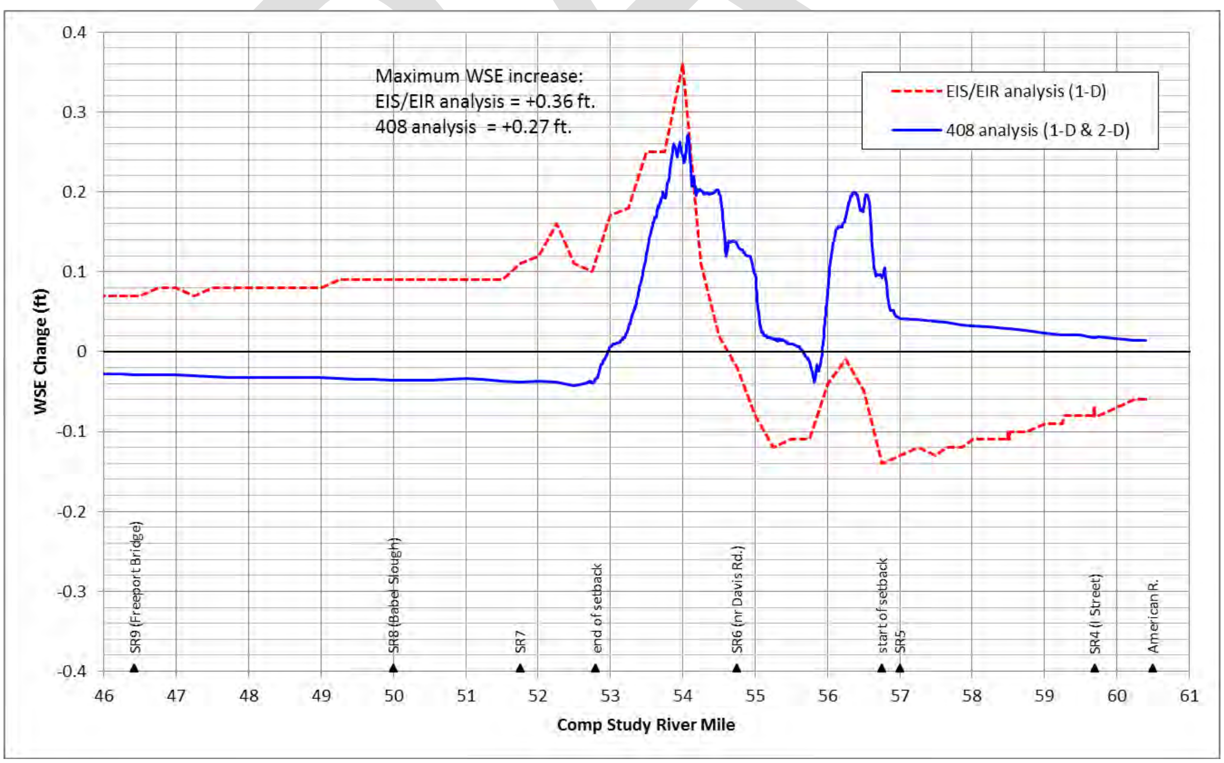

Figure 12. Computed Water Surface Elevation (WSE) change, 408 Analysis and EIS/EIR Analysis; Preferred Alternative (Alternative 5), 0.2% Annual Exceedance Probability (AEP)

Appendix B.2

Supplemental Memo: Refined Hydraulic Impact Analysis for the Southport Sacramento River Early Implementation Project Environmental Impact Statement/ Environmental Impact Report—MBK Engineers, September 9, 2013

MBK ENGINEERS

Hydraulic Impact Analysis For the Southport Sacramento River Early Implementation Project Environmental Impact Statement/ Environmental Impact Report

July 26, 2013

Prepared by: Mike Archer, P.E.

Reviewed by: Don Trieu, P.E.

1.0 Background

As part of the Southport Sacramento River Early Implementation Project (Project), the West Sacramento Area Flood Control Agency (WSAFCA) is studying five alternative projects with the purpose of achieving 0.5% (1 in 200) Annual Exceedance Probability (AEP) flood protection for a 5.6 mile reach of Sacramento River levee in the Southport project area. The Project area extends from about 0.4 miles downstream of the W.G. Stone Lock to the South Cross Levee (see Figures 1 and 2). The alternatives are described in detail in Section 6 of this Technical Memorandum. MBK Engineers (MBK) has performed a hydraulic impact analysis of the proposed alternatives, which is presented herein.

2.0 Purpose

The purpose of the hydraulic impact analysis is to determine impacts to water surface elevations and flows as a result of the proposed Project. The analysis is needed to satisfy the requirements of the National Environmental Policy Act (NEPA) and the California Environmental Quality Act (CEQA) for disclosing environmental effects and recommended mitigation measures related to a proposed action.

The hydraulic impacts of the levee alterations proposed as part of the Project were evaluated for the following flood events:

- **1% (1 in 100) AEP**: approximating the conditions associated with the Sacramento River Flood Control Project's (SRFCP) 1957 water surface profiles that serve as the minimum design standard for the SRFCP and the base flood elevations that govern management of SRFCP protected floodplains under the National Flood Insurance Program,

- **0.5% (1 in 200) AEP**: approximating the conditions associated with the recently adopted State of California Urban Levee Design Criteria (ULDC),

- **0.2% (1 in 500) AEP**: representing an extreme flood event; the largest flood event for which hydrologic input data has been developed for the hydraulic simulation model.

Each of the above flood events was evaluated for the following conditions:

- **Existing**: The levee system and reservoir operation criteria as existed in January 2013.

- **Current With Project**: Existing condition with each of the proposed Project alternatives. Each Project alternative is evaluated separately.

- **No Action (Future Without Project)**: Likely future conditions without Project. Assumes implementation of the Federally authorized improvements to Folsom Dam (also known as the Folsom Joint Federal Project [JFP], or Folsom JFP), and anticipated improvements to levees protecting existing urban areas so as to provide those areas with 0.5% AEP flood protection.

- **Future With Project**: No Action condition with the addition of each of the proposed Project alternatives. Each Project alternative is evaluated separately.

3.0 Hydraulic Model

Release 4 of the U.S. Army Corps of Engineers (USACE) Sacramento River Basin HEC-RAS hydraulic simulation model (Model) was used for this analysis. The USACE release memo for the Model is provided in Appendix A. HEC-RAS is software designed to perform one-dimensional hydraulic calculations for a full network of natural and constructed channels. It was developed by the USACE Hydrologic Engineering Center. Version 4.1 of HEC-RAS has been utilized for the analysis documented herein.

The extents of the Model are shown in Figure 3. It includes the Sacramento River from Colusa to Collinsville in the Sacramento-San Joaquin Delta, the Sutter and Yolo Bypasses, the Feather River below Oroville Dam, the American River below Folsom Dam, and other major tributaries and distributaries.

For this analysis, the Model was modified to include the recently constructed USACE Sacramento River Bank Protection Project Erosion Repair Site River Mile (RM) 57.2R (see Figure 4).

4.0 Hydrology

As part of the Sacramento and San Joaquin River Basins Comprehensive Study, USACE developed hydrologic input data for the Model for numerous storm "centerings." The centerings relied on historical storm patterns in the upstream basin to define the shape and magnitude of the flow contributions from each of the basins, and were designed to stress specific locations within the system. The hydrologic data sets associated with the storm centerings designed to stress the SRFCP at the latitude of Sacramento (Sacramento Centering) and at Folsom Dam (American River Centering) were used for this analysis.

The USACE hydrologic input data included seven flood events: 50% AEP, 10% AEP, 4% AEP, 2% AEP, 1% AEP, 0.5% AEP, and 0.2% AEP. As noted previously in Section 2, the analysis presented herein used the 1% AEP, 0.5% AEP, and 0.2% AEP flood events.

The hydrology of the American River in the Model, representing Folsom Dam releases, differs between the Existing Condition and Without/With Project Condition due to the Folsom JFP. The effect of the Folsom JFP on the peak American River flow in the analyzed flood events is summarized in Table 1.

Table 1. Peak Folsom Dam Release to American River in Analyzed Flood Events

	Peak Flow (cfs)	
Flood Event	**Without JFP**	**With JFP**
1% AEP	145,000	115,000
0.5% AEP	321,000	160,000
0.2% AEP	513,200	405,500

cfs – cubic feet per second

5.0 Levee Performance Assumptions

An important assumption in performing hydraulic simulations of leveed systems on a regional basis is defining if, when, and how levee failures will occur. The analysis presented herein assumed levees would act as weirs when overtopped and not degrade or fail.

6.0 Project Alternatives

6.1 Alternative 1 - Adjacent Levee

Alternative 1 involves the construction of an adjacent levee landward of the Sacramento River levee (see Figure 5). At Bees Lakes, the new levee will not be adjacent to the existing levee, but rather will be setback to the west side of Bees Lakes. This alternative makes no alterations on the river side of the existing levee, therefore, no modification of hydraulic model cross-sections was required. The alternative includes vegetation removal, the extent and degree of which is not known with any detail at this time. For the purpose of this analysis, vegetation removal was conservatively represented in the hydraulic model by reducing the Manning's n-value roughness coefficient by almost 25%, from 0.033 to 0.025, on the right bank of the Sacramento River for the entire length of the Project reach.

6.2 Alternative 2 – Setback Levee with Bees Lakes Flow-through

Alternative 2 involves the construction of a setback levee just over four miles long with a typical offset distance of approximately 400 feet (see Figure 6). The existing levee will be partially degraded for its entire length and breached at five locations to allow water to flow into and out of the offset area. The levee setback was incorporated into the Model by modifying the affected cross-sections. Plots of the affected cross-sections showing the modifications are provided in Appendix B. Manning's n-value roughness coefficients in the offset area were based on a proposed planting plan and corresponding mature condition Manning's n-values described in a memorandum prepared by cbec eco engineering (cbec) with subject "Southport EIP – Roughness Value Development for the Offset Area under Interim and Mature Vegetative Conditions (DRAFT)," dated 8/28/12. Where the cbec memo provides n-value ranges, the highest value in the range was used. The n-values used in the Model in the offset area ranged from 0.035 to 0.150, with a weighted mean of about 0.11. Road embankments at the same elevation as the setback levee will be constructed at the upstream and downstream ends of the Bees Lakes area to allow for access to marinas on the Sacramento River. Culverts will be installed in the embankments to allow for hydraulic connectivity between the Sacramento River and Bees Lakes. The number of culverts and culvert dimensions are not known at this time. For the analysis, ten 10 foot diameter culverts were assumed for each embankment.

6.3 Alternative 3 – Slope Flattening

Alternative 3 involves flattening the water side slope of the levee to a 3:1 slope throughout the Project reach (see Figure 7). The existing water side slope is approximately 2:1. The slope flattening was incorporated into the Model by modifying the affected cross-sections. Plots of the affected cross-sections showing the modifications are provided in Appendix C. Alternative 3 also includes vegetation removal similar to Alternative 1, modeled as described in Section 6.1.

6.4 Alternative 4 – Blended Setback Levee

Alternative 4 involves the construction of a setback levee about two miles in length starting downstream of Bees Lakes (see Figure 8). The remainder of the Project reach will have adjacent levee similar to Alternative 1, along with the vegetation removal component as described in Alternative 1. Bees Lakes is not hydraulically connected to the Sacramento River. This alternative was incorporated into the model using the cross-sections for River Stations (RS) 53.50 through 55.00 from Alternative 2 (see Appendix B), along with the corresponding offset area n-values which range from 0.035 to 0.150.

6.5 Alternative 5 – Setback Levee with Isolated Bees Lake

Alternative 5 (see Figure 9) is identical to Alternative 2 except for the following:

- There are no culverts in the marina access road embankments upstream and downstream of the Bees Lakes area, removing hydraulic connectivity with the Sacramento River.

- Slope flattening, rather than adjacent levee, is used for about 0.8 miles at the downstream end of the Project reach.

7.0 Results

Impacts to the computed maximum water surface elevations and peak flows have been determined and are discussed below for the following condition changes:

1. Existing to Current With Project
2. Existing to No Action
3. Existing to Future With Project
4. No Action to Future With Project

The computed maximum water surface elevations for the 1% AEP, 0.5% AEP, and 0.2% AEP flood events are provided at several index points in Tables 2 through 4, respectively. The index point locations are shown in Figure 10. The impacts on the maximum water surface elevation of going from Existing to Current With Project, Existing to Future With Project, and No Action to Future With Project are summarized for each Project Alternative separately in Tables 5 through 9. The impact on the maximum water surface elevation of going from Existing to No Action is summarized in Table 10. Profile plots of the computed maximum water surface elevations, along with profile plots showing the impacts, for all affected reaches are provided in Figures 11 through 130. In a similar manner, the computed peak flows for the 1% AEP, 0.5% AEP, and 0.2% AEP flood events are provided at several key locations in Tables 11 through 13, respectively. The impacts on the peak flow of going from Existing to Current With Project, Existing to Future With Project, and No Action to Future With Project are summarized for each Project Alternative separately in Tables 14 through 18. The impact on the peak flow of going from Existing to No Action is summarized in Table 19. The computed impacts are discussed in the following sections.

There are no noticeable effects on the flood duration and for all of the Project Alternatives and flood frequencies. There are also no significant effects on the computed mean velocities.

The project is located about 3.6 miles downstream of the American River. During the large flood events evaluated herein, the Sacramento Weir gates are open and flows from the American River are split with some heading upstream to the Sacramento Weir and the remainder heading downstream toward the Project site. If the Project results in a water surface elevation change in the Sacramento River, the American River flow split can be effected. If the water surface elevation is lower, the flow heading downstream would increase and the flow heading to the Sacramento Weir would decrease. Additionally, the Sacramento Weir gate operation is tied to the stage in the Sacramento River at I Street gage which is located about 0.7 miles downstream of the American River and 2.9 miles upstream of the Project. The hydraulic model accounts for the effects of the Project on the American River flow split and the Sacramento Weir operation. The effect of the Project Alternatives on the American River Flow split and the Sacramento Weir flow can be seen in peak flow impacts shown in Tables 11 through 19.

7.1 Existing to Current With Project

7.1.1 Existing to Current With Alternative 1

The only difference between Existing and Alternative 1 is vegetation removal on the right bank throughout the Project reach, which is represented in the analysis by a 25% reduction in the Manning's n-value roughness coefficient on the right bank. This alternative has no measurable impact to the peak stage or peak flow in any of the events analyzed (see columns [1], [4], and [7] in Tables 5 and 14).

7.1.2 Existing to Current With Alternative 2

Alternative 2 consists of a four mile long levee setback with adjacent levee and vegetation removal for the remainder of the Project reach. The Bees Lakes area is hydraulically connected to the Sacramento River with culverts. The computed impacts are summarized in Tables 6 and 15.

7.1.2.1 **1% AEP** (column [1] in Tables 6 and 15)

In the 1% AEP event, Alternative 2 results in a decrease of 0.07 feet in the peak stage at the upstream end of the Project reach (RM 57.00) and an increase of 0.14 feet at the downstream end of the Project reach (RM 51.75). The maximum computed increase in peak stage is 0.31 feet at RM 54.00. Five miles downstream of the Project, at the Freeport bridge, the peak stage increase is 0.13 feet. Twenty-five miles downstream of the Project, at Walnut Grove, the peak stage increase is 0.05 feet. The increase in stage downstream of the Project is the result of a 0.9% increase in the peak flow in the Sacramento River below the American River, from

126,000 cfs to 127,100 cfs. The increase in flow is due to the effect of the peak stage decrease upstream of the Project on the flow split at the confluence of the Sacramento and American Rivers. The impact to peak stages at index points on the Yolo Bypass, Sacramento Bypass, and Sacramento River Deep Water Ship Channel is negligible, ranging from -0.01 feet to -0.03 feet.

7.1.2.2 0.5% AEP (column [4] in Tables 6 and 15)

In the 0.5% AEP event, Alternative 2 results in a decrease of 0.11 feet in the peak stage at the upstream end of the Project reach (RM 57.00), and an increase of 0.11 feet at the downstream end of the Project reach (RM 51.75). The maximum computed increase in peak stage is 0.34 feet at RM 54.00. Five miles downstream of the Project, at the Freeport bridge, the peak stage increase is 0.09 feet. Twenty-five miles downstream of the Project, at Walnut Grove, the peak stage increase is 0.05 feet. The increase in stage downstream of the Project is the result of a 1.1% increase in the peak flow in the Sacramento River below the American River, from 149,200 cfs to 150,900 cfs. The increase in flow is due to the effect of the peak stage decrease upstream of the Project on the flow split at the confluence of the Sacramento and American Rivers. The impact to peak stages at index points on the Yolo Bypass, Sacramento Bypass, and Sacramento River Deep Water Ship Channel is negligible, ranging from zero to -0.04 feet.

7.1.2.3 0.2% AEP (column [7] in Tables 6 and 15)

In the 0.2% AEP event, Alternative 2 results in a decrease of 0.12 feet in the peak stage at the upstream end of the Project reach (RM 57.00), and an increase of 0.09 feet at the downstream end of the Project reach (RM 51.75). The maximum computed increase in peak stage is 0.36 feet at RM 54.00. Five miles downstream of the Project, at the Freeport bridge, the peak stage increase is 0.07 feet. Twenty-five miles downstream of the Project, at Walnut Grove, the peak stage increase is 0.02 feet. The increase in stage downstream of the Project is the result of a 1.2% increase in the peak flow in the Sacramento River below the American River, from 163,600 cfs to 165,500 cfs. The increase in flow is due to the effect of the peak stage decrease upstream of the Project on the flow split at the confluence of the Sacramento and American Rivers. The impact to peak stages at index points on the Yolo Bypass, Sacramento Bypass, and Sacramento River Deep Water Ship Channel is negligible, ranging from zero to -0.03 feet.

7.1.3 Existing to Current With Alternative 3

Alternative 3 involves slope flattening on the water side of the existing levee and vegetation removal. This alternative has no measurable impact to the peak stage or peak flow in any of the events analyzed (see columns [1], [4], and [7] in Tables 7 and 16).

7.1.4 Existing to Current With Alternative 4

Alternative 4 consists of a two mile long levee setback starting downstream of Bees Lakes with adjacent levee and vegetation removal for the remainder of the Project reach. The Bees Lakes area is not hydraulically connected to the Sacramento River. The computed impacts are summarized in Tables 8 and 17.

7.1.4.1 **1% AEP** (column [1] in Tables 8 and 17)

In the 1% AEP event, Alternative 4 results in a decrease of 0.06 feet in the peak stage at the upstream end of the Project reach (RM 57.00), and an increase of 0.11 feet at the downstream end of the Project reach (RM 51.75). The maximum computed increase in peak stage is 0.28 feet at RM 54.00. Five miles downstream of the Project, at the Freeport bridge, the peak stage increase is 0.10 feet. Twenty-five miles downstream of the Project, at Walnut Grove, the peak stage increase is 0.04 feet. The increase in stage downstream of the Project is the result of a 0.6% increase in the peak flow in the Sacramento River below the American River, from 126,000 cfs to 126,800 cfs. The increase in flow is due to the effect of the peak stage decrease upstream of the Project on the flow split at the confluence of the Sacramento and American Rivers. The impact to peak stages at index points on the Yolo Bypass, Sacramento Bypass, and Sacramento River Deep Water Ship Channel is negligible, ranging from zero to -0.02 feet.

7.1.4.2 **0.5% AEP** (column [4] in Tables 8 and 17)

In the 0.5% AEP event, Alternative 4 results in a decrease of 0.09 feet in the peak stage at the upstream end of the Project reach (RM 57.00), and an increase of 0.09 feet at the downstream end of the Project reach (RM 51.75). The maximum computed increase in peak stage is 0.32 feet at RM 54.00. Five miles downstream of the Project, at the Freeport bridge, the peak stage increase is 0.07 feet. Twenty-five miles downstream of the Project, at Walnut Grove, the peak stage increase is 0.04 feet. The increase in stage downstream of the Project is the result of a 0.9% increase in the peak flow in the Sacramento River below the American River, from 149,200 cfs to 150,500 cfs. The increase in flow is due to the effect of the peak stage decrease upstream of the Project on the flow split at the confluence of the Sacramento and American Rivers. The impact to peak stages at index points on the Yolo Bypass, Sacramento Bypass, and Sacramento River Deep Water Ship Channel is negligible, ranging from zero to -0.03 feet.

7.1.4.3 **0.2% AEP** (column [7] in Tables 8 and 17)

In the 0.2% AEP event, Alternative 4 results a decrease of 0.10 feet in the peak stage at the upstream end of the Project reach (RM 57.00), and an increase of 0.07 feet at the downstream end of the Project reach (RM 51.75). The maximum computed increase in peak stage is 0.34 feet at RM 54.00. Five miles downstream of the Project, at the Freeport bridge, the peak stage increase is 0.05 feet. Twenty-five miles downstream of the Project, at Walnut Grove, the peak stage increase is 0.02 feet. The increase in stage downstream of the Project is the result of a 0.9% increase in the peak flow in the Sacramento River below the American River, from 163,600 cfs to 165,000 cfs. The increase in flow is due to the effect of the peak stage decrease upstream of the Project on the flow split at the confluence of the Sacramento and American Rivers. The impact to peak stages at index points on the Yolo Bypass, Sacramento Bypass, and Sacramento River Deep Water Ship Channel is negligible, ranging from zero to -0.03 feet.

7.1.5 Existing to Current With Alternative 5

Alternative 5 is similar to Alternative 2 with a four mile long levee setback and adjacent levee with vegetation removal for the remainder of the Project reach. It differs from Alternative 2 in that the Bees Lakes area is not hydraulically connected to the Sacramento River, and the adjacent levee is replaced with slope flattening for about 0.8 miles at the downstream end of the Project reach. The computed impacts are summarized in Tables 9 and 18.

7.1.5.1 1% AEP (column [1] in Tables 9 and 18)

In the 1% AEP event, Alternative 5 results in a decrease of 0.07 feet in the peak stage at the upstream end of the Project reach (RM 57.00), and an increase of 0.14 feet at the downstream end of the Project reach (RM 51.75). The maximum computed increase in peak stage is 0.30 feet at RM 54.00. Five miles downstream of the Project, at the Freeport bridge, the peak stage increase is 0.13 feet. Twenty-five miles downstream of the Project, at Walnut Grove, the peak stage increase is 0.05feet. The increase in stage downstream of the Project is the result of a 0.9% increase in the peak flow in the Sacramento River below the American River, from 126,000 cfs to 127,100 cfs. The increase in flow is due to the effect of the peak stage decrease upstream of the Project on the flow split at the confluence of the Sacramento and American Rivers. The impact to peak stages at index points on the Yolo Bypass, Sacramento Bypass, and Sacramento River Deep Water Ship Channel is negligible, ranging from zero to -0.03 feet.

7.1.5.2 0.5% AEP (column [4] in Tables 9 and 18)

In the 0.5% AEP event, Alternative 5 results in a decrease of 0.11 feet in the peak stage at the upstream end of the Project reach (RM 57.00), and an increase of 0.12 feet at the downstream end of the Project reach (RM 51.75). The maximum computed increase in peak stage is 0.33 feet at RM 54.00. Five miles downstream of the Project, at the Freeport bridge, the peak stage increase is 0.09 feet. Twenty-five miles downstream of the Project, at Walnut Grove, the peak stage increase is 0.05 feet. The increase in stage downstream of the Project is the result of a 1.1% increase in the peak flow in the Sacramento River below the American River, from 149,200 cfs to 150,900 cfs. The increase in flow is due to the effect of the peak stage decrease upstream of the Project on the flow split at the confluence of the Sacramento and American Rivers. The impact to peak stages at index points on the Yolo Bypass, Sacramento Bypass, and Sacramento River Deep Water Ship Channel is negligible, ranging from zero to -0.04 feet.

7.1.5.3 0.2% AEP (column [7] in Tables 9 and 18)

In the 0.2% AEP event, Alternative 5 results in a decrease of 0.13 feet in the peak stage at the upstream end of the Project reach (RM 57.00), and an increase of 0.10 feet at the downstream end of the Project reach (RM 51.75). The maximum computed increase in peak stage is 0.35 feet at RM 54.00. Five miles downstream of the Project, at the Freeport bridge, the peak stage increase is 0.07 feet. Twenty-five miles downstream of the Project, at Walnut Grove, the peak stage increase is 0.02 feet. The increase in stage downstream of the Project is the result of a 1.2% increase in the peak flow in the Sacramento River below the American River, from

163,600 cfs to 165,500 cfs. The increase in flow is due to the effect of the peak stage decrease upstream of the Project on the flow split at the confluence of the Sacramento and American Rivers. The impact to peak stages at index points on the Yolo Bypass, Sacramento Bypass, and Sacramento River Deep Water Ship Channel is negligible, ranging from zero to -0.03 feet.

7.2 Existing to No Action

This comparison shows the effects of the Folsom JFP and of bringing all urban levees up to a 0.5% AEP level of protection. As noted in Section 4, the Folsom JFP results in significant reductions in peak American River flows for the flood events analyzed. The computed impacts are summarized in Tables 10 and 19.

7.2.1 **1% AEP** (column [1] in Tables 10 and 19)

In the 1% AEP event, No Action results in a decrease in the computed peak stage at all index points, with a maximum decrease of 0.60 feet at Index Point SR2 (Sacramento River at Sacramento Weir). The peak stage decrease at the Project site ranges from 0.52 feet to 0.55 feet. Peak flow at the Project site decreases 3.0%, from 126,000 cfs to 122,200 cfs.

7.2.2 **0.5% AEP** (column [2] in Tables 10 and 19)

In the 0.5% AEP event, No Action results in a decrease in the computed peak stage at all index points, with a maximum decrease of 1.56 feet at Index Point SR4 (Sacramento River at I Street bridge). The peak stage decrease at the Project site ranges from 1.25 feet to 1.47 feet. Peak flow at the Project site decreases 10.2%, from 149,200 cfs to 134,000 cfs.

7.2.3 **0.2% AEP** (column [4] in Tables 10 and 19)

In the 0.2% AEP event, No Action results in very little change in the computed peak stage at all index points, likely due to extensive levee overtopping, both with and without the Folsom JFP and urban levee raises. The impact ranges from an increase of 0.06 feet to a decrease of 0.06 feet.

7.3 Existing to Future With Project

7.3.1 Existing to Future With Alternative 1

As noted in Section 7.1.1, Alternative 1 by itself has no measurable impact to the peak stage or peak flow in any of the events analyzed. Therefore, the impacts for this scenario, which are shown in columns [2], [5], and [8] in Tables 5 and 14, are due almost entirely to the Folsom JFP and 0.5% AEP urban levees and are essentially the same as those for the Existing to No Action scenario discussed in Section 7.2.

7.3.2 Existing to Future With Alternative 2

Alternative 2 consists of a four mile long levee setback with adjacent levee and vegetation removal for the remainder of the Project reach. The Bees Lakes area is hydraulically connected to the Sacramento River with culverts. The future component for this scenario is the addition of the Folsom JFP and urban levees raised to provide 0.5% AEP level of protection where necessary. The computed impacts are summarized in Tables 6 and 15.

7.3.2.1 1% AEP (column [2] in Tables 6 and 15)

In the 1% AEP event, this scenario results in a decrease of 0.62 feet in the peak stage at the upstream end of the Project reach (RM 57.00), and a decrease of 0.38 feet at the downstream end of the Project reach (RM 51.75). There is no measurable increase in peak stage anywhere in the system. Five miles downstream of the Project, at the Freeport bridge, the peak stage decrease is 0.35 feet. Twenty-five miles downstream of the Project, at Walnut Grove, the peak stage decrease is 0.05 feet. The decrease in stage downstream of the Project is the result of a 2.2% decrease in the peak flow in the Sacramento River below the American River, from 126,000 cfs to 123,200 cfs. The decrease in flow is primarily due to reduced flow in the American River as a result of the Folsom JFP. The impact to peak stages at index points on the Yolo Bypass, Sacramento Bypass, and Sacramento River Deep Water Ship Channel ranges from -0.11 feet to -0.48 feet.

7.3.2.2 0.5% AEP (column [5] in Tables 6 and 15)

In the 0.5% AEP event, this scenario results in a decrease of 1.55 feet in the peak stage at the upstream end of the Project reach (RM 57.00), and a decrease of 1.10 feet at the downstream end of the Project reach (RM 51.75). There is no measurable increase in peak stage anywhere in the system. Five miles downstream of the Project, at the Freeport bridge, the peak stage decrease is 0.93 feet. Twenty-five miles downstream of the Project, at Walnut Grove, the peak stage decrease is 0.27 feet. The decrease in stage downstream of the Project is the result of a 9.4% decrease in the peak flow in the Sacramento River below the American River, from 149,200 cfs to 135,200 cfs. The decrease in flow is primarily due to reduced flow in the American River as a result of the Folsom JFP. The impact to peak stages at index points on the Yolo Bypass, Sacramento Bypass, and Sacramento River Deep Water Ship Channel ranges from -0.13 feet to -0.90 feet.

7.3.2.3 0.2% AEP (column [8] in Tables 6 and 15)

In the 0.2% AEP event, this scenario results in a decrease of 0.09 feet in the peak stage at the upstream end of the Project reach (RM 57.00), and an increase of 0.14 feet at the downstream end of the Project reach (RM 51.75). The maximum computed change in peak stage is +0.41 feet at RM 54.00.. Five miles downstream of the Project, at the Freeport bridge, the peak stage increase is 0.11 feet. Twenty-five miles downstream of the Project, at Walnut Grove, the peak stage increase is 0.02 feet. The impact to peak stages at index points on the Yolo Bypass, Sacramento Bypass, and Sacramento River Deep Water Ship Channel ranges from -0.07 feet to +0.01 feet.

7.3.3 Existing to Future With Alternative 3

As noted in Section 7.1.3, Alternative 3 by itself has no measurable impact to the peak stage or peak flow in any of the events analyzed. Therefore, the impacts for this scenario, which are shown in columns [2], [5], and [8] in Tables 7 and 16, are due almost entirely to the Folsom JFP and 0.5% AEP urban levees and are essentially the same as those for the Existing to No Action scenario discussed in Section 7.2.

7.3.4 Existing to Future With Alternative 4

Alternative 4 consists of a two mile long levee setback starting downstream of Bees Lakes with adjacent levee and vegetation removal for the remainder of the Project reach. The Bees Lakes area is not hydraulically connected to the Sacramento River. The future component for this scenario is the addition of the Folsom JFP and urban levees raised to provide 0.5% AEP level of protection where necessary. The computed impacts are summarized in Tables 8 and 17.

7.3.4.1 **1% AEP** (column [2] in Tables 8 and 17)

In the 1% AEP event, this scenario results in a decrease of 0.61 feet in the peak stage at the upstream end of the Project reach (RM 57.00), and a decrease of 0.41 feet at the downstream end of the Project reach (RM 51.75). There is no measurable increase in peak stage anywhere in the system. Five miles downstream of the Project, at the Freeport bridge, the peak stage decrease is 0.38 feet. Twenty-five miles downstream of the Project, at Walnut Grove, the peak stage decrease is 0.06 feet. The decrease in stage downstream of the Project is the result of a 2.4% decrease in the peak flow in the Sacramento River below the American River, from 126,000 cfs to 123,000 cfs. The decrease in flow is primarily due to reduced flow in the American River as a result of the Folsom JFP. The impact to peak stages at index points on the Yolo Bypass, Sacramento Bypass, and Sacramento River Deep Water Ship Channel ranges from -0.10 feet to -0.47 feet.

7.3.4.2 **0.5% AEP** (column [5] in Tables 8 and 17)

In the 0.5% AEP event, this scenario results in a decrease of 1.53 feet in the peak stage at the upstream end of the Project reach (RM 57.00), and a decrease of 1.14 feet at the downstream end of the Project reach (RM 51.75). There is no measurable increase in peak stage anywhere in the system. Five miles downstream of the Project, at the Freeport bridge, the peak stage decrease is 0.96 feet. Twenty-five miles downstream of the Project, at Walnut Grove, the peak stage decrease is 0.28 feet. The decrease in stage downstream of the Project is the result of a 9.6% decrease in the peak flow in the Sacramento River below the American River, from 149,200 cfs to 134,900 cfs. The decrease in flow is primarily due to reduced flow in the American River as a result of the Folsom JFP. The impact to peak stages at index points on the Yolo Bypass, Sacramento Bypass, and Sacramento River Deep Water Ship Channel ranges from -0.13 feet to -0.89 feet.

7.3.4.3 **0.2% AEP** (column [8] in Tables 8 and 17)

In the 0.2% AEP event, this scenario results in a decrease of 0.06 feet in the peak stage at the upstream end of the Project reach (RM 57.00), and an increase of 0.12 feet at the downstream end of the Project reach (RM 51.75). The maximum computed change in peak stage is +0.39 feet at RM 54.00. Five miles downstream of the Project, at the Freeport bridge, the peak stage increase is 0.10 feet. Twenty-five miles downstream of the Project, at Walnut Grove, the peak stage increase is 0.01 feet. The impact to peak stages at index points on the Yolo Bypass, Sacramento Bypass, and Sacramento River Deep Water Ship Channel ranges from -0.07 feet to +0.01 feet.

7.3.5 Existing to Future With Alternative 5

Alternative 5 is similar to Alternative 2 with a four mile long levee setback and adjacent levee with vegetation removal for the remainder of the Project reach. It differs from Alternative 2 in that the Bees Lakes area is not hydraulically connected to the Sacramento River and the adjacent levee is replaced with slope flattening for about 0.8 miles at the downstream end of the Project reach. The future component for this scenario is the addition of the Folsom JFP and urban levees raised to provide 0.5% AEP level of protection where necessary. The computed impacts are summarized in Tables 9 and 18.

7.3.5.1 **1% AEP** (column [2] in Tables 9 and 18)

In the 1% AEP event, this scenario results in a decrease of 0.62 feet in the peak stage at the upstream end of the Project reach (RM 57.00), and a decrease of 0.38 feet at the downstream end of the Project reach (RM 51.75). There is no measurable increase in peak stage anywhere in the system. Five miles downstream of the Project, at the Freeport bridge, the peak stage decrease is 0.35 feet. Twenty-five miles downstream of the Project, at Walnut Grove, the peak stage decrease is 0.05 feet. The decrease in stage downstream of the Project is the result of a 2.2% decrease in the peak flow in the Sacramento River below the American River, from 126,000 cfs to 123,200 cfs. The decrease in flow is primarily due to reduced flow in the American River as a result of the Folsom JFP. The impact to peak stages at index points on the Yolo Bypass, Sacramento Bypass, and Sacramento River Deep Water Ship Channel ranges from -0.11 feet to -0.48 feet.

7.3.5.2 **0.5% AEP** (column [5] in Tables 9 and 18)

In the 0.5% AEP event, this scenario results in a decrease of 1.55 feet in the peak stage at the upstream end of the Project reach (RM 57.00), and a decrease of 1.10 feet at the downstream end of the Project reach (RM 51.75). There is no measurable increase in peak stage anywhere in the system. Five miles downstream of the Project, at the Freeport bridge, the peak stage decrease is 0.93 feet. Twenty-five miles downstream of the Project, at Walnut Grove, the peak stage decrease is 0.27 feet. The decrease in stage downstream of the Project is the result of a 9.4% decrease in the peak flow in the Sacramento River below the American River, from 149,200 cfs to 135,200 cfs. The decrease in flow is primarily due to reduced flow in the

American River as a result of the Folsom JFP. The impact to peak stages at index points on the Yolo Bypass, Sacramento Bypass, and Sacramento River Deep Water Ship Channel ranges from -0.13 feet to -0.90 feet.

7.3.5.3 0.2% AEP (column [8] in Tables 9 and 18)

In the 0.2% AEP event, this scenario results in a decrease of 0.09 feet in the peak stage at the upstream end of the Project reach (RM 57.00), and an increase of 0.15 feet at the downstream end of the Project reach (RM 51.75). The maximum computed change in peak stage is +0.40 feet at RM 54.00. Five miles downstream of the Project, at the Freeport bridge, the peak stage increase is 0.11 feet. Twenty-five miles downstream of the Project, at Walnut Grove, the peak stage increase is 0.02 feet. The impact to peak stages at index points on the Yolo Bypass, Sacramento Bypass, and Sacramento River Deep Water Ship Channel ranges from -0.07 feet to +0.01 feet.

7.4 No Action to Future With Project

7.4.1 No Action to Future With Alternative 1

The only difference between No Action and Future With Alternative 1 is vegetation removal on the right bank throughout the Project reach, which is represented in the analysis by a 25% reduction in the Manning's n-value roughness coefficient on the right bank. This alternative has no measurable impact to the peak stage or peak flow in any of the events analyzed (see columns [3], [6], and [9] in Tables 5 and 14).

7.4.2 No Action to Future With Alternative 2

Alternative 2 consists of a four mile long levee setback with adjacent levee and vegetation removal for the remainder of the Project reach. The Bees Lakes area is hydraulically connected to the Sacramento River with culverts. The computed impacts are summarized in Tables 6 and 15.

7.4.2.1 1% AEP (column [3] in Tables 6 and 15)

In the 1% AEP event, Alternative 2 results in a decrease of 0.07 feet in the peak stage at the upstream end of the Project reach (RM 57.00) and an increase of 0.14 feet at the downstream end of the Project reach (RM 51.75). The maximum computed change in peak stage is +0.30 feet at RM 54.00. Five miles downstream of the Project, at the Freeport bridge, the peak stage increase is 0.13 feet. Twenty-five miles downstream of the Project, at Walnut Grove, the peak stage increase is 0.05 feet. The increase in stage downstream of the Project is the result of a 0.8% increase in the peak flow in the Sacramento River below the American River, from 122,200 cfs to 123,200 cfs. The increase in flow is due to the effect of the peak stage decrease upstream of the Project on the flow split at the confluence of the Sacramento and American Rivers. The impact to peak stages at index points on the Yolo Bypass, Sacramento Bypass, and Sacramento River Deep Water Ship Channel is negligible, ranging from zero to -0.03 feet.

7.4.2.2 **0.5% AEP** (column [6] in Tables 6 and 15)

In the 0.5% AEP event, Alternative 2 results in a decrease of 0.08 feet in the peak stage at the upstream end of the Project reach (RM 57.00), and an increase of 0.15 feet at the downstream end of the Project reach (RM 51.75). The maximum computed change in peak stage is +0.33 feet at RM 54.00. Five miles downstream of the Project, at the Freeport bridge, the peak stage increase is 0.13 feet. Twenty-five miles downstream of the Project, at Walnut Grove, the peak stage increase is 0.05 feet. The increase in stage downstream of the Project is the result of a 0.9% increase in the peak flow in the Sacramento River below the American River, from 134,000 cfs to 135,200 cfs. The increase in flow is due to the effect of the peak stage decrease upstream of the Project on the flow split at the confluence of the Sacramento and American Rivers. The impact to peak stages at index points on the Yolo Bypass, Sacramento Bypass, and Sacramento River Deep Water Ship Channel is negligible, ranging from zero to -0.03 feet.

7.4.2.3 **0.2% AEP** (column [9] in Tables 6 and 15)

In the 0.2% AEP event, Alternative 2 results in a decrease of 0.13 feet in the peak stage at the upstream end of the Project reach (RM 57.00), and an increase of 0.10 feet at the downstream end of the Project reach (RM 51.75). The maximum computed change in peak stage is +0.37 feet at RM 54.00. Five miles downstream of the Project, at the Freeport bridge, the peak stage increase is 0.07 feet. Twenty-five miles downstream of the Project, at Walnut Grove, the peak stage increase is 0.03 feet. The increase in stage downstream of the Project is the result of a 1.7% increase in the peak flow in the Sacramento River below the American River, from 155,500 cfs to 158,100 cfs. The increase in flow is due to the effect of the peak stage decrease upstream of the Project on the flow split at the confluence of the Sacramento and American Rivers. The impact to peak stages at index points on the Yolo Bypass, Sacramento Bypass, and Sacramento River Deep Water Ship Channel is negligible, ranging from zero to -0.04 feet.

7.4.3 No Action to Future With Alternative 3

Alternative 3 involves slope flattening on the water side of the existing levee and vegetation removal. This alternative has no measurable impact to the peak stage or peak flow in any of the events analyzed (see columns [3], [6], and [9] in Tables 7 and 16).

7.4.4 No Action to Future With Alternative 4

Alternative 4 consists of a two mile long levee setback starting downstream of Bees Lakes with adjacent levee and vegetation removal for the remainder of the Project reach. The Bees Lakes area is not hydraulically connected to the Sacramento River. The computed impacts are summarized in Tables 8 and 17.

7.4.4.1 **1% AEP** (column [3] in Tables 8 and 17)

In the 1% AEP event, Alternative 4 results in a decrease of 0.06 feet in the peak stage at the upstream end of the Project reach (RM 57.00), and an increase of 0.11 feet at the downstream end of the Project reach (RM 51.75). The maximum computed change in peak stage is +0.27

feet at RM 54.00. Five miles downstream of the Project, at the Freeport bridge, the peak stage increase is 0.10 feet. Twenty-five miles downstream of the Project, at Walnut Grove, the peak stage increase is 0.04 feet. The increase in stage downstream of the Project is the result of a 0.7% increase in the peak flow in the Sacramento River below the American River, from 122,200 cfs to 123,000 cfs. The increase in flow is due to the effect of the peak stage decrease upstream of the Project on the flow split at the confluence of the Sacramento and American Rivers. The impact to peak stages at index points on the Yolo Bypass, Sacramento Bypass, and Sacramento River Deep Water Ship Channel is negligible, ranging from zero to -0.02 feet.

7.4.4.2 **0.5% AEP** (column [6] in Tables 8 and 17)

In the 0.5% AEP event, Alternative 4 results in a decrease of 0.06 feet in the peak stage at the upstream end of the Project reach (RM 57.00), and an increase of 0.11 feet at the downstream end of the Project reach (RM 51.75). The maximum computed change in peak stage is +0.31 feet at RM 54.00. Five miles downstream of the Project, at the Freeport bridge, the peak stage increase is 0.10 feet. Twenty-five miles downstream of the Project, at Walnut Grove, the peak stage increase is 0.04 feet. The increase in stage downstream of the Project is the result of a 0.7% increase in the peak flow in the Sacramento River below the American River, from 134,000 cfs to 134,900 cfs. The increase in flow is due to the effect of the peak stage decrease upstream of the Project on the flow split at the confluence of the Sacramento and American Rivers. The impact to peak stages at index points on the Yolo Bypass, Sacramento Bypass, and Sacramento River Deep Water Ship Channel is negligible, ranging from zero to -0.02 feet.

7.4.4.3 **0.2% AEP** (column [9] in Tables 8 and 17)

In the 0.2% AEP event, Alternative 4 results a decrease of 0.10 feet in the peak stage at the upstream end of the Project reach (RM 57.00), and an increase of 0.08 feet at the downstream end of the Project reach (RM 51.75). The maximum computed change in peak stage is +0.35 feet at RM 54.00. Five miles downstream of the Project, at the Freeport bridge, the peak stage increase is 0.06 feet. Twenty-five miles downstream of the Project, at Walnut Grove, the peak stage increase is 0.02 feet. The increase in stage downstream of the Project is the result of a 1.3% increase in the peak flow in the Sacramento River below the American River, from 155,500 cfs to 157,500 cfs. The increase in flow is due to the effect of the peak stage decrease upstream of the Project on the flow split at the confluence of the Sacramento and American Rivers. The impact to peak stages at index points on the Yolo Bypass, Sacramento Bypass, and Sacramento River Deep Water Ship Channel is negligible, ranging from zero to -0.03 feet.

7.4.5 No Action to Future With Alternative 5

Alternative 5 is similar to Alternative 2 with a four mile long levee setback and adjacent levee with vegetation removal for the remainder of the Project reach. It differs from Alternative 2 in that the Bees Lakes area is not hydraulically connected to the Sacramento River, and the adjacent levee is replaced with slope flattening for about 0.8 miles at the downstream end of the Project reach. The computed impacts are summarized in Tables 9 and 18.

7.4.5.1 **1% AEP** (column [3] in Tables 9 and 18)

In the 1% AEP event, Alternative 5 results in a decrease of 0.07 feet in the peak stage at the upstream end of the Project reach (RM 57.00), and an increase of 0.14 feet at the downstream end of the Project reach (RM 51.75). The maximum computed change in peak stage is +0.29 feet at RM 54.00. Five miles downstream of the Project, at the Freeport bridge, the peak stage increase is 0.13 feet. Twenty-five miles downstream of the Project, at Walnut Grove, the peak stage increase is 0.05feet. The increase in stage downstream of the Project is the result of a 0.8% increase in the peak flow in the Sacramento River below the American River, from 122,200 cfs to 123,200 cfs. The increase in flow is due to the effect of the peak stage decrease upstream of the Project on the flow split at the confluence of the Sacramento and American Rivers. The impact to peak stages at index points on the Yolo Bypass, Sacramento Bypass, and Sacramento River Deep Water Ship Channel is negligible, ranging from zero to -0.03 feet.

7.4.5.2 **0.5% AEP** (column [6] in Tables 9 and 18)

In the 0.5% AEP event, Alternative 5 results in a decrease of 0.08 feet in the peak stage at the upstream end of the Project reach (RM 57.00), and an increase of 0.15 feet at the downstream end of the Project reach (RM 51.75). The maximum computed change in peak stage is +0.32 feet at RM 54.00. Five miles downstream of the Project, at the Freeport bridge, the peak stage increase is 0.13 feet. Twenty-five miles downstream of the Project, at Walnut Grove, the peak stage increase is 0.05 feet. The increase in stage downstream of the Project is the result of a 0.9% increase in the peak flow in the Sacramento River below the American River, from 134,000 cfs to 135,200 cfs. The increase in flow is due to the effect of the peak stage decrease upstream of the Project on the flow split at the confluence of the Sacramento and American Rivers. The impact to peak stages at index points on the Yolo Bypass, Sacramento Bypass, and Sacramento River Deep Water Ship Channel is negligible, ranging from zero to -0.03 feet.

7.4.5.3 **0.2% AEP** (column [9] in Tables 9 and 18)

In the 0.2% AEP event, Alternative 5 results in a decrease of 0.13 feet in the peak stage at the upstream end of the Project reach (RM 57.00), and an increase of 0.11 feet at the downstream end of the Project reach (RM 51.75). The maximum computed change in peak stage is +0.36 feet at RM 54.00. Five miles downstream of the Project, at the Freeport bridge, the peak stage increase is 0.07 feet. Twenty-five miles downstream of the Project, at Walnut Grove, the peak stage increase is 0.03 feet. The increase in stage downstream of the Project is the result of a 1.7% increase in the peak flow in the Sacramento River below the American River, from 155,500 cfs to 158,100 cfs. The increase in flow is due to the effect of the peak stage decrease upstream of the Project on the flow split at the confluence of the Sacramento and American Rivers. The impact to peak stages at index points on the Yolo Bypass, Sacramento Bypass, and Sacramento River Deep Water Ship Channel is negligible, ranging from zero to -0.04 feet.

Table 2. 1% AEP Computed Maximum Water Surface Elevations

ID	Location	River Mile	1% AEP Computed Maximum Water Surface Elevation (ft. NAVD88)											
			Existing	**Current with Alt. 1**	**Current with Alt. 2**	**Current with Alt. 3**	**Current with Alt. 4**	**Current with Alt. 5**	**No Action** (Future without Project)	**Future with Alt. 1**	**Future with Alt. 2**	**Future with Alt. 3**	**Future with Alt. 4**	**Future with Alt. 5**
SR1	Sacramento R at Natomas Cross Canal	79.205	43.55	43.55	43.55	43.55	43.55	43.55	43.50	43.50	43.49	43.50	43.50	43.49
SR2	Sacramento R at Sacramento Weir	63.81	34.72	34.72	34.68	34.72	34.69	34.68	34.12	34.12	34.08	34.11	34.09	34.08
SR3	Sacramento R at I-80	62.971	35.27	35.27	35.23	35.26	35.24	35.23	34.71	34.71	34.68	34.71	34.68	34.68
SR4	Sacramento R at I Street bridge	59.692	34.96	34.96	34.92	34.96	34.93	34.92	34.39	34.39	34.35	34.39	34.36	34.35
SR5	Sacramento R at upstream end of Project reach	57.003	33.75	33.75	33.68	33.74	33.69	33.68	33.20	33.20	33.13	33.19	33.14	33.13
SR6	Sacramento R near Davis Road	54.746	32.86	32.86	32.89	32.85	32.86	32.88	32.32	32.32	32.35	32.32	32.32	32.34
SR7	Sacramento R at downstream end of Project reach	51.754	31.41	31.41	31.55	31.43	31.52	31.55	30.89	30.90	31.03	30.91	31.00	31.03
SR8	Sacramento R at Babel Slough	50	30.74	30.74	30.87	30.75	30.84	30.88	30.23	30.23	30.36	30.25	30.33	30.36
SR9	Sacramento R at Freeport bridge	46.427	29.04	29.04	29.17	29.05	29.14	29.17	28.56	28.57	28.69	28.58	28.66	28.69
SR10	Sacramento R at Walnut Grove	26.75	17.27	17.27	17.32	17.27	17.31	17.32	17.17	17.17	17.22	17.18	17.21	17.22
SB1	Sacramento Bypass below Sacramento Weir	1.49	33.73	33.73	33.70	33.73	33.71	33.70	33.28	33.28	33.25	33.28	33.26	33.25
YB1	Yolo Bypass at I-5	50.496	34.78	34.77	34.77	34.77	34.77	34.77	34.65	34.65	34.64	34.65	34.64	34.64
YB2	Yolo Bypass at Sacramento Bypass	44.13	31.95	31.95	31.94	31.95	31.94	31.94	31.76	31.76	31.75	31.76	31.75	31.75
YB3	Yolo Bypass at Lisbon Gage	35.672	28.72	28.72	28.70	28.71	28.71	28.70	28.58	28.58	28.56	28.58	28.57	28.56
SC1	Ship Channel at Turning Basin	42.954	19.88	19.88	19.88	19.88	19.88	19.88	19.78	19.78	19.77	19.78	19.78	19.77
SC2	Ship Channel at Cache Slough	19.536	19.89	19.89	19.88	19.89	19.89	19.88	19.78	19.78	19.77	19.78	19.77	19.77

Table 3. 0.5% AEP Computed Maximum Water Surface Elevations

ID	Location	River Mile	0.5% AEP Computed Maximum Water Surface Elevation (ft. NAVD88)											
			Existing	Current with Alt. 1	Current with Alt. 2	Current with Alt. 3	Current with Alt. 4	Current with Alt. 5	No Action (Future without Project)	Future with Alt. 1	Future with Alt. 2	Future with Alt. 3	Future with Alt. 4	Future with Alt. 5
SR1	Sacramento R at Natomas Cross Canal	79.205	44.61	44.61	44.61	44.61	44.61	44.61	44.57	44.57	44.57	44.57	44.57	44.57
SR2	Sacramento R at Sacramento Weir	63.81	37.55	37.55	37.50	37.55	37.52	37.50	36.01	36.01	35.96	36.00	35.97	35.96
SR3	Sacramento R at I-80	62.971	37.68	37.68	37.64	37.68	37.65	37.64	36.51	36.51	36.47	36.50	36.48	36.47
SR4	Sacramento R at I Street bridge	59.692	37.80	37.80	37.74	37.80	37.75	37.74	36.24	36.24	36.20	36.24	36.21	36.20
SR5	Sacramento R at upstream end of Project reach	57.003	36.49	36.49	36.38	36.48	36.40	36.38	35.02	35.02	34.94	35.01	34.96	34.94
SR6	Sacramento R near Davis Road	54.746	35.50	35.51	35.51	35.50	35.49	35.50	34.11	34.11	34.14	34.11	34.12	34.13
SR7	Sacramento R at downstream end of Project reach	51.754	33.88	33.88	33.99	33.90	33.97	34.00	32.63	32.63	32.78	32.65	32.74	32.78
SR8	Sacramento R at Babel Slough	50	33.16	33.17	33.27	33.18	33.25	33.28	31.96	31.96	32.10	31.97	32.07	32.10
SR9	Sacramento R at Freeport bridge	46.427	31.27	31.27	31.36	31.28	31.34	31.36	30.21	30.21	30.34	30.23	30.31	30.34
SR10	Sacramento R at Walnut Grove	26.75	18.37	18.37	18.42	18.38	18.41	18.42	18.05	18.05	18.10	18.05	18.09	18.10
SB1	Sacramento Bypass below Sacramento Weir	1.49	35.85	35.85	35.81	35.84	35.82	35.81	34.98	34.98	34.95	34.98	34.96	34.95
YB1	Yolo Bypass at I-5	50.496	36.09	36.09	36.09	36.09	36.09	36.09	35.94	35.94	35.93	35.93	35.93	35.93
YB2	Yolo Bypass at Sacramento Bypass	44.13	33.32	33.32	33.30	33.31	33.31	33.30	33.06	33.06	33.05	33.06	33.05	33.05
YB3	Yolo Bypass at Lisbon Gage	35.672	29.80	29.80	29.79	29.80	29.79	29.79	29.62	29.62	29.61	29.62	29.61	29.61
SC1	Ship Channel at Turning Basin	42.954	21.05	21.05	21.05	21.05	21.05	21.05	20.92	20.92	20.92	20.92	20.92	20.92
SC2	Ship Channel at Cache Slough	19.536	20.83	20.83	20.82	20.83	20.83	20.82	20.69	20.69	20.69	20.69	20.69	20.69

Table 4. 0.2% AEP Computed Maximum Water Surface Elevations

ID	Location	River Mile	0.2% AEP Computed Maximum Water Surface Elevation (ft. NAVD88)											
			Existing	Current with Alt. 1	Current with Alt. 2	Current with Alt. 3	Current with Alt. 4	Current with Alt. 5	No Action (Future without Project)	Future with Alt. 1	Future with Alt. 2	Future with Alt. 3	Future with Alt. 4	Future with Alt. 5
SR1	Sacramento R at Natomas Cross Canal	79.205	45.62	45.62	45.62	45.62	45.62	45.62	45.63	45.63	45.63	45.63	45.63	45.63
SR2	Sacramento R at Sacramento Weir	63.81	38.26	38.25	38.22	38.25	38.23	38.22	38.28	38.28	38.23	38.28	38.24	38.23
SR3	Sacramento R at I-80	62.971	38.43	38.42	38.39	38.42	38.40	38.39	38.46	38.46	38.42	38.46	38.43	38.42
SR4	Sacramento R at I Street bridge	59.692	38.45	38.45	38.39	38.44	38.41	38.39	38.51	38.51	38.43	38.50	38.45	38.43
SR5	Sacramento R at upstream end of Project reach	57.003	37.07	37.07	36.95	37.06	36.97	36.94	37.11	37.11	36.98	37.09	37.01	36.98
SR6	Sacramento R near Davis Road	54.746	36.02	36.02	36.01	36.01	35.99	36.00	36.06	36.06	36.05	36.05	36.03	36.04
SR7	Sacramento R at downstream end of Project reach	51.754	34.27	34.27	34.36	34.29	34.34	34.37	34.31	34.32	34.41	34.33	34.39	34.42
SR8	Sacramento R at Babel Slough	50	33.54	33.54	33.63	33.55	33.61	33.63	33.58	33.58	33.67	33.59	33.65	33.67
SR9	Sacramento R at Freeport bridge	46.427	31.59	31.59	31.66	31.60	31.64	31.66	31.63	31.63	31.70	31.64	31.69	31.70
SR10	Sacramento R at Walnut Grove	26.75	18.86	18.86	18.88	18.86	18.88	18.88	18.85	18.85	18.88	18.86	18.87	18.88
SB1	Sacramento Bypass below Sacramento Weir	1.49	36.73	36.73	36.70	36.72	36.71	36.70	36.77	36.77	36.73	36.76	36.74	36.73
YB1	Yolo Bypass at I-5	50.496	37.21	37.21	37.21	37.21	37.21	37.21	37.22	37.22	37.22	37.22	37.22	37.22
YB2	Yolo Bypass at Sacramento Bypass	44.13	34.26	34.26	34.23	34.25	34.23	34.23	34.21	34.21	34.20	34.21	34.21	34.20
YB3	Yolo Bypass at Lisbon Gage	35.672	30.62	30.62	30.60	30.62	30.60	30.60	30.57	30.57	30.56	30.57	30.56	30.56
SC1	Ship Channel at Turning Basin	42.954	22.43	22.43	22.42	22.43	22.42	22.42	22.38	22.38	22.36	22.38	22.37	22.36
SC2	Ship Channel at Cache Slough	19.536	22.31	22.31	22.29	22.31	22.30	22.29	22.25	22.25	22.24	22.25	22.24	22.24

Table 5. Impact on Maximum Water Surface Elevation – Project Alternative 1

ID	Location	River Mile	Impact on Maximum Water Surface Elevation (feet)								
			1% AEP			0.5% AEP			0.2% AEP		
			Existing to Current with Alt. 1	Existing to Future with Alt. 1	No Action to Future with Alt. 1	Existing to Current with Alt. 1	Existing to Future with Alt. 1	No Action to Future with Alt. 1	Existing to Current with Alt. 1	Existing to Future with Alt. 1	No Action to Future with Alt. 1
			[1]	[2]	[3]	[4]	[5]	[6]	[7]	[8]	[9]
SR1	Sacramento R at Natomas Cross Canal	79.205	0	-0.05	0	0	-0.04	0	0	+0.01	0
SR2	Sacramento R at Sacramento Weir	63.81	0	-0.60	0	0	-1.54	0	-0.01	+0.02	0
SR3	Sacramento R at I-80	62.971	0	-0.56	0	0	-1.17	0	-0.01	+0.03	0
SR4	Sacramento R at I Street bridge	59.692	0	-0.57	0	0	-1.56	0	0	+0.06	0
SR5	Sacramento R at upstream end of Project reach	57.003	0	-0.55	0	0	-1.47	0	0	+0.04	0
SR6	Sacramento R near Davis Road	54.746	0	-0.54	0	+0.01	-1.39	0	0	+0.04	0
SR7	Sacramento R at downstream end of Project reach	51.754	0	-0.51	+0.01	0	-1.25	0	0	+0.05	+0.01
SR8	Sacramento R at Babel Slough	50	0	-0.51	0	+0.01	-1.20	0	0	+0.04	0
SR9	Sacramento R at Freeport bridge	46.427	0	-0.47	+0.01	0	-1.06	0	0	+0.04	0
SR10	Sacramento R at Walnut Grove	26.75	0	-0.10	0	0	-0.32	0	0	-0.01	0
SR max	Maximum impact on Sacramento R (RM)	[1]	+0.01	-0.01	+0.01	+0.01	-0.04	+0.01	+0.01	+0.08 (59.003)	+0.01
SB1	Sacramento Bypass below Sacramento Weir	1.49	0	-0.45	0	0	-0.87	0	0	+0.04	0
SB max	Maximum impact on Sacramento Bypass (RM)	[1]	0	-0.19	0	0	-0.26	0	0	+0.05	0
YB1	Yolo Bypass at I-5	50.496	-0.01	-0.13	0	0	-0.15	0	0	+0.01	0
YB2	Yolo Bypass at Sacramento Bypass	44.13	0	-0.19	0	0	-0.26	0	0	-0.05	0
YB3	Yolo Bypass at Lisbon Gage	35.672	0	-0.14	0	0	-0.18	0	0	-0.05	0
YB max	Maximum impact on Yolo Bypass (RM)	[1]	0	-0.09	0	0	-0.12	0	0	+0.01	0
SC1	Ship Channel at Turning Basin	42.954	0	-0.10	0	0	-0.13	0	0	-0.05	0
SC2	Ship Channel at Cache Slough	19.536	0	-0.11	0	0	-0.14	0	0	-0.06	0
SC max	Maximum impact on Ship Channel (RM)	[1]	0	-0.10	0	0	-0.12	+0.01	0	-0.05	0

[1] Location not shown if maximum water surface elevation impact is less than +0.05.

Table 6. Impact on Maximum Water Surface Elevation – Project Alternative 2

ID	Location	River Mile	Impact on Maximum Water Surface Elevation (feet) 1% AEP Existing to Current with Alt. 2 [1]	1% AEP Existing to Future with Alt. 2 [2]	1% AEP No Action to Future with Alt. 2 [3]	0.5% AEP Existing to Current with Alt. 2 [4]	0.5% AEP Existing to Future with Alt. 2 [5]	0.5% AEP No Action to Future with Alt. 2 [6]	0.2% AEP Existing to Current with Alt. 2 [7]	0.2% AEP Existing to Future with Alt. 2 [8]	0.2% AEP No Action to Future with Alt. 2 [9]
SR1	Sacramento R at Natomas Cross Canal	79.205	0	-0.06	-0.01	0	-0.04	0	0	+0.01	0
SR2	Sacramento R at Sacramento Weir	63.81	-0.04	-0.64	-0.04	-0.05	-1.59	-0.05	-0.04	-0.03	-0.05
SR3	Sacramento R at I-80	62.971	-0.04	-0.59	-0.03	-0.04	-1.21	-0.04	-0.04	-0.01	-0.04
SR4	Sacramento R at I Street bridge	59.692	-0.04	-0.61	-0.04	-0.06	-1.60	-0.04	-0.06	-0.02	-0.08
SR5	Sacramento R at upstream end of Project reach	57.003	-0.07	-0.62	-0.07	-0.11	-1.55	-0.08	-0.12	-0.09	-0.13
SR6	Sacramento R near Davis Road	54.746	+0.03	-0.51	+0.03	+0.01	-1.36	+0.03	-0.01	+0.03	-0.01
SR7	Sacramento R at downstream end of Project reach	51.754	+0.14	-0.38	+0.14	+0.11	-1.10	+0.15	+0.09	+0.14	+0.10
SR8	Sacramento R at Babel Slough	50	+0.13	-0.38	+0.13	+0.11	-1.06	+0.14	+0.09	+0.13	+0.09
SR9	Sacramento R at Freeport bridge	46.427	+0.13	-0.35	+0.13	+0.09	-0.93	+0.13	+0.07	+0.11	+0.07
SR10	Sacramento R at Walnut Grove	26.75	+0.05	-0.05	+0.05	+0.05	-0.27	+0.05	+0.02	+0.02	+0.03
SR max	Maximum impact on Sacramento R (RM)	1	+0.31 (54.00)	+0.01	+0.30 (54.00)	+0.34 (54.00)	-0.04	+0.33 (54.00)	+0.36 (54.00)	+0.41 (54.00)	+0.37 (54.00)
SB1	Sacramento Bypass below Sacramento Weir	1.49	-0.03	-0.48	-0.03	-0.04	-0.90	-0.03	-0.03	0	-0.04
SB max	Maximum impact on Sacramento Bypass (RM)	1	-0.01	-0.20	-0.01	-0.01	-0.27	-0.01	-0.02	+0.01	-0.01
YB1	Yolo Bypass at I-5	50.496	-0.01	-0.14	-0.01	0	-0.16	-0.01	0	+0.01	-0.01
YB2	Yolo Bypass at Sacramento Bypass	44.13	-0.01	-0.20	-0.01	-0.02	-0.27	-0.01	-0.03	-0.06	0
YB3	Yolo Bypass at Lisbon Gage	35.672	-0.02	-0.16	-0.01	-0.01	-0.19	-0.01	-0.02	-0.06	-0.01
YB max	Maximum impact on Yolo Bypass (RM)	1	0	-0.09	-0.02	0	-0.11	-0.01	0	+0.01	-0.01
SC1	Ship Channel at Turning Basin	42.954	0	-0.11	0	0	-0.13	0	-0.01	-0.07	0
SC2	Ship Channel at Cache Slough	19.536	-0.01	-0.12	-0.01	-0.01	-0.14	0	-0.02	-0.07	-0.02
SC max	Maximum impact on Ship Channel (RM)	1	0	-0.10	-0.01	0	-0.12	0	-0.01	-0.06	-0.01

1 Location not shown if maximum water surface elevation impact is less than +0.05.

Table 7. Impact on Maximum Water Surface Elevation – Project Alternative 3

ID	Location	River Mile	Impact on Maximum Water Surface Elevation (feet)								
			1% AEP			0.5% AEP			0.2% AEP		
			Existing to Current with Alt. 3	Existing to Future with Alt. 3	No Action to Future with Alt. 3	Existing to Current with Alt. 3	Existing to Future with Alt. 3	No Action to Future with Alt. 3	Existing to Current with Alt. 3	Existing to Future with Alt. 3	No Action to Future with Alt. 3
			[1]	[2]	[3]	[4]	[5]	[6]	[7]	[8]	[9]
SR1	Sacramento R at Natomas Cross Canal	79.205	0	-0.05	0	0	-0.04	0	0	+0.01	0
SR2	Sacramento R at Sacramento Weir	63.81	0	-0.61	-0.01	0	-1.55	-0.01	-0.01	+0.02	0
SR3	Sacramento R at I-80	62.971	-0.01	-0.56	0	0	-1.18	-0.01	-0.01	+0.03	0
SR4	Sacramento R at I Street bridge	59.692	0	-0.57	0	0	-1.56	0	-0.01	+0.05	-0.01
SR5	Sacramento R at upstream end of Project reach	57.003	-0.01	-0.56	-0.01	-0.01	-1.48	-0.01	-0.01	+0.02	-0.02
SR6	Sacramento R near Davis Road	54.746	-0.01	-0.54	0	0	-1.39	0	-0.01	+0.03	-0.01
SR7	Sacramento R at downstream end of Project reach	51.754	+0.02	-0.50	+0.02	+0.02	-1.23	+0.02	+0.02	+0.06	+0.02
SR8	Sacramento R at Babel Slough	50	+0.01	-0.49	+0.02	+0.02	-1.19	+0.01	+0.01	+0.05	+0.01
SR9	Sacramento R at Freeport bridge	46.427	+0.01	-0.46	+0.02	+0.01	-1.04	+0.02	+0.01	+0.05	+0.01
SR10	Sacramento R at Walnut Grove	26.75	0	-0.09	+0.01	+0.01	-0.32	0	0	0	+0.01
SR max	Maximum impact on Sacramento R (RM)	[1]	+0.05 (52.255)	-0.01	+0.05 (52.255)	+0.06 (52.255)	-0.04	+0.06 (52.255)	+0.06 (52.255)	+0.11 (52.255)	+0.07 (52.255)
SB1	Sacramento Bypass below Sacramento Weir	1.49	0	-0.45	0	-0.01	-0.87	0	-0.01	+0.03	-0.01
SB max	Maximum impact on Sacramento Bypass (RM)	[1]	0	-0.19	0	0	-0.26	0	0	+0.04	0
YB1	Yolo Bypass at I-5	50.496	-0.01	-0.13	0	0	-0.16	0	0	+0.01	0
YB2	Yolo Bypass at Sacramento Bypass	44.13	0	-0.19	0	-0.01	-0.26	-0.01	-0.01	-0.05	0
YB3	Yolo Bypass at Lisbon Gage	35.672	-0.01	-0.14	0	0	-0.18	0	0	-0.05	0
YB max	Maximum impact on Yolo Bypass (RM)	[1]	0	-0.09	0	0	-0.10	0	0	+0.01	0
SC1	Ship Channel at Turning Basin	42.954	0	-0.10	0	0	-0.13	0	0	-0.05	0
SC2	Ship Channel at Cache Slough	19.536	0	-0.11	0	0	-0.14	0	0	-0.06	0
SC max	Maximum impact on Ship Channel (RM)	[1]	0	-0.10	0	0	-0.12	0	0	-0.05	0

[1] Location not shown if maximum water surface elevation impact is less than +0.05.

Table 8. Impact on Maximum Water Surface Elevation – Project Alternative 4

ID	Location	River Mile	Impact on Maximum Water Surface Elevation (feet)								
			1% AEP			0.5% AEP			0.2% AEP		
			Existing to Current with Alt. 4	Existing to Future with Alt. 4	No Action to Future with Alt. 4	Existing to Current with Alt. 4	Existing to Future with Alt. 4	No Action to Future with Alt. 4	Existing to Current with Alt. 4	Existing to Future with Alt. 4	No Action to Future with Alt. 4
			[1]	[2]	[3]	[4]	[5]	[6]	[7]	[8]	[9]
SR1	Sacramento R at Natomas Cross Canal	79.205	0	-0.05	0	0	-0.04	0	0	+0.01	0
SR2	Sacramento R at Sacramento Weir	63.81	-0.03	-0.63	-0.03	-0.03	-1.58	-0.04	-0.03	-0.02	-0.04
SR3	Sacramento R at I-80	62.971	-0.03	-0.59	-0.03	-0.03	-1.20	-0.03	-0.03	0	-0.03
SR4	Sacramento R at I Street bridge	59.692	-0.03	-0.60	-0.03	-0.05	-1.59	-0.03	-0.04	0	-0.06
SR5	Sacramento R at upstream end of Project reach	57.003	-0.06	-0.61	-0.06	-0.09	-1.53	-0.06	-0.10	-0.06	-0.10
SR6	Sacramento R near Davis Road	54.746	0	-0.54	0	-0.01	-1.38	+0.01	-0.03	+0.01	-0.03
SR7	Sacramento R at downstream end of Project reach	51.754	+0.11	-0.41	+0.11	+0.09	-1.14	+0.11	+0.07	+0.12	+0.08
SR8	Sacramento R at Babel Slough	50	+0.10	-0.41	+0.10	+0.09	-1.09	+0.11	+0.07	+0.11	+0.07
SR9	Sacramento R at Freeport bridge	46.427	+0.10	-0.38	+0.10	+0.07	-0.96	+0.10	+0.05	+0.10	+0.06
SR10	Sacramento R at Walnut Grove	26.75	+0.04	-0.06	+0.04	+0.04	-0.28	+0.04	+0.02	+0.01	+0.02
SR max	Maximum impact on Sacramento R (RM)	1	+0.28 (54.00)	+0.01	+0.27 (54.00)	+0.32 (54.00)	-0.04	+0.31 (54.00)	+0.34 (54.00)	+0.39 (54.00)	+0.35 (54.00)
SB1	Sacramento Bypass below Sacramento Weir	1.49	-0.02	-0.47	-0.02	-0.03	-0.89	-0.02	-0.02	+0.01	-0.03
SB max	Maximum impact on Sacramento Bypass (RM)	1	-0.01	-0.20	-0.01	-0.01	-0.27	-0.01	-0.01	+0.02	0
YB1	Yolo Bypass at I-5	50.496	-0.01	-0.14	-0.01	0	-0.16	-0.01	0	+0.01	0
YB2	Yolo Bypass at Sacramento Bypass	44.13	-0.01	-0.20	-0.01	-0.01	-0.27	-0.01	-0.03	-0.05	0
YB3	Yolo Bypass at Lisbon Gage	35.672	-0.01	-0.15	-0.01	-0.01	-0.19	-0.01	-0.02	-0.06	0
YB max	Maximum impact on Yolo Bypass (RM)	1	0	-0.09	-0.01	0	-0.11	-0.01	0	+0.01	-0.01
SC1	Ship Channel at Turning Basin	42.954	0	-0.10	0	0	-0.13	0	-0.01	-0.06	0
SC2	Ship Channel at Cache Slough	19.536	0	-0.12	0	0	-0.14	0	-0.01	-0.07	-0.01
SC max	Maximum impact on Ship Channel (RM)	1	0	-0.10	-0.01	0	-0.12	0	-0.01	-0.06	-0.01

[1] Location not shown if maximum water surface elevation impact is less than +0.05.

Table 9. Impact on Maximum Water Surface Elevation – Project Alternative 5

ID	Location	River Mile	Impact on Maximum Water Surface Elevation (feet)								
			1% AEP			0.5% AEP			0.2% AEP		
			Existing to Current with Alt. 5	Existing to Future with Alt. 5	No Action to Future with Alt. 5	Existing to Current with Alt. 5	Existing to Future with Alt. 5	No Action to Future with Alt. 5	Existing to Current with Alt. 5	Existing to Future with Alt. 5	No Action to Future with Alt. 5
			[1]	[2]	[3]	[4]	[5]	[6]	[7]	[8]	[9]
SR1	Sacramento R at Natomas Cross Canal	79.205	0	-0.06	-0.01	0	-0.04	0	0	+0.01	0
SR2	Sacramento R at Sacramento Weir	63.81	-0.04	-0.64	-0.04	-0.05	-1.59	-0.05	-0.04	-0.03	-0.05
SR3	Sacramento R at I-80	62.971	-0.04	-0.59	-0.03	-0.04	-1.21	-0.04	-0.04	-0.01	-0.04
SR4	Sacramento R at I Street bridge	59.692	-0.04	-0.61	-0.04	-0.06	-1.60	-0.04	-0.06	-0.02	-0.08
SR5	Sacramento R at upstream end of Project reach	57.003	-0.07	-0.62	-0.07	-0.11	-1.55	-0.08	-0.13	-0.09	-0.13
SR6	Sacramento R near Davis Road	54.746	+0.02	-0.52	+0.02	0	-1.37	+0.02	-0.02	+0.02	-0.02
SR7	Sacramento R at downstream end of Project reach	51.754	+0.14	-0.38	+0.14	+0.12	-1.10	+0.15	+0.10	+0.15	+0.11
SR8	Sacramento R at Babel Slough	50	+0.14	-0.38	+0.13	+0.12	-1.06	+0.14	+0.09	+0.13	+0.09
SR9	Sacramento R at Freeport bridge	46.427	+0.13	-0.35	+0.13	+0.09	-0.93	+0.13	+0.07	+0.11	+0.07
SR10	Sacramento R at Walnut Grove	26.75	+0.05	-0.05	+0.05	+0.05	-0.27	+0.05	+0.02	+0.02	+0.03
SR max	Maximum impact on Sacramento R (RM)	[1]	+0.30 (54.00)	+0.01	+0.29 (54.00)	+0.33 (54.00)	-0.04	+0.32 (54.00)	+0.35 (54.00)	+0.40 (54.00)	+0.36 (54.00)
SB1	Sacramento Bypass below Sacramento Weir	1.49	-0.03	-0.48	-0.03	-0.04	-0.90	-0.03	-0.03	0	-0.04
SB max	Maximum impact on Sacramento Bypass (RM)	[1]	-0.01	-0.14	-0.01	-0.01	-0.27	-0.01	-0.02	+0.01	-0.01
YB1	Yolo Bypass at I-5	50.496	-0.01	-0.20	-0.01	0	-0.16	-0.01	0	+0.01	-0.01
YB2	Yolo Bypass at Sacramento Bypass	44.13	-0.02	-0.16	-0.01	-0.02	-0.27	-0.01	-0.03	-0.06	0
YB3	Yolo Bypass at Lisbon Gage	35.672	0	-0.09	-0.01	-0.01	-0.19	-0.01	-0.02	-0.06	-0.01
YB max	Maximum impact on Yolo Bypass (RM)	[1]	0	-0.11	-0.02	0	-0.11	-0.01	0	+0.01	-0.01
SC1	Ship Channel at Turning Basin	42.954	-0.01	-0.12	0	0	-0.13	0	-0.01	-0.07	0
SC2	Ship Channel at Cache Slough	19.536	0	-0.10	-0.01	-0.01	-0.14	0	-0.02	-0.07	-0.02
SC max	Maximum impact on Ship Channel (RM)	[1]	-0.01	-0.14	-0.01	0	-0.12	0	-0.01	-0.06	-0.01

[1] Location not shown if maximum water surface elevation impact is less than +0.05.

Table 10. Impact on Maximum Water Surface Elevation – Existing to No Action

ID	Location	River Mile	Impact on Maximum Water Surface Elevation (feet) 1% AEP [1]	0.5% AEP [2]	0.2% AEP [3]
SR1	Sacramento R at Natomas Cross Canal	79.205	-0.05	-0.04	+0.01
SR2	Sacramento R at Sacramento Weir	63.81	-0.60	-1.54	+0.02
SR3	Sacramento R at I-80	62.971	-0.56	-1.17	+0.03
SR4	Sacramento R at I Street bridge	59.692	-0.57	-1.56	+0.06
SR5	Sacramento R at upstream end of Project reach	57.003	-0.55	-1.47	+0.04
SR6	Sacramento R near Davis Road	54.746	-0.54	-1.39	+0.04
SR7	Sacramento R at downstream end of Project reach	51.754	-0.52	-1.25	+0.04
SR8	Sacramento R at Babel Slough	50	-0.51	-1.20	+0.04
SR9	Sacramento R at Freeport bridge	46.427	-0.48	-1.06	+0.04
SR10	Sacramento R at Walnut Grove	26.75	-0.10	-0.32	-0.01
SR max	Maximum impact on Sacramento R (RM)	[1]	-0.01	-0.04	+0.08 (59.003)
SB1	Sacramento Bypass below Sacramento Weir	1.49	-0.45	-0.87	+0.04
SB max	Maximum impact on Sacramento Bypass (RM)	[1]	-0.19	-0.26	+0.05 (1.68)
YB1	Yolo Bypass at I-5	50.496	-0.13	-0.15	+0.01
YB2	Yolo Bypass at Sacramento Bypass	44.13	-0.19	-0.26	-0.05
YB3	Yolo Bypass at Lisbon Gage	35.672	-0.14	-0.18	-0.05
YB max	Maximum impact on Yolo Bypass (RM)	[1]	-0.09	-0.10	+0.01
SC1	Ship Channel at Turning Basin	42.954	-0.10	-0.13	-0.05
SC2	Ship Channel at Cache Slough	19.536	-0.11	-0.14	-0.06
SC max	Maximum impact on Ship Channel (RM)	[1]	-0.10	-0.12	-0.05

[1] Location not shown if maximum water surface elevation impact is less than +0.05.

Table 11. 1% AEP Computed Peak Flow

Location	1% AEP Computed Peak Flow (cfs) Existing	Current with Alt. 1	Current with Alt. 2	Current with Alt. 3	Current with Alt. 4	Current with Alt. 5	No Action (Future without Project)	Future with Alt. 1	Future with Alt. 2	Future with Alt. 3	Future with Alt. 4	Future with Alt. 5
Sacramento R. abv Sacramento Weir	119,300	119,300	119,400	119,300	119,400	119,400	120,200	120,200	120,300	120,200	120,300	120,300
Sacramento R. blw Sacramento Weir [1]	75,300	75,300	75,400	75,300	75,400	75,400	75,300	75,300	75,400	75,300	75,400	75,400
	-34,400	-34,400	-33,300	-34,300	-33,600	-33,300	-14,100	-14,100	-13,100	-14,000	-13,300	-13,100
Sacramento R. blw American R.	126,000	126,000	127,100	126,100	126,800	127,100	122,200	122,200	123,200	122,300	123,000	123,200
Sacramento R. at Freeport	125,700	125,700	126,700	125,800	126,400	126,700	121,800	121,800	122,800	121,900	122,600	122,800
Sacramento Weir	133,300	133,300	132,400	133,200	132,600	132,400	116,500	116,500	115,600	116,400	115,800	115,600
Fremont Weir	401,900	401,900	401,800	401,900	401,800	401,800	400,700	400,700	400,700	400,700	400,700	400,700
Yolo Byp. blw Sacramento Byp.	551,600	551,600	550,600	551,500	550,900	550,600	537,100	537,100	536,200	537,000	536,400	536,200

[1] Due to Sacramento Weir operation, flow at this location is bi-directional. Positive value is peak flow in downstream direction; negative value is peak flow in upstream direction.

Table 12. 0.5% AEP Computed Peak Flow

Location	0.5% AEP Computed Peak Flow (cfs) Existing	Current with Alt. 1	Current with Alt. 2	Current with Alt. 3	Current with Alt. 4	Current with Alt. 5	No Action (Future without Project)	Future with Alt. 1	Future with Alt. 2	Future with Alt. 3	Future with Alt. 4	Future with Alt. 5
Sacramento R. abv Sacramento Weir	123,800	123,800	123,900	123,800	123,900	123,900	124,000	124,000	124,100	124,000	124,100	124,200
Sacramento R. blw Sacramento Weir [1]	76,300	76,300	76,400	76,300	76,400	76,400	76,300	76,300	76,400	76,300	76,400	76,400
	-94,200	-94,200	-92,800	-94,000	-93,200	-92,800	-39,600	-39,600	-38,400	-39,500	-38,700	-38,400
Sacramento R. blw American R.	149,200	149,200	150,900	149,400	150,500	150,900	134,000	134,000	135,200	134,100	134,900	135,200
Sacramento R. at Freeport	145,200	145,200	146,400	145,300	146,100	146,400	133,700	133,700	134,900	133,900	134,600	134,900
Sacramento Weir	196,200	196,200	195,200	196,100	195,400	195,200	151,500	151,500	150,500	151,400	150,700	150,500
Fremont Weir	445,700	445,700	445,700	445,700	445,700	445,700	445,800	445,800	445,800	445,800	445,800	445,800
Yolo Byp. blw Sacramento Byp.	656,800	656,800	655,800	656,700	656,000	655,800	632,600	632,600	631,700	632,500	631,900	631,600

[1] Due to Sacramento Weir operation, flow at this location is bi-directional. Positive value is peak flow in downstream direction; negative value is peak flow in upstream direction.

Table 13. 0.2% AEP Computed Peak Flow

Location	Existing	Current with Alt. 1	Current with Alt. 2	Current with Alt. 3	Current with Alt. 4	Current with Alt. 5	No Action (Future without Project)	Future with Alt. 1	Future with Alt. 2	Future with Alt. 3	Future with Alt. 4	Future with Alt. 5
	0.2% AEP Computed Peak Flow (cfs)											
Sacramento R. abv Sacramento Weir	126,100	126,100	126,200	126,100	126,200	126,200	126,400	126,400	126,600	126,500	126,600	126,600
Sacramento R. blw Sacramento Weir [1]	77,600	77,600	77,700	77,600	77,700	77,700	77,600	77,600	77,700	77,600	77,700	77,700
	-99,900	-99,900	-98,600	-99,800	-98,900	-98,600	-88,500	-88,500	-86,700	-88,300	-87,200	-86,700
Sacramento R. blw American R.	163,600	163,600	165,500	163,800	165,000	165,500	155,500	155,500	158,100	155,800	157,500	158,100
Sacramento R. at Freeport	149,300	149,300	150,400	149,400	150,100	150,400	149,600	149,600	150,700	149,700	150,500	150,700
Sacramento Weir	204,200	204,200	203,300	204,100	203,500	203,300	199,800	199,800	198,500	199,700	198,800	198,500
Fremont Weir	498,000	498,000	498,000	498,000	498,000	498,000	498,300	498,300	498,300	498,300	498,300	498,300
Yolo Byp. blw Sacramento Byp.	726,500	726,600	723,600	726,200	724,200	723,500	724,500	724,500	723,700	724,400	723,900	723,700

[1] Due to Sacramento Weir operation, flow at this location is bi-directional. Positive value is peak flow in downstream direction; negative value is peak flow in upstream direction.

Table 14. Impact on Peak Flow – Project Alternative 1

Location	Impact on Peak Flow (%)								
	1% AEP			0.5% AEP			0.2% AEP		
	Existing to Current with Alt. 1	Existing to Future with Alt. 1	No Action to Future with Alt. 1	Existing to Current with Alt. 1	Existing to Future with Alt. 1	No Action to Future with Alt. 1	Existing to Current with Alt. 1	Existing to Future with Alt. 1	No Action to Future with Alt. 1
	[1]	[2]	[3]	[4]	[5]	[6]	[7]	[8]	[9]
Sacramento R. abv Sacramento Weir	0%	+0.8%	0%	0%	+0.2%	0%	0%	+0.2%	0%
Sacramento R. blw Sacramento Weir[1]									
peak flow in downstream direction	0%	0%	0%	0%	0%	0%	0%	0%	0%
peak flow in upstream direction	0%	-59.0%	0%	0%	-58.0%	0%	0%	-11.4%	0%
Sacramento R. blw American R.	0%	-3.0%	0%	0%	-10.2%	0%	0%	-5.0%	0%
Sacramento R. at Freeport	0%	-3.1%	0%	0%	-7.9%	0%	0%	+0.2%	0%
Sacramento Weir	0%	-12.6%	0%	0%	-22.8%	0%	0%	-2.2%	0%
Fremont Weir	0%	-0.3%	0%	0%	0%	0%	0%	+0.1%	0%
Yolo Byp. blw Sacramento Byp.	0%	-2.6%	0%	0%	-3.7%	0%	0%	-0.3%	0%

[1] Due to Sacramento Weir operation, flow at this location is bi-directional.

Table 15. Impact on Peak Flow – Project Alternative 2

Location	Impact on Peak Flow (%)								
	1% AEP			0.5% AEP			0.2% AEP		
	Existing to Current with Alt. 2	Existing to Future with Alt. 2	No Action to Future with Alt. 2	Existing to Current with Alt. 2	Existing to Future with Alt. 2	No Action to Future with Alt. 2	Existing to Current with Alt. 2	Existing to Future with Alt. 2	No Action to Future with Alt. 2
	[1]	[2]	[3]	[4]	[5]	[6]	[7]	[8]	[9]
Sacramento R. abv Sacramento Weir	+0.1%	+0.8%	+0.1%	+0.1%	+0.2%	+0.1%	+0.1%	+0.4%	+0.2%
Sacramento R. blw Sacramento Weir[1]									
peak flow in downstream direction	+0.1%	+0.1%	+0.1%	+0.1%	+0.1%	+0.1%	+0.1%	+0.1%	+0.1%
peak flow in upstream direction	-3.2%	-61.9%	-7.1%	-1.5%	-59.2%	-3.0%	-1.3%	-13.2%	-2.0%
Sacramento R. blw American R.	+0.9%	-2.2%	+0.8%	+1.1%	-9.4%	+0.9%	+1.2%	-3.4%	+1.7%
Sacramento R. at Freeport	+0.8%	-2.3%	+0.8%	+0.8%	-7.1%	+0.9%	+0.7%	+0.9%	+0.7%
Sacramento Weir	-0.7%	-13.3%	-0.8%	-0.5%	-23.3%	-0.7%	-0.4%	-2.8%	-0.7%
Fremont Weir	0%	-0.3%	0%	0%	0%	0%	0%	+0.1%	0%
Yolo Byp. blw Sacramento Byp.	-0.2%	-2.8%	-0.2%	-0.2%	-3.8%	-0.1%	-0.4%	-0.4%	-0.1%

[1] Due to Sacramento Weir operation, flow at this location is bi-directional.

Table 16. Impact on Peak Flow – Project Alternative 3

Location	Impact on Peak Flow (%)								
	1% AEP			0.5% AEP			0.2% AEP		
	Existing to Current with Alt. 3	Existing to Future with Alt. 3	No Action to Future with Alt. 3	Existing to Current with Alt. 3	Existing to Future with Alt. 3	No Action to Future with Alt. 3	Existing to Current with Alt. 3	Existing to Future with Alt. 3	No Action to Future with Alt. 3
	[1]	[2]	[3]	[4]	[5]	[6]	[7]	[8]	[9]
Sacramento R. abv Sacramento Weir	0%	+0.8%	0%	0%	+0.2%	0%	0%	+0.3%	+0.1%
Sacramento R. blw Sacramento Weir[1]									
peak flow in downstream direction	0%	0%	0%	0%	0%	0%	0%	0%	0%
peak flow in upstream direction	-0.3%	-59.3%	-0.7%	-0.2%	-58.1%	-0.3%	-0.1%	-11.6%	-0.2%
Sacramento R. blw American R.	+0.1%	-2.9%	+0.1%	+0.1%	-10.1%	+0.1%	+0.1%	-4.8%	+0.2%
Sacramento R. at Freeport	+0.1%	-3.0%	+0.1%	+0.1%	-7.8%	+0.1%	+0.1%	+0.3%	+0.1%
Sacramento Weir	-0.1%	-12.7%	-0.1%	-0.1%	-22.8%	-0.1%	0%	-2.2%	-0.1%
Fremont Weir	0%	-0.3%	0%	0%	0%	0%	0%	+0.1%	0%
Yolo Byp. blw Sacramento Byp.	0%	-2.6%	0%	0%	-3.7%	0%	0%	-0.3%	0%

[1] Due to Sacramento Weir operation, flow at this location is bi-directional.

Table 17. Impact on Peak Flow – Project Alternative 4									
	Impact on Peak Flow (%)								
	1% AEP			0.5% AEP			0.2% AEP		
Location	Existing to Current with Alt. 4	Existing to Future with Alt. 4	No Action to Future with Alt. 4	Existing to Current with Alt. 4	Existing to Future with Alt. 4	No Action to Future with Alt. 4	Existing to Current with Alt. 4	Existing to Future with Alt. 4	No Action to Future with Alt. 4
	[1]	[2]	[3]	[4]	[5]	[6]	[7]	[8]	[9]
Sacramento R. abv Sacramento Weir	+0.1%	+0.8%	+0.1%	+0.1%	+0.2%	+0.1%	+0.1%	+0.4%	+0.2%
Sacramento R. blw Sacramento Weir[1]									
peak flow in downstream direction	+0.1%	+0.1%	+0.1%	+0.1%	+0.1%	+0.1%	+0.1%	+0.1%	+0.1%
peak flow in upstream direction	-2.3%	-61.3%	-5.7%	-1.1%	-58.9%	-2.3%	-1.0%	-12.7%	-1.5%
Sacramento R. blw American R.	+0.6%	-2.4%	+0.7%	+0.9%	-9.6%	+0.7%	+0.9%	-3.7%	+1.3%
Sacramento R. at Freeport	+0.6%	-2.5%	+0.7%	+0.6%	-7.3%	+0.7%	+0.5%	+0.8%	+0.6%
Sacramento Weir	-0.5%	-13.1%	-0.6%	-0.4%	-23.2%	-0.5%	-0.3%	-2.6%	-0.5%
Fremont Weir	0%	-0.3%	0%	0%	0%	0%	0%	+0.1%	0%
Yolo Byp. blw Sacramento Byp.	-0.1%	-2.8%	-0.1%	-0.1%	-3.8%	-0.1%	-0.3%	-0.4%	-0.1%

[1] Due to Sacramento Weir operation, flow at this location is bi-directional.

Table 18. Impact on Peak Flow – Project Alternative 5									
	Impact on Peak Flow (%)								
	1% AEP			0.5% AEP			0.2% AEP		
Location	Existing to Current with Alt. 5	Existing to Future with Alt. 5	No Action to Future with Alt. 5	Existing to Current with Alt. 5	Existing to Future with Alt. 5	No Action to Future with Alt. 5	Existing to Current with Alt. 5	Existing to Future with Alt. 5	No Action to Future with Alt. 5
	[1]	[2]	[3]	[4]	[5]	[6]	[7]	[8]	[9]
Sacramento R. abv Sacramento Weir	+0.1%	+0.8%	+0.1%	+0.1%	+0.3%	+0.2%	+0.1%	+0.4%	+0.2%
Sacramento R. blw Sacramento Weir[1]									
peak flow in downstream direction	+0.1%	+0.1%	+0.1%	+0.1%	+0.1%	+0.1%	+0.1%	+0.1%	+0.1%
peak flow in upstream direction	-3.2%	-61.9%	-7.1%	-1.5%	-59.2%	-3.0%	-1.3%	-13.2%	-2.0%
Sacramento R. blw American R.	+0.9%	-2.2%	+0.8%	+1.1%	-9.4%	+0.9%	+1.2%	-3.4%	+1.7%
Sacramento R. at Freeport	+0.8%	-2.3%	+0.8%	+0.8%	-7.1%	+0.9%	+0.7%	+0.9%	+0.7%
Sacramento Weir	-0.7%	-13.3%	-0.8%	-0.5%	-23.3%	-0.7%	-0.4%	-2.8%	-0.7%
Fremont Weir	0%	-0.3%	0%	0%	0%	0%	0%	+0.1%	0%
Yolo Byp. blw Sacramento Byp.	-0.2%	-2.8%	-0.2%	-0.2%	-3.8%	-0.2%	-0.4%	-0.4%	-0.1%

[1] Due to Sacramento Weir operation, flow at this location is bi-directional.

Table 19. Impact on Peak Flow – Existing to No Action			
Location	**Impact on Peak Flow (%)**		
	1% AEP	**0.5% AEP**	**0.2% AEP**
	[1]	[2]	[3]
Sacramento R. abv Sacramento Weir	+0.8%	+0.2%	+0.2%
Sacramento R. blw Sacramento Weir[1]			
peak flow in downstream direction	0%	0%	0%
peak flow in upstream direction	-59.0%	-58.0%	-11.4%
Sacramento R. blw American R.	-3.0%	-10.2%	-5.0%
Sacramento R. at Freeport	-3.1%	-7.9%	+0.2%
Sacramento Weir	-12.6%	-22.8%	-2.2%
Fremont Weir	-0.3%	0%	+0.1%
Yolo Byp. blw Sacramento Byp.	-2.6%	-3.7%	-0.3%

1 Due to Sacramento Weir operation, flow at this location is bi-directional.

FIGURES

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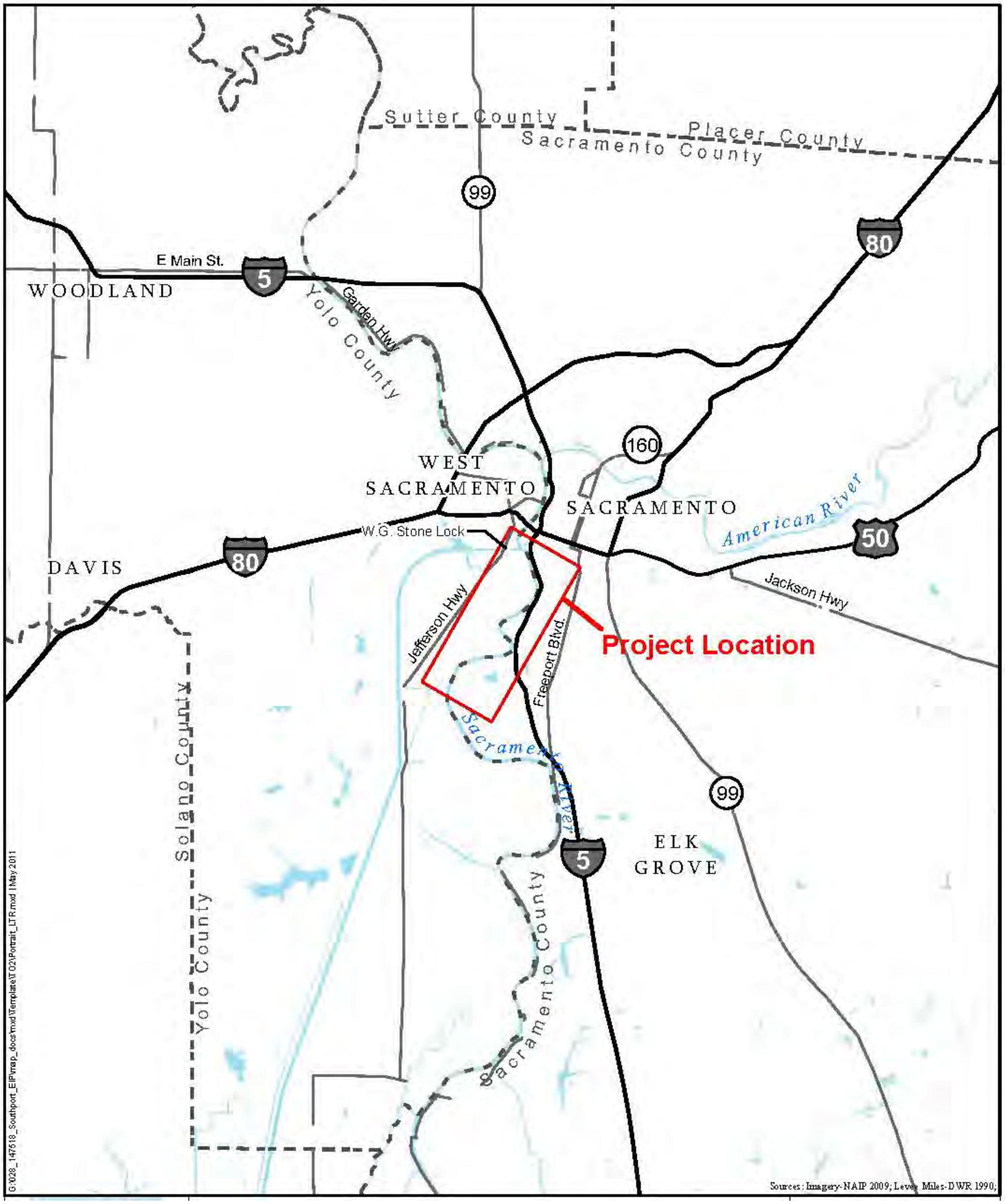
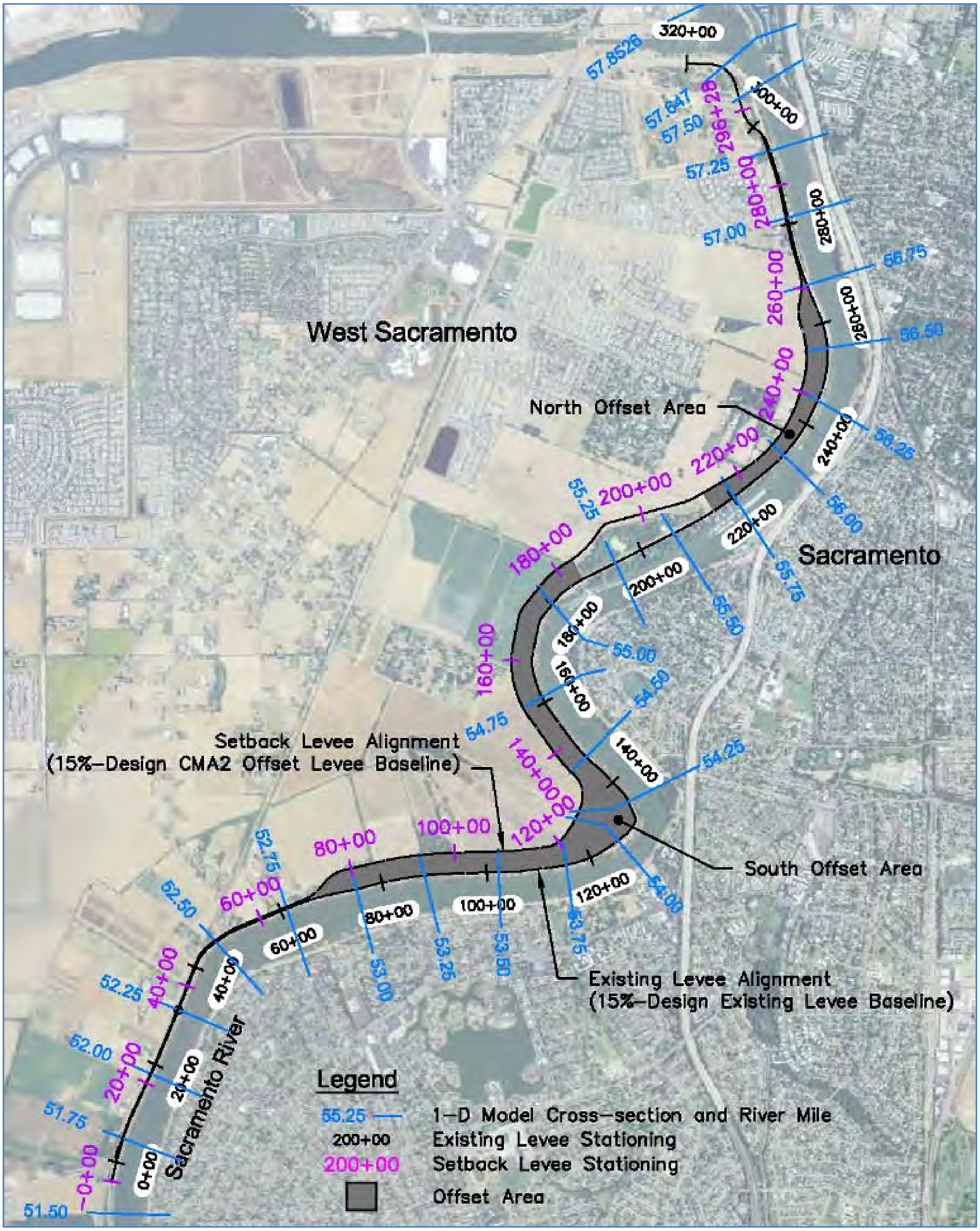


Figure 1. Project Location Map



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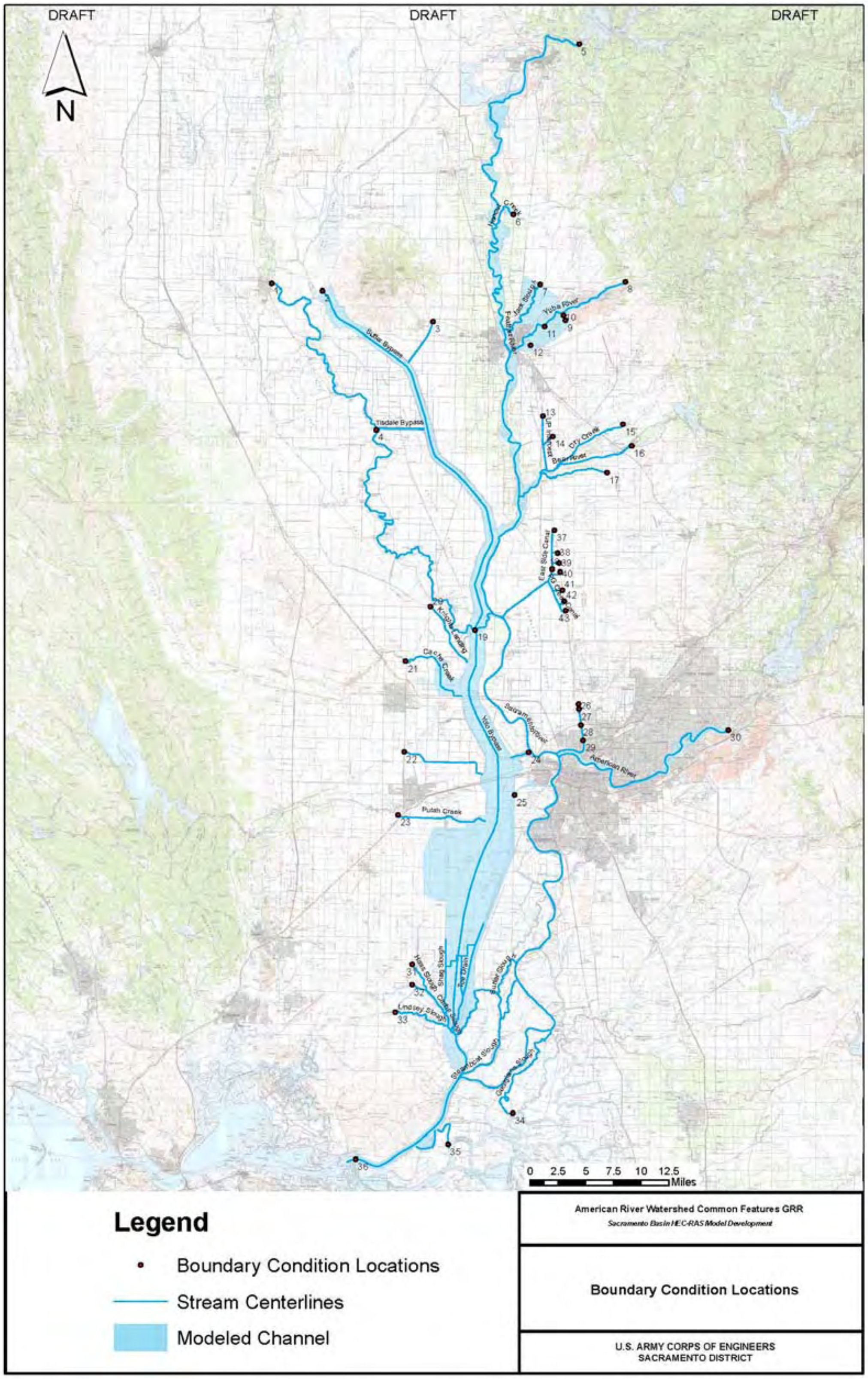
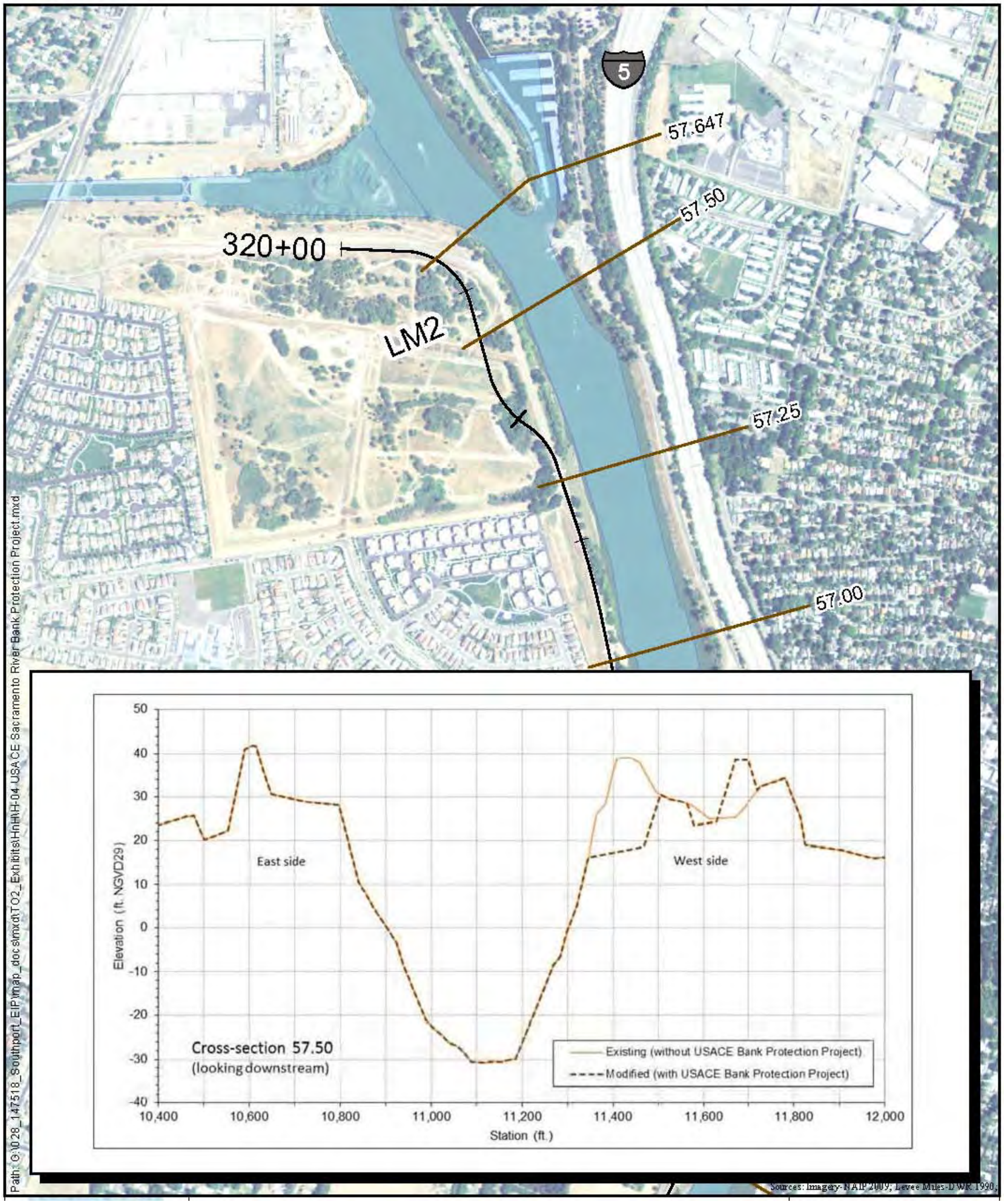


Figure 3. USACE Sacramento River HEC-RAS Model Extents (Source: USACE)



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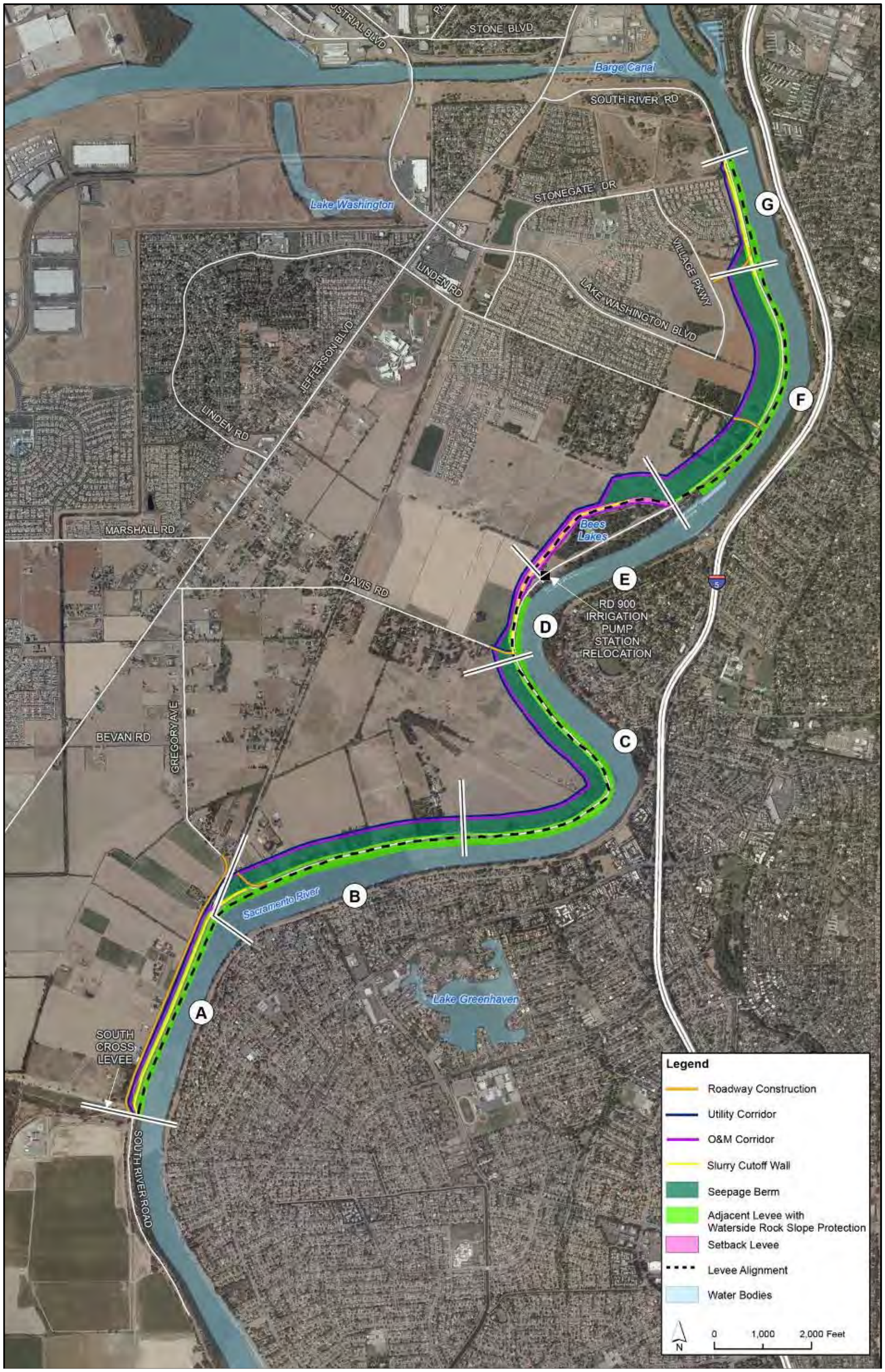

Figure 5. With Project, Alternative 1 (Source: ICF International)

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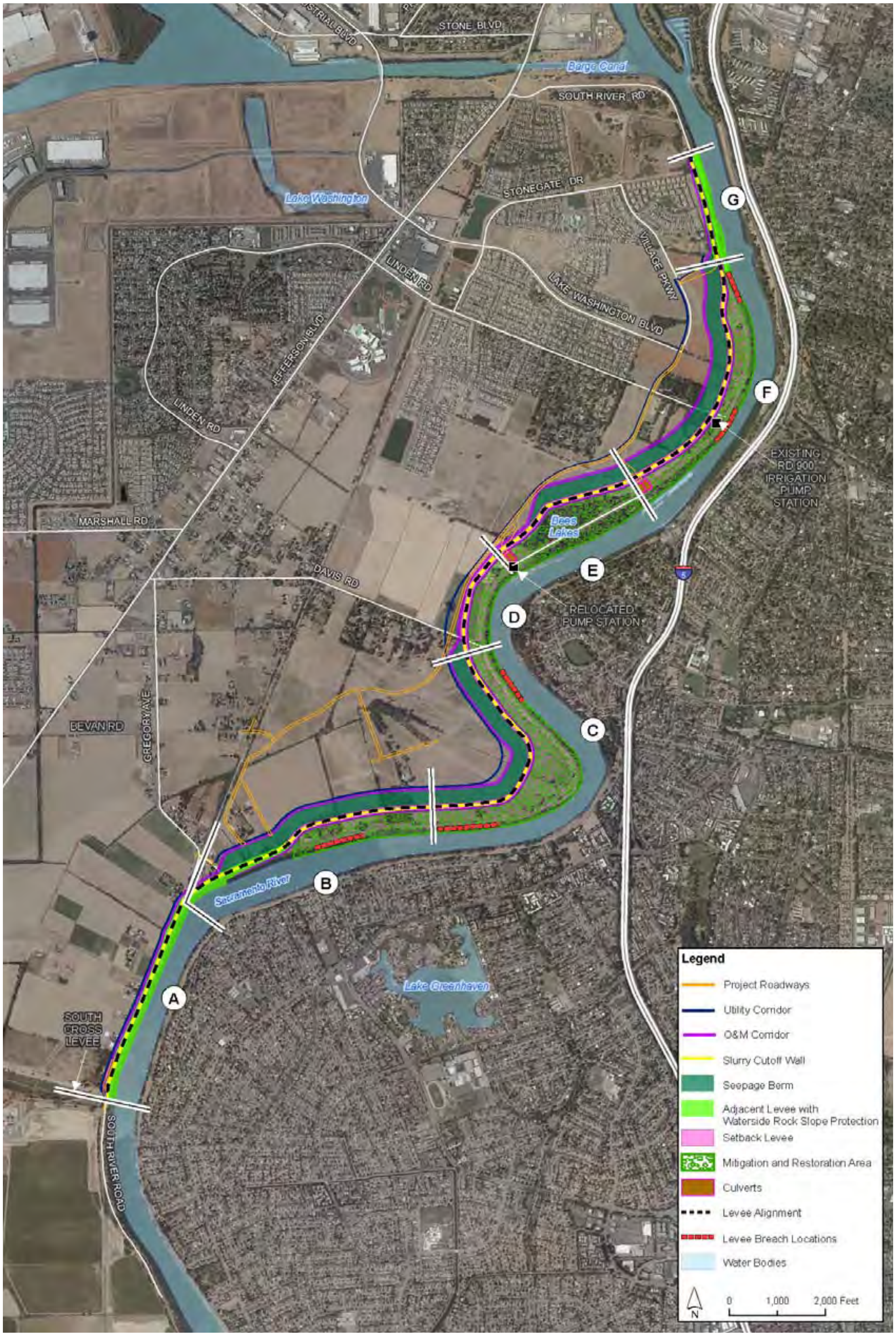

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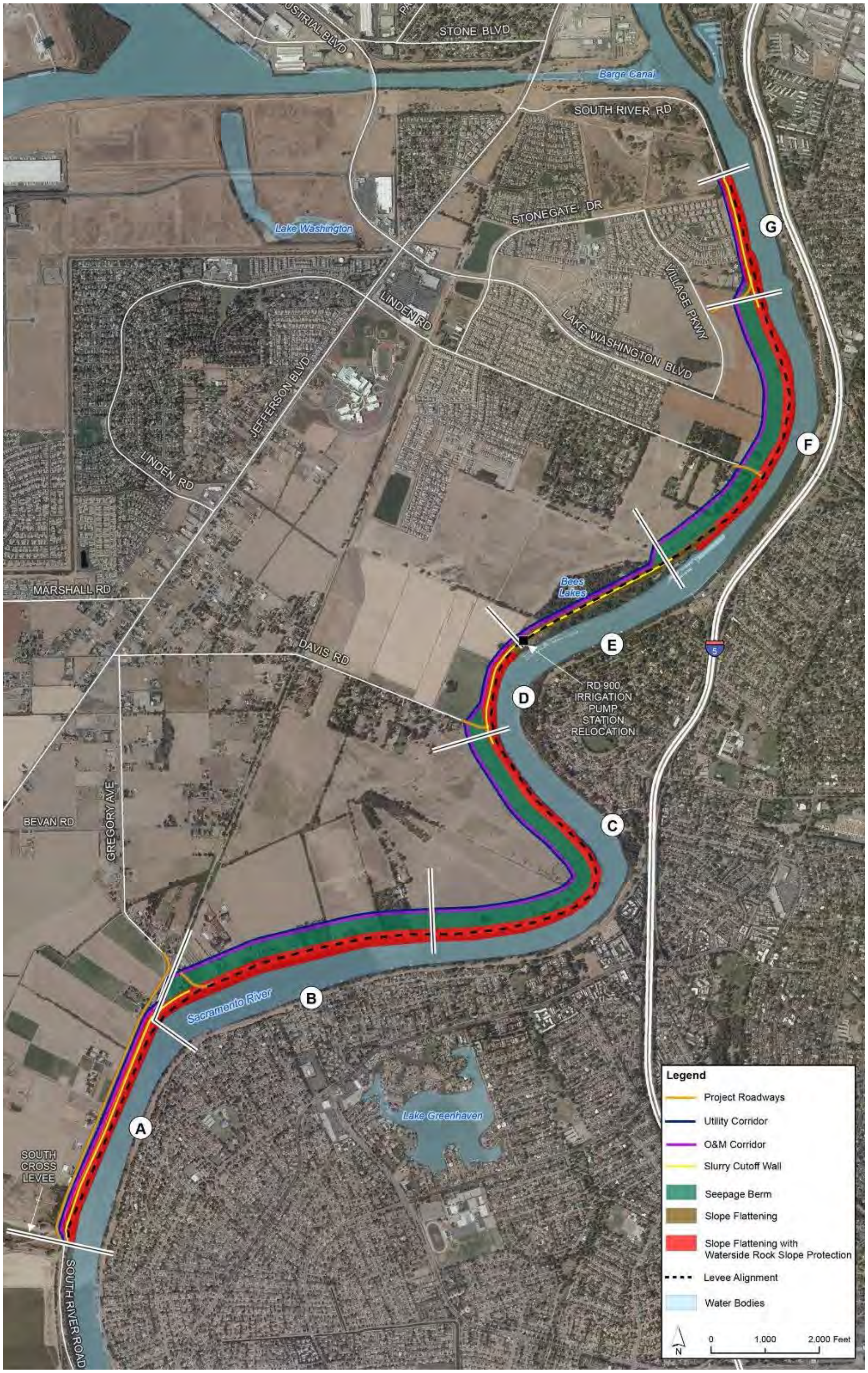

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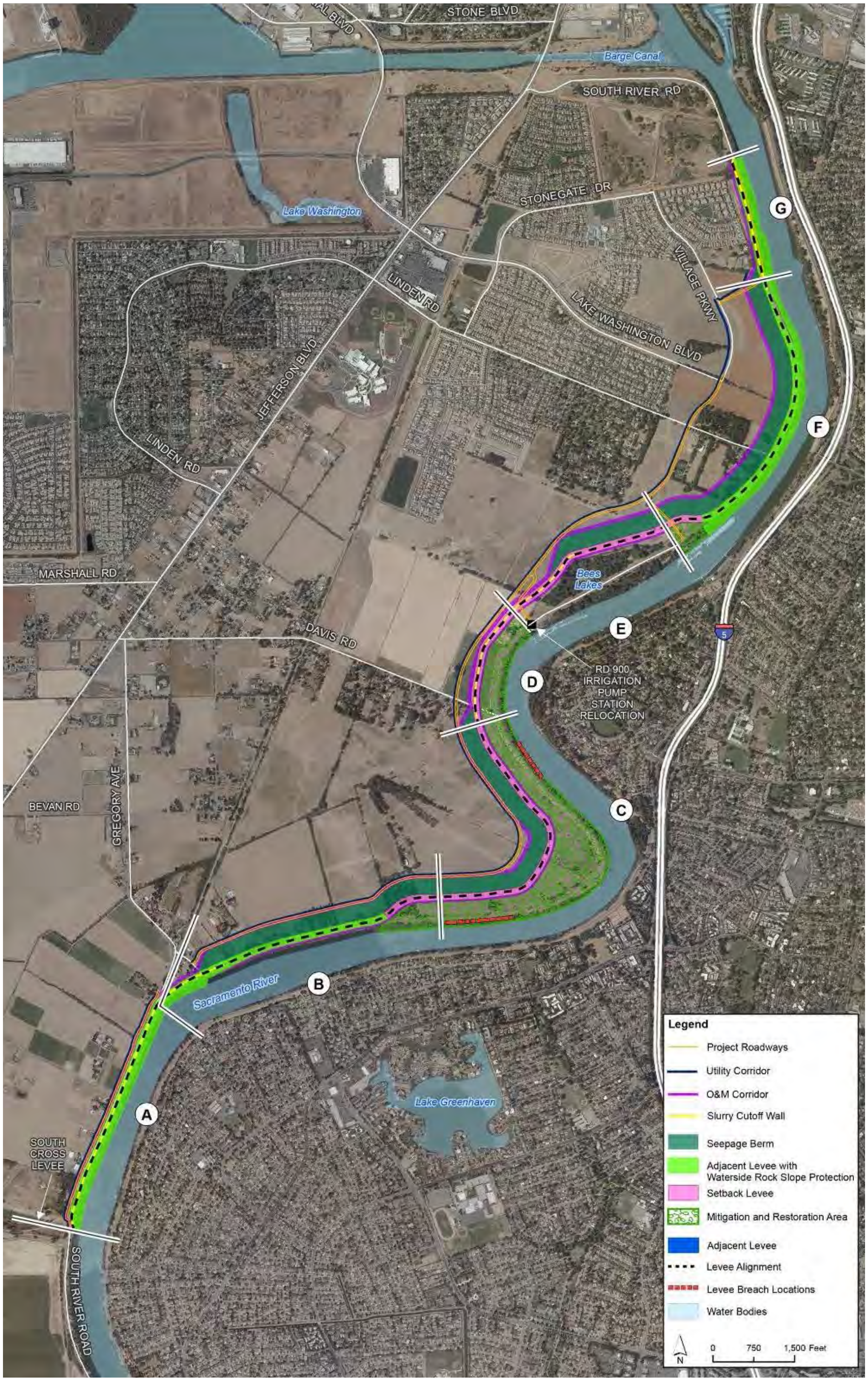

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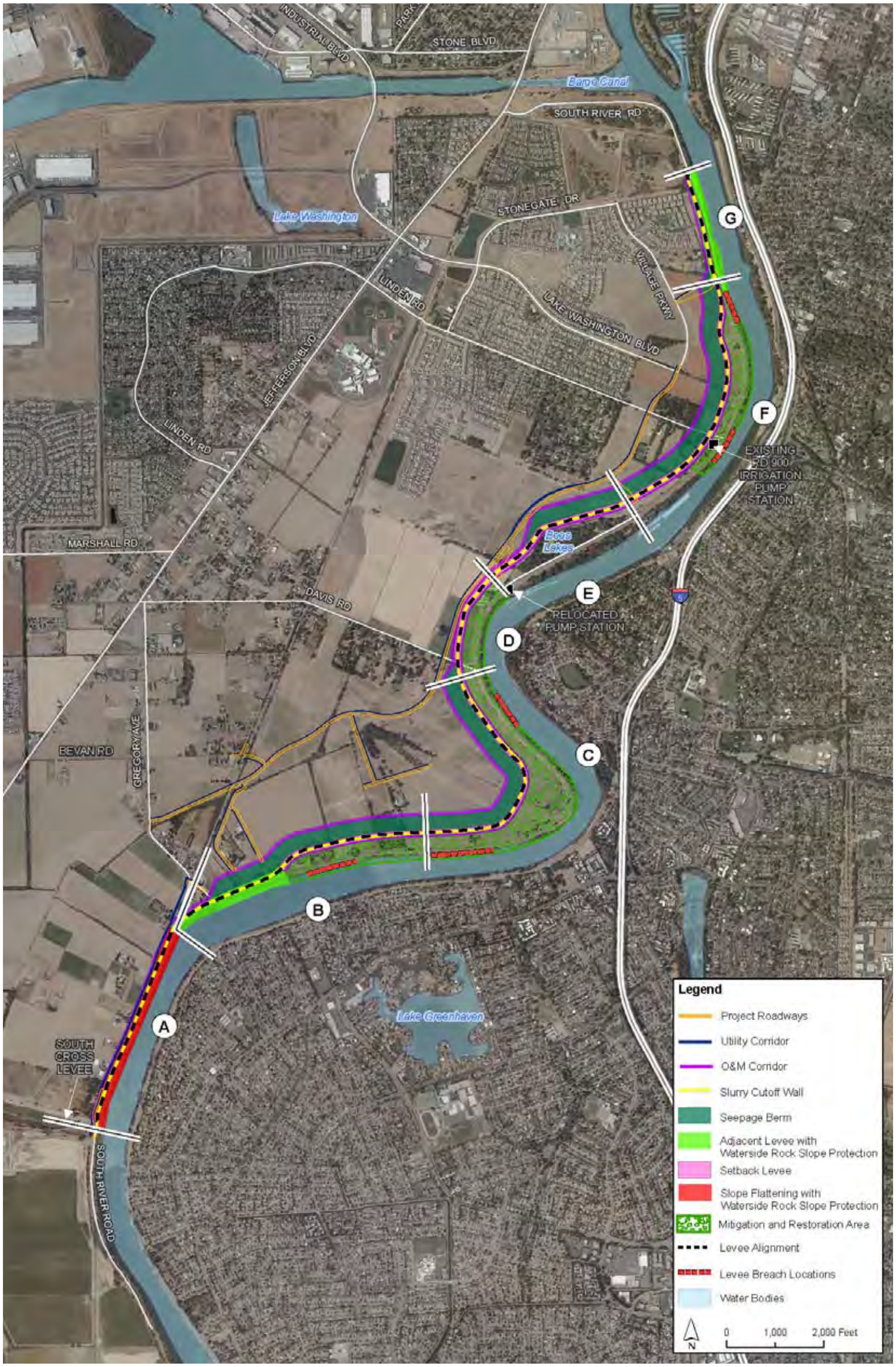

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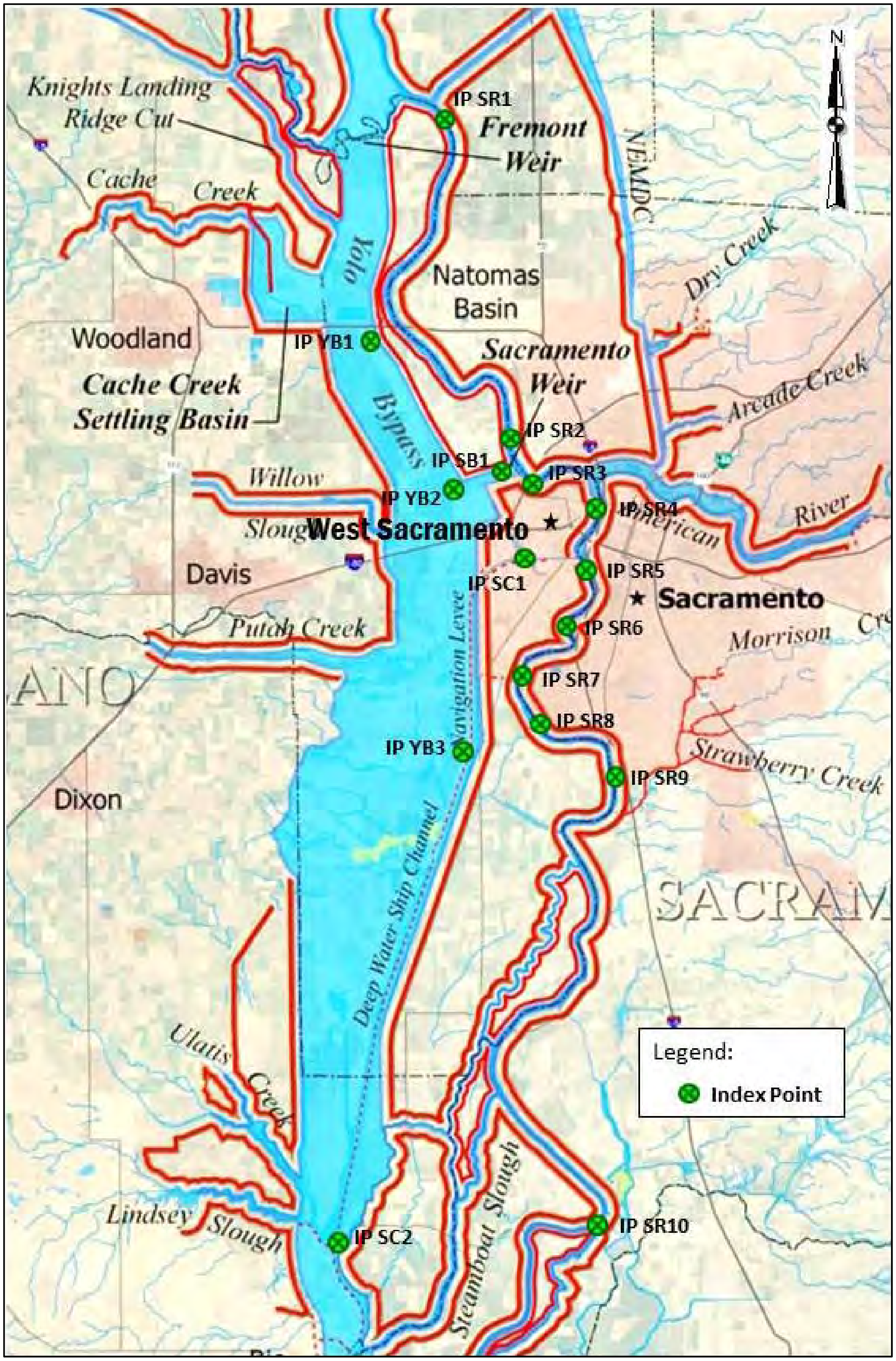

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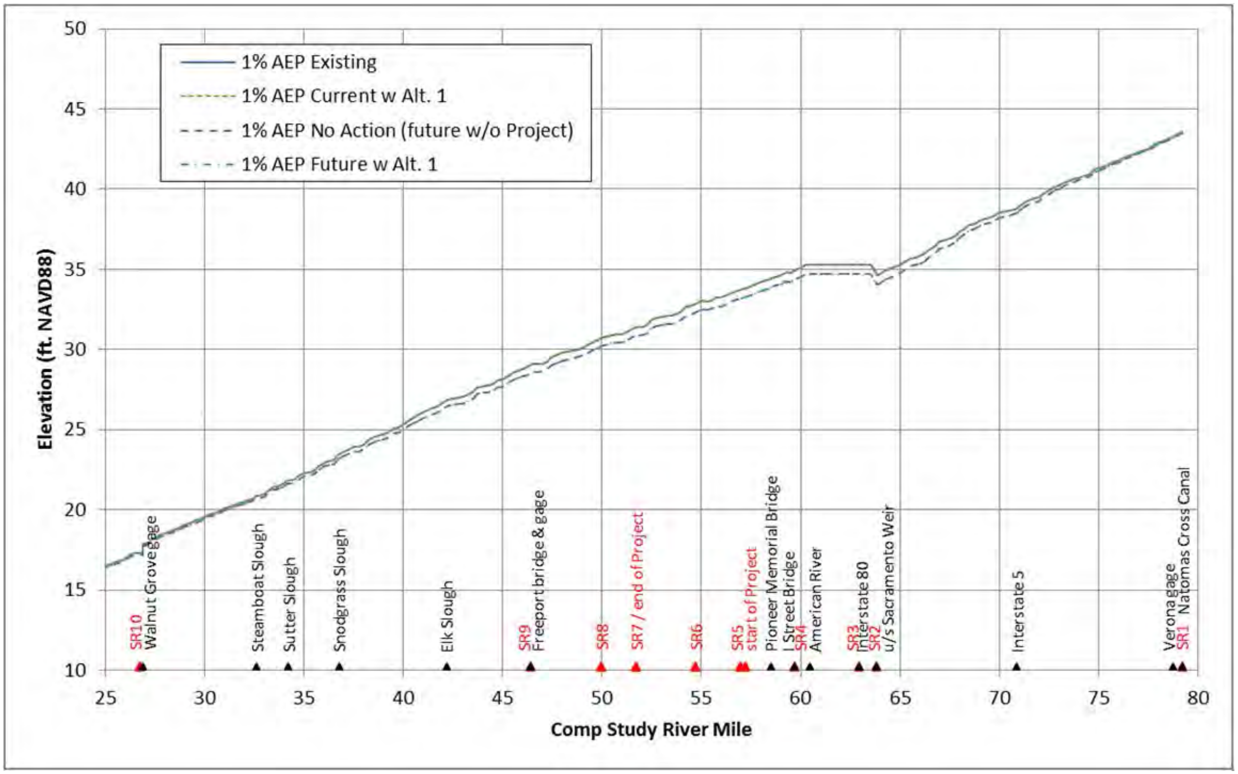


Figure 11. Maximum Water Surface Elevations; Sacramento River; Alternative 1; 1% AEP

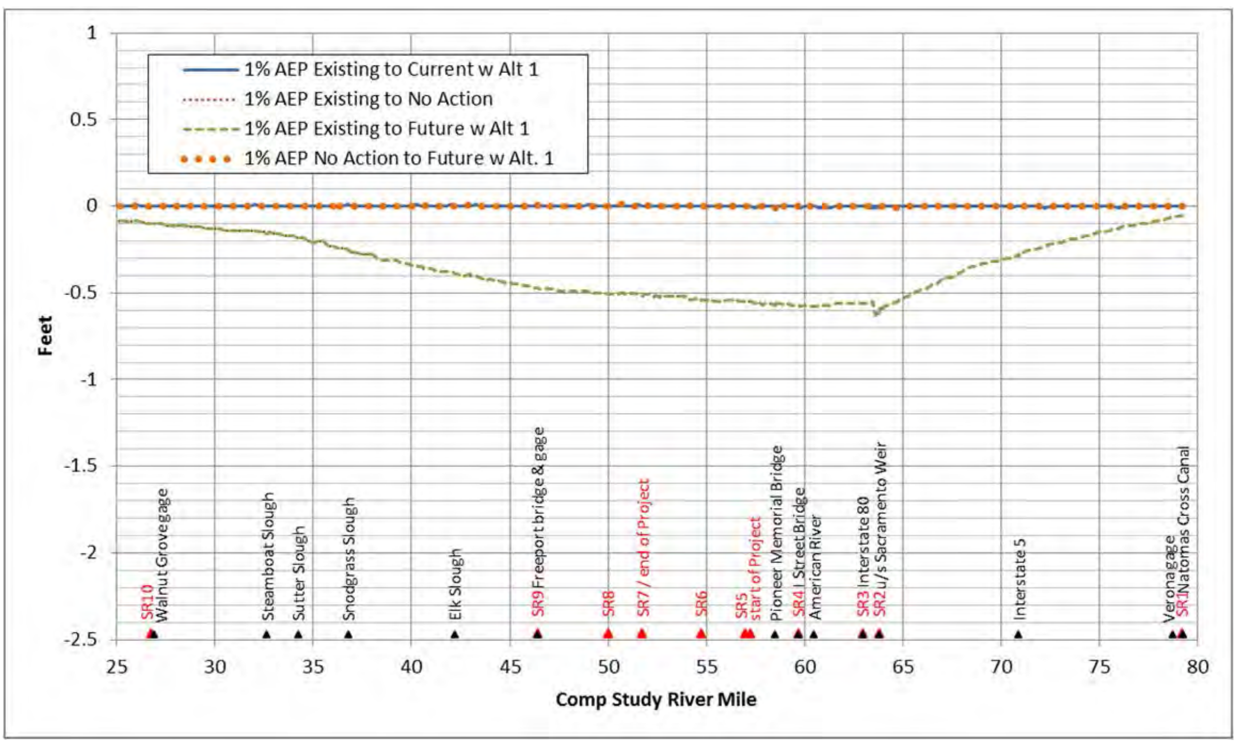
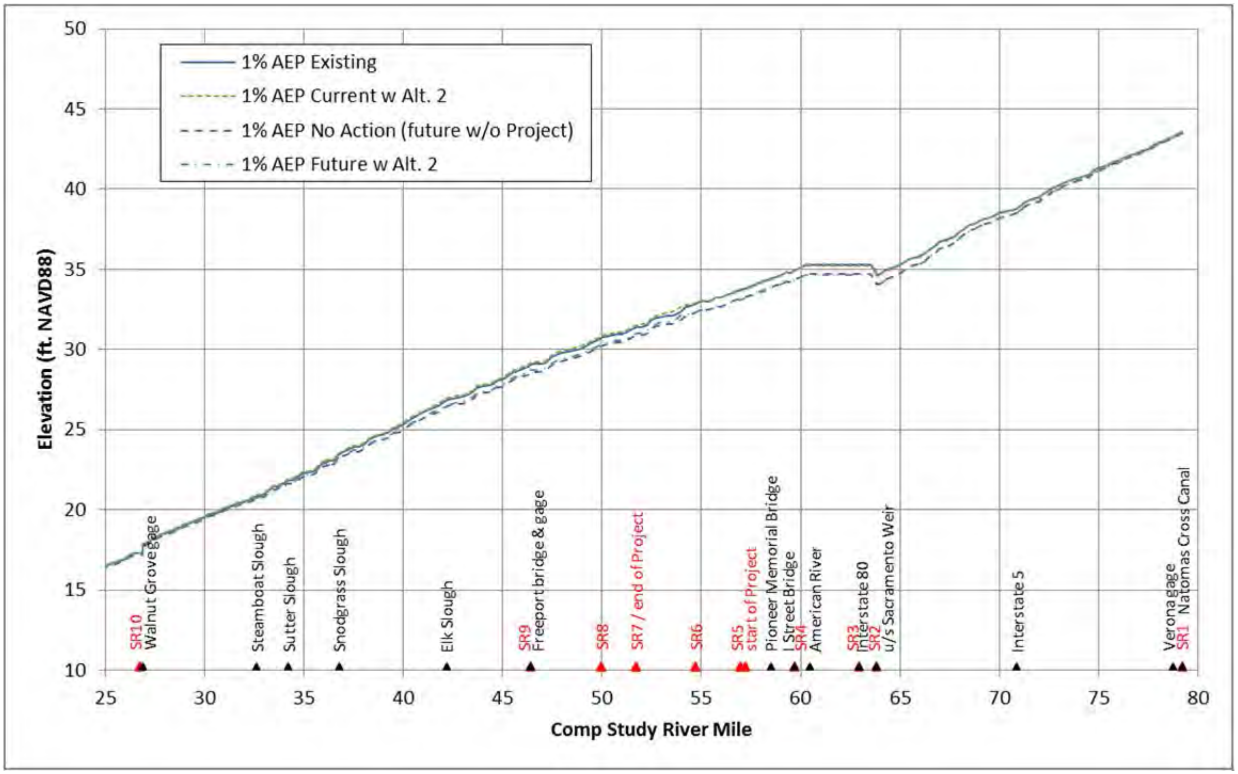


Figure 12. Maximum Water Surface Elevation Impacts; Sacramento River; Alternative 1; 1% AEP



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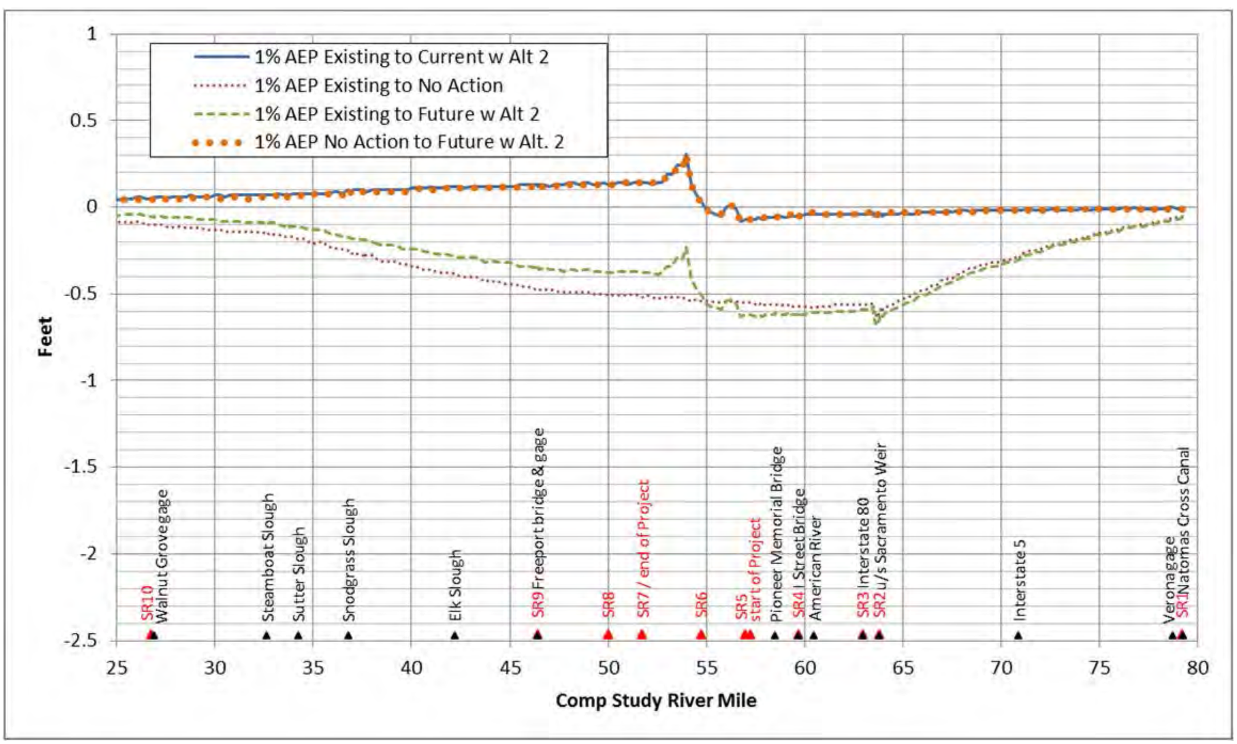

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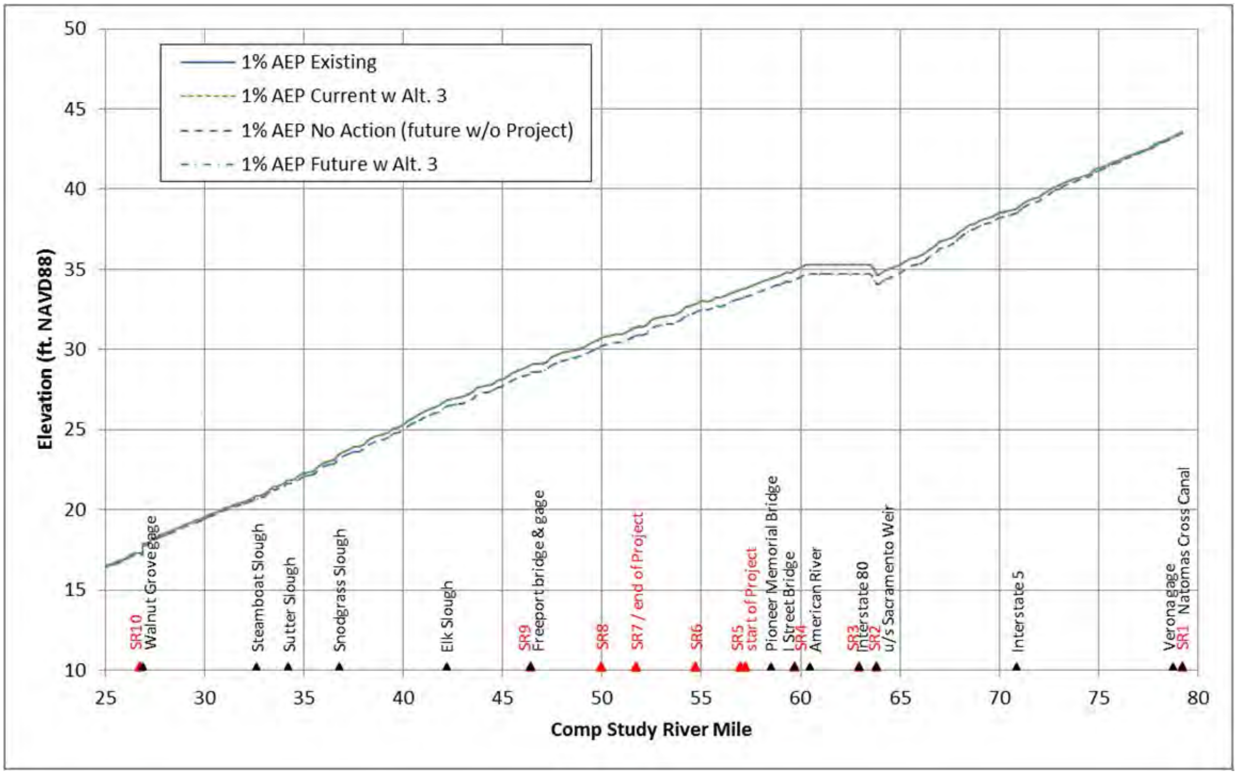

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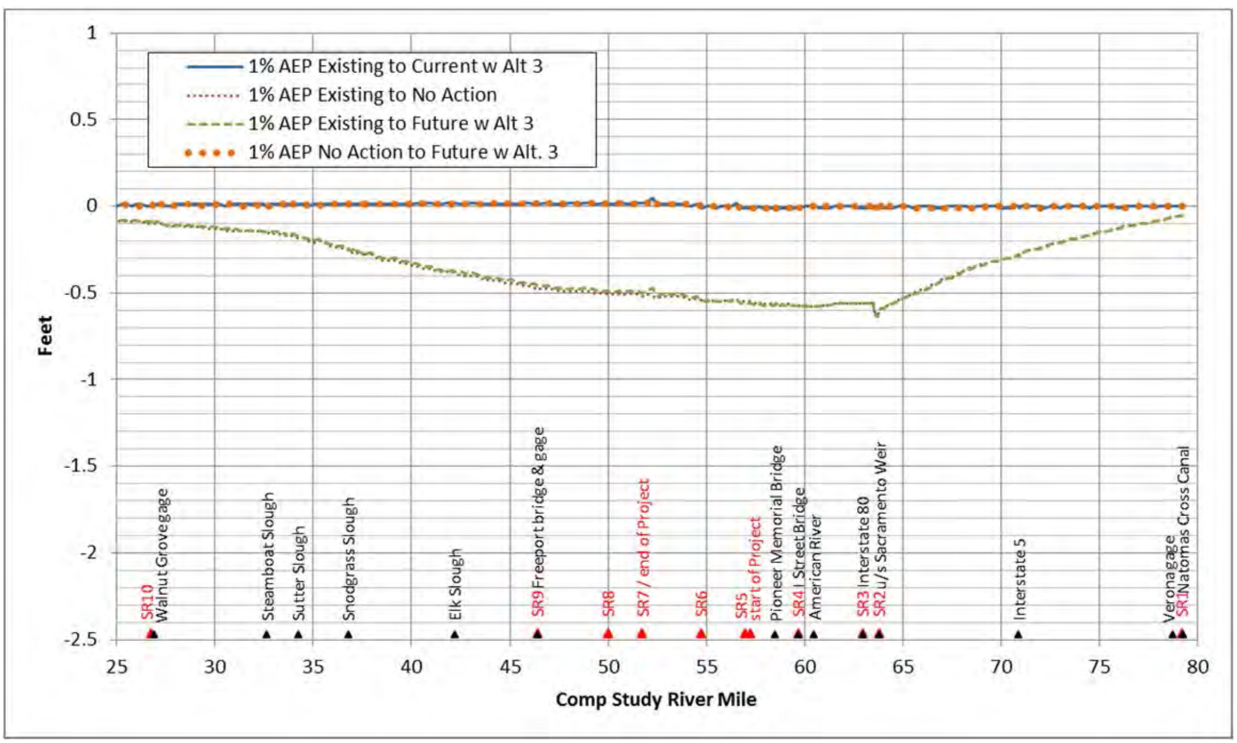

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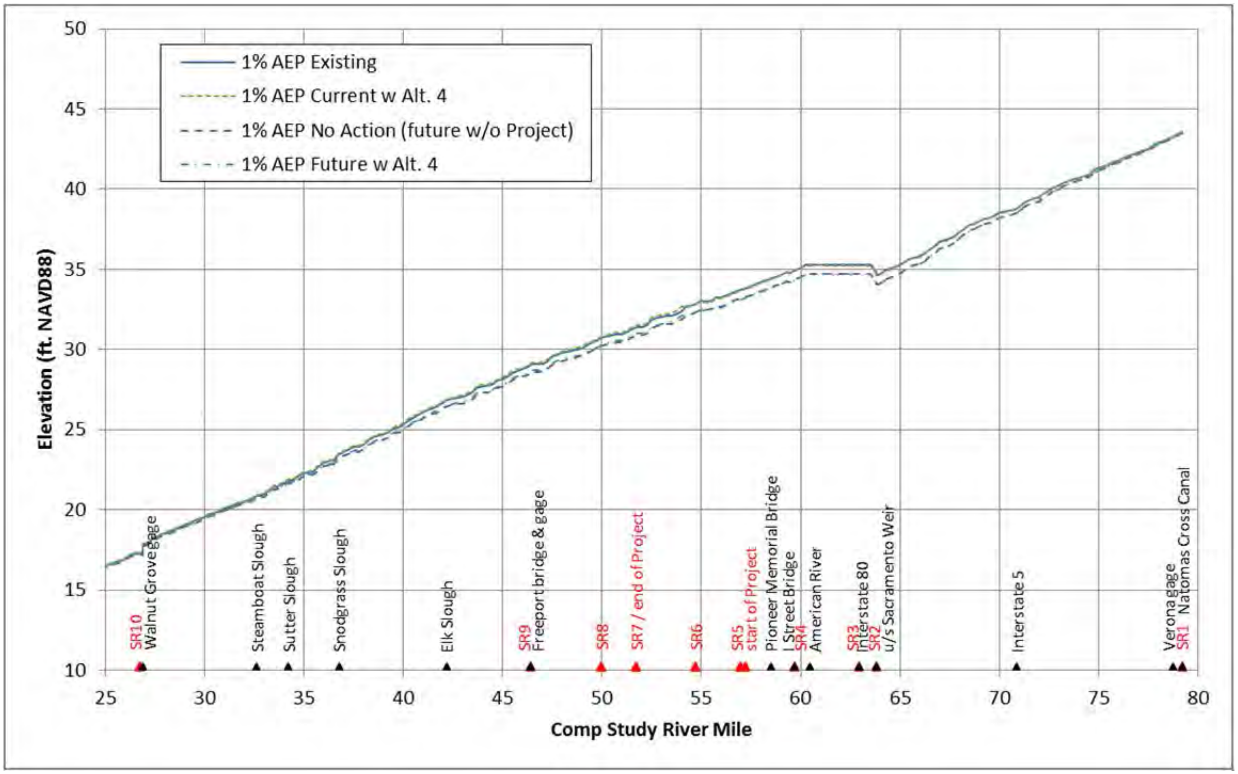

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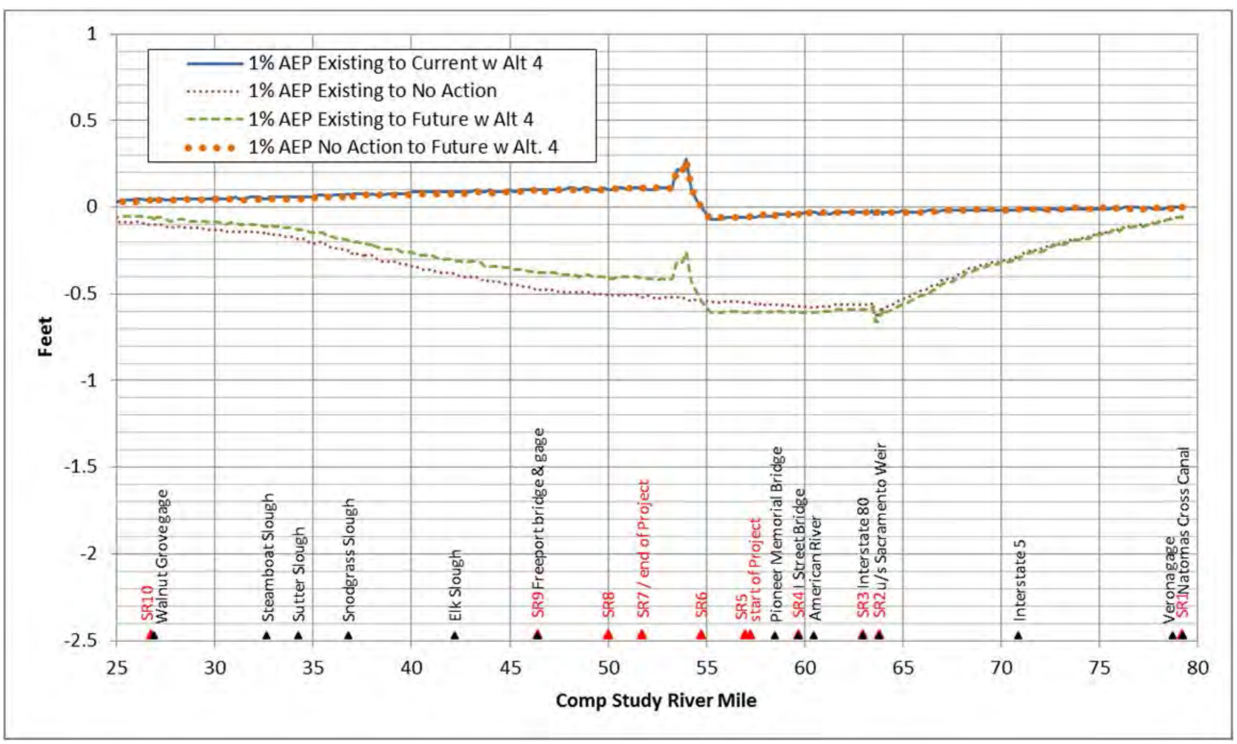
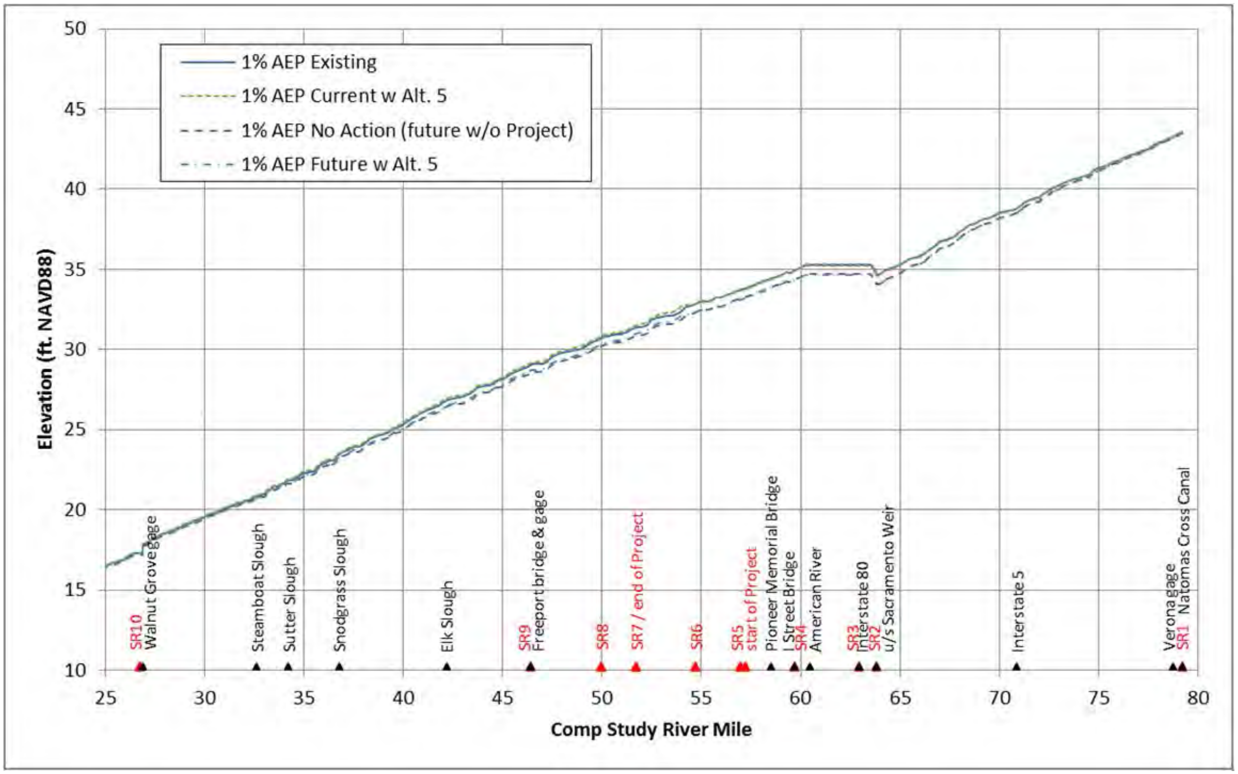


Figure 18. Maximum Water Surface Elevation Impacts; Sacramento River; Alternative 4; 1% AEP



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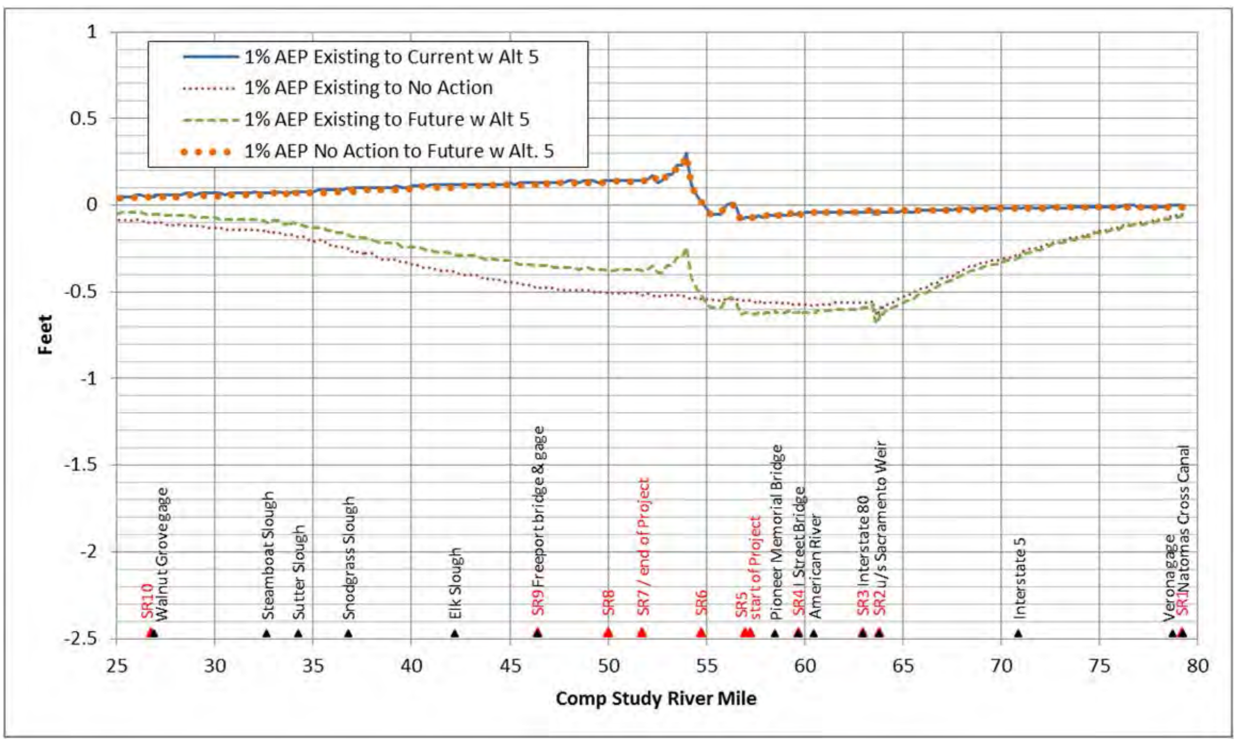

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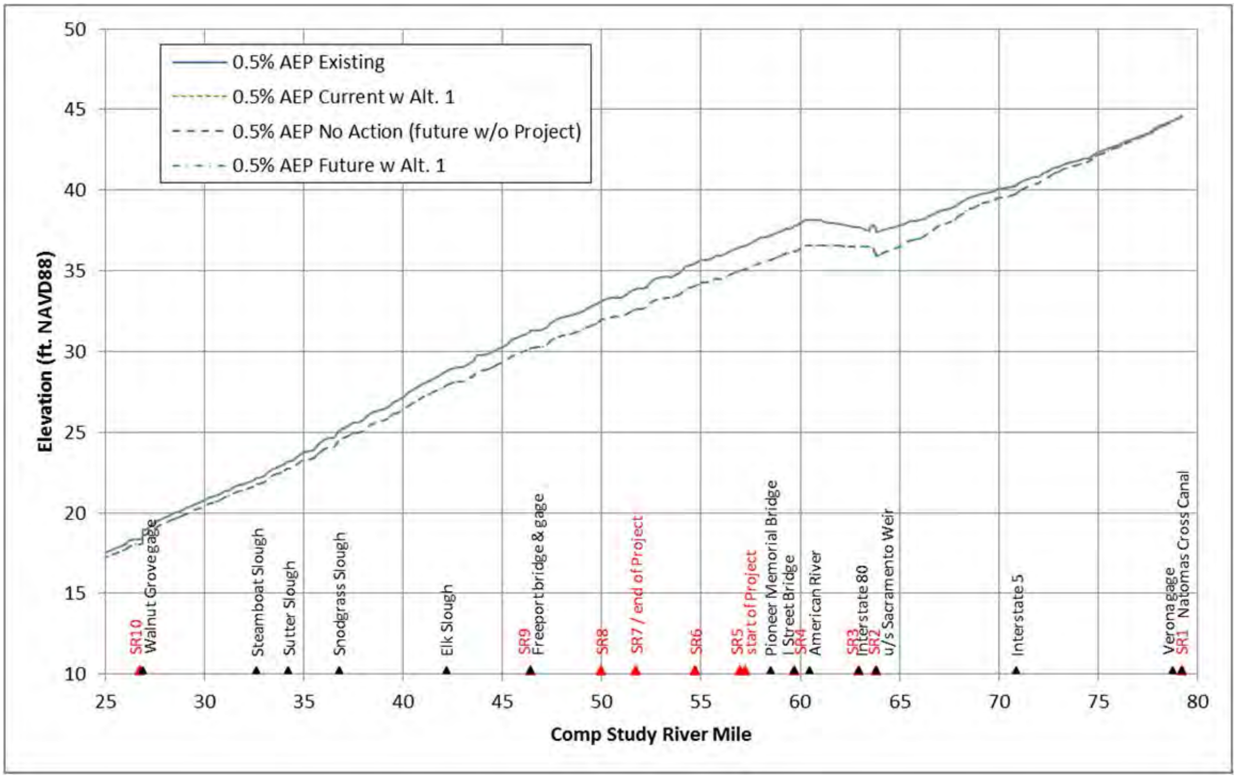

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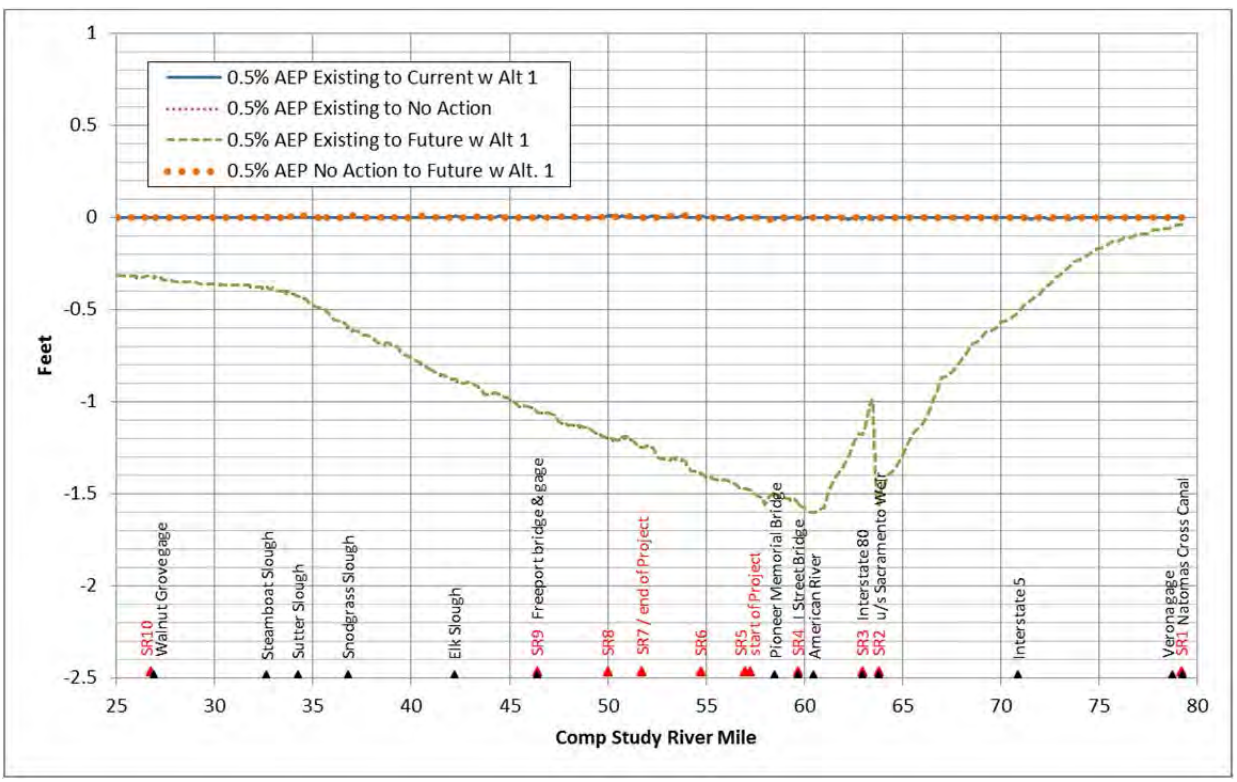

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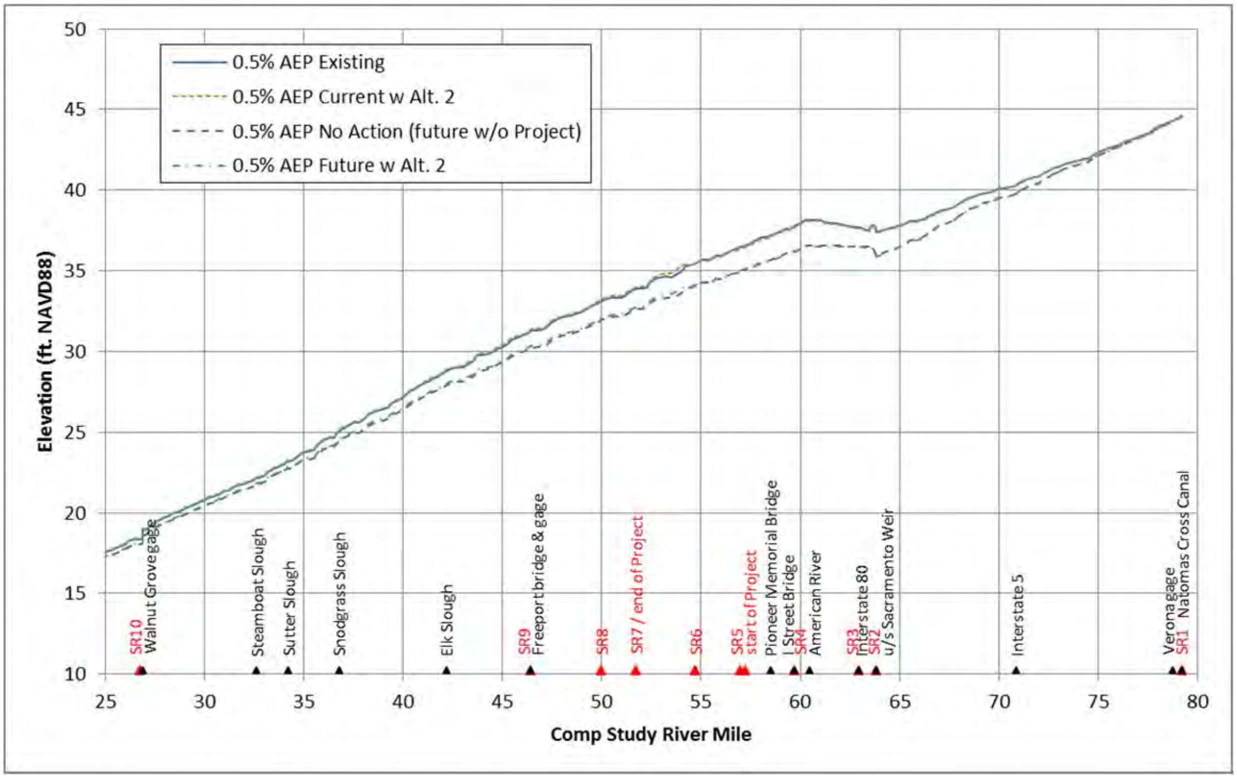

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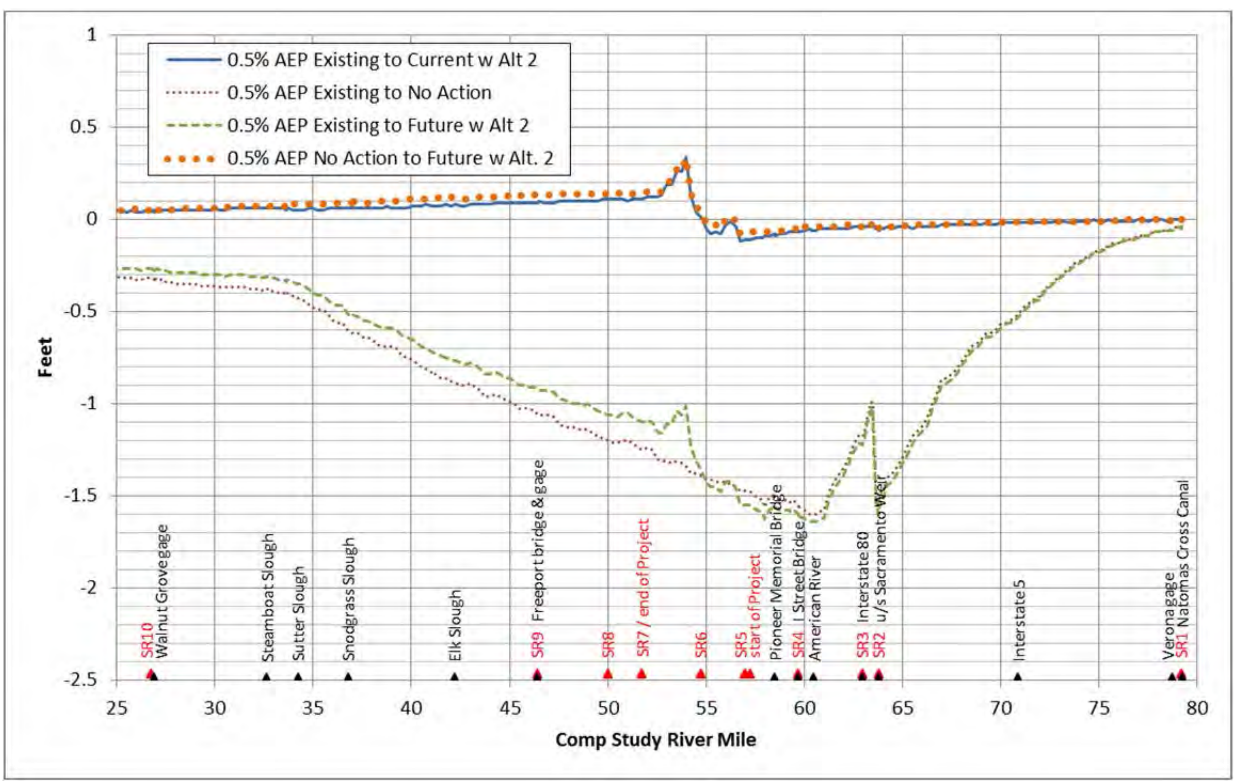

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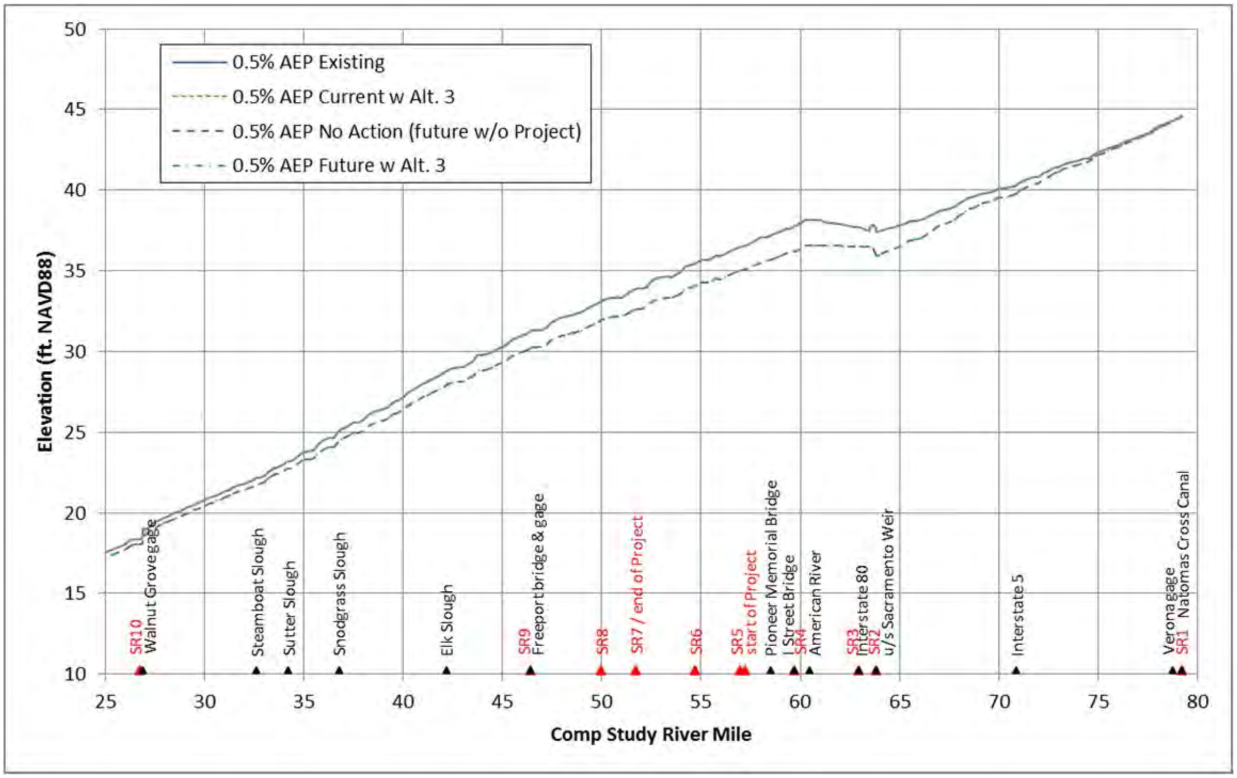

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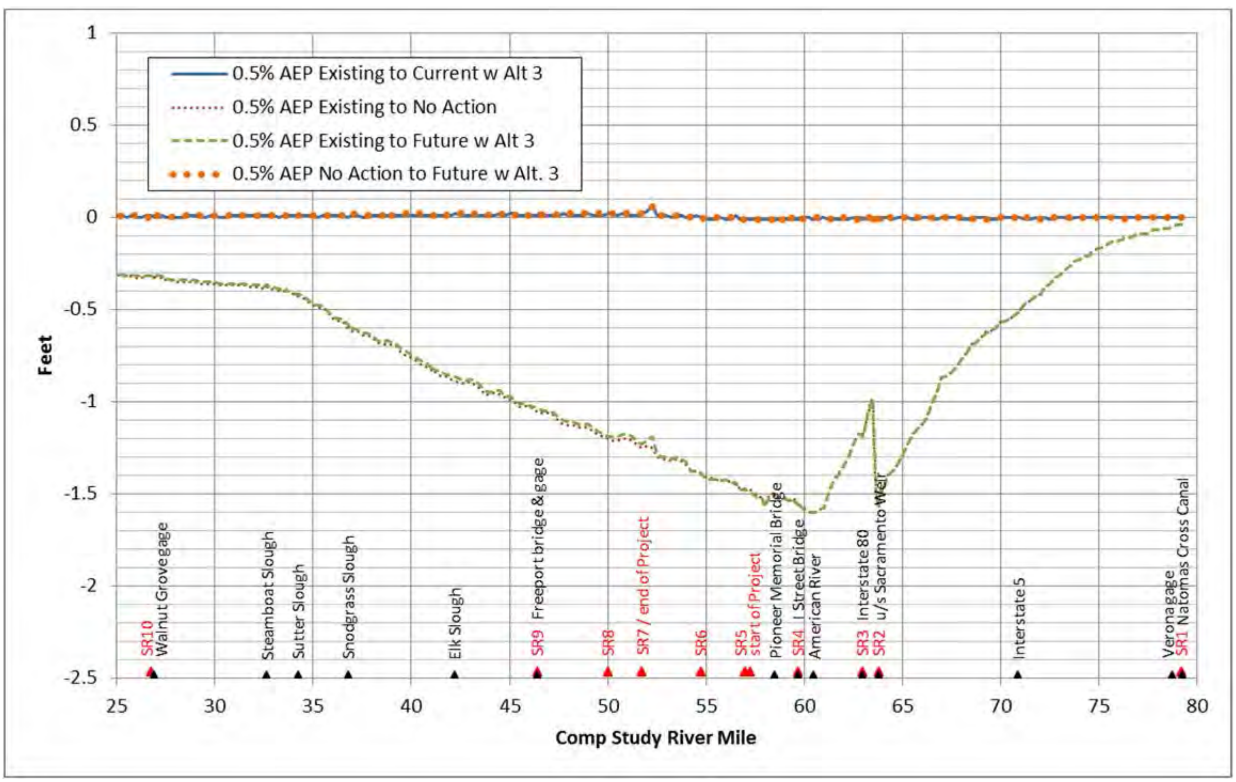

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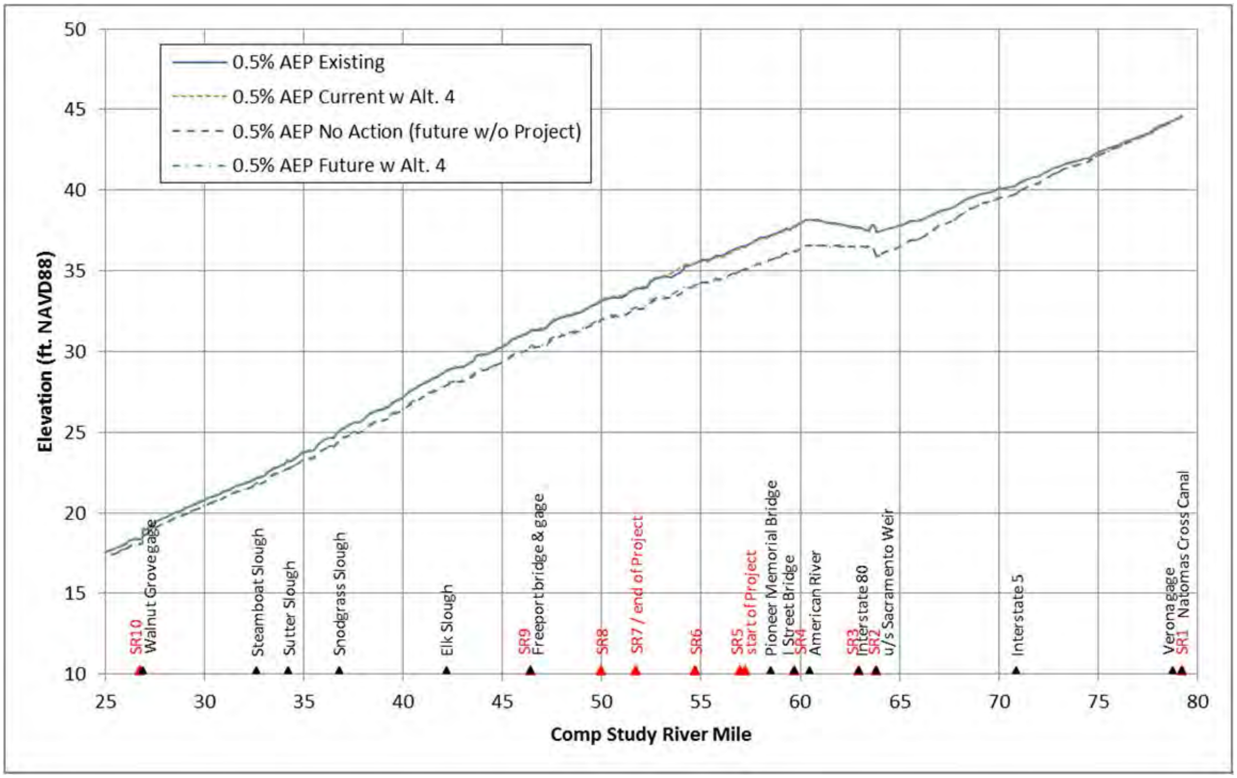
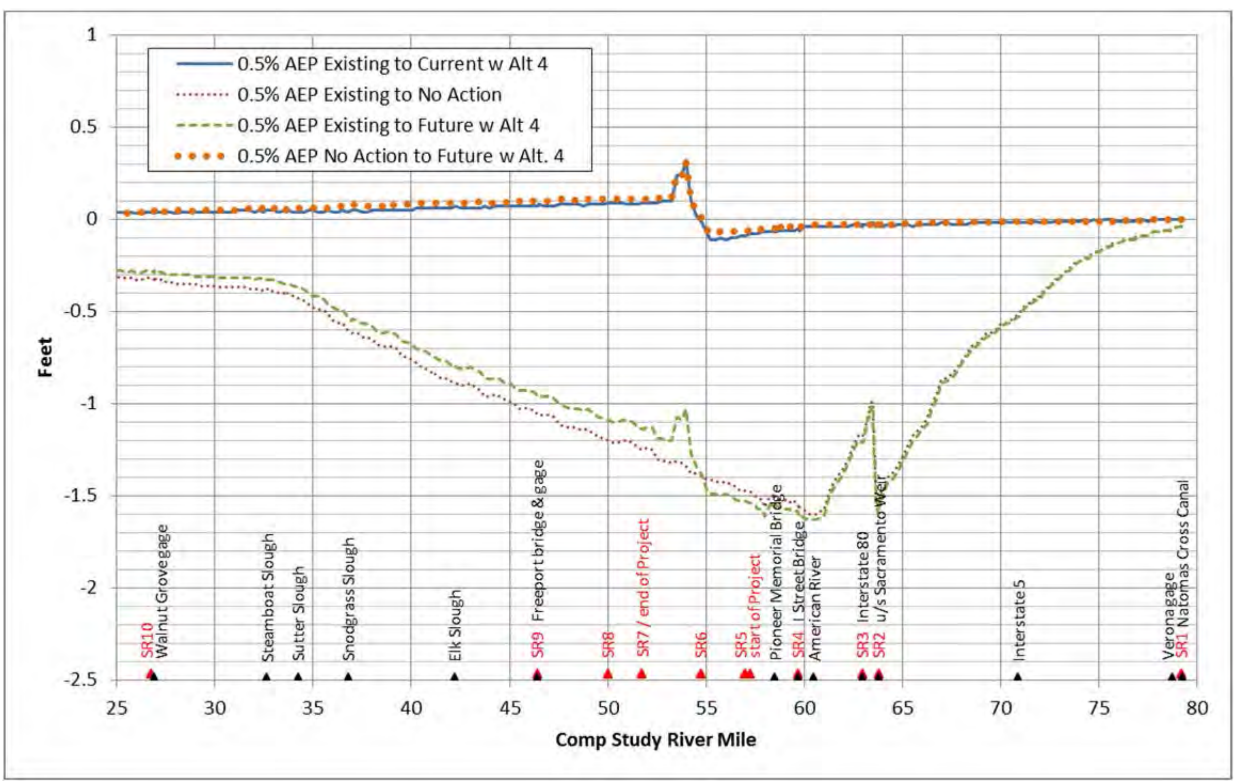


Figure 27. Maximum Water Surface Elevations; Sacramento River; Alternative 4; 0.5% AEP



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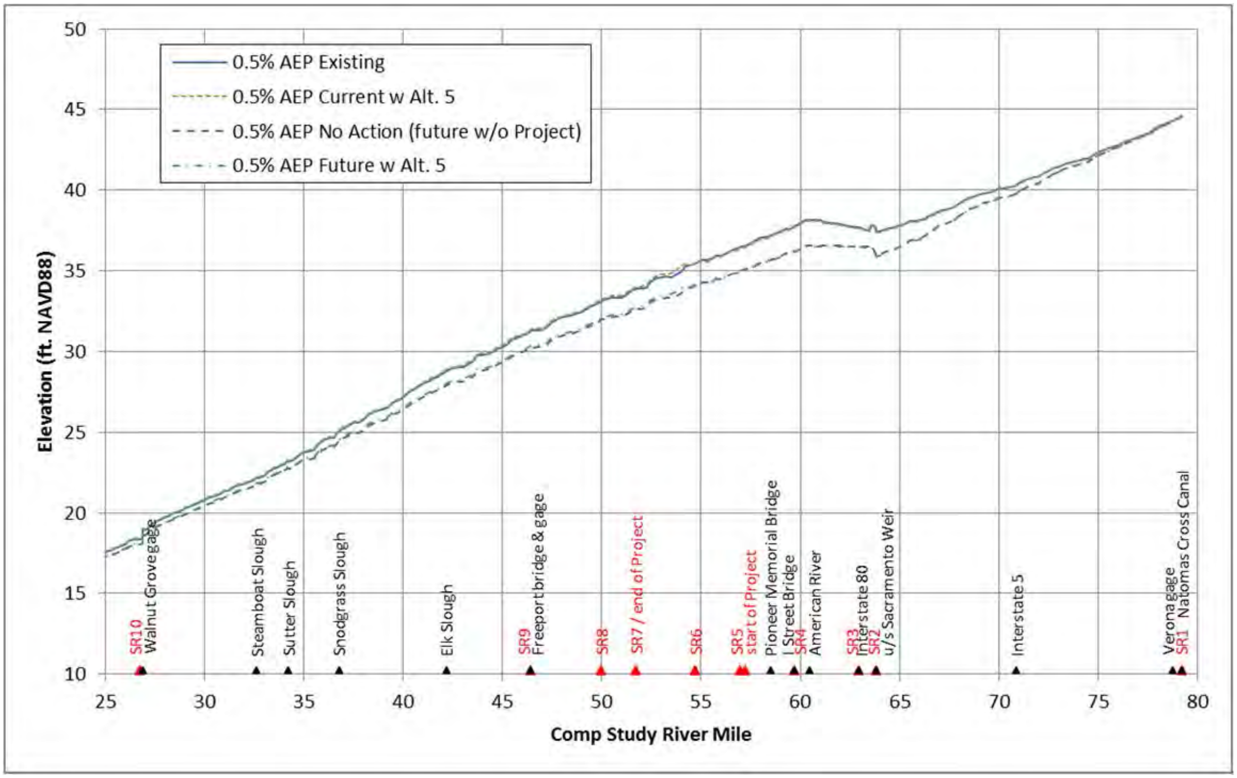

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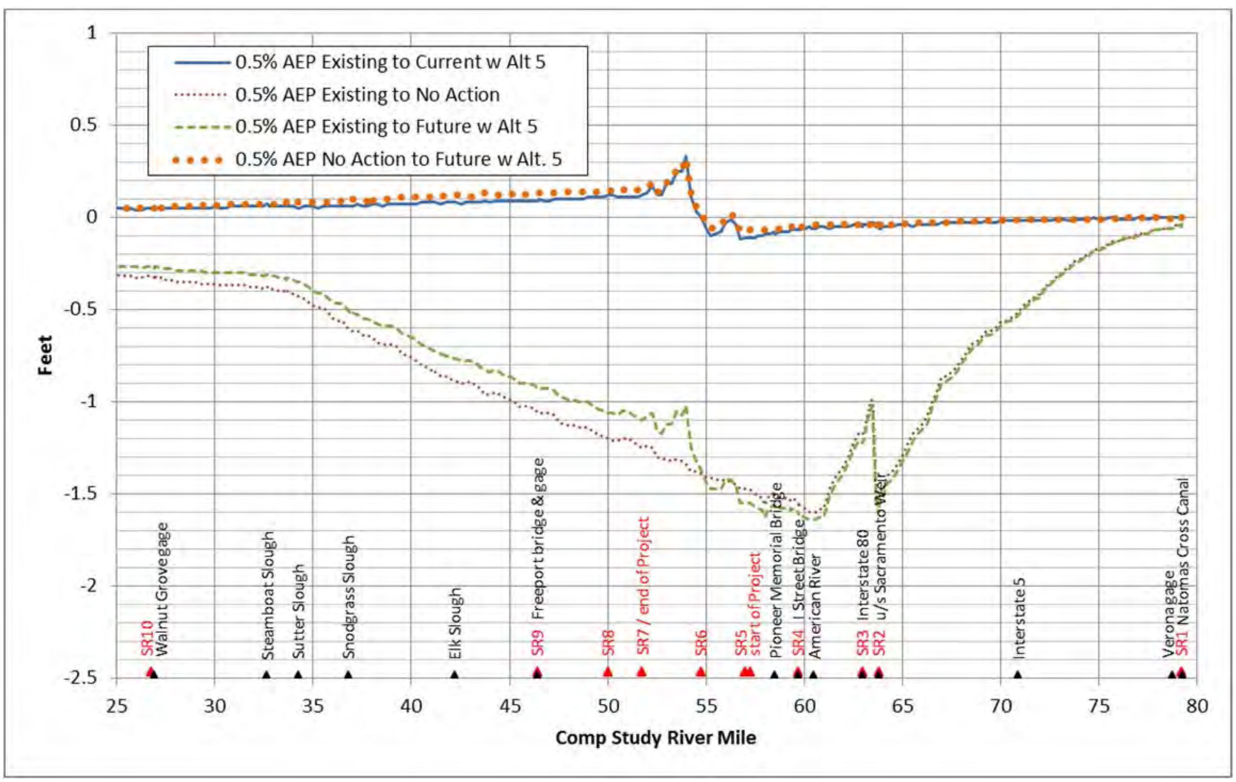

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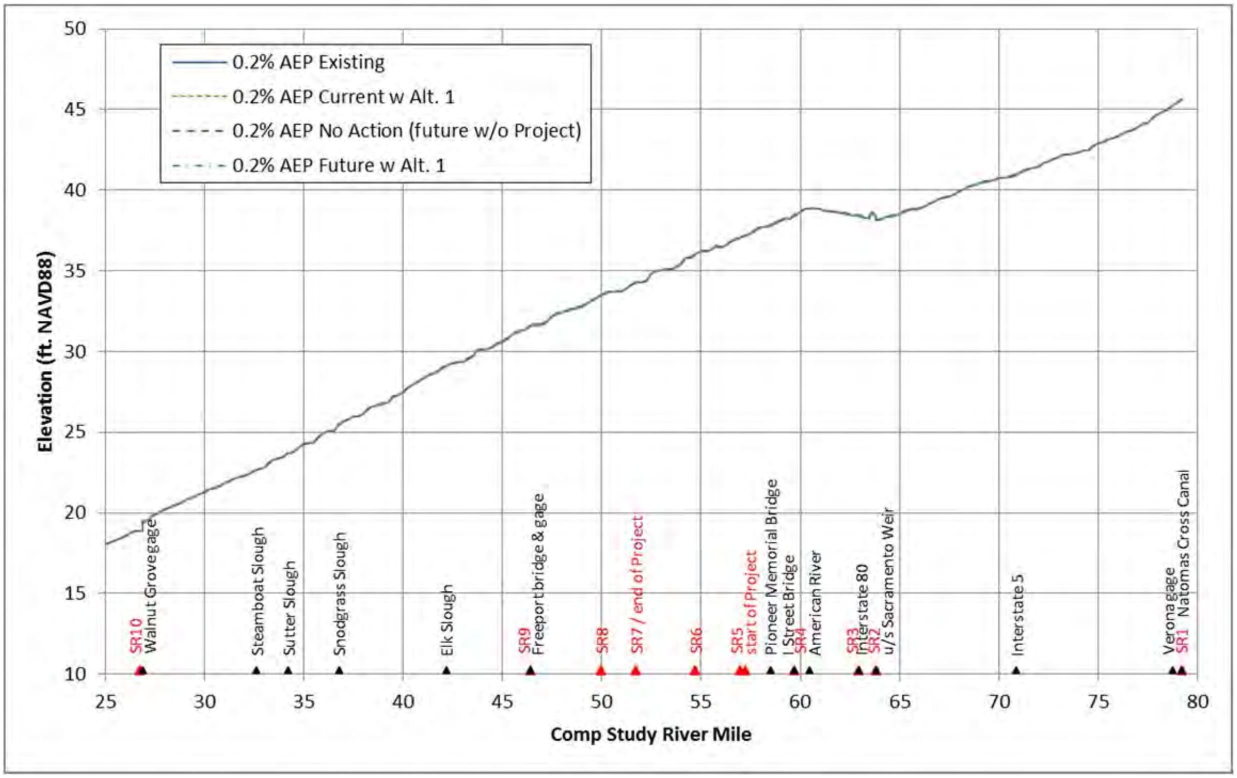

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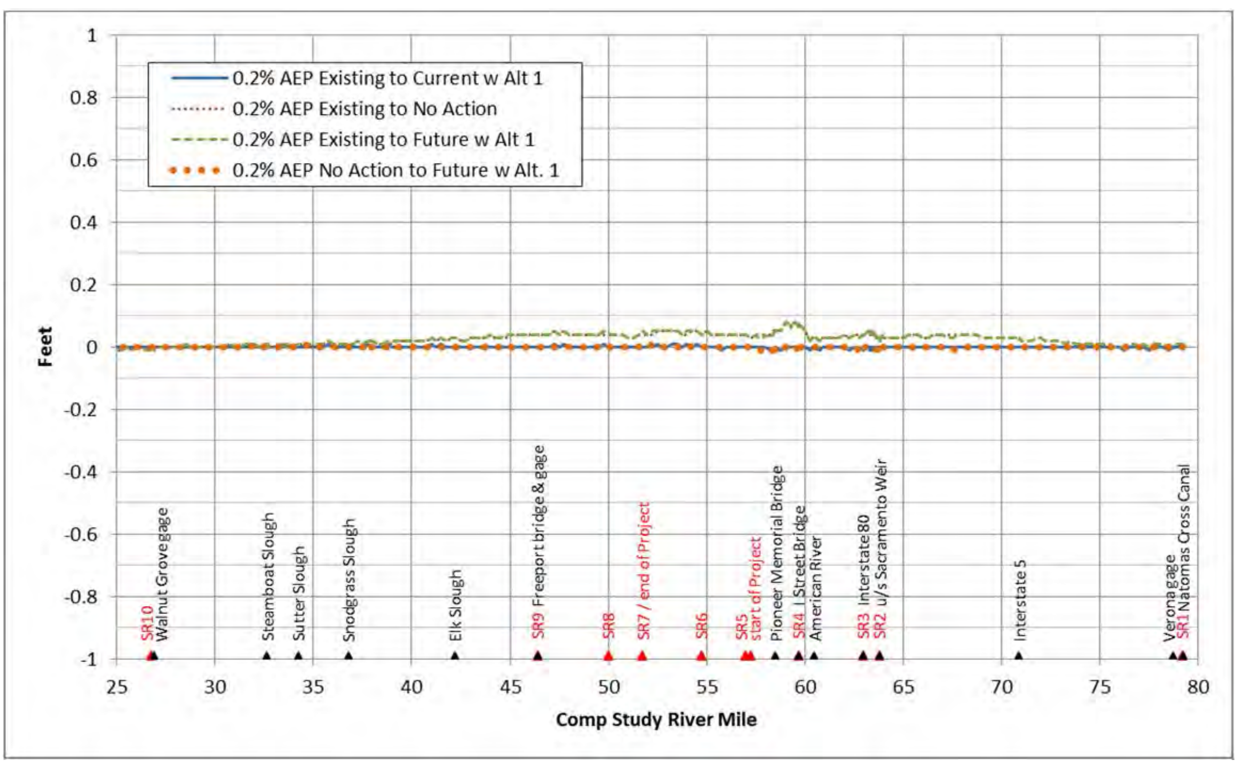

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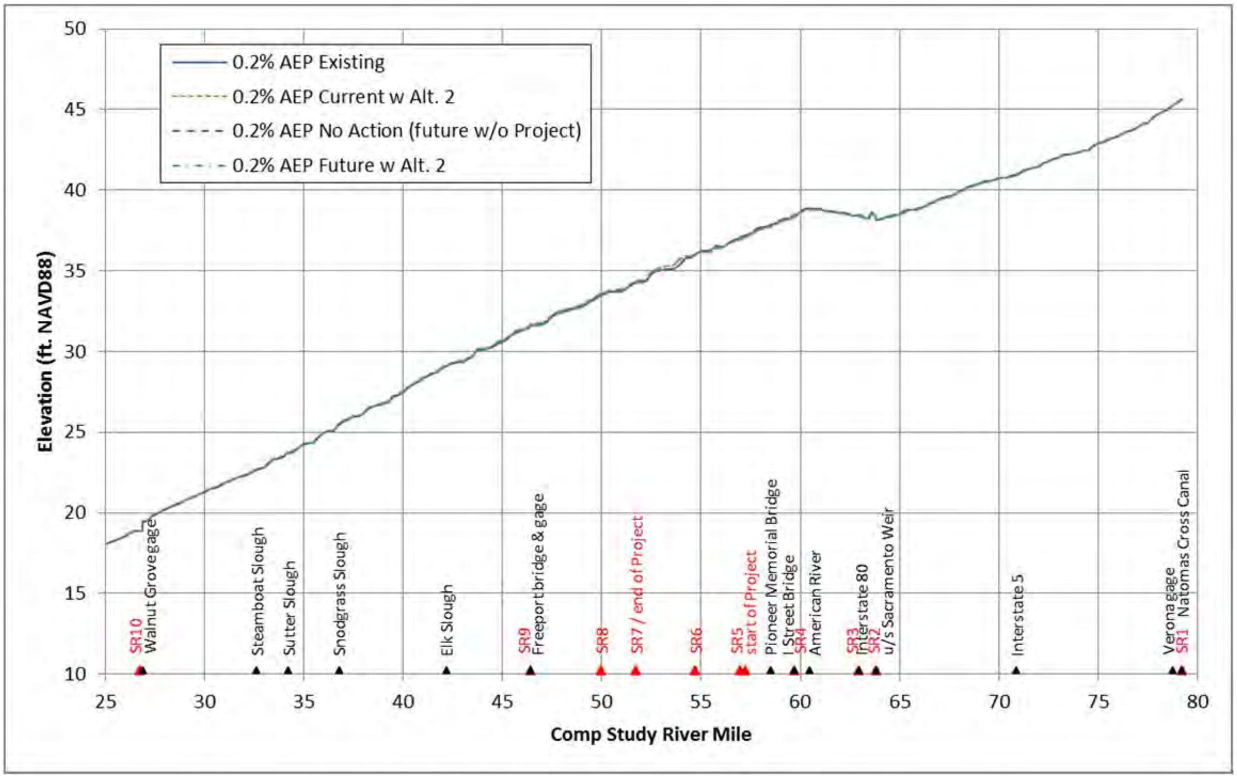

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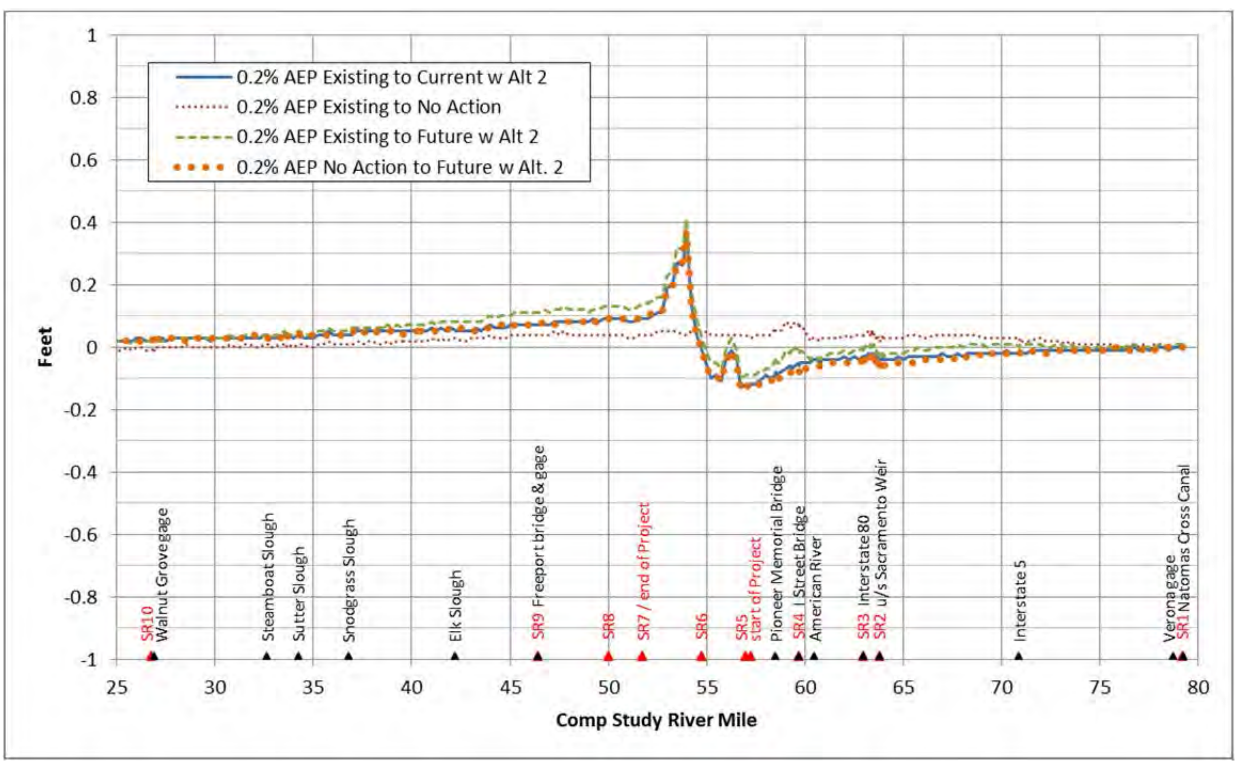

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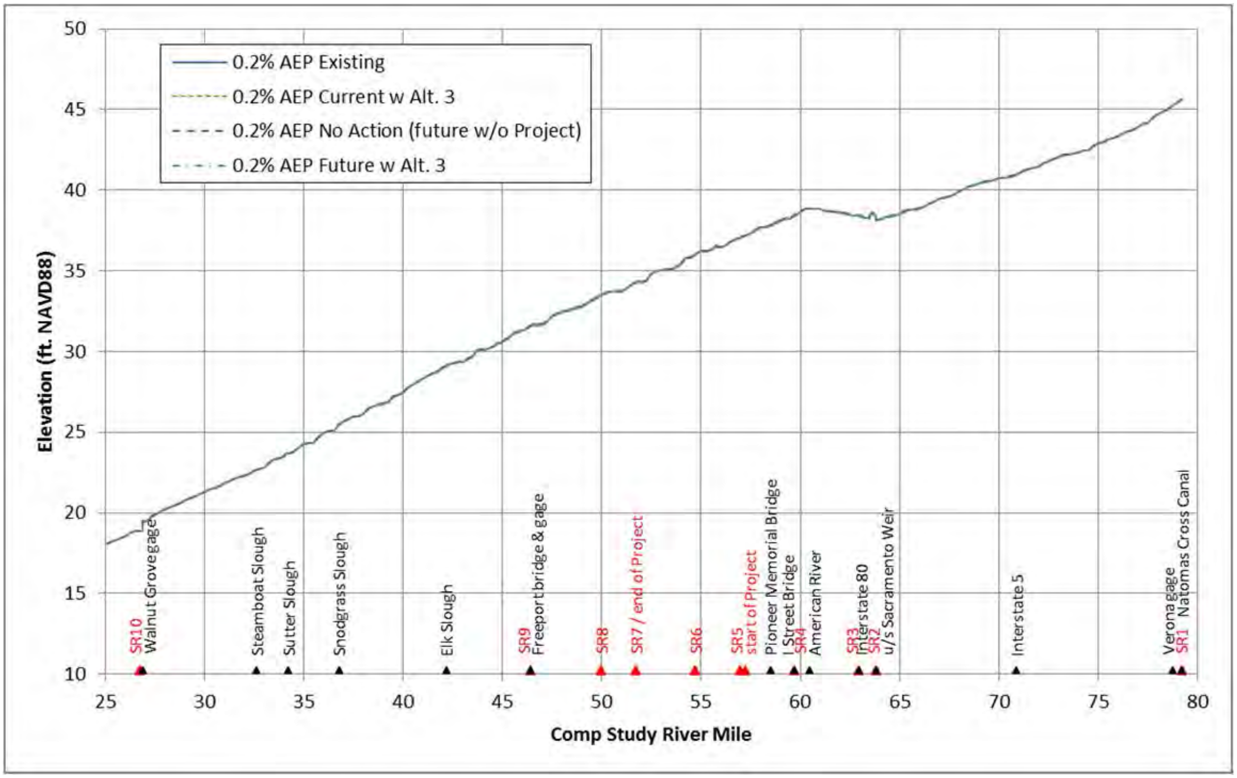

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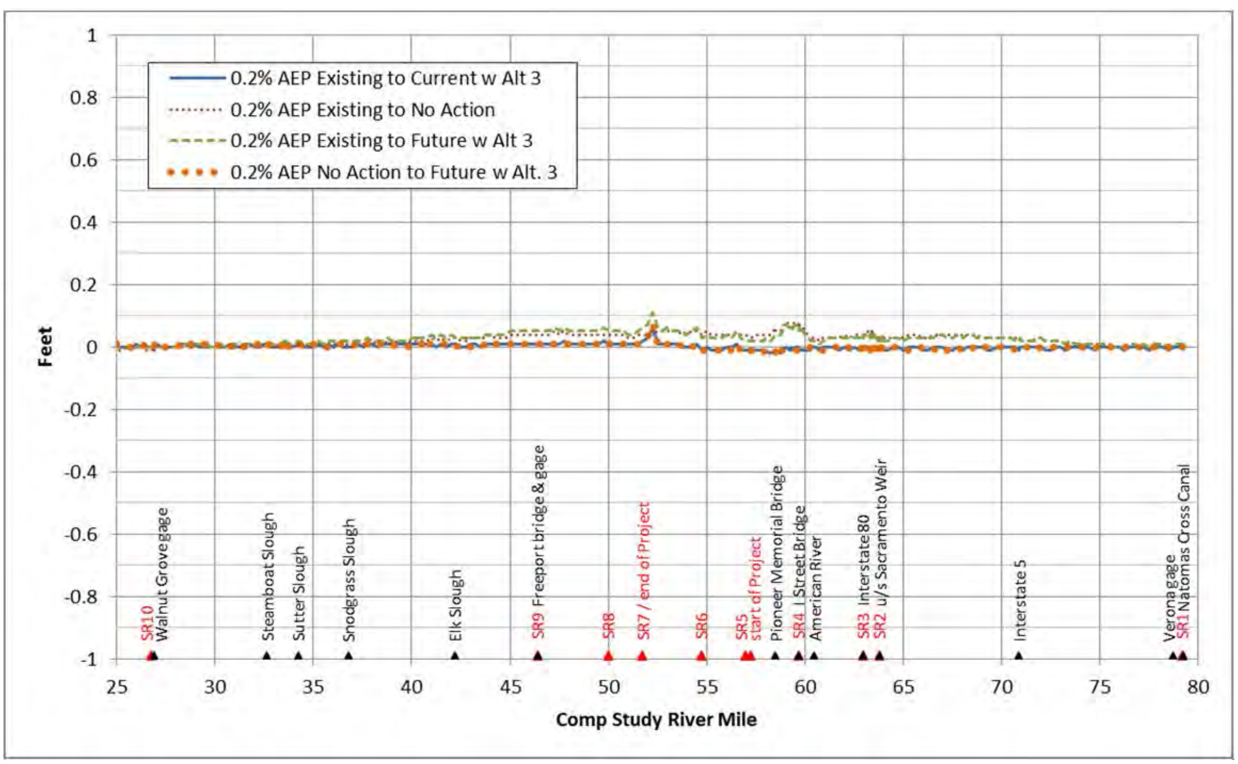
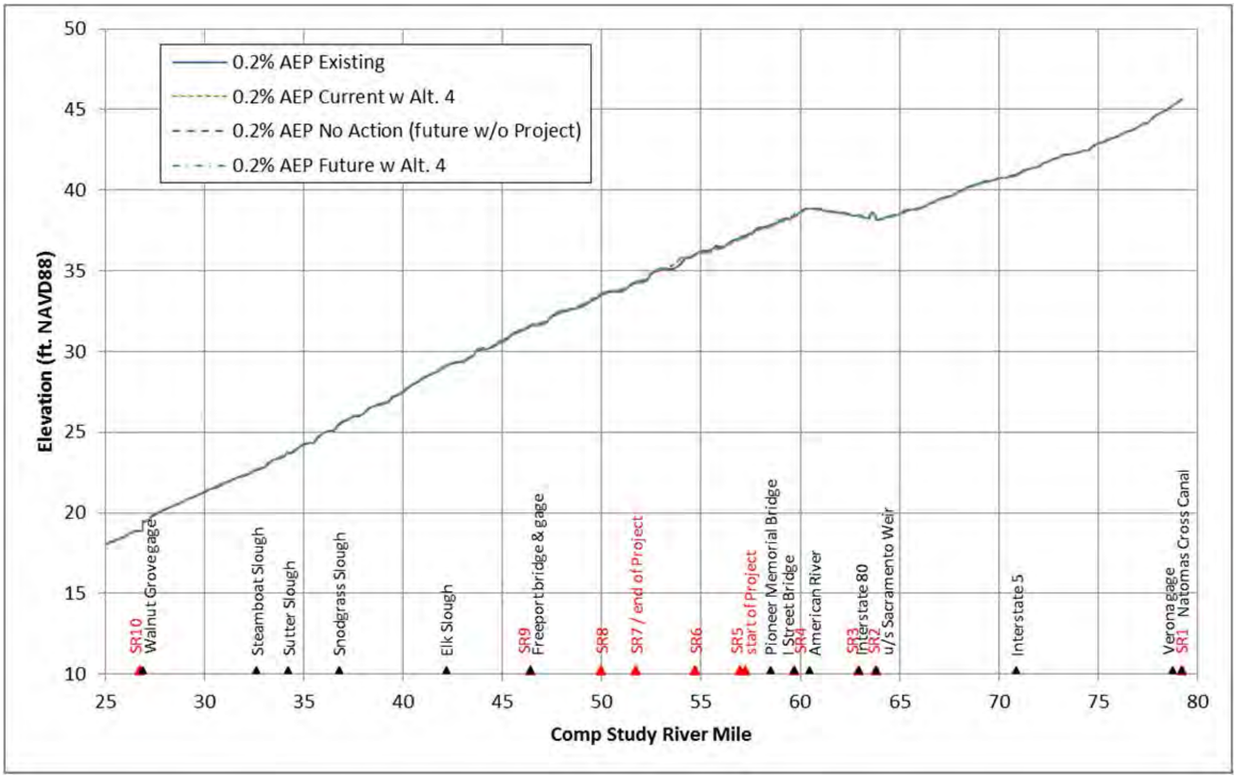


Figure 36. Maximum Water Surface Elevation Impacts; Sacramento River; Alternative 3; 0.2% AEP



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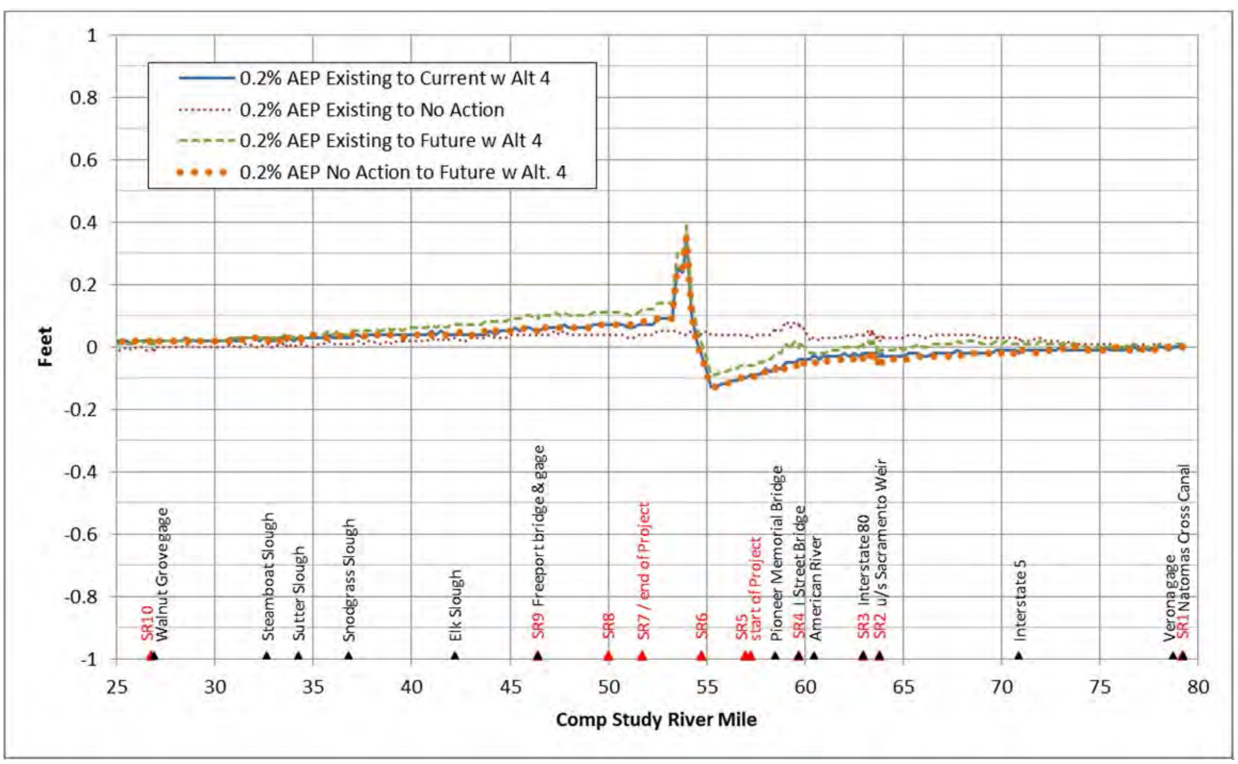
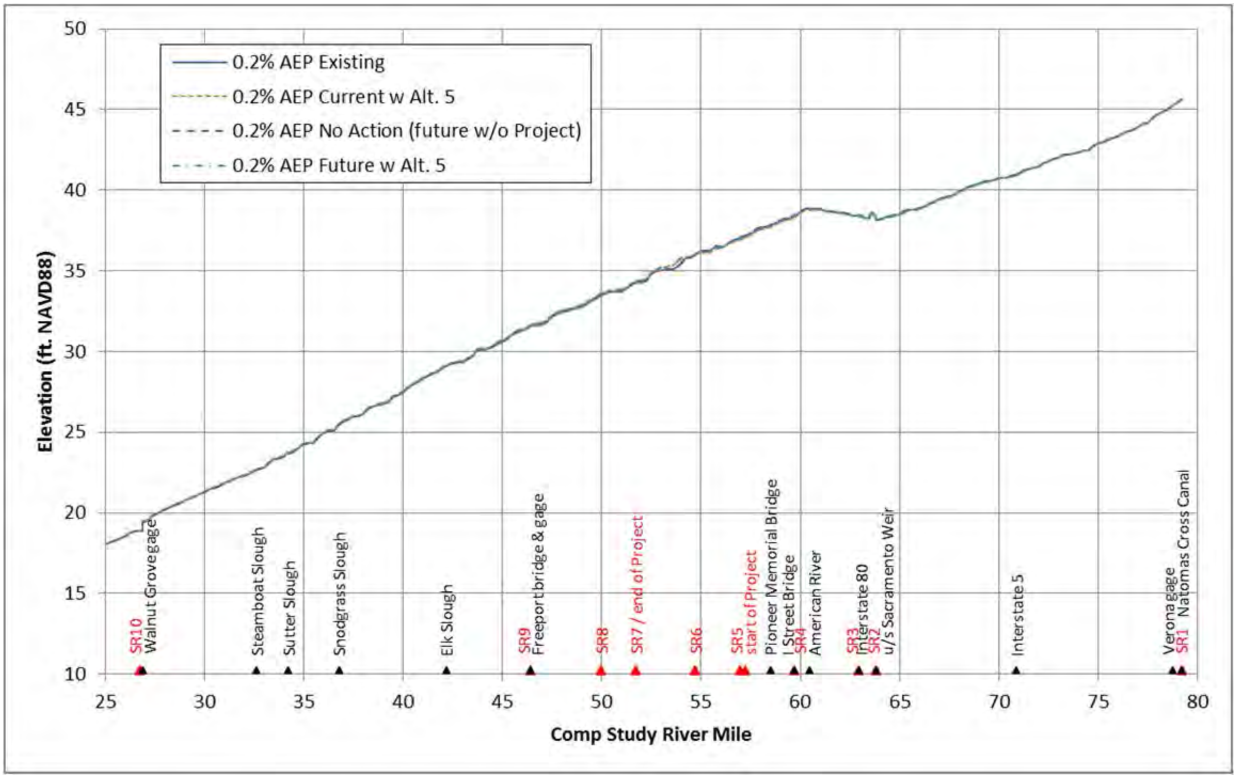


Figure 38. Maximum Water Surface Elevation Impacts; Sacramento River; Alternative 4; 0.2% AEP



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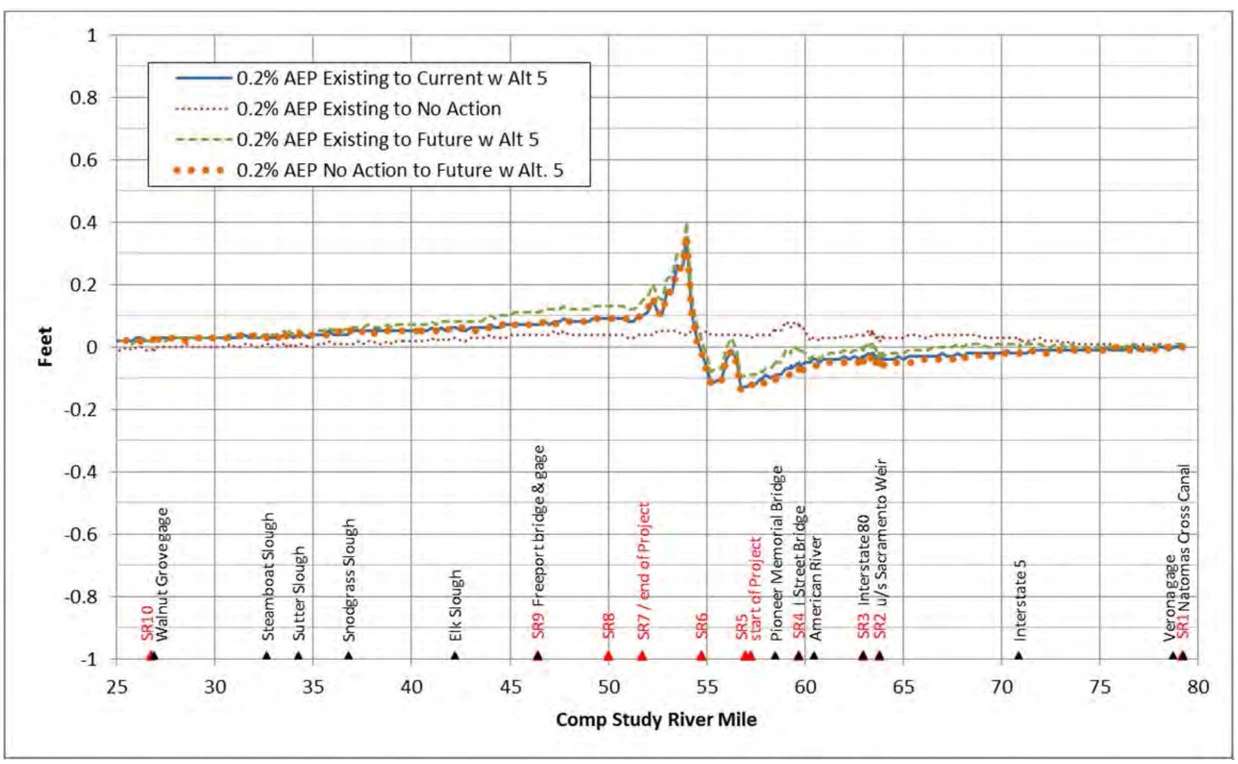
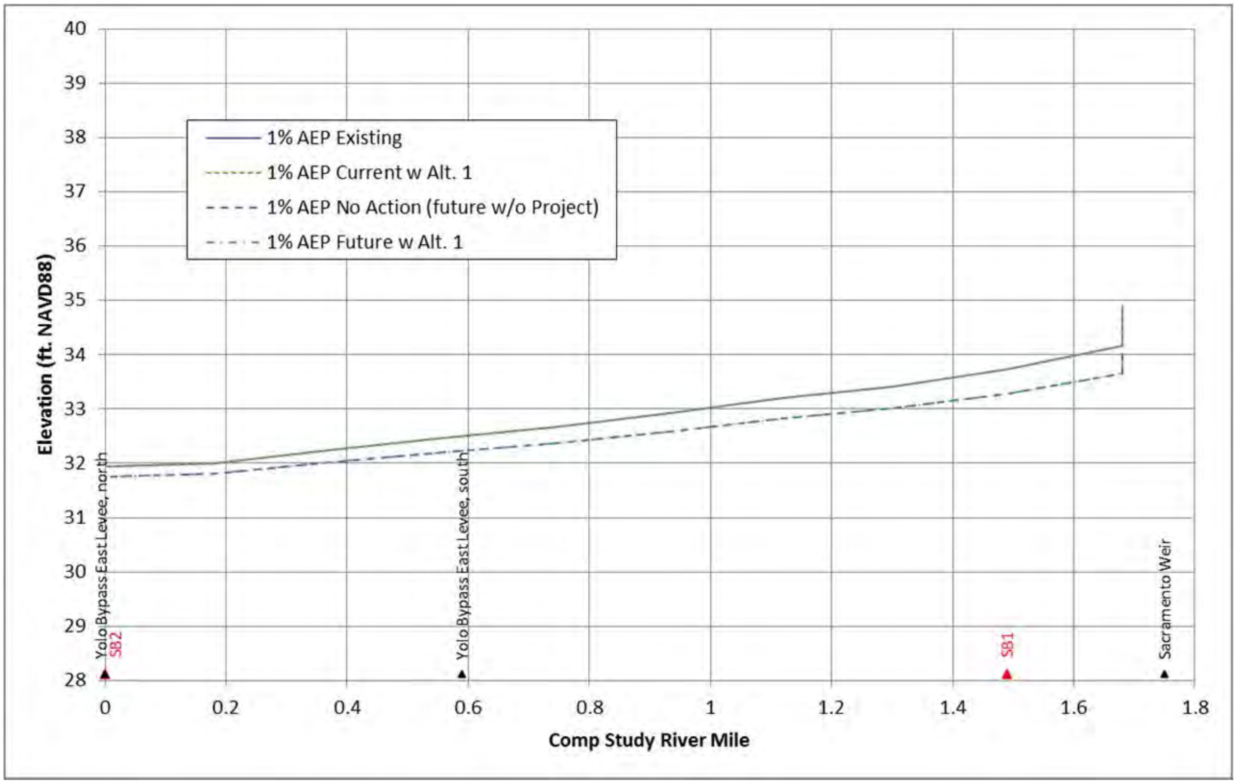


Figure 40. Maximum Water Surface Elevation Impacts; Sacramento River; Alternative 5; 0.2% AEP



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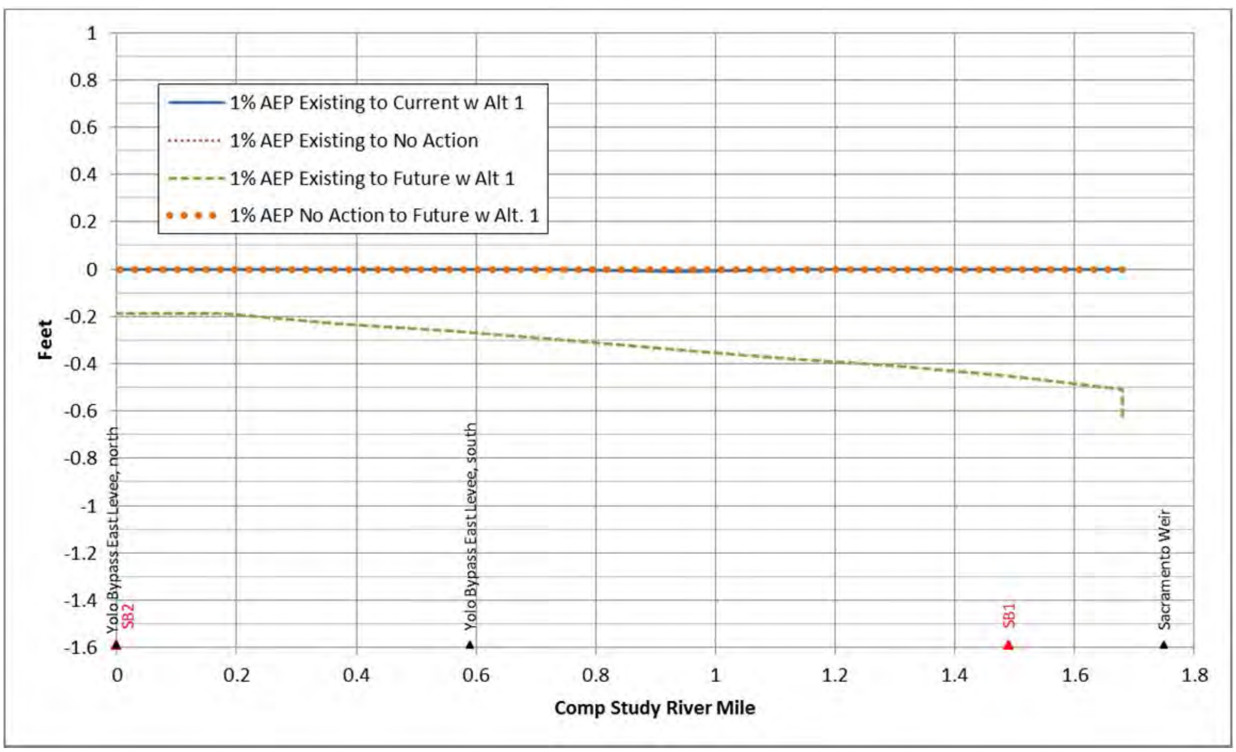

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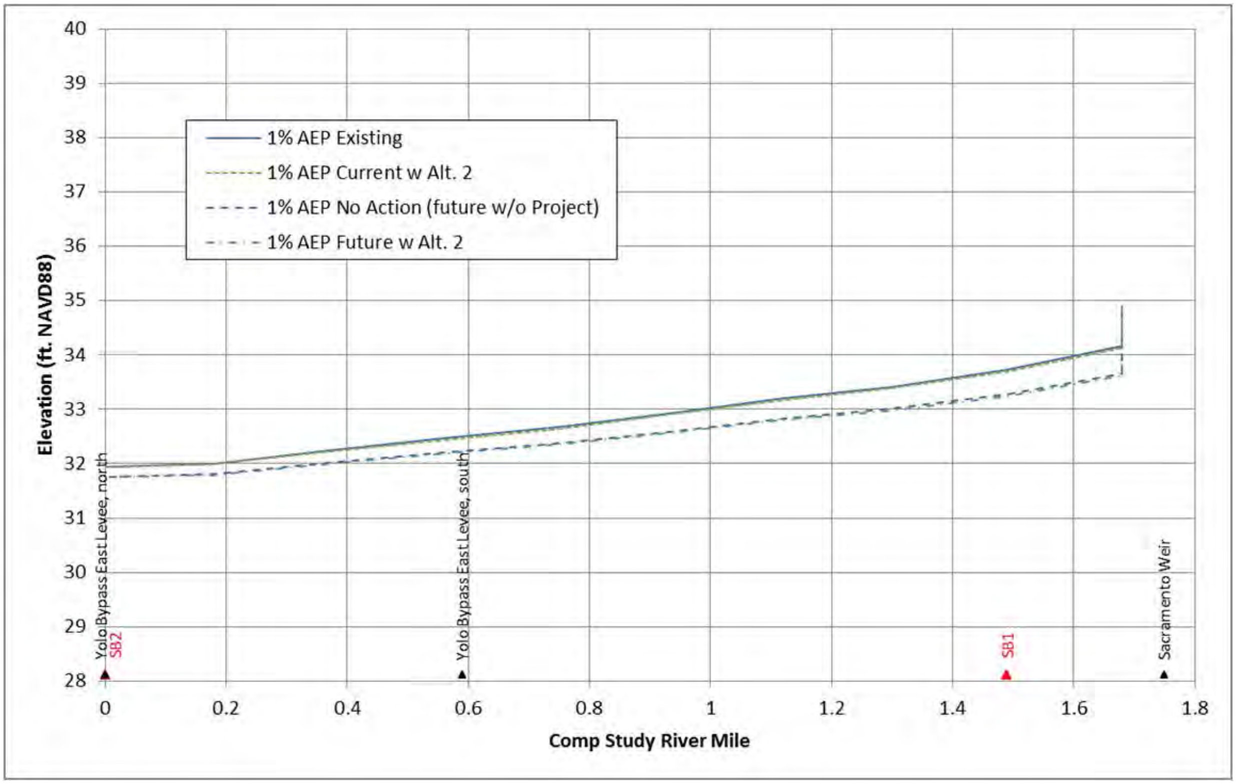

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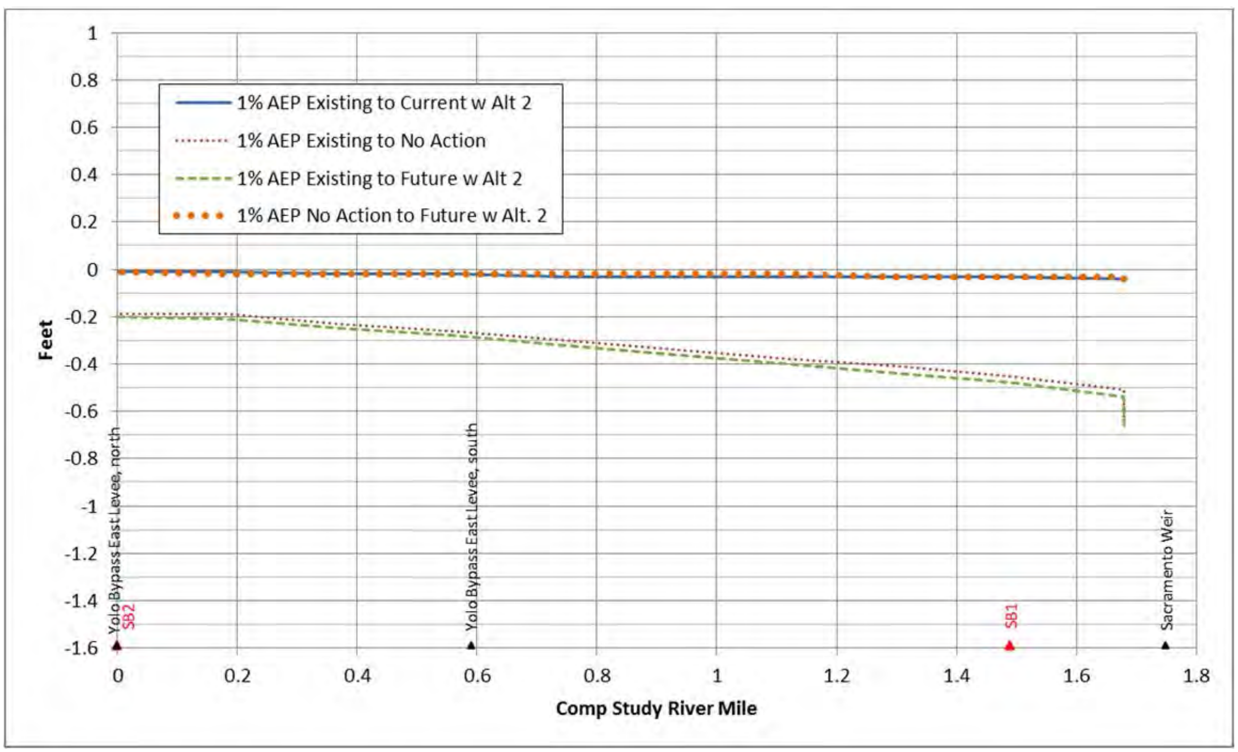

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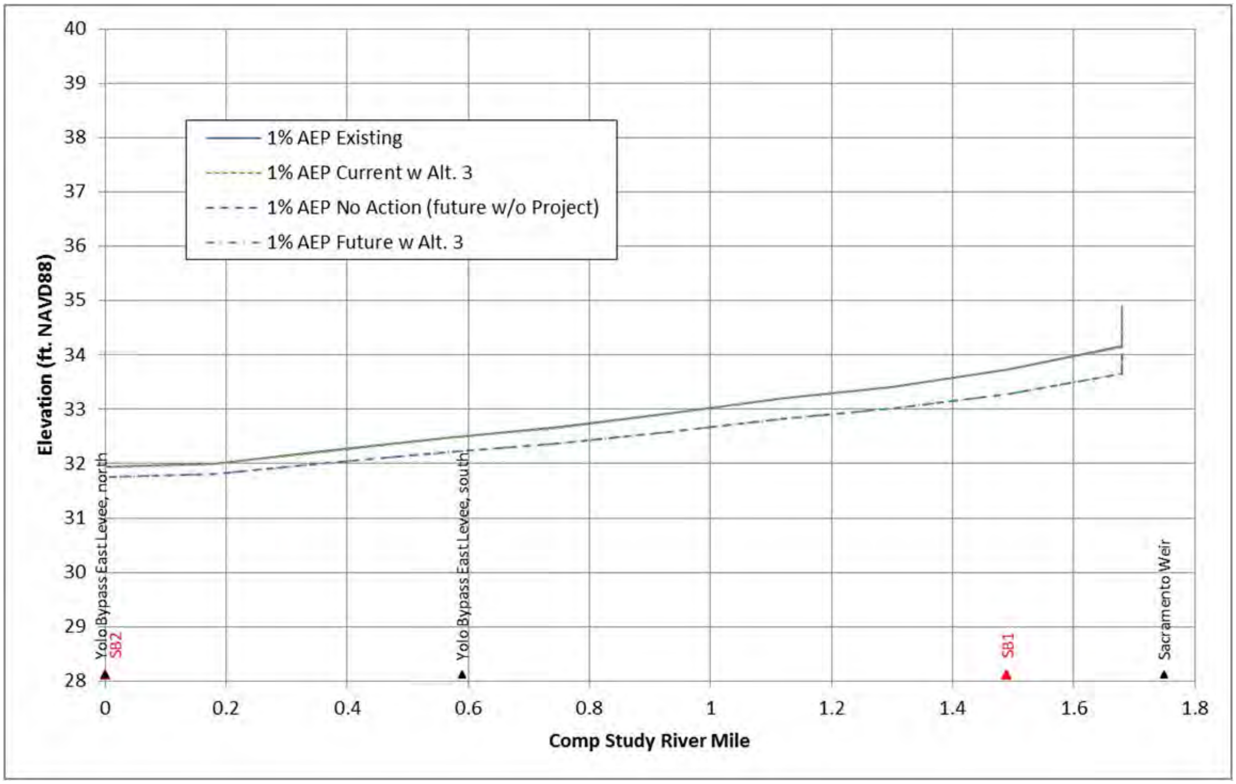

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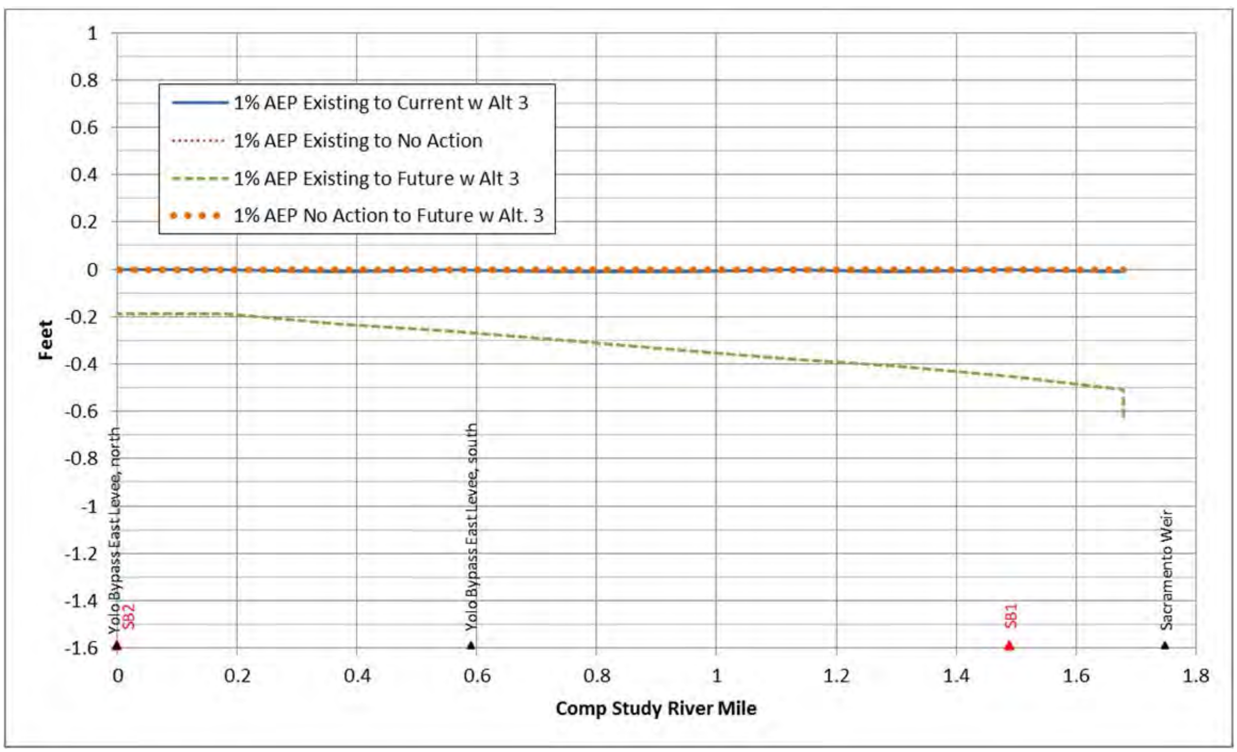

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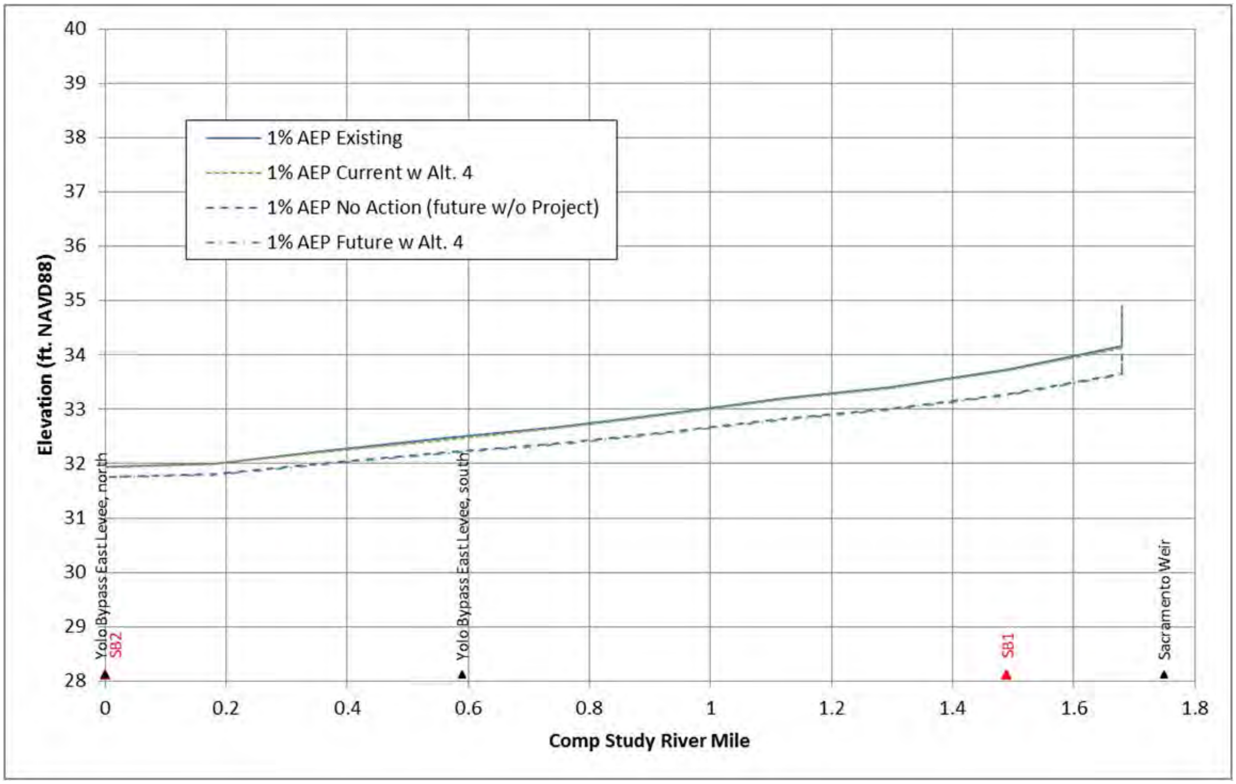

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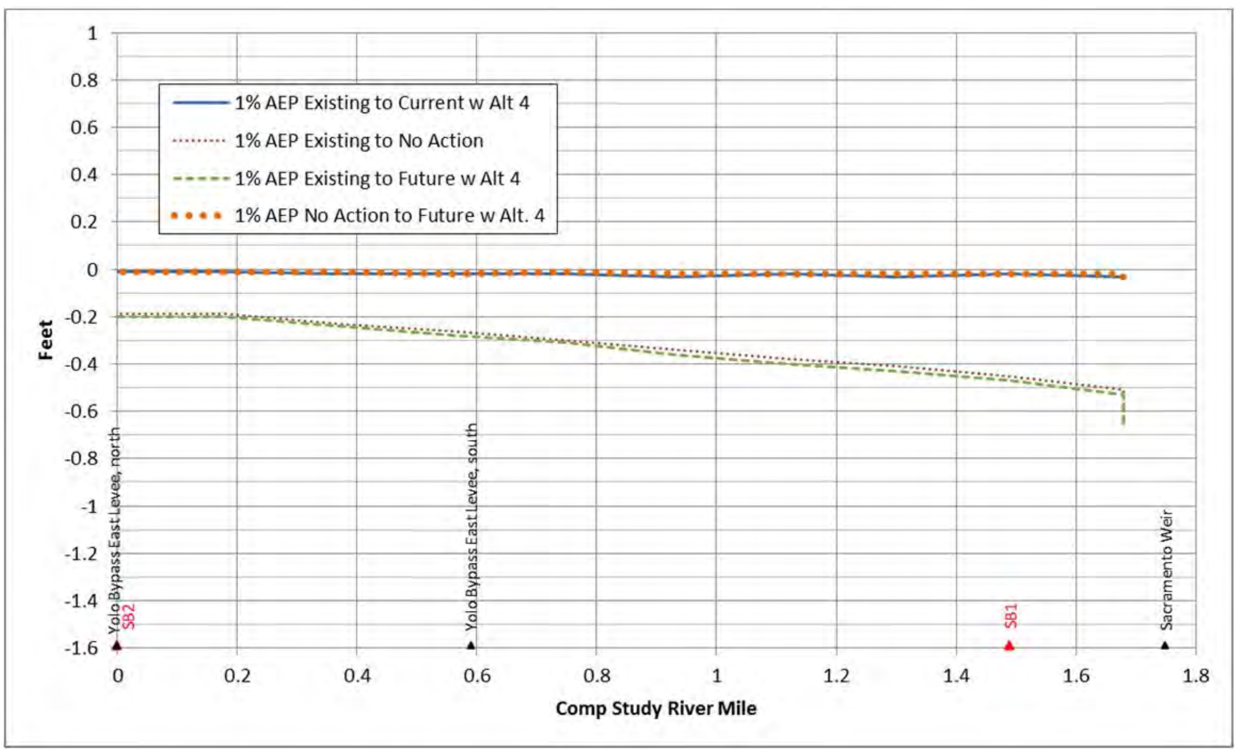

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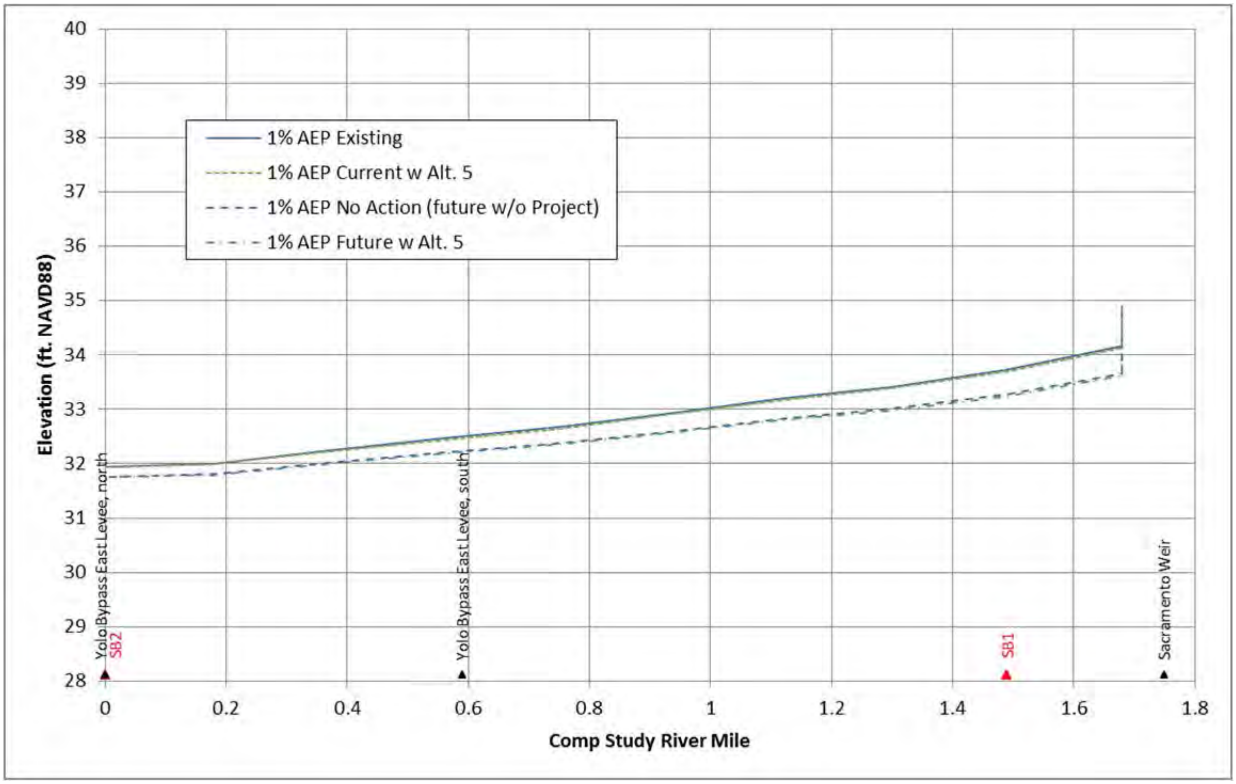

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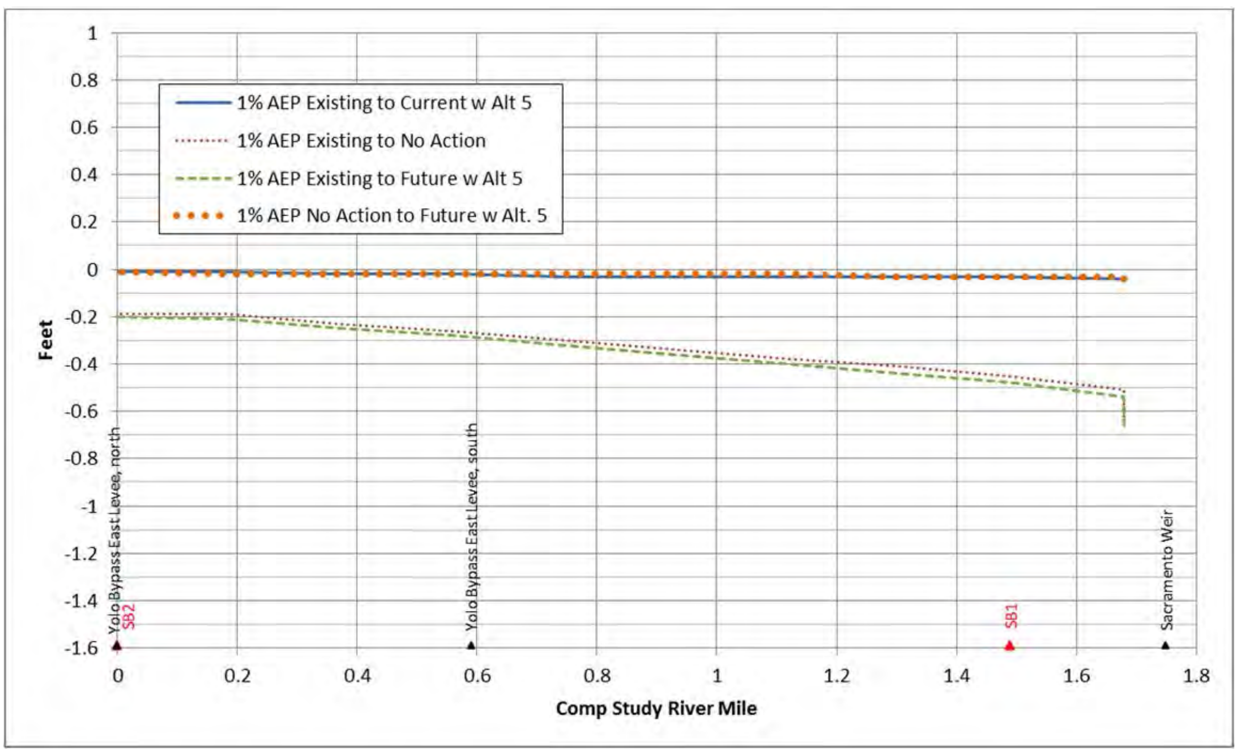

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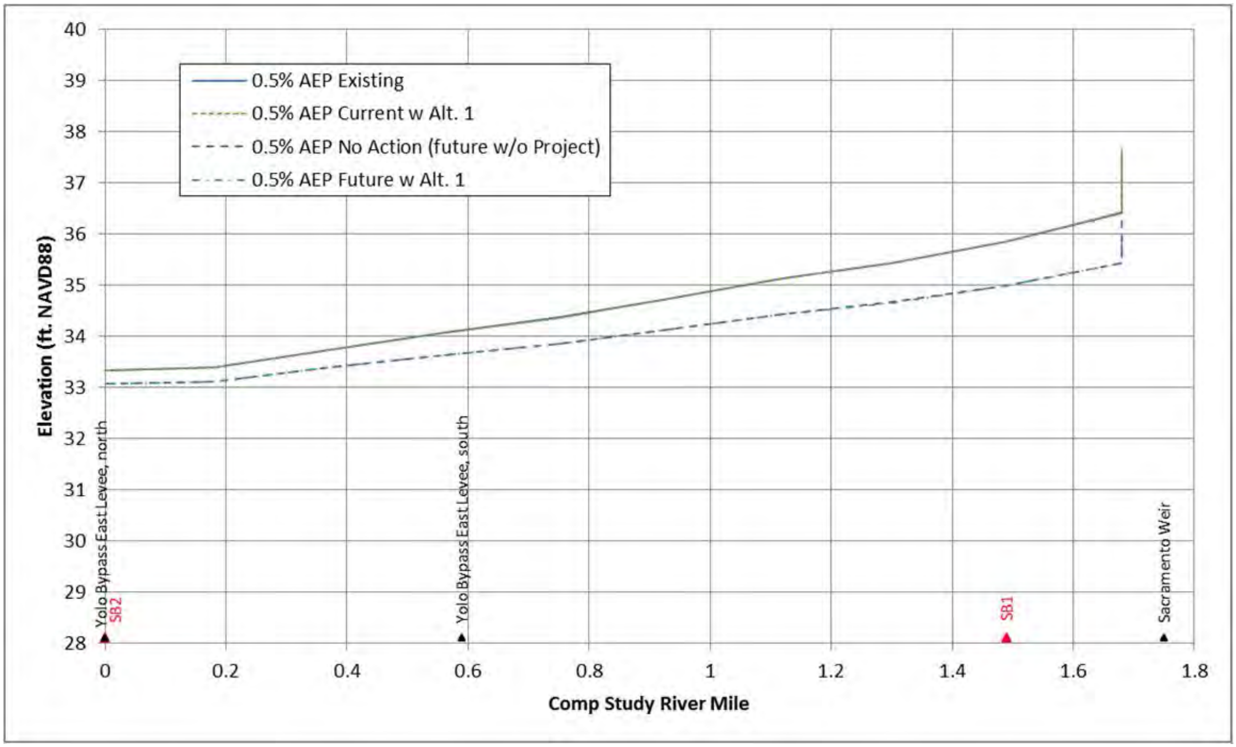

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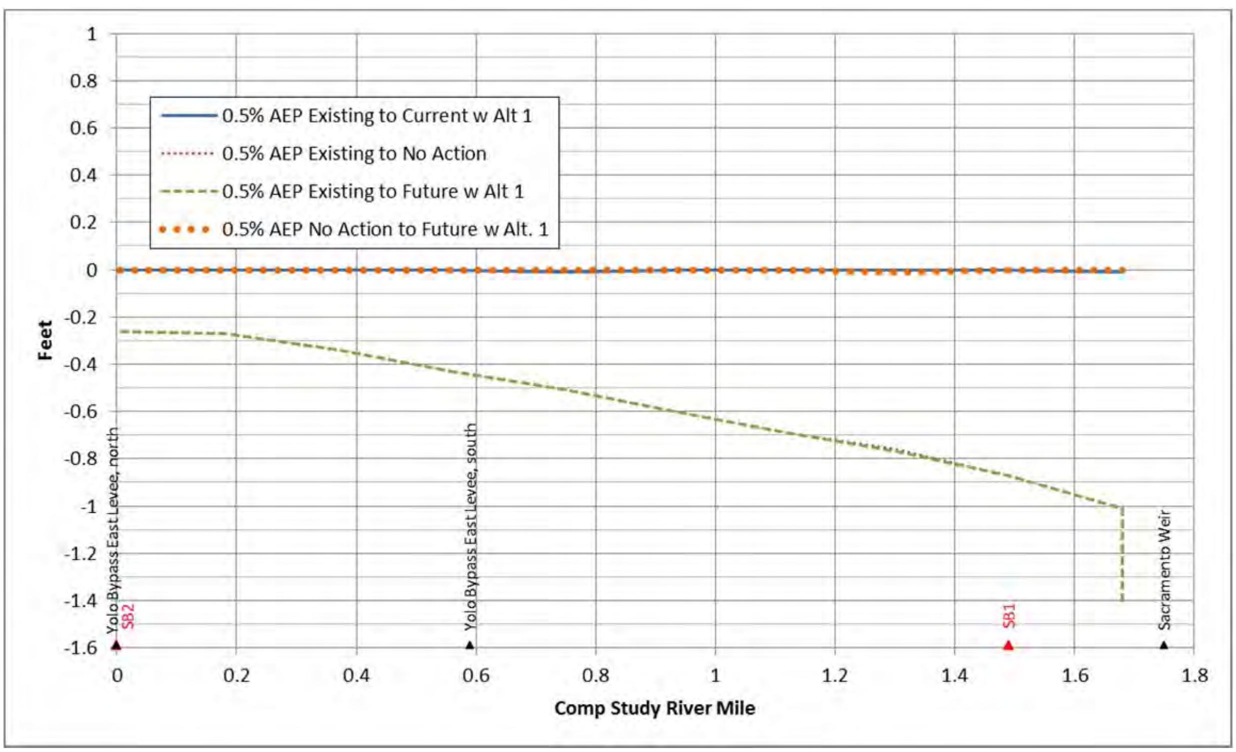
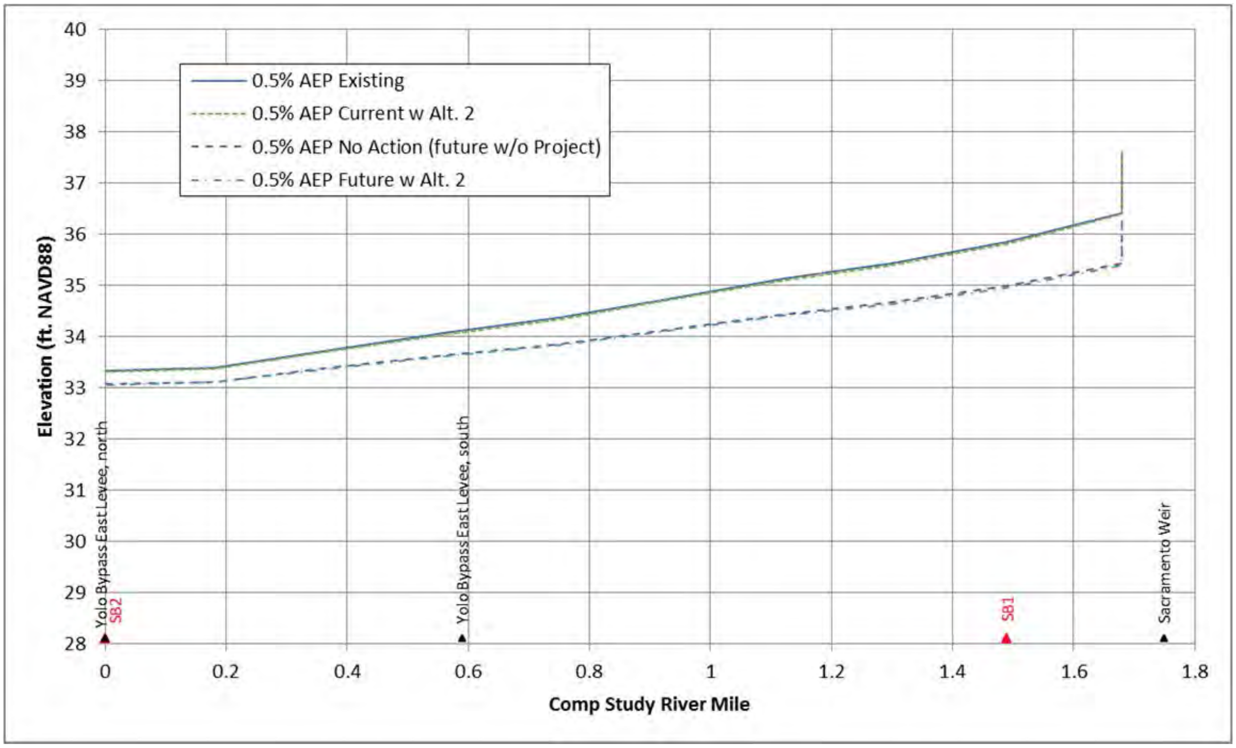


Figure 52. Maximum Water Surface Elevation Impacts; Sacramento Bypass; Alternative 1; 0.5% AEP



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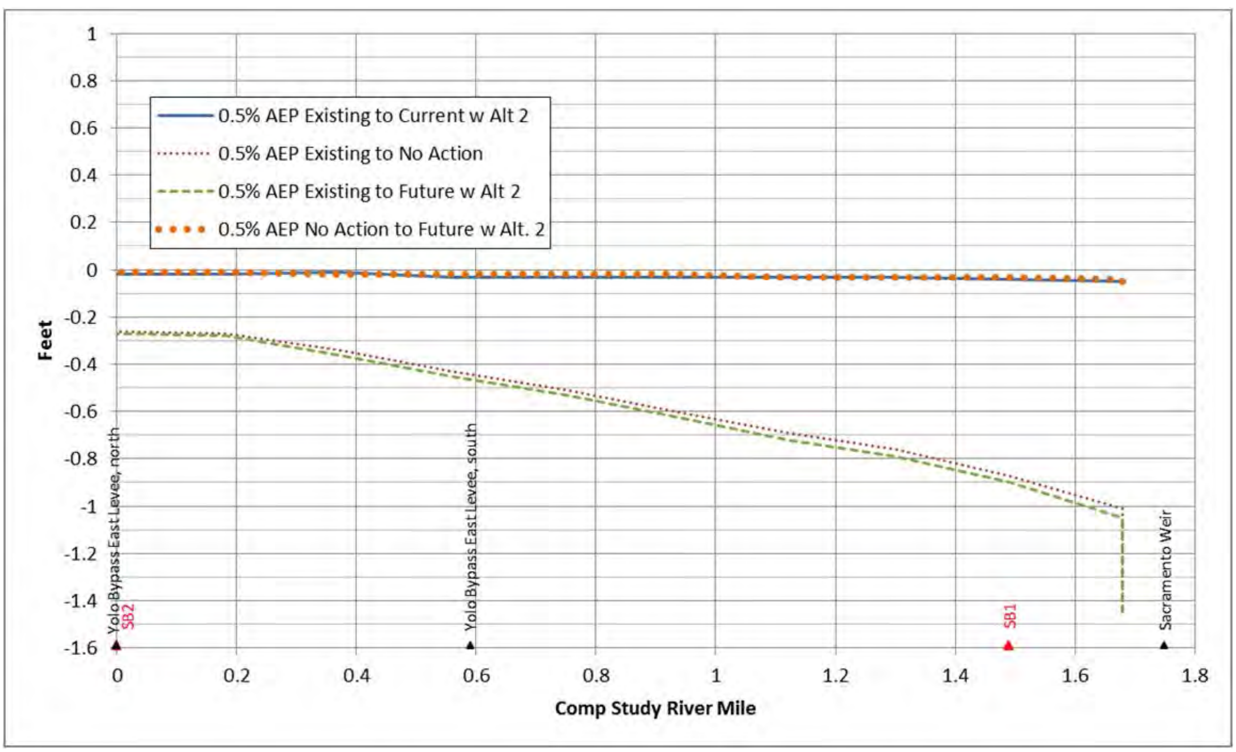

Figure 54. Maximum Water Surface Elevation Impacts; Sacramento Bypass; Alternative 2; 0.5% AEP

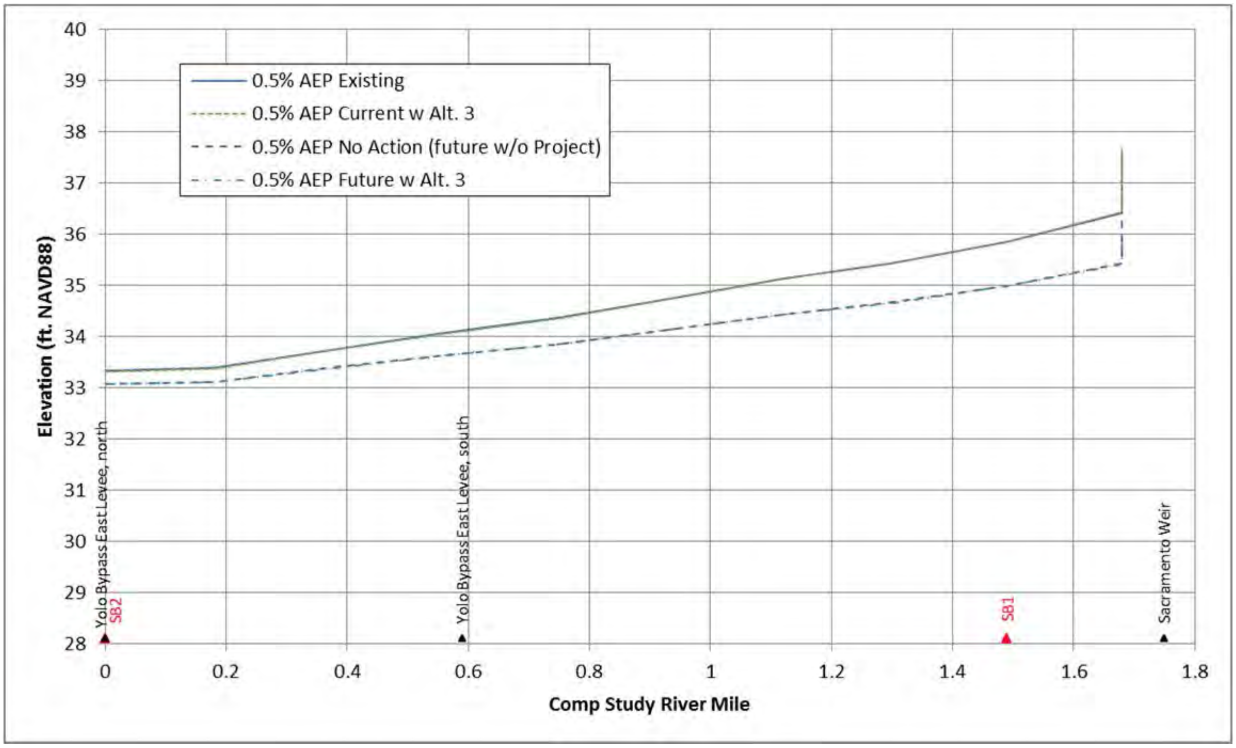

Figure 55. Maximum Water Surface Elevations; Sacramento Bypass; Alternative 3; 0.5% AEP

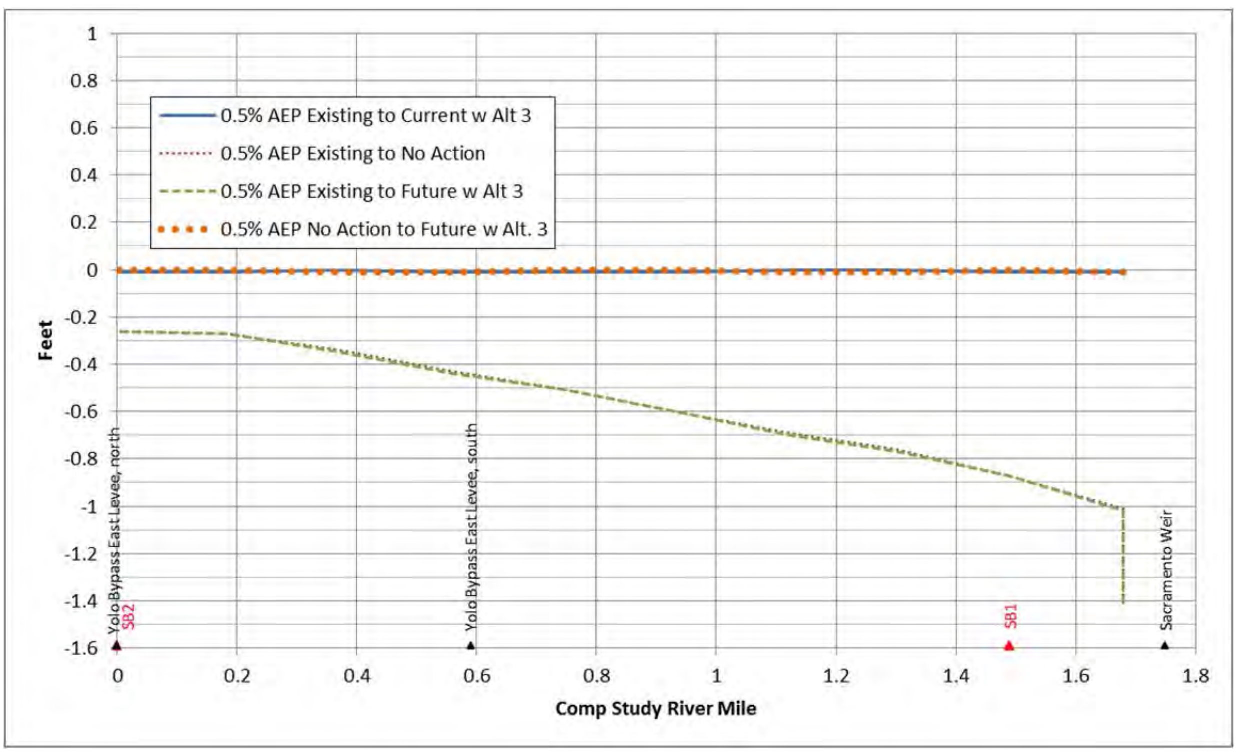

Figure 56. Maximum Water Surface Elevation Impacts; Sacramento Bypass; Alternative 3; 0.5% AEP

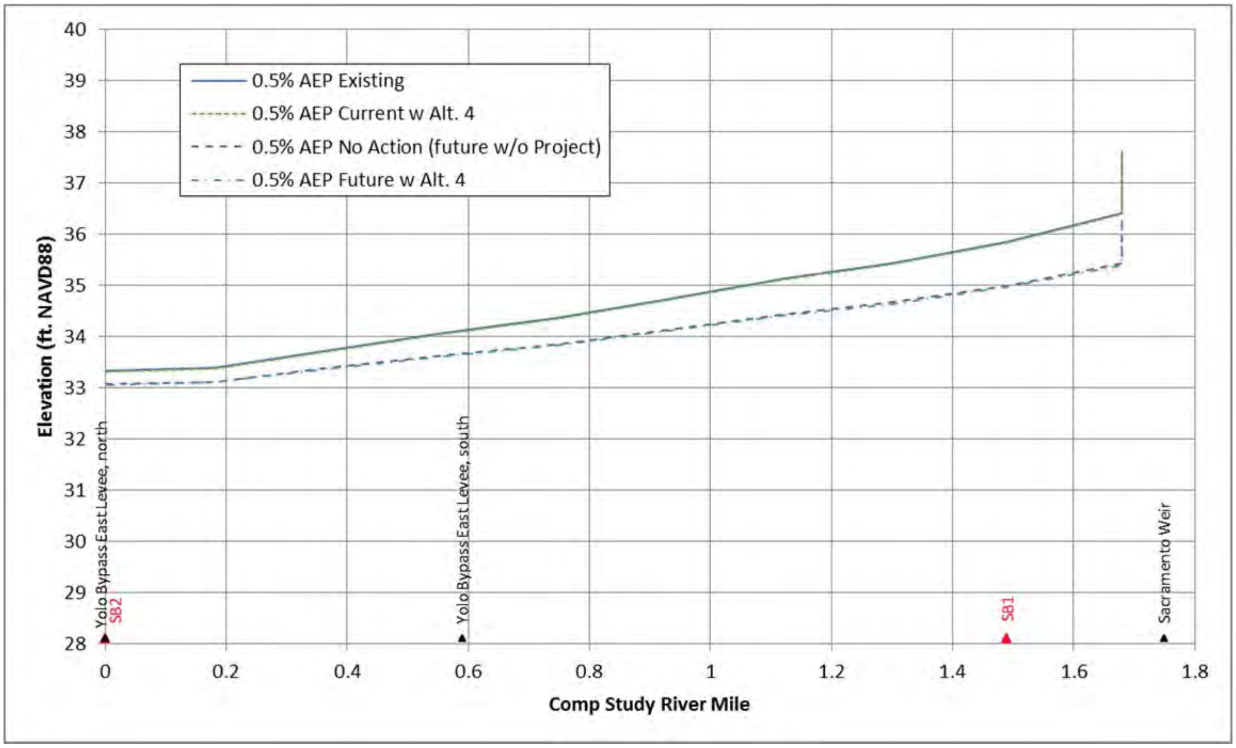

Figure 57. Maximum Water Surface Elevations; Sacramento Bypass; Alternative 4; 0.5% AEP

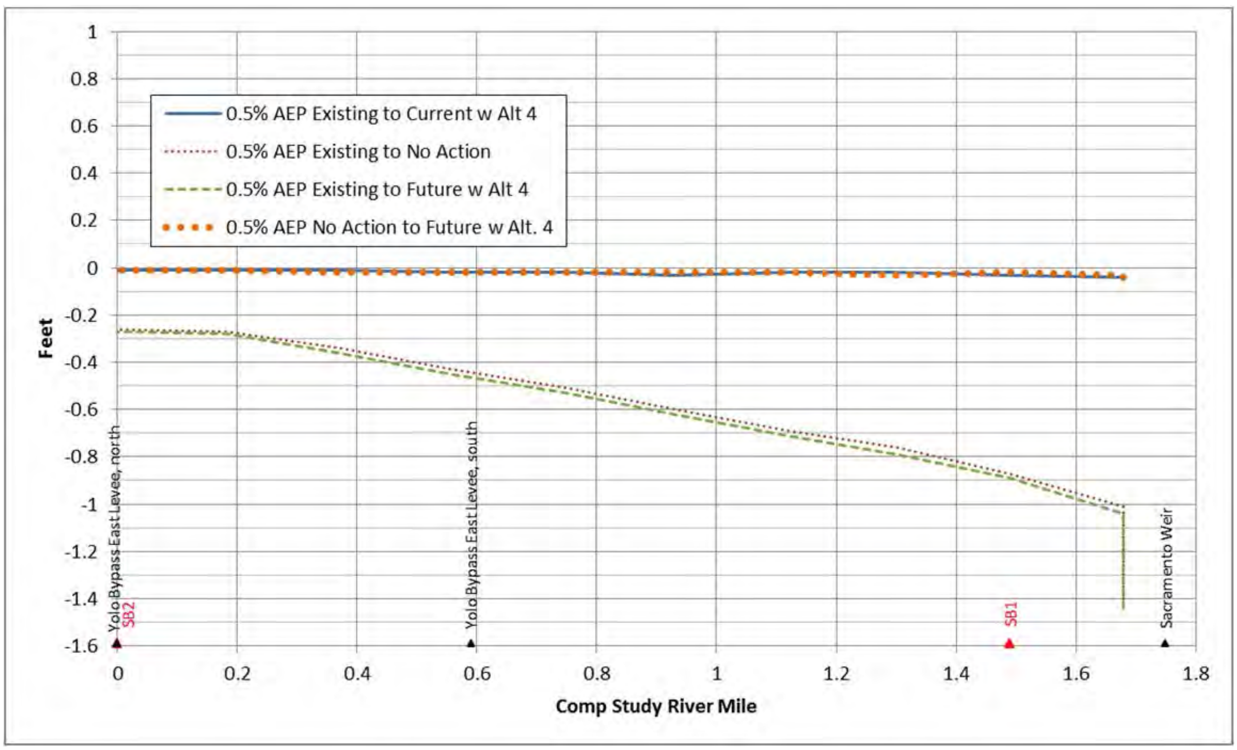

Figure 58. Maximum Water Surface Elevation Impacts; Sacramento Bypass; Alternative 4; 0.5% AEP

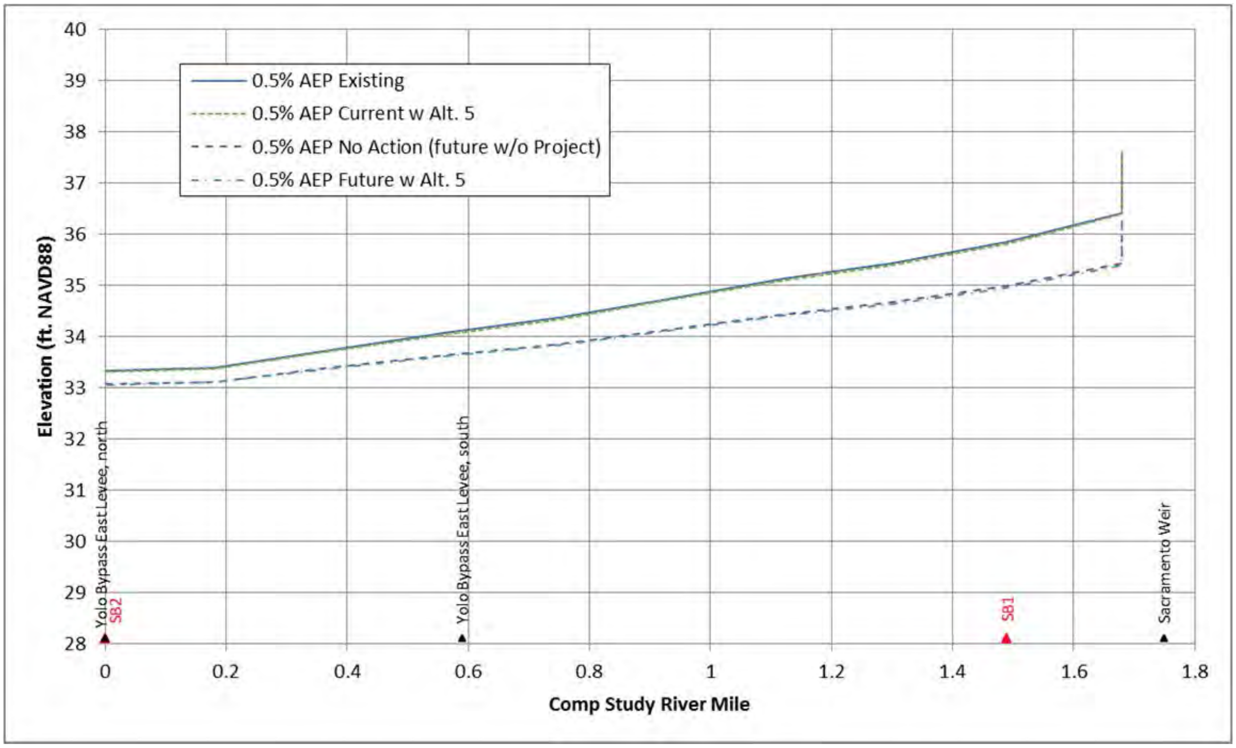

Figure 59. Maximum Water Surface Elevations; Sacramento Bypass; Alternative 5; 0.5% AEP

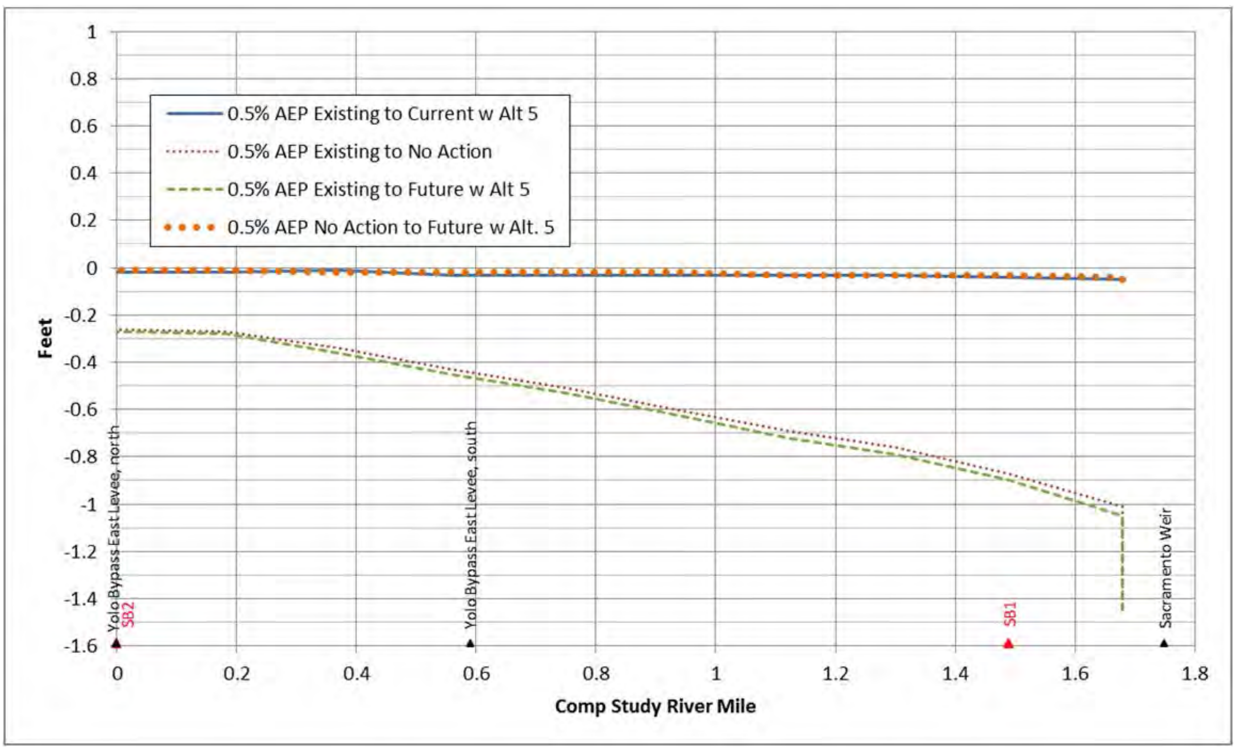

Figure 60. Maximum Water Surface Elevation Impacts; Sacramento Bypass; Alternative 5; 0.5% AEP

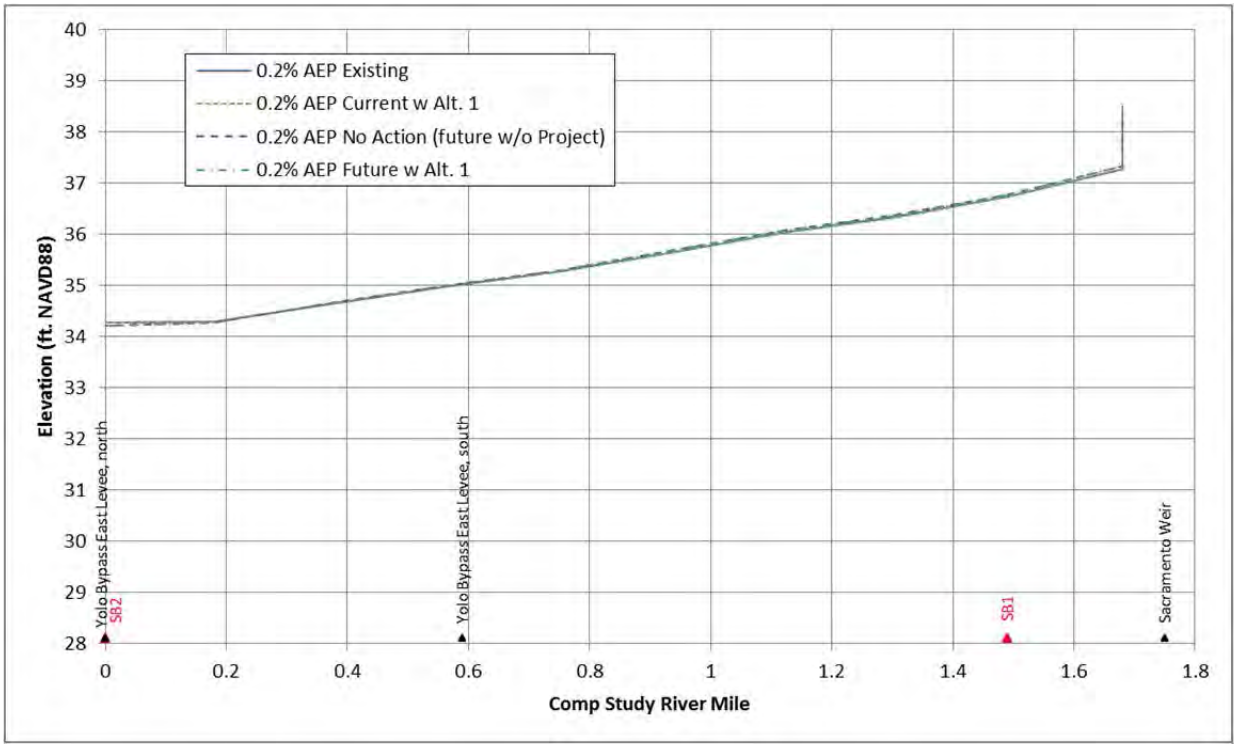

Figure 61. Maximum Water Surface Elevations; Sacramento Bypass; Alternative 1; 0.2% AEP

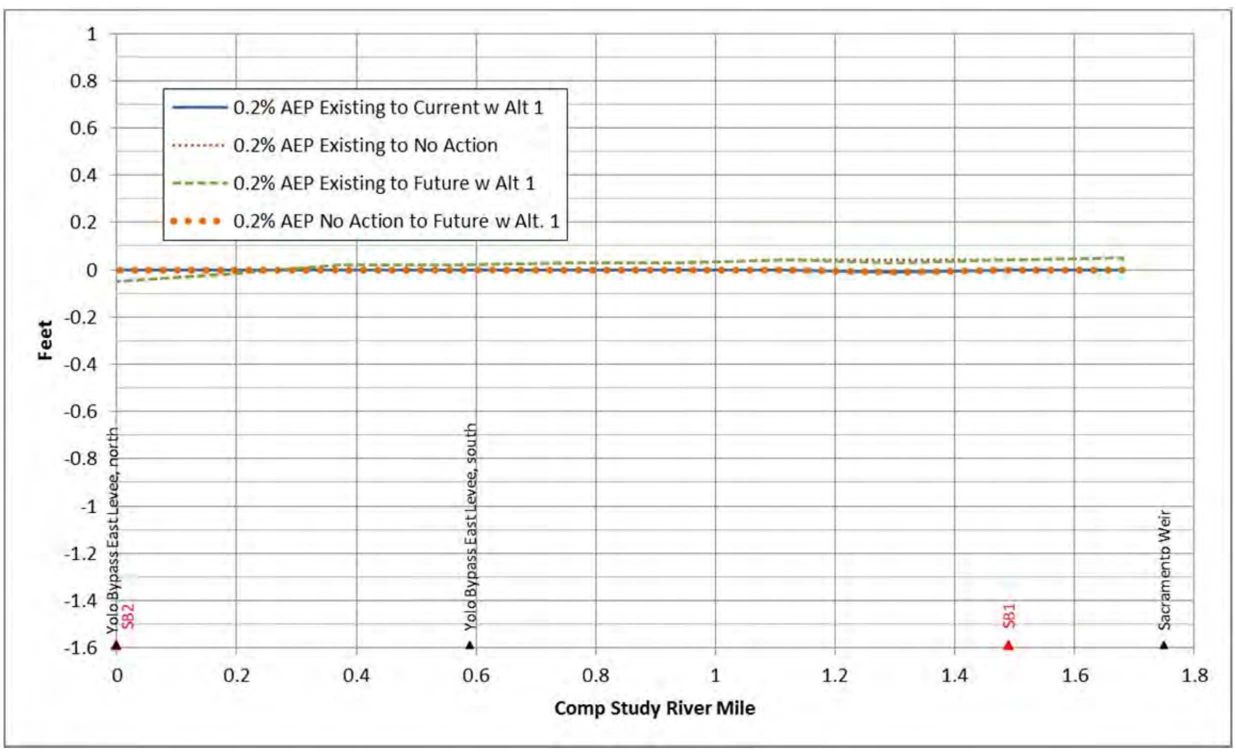

Figure 62. Maximum Water Surface Elevation Impacts; Sacramento Bypass; Alternative 1; 0.2% AEP

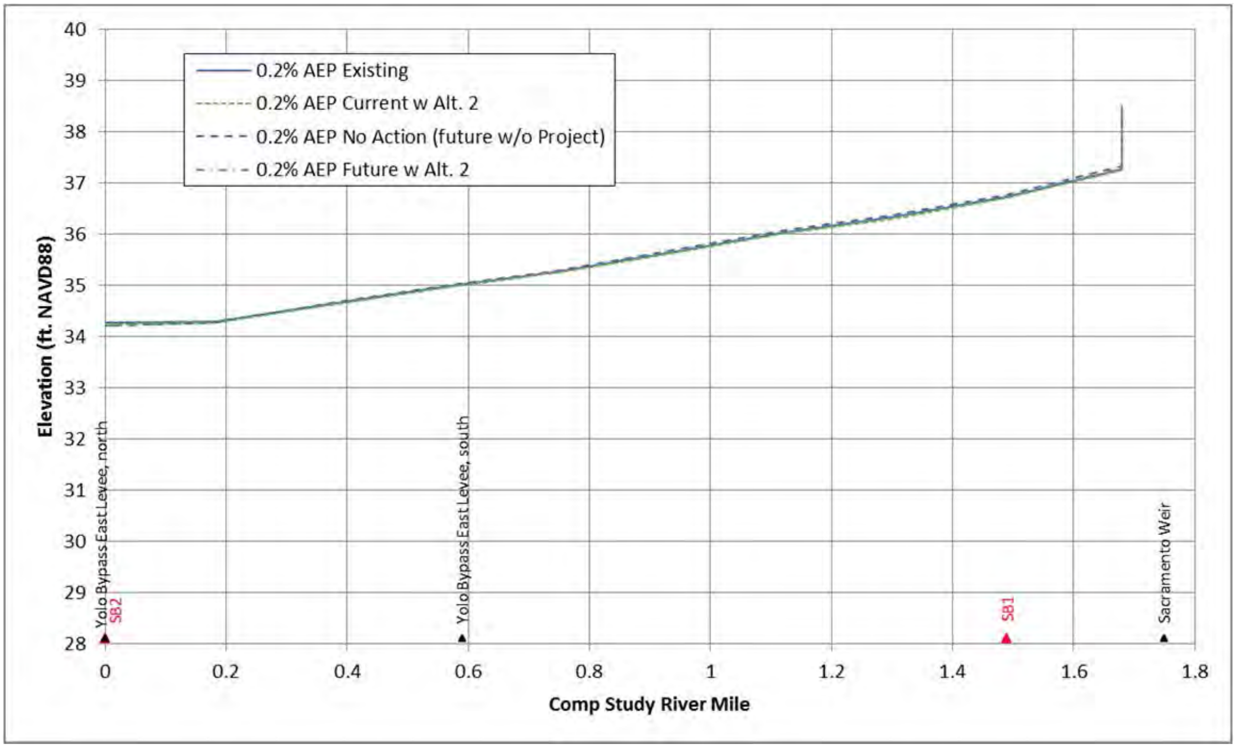

Figure 63. Maximum Water Surface Elevations; Sacramento Bypass; Alternative 2; 0.2% AEP

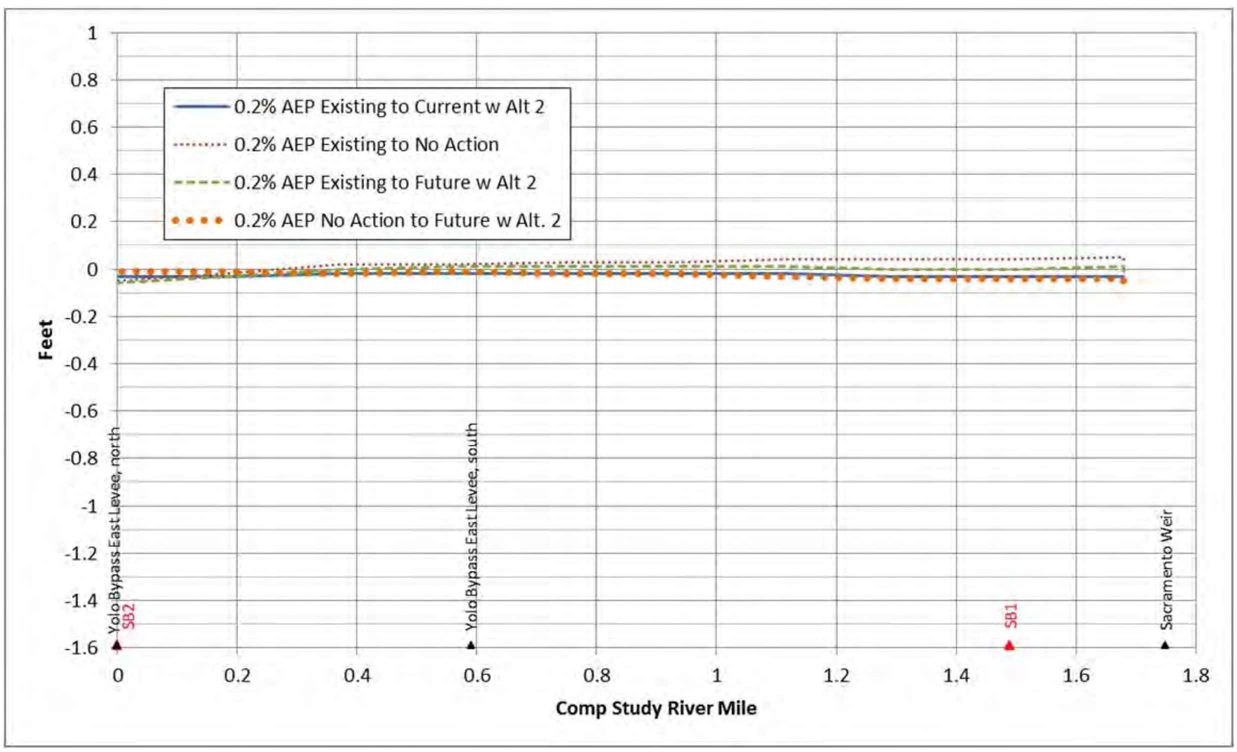

Figure 64. Maximum Water Surface Elevation Impacts; Sacramento Bypass; Alternative 2; 0.2% AEP

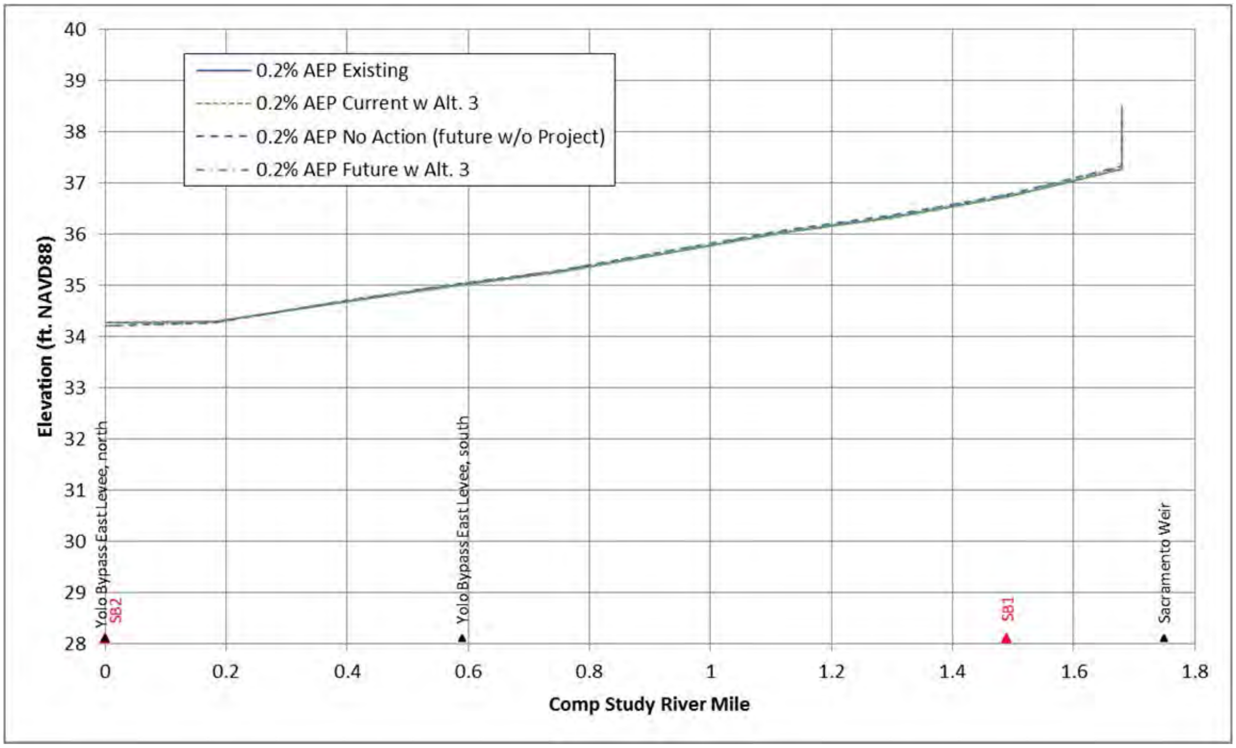

Figure 65. Maximum Water Surface Elevations; Sacramento Bypass; Alternative 3; 0.2% AEP

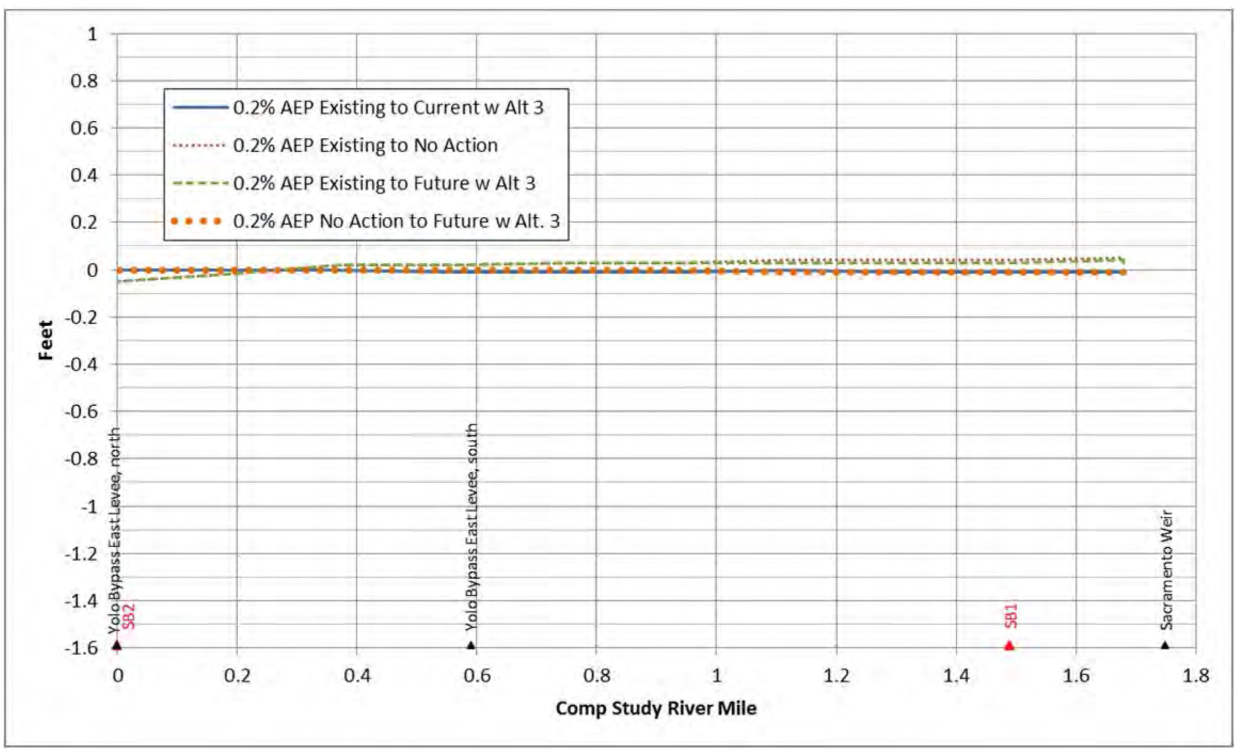

Figure 66. Maximum Water Surface Elevation Impacts; Sacramento Bypass; Alternative 3; 0.2% AEP

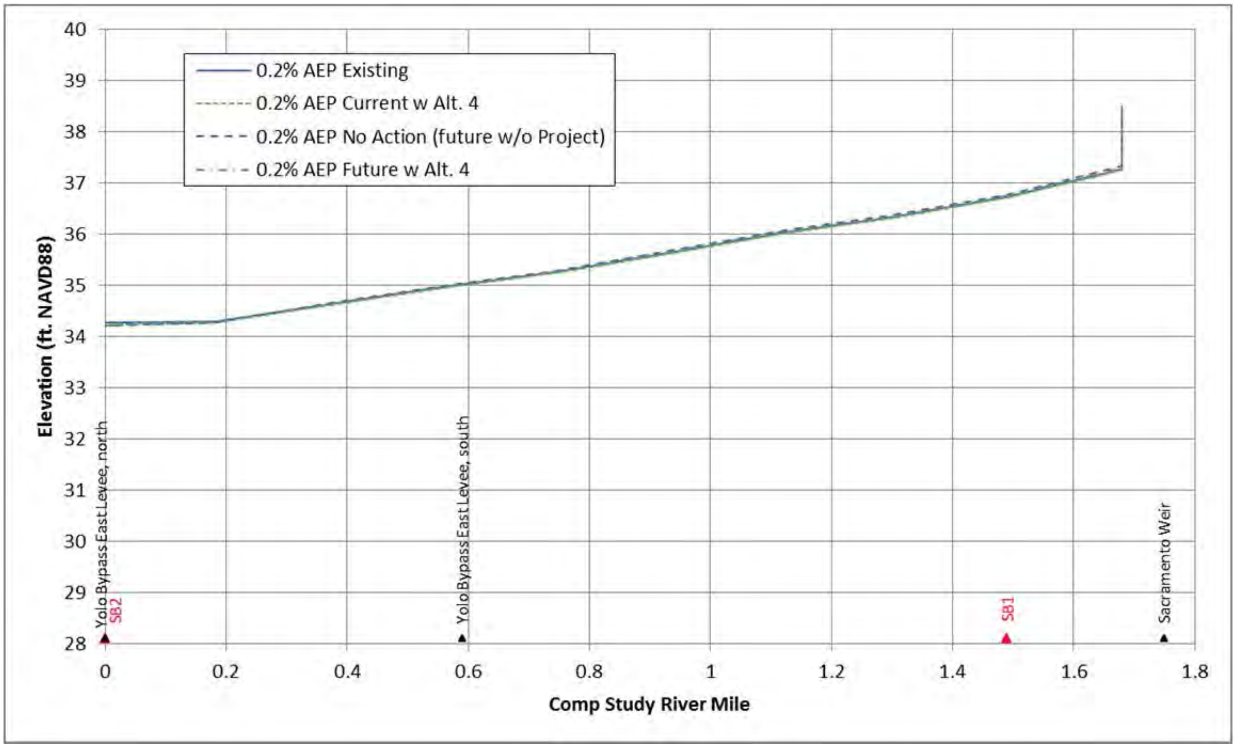

Figure 67. Maximum Water Surface Elevations; Sacramento Bypass; Alternative 4; 0.2% AEP

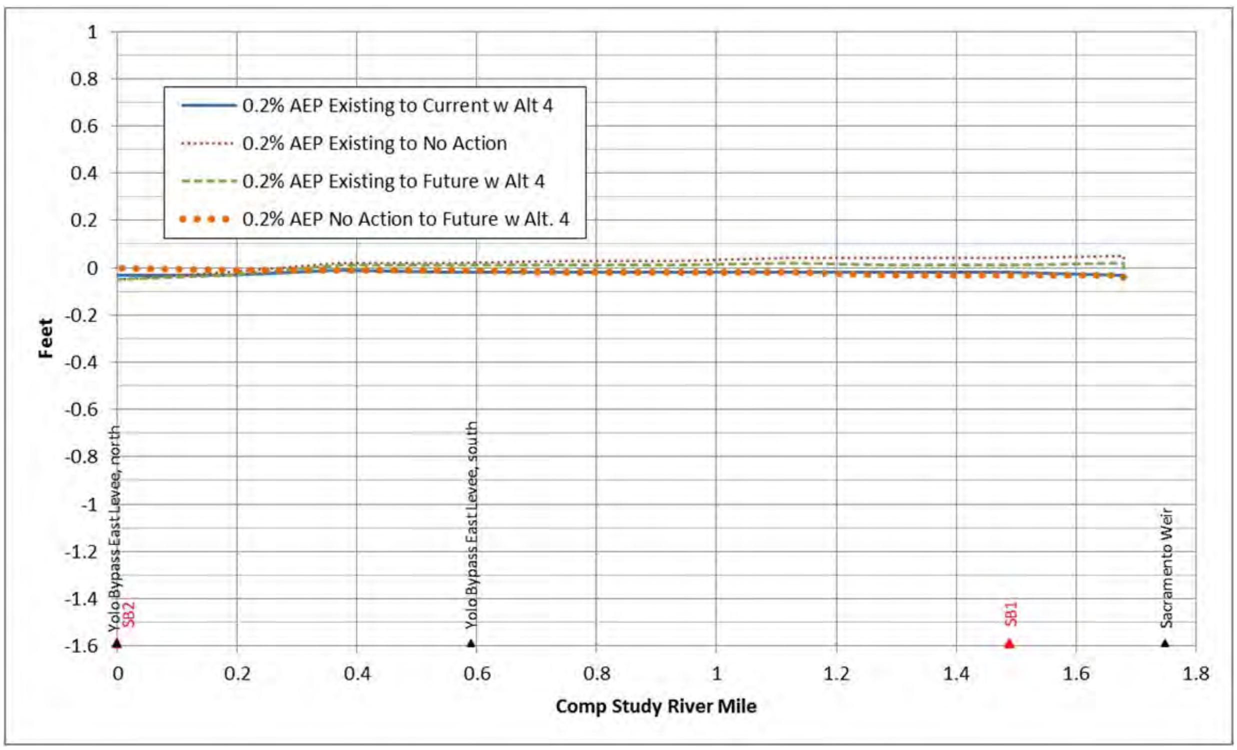

Figure 68. Maximum Water Surface Elevation Impacts; Sacramento Bypass; Alternative 4; 0.2% AEP

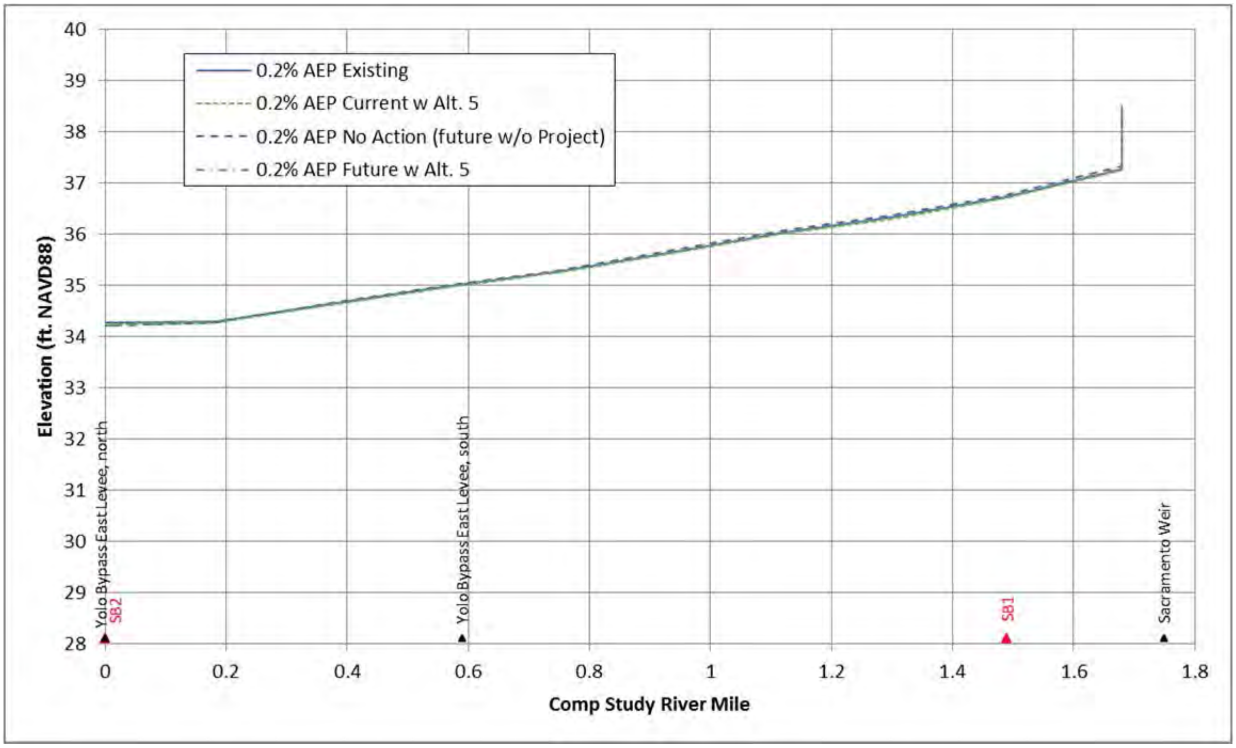

Figure 69. Maximum Water Surface Elevations; Sacramento Bypass; Alternative 5; 0.2% AEP

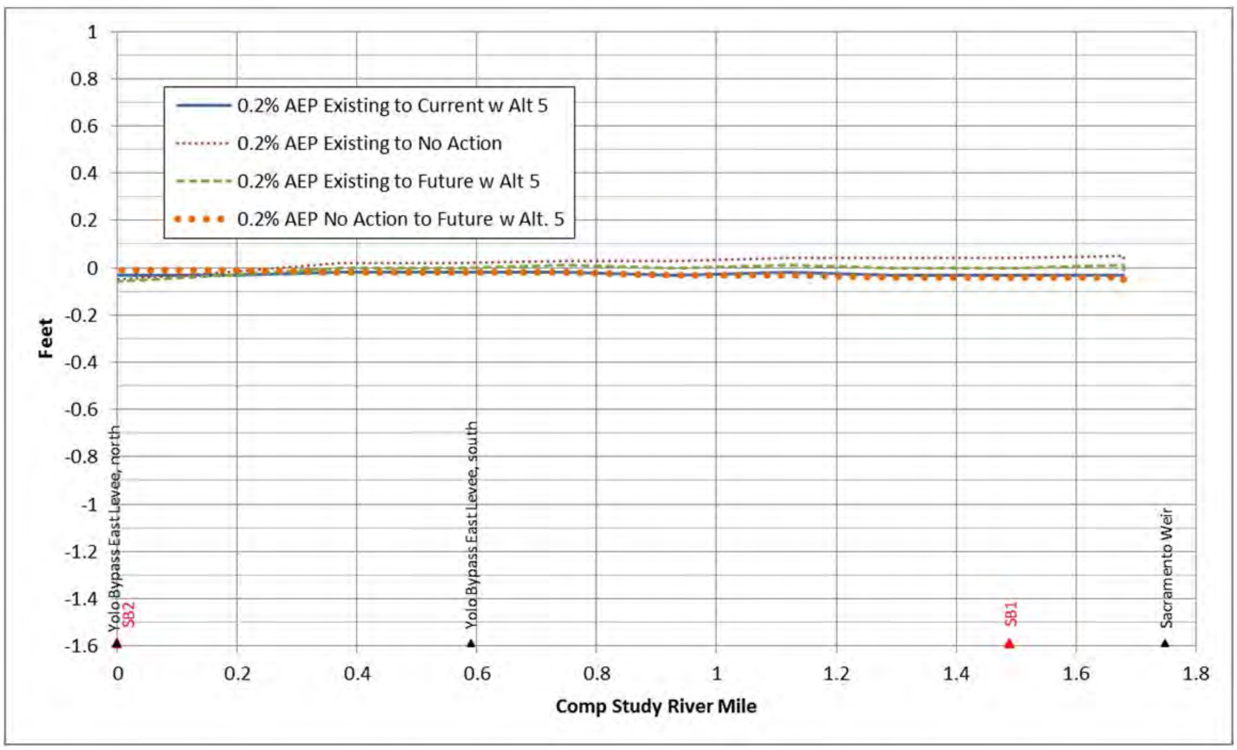

Figure 70. Maximum Water Surface Elevation Impacts; Sacramento Bypass; Alternative 5; 0.2% AEP

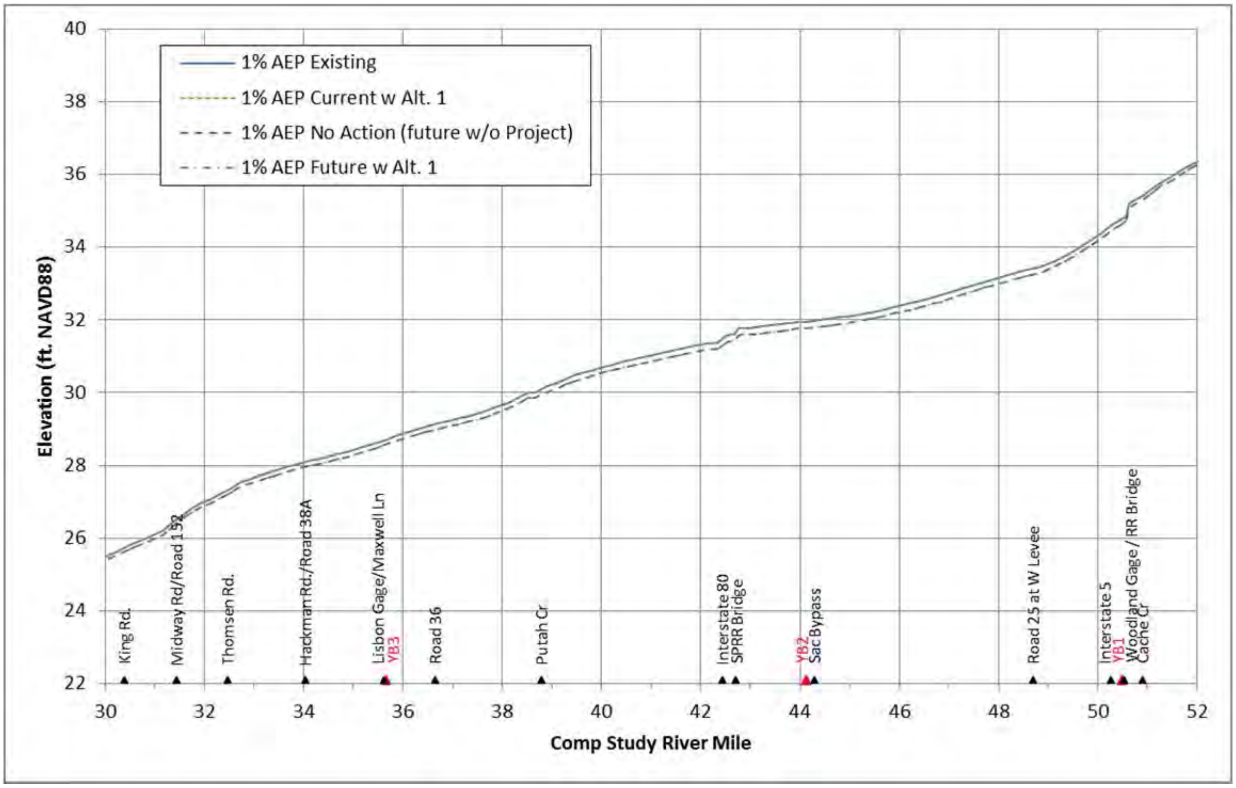

Figure 71. Maximum Water Surface Elevations; Yolo Bypass; Alternative 1; 1% AEP

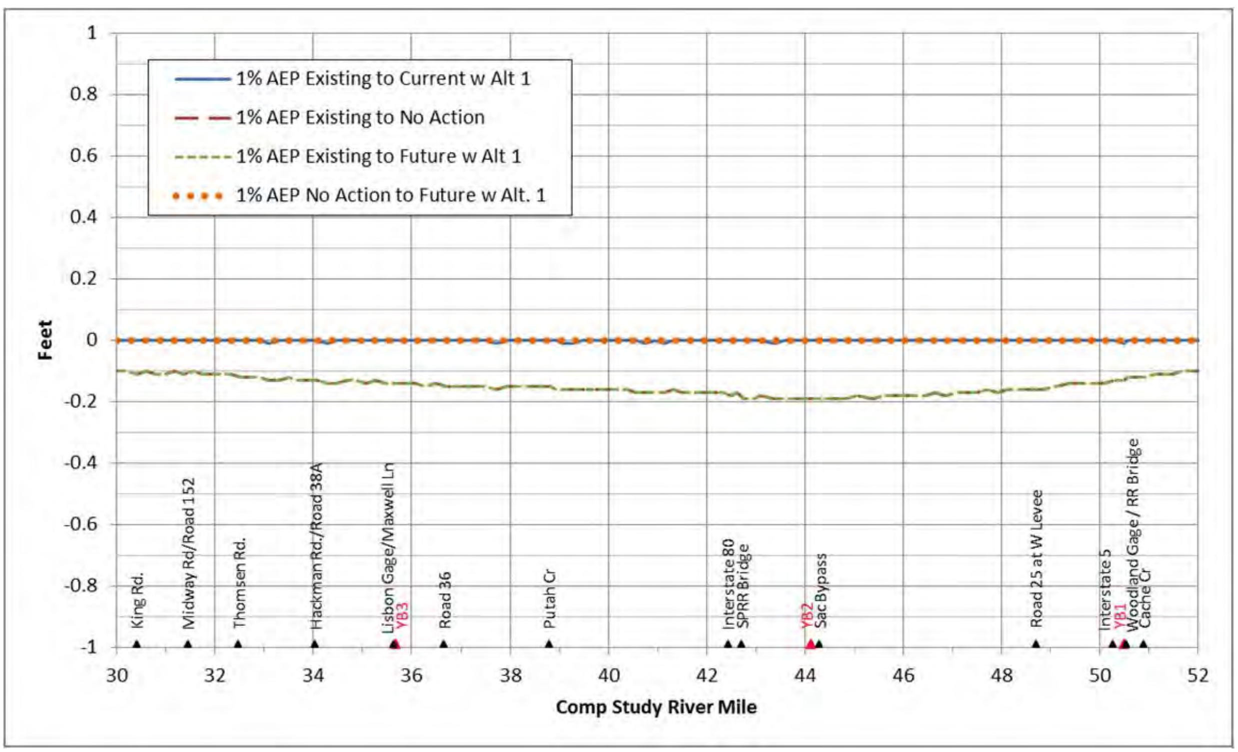

Figure 72. Maximum Water Surface Elevation Impacts; Yolo Bypass; Alternative 1; 1% AEP

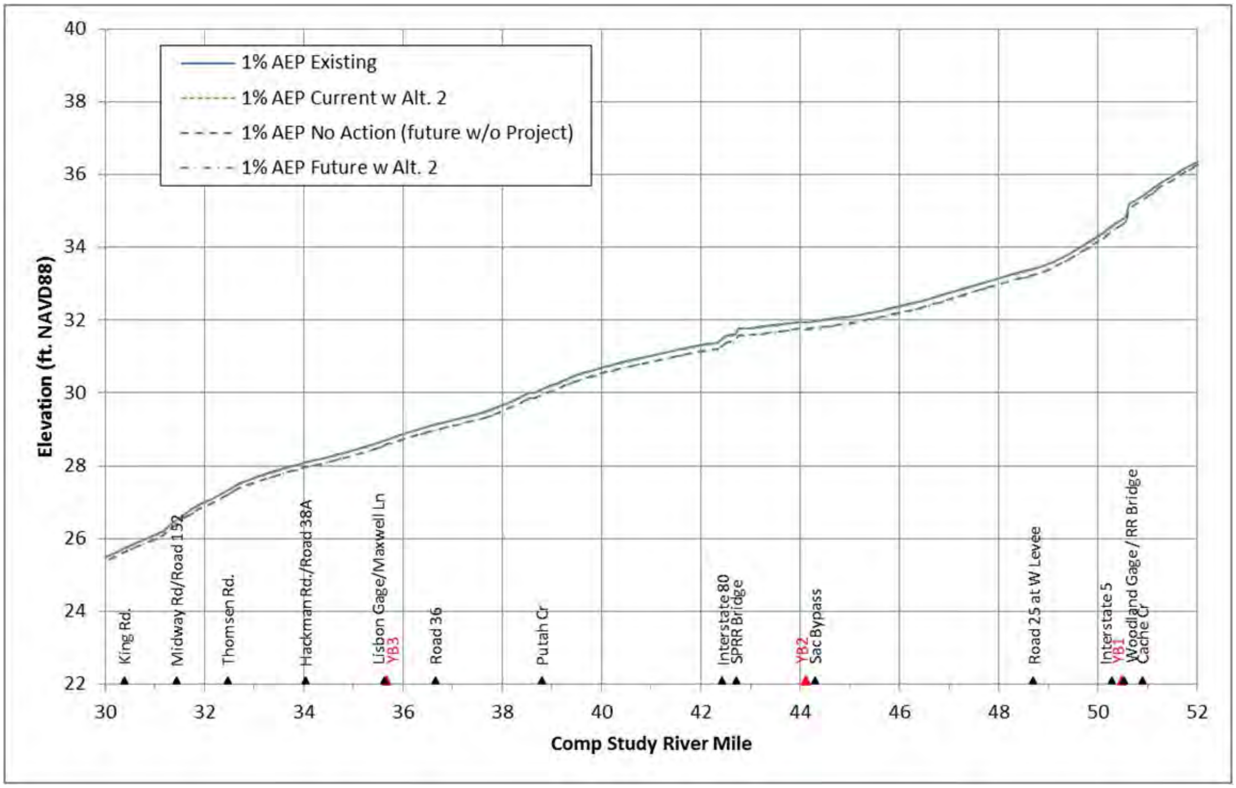

Figure 73. Maximum Water Surface Elevations; Yolo Bypass; Alternative 2; 1% AEP

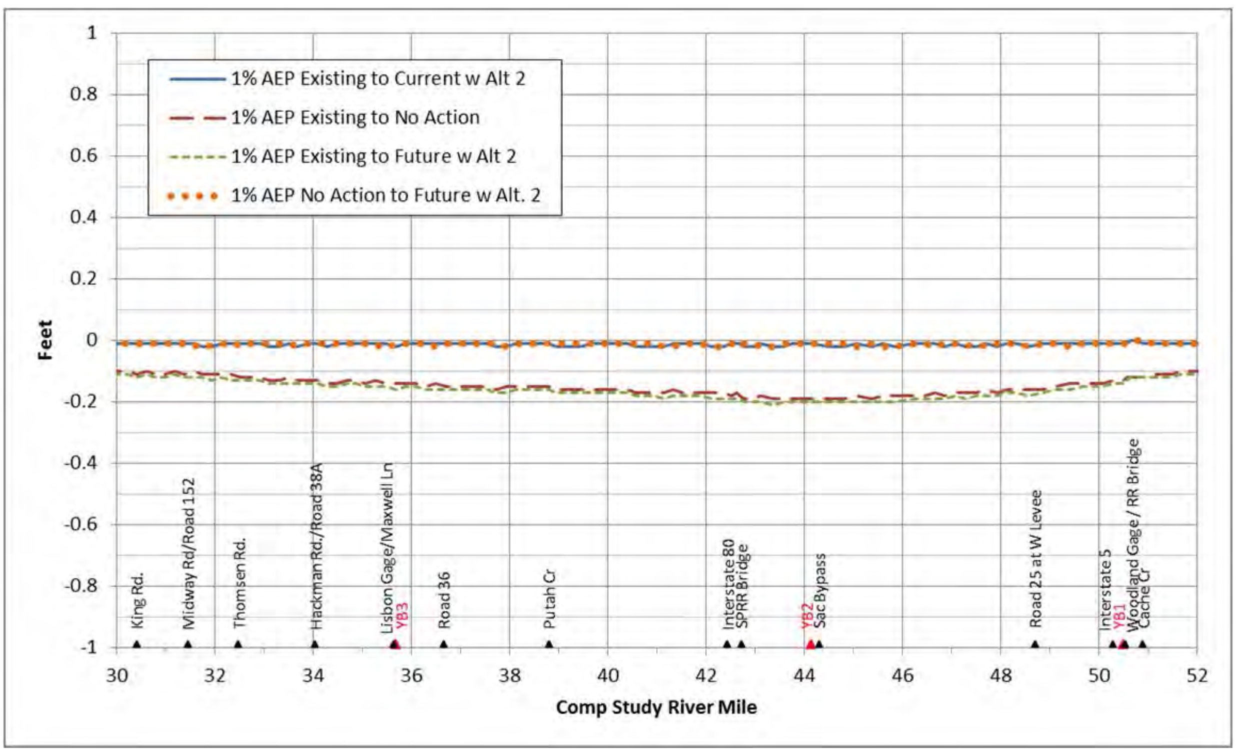

Figure 74. Maximum Water Surface Elevation Impacts; Yolo Bypass; Alternative 2; 1% AEP

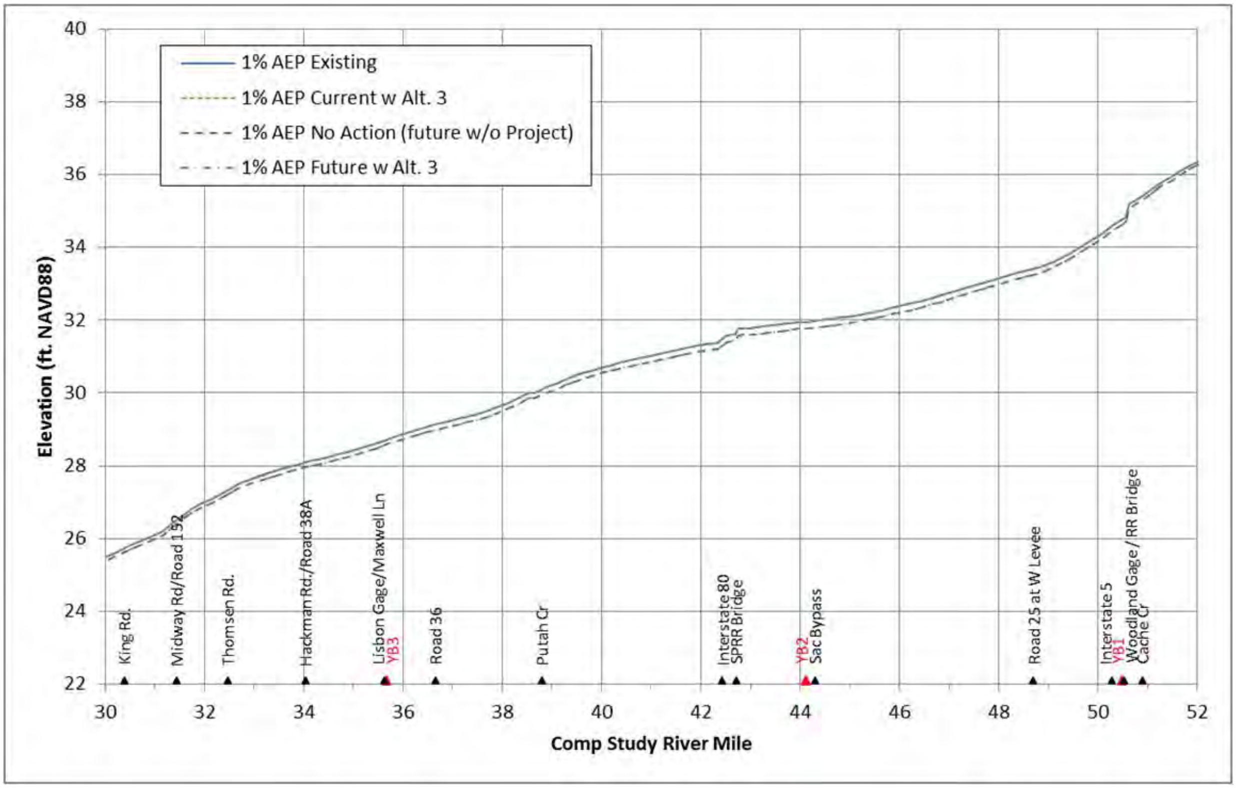

Figure 75. Maximum Water Surface Elevations; Yolo Bypass; Alternative 3; 1% AEP

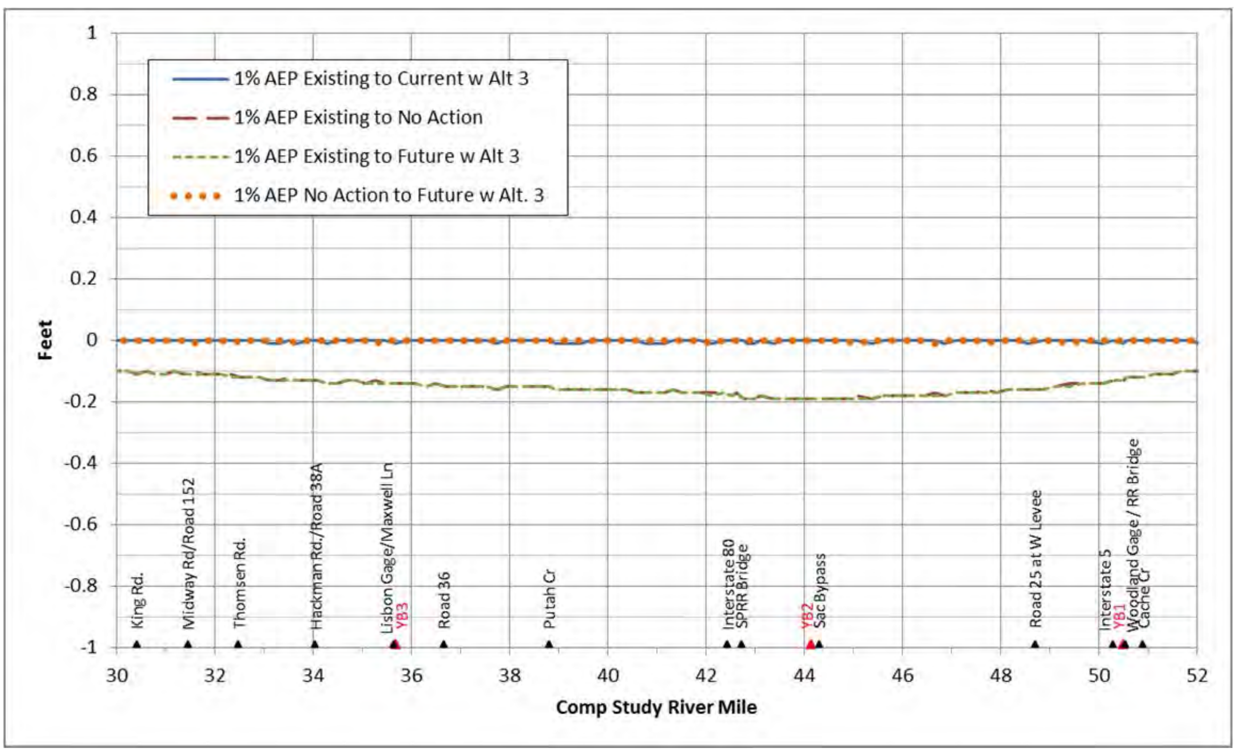

Figure 76. Maximum Water Surface Elevation Impacts; Yolo Bypass; Alternative 3; 1% AEP

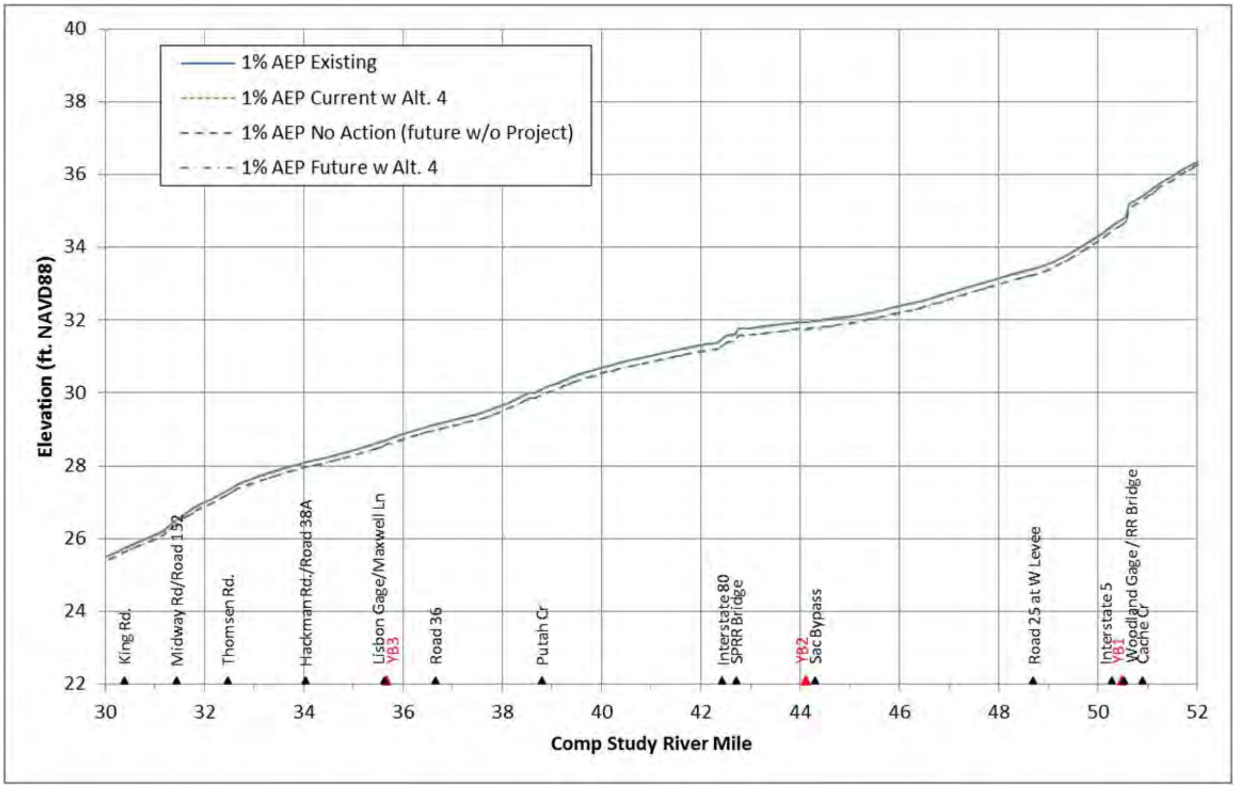

Figure 77. Maximum Water Surface Elevations; Yolo Bypass; Alternative 4; 1% AEP

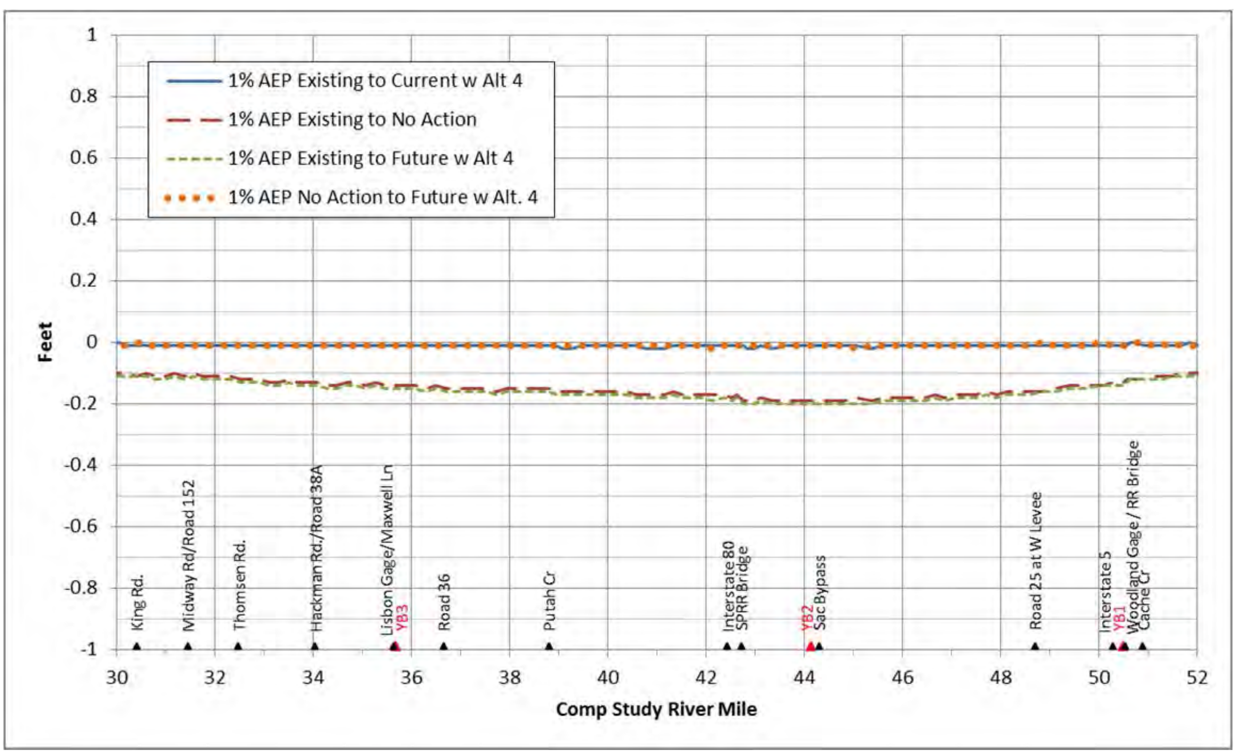

Figure 78. Maximum Water Surface Elevation Impacts; Yolo Bypass; Alternative 4; 1% AEP

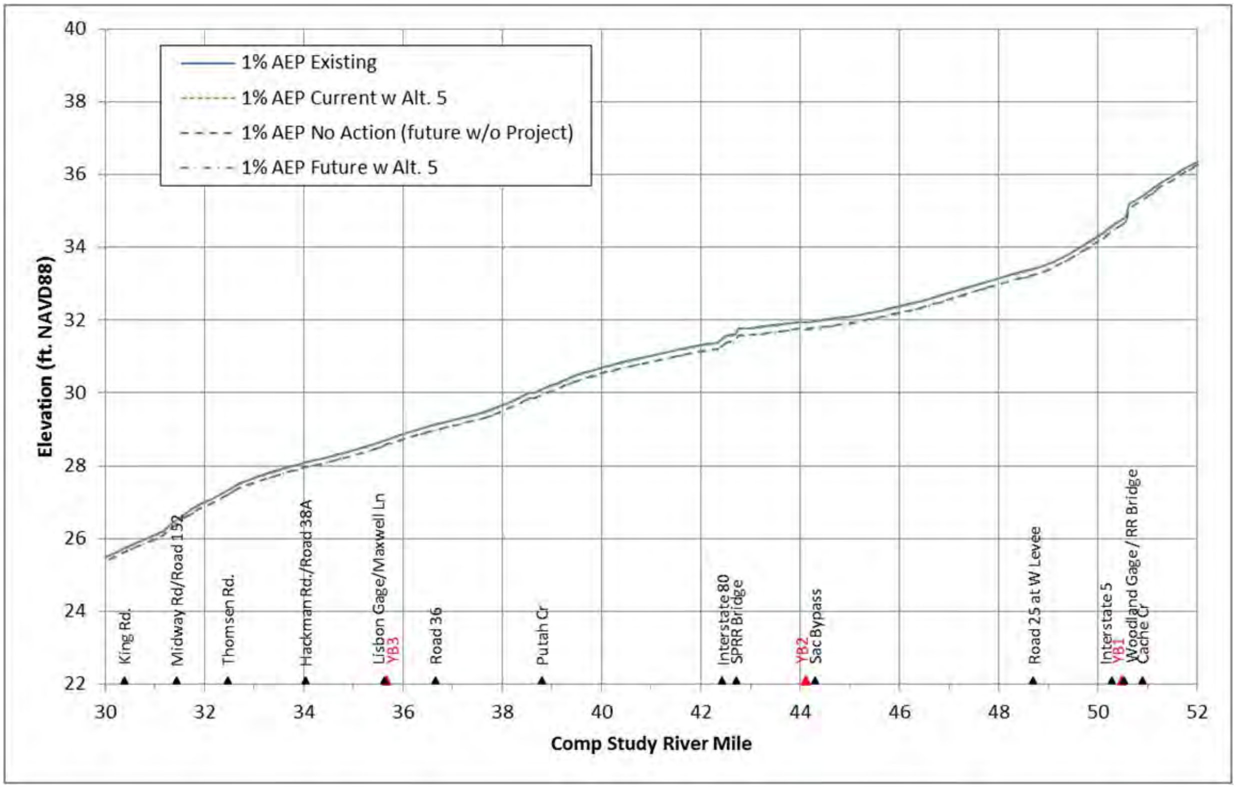

Figure 79. Maximum Water Surface Elevations; Yolo Bypass; Alternative 5; 1% AEP

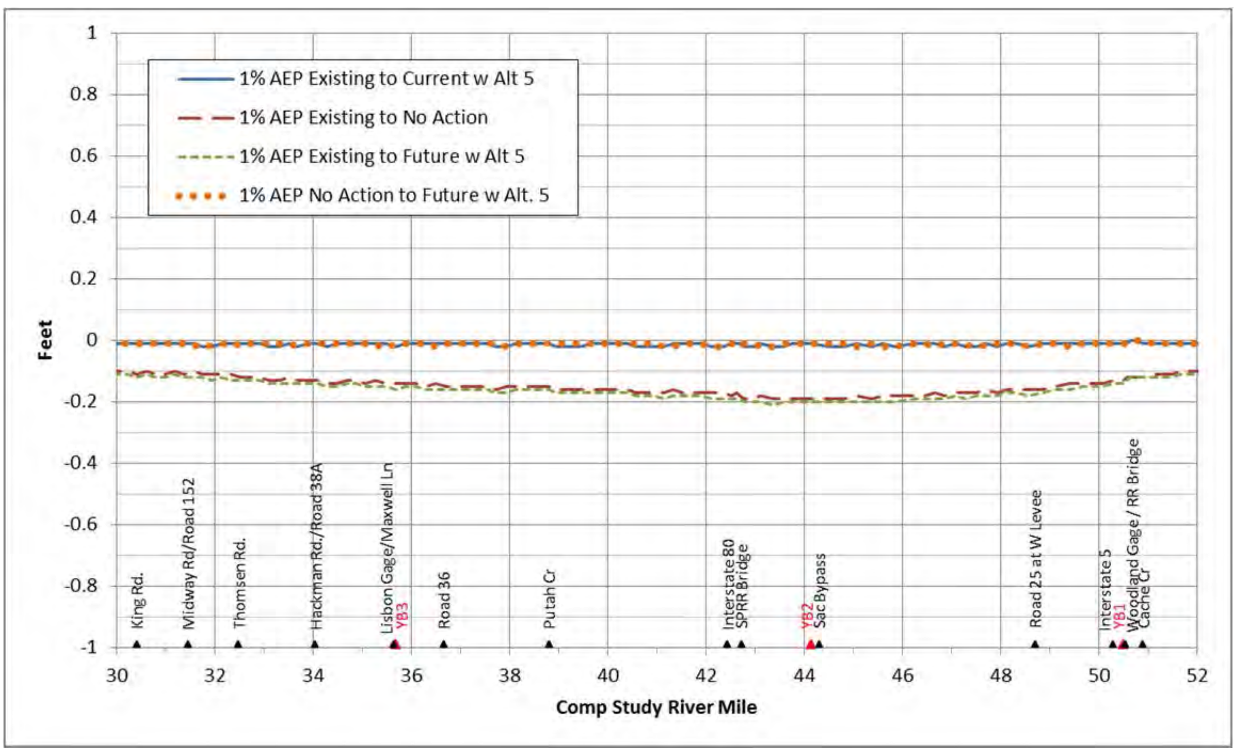

Figure 80. Maximum Water Surface Elevation Impacts; Yolo Bypass; Alternative 5; 1% AEP

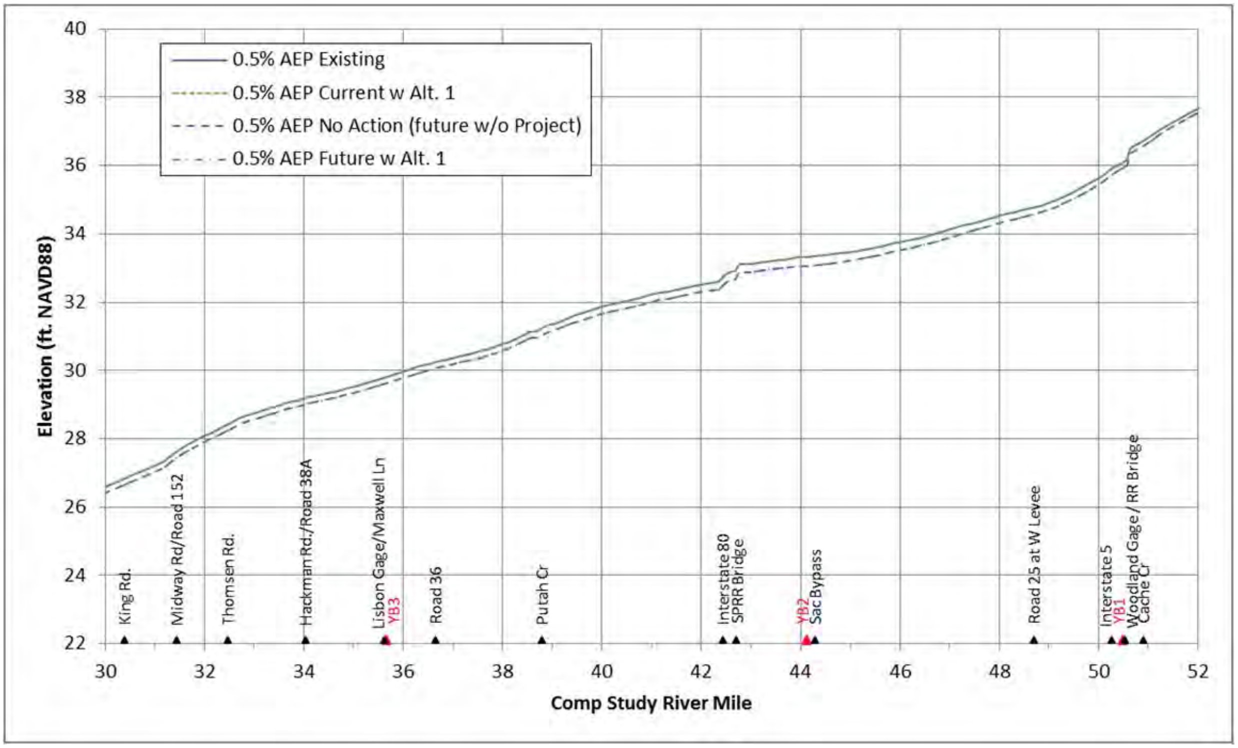

Figure 81. Maximum Water Surface Elevations; Yolo Bypass; Alternative 1; 0.5% AEP

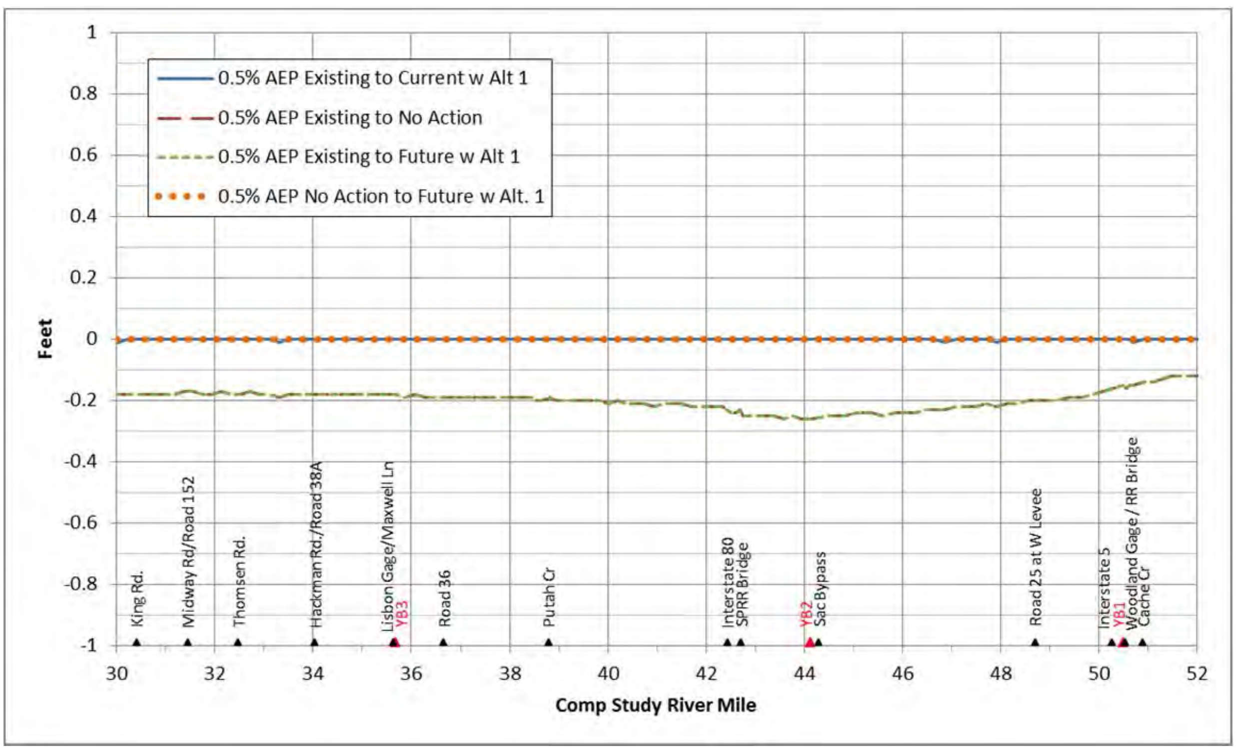

Figure 82. Maximum Water Surface Elevation Impacts; Yolo Bypass; Alternative 1; 0.5% AEP

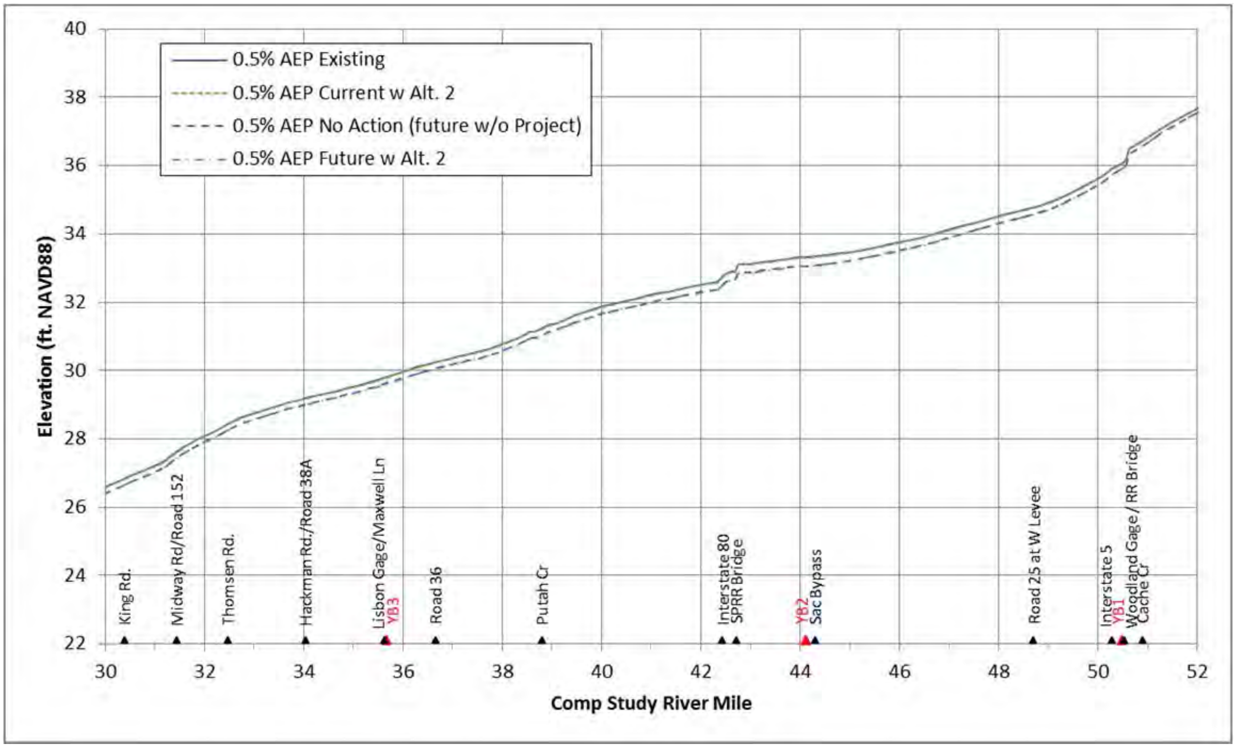

Figure 83. Maximum Water Surface Elevations; Yolo Bypass; Alternative 2; 0.5% AEP

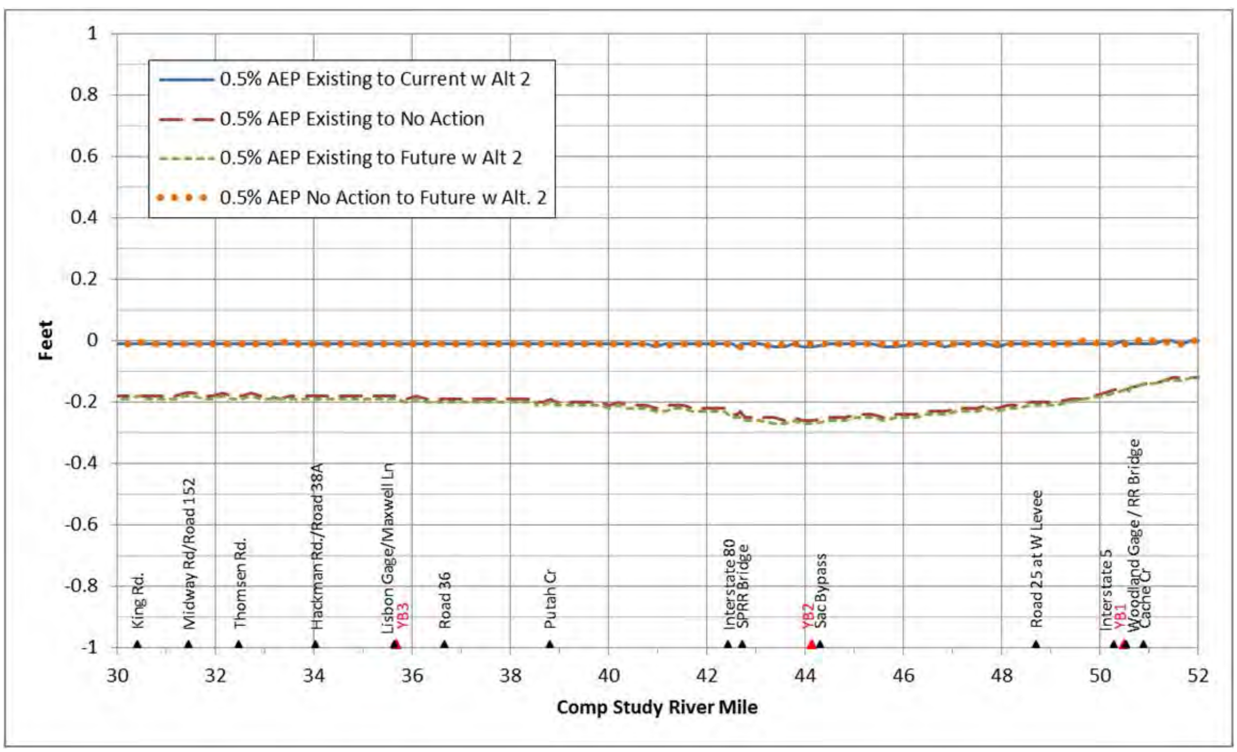

Figure 84. Maximum Water Surface Elevation Impacts; Yolo Bypass; Alternative 2; 0.5% AEP

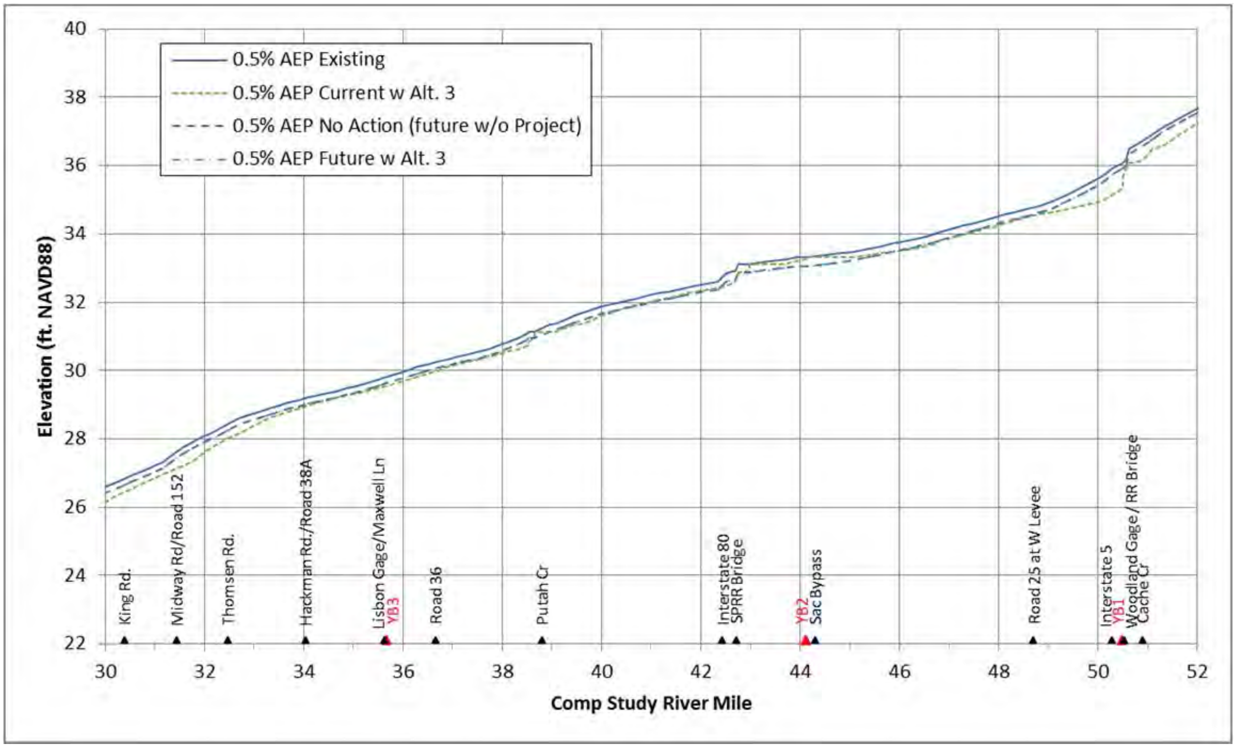

Figure 85. Maximum Water Surface Elevations; Yolo Bypass; Alternative 3; 0.5% AEP

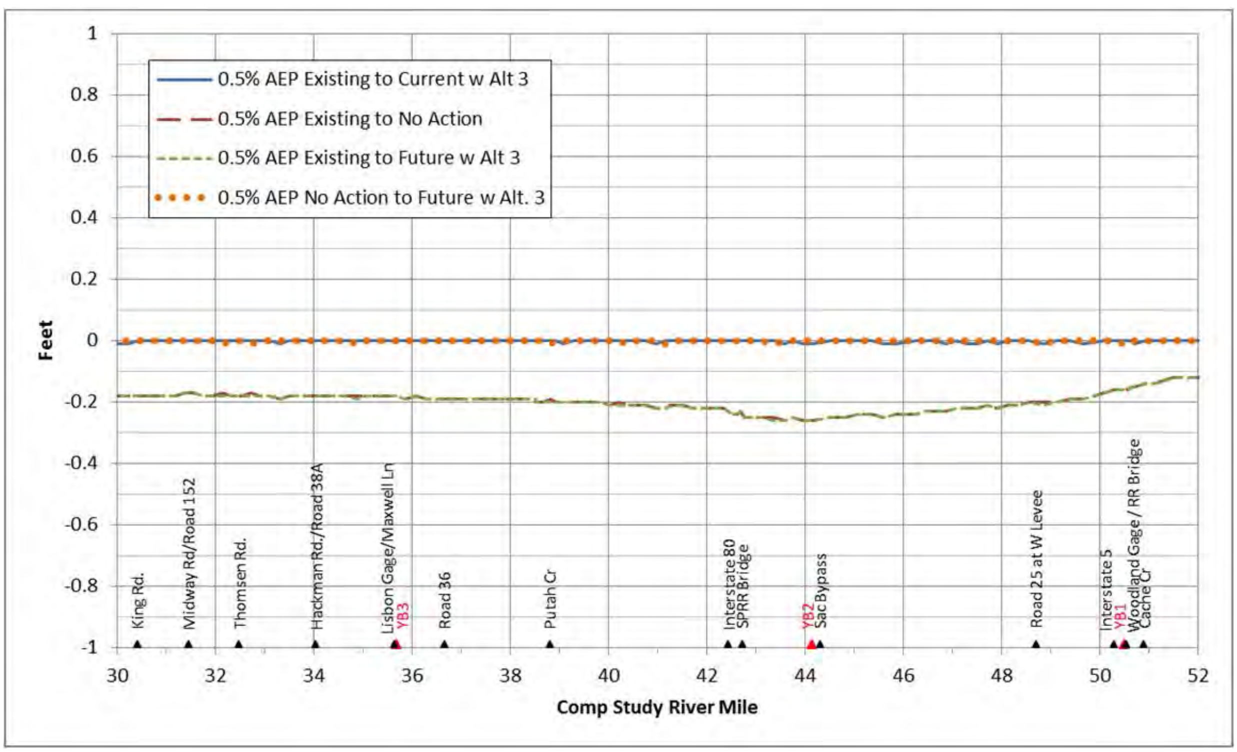

Figure 86. Maximum Water Surface Elevation Impacts; Yolo Bypass; Alternative 3; 0.5% AEP

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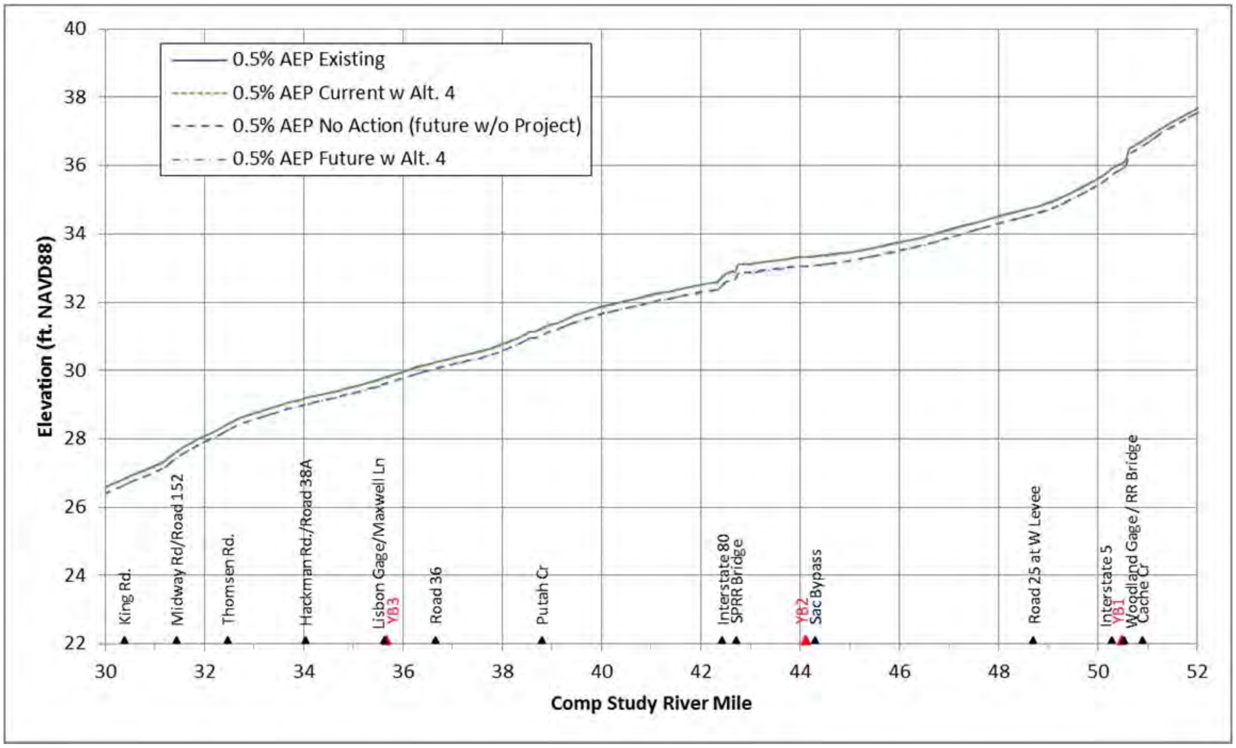


Figure 87. Maximum Water Surface Elevations; Yolo Bypass; Alternative 4; 0.5% AEP

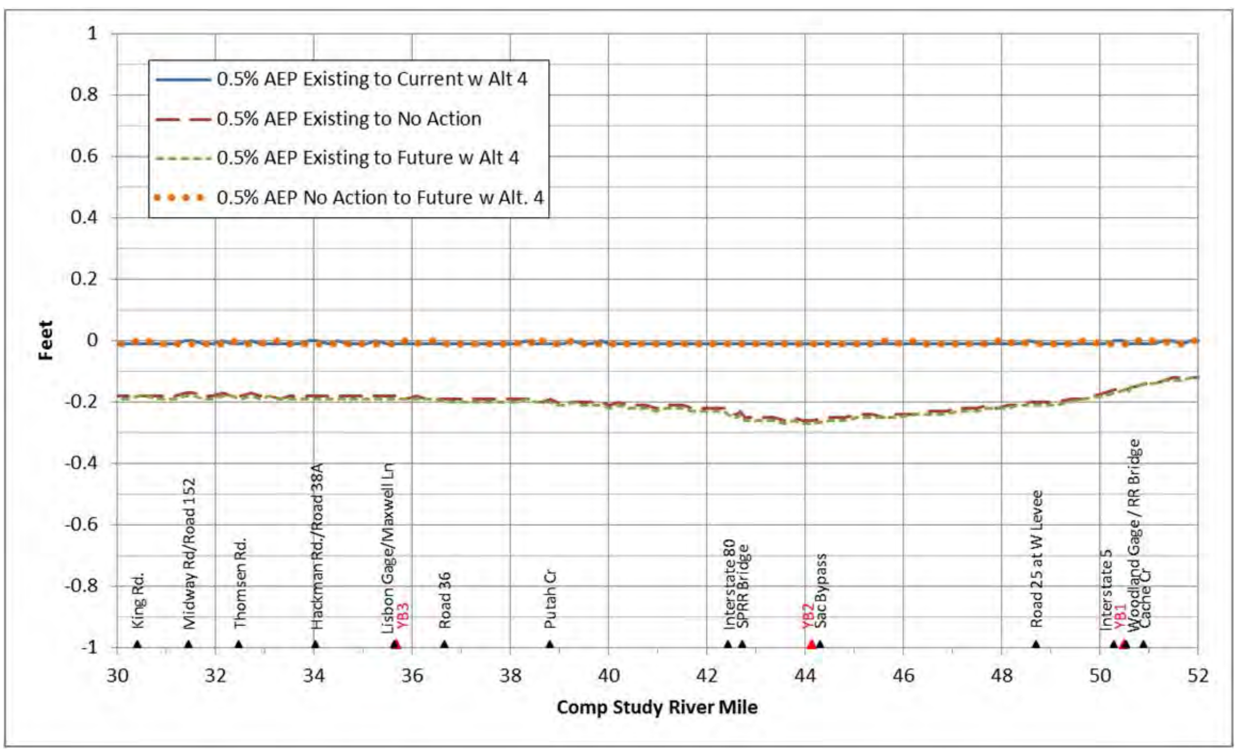


Figure 88. Maximum Water Surface Elevation Impacts; Yolo Bypass; Alternative 4; 0.5% AEP

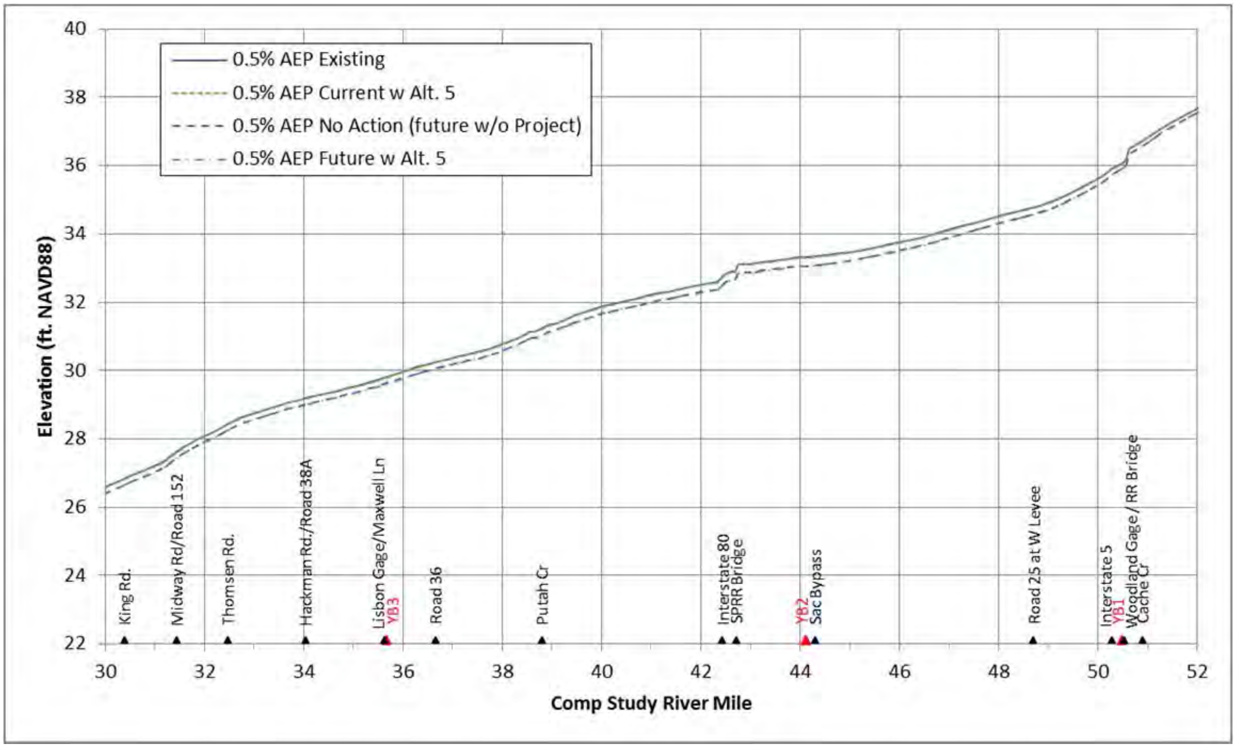


Figure 89. Maximum Water Surface Elevations; Yolo Bypass; Alternative 5; 0.5% AEP

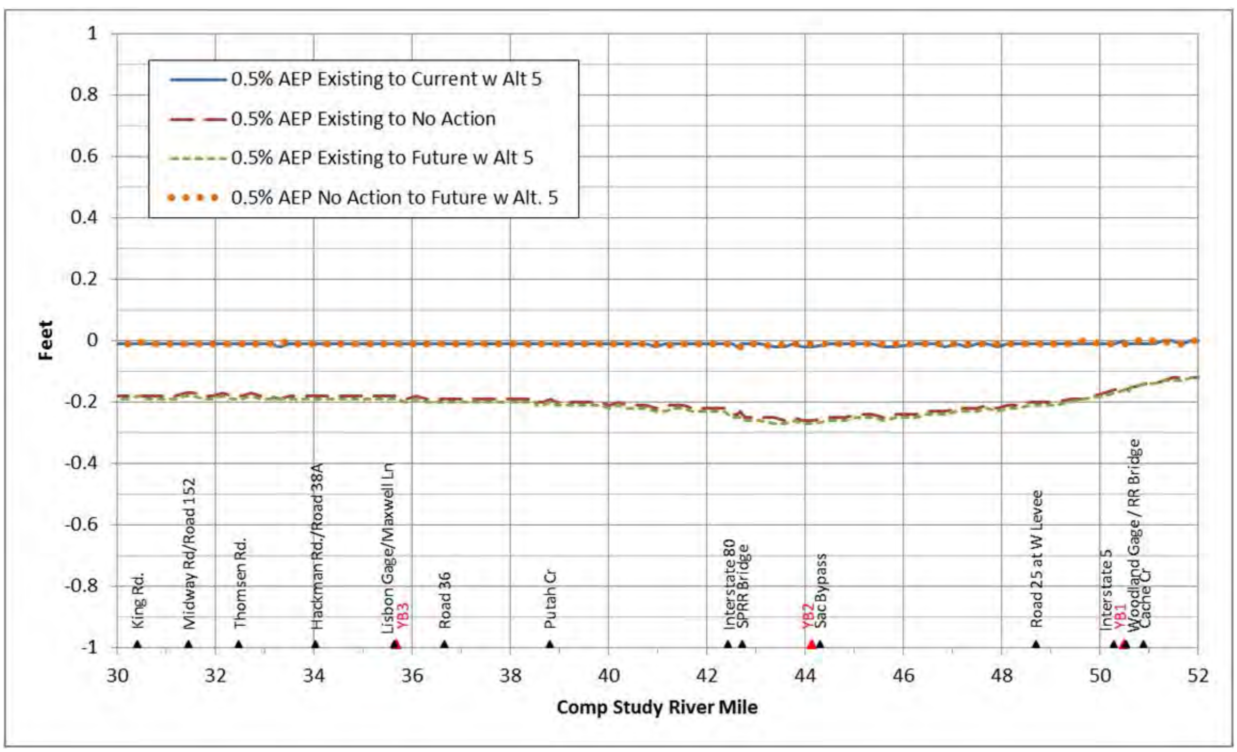


Figure 90. Maximum Water Surface Elevation Impacts; Yolo Bypass; Alternative 5; 0.5% AEP

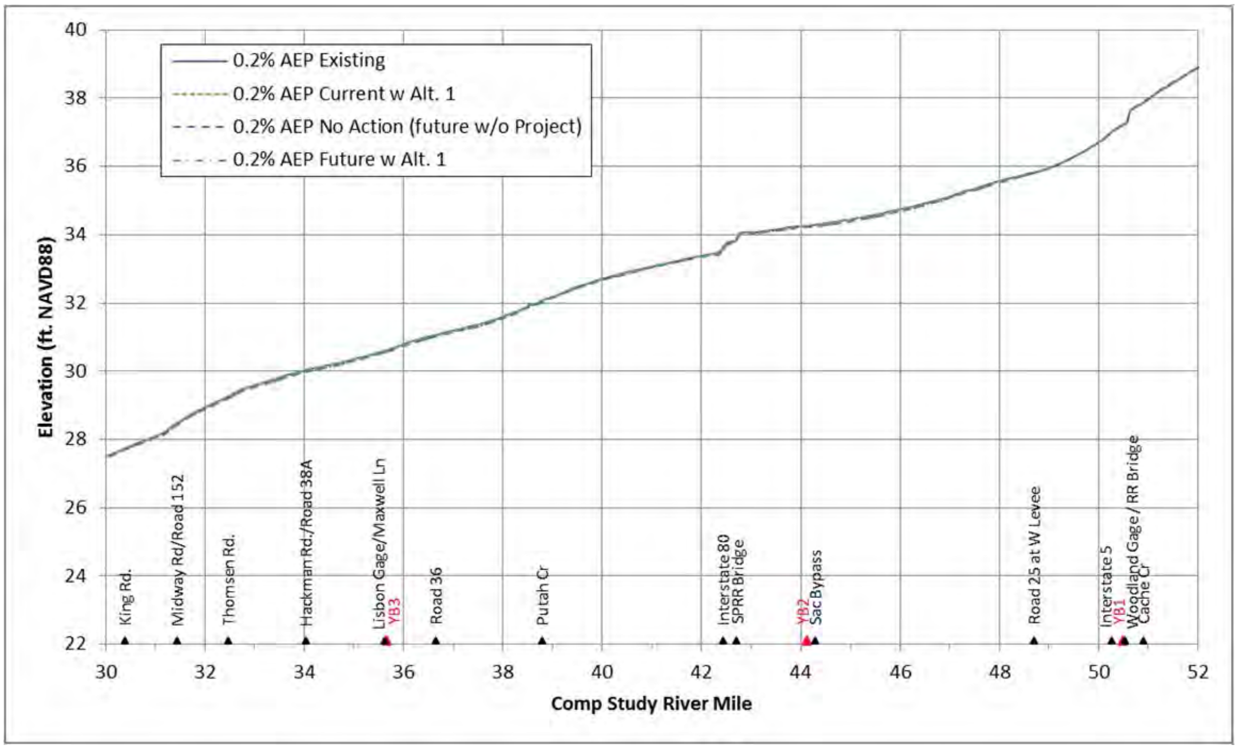


Figure 91. Maximum Water Surface Elevations; Yolo Bypass; Alternative 1; 0.2% AEP

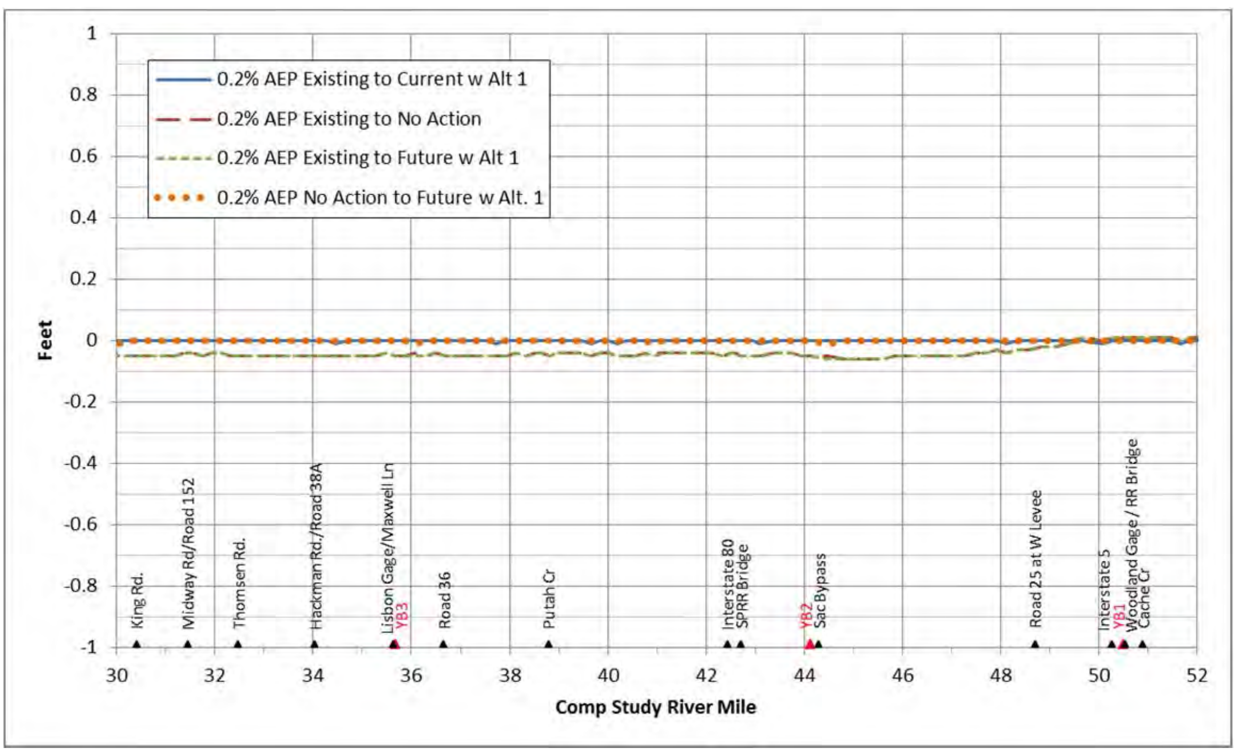


Figure 92. Maximum Water Surface Elevation Impacts; Yolo Bypass; Alternative 1; 0.2% AEP

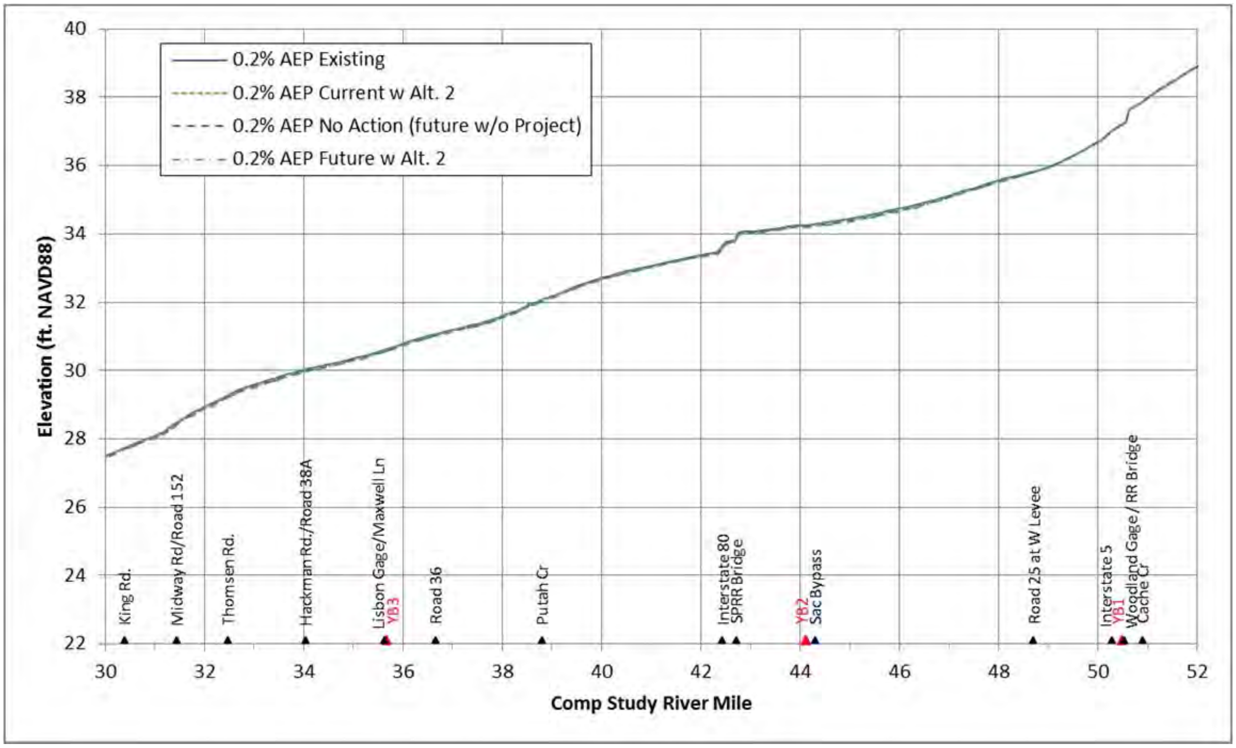


Figure 93. Maximum Water Surface Elevations; Yolo Bypass; Alternative 2; 0.2% AEP

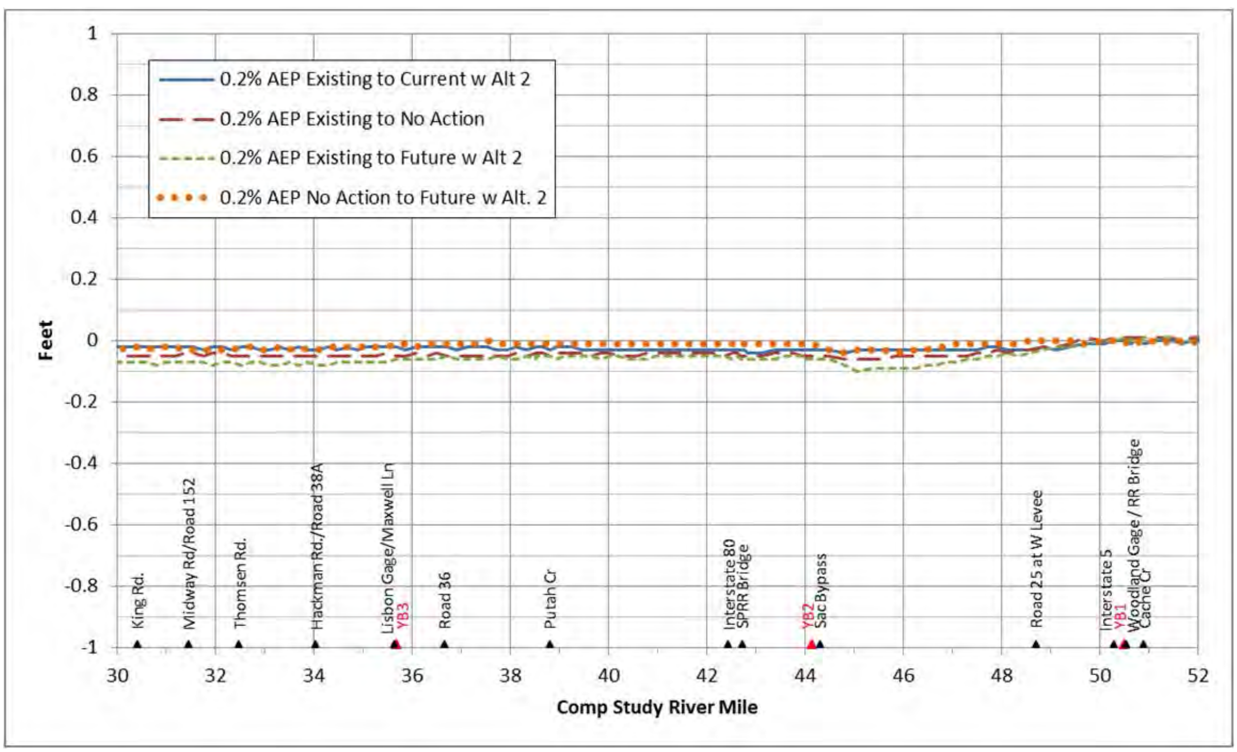


Figure 94. Maximum Water Surface Elevation Impacts; Yolo Bypass; Alternative 2; 0.2% AEP

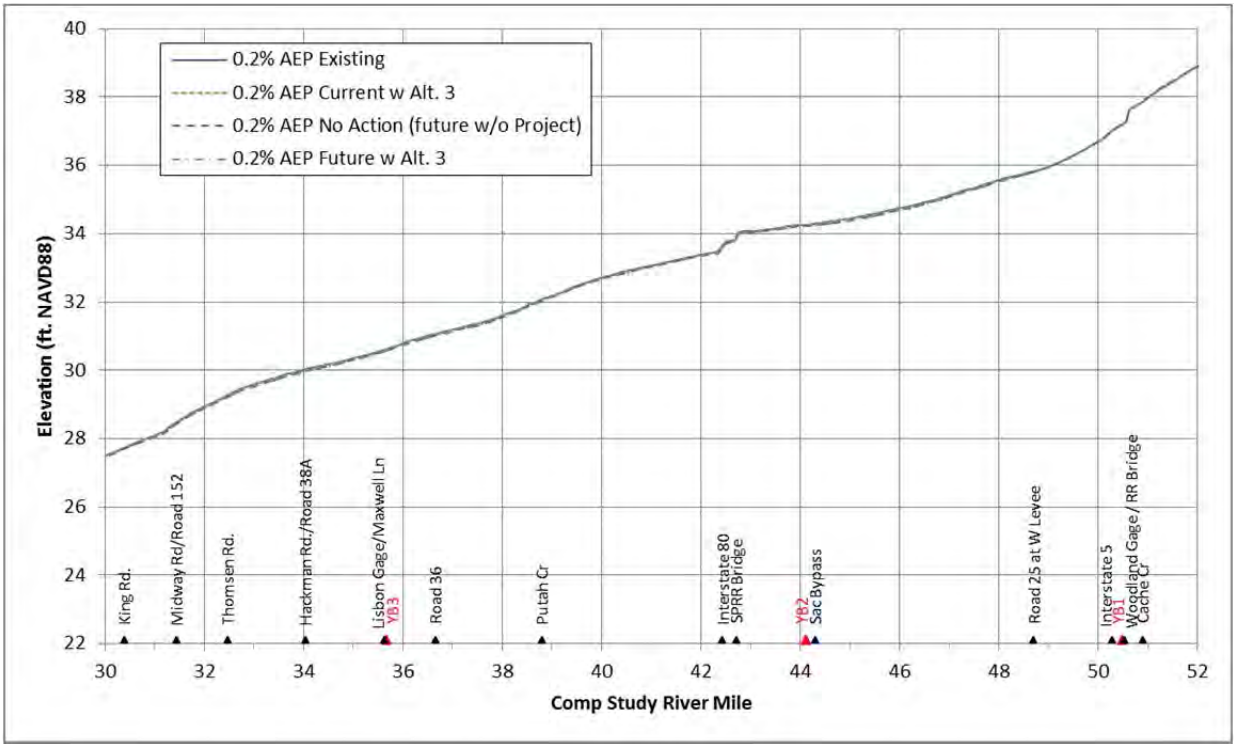


Figure 95. Maximum Water Surface Elevations; Yolo Bypass; Alternative 3; 0.2% AEP

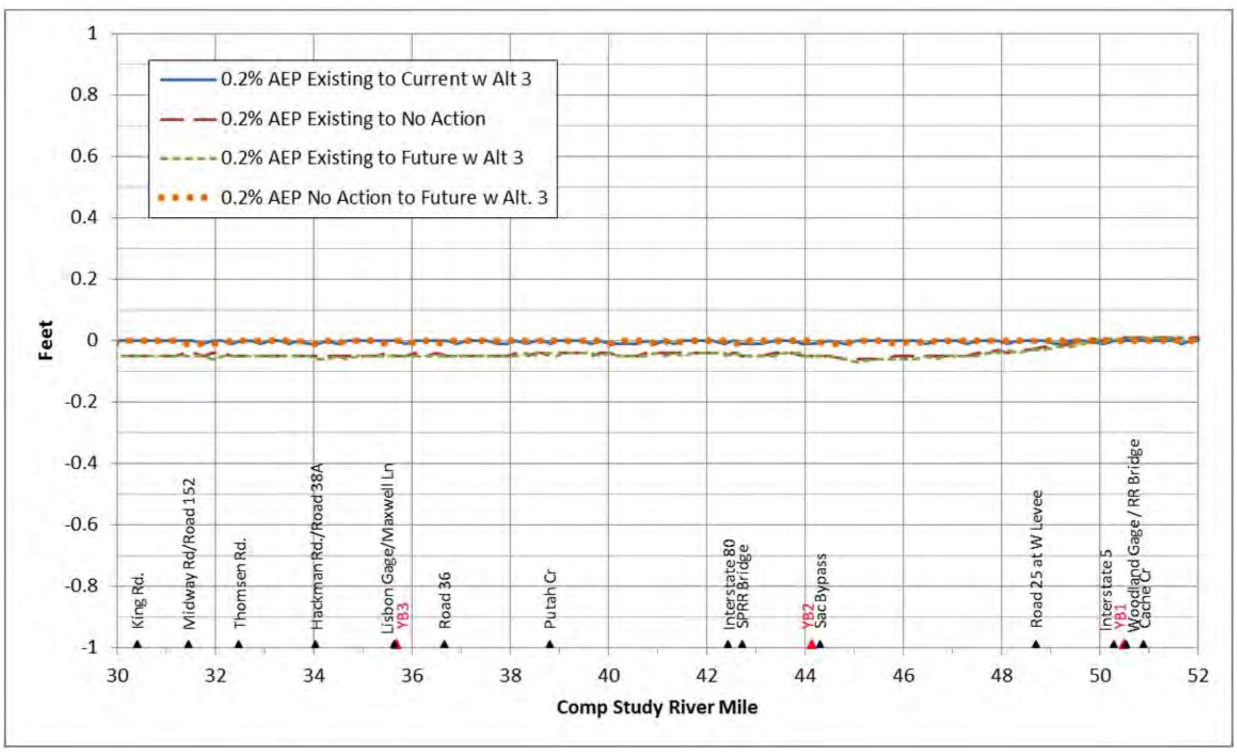


Figure 96. Maximum Water Surface Elevation Impacts; Yolo Bypass; Alternative 3; 0.2% AEP

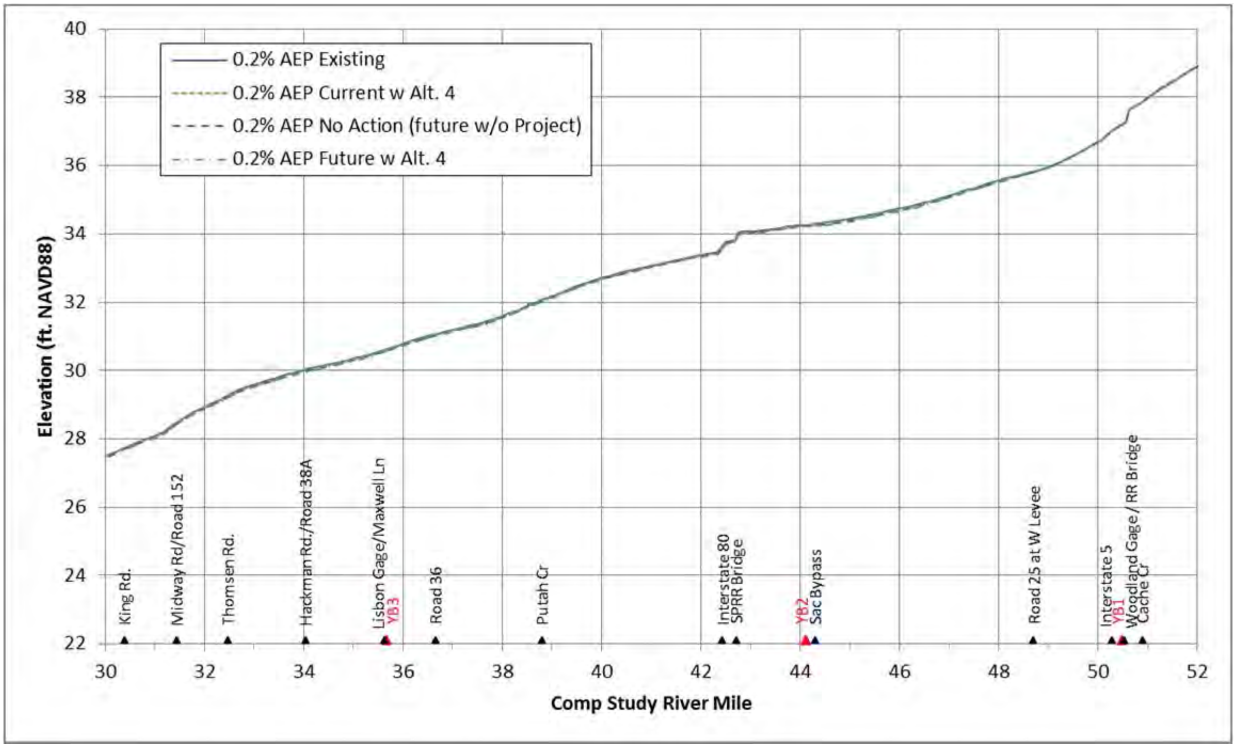


Figure 97. Maximum Water Surface Elevations; Yolo Bypass; Alternative 4; 0.2% AEP

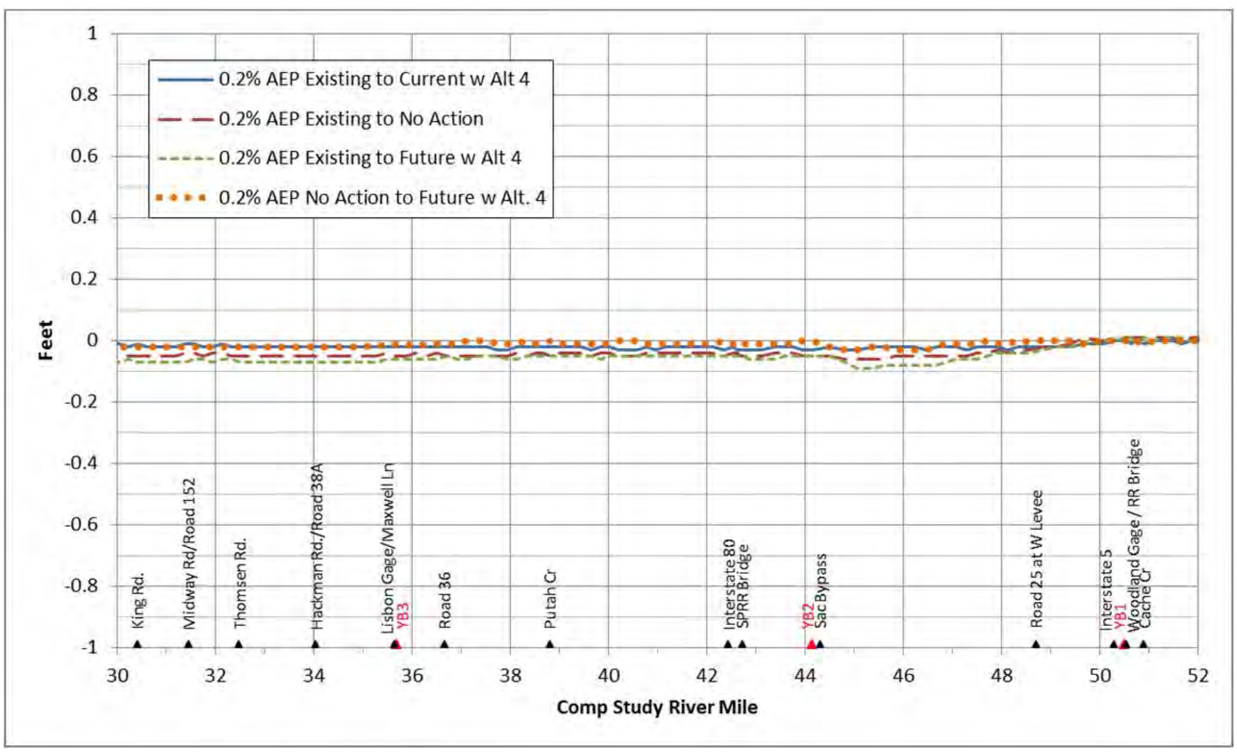


Figure 98. Maximum Water Surface Elevation Impacts; Yolo Bypass; Alternative 4; 0.2% AEP

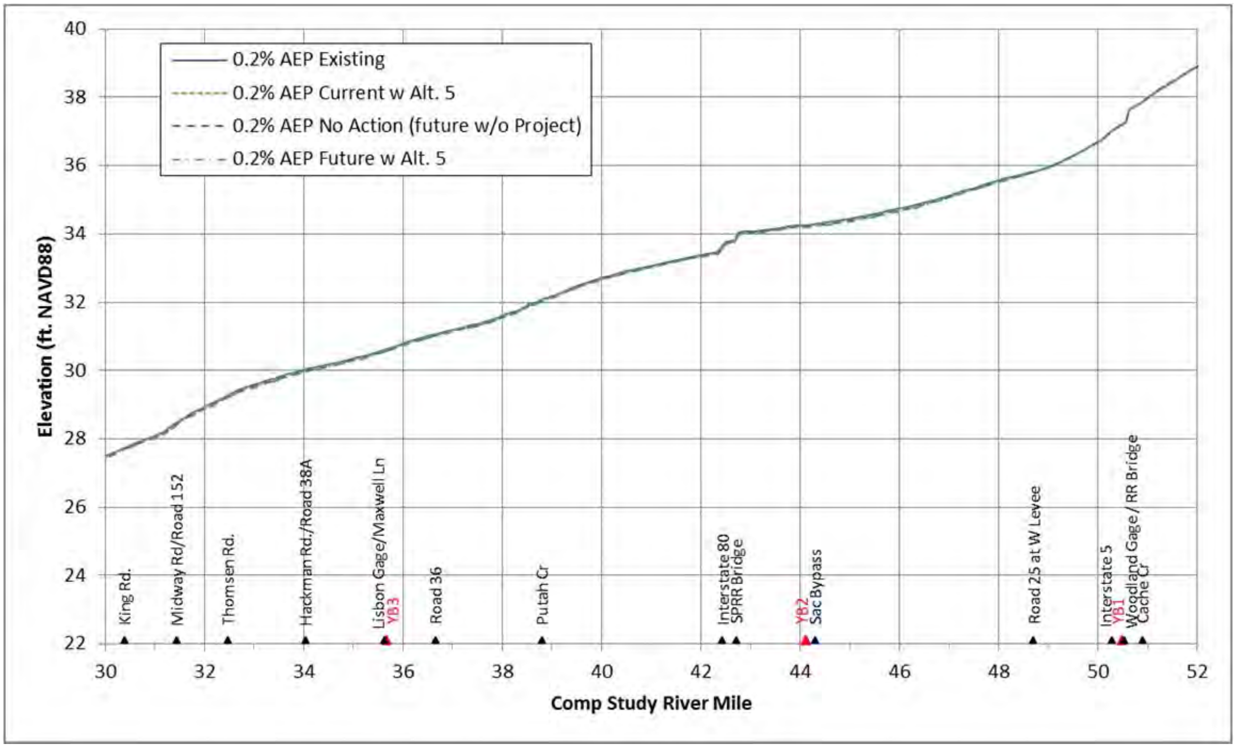


Figure 99. Maximum Water Surface Elevations; Yolo Bypass; Alternative 5; 0.2% AEP

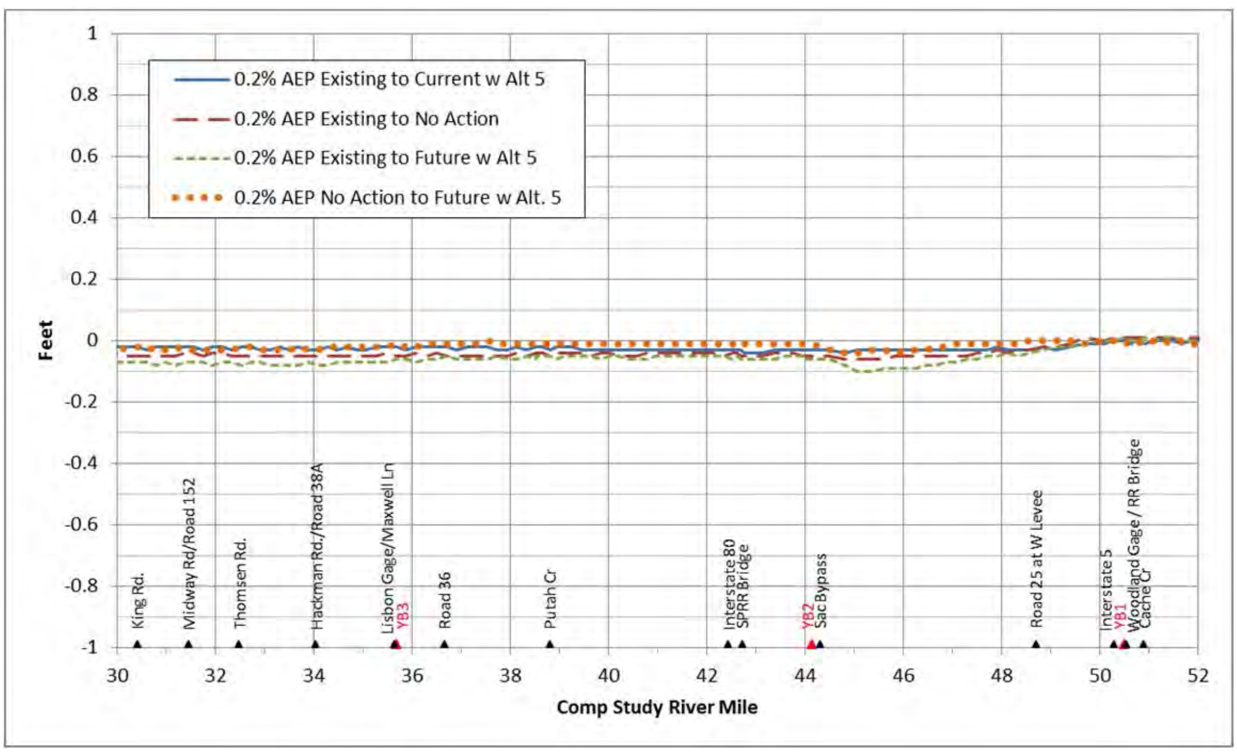


Figure 100. Maximum Water Surface Elevation Impacts; Yolo Bypass; Alternative 5; 0.2% AEP

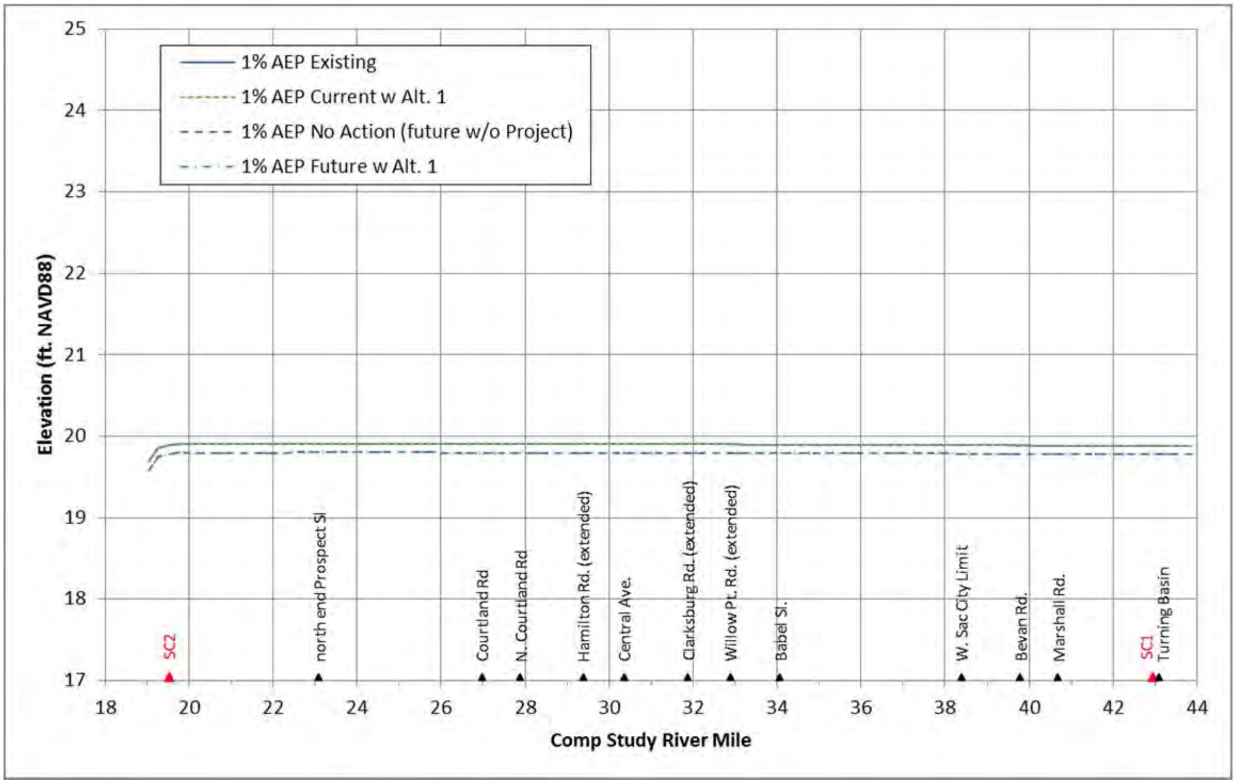


Figure 101. Maximum Water Surface Elevations; Deep Water Ship Channel; Alternative 1; 1% AEP

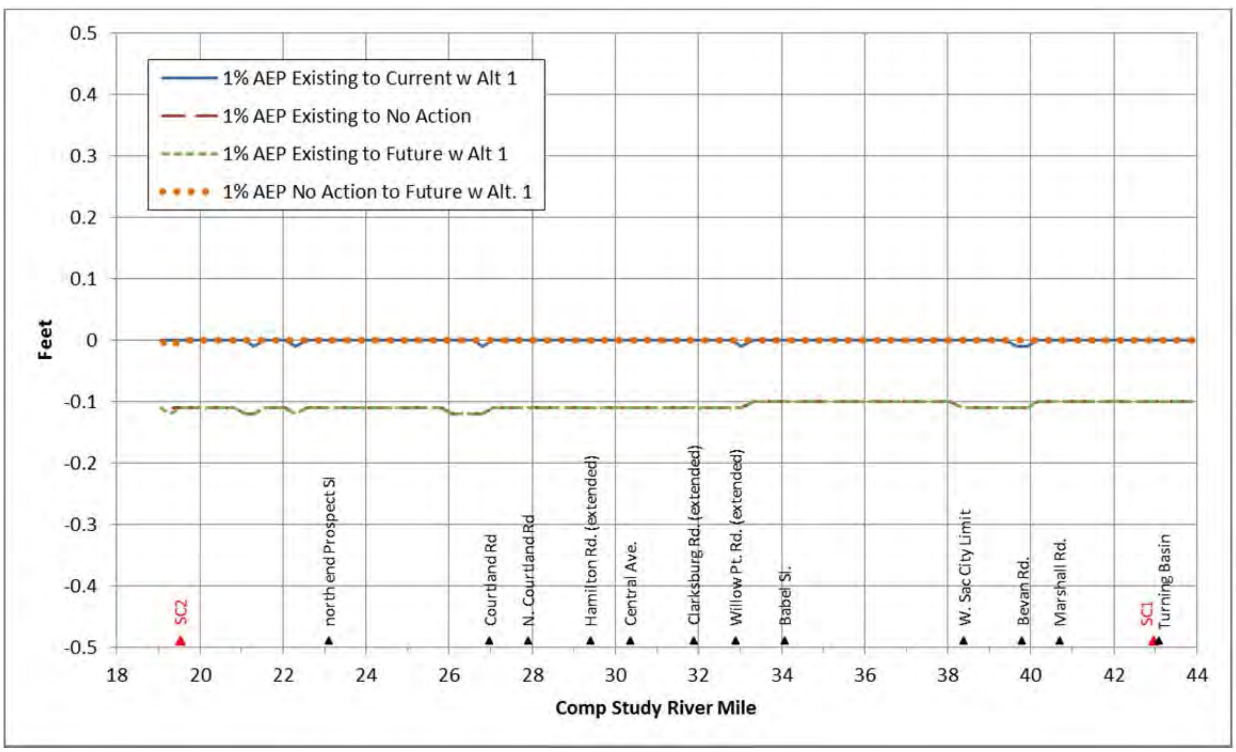


Figure 102. Maximum Water Surface Elevation Impacts; Deep Water Ship Channel; Alternative 1; 1% AEP

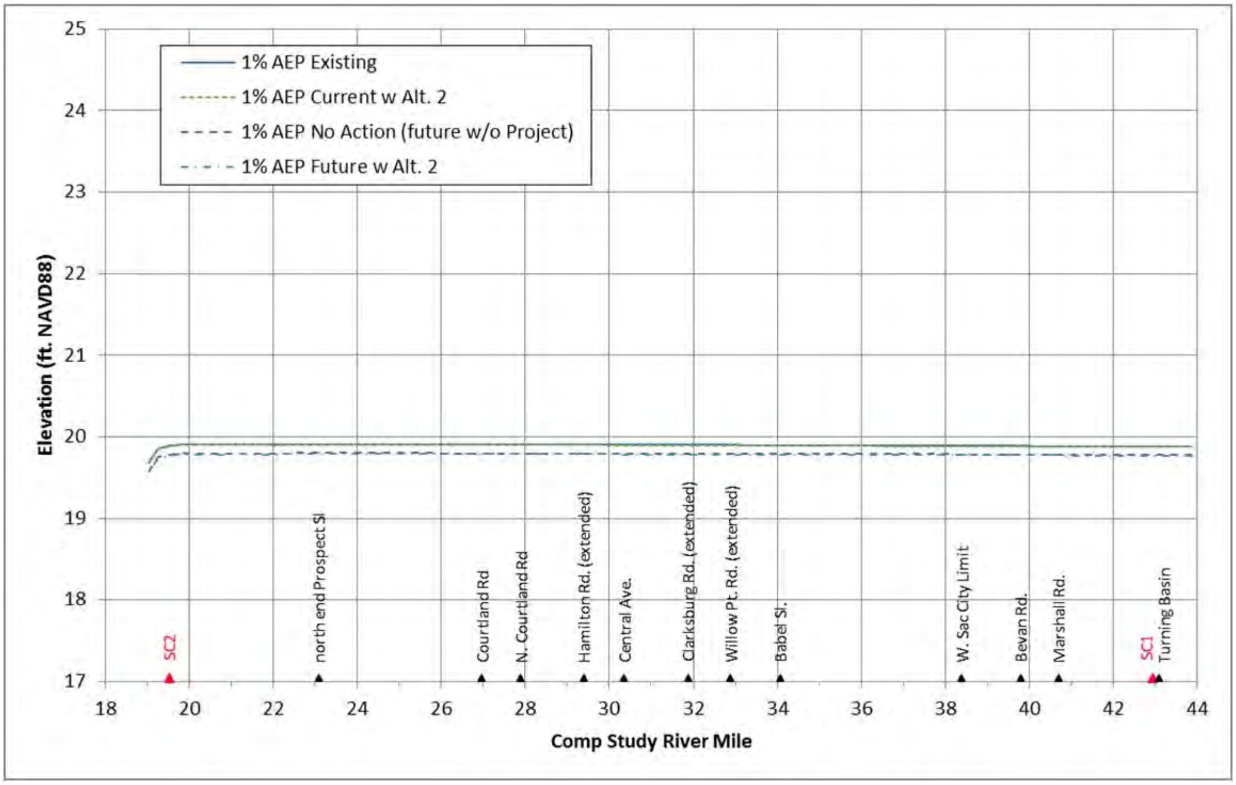


Figure 103. Maximum Water Surface Elevations; Deep Water Ship Channel; Alternative 2; 1% AEP

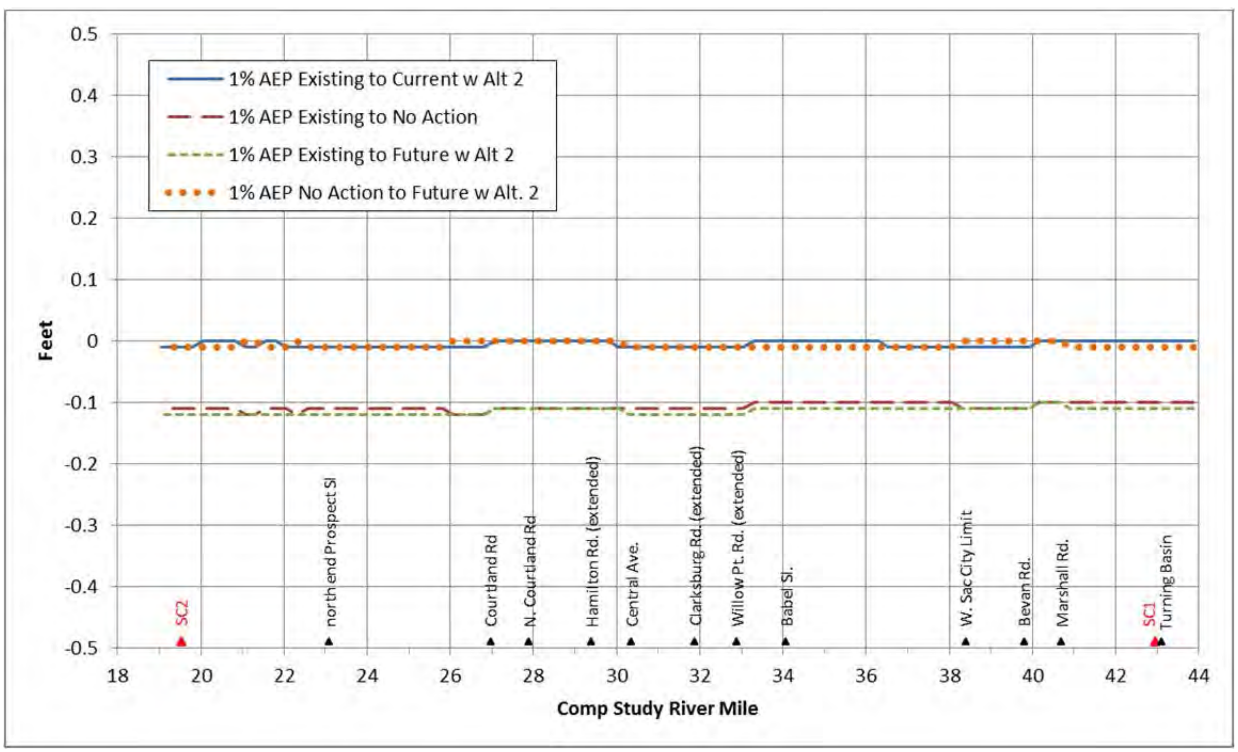


Figure 104. Maximum Water Surface Elevation Impacts; Deep Water Ship Channel; Alternative 2; 1% AEP

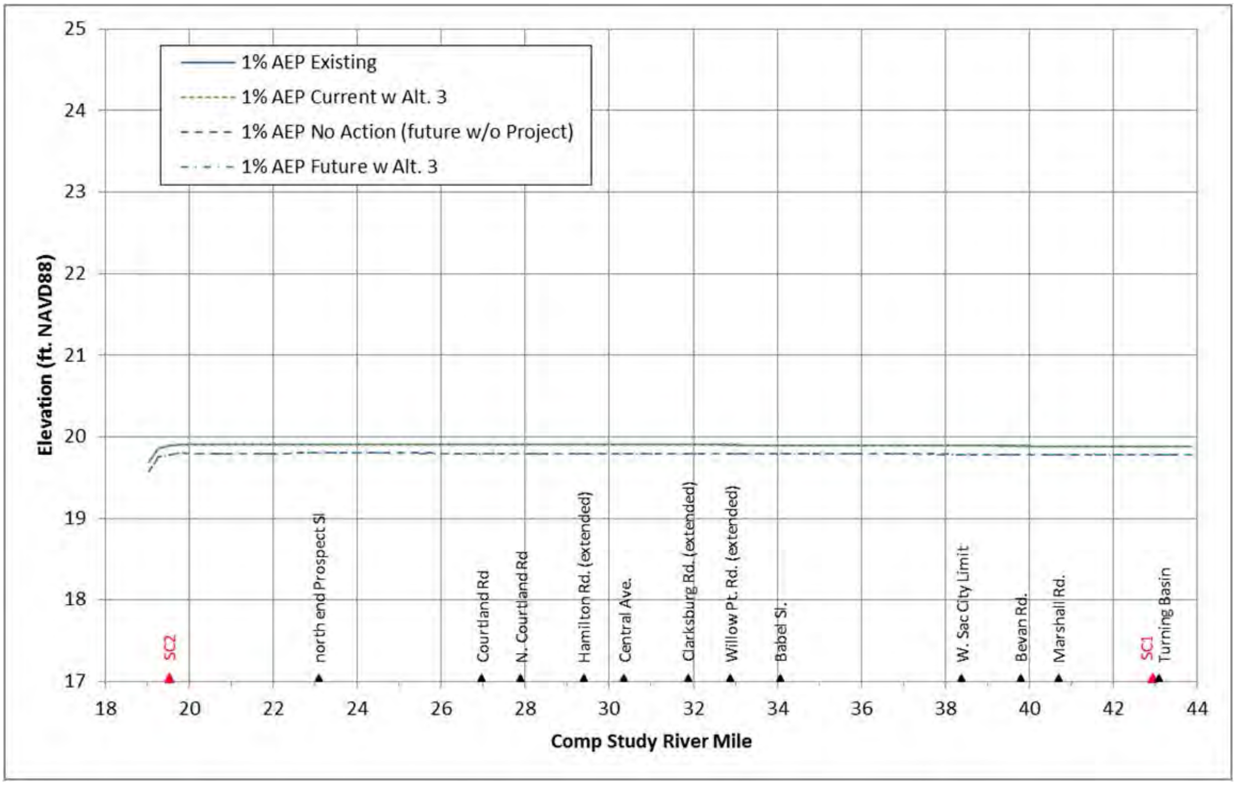


Figure 105. Maximum Water Surface Elevations; Deep Water Ship Channel; Alternative 3; 1% AEP

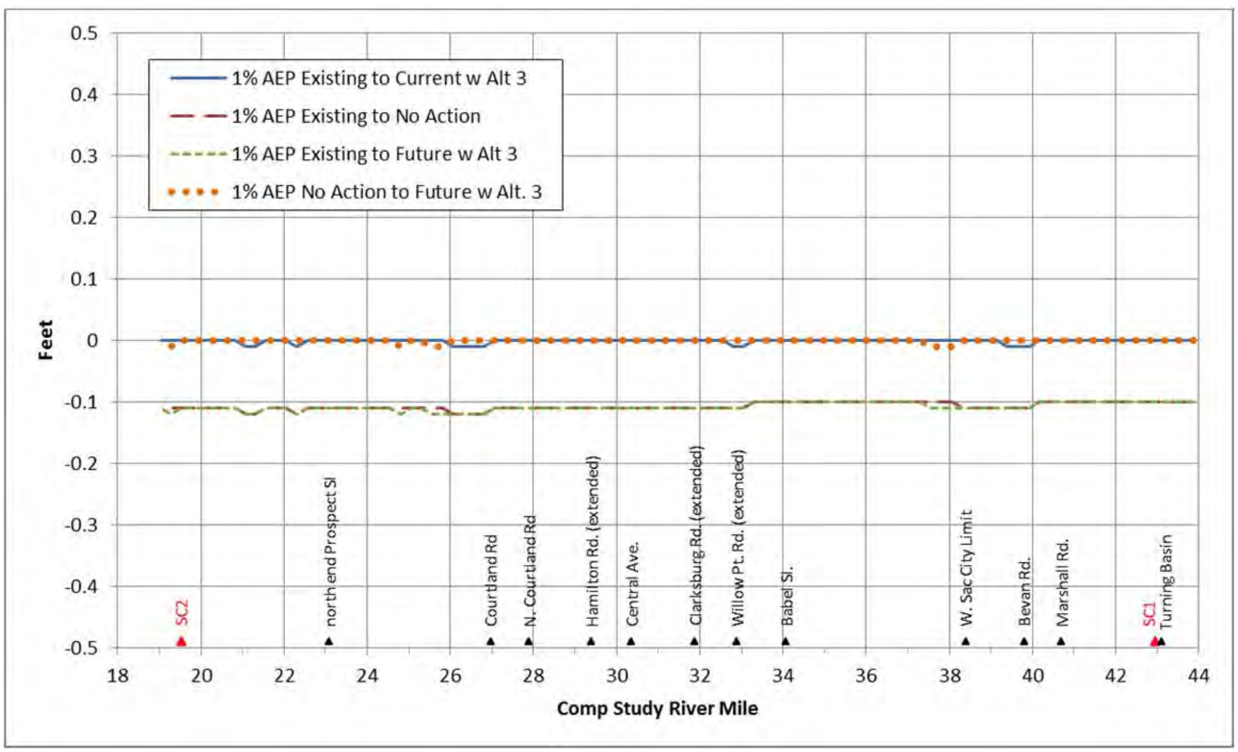


Figure 106. Maximum Water Surface Elevation Impacts; Deep Water Ship Channel; Alternative 3; 1% AEP

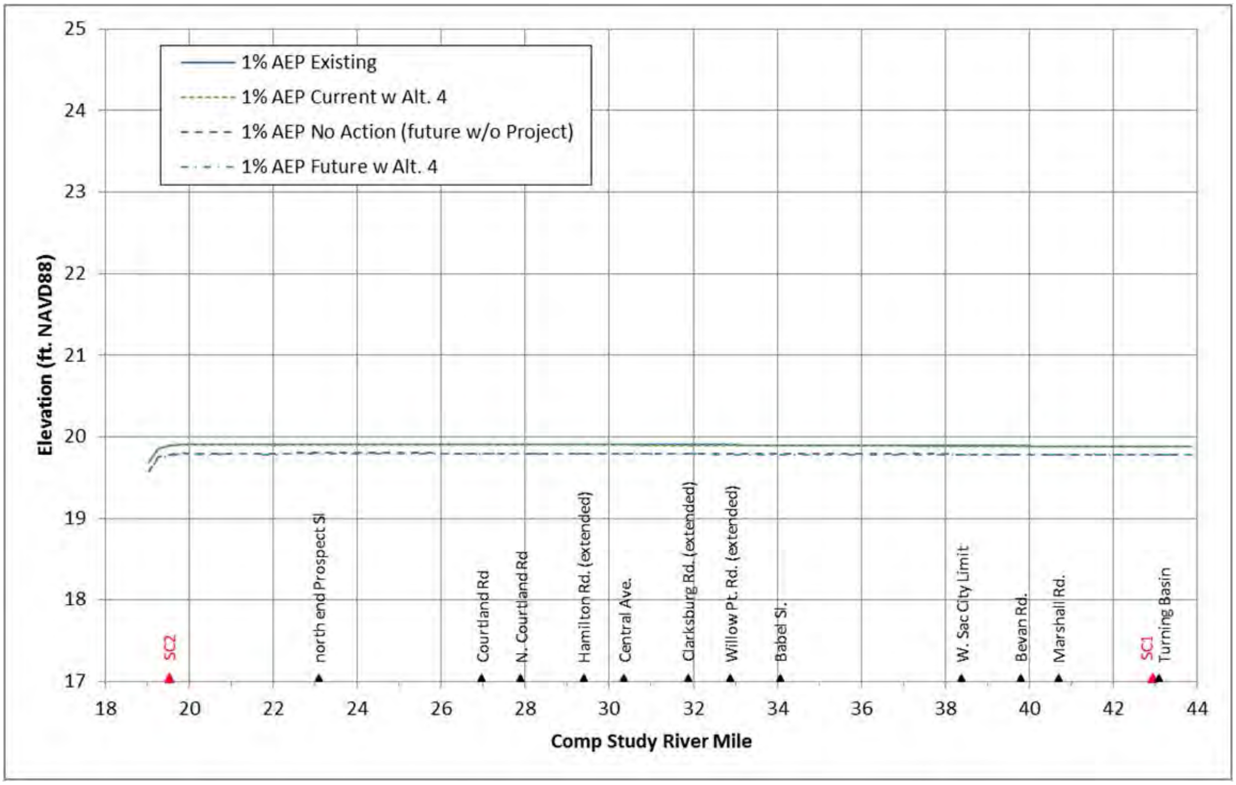


Figure 107. Maximum Water Surface Elevations; Deep Water Ship Channel; Alternative 4; 1% AEP

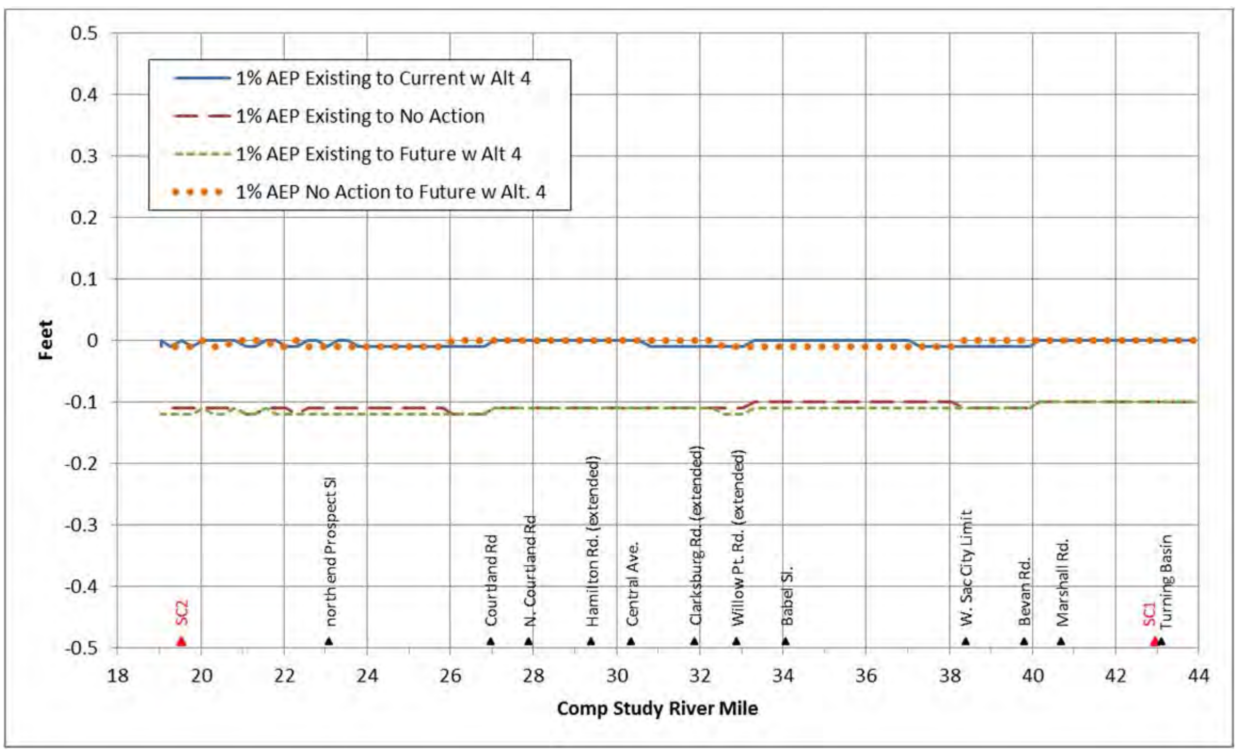


Figure 108. Maximum Water Surface Elevation Impacts; Deep Water Ship Channel; Alternative 4; 1% AEP

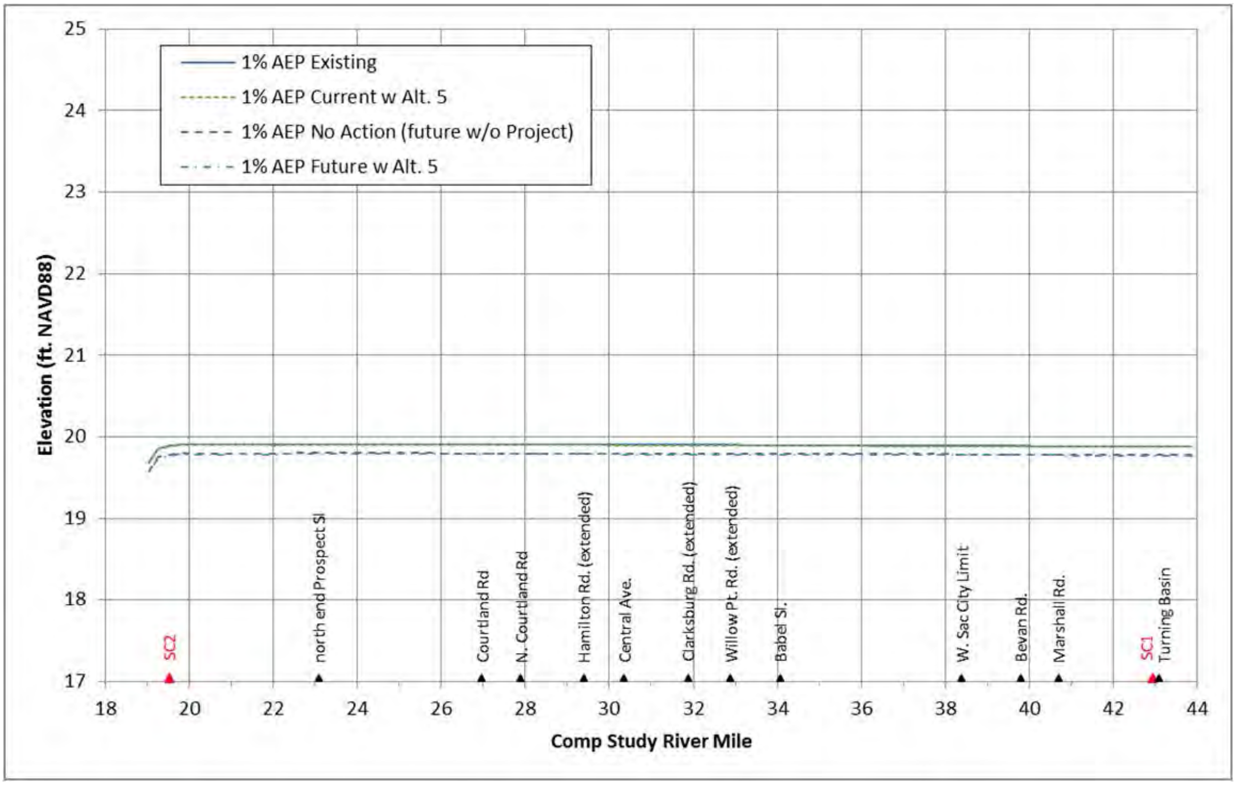


Figure 109. Maximum Water Surface Elevations; Deep Water Ship Channel; Alternative 5; 1% AEP

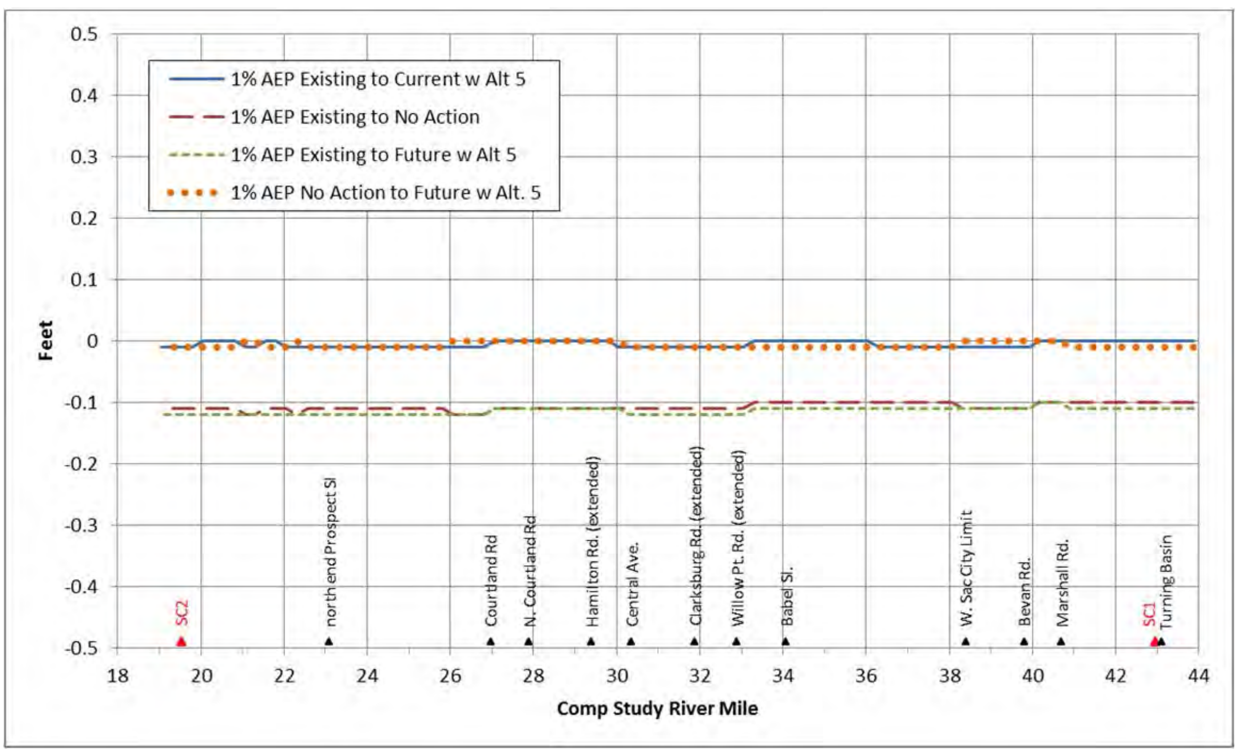


Figure 110. Maximum Water Surface Elevation Impacts; Deep Water Ship Channel; Alternative 5; 1% AEP

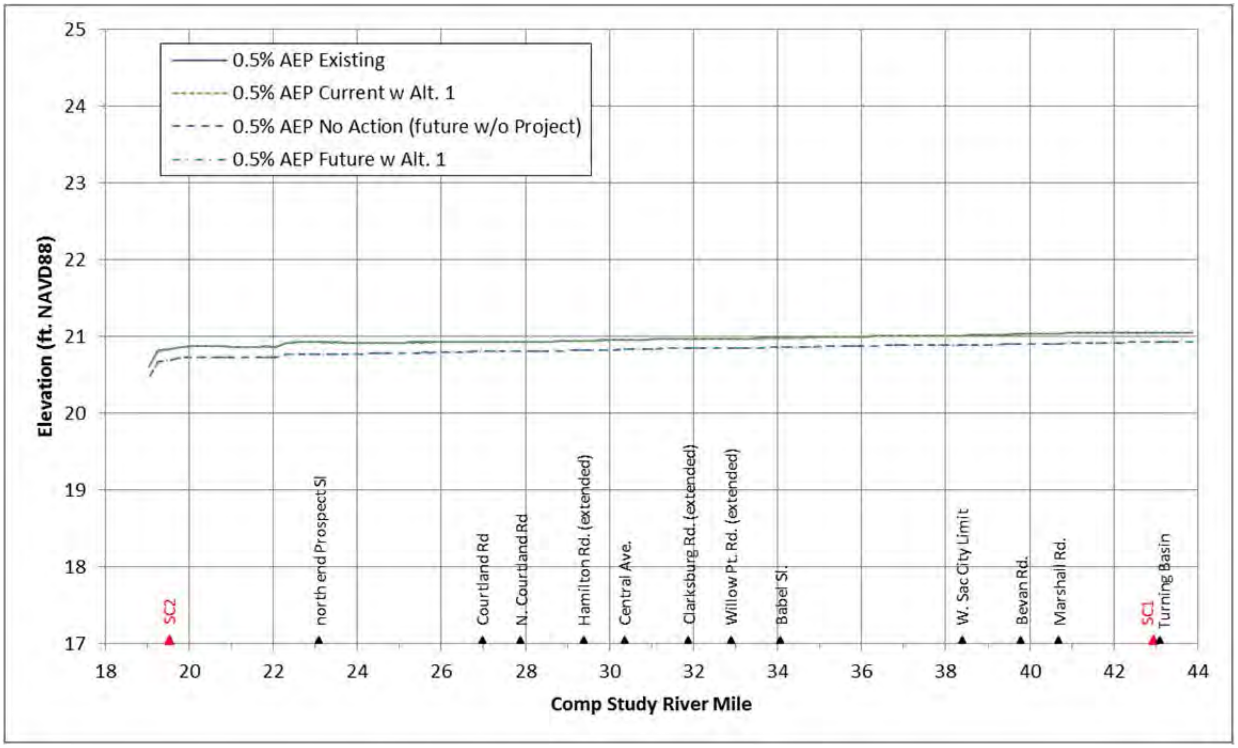


Figure 111. Maximum Water Surface Elevations; Deep Water Ship Channel; Alternative 1; 0.5% AEP

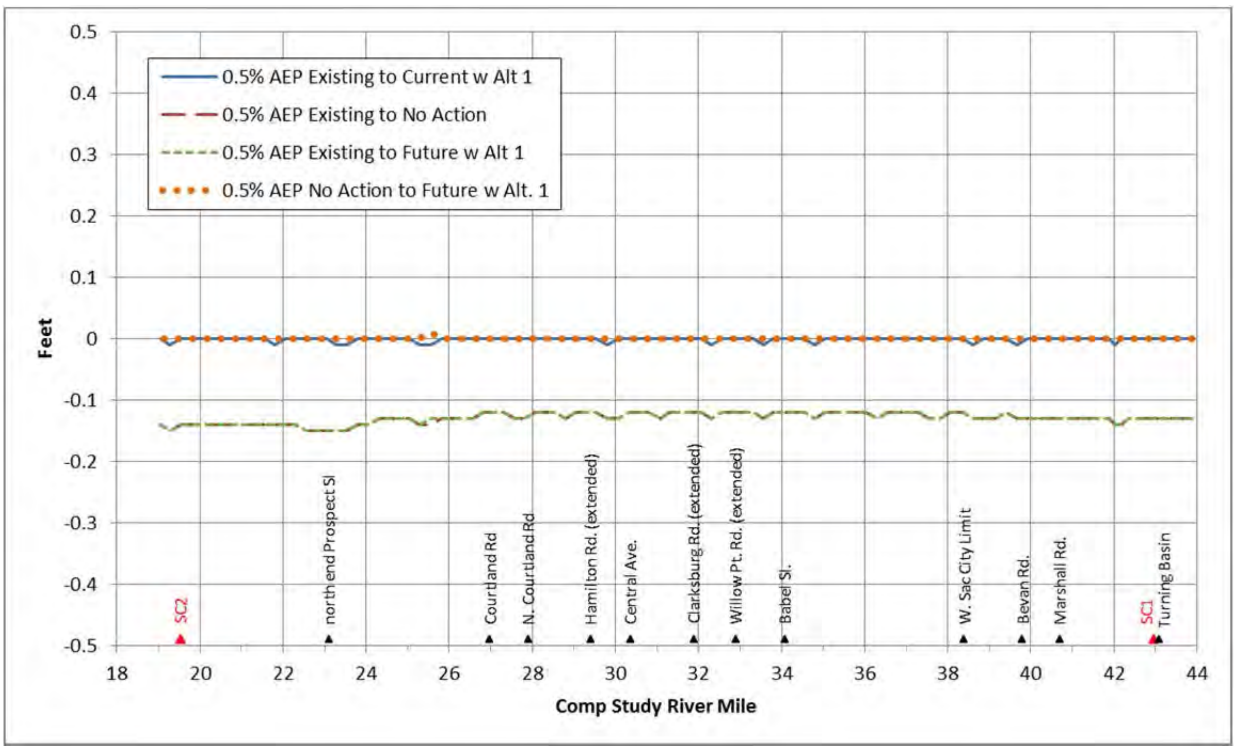


Figure 112. Maximum Water Surface Elevation Impacts; Deep Water Ship Channel; Alternative 1; 0.5% AEP

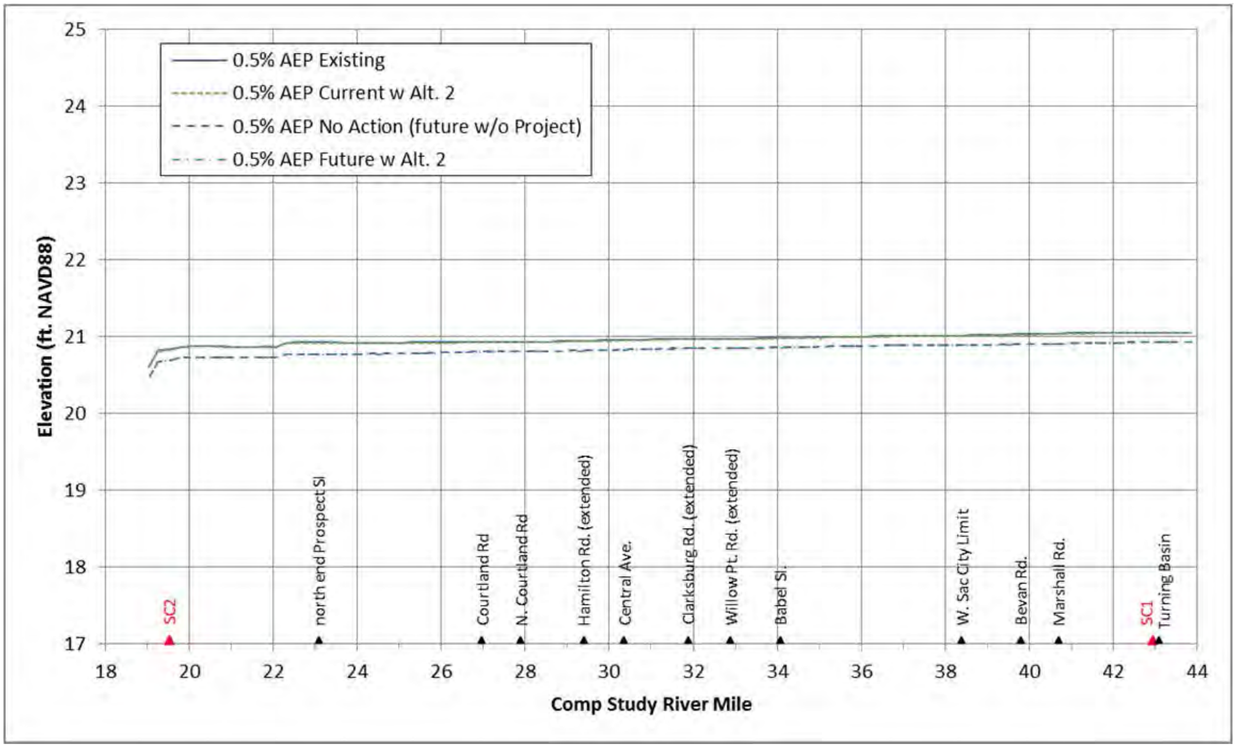


Figure 113. Maximum Water Surface Elevations; Deep Water Ship Channel; Alternative 2; 0.5% AEP

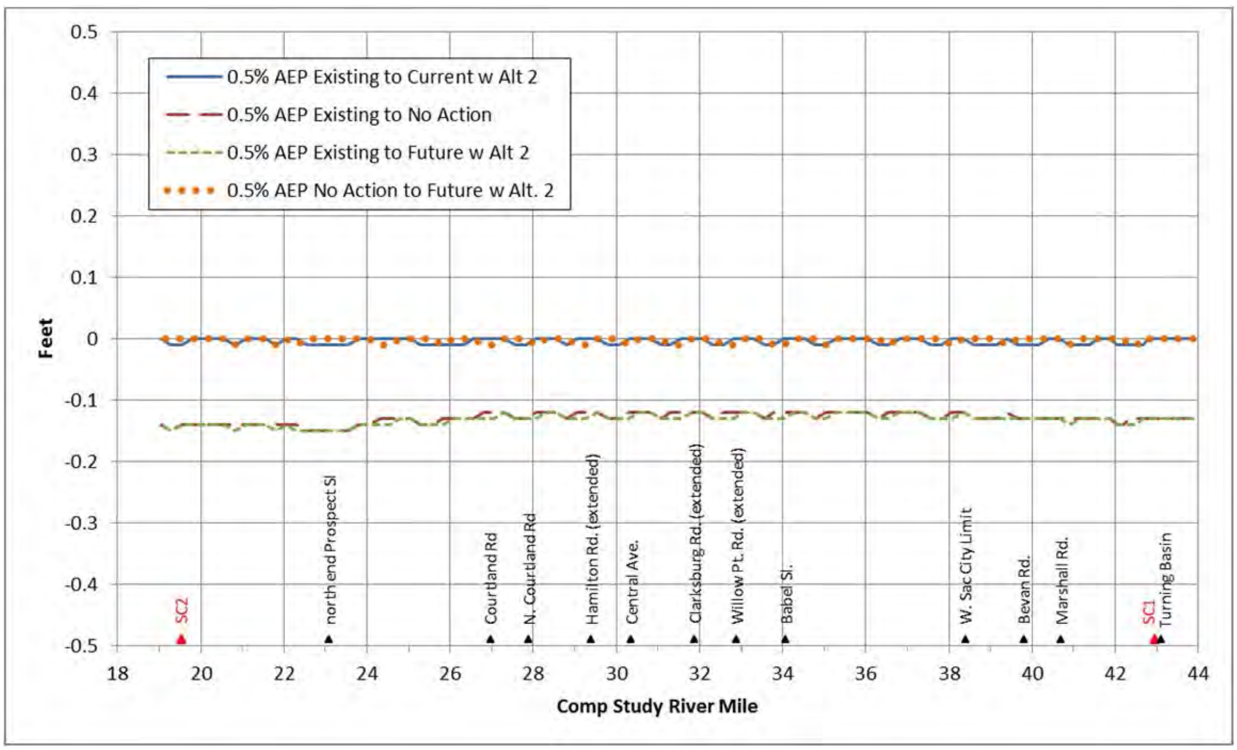


Figure 114. Maximum Water Surface Elevation Impacts; Deep Water Ship Channel; Alternative 2; 0.5% AEP

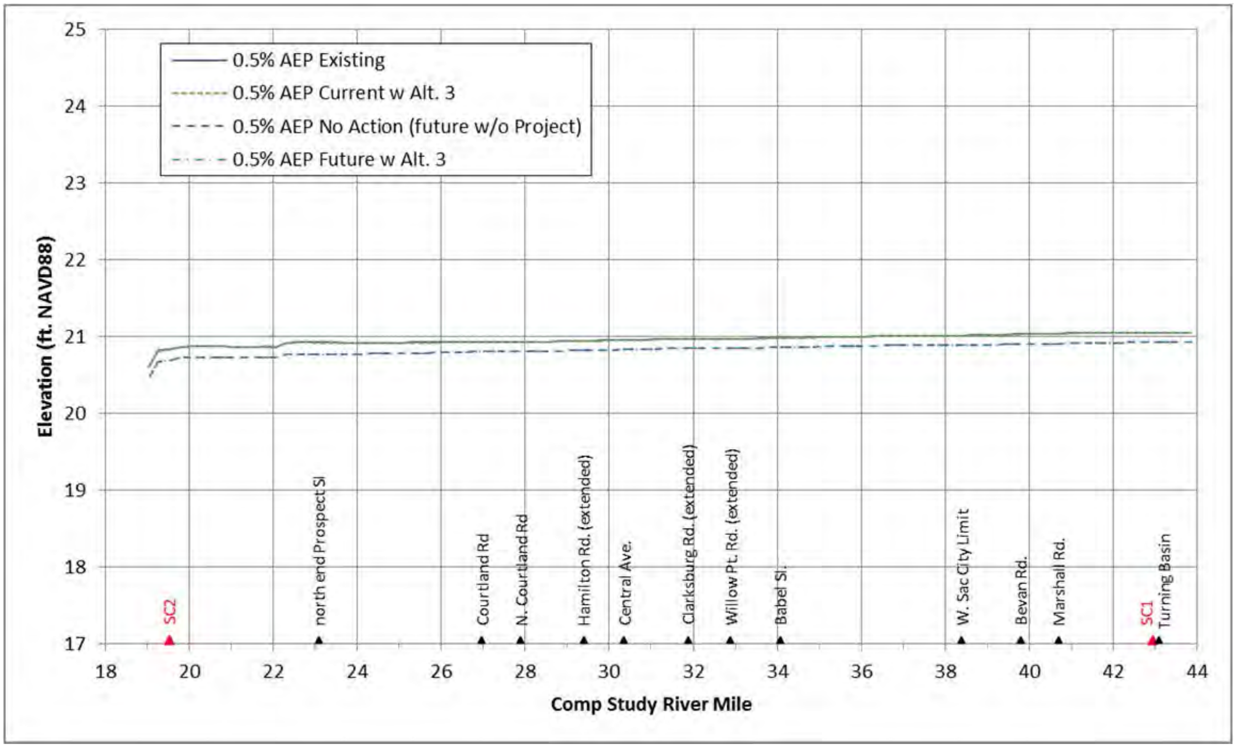


Figure 115. Maximum Water Surface Elevations; Deep Water Ship Channel; Alternative 3; 0.5% AEP

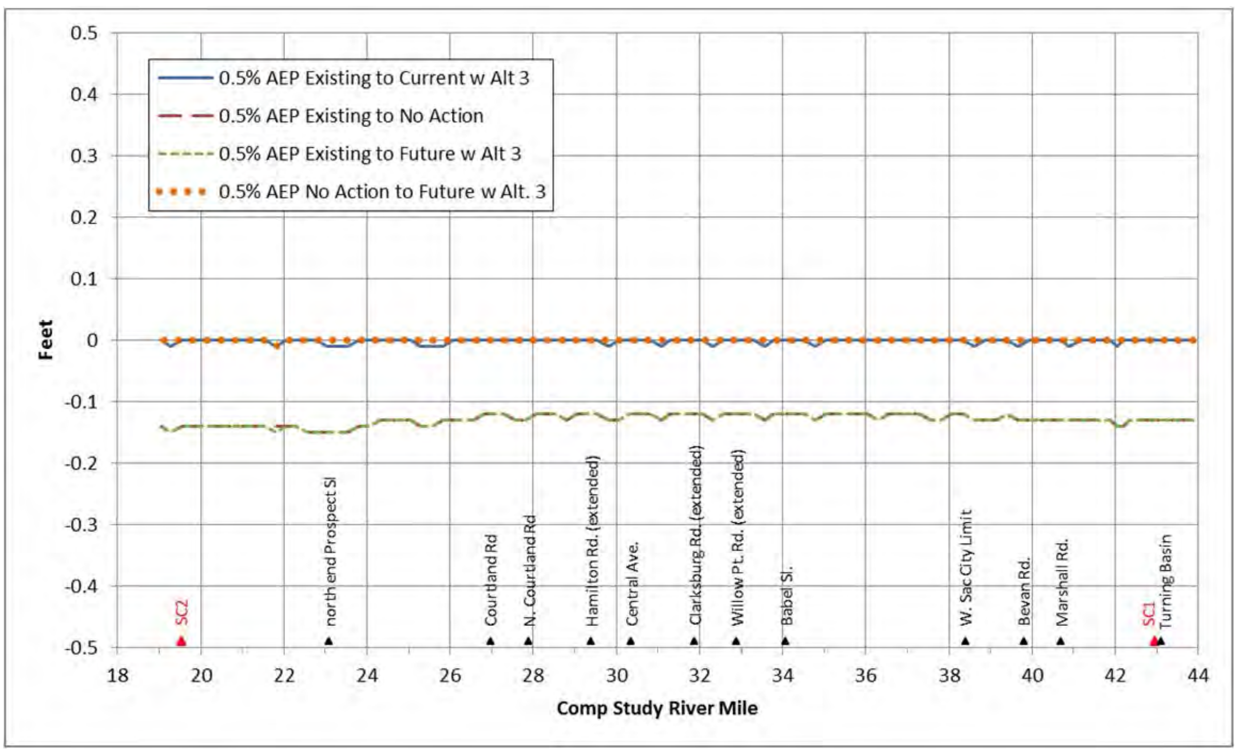


Figure 116. Maximum Water Surface Elevation Impacts; Deep Water Ship Channel; Alternative 3; 0.5% AEP

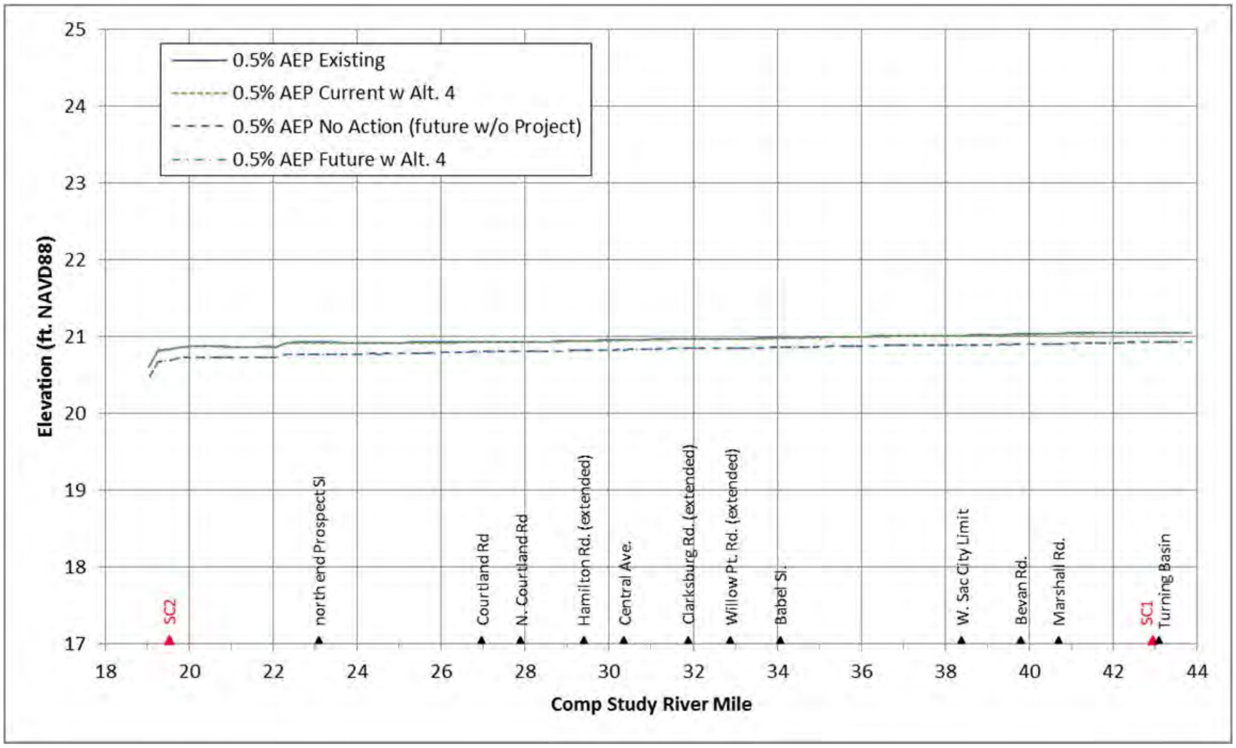


Figure 117. Maximum Water Surface Elevations; Deep Water Ship Channel; Alternative 4; 0.5% AEP

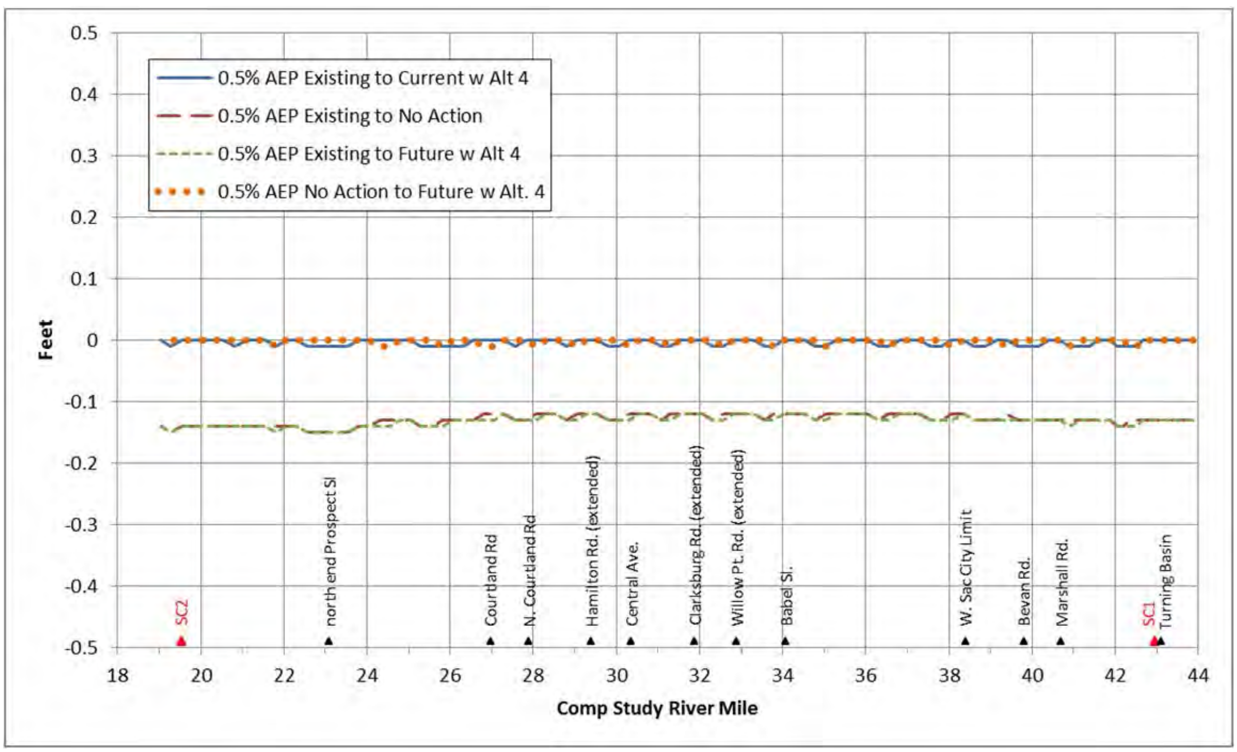


Figure 118. Maximum Water Surface Elevation Impacts; Deep Water Ship Channel; Alternative 4; 0.5% AEP

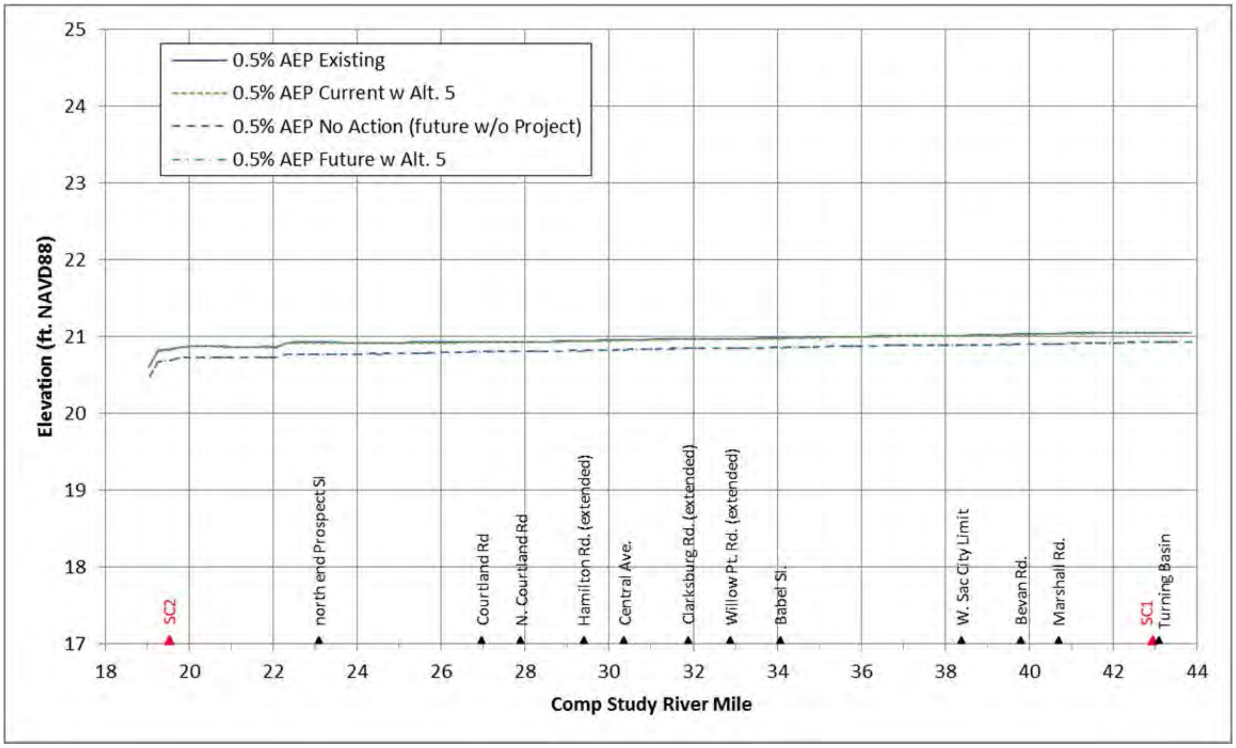


Figure 119. Maximum Water Surface Elevations; Deep Water Ship Channel; Alternative 5; 0.5% AEP

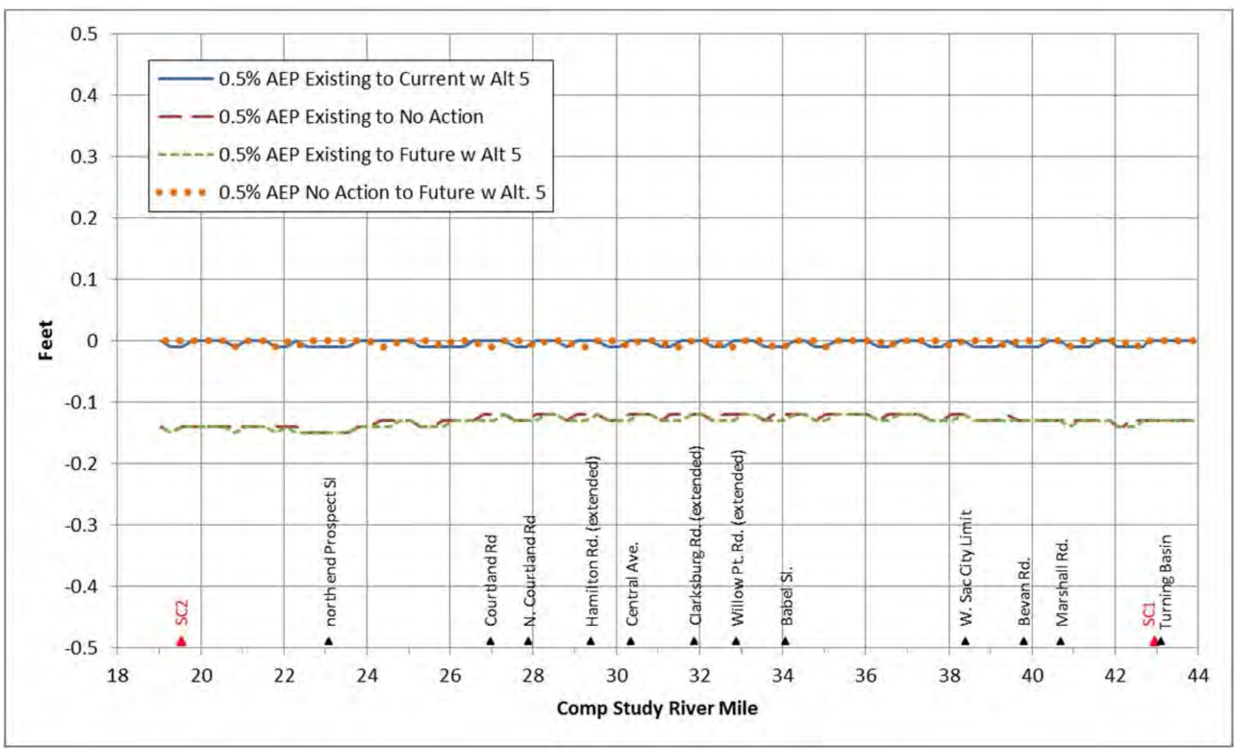


Figure 120. Maximum Water Surface Elevation Impacts; Deep Water Ship Channel; Alternative 5; 0.5% AEP

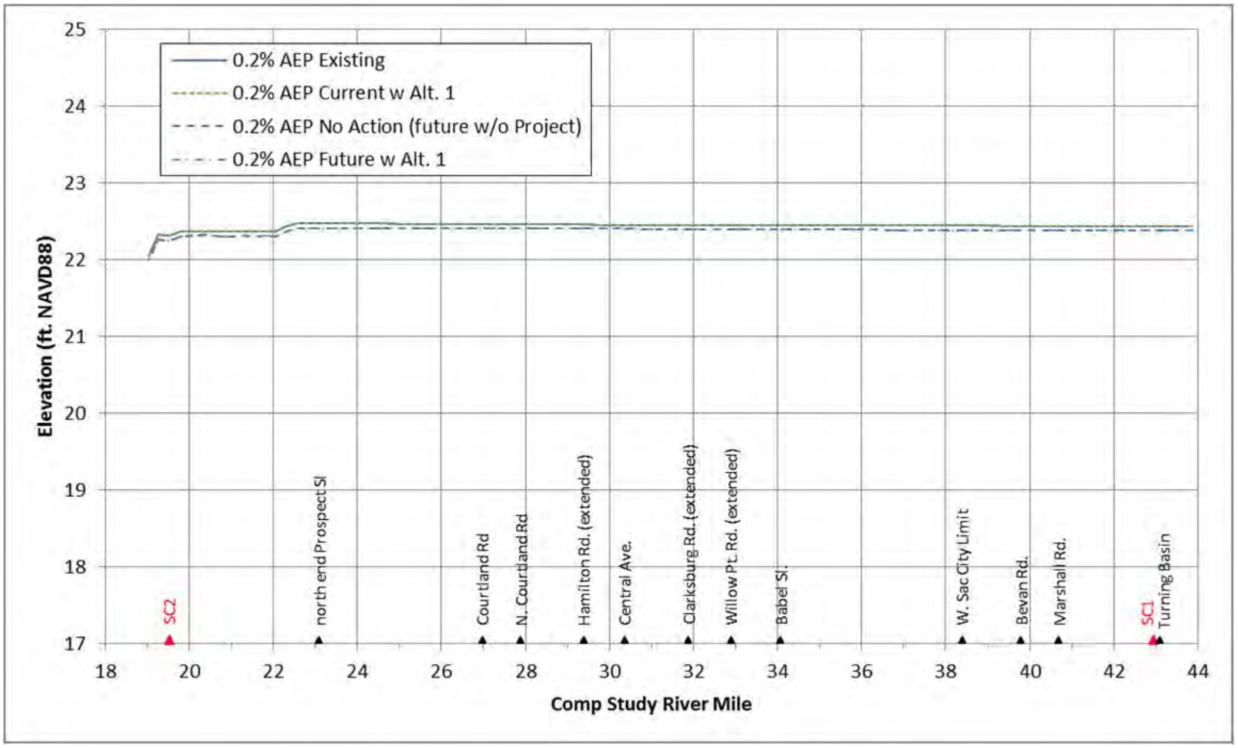


Figure 121. Maximum Water Surface Elevations; Deep Water Ship Channel; Alternative 1; 0.2% AEP

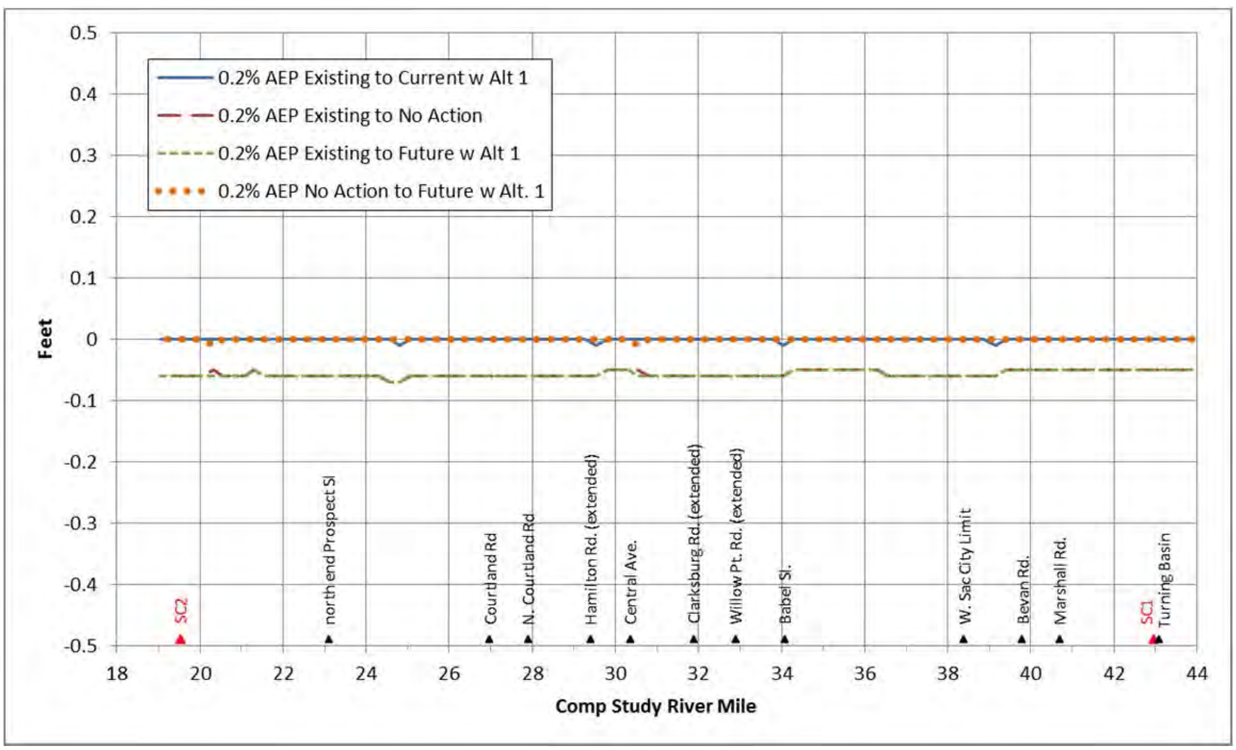


Figure 122. Maximum Water Surface Elevation Impacts; Deep Water Ship Channel; Alternative 1; 0.2% AEP

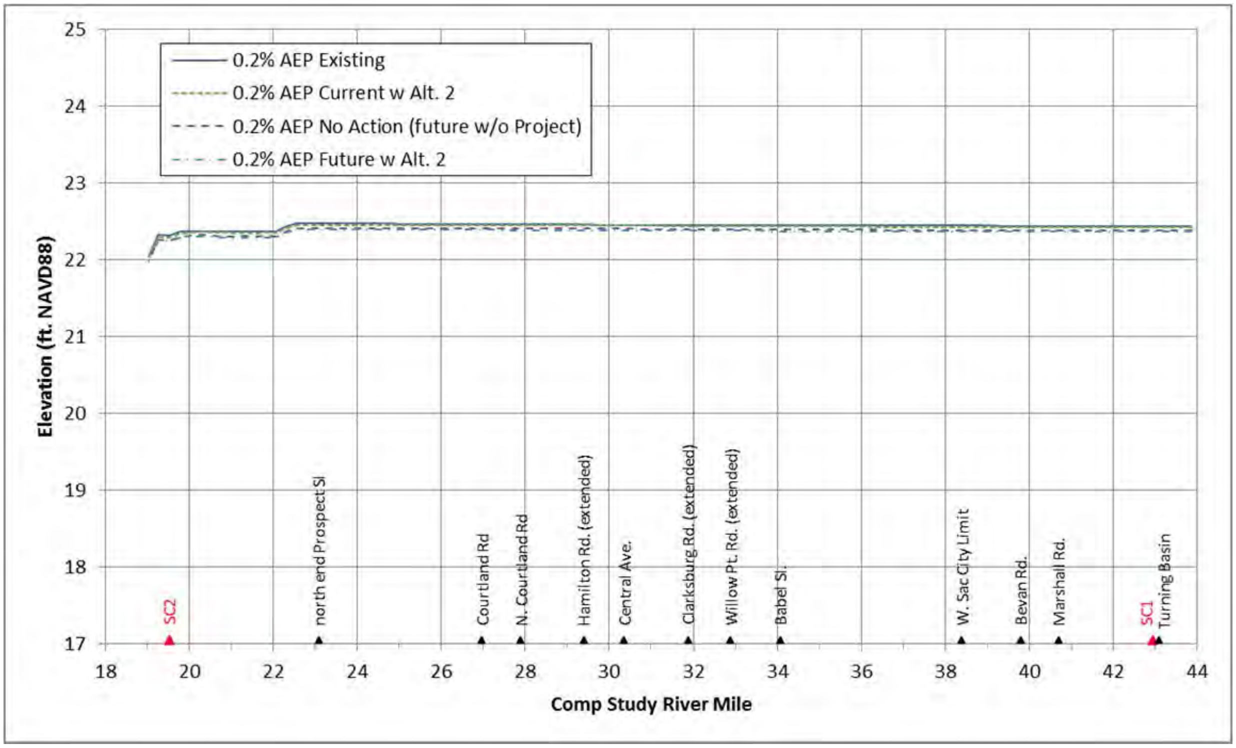


Figure 123. Maximum Water Surface Elevations; Deep Water Ship Channel; Alternative 2; 0.2% AEP

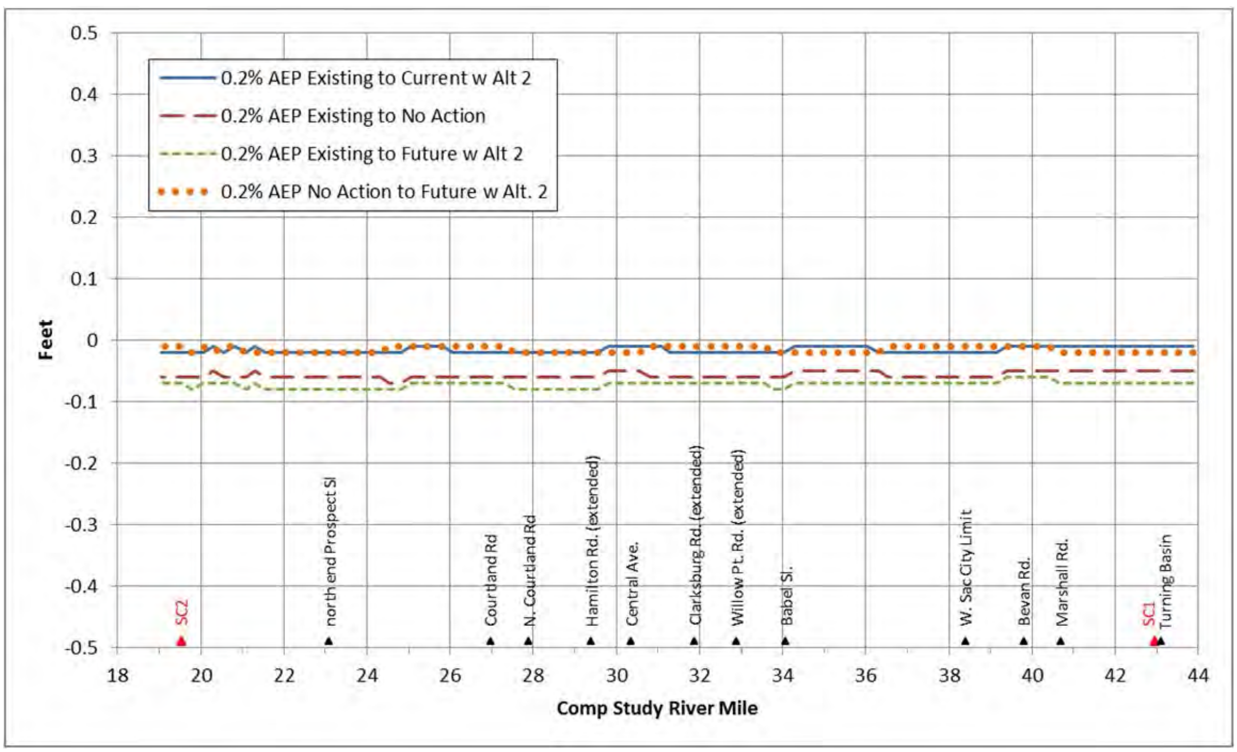


Figure 124. Maximum Water Surface Elevation Impacts; Deep Water Ship Channel; Alternative 2; 0.2% AEP

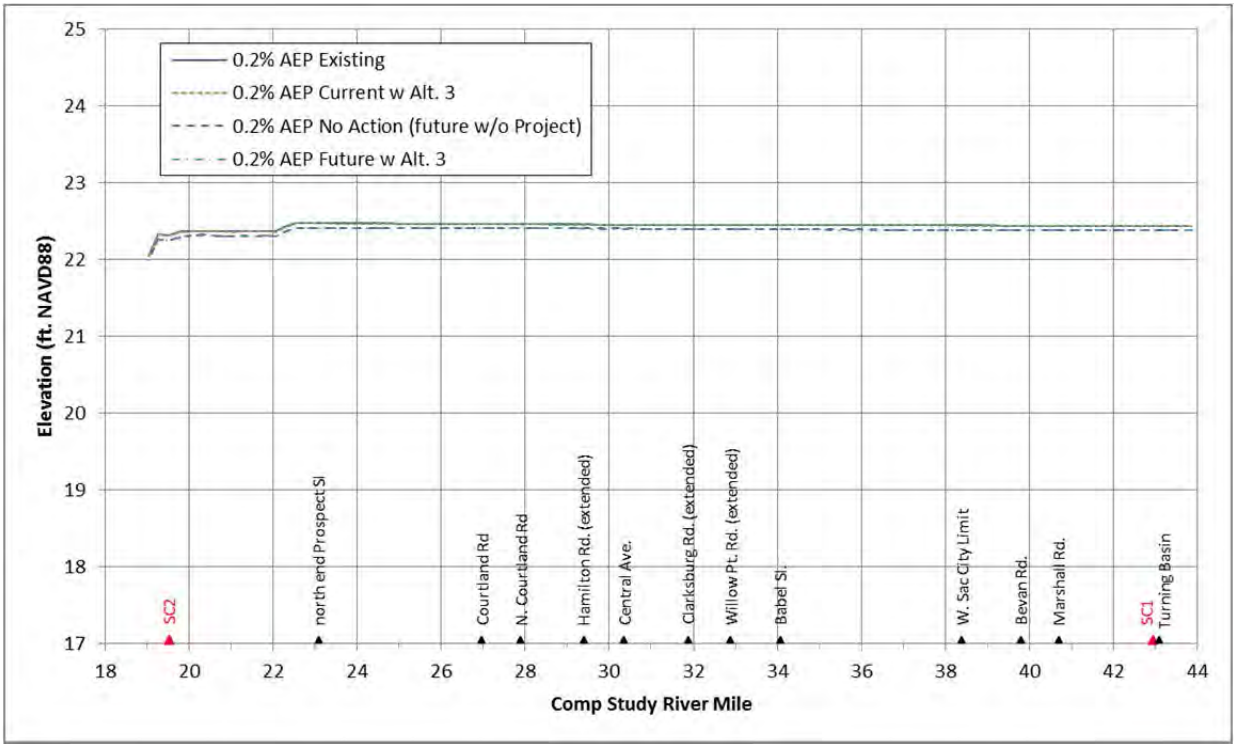


Figure 125. Maximum Water Surface Elevations; Deep Water Ship Channel; Alternative 3; 0.2% AEP

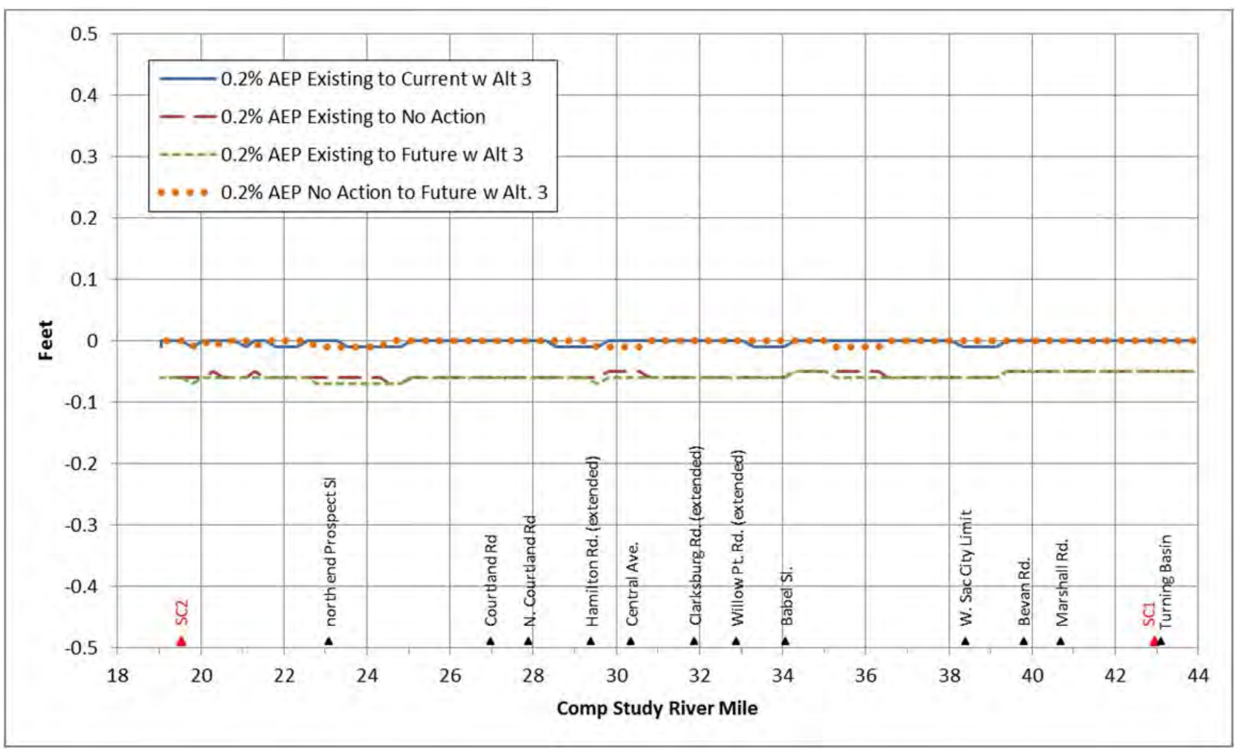


Figure 126. Maximum Water Surface Elevation Impacts; Deep Water Ship Channel; Alternative 3; 0.2% AEP

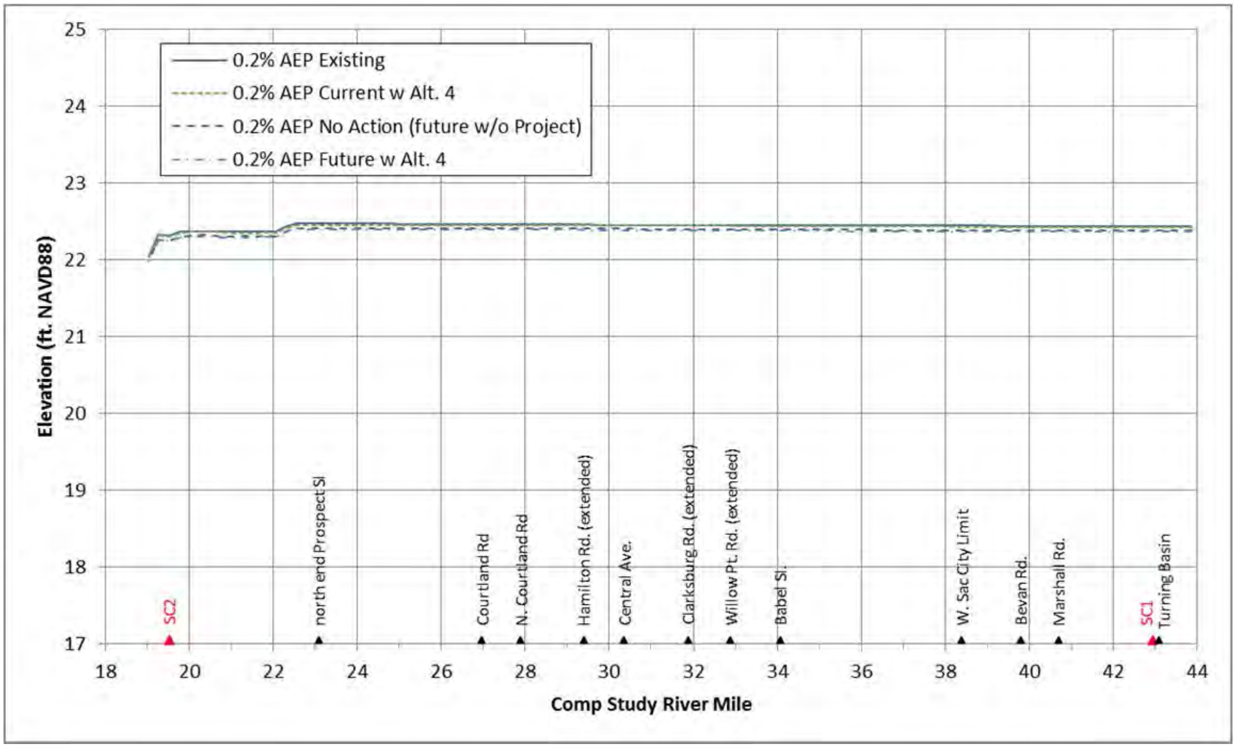


Figure 127. Maximum Water Surface Elevations; Deep Water Ship Channel; Alternative 4; 0.2% AEP

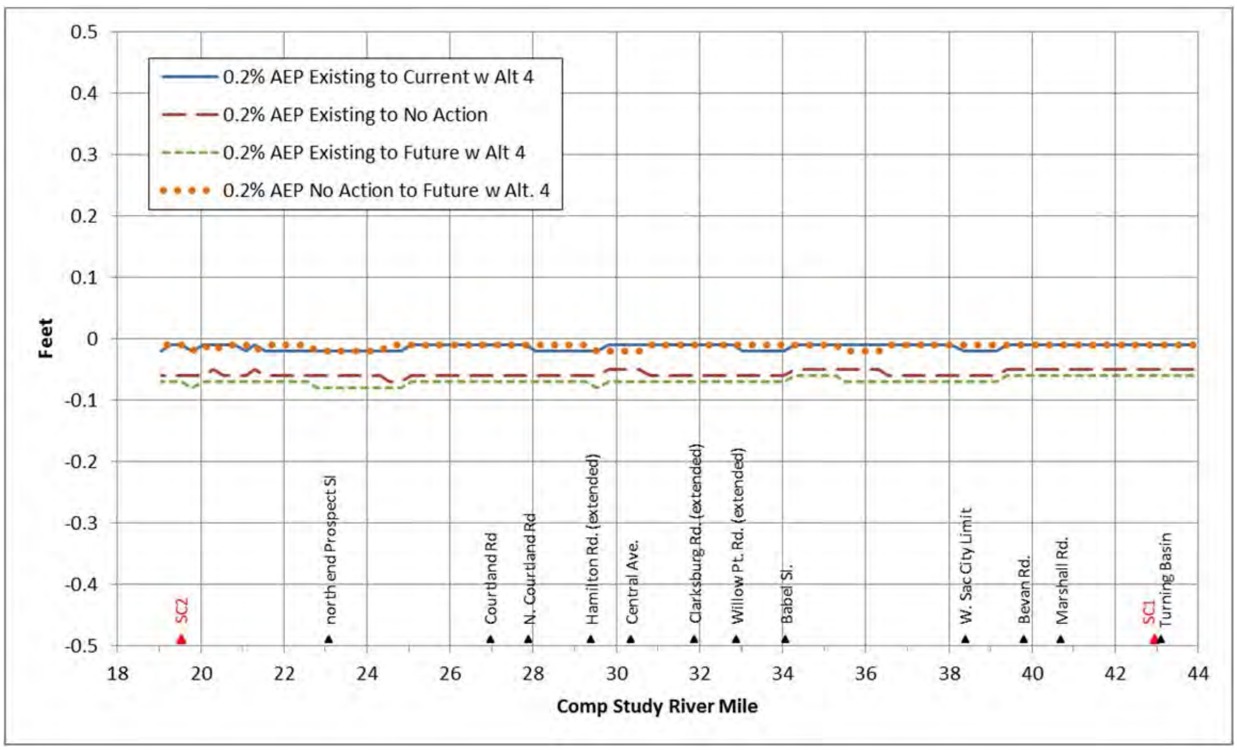


Figure 128. Maximum Water Surface Elevation Impacts; Deep Water Ship Channel; Alternative 4; 0.2% AEP

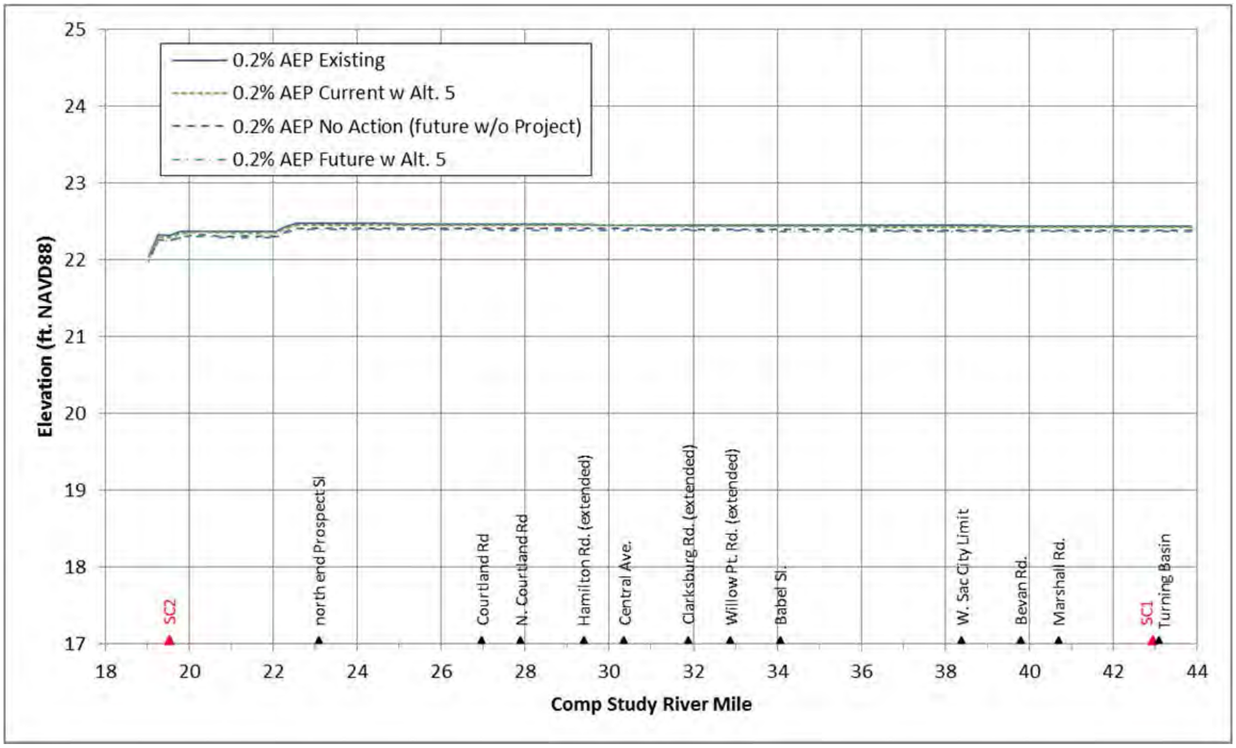


Figure 129. Maximum Water Surface Elevations; Deep Water Ship Channel; Alternative 5; 0.2% AEP

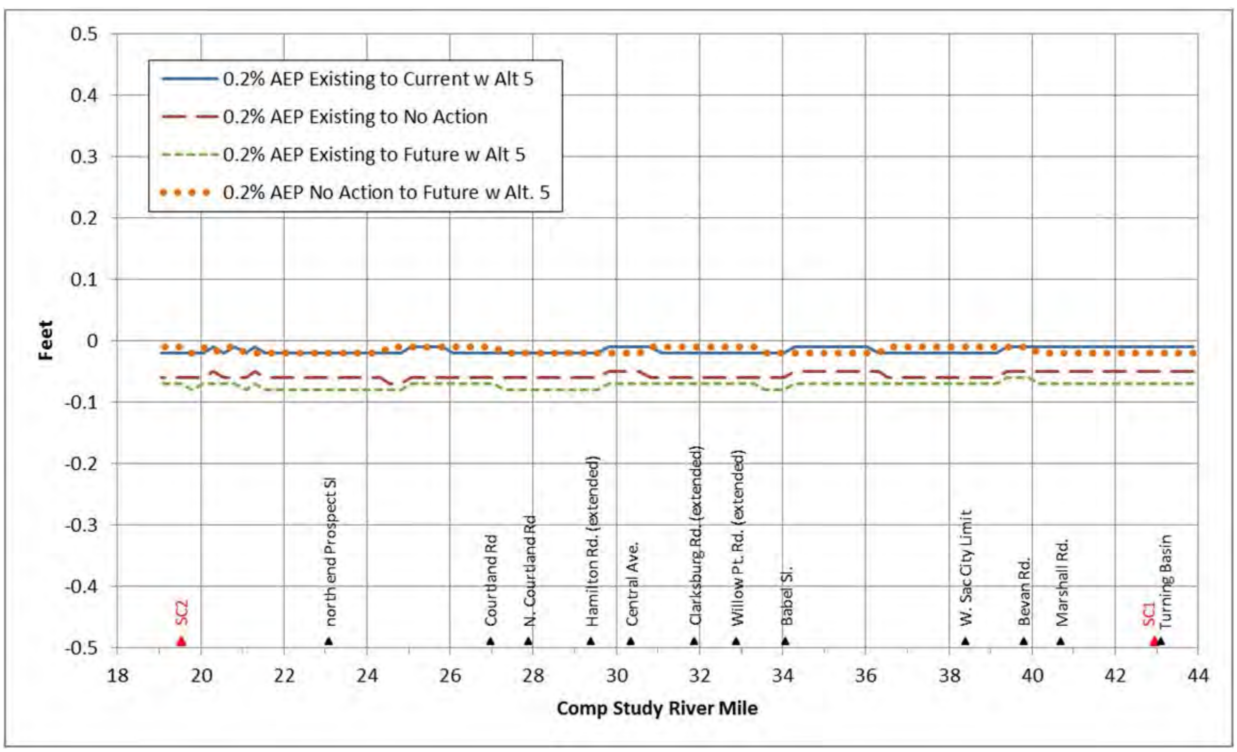


Figure 130. Maximum Water Surface Elevation Impacts; Deep Water Ship Channel; Alternative 5; 0.2% AEP

APPENDIX A

USACE Memorandum for Record for
Sacramento River Basin HEC-RAS Model Release 4

CESPK-ED-HD 4 June 2012

MEMORANDUM FOR RECORD

SUBJECT: Sacramento River Basin HEC-RAS Model Release 4 (NAVD'88 Version)

1. REFERENCES

a. Memorandum for Record, CESPK-ED-HD. Subject: Sacramento River Basin HEC-RAS Model Release 3, 1 February 2011.
b. Memorandum for Record, CESPK-ED-HD. Subject: Sacramento River Basin HEC-RAS Model Release 2, 13 December 2010.
c. Memorandum for Record, CESPK-ED-DH. Subject: Sacramento River Basin HEC-RAS Model Release, 24 March 2009.
d. Hydrologic Engineering Center, HEC-RAS River Analysis Program Version 4.0.0, March 2008.
e. Hydrologic Engineering Center, HEC-RAS User's Manual Version 4.0.0, March 2008.

2. PURPOSE. This memo documents a fourth release of the Sacramento Basin HEC-RAS model. Previous releases of the model (see References 1.b, 1.c) were based on the NGVD'29 vertical datum and the latest model geometry is based on NAVD'88 vertical datum. The release of this model only includes runs for the synthetic hydrology (0.5, 0.1 0.04, 0.02, 0.01, 0.005, 0.002 annual exceedance probability) and does not include detailed documentation. Documentation of model construct and results will be available in the future.

The intent with the release of this model at this time is to allow for interested parties to make comments on the model construct and view results that were already handed off to the American River Common Features Project Delivery Team for use in their alternatives analysis. Along with this memo it will also be available for interested parties to review and provide comment on. This updated model has yet to undergo agency technical review (ATR) completed by another USACE district and this is planned to take place in the future. Based on these reviews, the USACE will generate a final model that will be released to any interested parties upon request.

The USACE makes no warranty regarding use of this model. The intent of this model release is to allow interested parties the opportunity to review the model and provide comment as a means to improve the model.

3. MODEL DEVELOPMENT. An extensive amount of effort was expended to convert the model from NGVD'29 to NAVD'88 vertical datum. Due to survey control stability issues in the topographic data collected by the USACE for the Comprehensive Study and the Sacramento River Bank Protection Project an appropriate conversion factor had to be determined to convert the data from NGVD'29 to NAVD'88. Model geometry in the HEC-RAS model was based upon this topographic data and therefore also had to be adjusted accordingly. In addition to the conversion of the model geometry, stream gage data and high water marks were also converted to NAVD'88. In some cases this required surveys of existing gages. Documentation on

CESPK-ED-HD 4 June 2012

MEMORANDUM FOR RECORD

SUBJECT: Sacramento River Basin HEC-RAS Model Release 4 (NAVD'88 Version)

conversion of the topographic data, stream gage data, and highwater mark data is forthcoming and will be provided in the near future.

The extents of the HEC-RAS model are the same as in previous releases of the model. The model was primarily calibrated using the 2006 flood event because it was a recent large event without levee failure. The 1997 event was also used during calibration. However, levee failures during the event created significant flow uncertainty in several reaches. Therefore, the 1997 event primarily served as model validation. The model geometry for the 2006 event is slightly different than the model geometry for the 1997 event. A pump station on the Natomas East Main Drain Canal (NEMDC) was added to the 2006 model geometry to reflect the addition of that pump station.

The geometry now includes updates to the system including the recently constructed setbacks on the Bear and Feather Rivers. Model runs were completed using HEC-RAS version 4.0.

4. SENSITIVITY ANALYSES. USACE recognizes some unresolved issues with the current state of the model and intends to conduct sensitivity analyses to gage how critical these issues are. Those include the sensitivity to higher or lower roughness values, the sensitivity to local inflows, and the sensitivity to flow splits at diversion weirs, in particular Tisdale Weir and the Fremont Weir. Any changes resulting from these analyses will be incorporated in the next release of the model.

5. For questions or comments on the model contact Ethan Thompson, P.E., (916) 557-7142, ethan.a.thompson@usace.army.mil or Jesse Schlunegger, P.E., (916) 557-6777, jesse.j.schlunegger@usace.army.mil.

Lea Adams, P.E.
Chief, Hydraulic Design Section
Sacramento District
U.S. Army Corps of Engineers

APPENDIX B

Setback Levee Cross-Section Plots

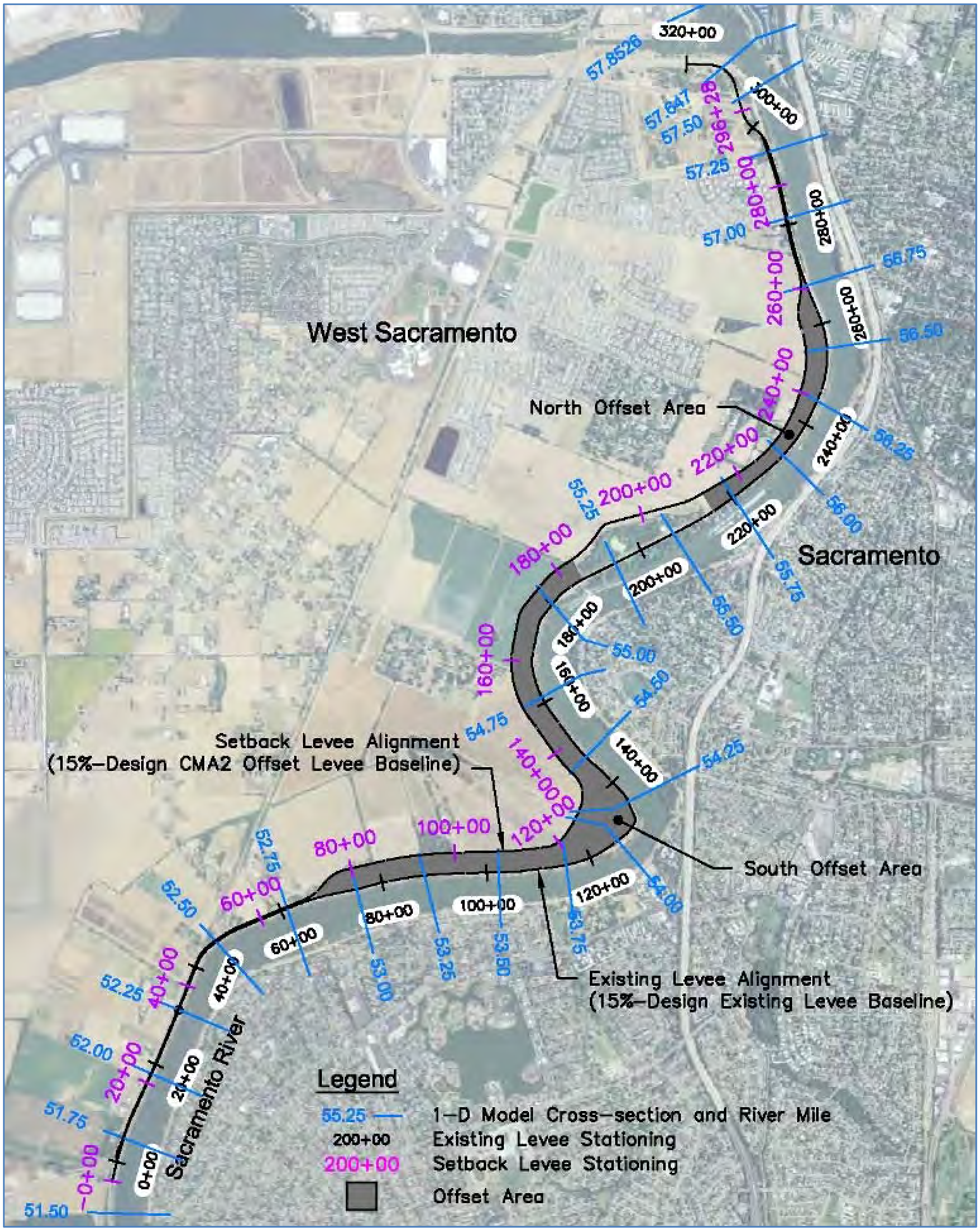


Figure B-1. Cross-section Locations

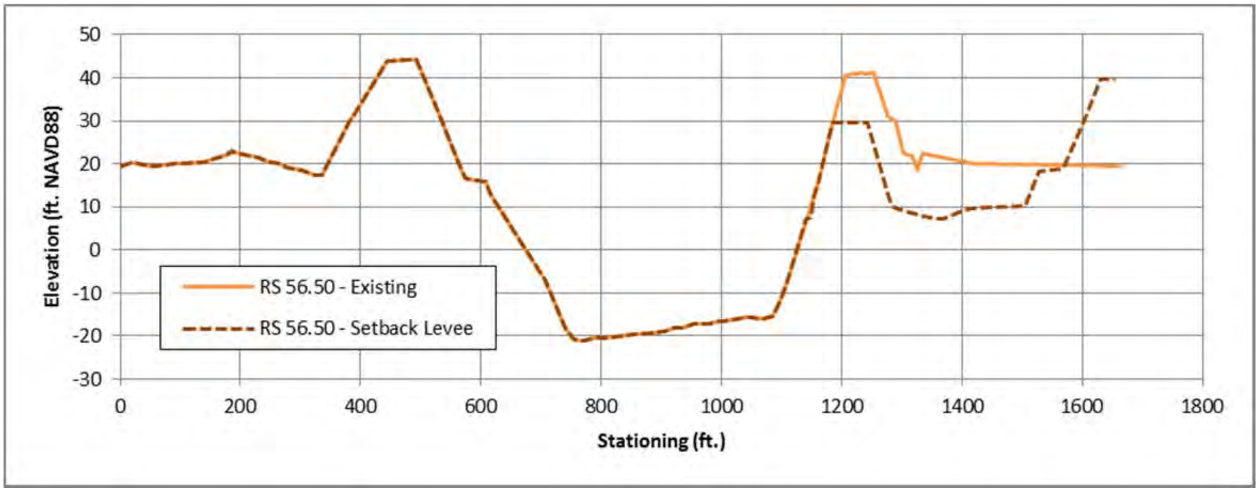


Figure B-2. Cross-section 56.50 (looking downstream)

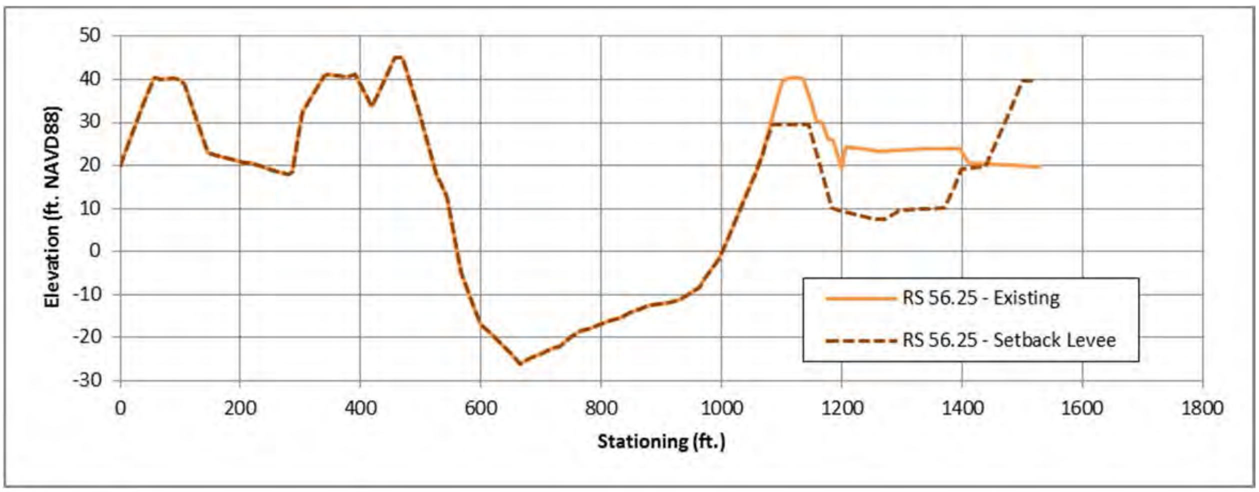


Figure B-3. Cross-section 56.25 (looking downstream)

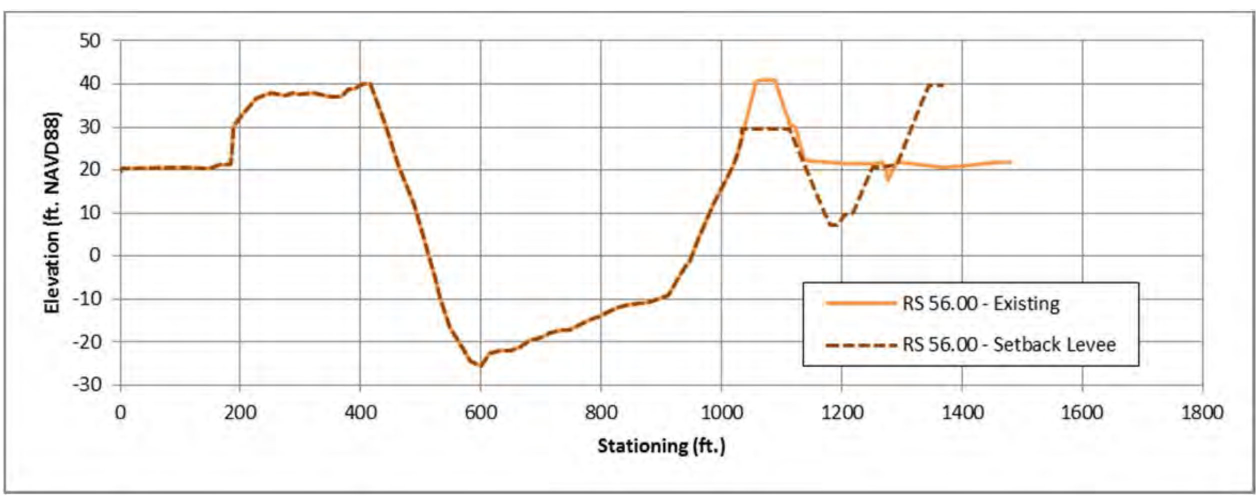


Figure B-4. Cross-section 56.00 (looking downstream)

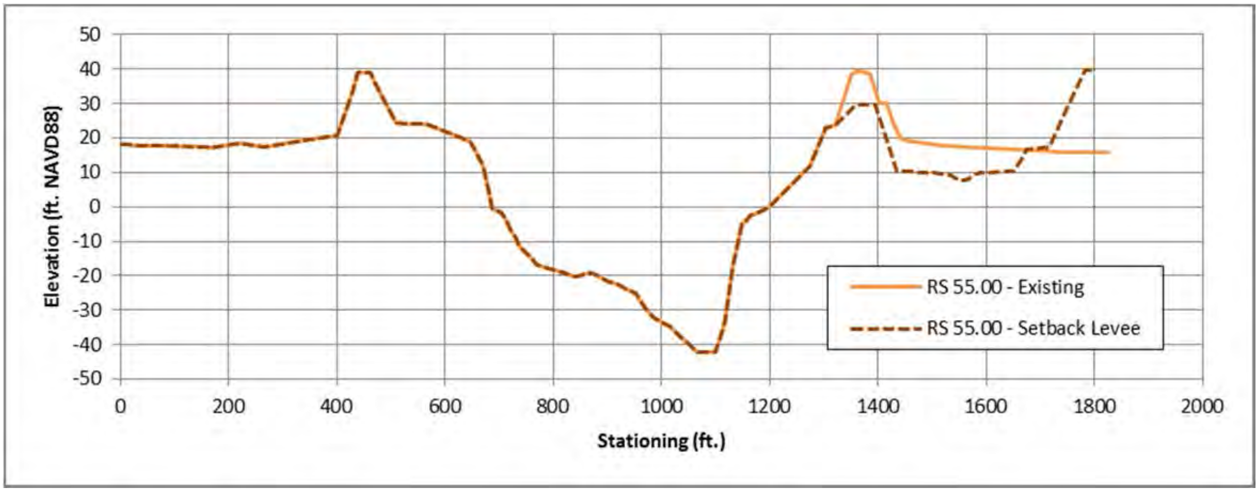


Figure B-5. Cross-section 55.00 (looking downstream)

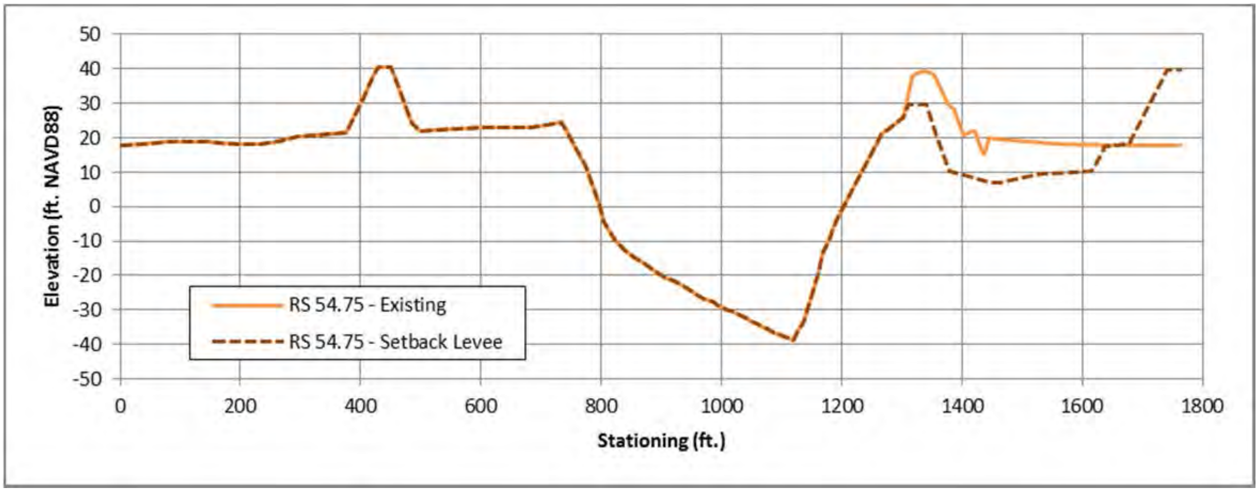


Figure B-6. Cross-section 54.75 (looking downstream)

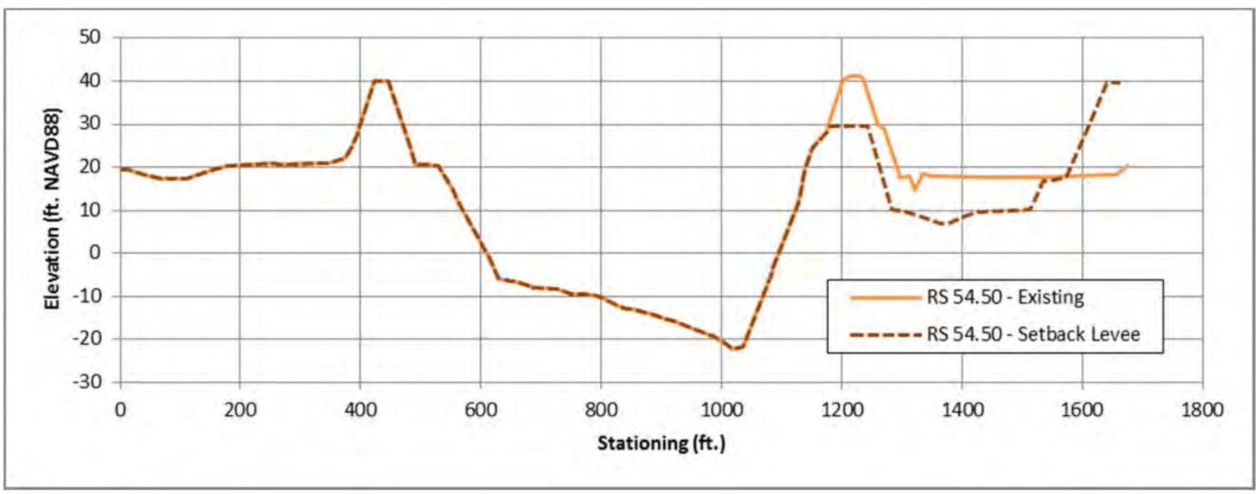


Figure B-7. Cross-section 54.50 (looking downstream)

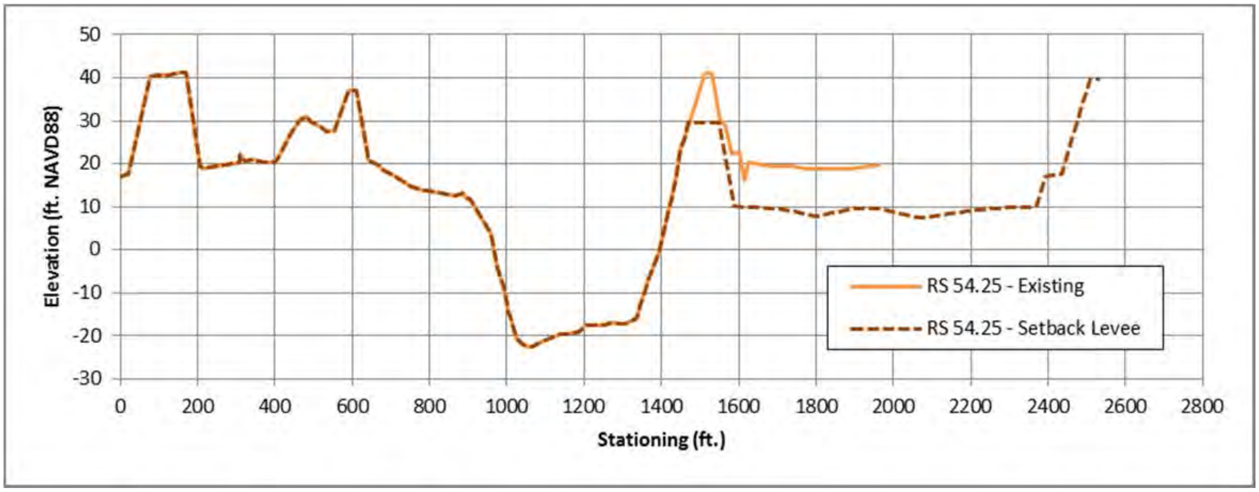


Figure B-8. Cross-section 54.25 (looking downstream)

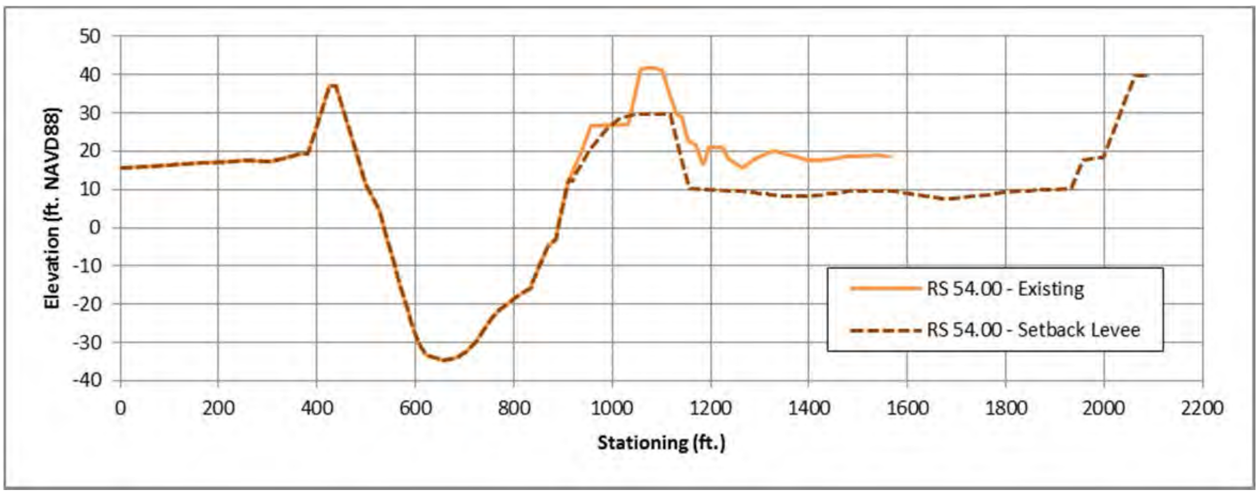


Figure B-9. Cross-section 54.00 (looking downstream)

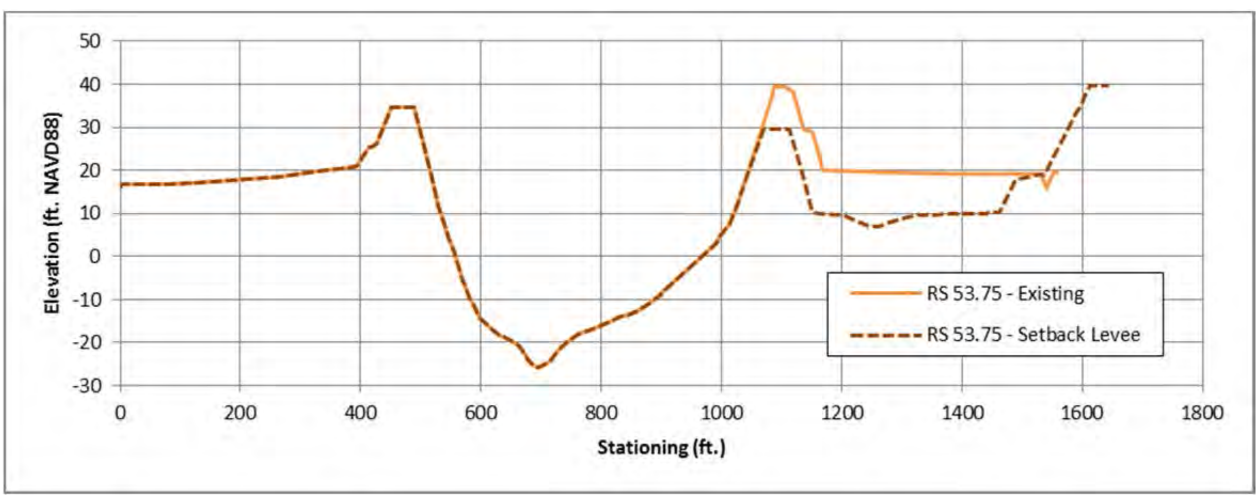


Figure B-10. Cross-section 53.75 (looking downstream)

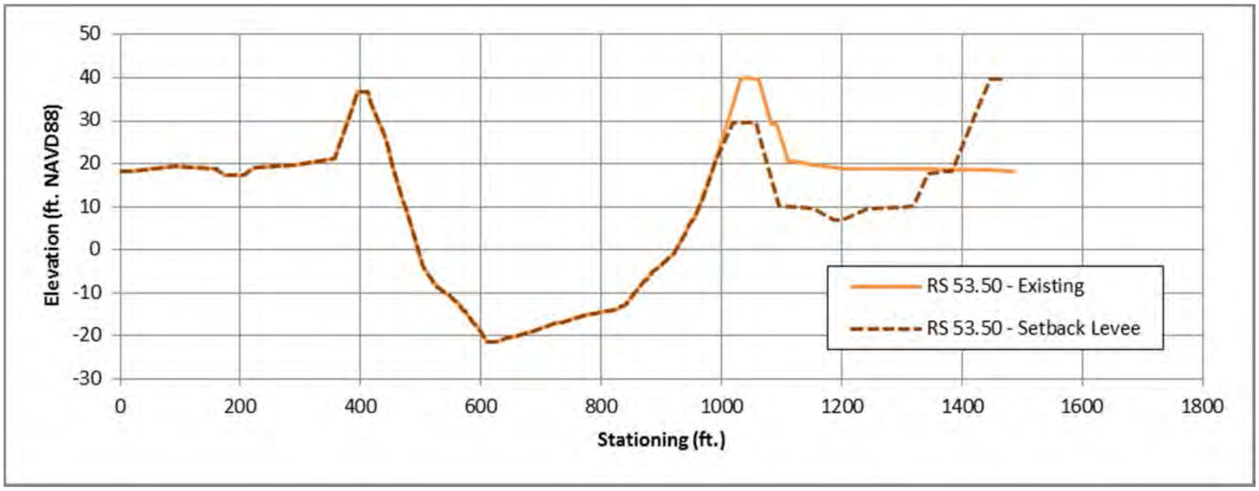


Figure B-11. Cross-section 53.50 (looking downstream)

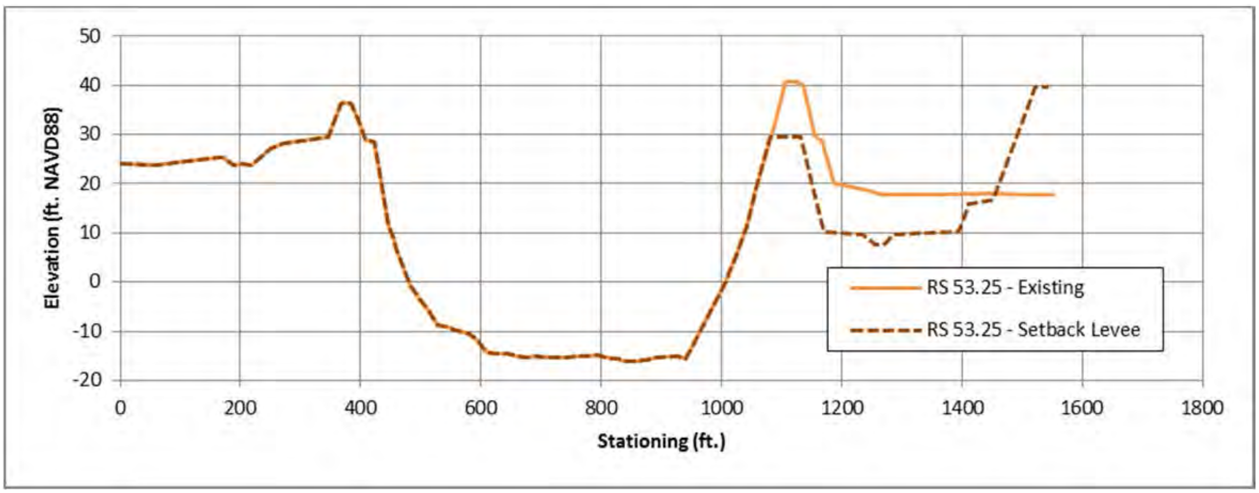


Exhibit B-12. Cross-section 53.25 (looking downstream)

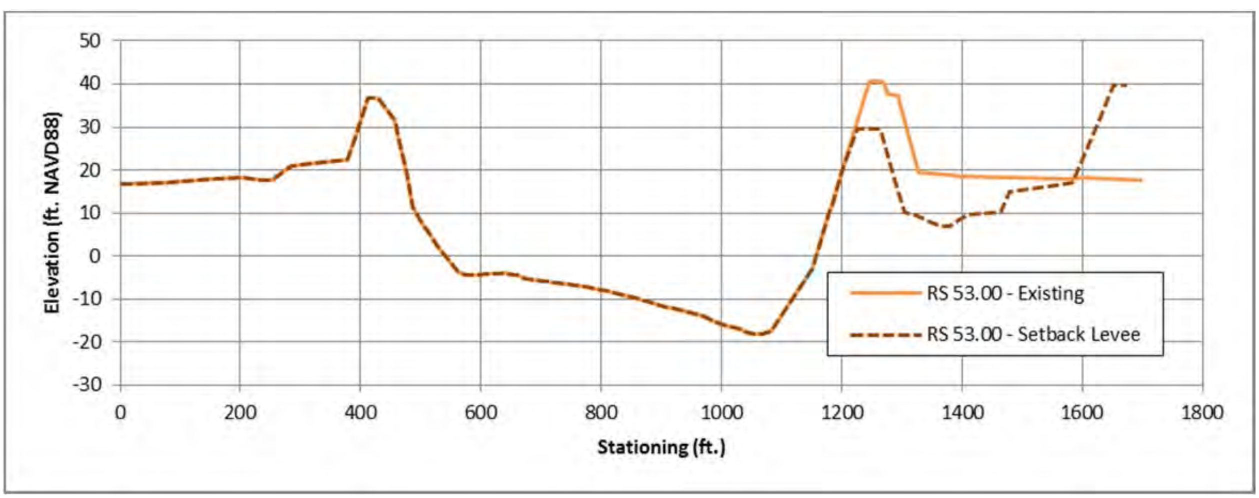


Exhibit B-13. Cross-section 53.00 (looking downstream)

APPENDIX C

Slope Flattening Cross-Section Plots

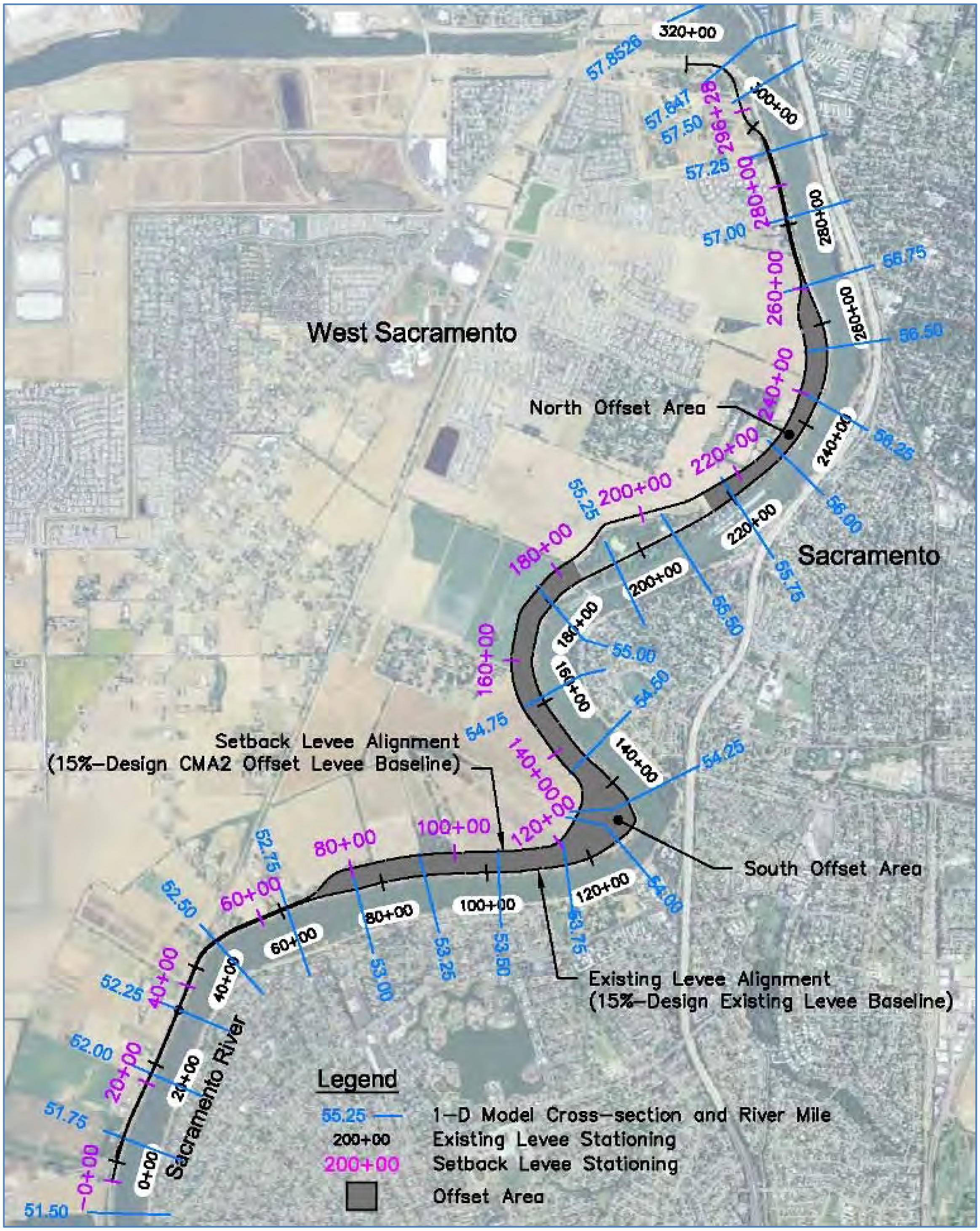


Figure C-1. Cross-section Locations

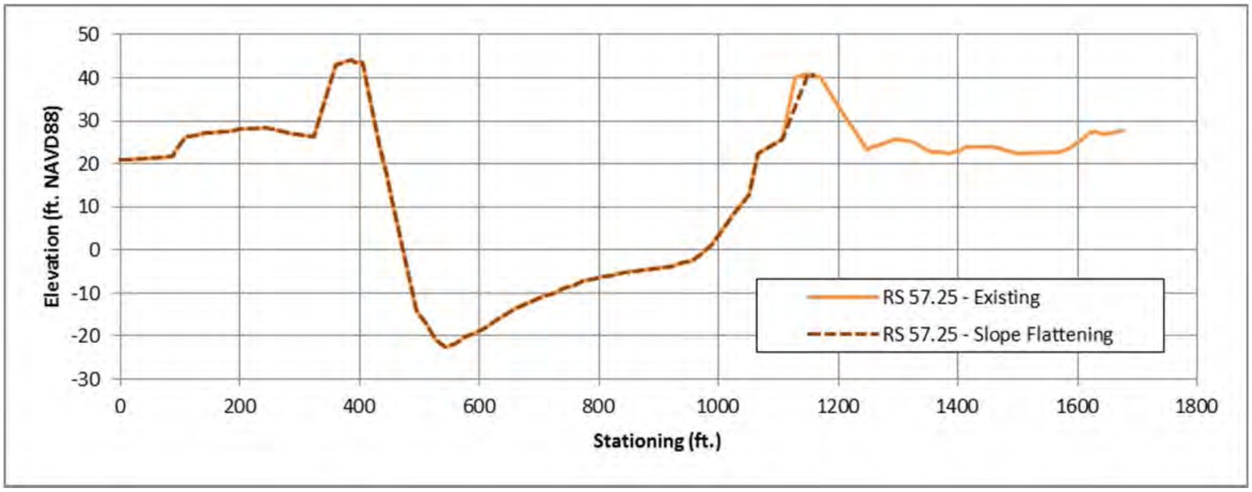


Figure C-2. Cross-section 57.25 (looking downstream)

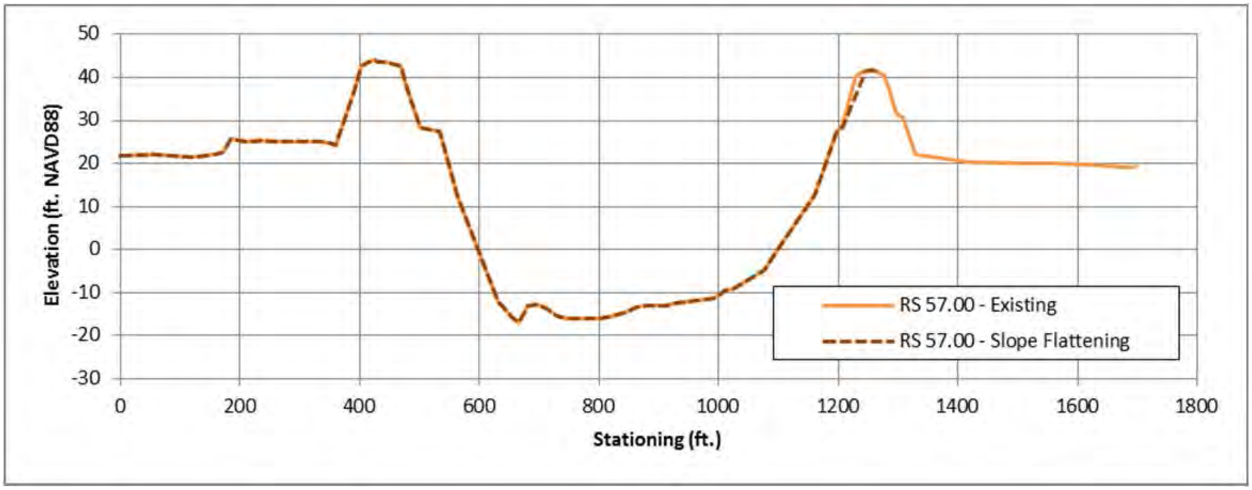


Figure C-3. Cross-section 57.00 (looking downstream)

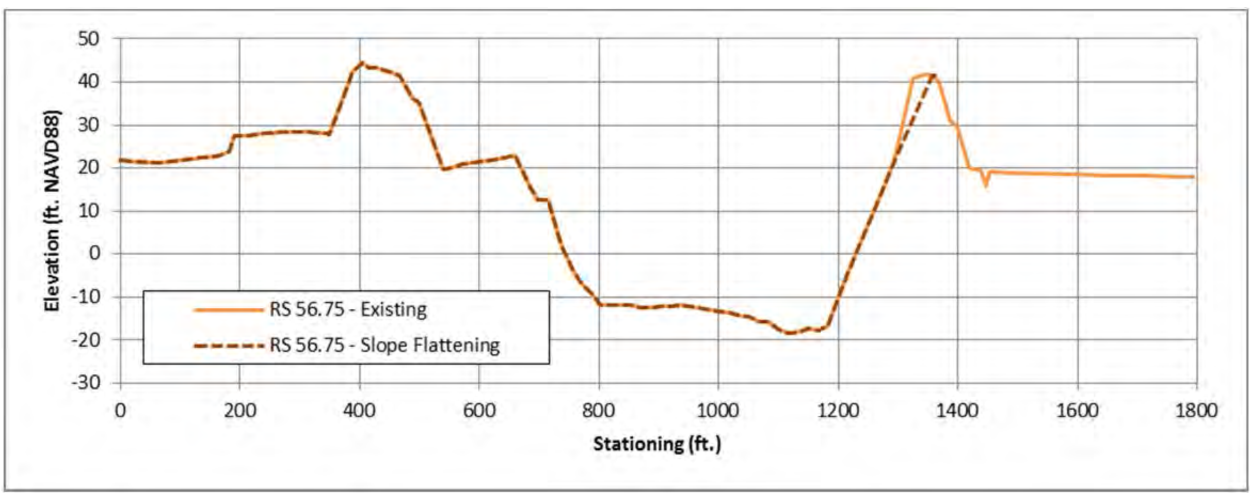


Figure C-4. Cross-section 56.75 (looking downstream)

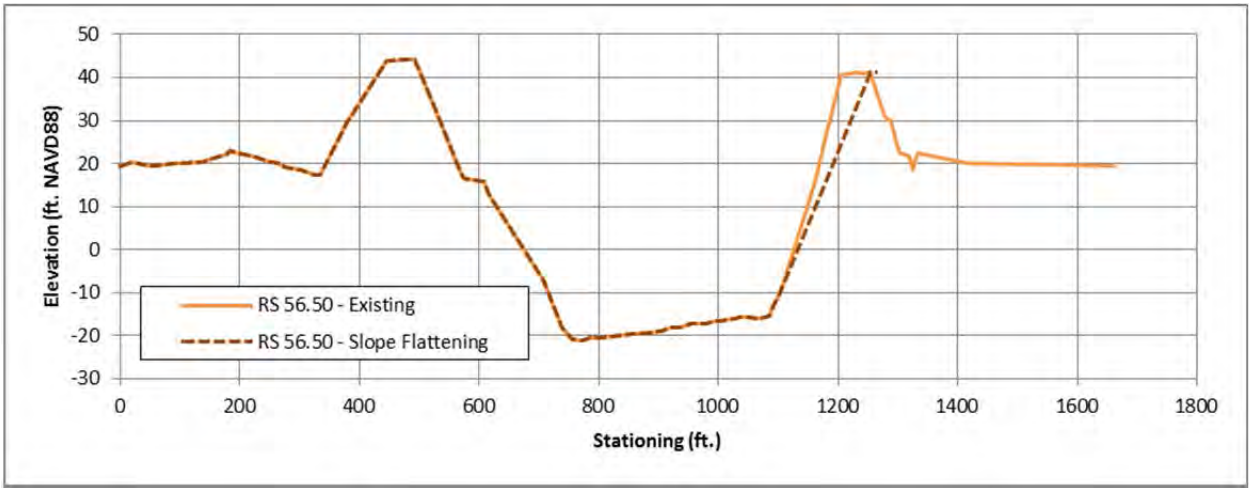


Figure C-5. Cross-section 56.50 (looking downstream)

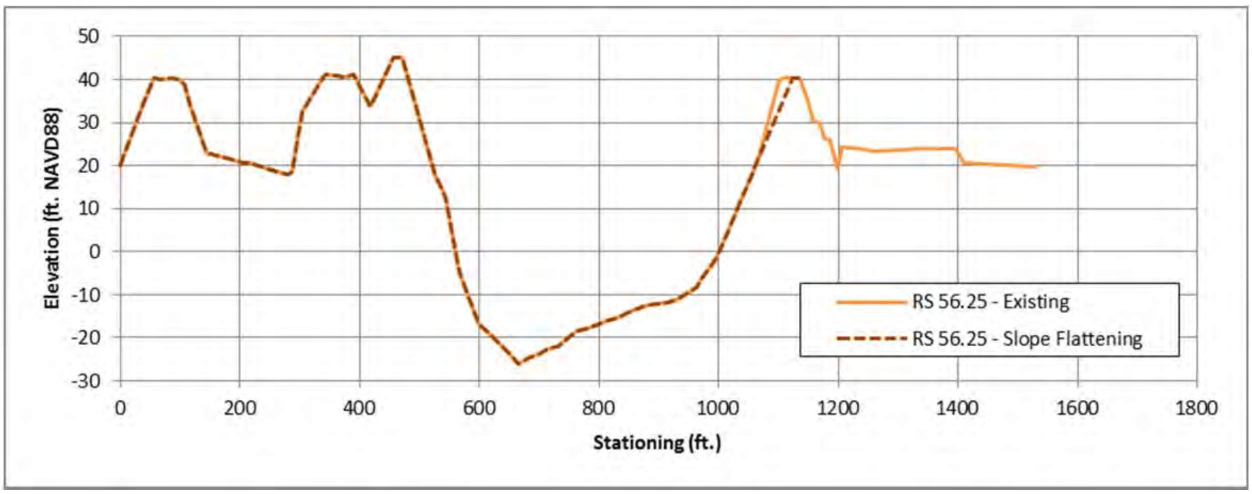


Figure C-6. Cross-section 56.25 (looking downstream)

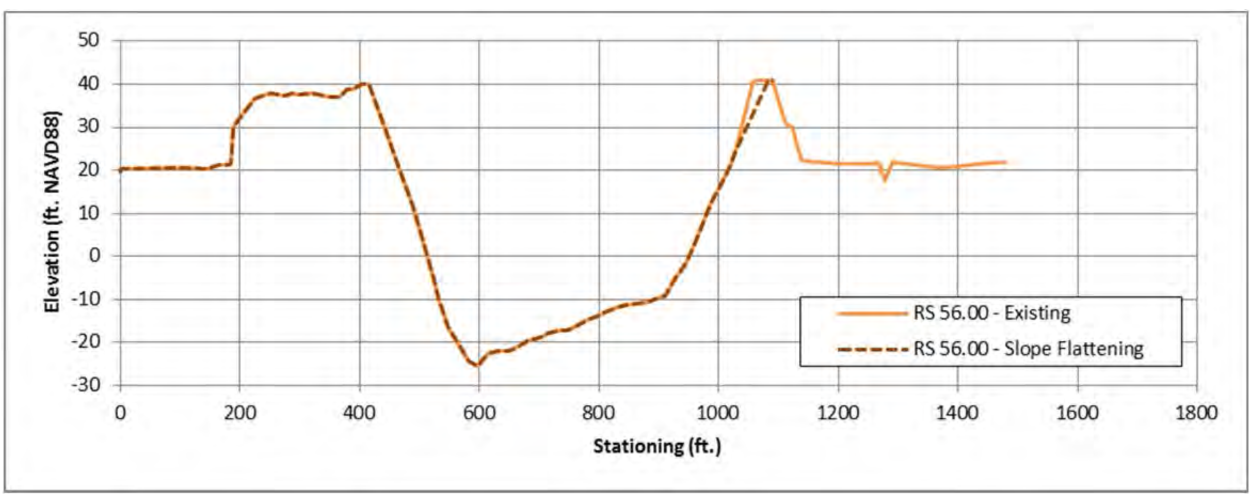


Figure C-7. Cross-section 56.00 (looking downstream)

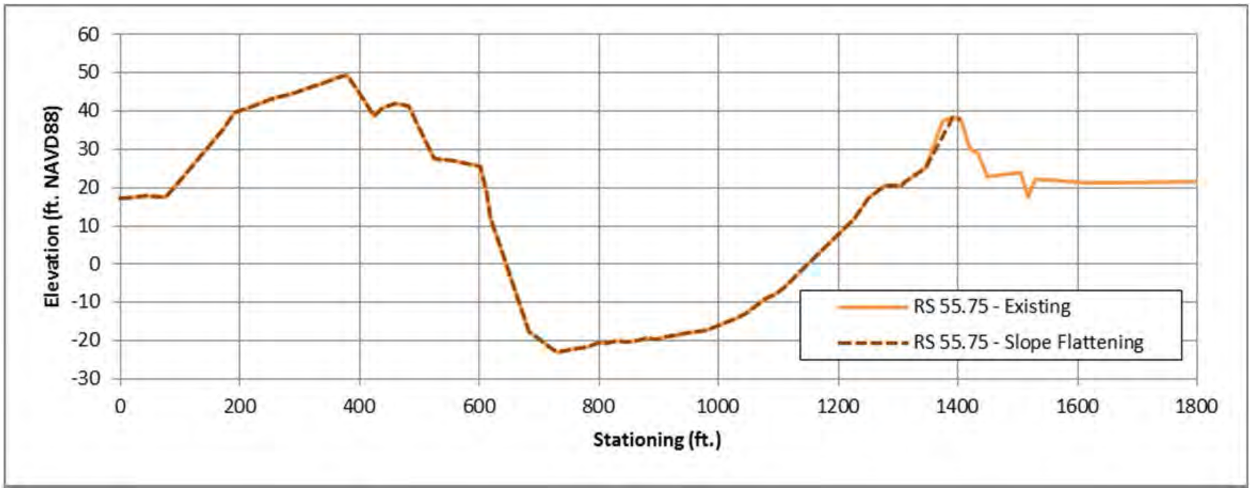


Figure C-8. Cross-section 55.75 (looking downstream)

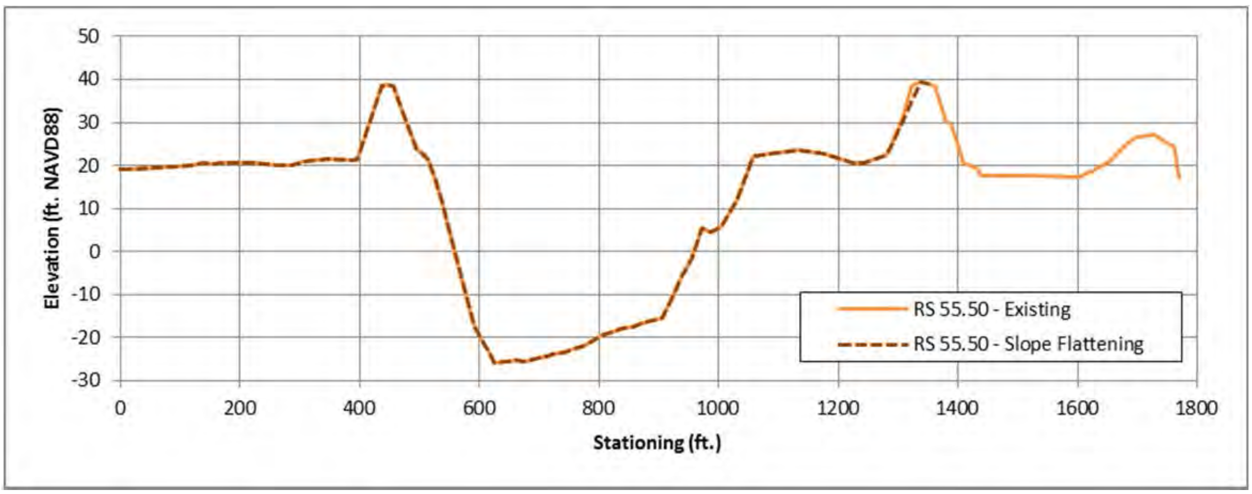


Figure C-9. Cross-section 55.50 (looking downstream)

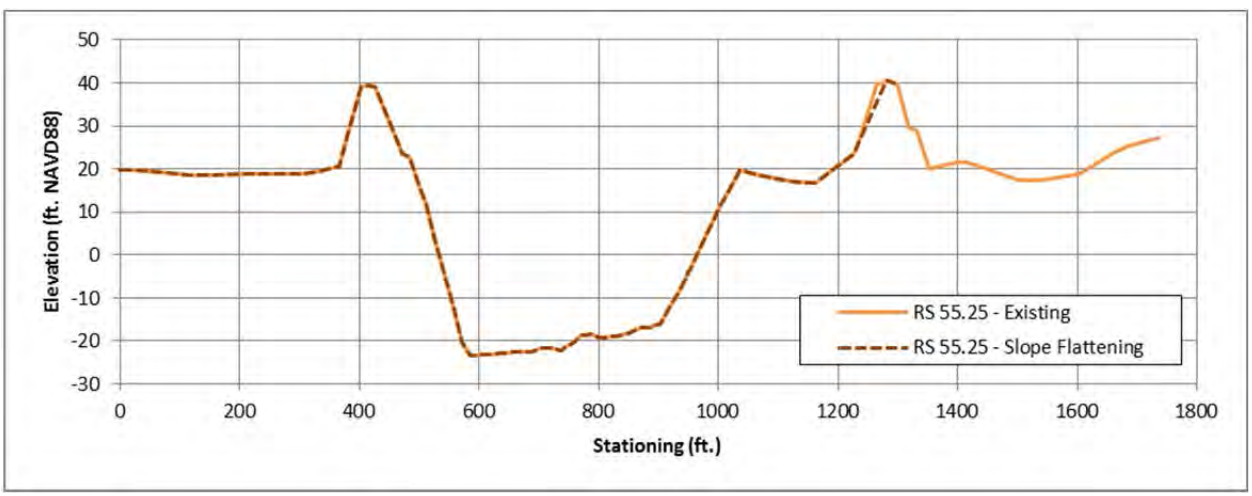


Figure C-10. Cross-section 55.25 (looking downstream)

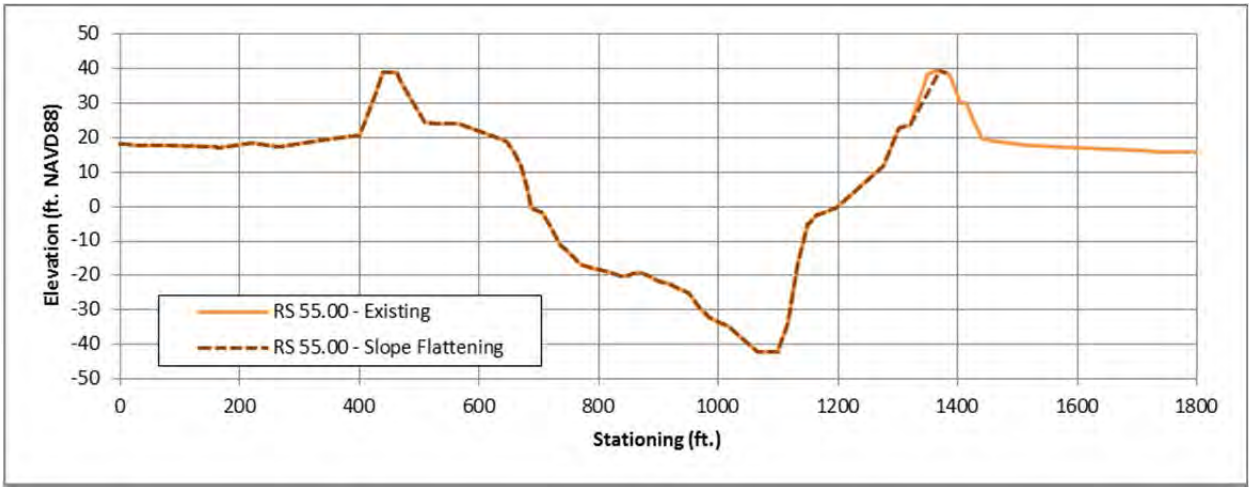


Figure C-11. Cross-section 55.00 (looking downstream)

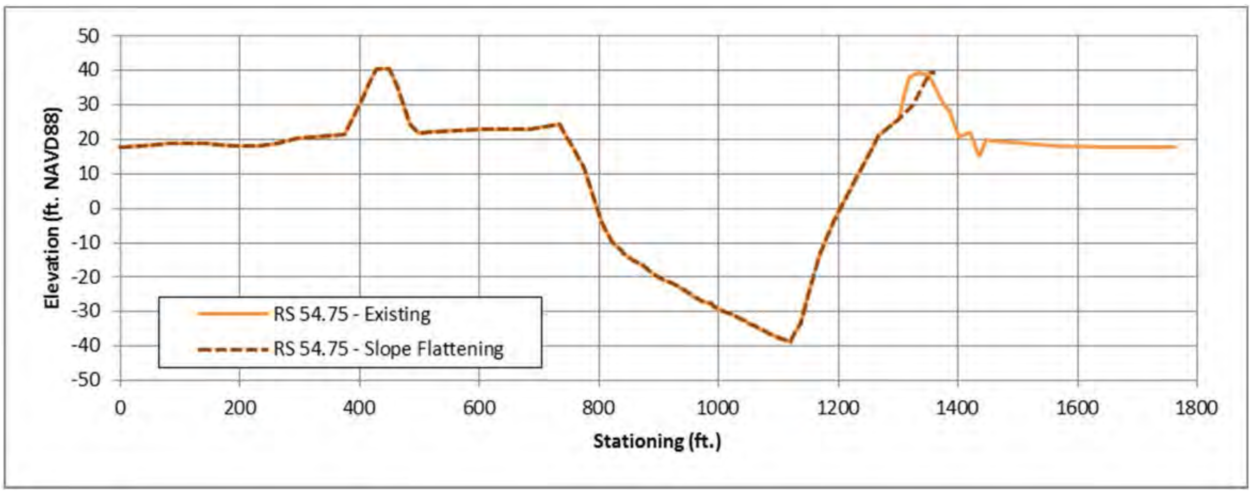


Figure C-12. Cross-section 54.75 (looking downstream)

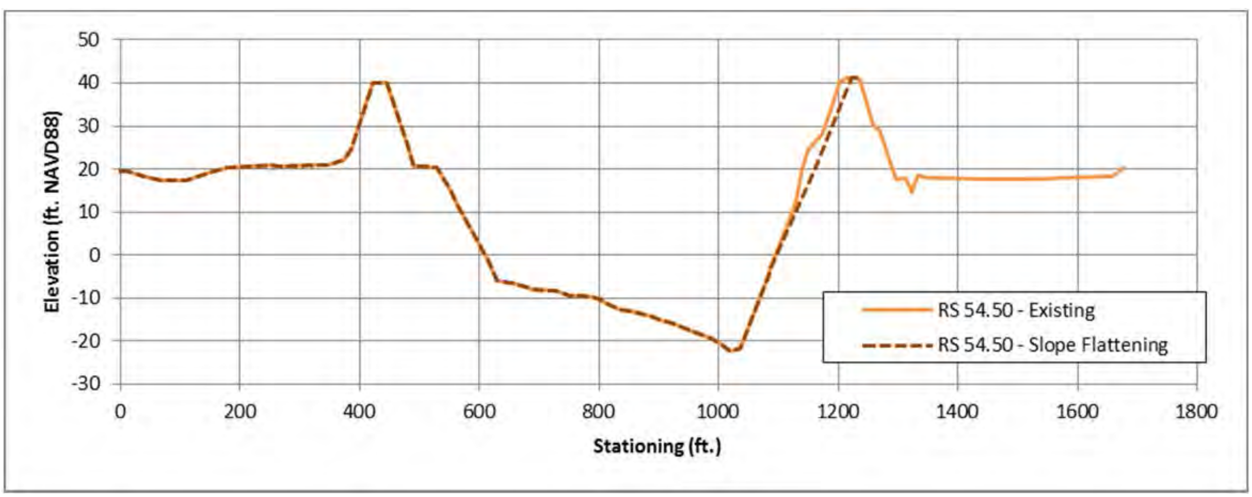


Figure C-13. Cross-section 54.50 (looking downstream)

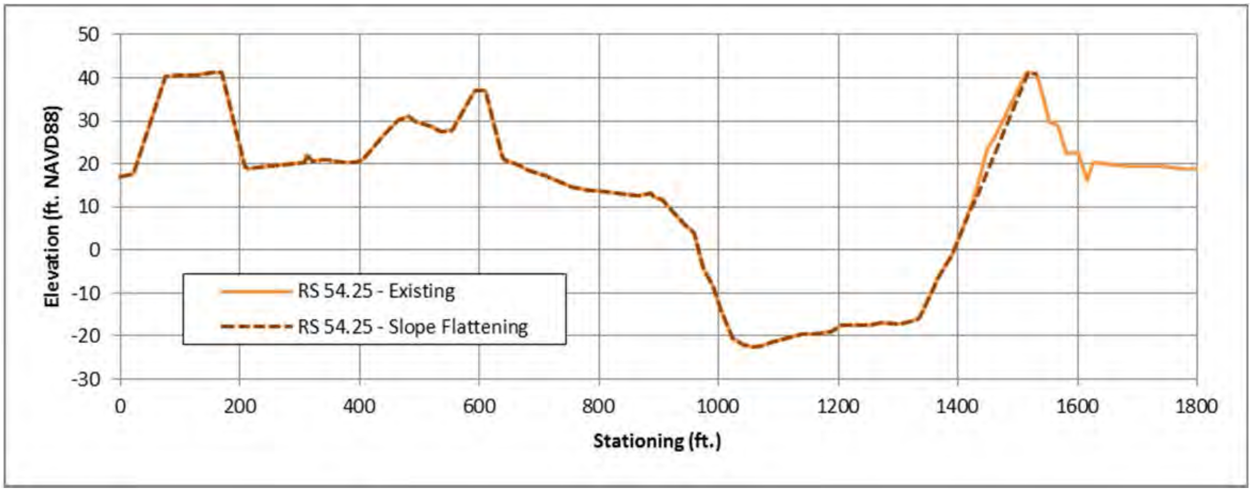


Figure C-14. Cross-section 54.25 (looking downstream)

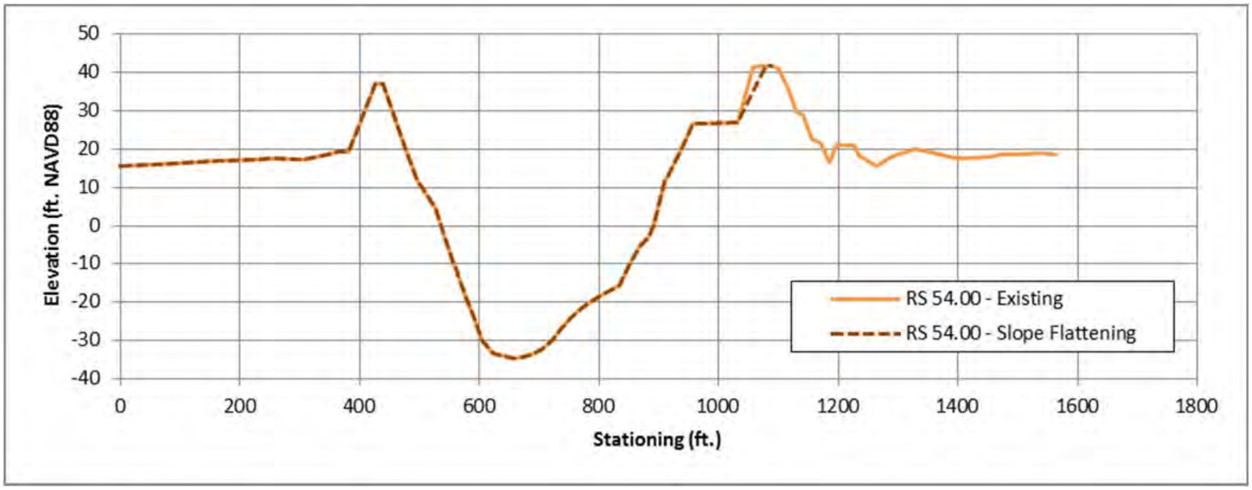


Figure C-15. Cross-section 54.00 (looking downstream)

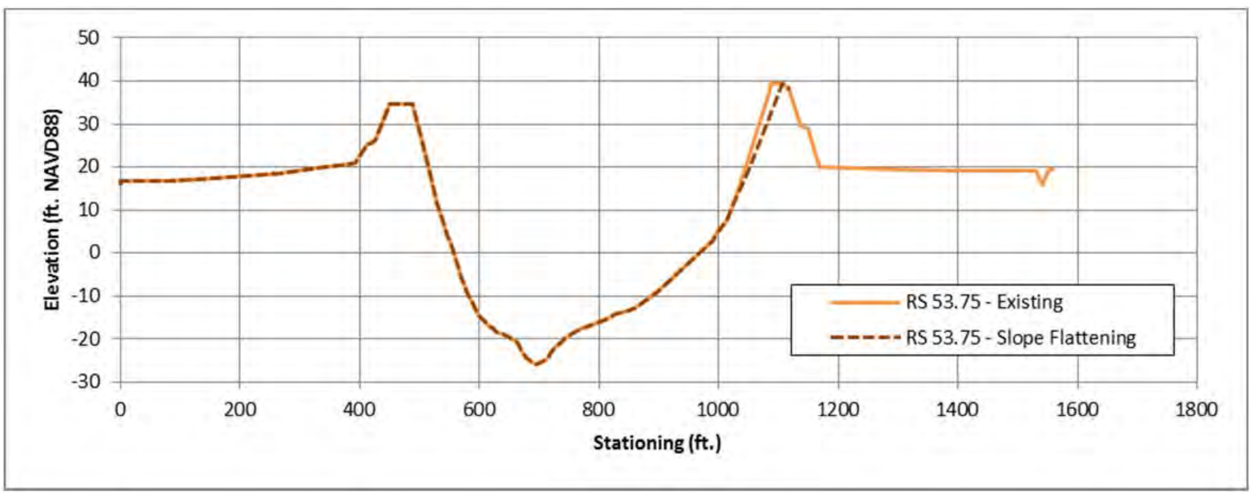


Figure C-16. Cross-section 53.75 (looking downstream)

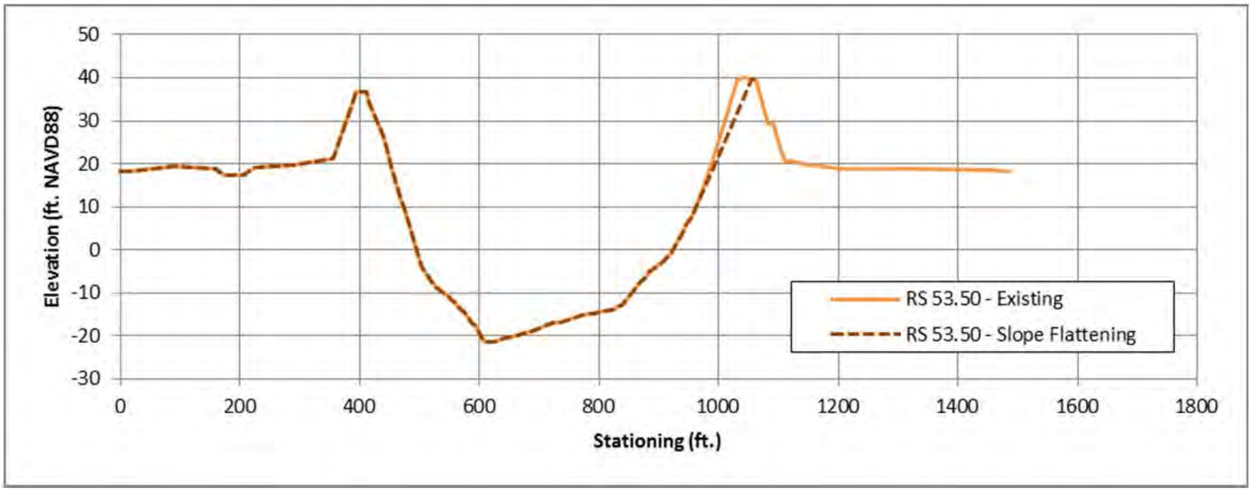


Figure C-17. Cross-section 53.50 (looking downstream)

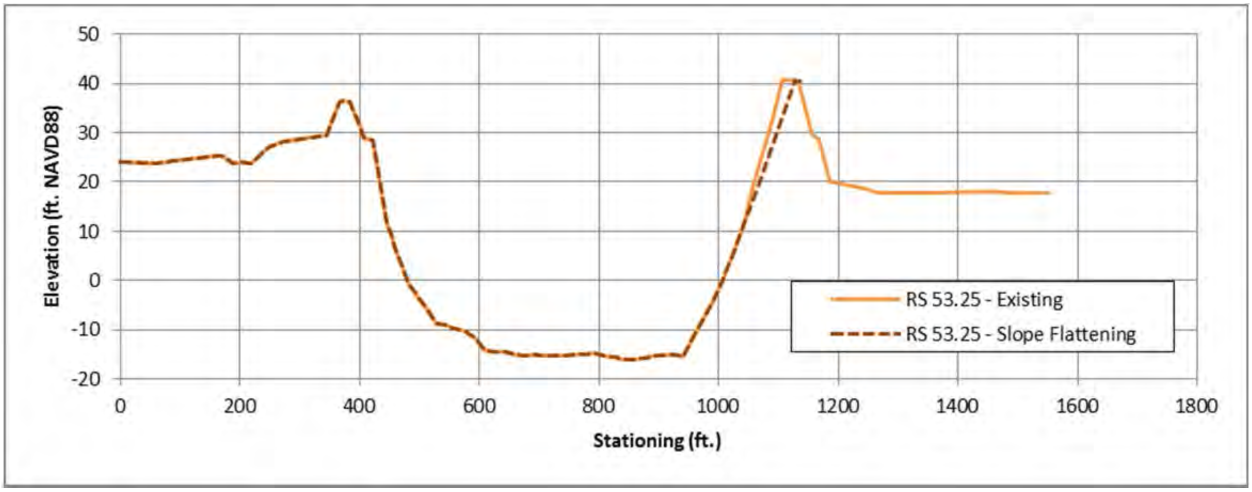


Exhibit C-18. Cross-section 53.25 (looking downstream)

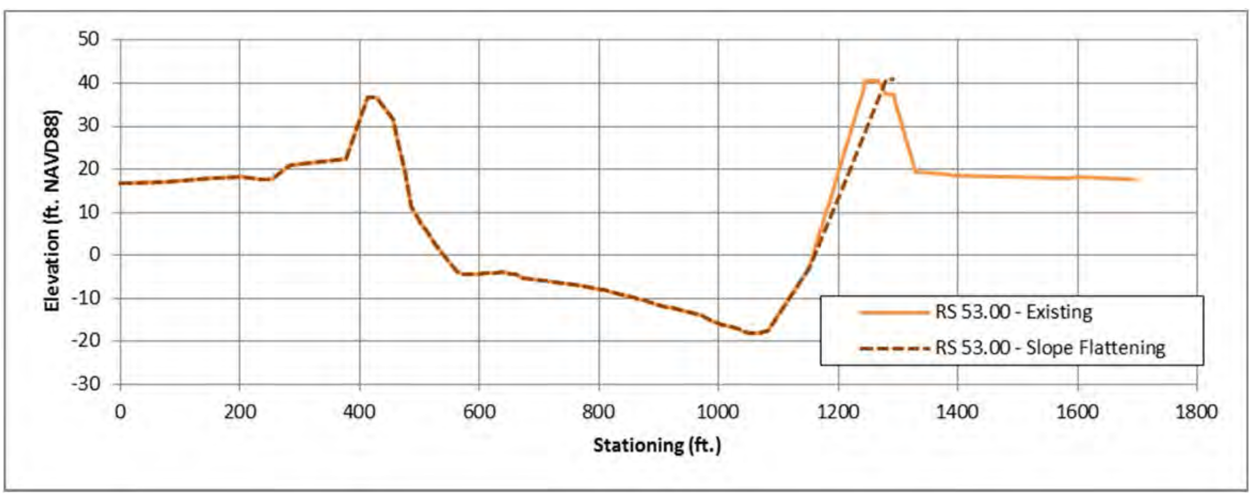


Exhibit C-19. Cross-section 53.00 (looking downstream)

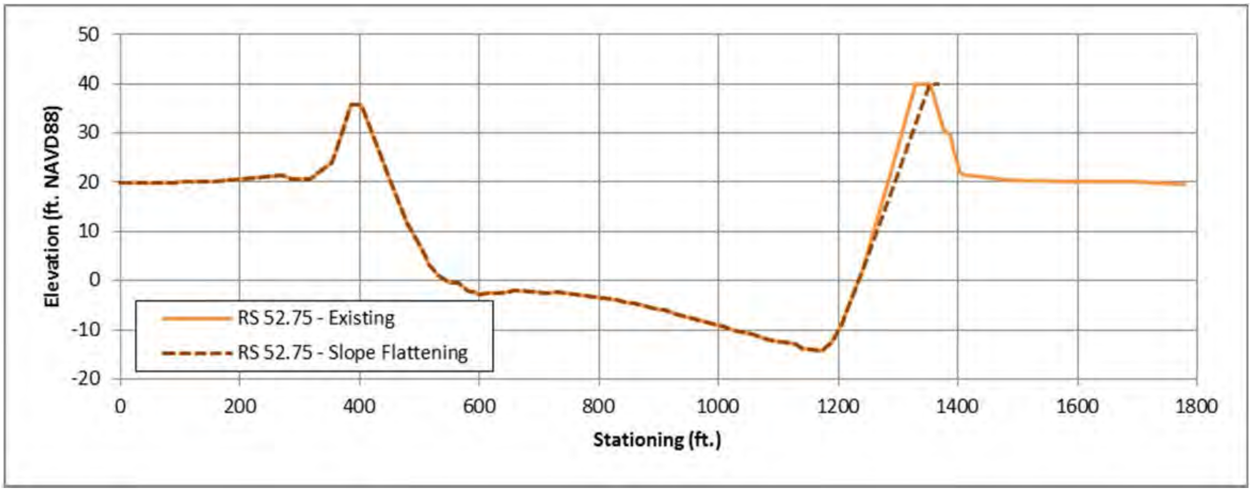


Exhibit C-20. Cross-section 52.75 (looking downstream)

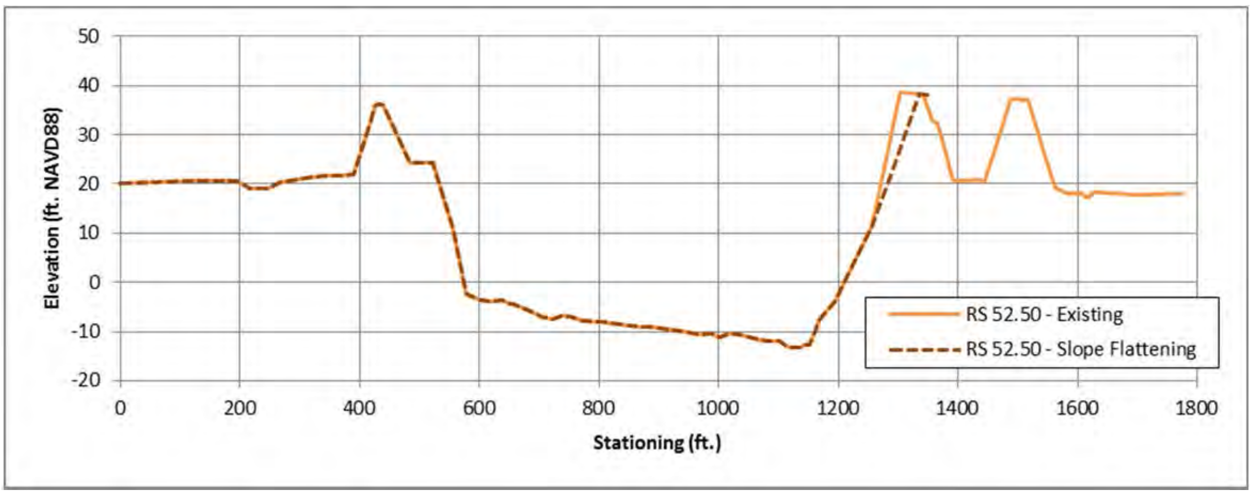


Exhibit C-21. Cross-section 52.50 (looking downstream)

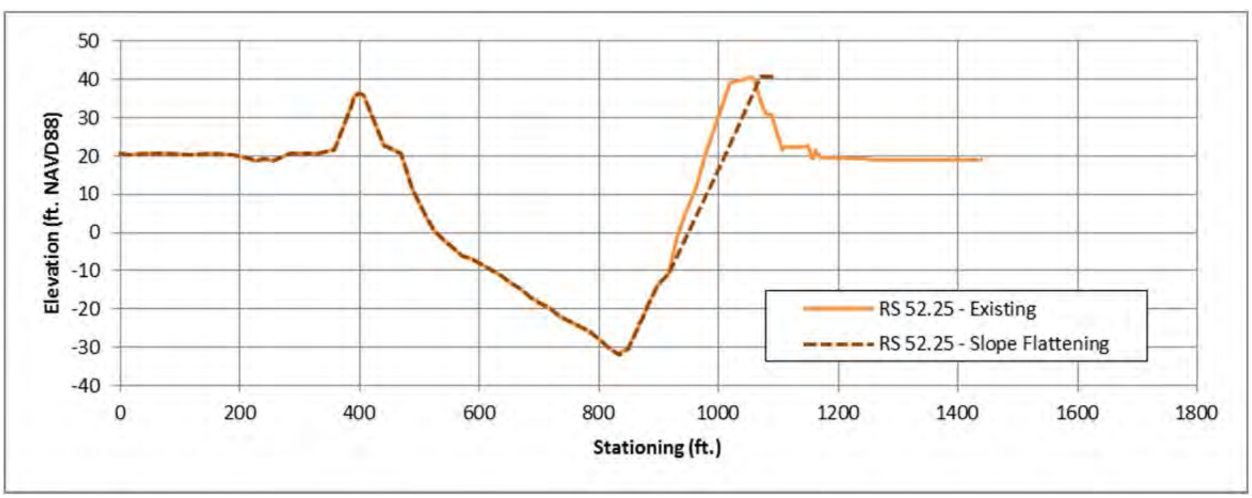


Exhibit C-22. Cross-section 52.25 (looking downstream)

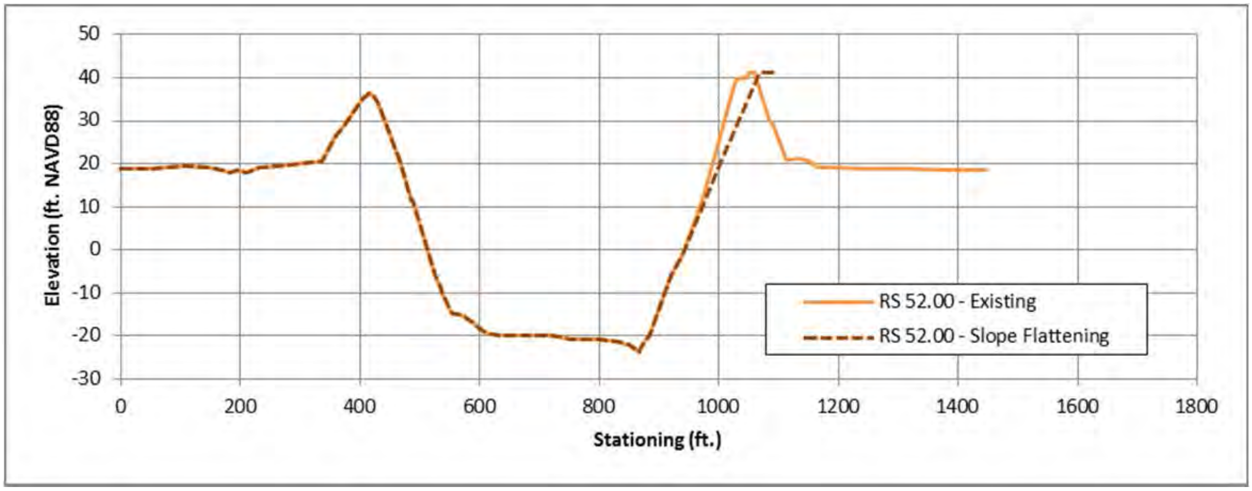


Exhibit C-23. Cross-section 52.00 (looking downstream)

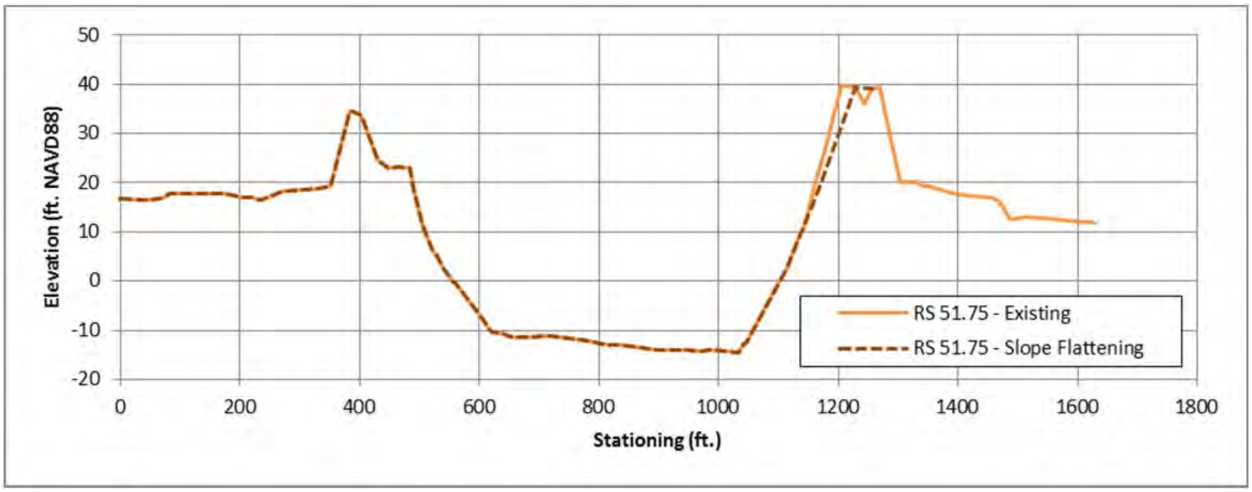


Exhibit C-24. Cross-section 51.75 (looking downstream)

Appendix B.3

Southport Sacramento River Early Implementation Project Interim Condition Hydraulic Impact Analysis—MBK Engineers, September 18, 2013

SOUTHPORT SACRAMENTO RIVER EARLY IMPLEMENTATION PROJECT

INTERIM CONDITION HYDRAULIC IMPACT ANALYSIS

September 18, 2013

Prepared by: Mike Archer, P.E.

Reviewed by: Don Trieu, P.E.

1.0 Purpose

MBK Engineers has evaluated a phased construction approach to reflect how the Southport Sacramento River Early Implementation Project (Southport Project) could be constructed. This memo summarizes that evaluation.

2.0 Proposed Interim Condition

The Southport Project levee setback offset area is effectively divided into two offset areas by Bees Lakes, as shown in Figure 1. The offset area upstream of Bees Lakes will be referred to as the Upper Offset Area and the segment downstream will be referred to as the Lower Offset Area. In both offset areas, the existing project levee, or remnant levee, will be degraded to an elevation of about 30 ft. NAVD88, which is based on the estimated 1/50 AEP flood stage. In the final Southport Project the Upper Offset Area remnant levee will be breached in two locations and the Lower Offset Area remnant levee will be breached in three locations, allowing for flow interchange between the Sacramento River and the offset areas on a relatively frequent basis. The proposed Interim Condition would allow for construction of the downstream-most breaches in the Upper and Lower Offset Areas initially, as shown in Figure 2, and postpone construction of the remaining three breaches to a later construction season.

3.0 Proposed Interim Condition Hydraulic Impact Analysis

The Southport Project RMA2 two-dimensional (2-D) hydraulic simulation model was used to evaluate the effects of the Interim Condition on the 1/100 AEP and 1/200 AEP flood stages. The 2-D analysis is a steady state analysis of the peak flow condition. The flows used in the analysis are summarized in Table 1. The flows vary due to the effects of the evaluated project on the flow split at the confluence of the Sacramento and American Rivers. The Manning's n roughness coefficients assumed for the offset areas were based on the "initial (year 0)" condition from the memorandum "Southport EIP – Roughness Value Development for the Offset Area under Interim and Mature Vegetative Conditions," prepared by cbec eco engineering (cbec) and dated August 28, 2012. The cbec memorandum provided a range of roughness values for the initial condition, the lowest of which was used for the Proposed Interim Condition analysis. Table 2 provides the maximum water surface elevation increases adjacent to the two offset areas. Profile plots of the computed water surface elevation changes along the Sacramento River East Levee are provided in Figures 3 and 4.

Table 1. Proposed Interim Condition Evaluation Flows, Sacramento River near I Street

Condition	Flow (cubic feet per second)	
	1/100 AEP	1/200 AEP
Without Project	122,200	134,600
Proposed Interim Condition	122,980	135,520

Table 2. Proposed Interim Condition Maximum Water Surface Increases

Condition	Increases, in feet	
	1/100 AEP	1/200 AEP
Upstream of Bees Lakes	+ 0.05	+ 0.10
Downstream of Bees Lakes	+ 0.12	+ 0.20

4.0 Comparison with Proposed Project Final Condition

As compared to the Proposed Project Final Condition (see Figures 3 and 4), the proposed Interim Condition showed some reduction in the flood stage change upstream of Bees Lakes, from a maximum of +0.09 ft. to +0.05 ft. in the 1/100 AEP flood and from +0.13 ft. to +0.10 ft. in the 1/200 AEP flood. The proposed Interim Condition had essentially no effect on the flood stage change downstream of Bees Lakes in the 1/100 AEP flood and showed a small increase in the 1/200 AEP flood. The reason for essentially no effect is likely the fact that the remnant levee is overtopped by 1 to 3 feet in the 1/100 AEP flood and by 3 to 5 feet in the 1/200 AEP. Due to this overtopping, not constructing the upper breaches of remnant levee had very little effect on the water surface elevation changes Tables 3 provide a comparison of water surface changes between the Interim Condition and the Final Condition.

Table 3. Proposed Interim Condition Maximum Water Surface Increases

Condition	Increases, in feet	
	Interim Condition	Final Condition
	1/100 AEP	
Upstream of Bees Lakes	+ 0.05	+ 0.09
Downstream of Bees Lakes	+ 0.12	+0.13
	1/200 AEP	
Upstream of Bees Lakes	+ 0.10	+ 0.13
Downstream of Bees Lakes	+ 0.20	+ 0.17

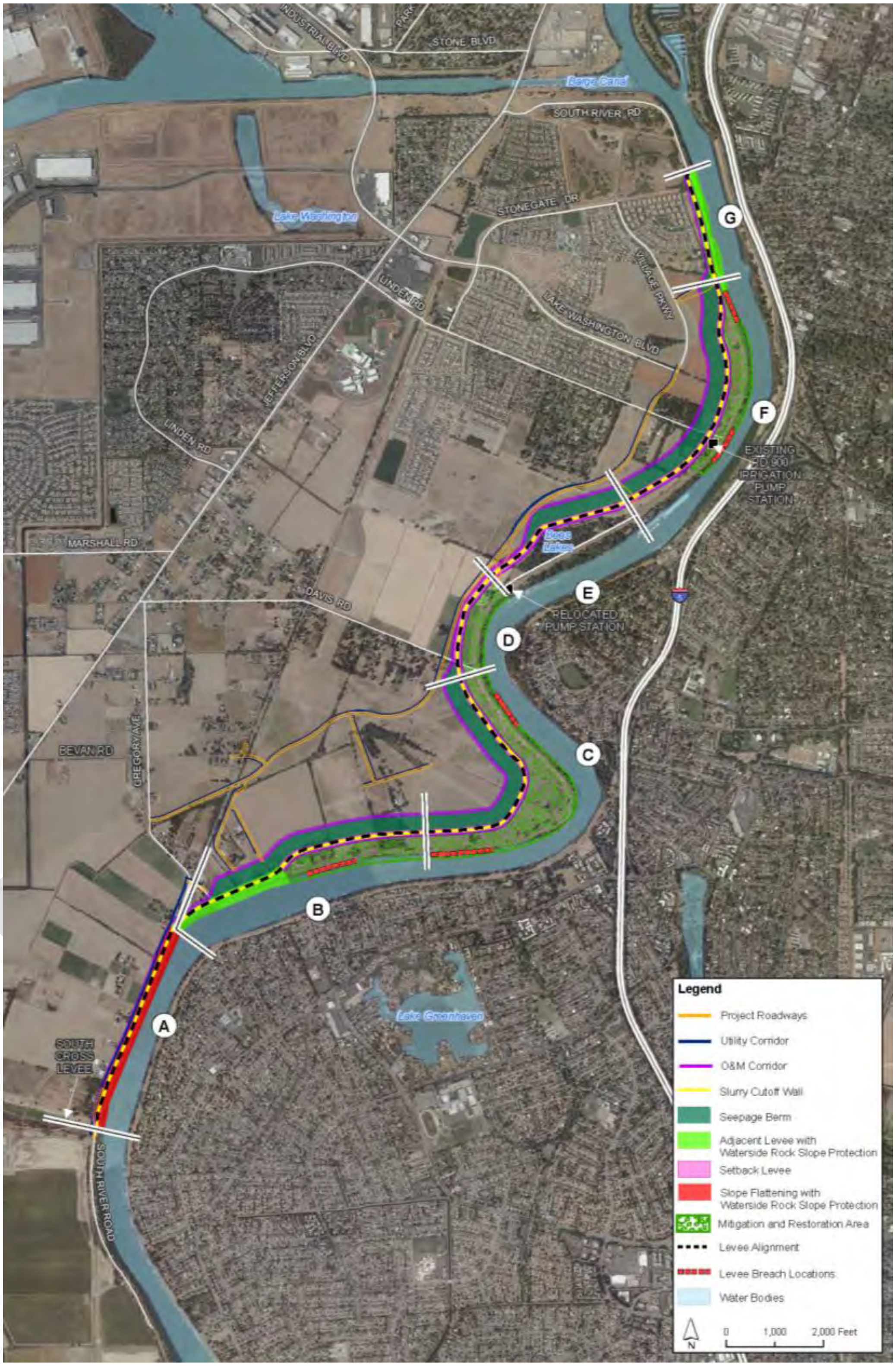


Figure 1. Southport Project, Alternative 5 (Source: ICF International)

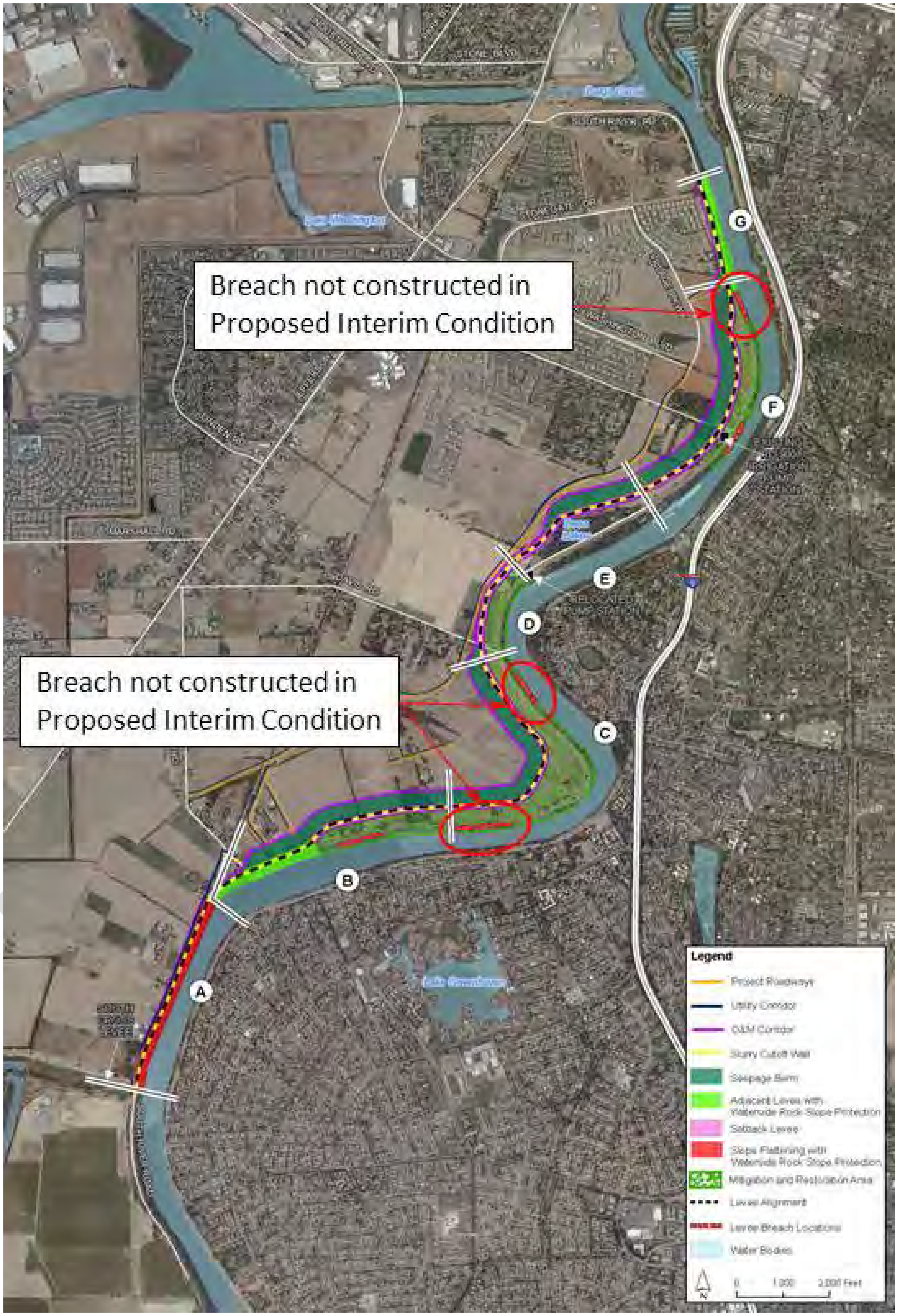


Figure 2. Proposed Interim Condition

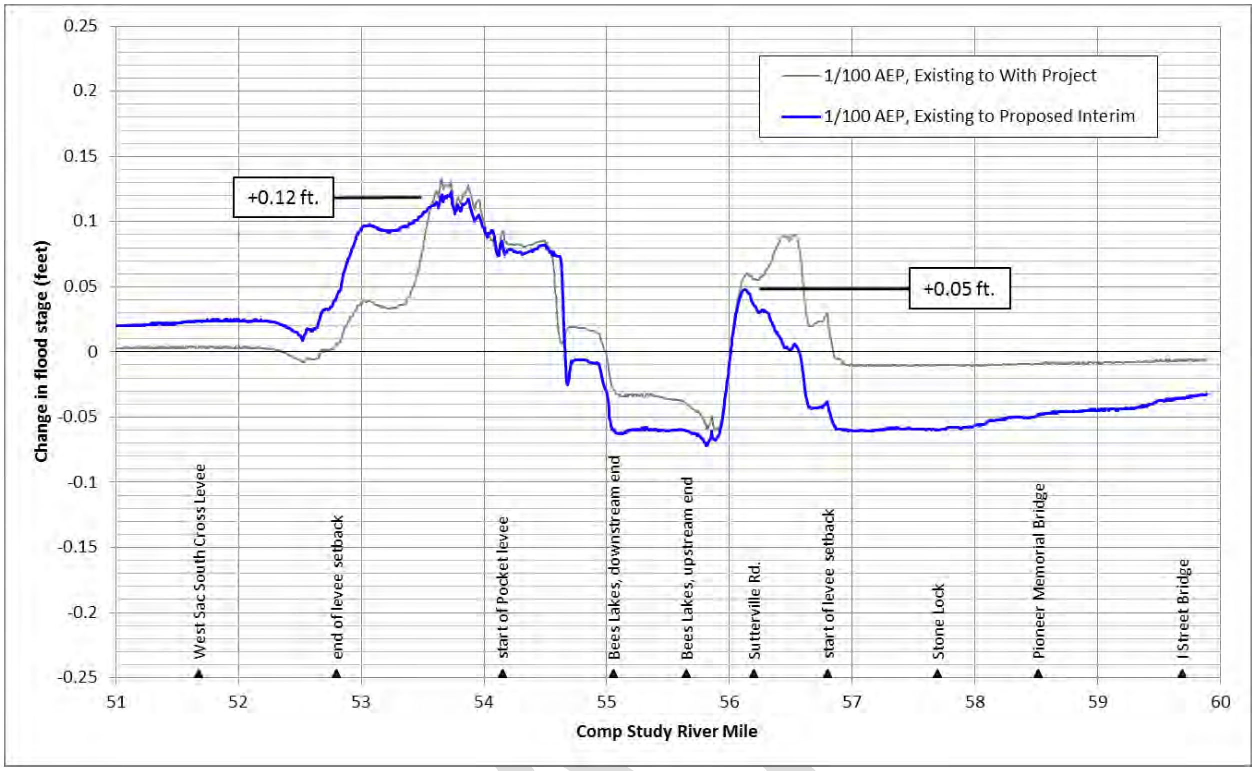


Figure 3. Water Surface Elevation Change due to Proposed Interim Condition, 1/100 AEP

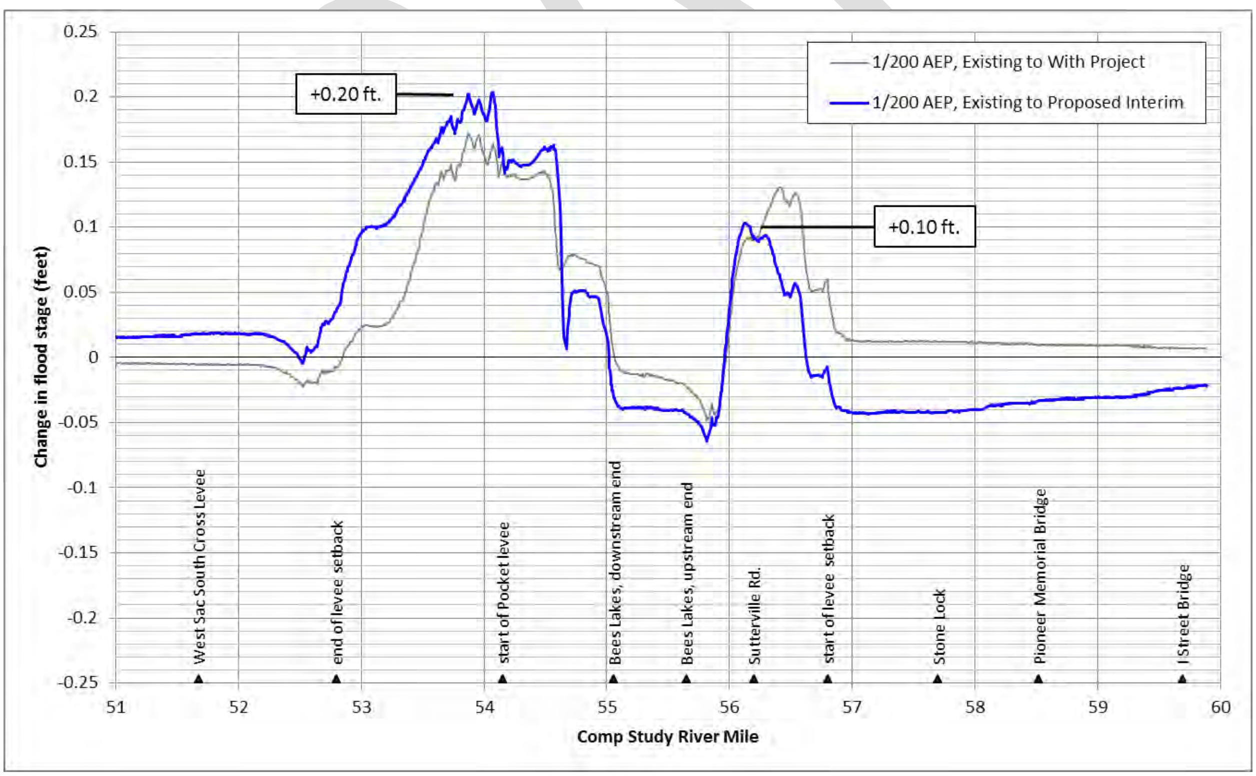


Figure 4. Water Surface Elevation Change due to Proposed Interim Condition, 1/200 AEP

Appendix B.4

Hydraulic Impact Analysis— MBK Engineers, June 29, 2011

HYDRAULIC IMPACT ANALYSIS

Southport Early Implementation Project ***June 29, 2011***

Prepared by: Mike Archer, P.E.

Reviewed by: Don Trieu, P.E.

1.0 Background

The West Sacramento Area Flood Control Agency (WSAFCA) is studying four "Combined Measure Alternative (CMA)" projects for 5 miles of the Sacramento River west levee downstream of Stone Lock in the City of West Sacramento. The project location is shown in Exhibit H-1. A more detailed site map of the project area showing river miles and reference locations is provided in Exhibit H-2. One of the alternatives consists of constructing an adjacent levee on the land side of the existing levee, one involves flattening of the water side slope of the existing levee, and two involve construction of setback levees with partial removal of the existing levee. The alternatives are described in detail in Section 7. MBK Engineers (MBK) has performed a hydraulic impact analysis of the proposed alternatives.

2.0 Purpose

The purpose of the hydraulic impact analysis is to determine the impacts of the proposed CMA's during a flood event with a 1 in 200 annual exceedence probability (200-year). The analysis is focused on answering the following:

1. Does an alternative create the hydraulic benefit of reduced water surface elevations?

2. Does an alternative induce undesirable changes in flow distribution within the Sacramento River waterway system?

3. Does an alternative sustain neutral hydraulic conditions?

4. How would an alternative function in combination with the Folsom Dam Joint Federal Project?

3.0 Hydraulic Model

Release 2 of the U.S. Army Corps of Engineers (USACE) Sacramento River Basin HEC-RAS hydraulic simulation model (Model) was used for this analysis. The extents of the Model are shown in Exhibit H-3. MBK evaluated the Model to ensure that it was adequate for the subject analysis. MBK concluded that the Model was adequate for the subject analysis but also found some areas where refinements could be made to improve the calibration in the WSAFCA study area. A detailed discussion of the Model refinements is provided in Attachment A.

4.0 Hydrology

USACE developed hydrologic input data for the Model for three hypothetical storm "centerings":

1. Sacramento River Mainstem at Latitude of Sacramento
2. Feather River at Shanghai Bend
3. American River

The centerings relied on historical storm patterns in the upstream basin to define the shape and magnitude of the flow contributions from each of the basins and were designed to stress the locations indicated by the centering name. All three centerings were used in the hydraulic impact analysis. The maximum water surface elevation and flow at any location were defined by the largest values computed from the simulations of the three centerings.

The USACE hydrologic input data included seven annual Exceedence probability flood events: 1 in 2 (2-year), 1 in 10 (10-year), 1 in 25 (25-year) , 1 in 50 (50-year) , 1 in 100 (100-year) , 1 in 200 (200-year) and 1 in 500 (500-year). As noted previously, the analysis presented herein used the 200-year event.

5.0 Levee Performance Assumptions

An important assumption in performing hydraulic simulations of leveed systems on a regional basis is defining if, when, and how levee failures will occur. The analysis presented herein assumed all urban levees, including West Sacramento, had a minimum of three feet of freeboard above the 200-year water surface. Non-urban levees were assumed to be at existing levee heights. Additionally, levees were assumed to act as weirs when overtopped and not degrade or fail.

6.0 Without Project Condition

The without project condition is the baseline to which the project alternatives are compared. This condition represents the flood control system of the Sacramento River and the current reservoir operations at Folsom Dam as of 2010. In addition, the USACE Sacramento River Bank Protection Project Erosion Repair Site RM 57.2R (see Exhibit H-4) was assumed to be part of the without project condition and was included in the model by modifying the cross-section at RM 57.5 as shown in Exhibit H-4. No modification was needed for the cross-section at RM 57.647.

7.0 Project Condition

There are four proposed Combined Measure Alternatives as described in the following Sections.

7.1 Combined Measure Alternative 1 (CMA1)

In CMA1, an adjacent levee is constructed on the land side of the existing levee from levee mile 2.3 to 7.8 (see Exhibit H-5). CMA1 does not change the existing cross-sectional flow area of the Sacramento River, so it does not differ hydraulically from the without project condition.

7.2 Combined Measure Alternative 2 (CMA2)

In CMA2, a setback levee is constructed from levee mile 2.7 to 7.8 (river mile 57.0 to 51.75) with a typical setback distance of approximately 400 feet (see Exhibit H-6). A 1,800 ft. length of the existing levee is degraded to original ground at the upstream end of the setback reach and a 2,600 ft. length is degraded to original ground at the downstream end. The remaining existing levee along the setback reach is assumed to be degraded to the 50-year water surface elevation. A typical cross-section showing modeled levee setback is provided in Exhibit H-7. Plots of all cross-sections modified in the Model are provided in Attachment B.

7.3 Combined Measure Alternative 3 (CMA3)

In CMA3, the water side slope of the existing levee is flattened to a 3:1 slope from levee mile 2.3 to 7.8 (see Exhibit H-8). The existing water side slope is approximately 2:1. The slope flattening slightly increases the cross-sectional flow area as shown in Exhibit H-9. However, the increase is very small, typically less than one half of one percent, and therefore has a negligible impact on the river hydraulics. For this reason, CMA3 is assumed to hydraulically be no different than the without project condition

7.4 Combined Measure Alternative 4 (CMA4)

In CMA4, a setback levee is constructed from levee mile 2.7 to 7.8 (river mile 57.0 to 51.75). The setback distance ranges from 1,000 feet to 3,000 feet (see Exhibit H-10). A 1,800 ft. length of the existing levee is degraded to original ground at the upstream end of the setback reach and a 2,600 ft. length is degraded to original ground at the downstream end. The remaining existing levee along the setback reach is assumed to be degraded to the 50-year water surface elevation. A typical cross-section showing modeled levee setback is provided in Exhibit H-7. Plots of all cross-sections modified in the Model are provided in Attachment B.

8.0 Cumulative Impact Analysis

A cumulative impact analysis was performed to determine how the project alternatives would function in combination with the Folsom Dam Joint Federal Project (JFP), which is currently under construction with completion planned in 2020. The JFP involves the construction of a new auxiliary spillway for Folsom Dam that will provide improved operational control during extreme hydrologic events. When complete, the JFP is expected to reduce the 200-year Folsom Dam peak release from 320,000 cubic feet per second (cfs) to 160,000 cfs.

9.0 Results

To assess the regional impact of the setback alignments, the computed maximum water surface elevations and flows from the Project Condition simulations were compared with those from the Without Project Condition simulations. The impacts are presented and discussed in the following sections for the following conditions:

- Current Conditions: The Without Project Condition and the Project Condition are both simulated with current Folsom Dam operations (without the JFP).
- Cumulative: The Without Project Condition is simulated with current Folsom Dam operations (without the JFP) and the Project Condition is simulated with future Folsom Dam operations (with JFP).
- Future Conditions: The Without Project Condition and the Project Condition are both simulated with future Folsom Dam operations (with the JFP).

Maximum water surface elevation profiles are provided for the impacted reaches along with summary tables showing values at selected locations which are shown in Exhibit H-2.

The hydraulic model is referenced to the National Geodetic Vertical Datum of 1929 (NGVD29). The computed water surface elevations as referenced in this report have been converted to the North American Vertical Datum of 1988 (NAVD88) by adding 2.57 feet, the West Sacramento Levee Improvement Program project specific value, as documented in the Kjeldsen, Sinnock & Neudeck (KSN), Inc. report entitled “Survey Control Report, City of West Sacramento, Geotechnical Problem Identification and Alternative Analysis” dated January 2007.

9.1 Current Conditions Impact

The current conditions analysis looks at the impacts of the CMA’s assuming current operations (no JFP) in both the without and with project scenarios.

In the Sacramento River, the levee setbacks result in a reduction in peak stage in the vicinity of the setbacks. However, due to the proximity of the setbacks to the American River, the reduction in stage in the Sacramento River results in an increase in flow in the Sacramento River downstream of the American River. The increase in flow offsets some of the stage decrease upstream of the levee setback but results in an increase in stage downstream of the setback. CMA2 and CMA4 increase the peak flow in the Sacramento River downstream of the American River by 1,500 cfs (1.0%) and 4,800 cfs (3.3%), respectively. CMA2 and CMA4 result in maximum reductions in stage on the Sacramento River of 0.08 ft. and 0.27 ft., respectively, upstream of the setback (RM 57.00). However, due to the increased flow in the Sacramento River, CMA2 and CMA4 show maximum stage increases downstream of the setbacks (RM 51.75) of 0.09 ft. and 0.28 ft., respectively. The increase is still apparent, though reduced to values of 0.05 ft. and 0.13 ft., twenty-six miles downstream at Walnut Grove. Profile plots of the Sacramento River maximum water surface elevations and project impacts are provided in Exhibit H-11. Peak stages and flows and corresponding project impacts are summarized for several key locations in Tables 1 and 2.

In the Yolo Bypass there were no computed increases in maximum water surface elevation. The maximum computed water surface impact in the Yolo Bypass was a negligible decrease of 0.03 ft. Profile plots of the Yolo Bypass maximum water surface elevations and project impacts are provided in Exhibit H-12. Peak stages and flows and corresponding project impacts are summarized for several key locations in Tables 1 and 2.

In the Sacramento Bypass, there were no computed increases in maximum water surface elevation. The maximum computed water surface impact in the Sacramento Bypass was a decrease of 0.07 ft. Profile plots of the Sacramento Bypass maximum water surface elevations and project impacts are provided in Exhibit H-13. Peak stages and flows and corresponding project impacts are summarized for several key locations in Tables 1 and 2.

The Project Condition alternatives have no impact on the maximum water surface elevation in the Sacramento River Deep Water Ship Channel (Ship Channel) and Port of Sacramento (Port) under current conditions. The computed peak stages in the Port and in the Ship Channel at its southern end near Cache Slough are provided in Table 1.

9.2 Cumulative Impact

The cumulative analysis looks at the impact of the CMA's in conjunction with the JFP. In the cumulative analysis there were no increases in the computed maximum water surface elevations relative to the without project current operations condition.

CMA2 resulted in a maximum decrease in the computed maximum water surface elevation in the Sacramento River of 1.9 ft. just upstream of the setback. CMA4 resulted in a maximum decrease in the computed maximum water surface elevation in the Sacramento River of 2.0 ft. just upstream of the setback. Profile plots of the Sacramento River maximum water surface elevations and project impacts are provided in Exhibit H-14. Peak stages and flows and corresponding project impacts are summarized for several key locations in Tables 3 and 4.

CMA2 resulted in a maximum decrease in the computed maximum water surface elevation in the Yolo Bypass of 0.16 ft. CMA4 resulted in a maximum decrease in the computed maximum water surface elevation in the Yolo Bypass of 0.17 ft. Profile plots of the Yolo Bypass maximum water surface elevations and project impacts are provided in Exhibit H-15. Peak stages and flows and corresponding project impacts are summarized for several key locations in Tables 3 and 4.

CMA2 resulted in a maximum decrease in the computed maximum water surface elevation in the Sacramento Bypass of 0.96 ft. CMA4 resulted in a maximum decrease in the computed maximum water surface elevation in the Sacramento Bypass of 1.00 ft. Profile plots of the Sacramento Bypass maximum water surface elevations and project impacts are provided in Exhibit H-16. Peak stages and flows and corresponding project impacts are summarized for several key locations in Tables 3 and 4.

The Project alternatives decrease the maximum water surface elevation in the Sacramento River Deep Water Ship Channel (Ship Channel) and Port of Sacramento (Port) by about 0.1 feet in the Cumulative analysis. The computed peak stages in the Port and in the Ship Channel at its southern end near Cache Slough are provided in Table 3.

9.3 Future Conditions Impact

The future conditions analysis looks at the impacts of the CMA's assuming implementation of the JFP in both the without and with project scenarios.

CMA2 and CMA4 increase the peak flow in the Sacramento River downstream of the American River by 700 cfs (0.5%) and 2,300 cfs (1.8%), respectively. CMA2 and CMA4 result in maximum reductions in stage on the Sacramento River of 0.06 ft. and 0.18 ft., respectively, upstream of the setback (RM 57.00). However, due to the increased flow in the Sacramento River CMA2 and CMA4 show maximum stage increases downstream of the setbacks (RM 51.75) of 0.10 ft. and 0.32 ft., respectively. The increase is still apparent, though reduced to values of 0.04 ft. and 0.14 ft., twenty-six miles downstream at Walnut Grove. Profile plots of the Sacramento River maximum water surface elevations and project impacts are provided in Exhibit H-17. Peak stages and flows and corresponding project impacts are summarized for several key locations in Tables 5 and 6.

In the Yolo Bypass there were no computed increases in maximum water surface elevation. The maximum computed water surface impact in the Yolo Bypass was a negligible decrease of 0.03 ft. Profile plots of the Yolo Bypass maximum water surface elevations and project impacts are provided in Exhibit H-18. Peak stages and flows and corresponding project impacts are summarized for several key locations in Tables 5 and 6.

In the Sacramento Bypass there were no computed increases in maximum water surface elevation. The maximum computed water surface impact in the Sacramento Bypass was a decrease of 0.06 ft. Profile plots of the Sacramento Bypass maximum water surface elevations and project impacts are provided in Exhibit H-19. Peak stages and flows and corresponding project impacts are summarized for several key locations in Tables 5 and 6.

The Project alternatives have no impact on the maximum water surface elevation in the Sacramento River Deep Water Ship Channel (Ship Channel) and Port of Sacramento (Port) under future conditions. The computed peak stages in the Port and in the Ship Channel at its southern end near Cache Slough are provided in Table 5.

Table 1. Impact on Maximum Water Surface Elevations, Current Conditions (without JFP)

ID	River	Location	Comp Study River Mile	Maximum Water Surface Elevation (ft. NAVD88)			Difference (ft)	
				Without Project	With Project			
					CMA2	CMA4	CMA2	CMA4
SR1	Sacramento River	At Natomas Cross Canal	79.205	44.57	44.57	44.57	0	0
SR2	Sacramento River	At Sacramento Weir	63.81	37.61	37.57	37.51	-0.04	-0.10
SR3	Sacramento River	At Interstate 80	62.97	37.72	37.69	37.63	-0.03	-0.09
SR4	Sacramento River	At I Street Bridge	59.692	37.96	37.92	37.81	-0.04	-0.15
SR5	Sacramento River	At upstream end of Levee Setback	57.00	36.28	36.20	36.01	-0.08	-0.27
SR6	Sacramento River	Near Davis Road	54.75	35.27	35.29	35.21	+0.02	-0.06
SR7	Sacramento River	At downstream end of Levee Setback	51.75	33.63	33.72	33.91	+0.09	+0.28
SR8	Sacramento River	At Babel Slough	50.00	32.88	32.97	33.16	+0.09	+0.28
SR9	Sacramento River	At Freeport Bridge	46.43	30.89	30.98	31.14	+0.09	+0.25
SR10	Sacramento River	At Walnut Grove	26.75	18.22	18.27	18.35	+0.05	+0.13
YB1	Yolo Bypass	At Interstate 5	50.496	36.58	36.58	36.57	0	-0.01
YB2	Yolo Bypass	At Sacramento Bypass	44.13	33.38	33.37	33.35	-0.01	-0.03
YB3	Yolo Bypass	At Lisbon Gage	35.672	29.45	29.44	29.43	-0.01	-0.02
SB1	Sacramento Bypass	Below Sacramento Weir	1.49	36.07	36.05	36.00	-0.02	-0.07
SB2	Sacramento Bypass	At Yolo Bypass	0	33.38	33.37	33.35	-0.01	-0.03
SC1	Deep Water Ship Channel	Port of Sacramento	42.984	17.09	17.09	17.09	0	0
SC2	Deep Water Ship Channel	At Cache Slough	19.54	17.03	17.03	17.03	0	0

Table 2. Impact on Maximum Flows, Current Conditions (without JFP)

Location	Maximum Flow (cfs) [1]			Difference (%)	
	Without Project	With Project			
		CMA2	CMA4	CMA2	CMA4
Sacramento River above Sacramento Weir	116,300	116,400	116,500	0.1%	0.2%
Sacramento River below Sacramento Weir	-98,900	-97,900	-95,800	-1.0%	-3.1%
Sacramento River below American River (upstream of Project)	147,400	148,900	152,200	1.0%	3.3%
Sacramento River at Freeport (downstream of Project)	144,400	145,300	147,000	0.6%	1.8%
Sacramento Weir	193,400	192,700	191,200	-0.4%	-1.1%
Fremont Weir	471,300	471,200	471,300	0%	0%
Yolo Bypass below Sacramento Bypass	655,300	654,800	653,400	-0.1%	-0.3%

[1] Negative value indicates flow in upstream direction. For the reach between the Sacramento Weir and the American River this occurs when the Sacramento Weir gates are open and the American River flow is high. The American River flow splits when it reaches the Sacramento River with some heading upstream towards the Sacramento Weir and the rest heading downstream.

Table 3. Cumulative Impact on Maximum Water Surface Elevations (Current operation without project to future operation with project)

ID	River	Location	Comp Study River Mile	Maximum Water Surface Elevation (ft. NAVD88)			Difference (ft)	
				Without Project	With Project			
					CMA2	CMA4	CMA2	CMA4
SR1	Sacramento River	At Natomas Cross Canal	79.205	44.57	44.55	44.55	-0.02	-0.02
SR2	Sacramento River	At Sacramento Weir	63.81	37.61	35.96	35.91	-1.65	-1.70
SR3	Sacramento River	At Interstate 80	62.97	37.72	36.45	36.40	-1.27	-1.32
SR4	Sacramento River	At I Street Bridge	59.692	37.96	36.06	35.99	-1.90	-1.97
SR5	Sacramento River	At upstream end of Levee Setback	57.00	36.28	34.39	34.27	-1.89	-2.01
SR6	Sacramento River	Near Davis Road	54.75	35.27	33.53	33.51	-1.74	-1.76
SR7	Sacramento River	At downstream end of Levee Setback	51.75	33.63	32.06	32.28	-1.57	-1.35
SR8	Sacramento River	At Babel Slough	50.00	32.88	31.34	31.56	-1.54	-1.32
SR9	Sacramento River	At Freeport Bridge	46.43	30.89	29.49	29.70	-1.40	-1.19
SR10	Sacramento River	At Walnut Grove	26.75	18.22	17.84	17.94	-0.38	-0.28
YB1	Yolo Bypass	At Interstate 5	50.496	36.58	36.50	36.50	-0.08	-0.08
YB2	Yolo Bypass	At Sacramento Bypass	44.13	33.38	33.24	33.22	-0.14	-0.16
YB3	Yolo Bypass	At Lisbon Gage	35.672	29.45	29.35	29.33	-0.10	-0.12
SB1	Sacramento Bypass	Below Sacramento Weir	1.49	36.07	35.11	35.07	-0.96	-1.00
SB2	Sacramento Bypass	At Yolo Bypass	0	33.38	33.24	33.22	-0.14	-0.16
SC1	Deep Water Ship Channel	Port of Sacramento	42.984	17.09	16.97	16.97	-0.12	-0.12
SC2	Deep Water Ship Channel	At Cache Slough	19.54	17.03	16.92	16.91	-0.11	-0.12

Table 4. Cumulative Impact on Maximum Flows (Current operation without project to future operation with project)

Location	Maximum Flow (cfs) [1]			Difference (%)	
	Without Project	With Project			
		CMA2	CMA4	CMA2	CMA4
Sacramento River above Sacramento Weir	116,300	116,400	116,500	0.1%	0.2%
Sacramento River below Sacramento Weir	-98,900	-48,700	-47,100	-50.8%	-52.4%
Sacramento River below American River (upstream of Project)	147,400	131,900	133,500	-10.5%	-9.4%
Sacramento River at Freeport (downstream of Project)	144,400	131,700	133,300	-8.8%	-7.7%
Sacramento Weir	193,400	146,700	145,200	-24.1%	-24.9%
Fremont Weir	471,300	471,400	471,400	0%	0%
Yolo Bypass below Sacramento Bypass	655,300	642,600	641,500	-1.9%	-2.1%

[1] Negative value indicates flow in upstream direction. For the reach between the Sacramento Weir and the American River this occurs when the Sacramento Weir gates are open and the American River flow is high. The American River flow splits when it reaches the Sacramento River with some heading upstream towards the Sacramento Weir and the rest heading downstream.

Table 5. Impact on Maximum Water Surface Elevations, Future Conditions (with JFP)

ID	River	Location	Comp Study River Mile	Maximum Water Surface Elevation (ft. NAVD88)			Difference (ft)	
				Without Project	With Project			
					CMA2	CMA4	CMA2	CMA4
SR1	Sacramento River	At Natomas Cross Canal	79.205	44.55	44.55	44.55	0	0
SR2	Sacramento River	At Sacramento Weir	63.81	35.99	35.96	35.91	-0.03	-0.08
SR3	Sacramento River	At Interstate 80	62.97	36.47	36.45	36.40	-0.02	-0.07
SR4	Sacramento River	At I Street Bridge	59.692	36.10	36.06	35.99	-0.04	-0.11
SR5	Sacramento River	At upstream end of Levee Setback	57.00	34.45	34.39	34.27	-0.06	-0.18
SR6	Sacramento River	Near Davis Road	54.75	33.50	33.53	33.51	+0.03	+0.01
SR7	Sacramento River	At downstream end of Levee Setback	51.75	31.96	32.06	32.28	+0.10	+0.32
SR8	Sacramento River	At Babel Slough	50.00	31.24	31.34	31.56	+0.10	+0.32
SR9	Sacramento River	At Freeport Bridge	46.43	29.40	29.49	29.70	+0.09	+0.30
SR10	Sacramento River	At Walnut Grove	26.75	17.80	17.84	17.94	+0.04	+0.14
YB1	Yolo Bypass	At Interstate 5	50.496	36.51	36.50	36.50	-0.01	-0.01
YB2	Yolo Bypass	At Sacramento Bypass	44.13	33.25	33.24	33.22	-0.01	-0.03
YB3	Yolo Bypass	At Lisbon Gage	35.672	29.35	29.35	29.33	0	-0.02
SB1	Sacramento Bypass	Below Sacramento Weir	1.49	35.13	35.11	35.07	-0.02	-0.06
SB2	Sacramento Bypass	At Yolo Bypass	0	33.25	33.24	33.22	-0.01	-0.03
SC1	Deep Water Ship Channel	Port of Sacramento	42.984	16.97	16.97	16.97	0	0
SC2	Deep Water Ship Channel	At Cache Slough	19.54	16.92	16.92	16.91	0	-0.01

Table 6. Impact on Maximum Flows, Future Conditions (with JFP)

Location	Maximum Flow (cfs) [1]			Difference (%)	
	Without Project	With Project			
		CMA2	CMA4	CMA2	CMA4
Sacramento River above Sacramento Weir	116,300	116,400	116,500	0.1%	0.2%
Sacramento River below Sacramento Weir	-49,600	-48,700	-47,100	-1.8%	-5.0%
Sacramento River below American River (upstream of Project)	131,200	131,900	133,500	0.5%	1.8%
Sacramento River at Freeport (downstream of Project)	131,000	131,700	133,300	0.5%	1.8%
Sacramento Weir	147,400	146,700	145,200	-0.5%	-1.5%
Fremont Weir	471,300	471,400	471,400	0.0%	0.0%
Yolo Bypass below Sacramento Bypass	643,400	642,600	641,500	-0.1%	-0.3%

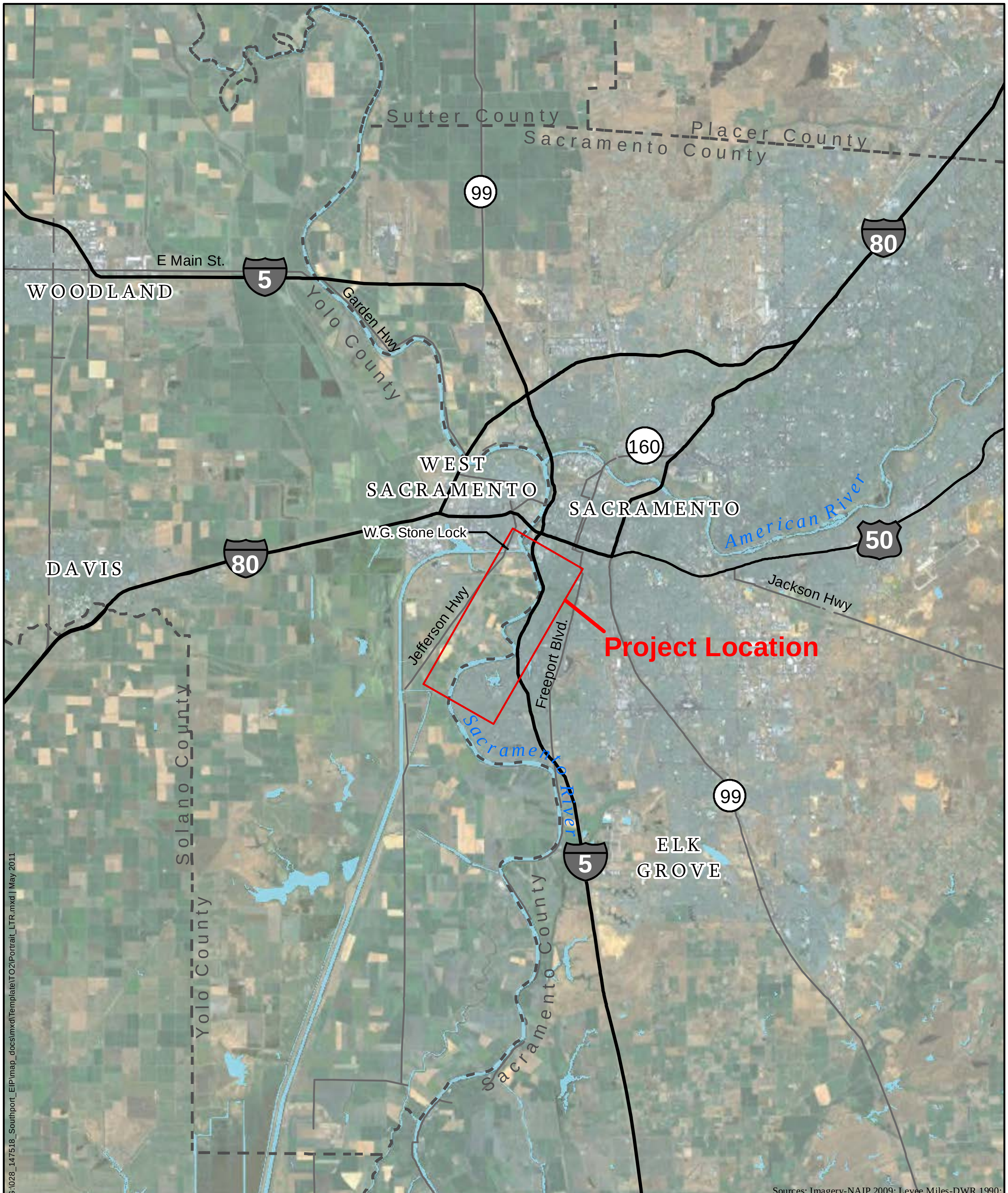
[1] Negative value indicates flow in upstream direction. For the reach between the Sacramento Weir and the American River this occurs when the Sacramento Weir gates are open and the American River flow is high. The American River flow splits when it reaches the Sacramento River with some heading upstream towards the Sacramento Weir and the rest heading downstream.

10.0 Future Hydraulic Analysis

The hydraulic analysis performed for the interim preliminary design phase focused on determining what the system wide hydraulic impacts would be as a result of the setback alternatives. We focused on determining if there were undesirable changes in flow distribution, beneficial and adverse water surface elevations changes, and assessing cumulative impacts.

During the next phase of this study (final preliminary design), the hydraulic analysis will continue to focus on answering questions presented in Section 2 but will further expand to address river reach and site specific questions. We will be developing a 2-dimensional hydraulic model of the Sacramento River in the project vicinity to corroborate our 1-dimensional model results and perform more detailed hydraulic analysis of features such as partial levee removal, setback grading, and habitat enhancement.

EXHIBITS



WSAFCA

Levee Improvement Program Southport Early Implementation Project

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Drawn
A. ARNOLD, HDR

- PROJECT_BOUNDARY
- COUNTIES
- INTERSTATE
- US HIGHWAY
- STATE HIGHWAY
- WATERWAYS AND WATER BODIES

Location Map

Exhibit H-1

Project Num: 453318-157784-028

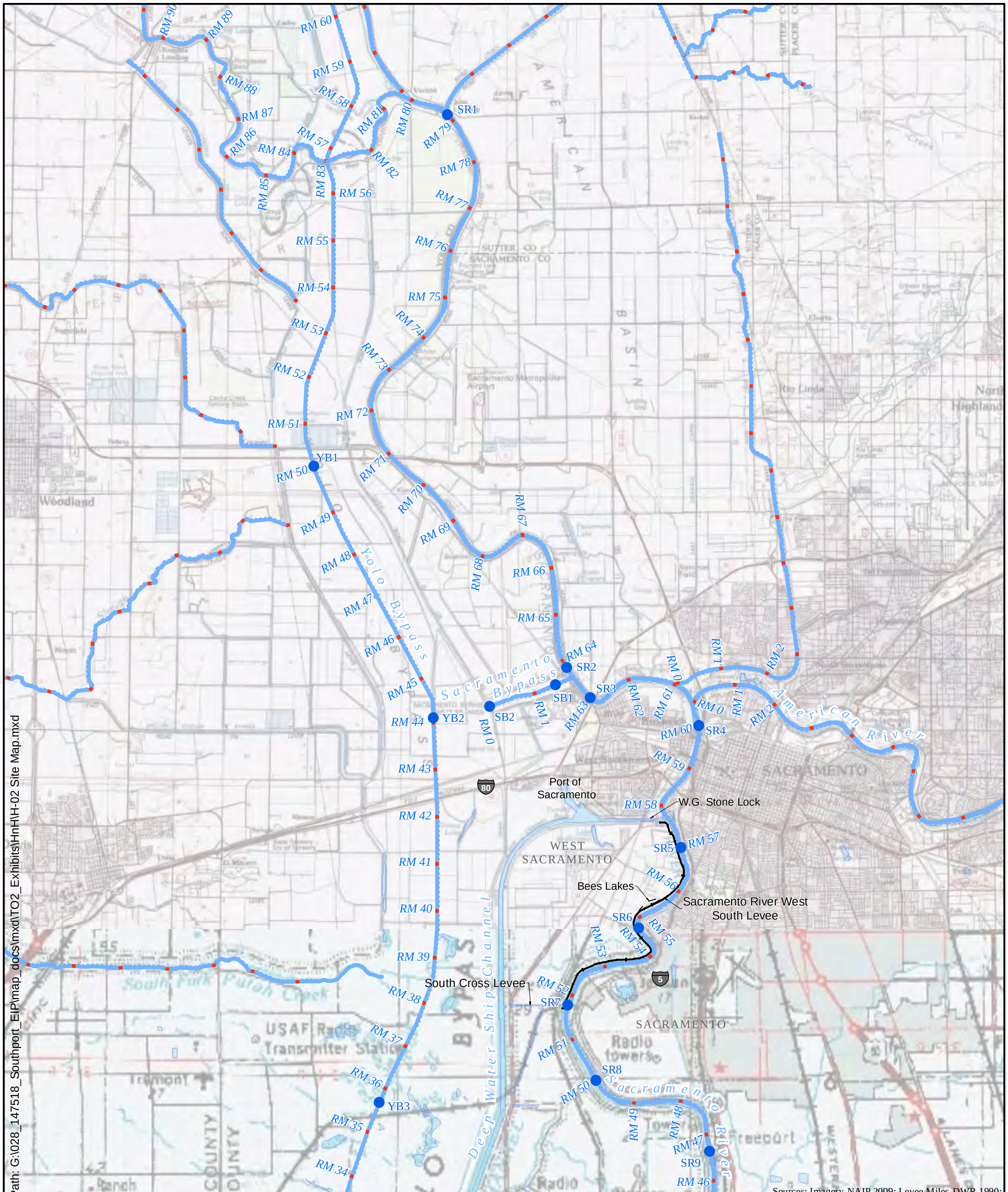
May 2011

N W E S

0 1 2 3 4

Miles

1 in = 4 miles



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Levee Improvement Program Southport Early Implementation Project

Project Manager
M. VECCHIO, HDR

Designed
M. ARCHER, MBK

Checked
B. JONES, HDR

Drawn
A. ARNOLD, HDR

SB1 INDEX POINT LOCATION

WATERWAY CENTERLINE

RM 57 RIVER MILES

PROJECT STATIONING

Site Map

Exhibit H-2

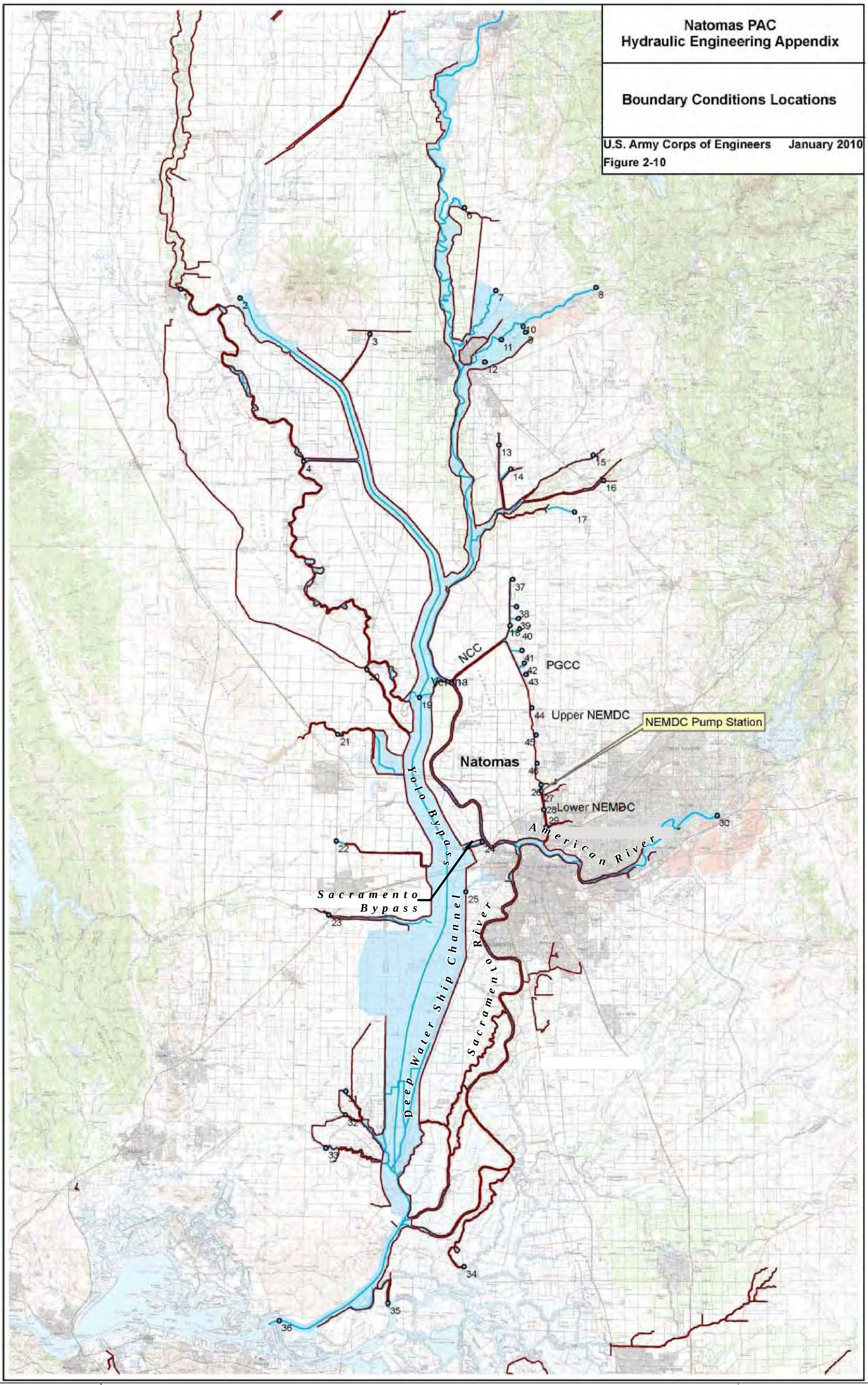
Project Num: 453318-157784-028

May 2011

N
W E
S

0 1 2 3

Miles
1 in = 3 miles



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Project Manager
M. VECCHIO, HDR

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USACE

Checked
USACE

Drawn
USACE

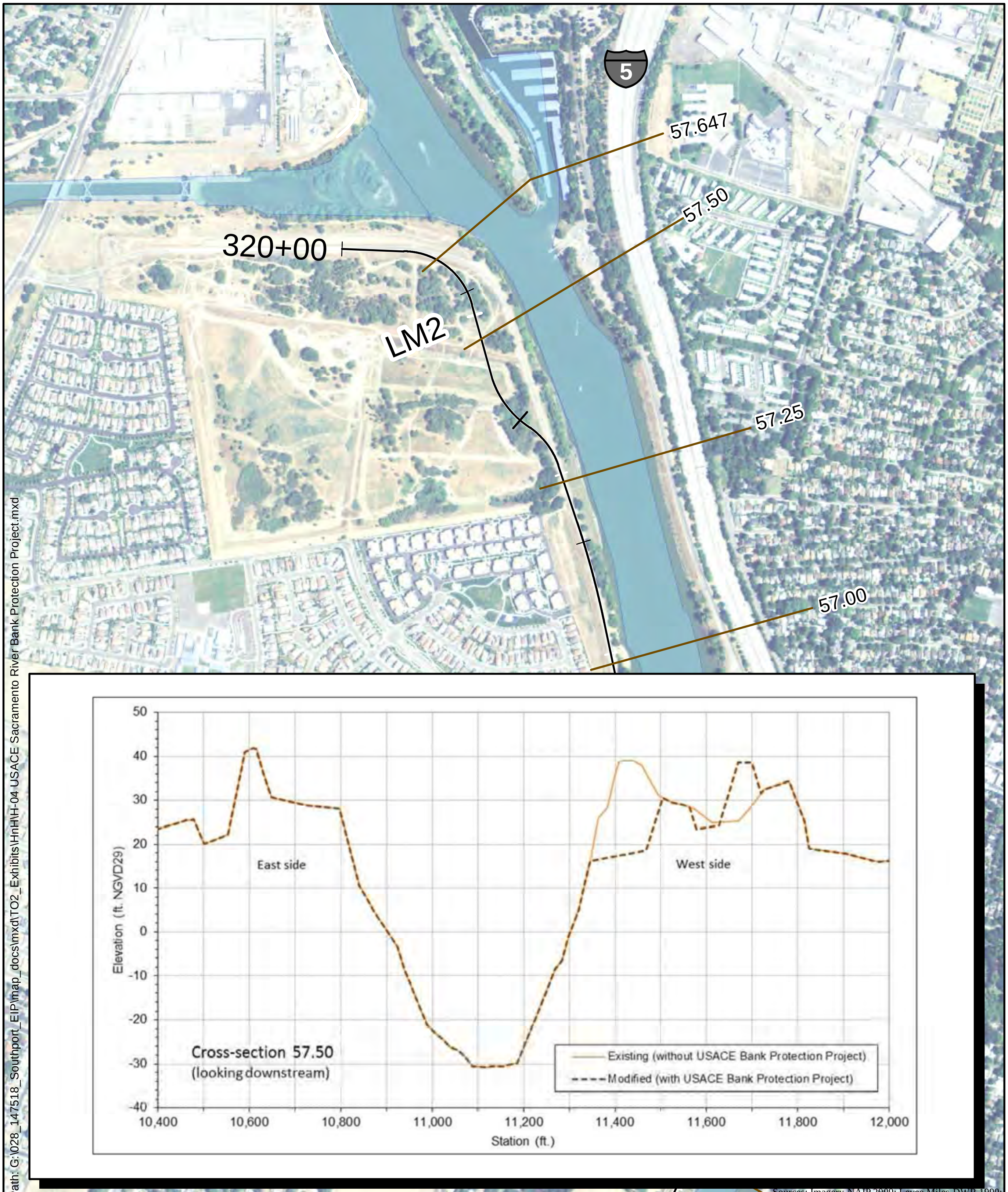
USACE RAS Model Extents

Exhibit H-3

Project Num: 453318-157784-028

May 2011

N W E S



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Levee Improvement Program Southport Early Implementation Project

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Checked
B. JONES, HDR

Drawn
A. ARNOLD, HDR

CROSS SECTIONS

PROJECT STATIONING

LM2 LEVEE MILES (DWR, 1990)

USACE Sacramento River Bank Protection Project

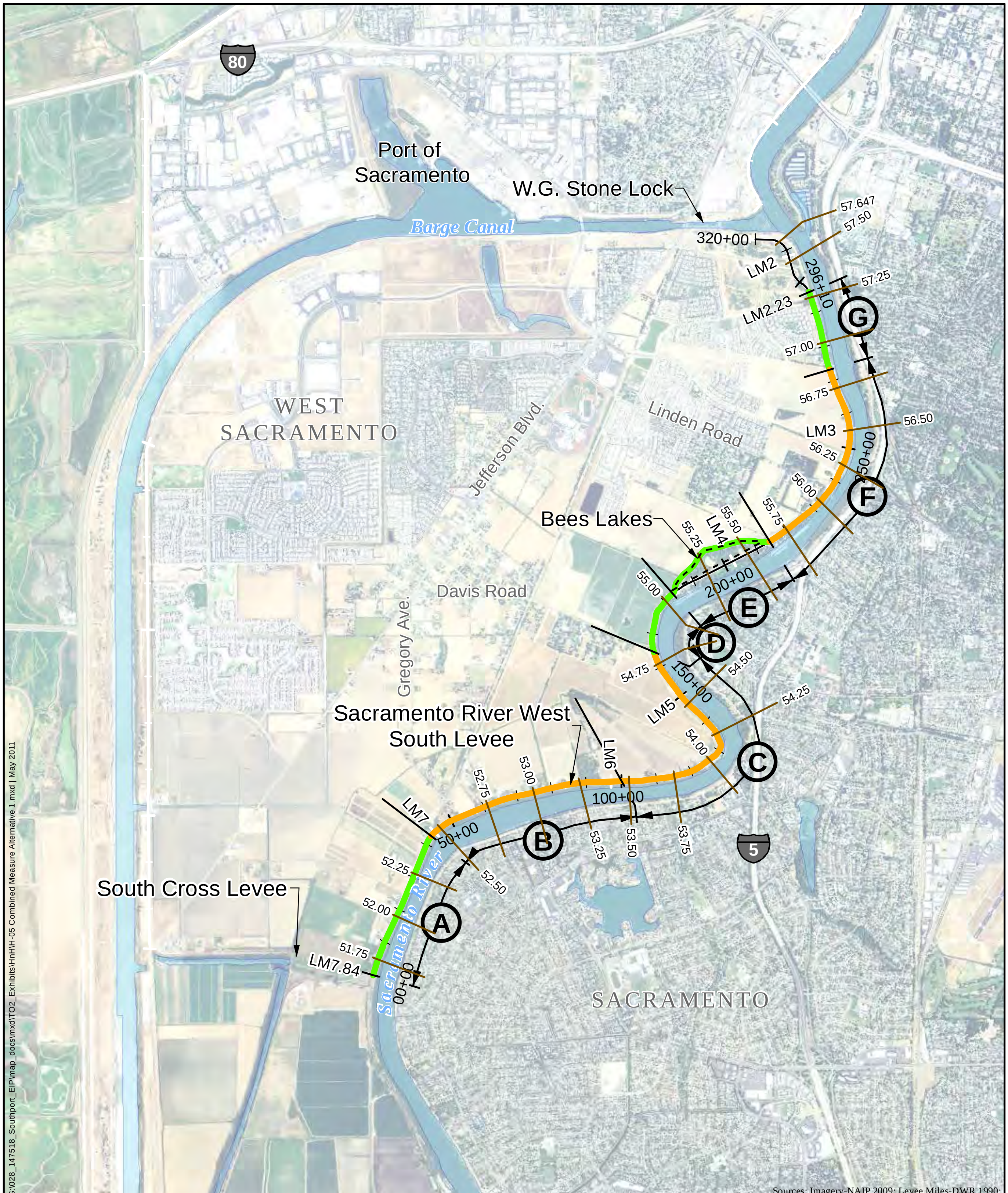
Exhibit H-4

Project Num: 453318-157784-028

May 2011

N
W E
S

0 500 1,000
Feet
1 in = 1,000 feet



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CMA 1 CROSS-SECTIONS
ADJACENT LEVEE WITH BERM
ADJACENT LEVEE WITH CUTOFF WALL

Combined Measure Alternative 1 (Adjacent Levee)

Exhibit H-5

Project Num: 453318-157784-028

May 2011

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Drawn
A. ARNOLD, HDR

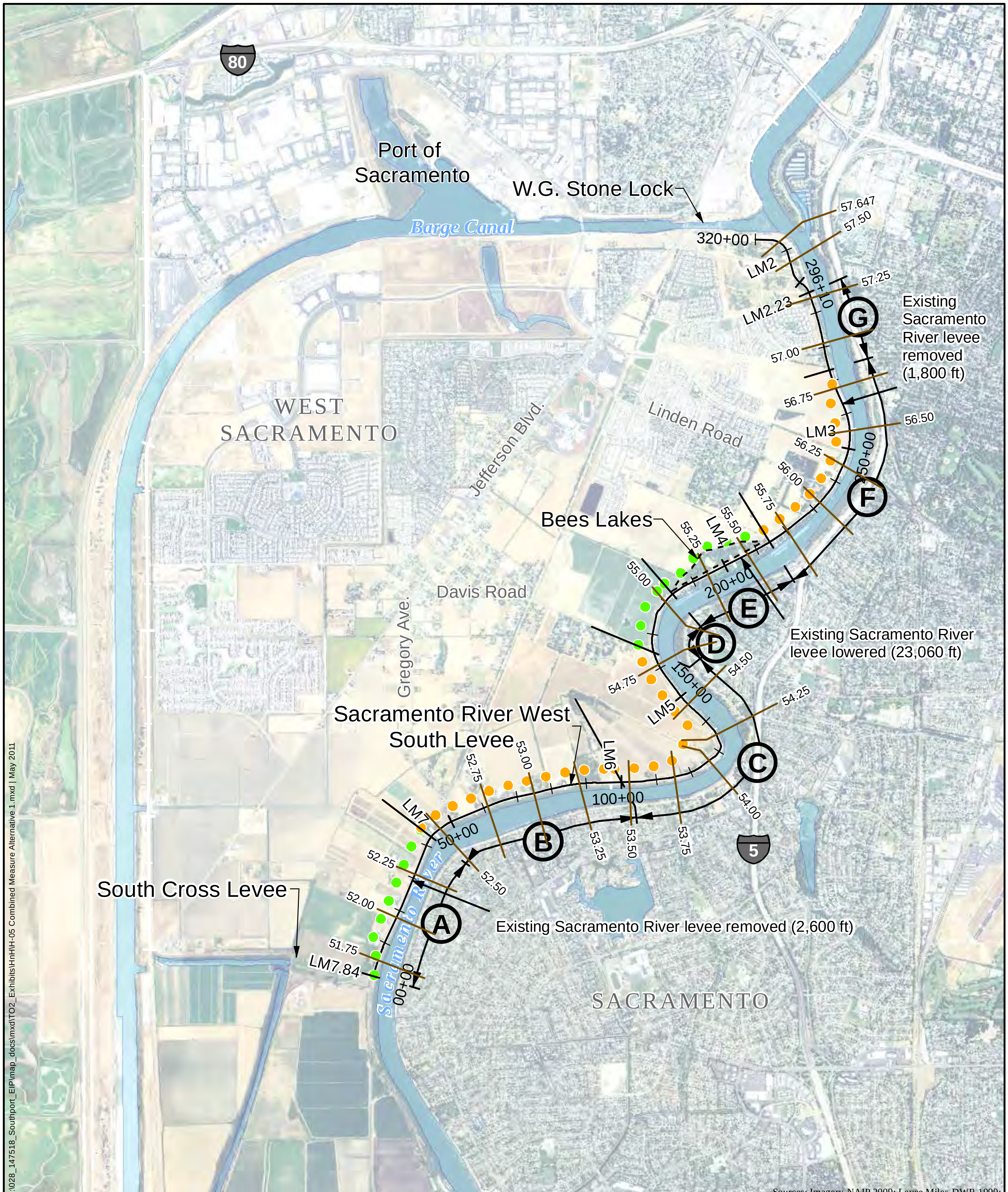
A COST EVALUATION SEGMENT

PROJECT STATIONING

LM2 LEVEE MILES (DWR, 1990)

N
W E
S

0 1,000 2,000 3,000 4,000
Feet
1 inch = 4,000 feet



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Levee Improvement Program Southport Early Implementation Project

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CMA 2 CROSS-SECTION

SETBACK LEVEE WITH BERM

SETBACK LEVEE WITH CUTOFF WALL

A COST EVALUATION SEGMENT

PROJECT STATIONING

LM2 LEVEE MILES (DWR, 1990)

Combined Measure Alternative 2 (400' Setback Levee)

Exhibit H-6

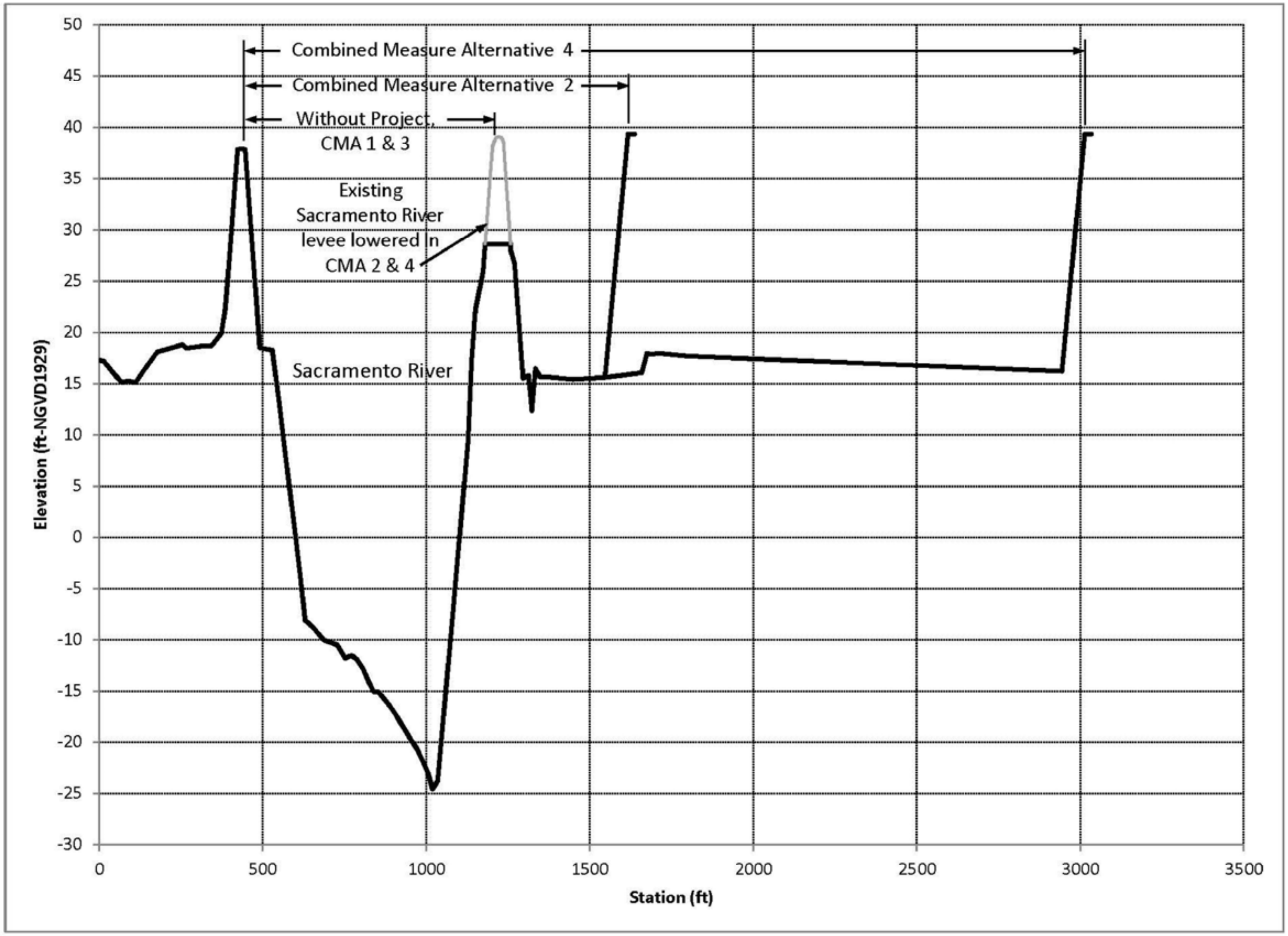
Project Num: 453318-157784-028

May 2011

N W E S

0 1,000 2,000 3,000 4,000
Feet
1 inch = 4,000 feet

[redacted]



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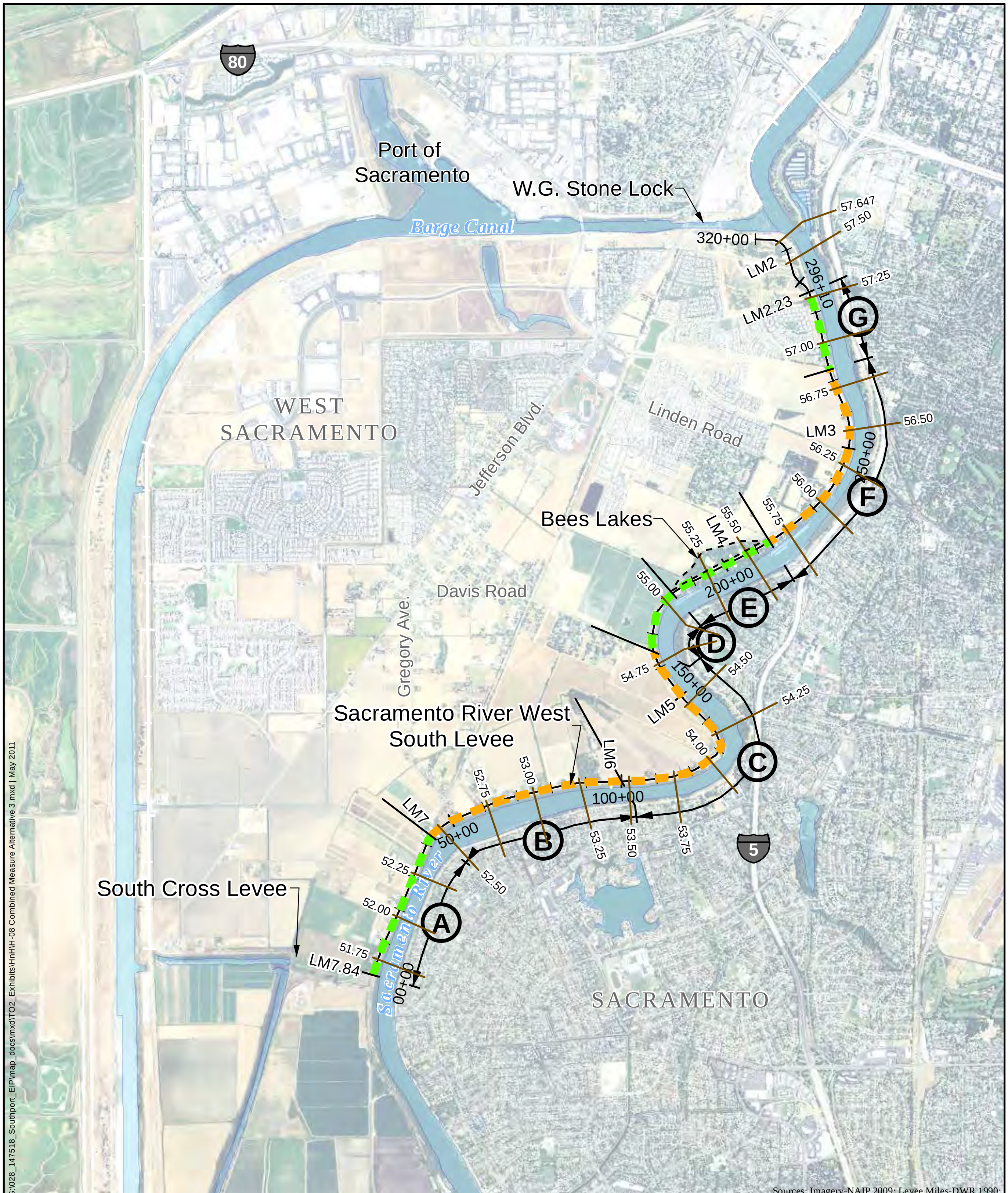
C. PLOTTS, HDR

Typical Cross-Sections, Levee Setbacks

Exhibit H-7

Project Num: 453318-147518-028

May 2011



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Levee Improvement Program Southport Early Implementation Project

Project Manager
M. VECCHIO, HDR
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G. PRESTON, MBK
Checked
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Drawn
A. ARNOLD, HDR

CMA 3 CROSS-SECTIONS
SLOPE FLATTENING WITH BERM
SLOPE FLATTENING WITH CUTOFF WALL

A COST EVALUATION SEGMENT

PROJECT STATIONING

LM2 LEVEE MILES (DWR, 1990)

Combined Measure Alternative 3 (In-Place Modifications)

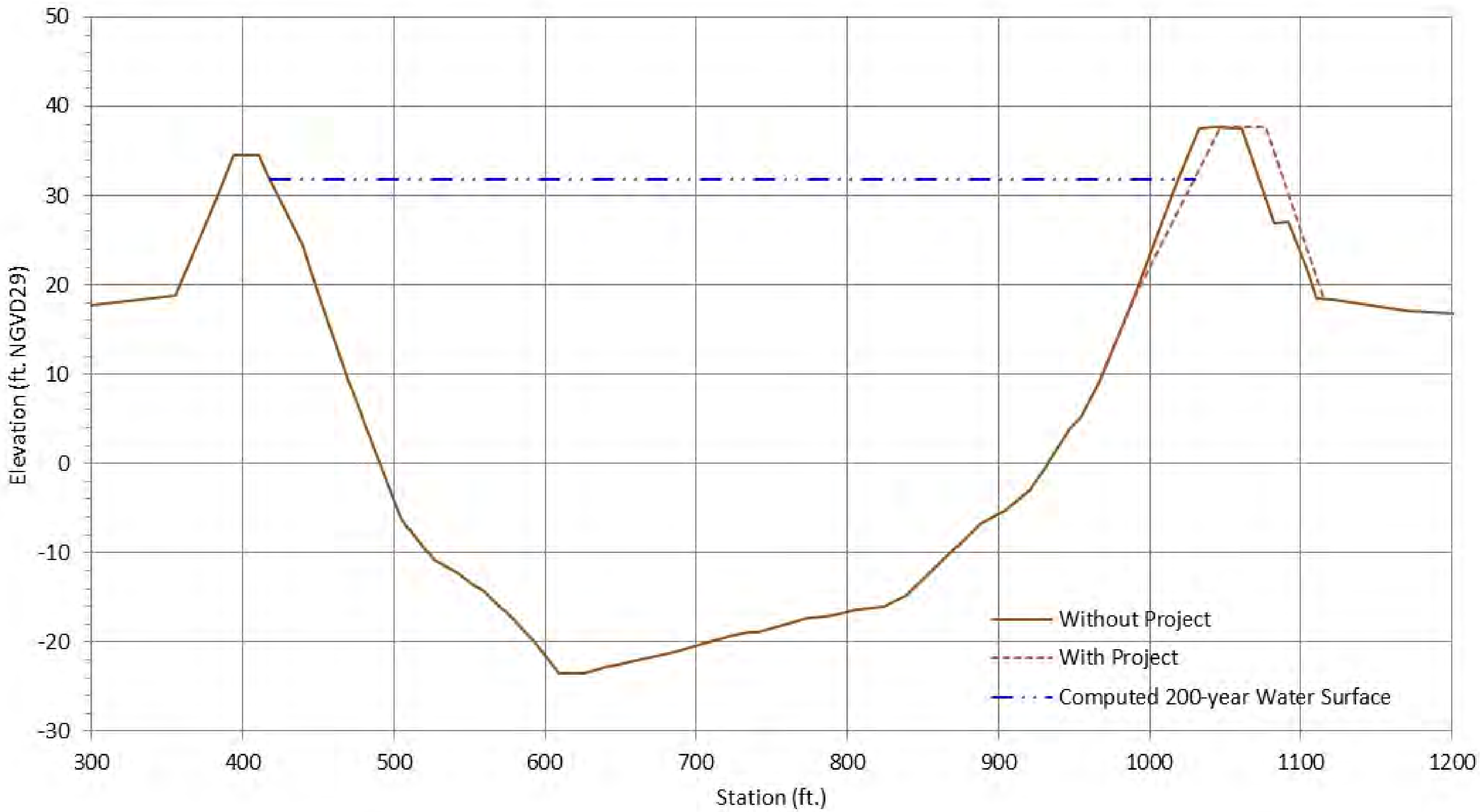
Exhibit H-8

Project Num: 453318-157784-028

May 2011

N
W E
S

0 1,000 2,000 3,000 4,000
Feet
1 inch = 4,000 feet



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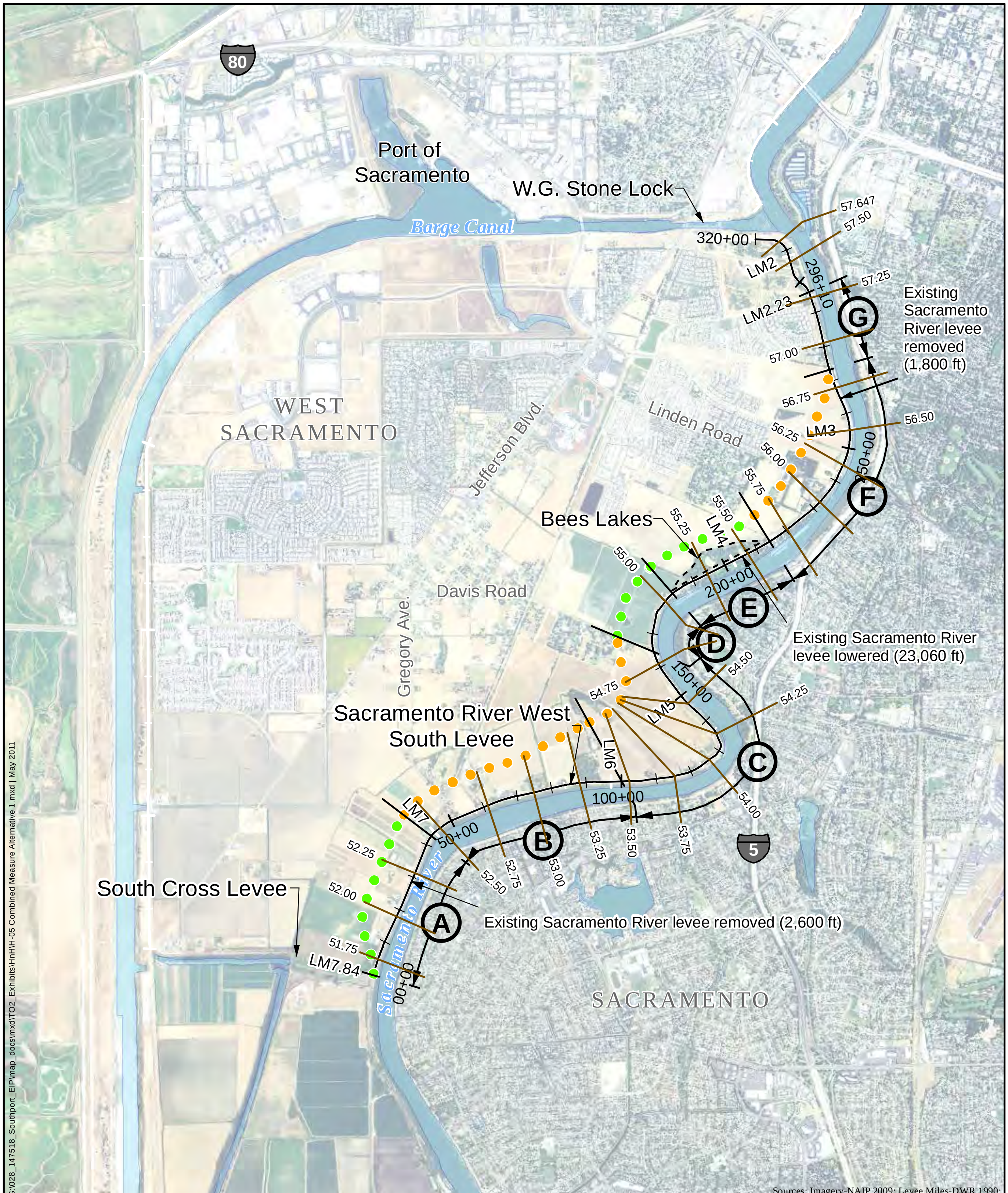
Checked	B. JONES, HDR
Drawn	C. PLOTTS, HDR

Typical Cross-Sections, CMA3

Exhibit H-9

Project Num: 453318-147518-028

May 2011



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M. VECCHIO, HDR

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CMA 4 CROSS-SECTION

SETBACK LEVEE WITH BERM

SETBACK LEVEE WITH CUTOFF WALL

A **COST EVALUATION SEGMENT**

PROJECT STATIONING

LM2 **LEVEE MILES (DWR, 1990)**

Combined Measure Alternative 4 - (1,000' Setback Alternative)

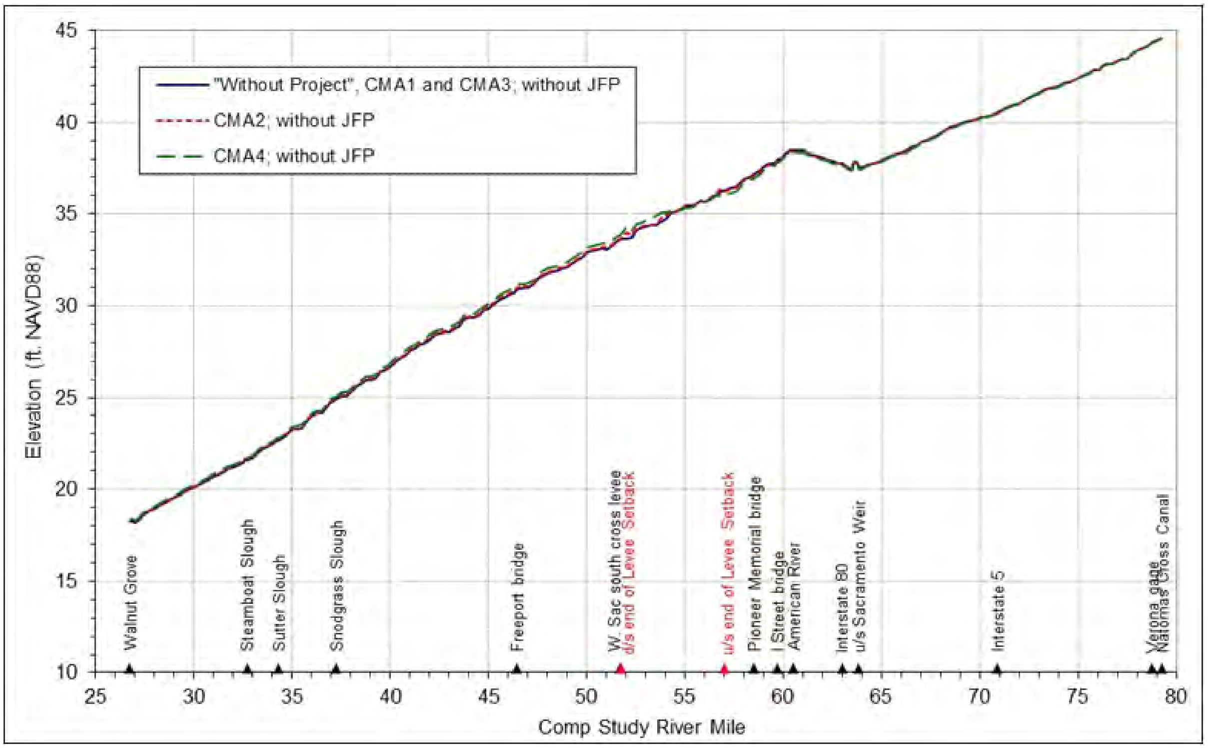
Exhibit H-10

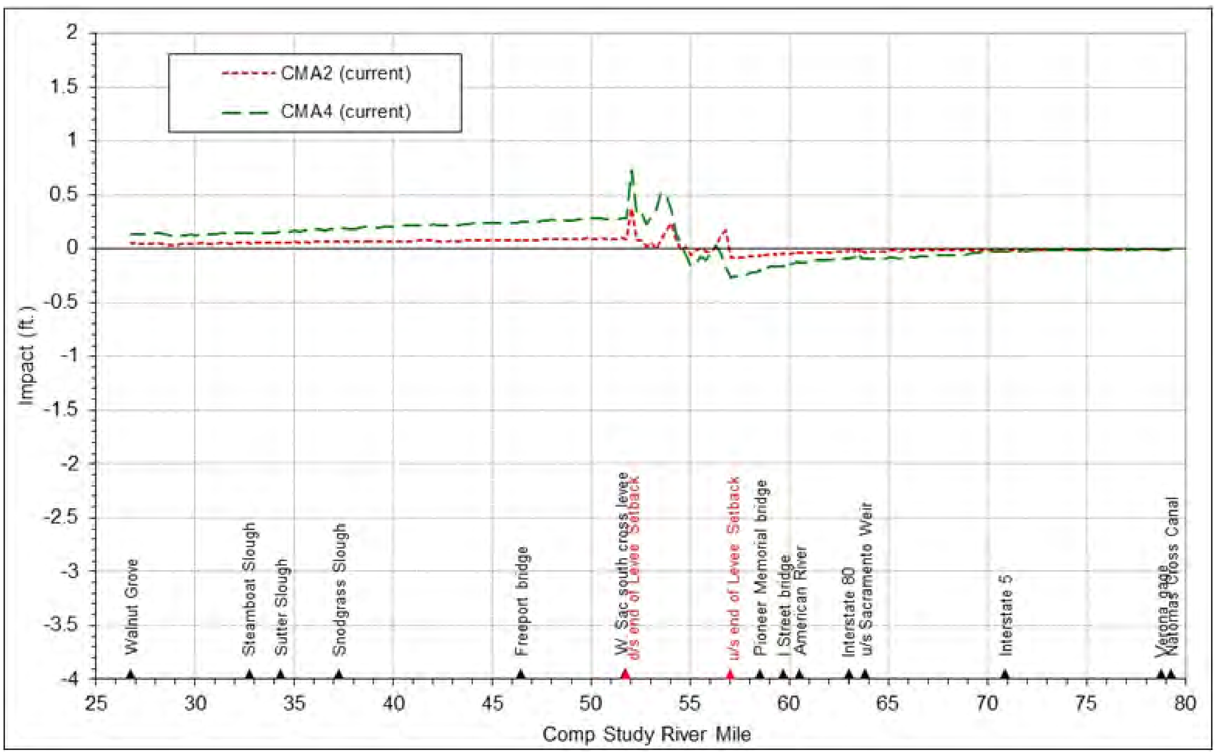
Project Num: 453318-157784-028

May 2011

N W E S

0 1,000 2,000 3,000 4,000
Feet
1 inch = 4,000 feet





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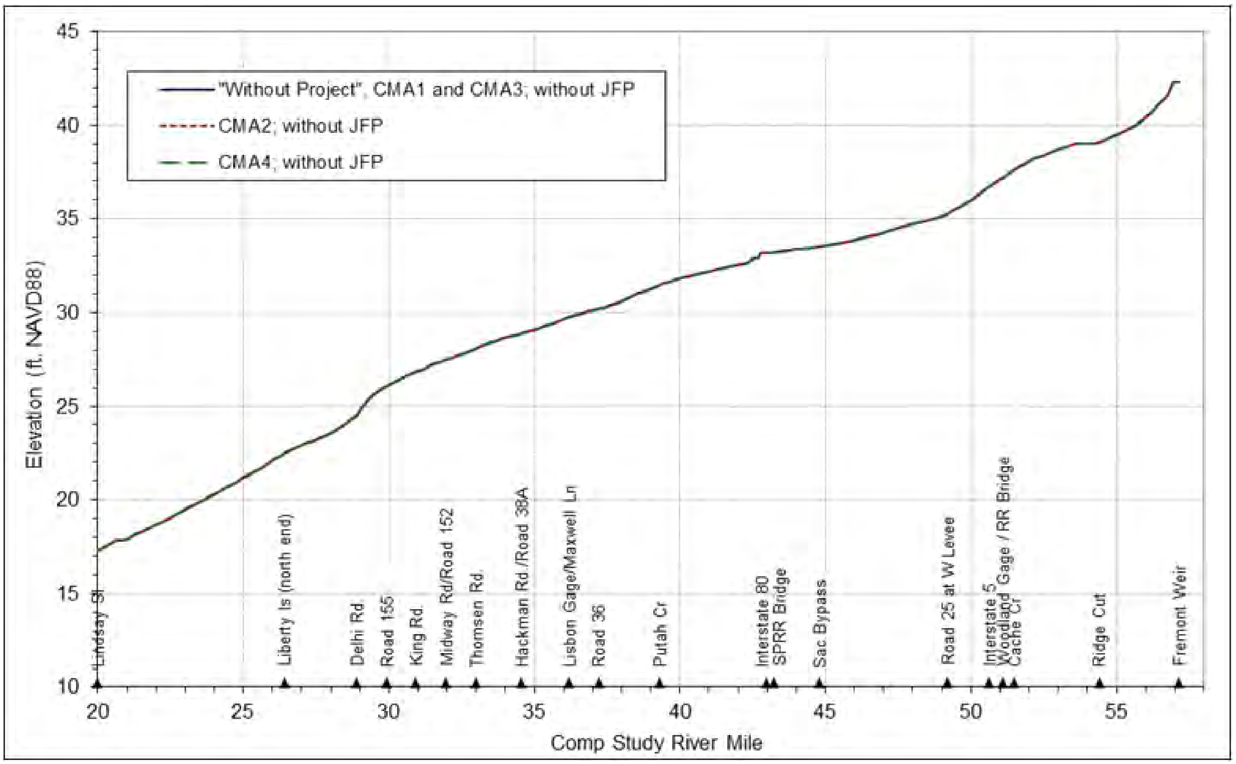
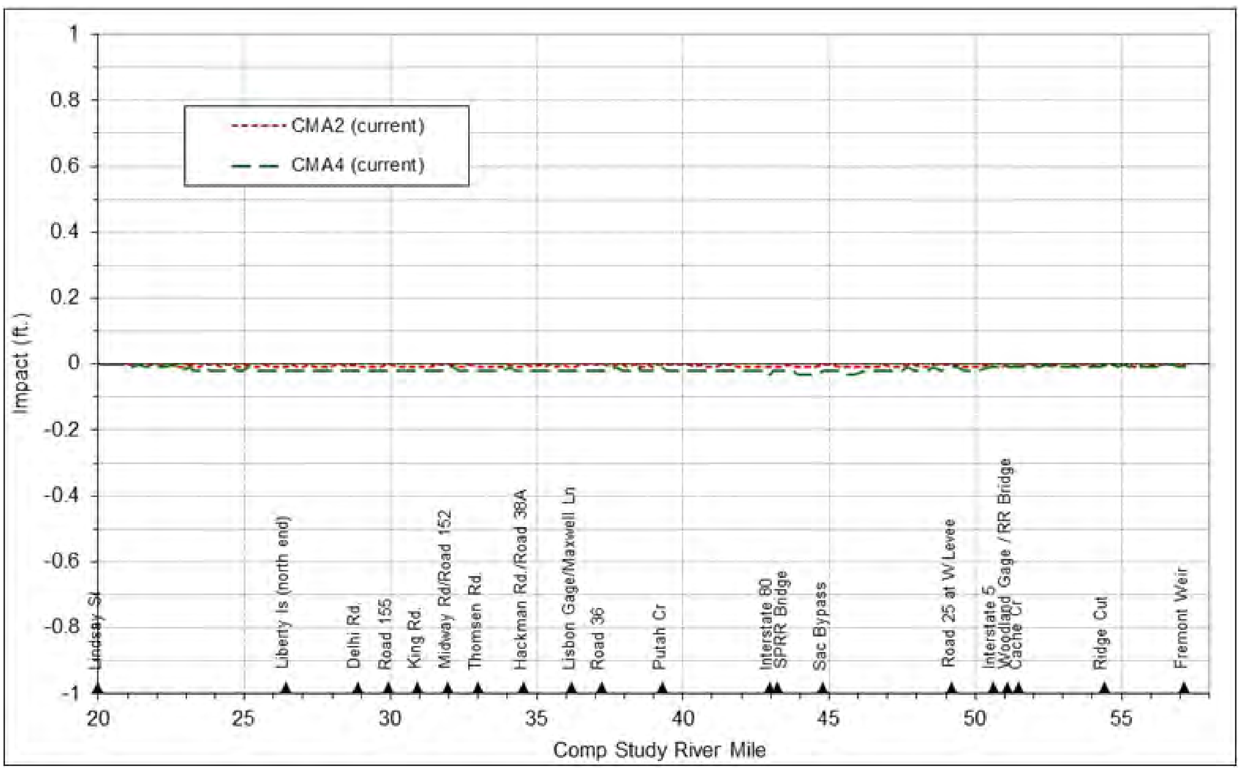
Maximum WSE and Impact Profiles, Sacramento River Current Conditions

Exhibit H-11

Project Num: 453318-157784-028

May 2011





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Levee Improvement Program Southport Early Implementation Project

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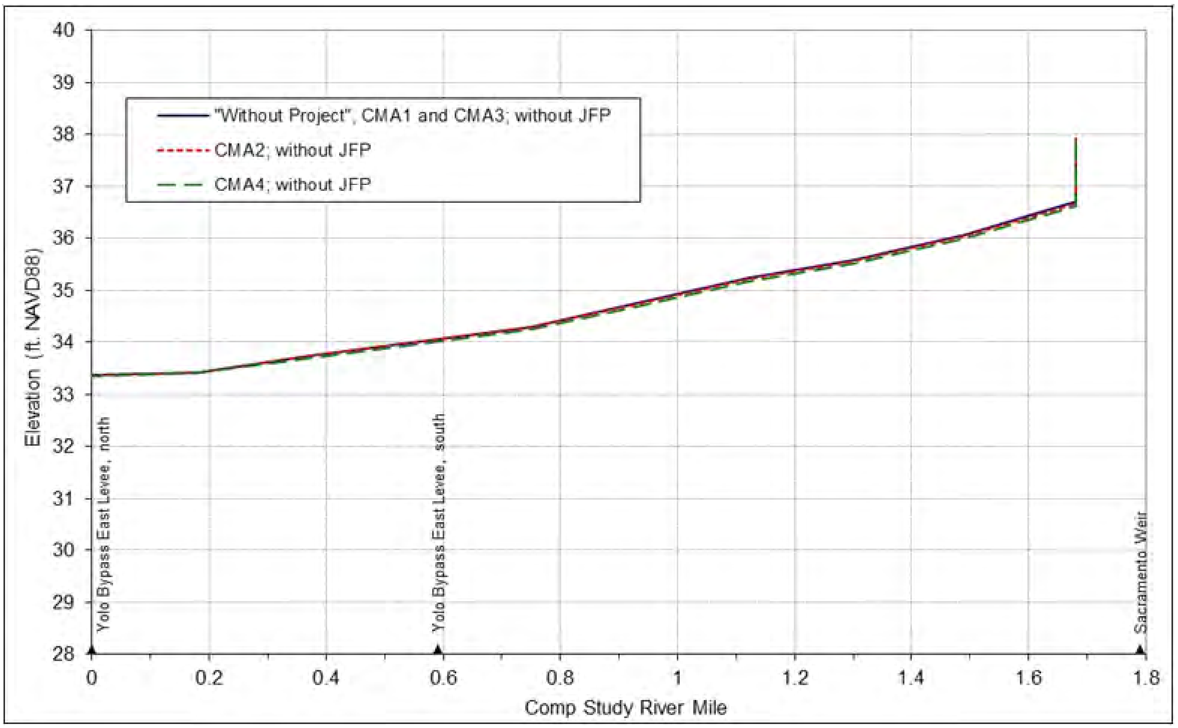
Drawn
C. PLOTTS, HDR

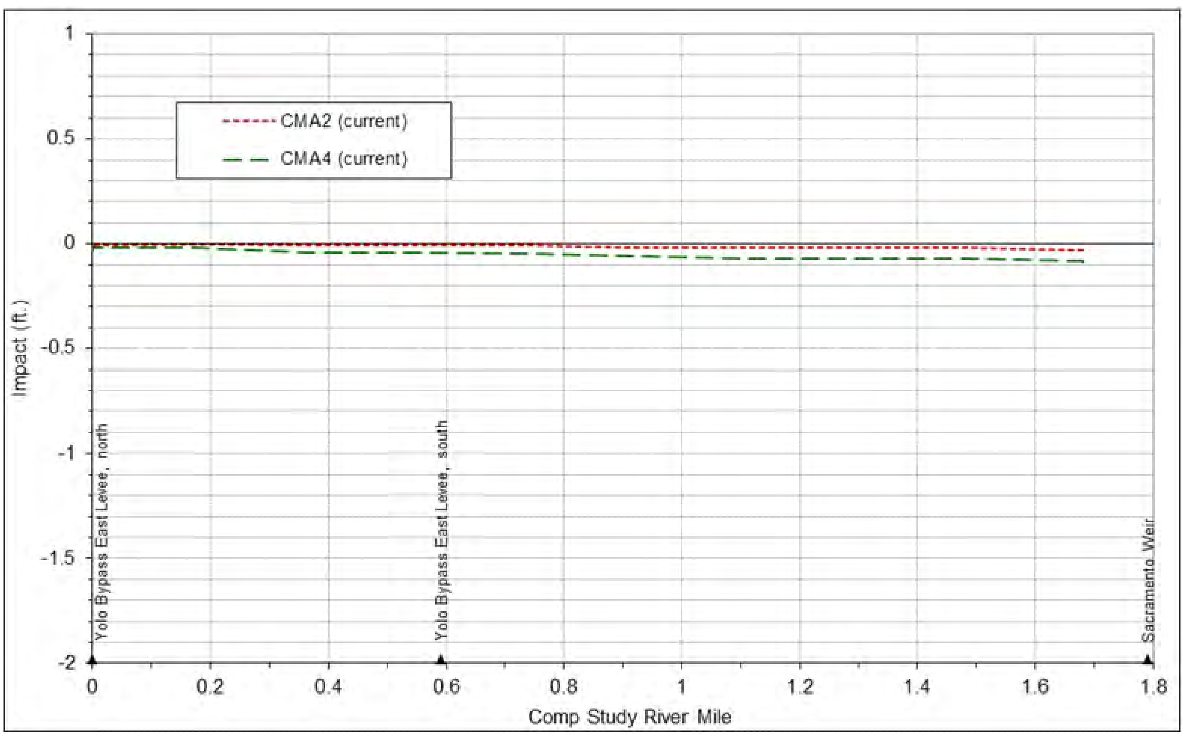
Maximum WSE and Impact Profiles, Yolo Bypass Current Conditions

Exhibit H-12

Project Num: 453318-157784-028

May 2011





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Levee Improvement Program Southport Early Implementation Project

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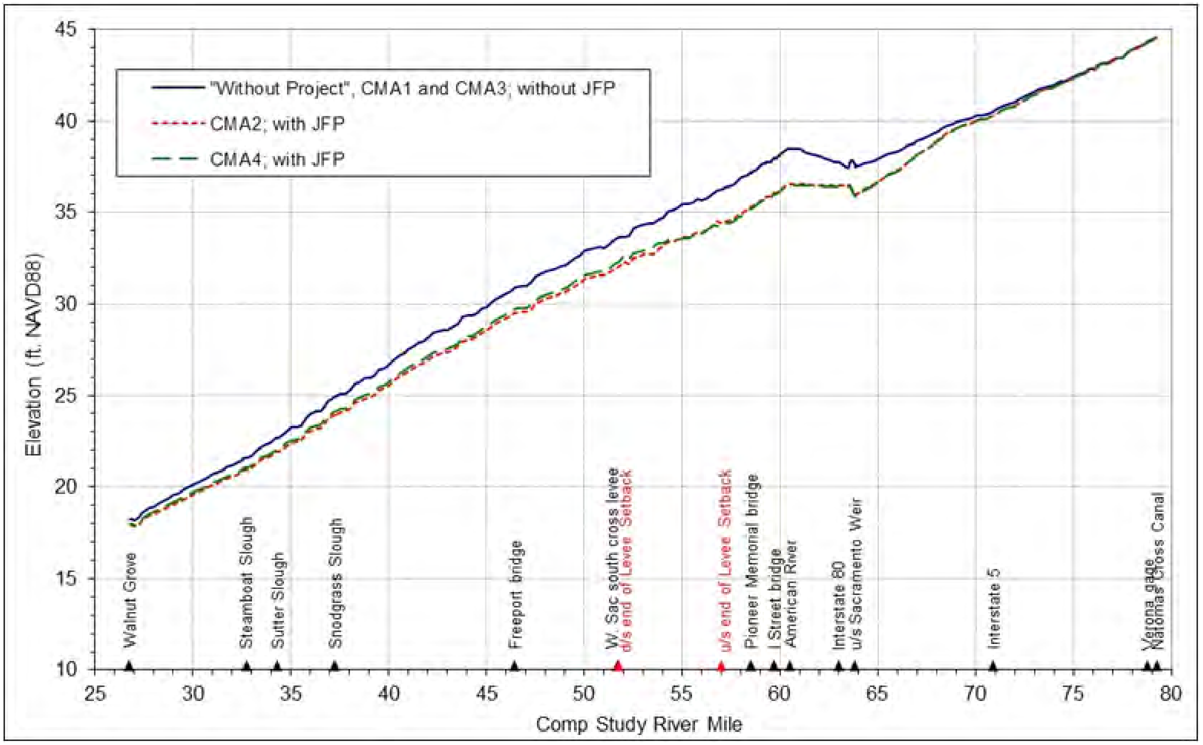
C. PLOTTS, HDR

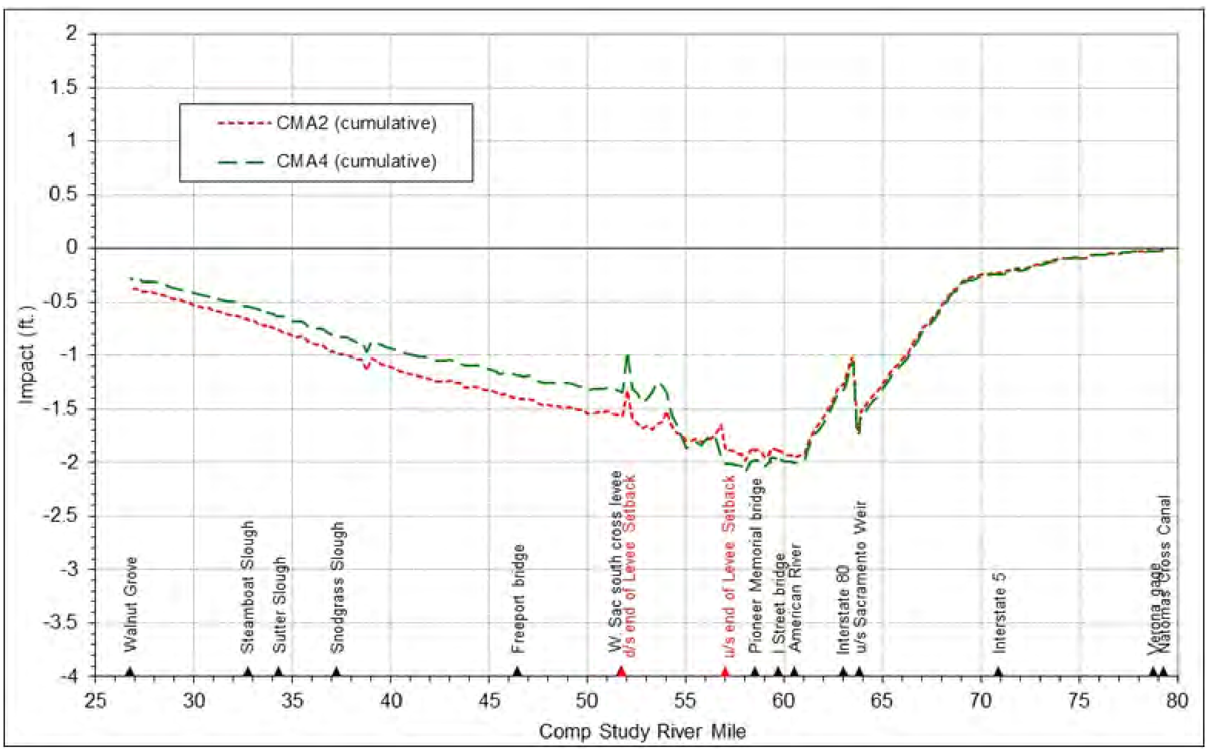
Maximum WSE and Impact Profiles, Sacramento Bypass Current Conditions

Exhibit H-13

Project Num: 453318-157784-028

May 2011





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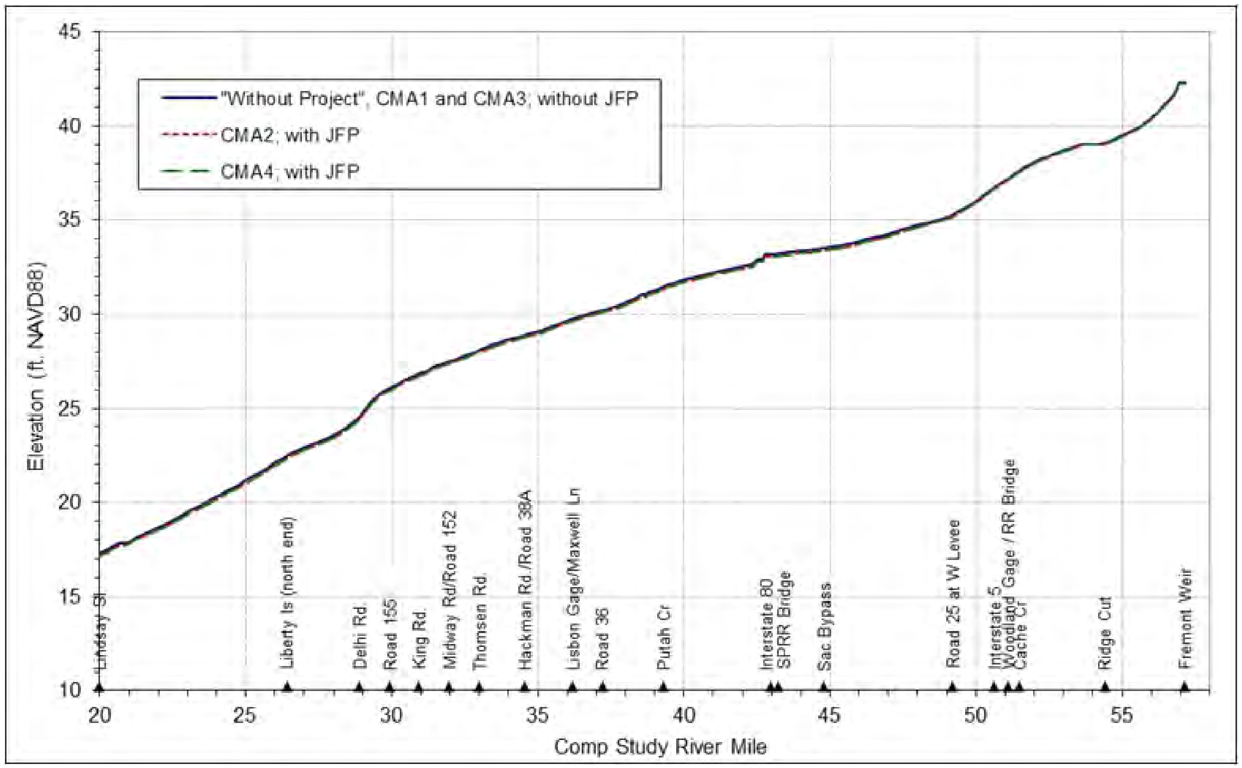
C. PLOTTS, HDR

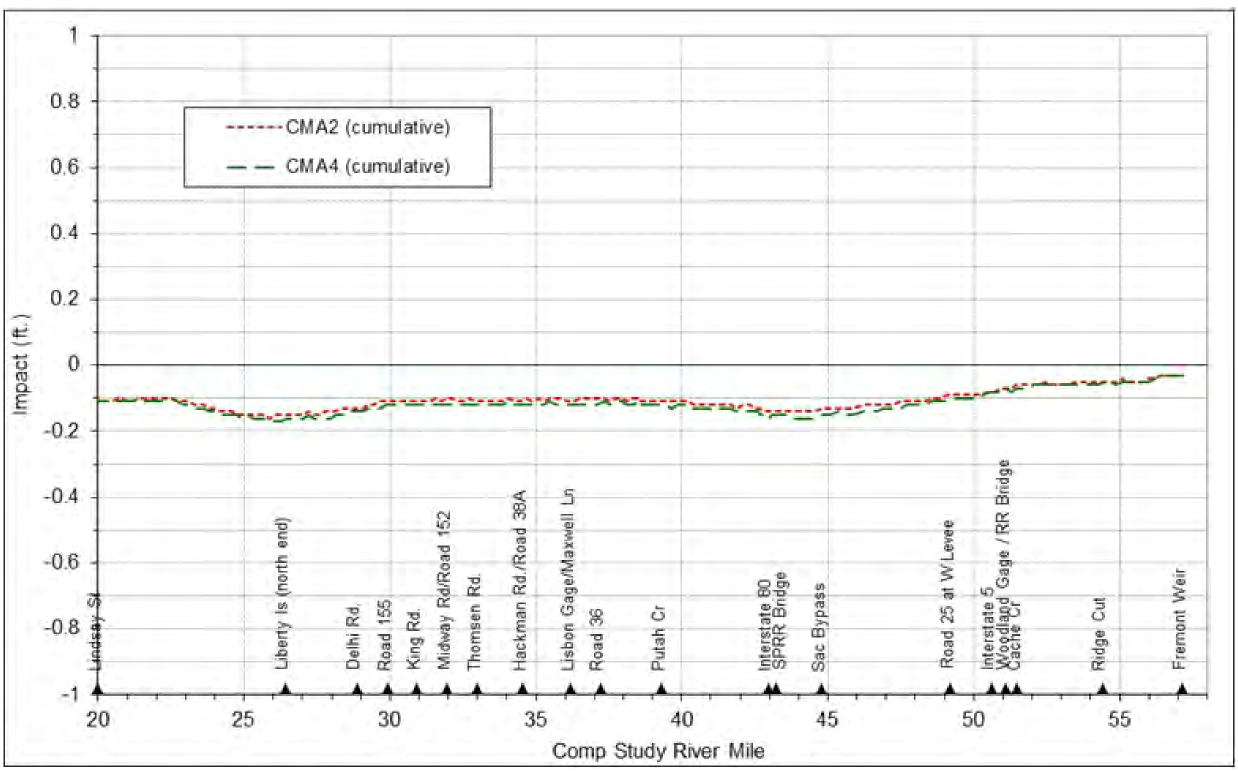
Maximum WSE and Impact Profiles, Sacramento River Cumulative Analysis

Exhibit H-14

Project Num: 453318-157784-028

May 2011





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Levee Improvement Program Southport Early Implementation Project

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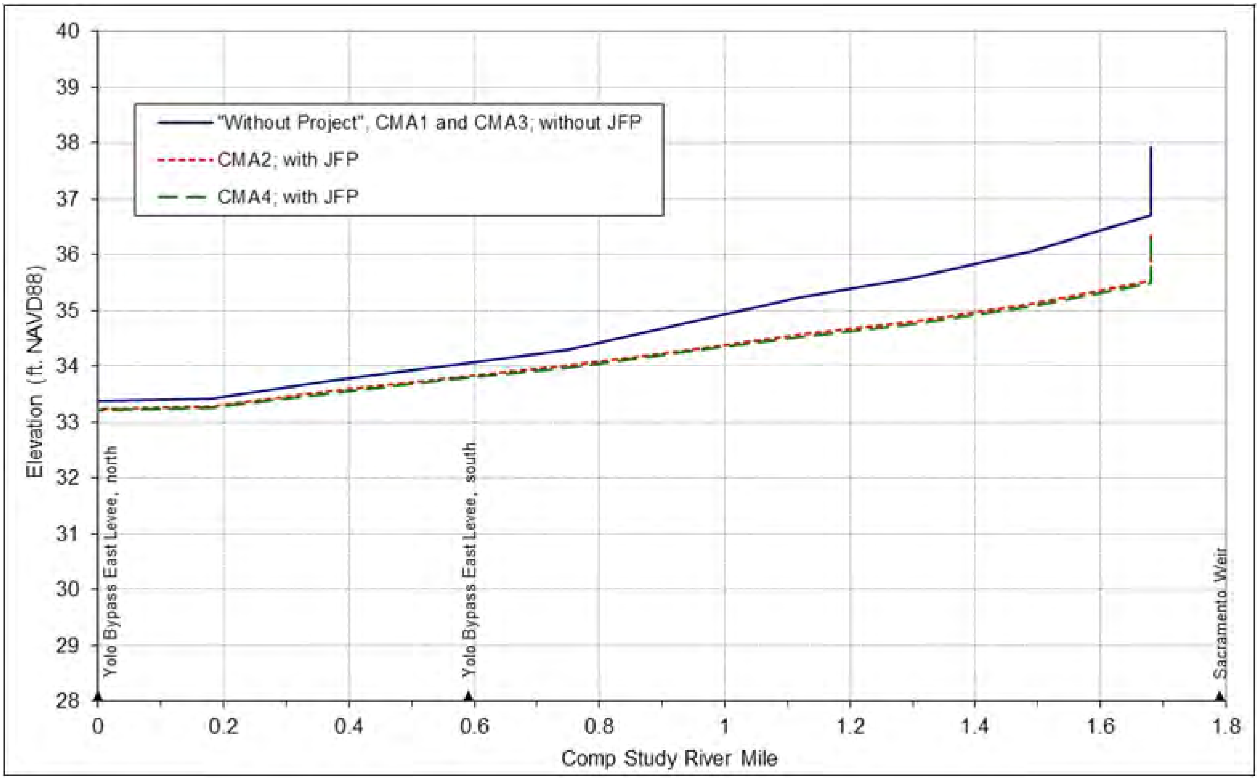
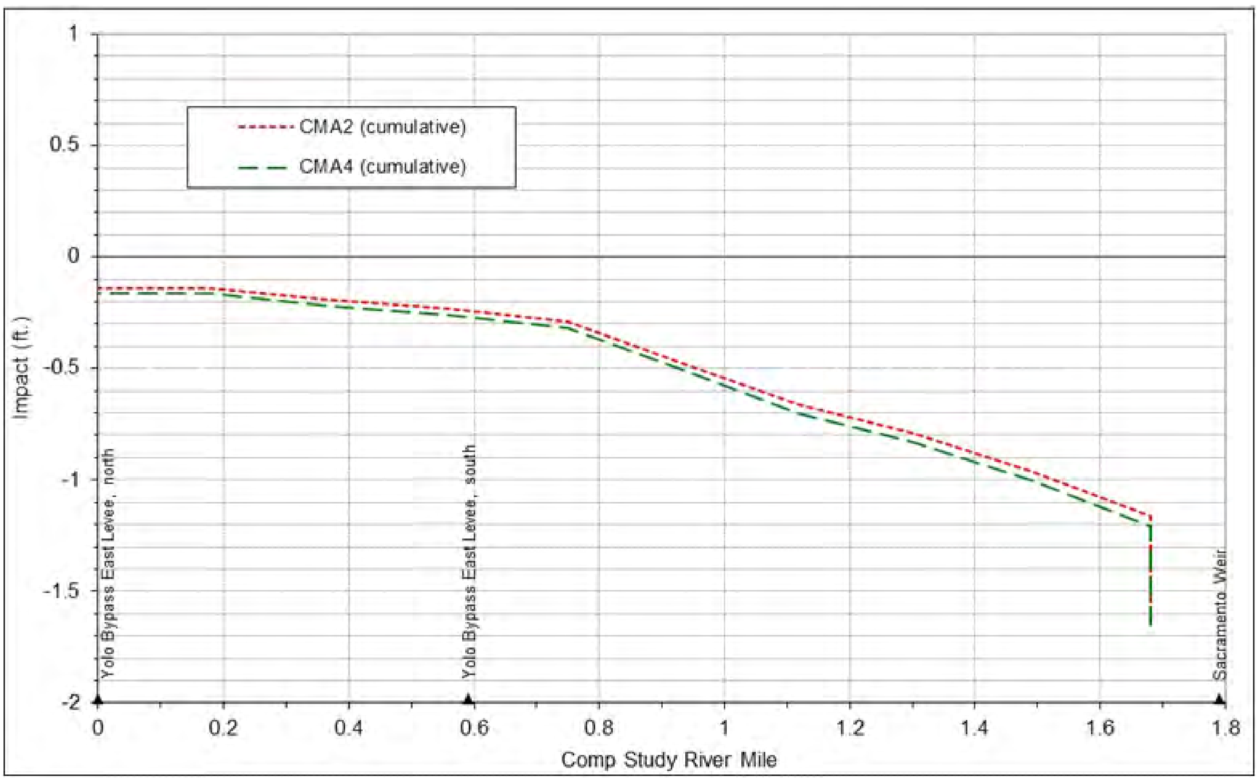
Maximum WSE and Impact Profiles, Yolo Bypass Cumulative Analysis

Exhibit H-15

Project Num: 453318-157784-028

May 2011





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Levee Improvement Program Southport Early Implementation Project

Project Manager
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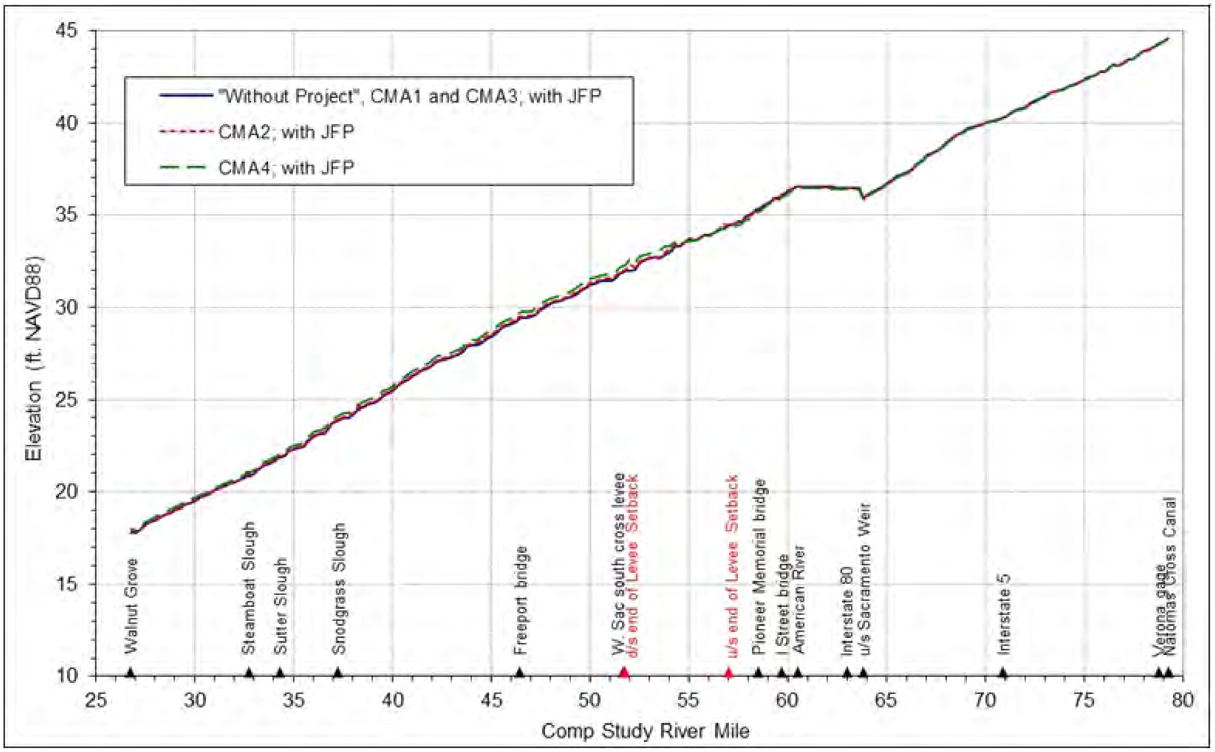
C. PLOTTS, HDR

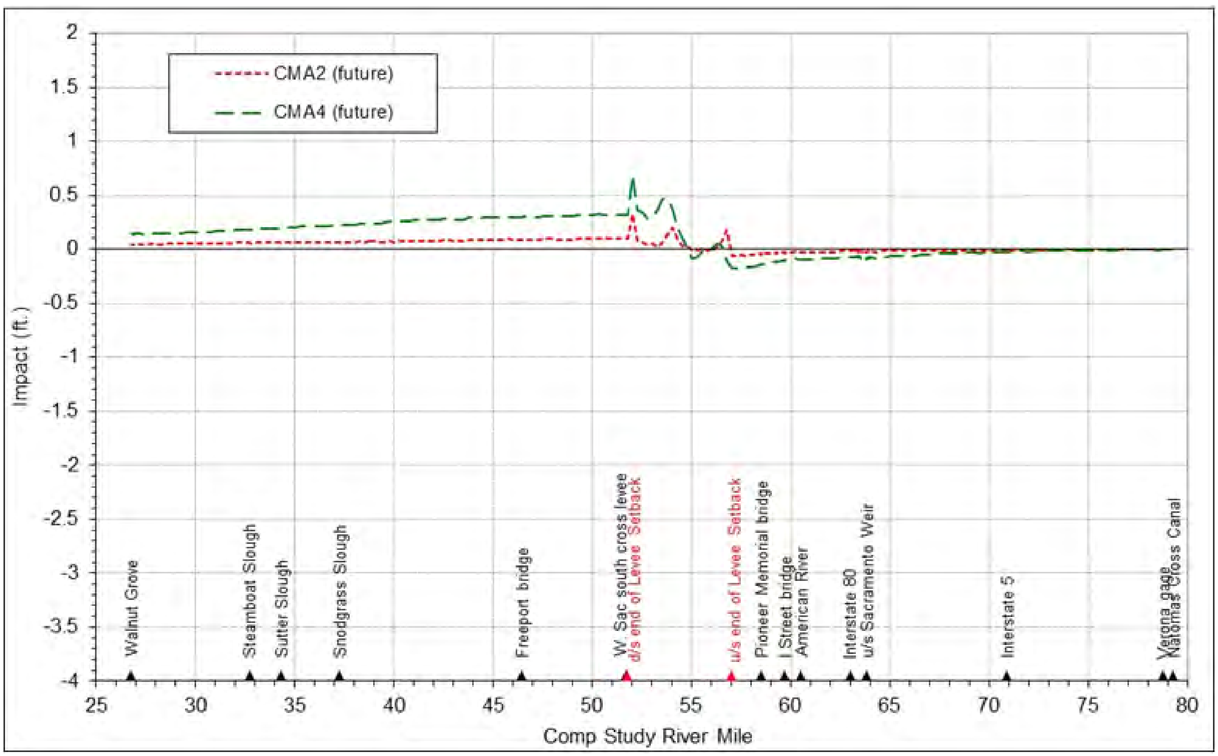
Maximum WSE and Impact Profiles, Sacramento Bypass Cumulative Analysis

Exhibit H-16

Project Num: 453318-157784-028

May 2011





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Project Manager
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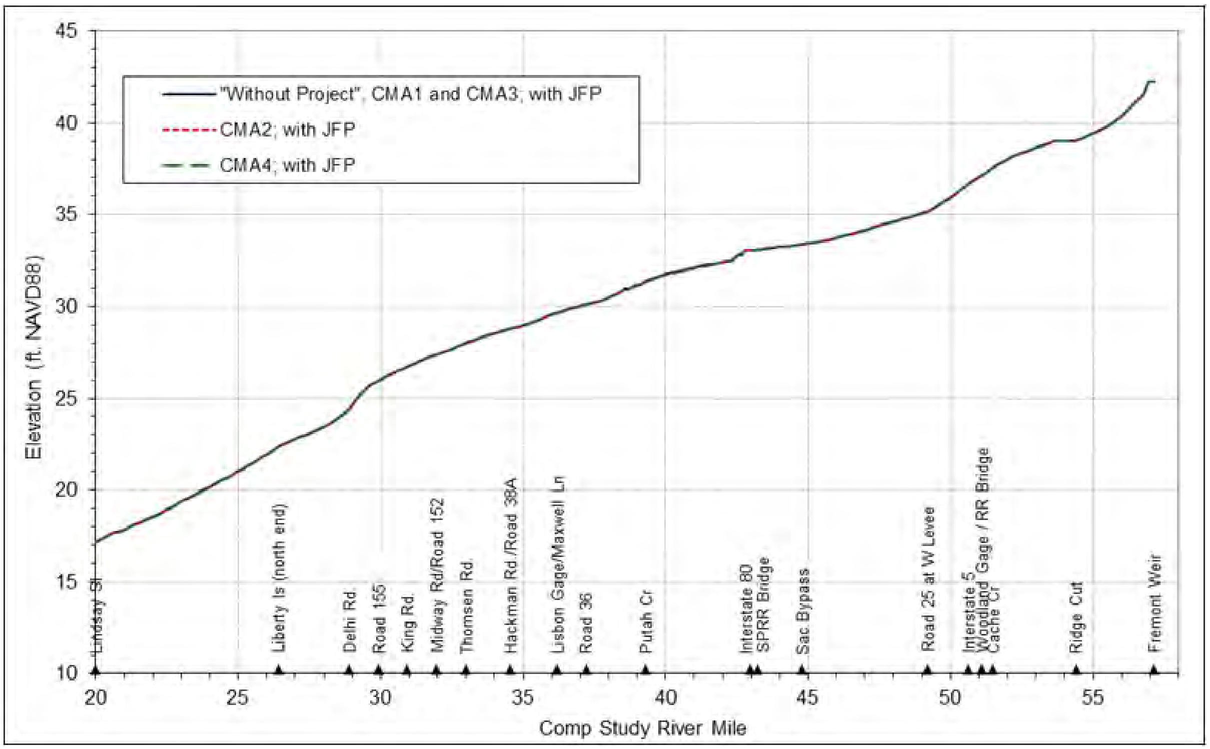
C. PLOTTS, HDR

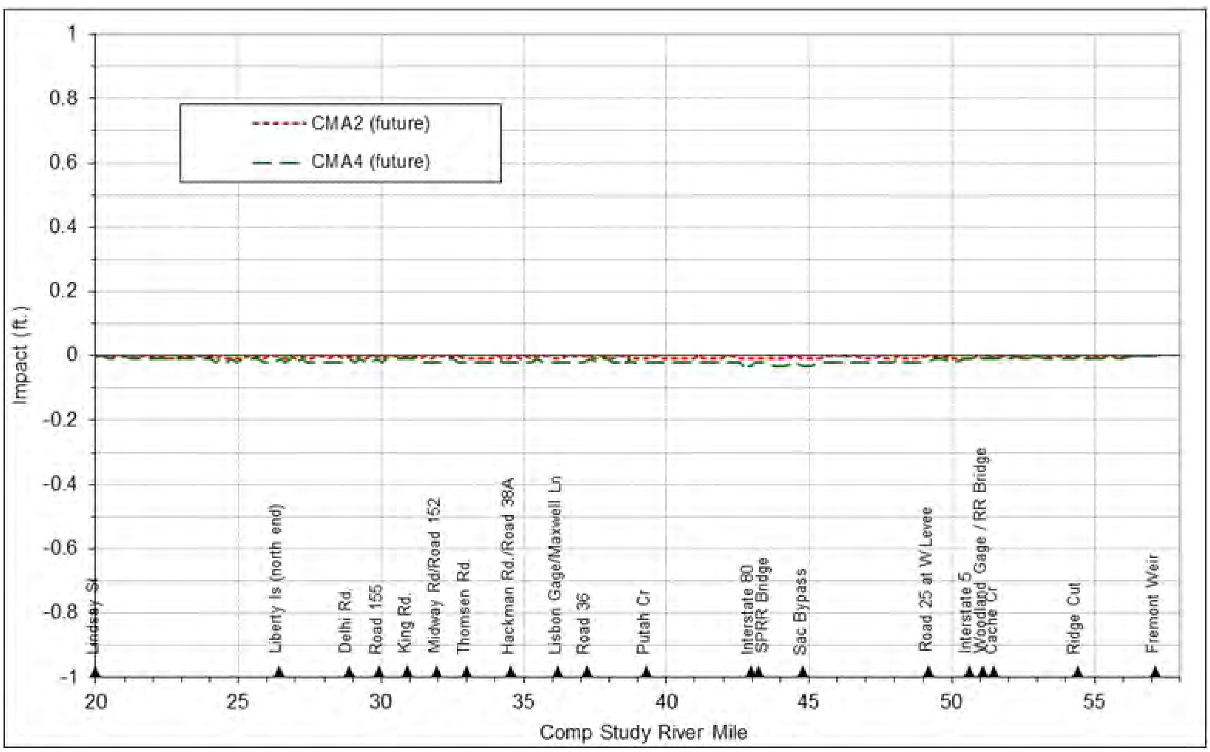
Maximum WSE and Impact Profiles, Sacramento River Future Conditions

Exhibit H-17

Project Num: 453318-157784-028

May 2011





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Levee Improvement Program Southport Early Implementation Project

Project Manager
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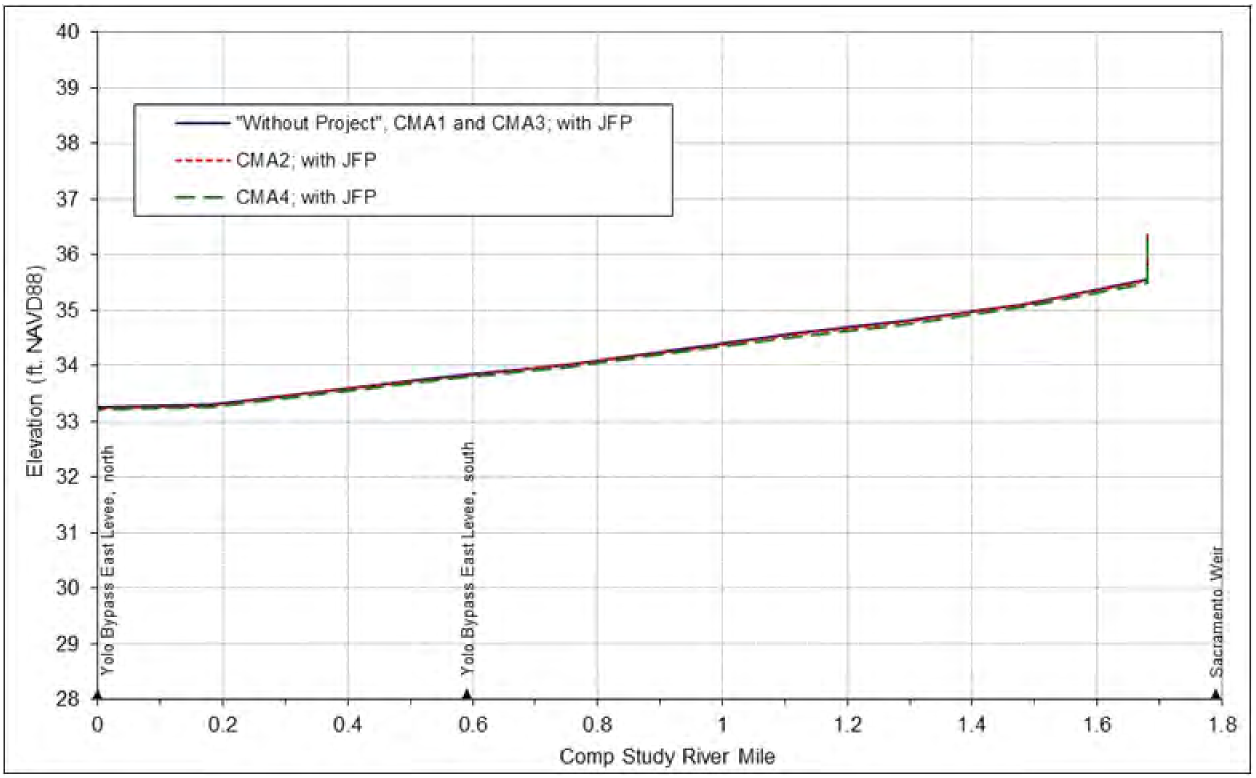
C. PLOTTS, HDR

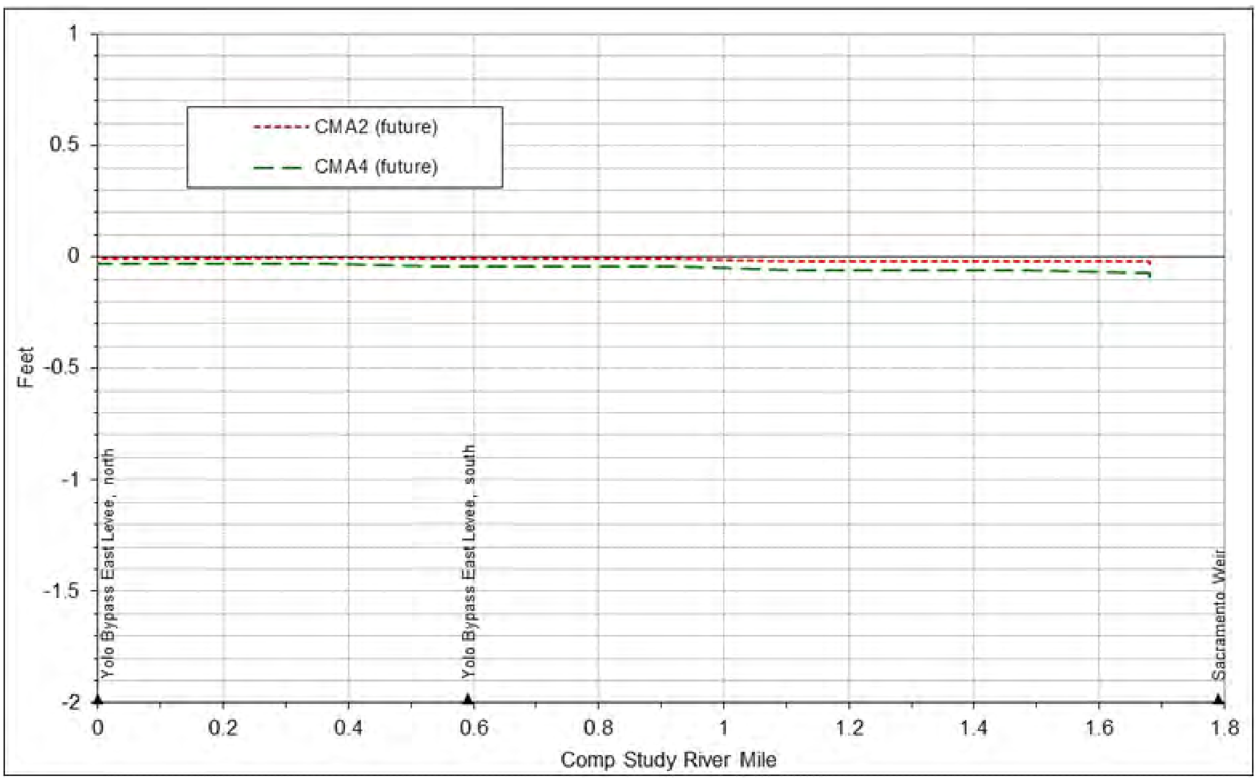
Maximum WSE and Impact Profiles, Yolo Bypass Future Conditions

Exhibit H-18

Project Num: 453318-157784-028

May 2011





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Levee Improvement Program Southport Early Implementation Project

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C. PLOTTS, HDR

Maximum WSE and Impact Profiles, Sacramento Bypass Future Conditions

Exhibit H-19

Project Num: 453318-157784-028

May 2011

ATTACHMENT A

MBK ENGINEERS

Water Resources • Flood Control • Water Rights

TECHNICAL MEMORANDUM

DATE: May 13, 2011

SUBJECT: USACE Sacramento River Basin HEC-RAS Model Calibration Refinement for West Sacramento Analyses

Prepared by: George Preston, P.E.

Reviewed by: Michael Archer, P.E.

The U.S. Army Corps of Engineers (USACE) developed a HEC-RAS model of the Sacramento River Basin (Model) (USACE 2010). Release 2 of the Model was made available by USACE in December 2010. The USACE release memo for the Model is provided in Attachment 1. The Model was calibrated to the January 1997 flood event. The extents of the Model are shown in Figure 1.

MBK Engineers (MBK) reviewed the Model for application with West Sacramento hydraulic analyses. The Model, as provided by the USACE, was developed and run using HEC-RAS 4.0. The MBK review and analysis were performed using HEC-RAS 4.1. Prior to the calibration evaluation, a HEC-RAS computational review was performed comparing computed model results from HEC-RAS 4.0 and 4.1, which showed essentially no differences.

Maximum water surface elevations computed with the Model compared well with observed data for the Sacramento River in the vicinity of West Sacramento (see Figure 2), while maximum water surfaces in the Yolo Bypass were overestimated, as shown in Figure 3. A review of peak flows, observed and computed, revealed the Model had a reasonable peak flow in the Sacramento River downstream of the Natomas Cross Canal, but that the peak flow in the Yolo Bypass was significantly overestimated, as shown in Table 1.

Table 1. USACE Model Peak Flow Comparison for Latitude of Verona

Gage Name	Gage ID	Observed Peak Flow (cfs)	Computed Peak Flow (cfs)	Percent Difference
Sacramento River at Verona	11425500 (USGS)	102,000	104,300	+2.3%
Yolo Bypass near Woodland	11453000 (USGS)	357,000	404,200	+13.2%

Based on a review of the Model input flows for the January 1997 calibration event, it was concluded that the input flow for the Sutter Bypass was the source of the most uncertainty. In order to improve the comparison of observed and computed flow values at the latitude of Verona, the model input flow for the Sutter Bypass was reduced 38%.

To improve the calibration of the Fremont Weir-Sacramento River flow split, MBK modified the Fremont Weir in the Model by changing the weir shape from “Broad Crested” to “Sharp Crested” and increased the weir coefficient from 1.4 to 1.8. Additionally, the Tisdale Weir in the model was modified by changing the weir shape from “Broad Crested” to “Sharp Crested” and the weir coefficient was increased from 3.0 to 3.2.

The following table and figures present computed model results from the refined model. Peak flows at the latitude of Verona are summarized in Table 2. Maximum water surface profiles are shown in Figures 4 and 5. Stage and flow hydrographs along with observed data in the vicinity of West Sacramento are shown in Figures 6 through 13.

Table 2. MBK Refined Model Peak Flow Comparison for Latitude of Verona

Gage Name	Gage ID	Observed Peak Flow (cfs)	Computed Peak Flow (cfs)	Percent Difference
Sacramento River at Verona	11425500 (USGS)	102,000	103,100	+1.1%
Yolo Bypass near Woodland	11453000 (USGS)	357,000	356,900	+0.0%

References

Unites States Army Corps of Engineers (USACE). Post-Authorization Change Report And Interim General Reevaluation Report, American River Watershed Common Features Project, Appendix C - Hydraulic Technical Documentation, August 2010.

Figures

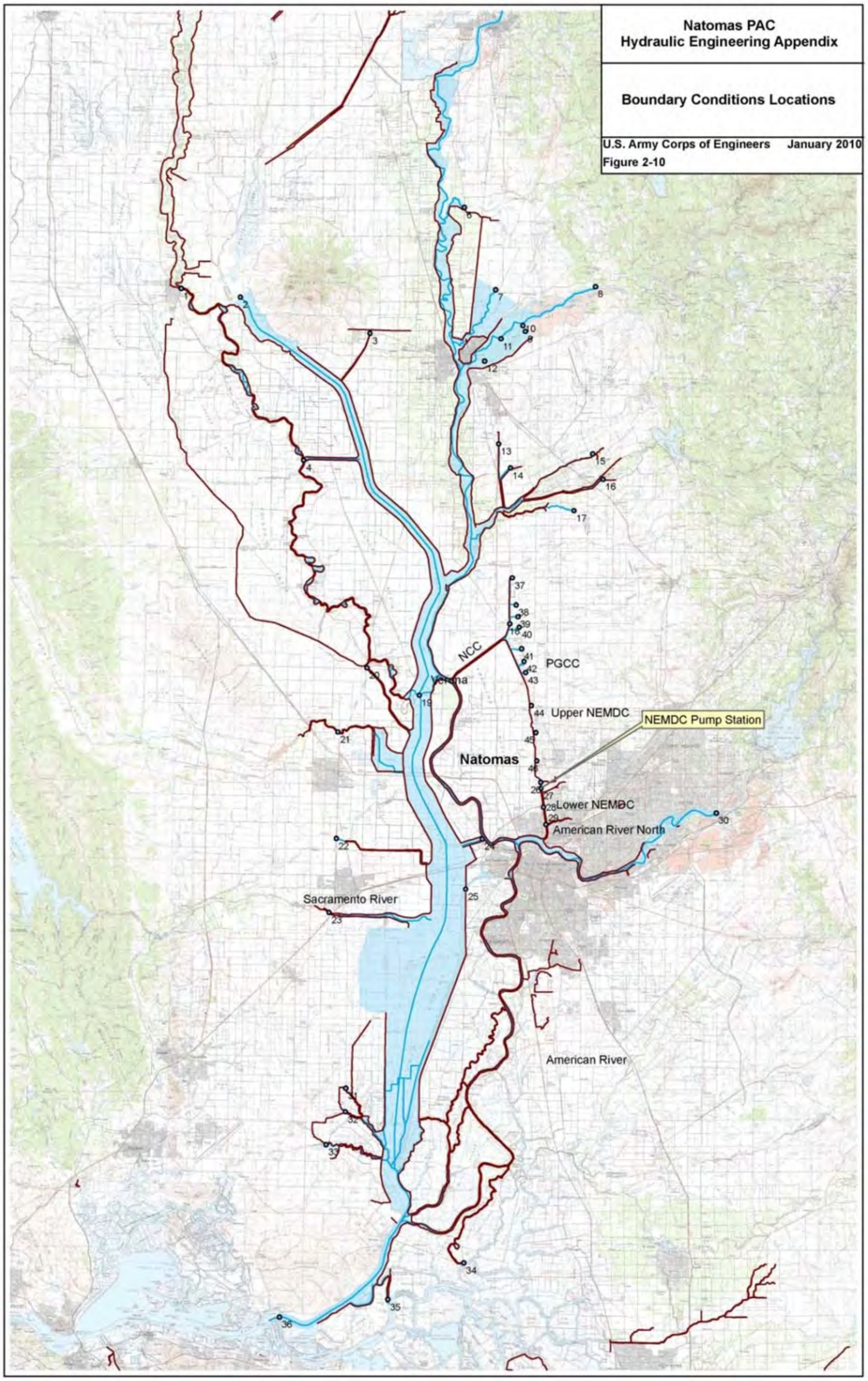

Figure 1. Model Extents (Source: USACE)

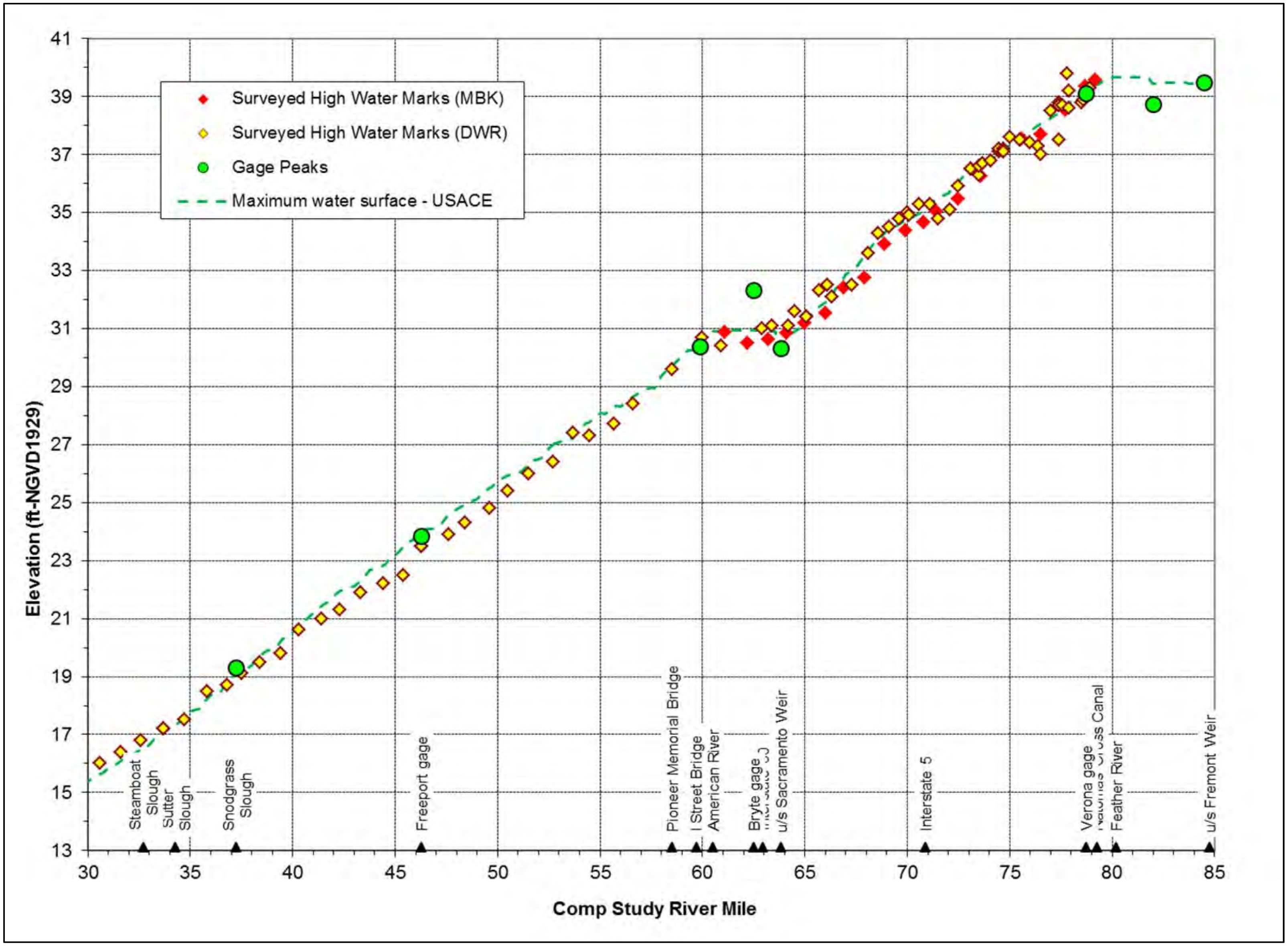

Figure 2. Sacramento River Maximum Water Surface Profile, January 1997 Calibration Simulation, USACE

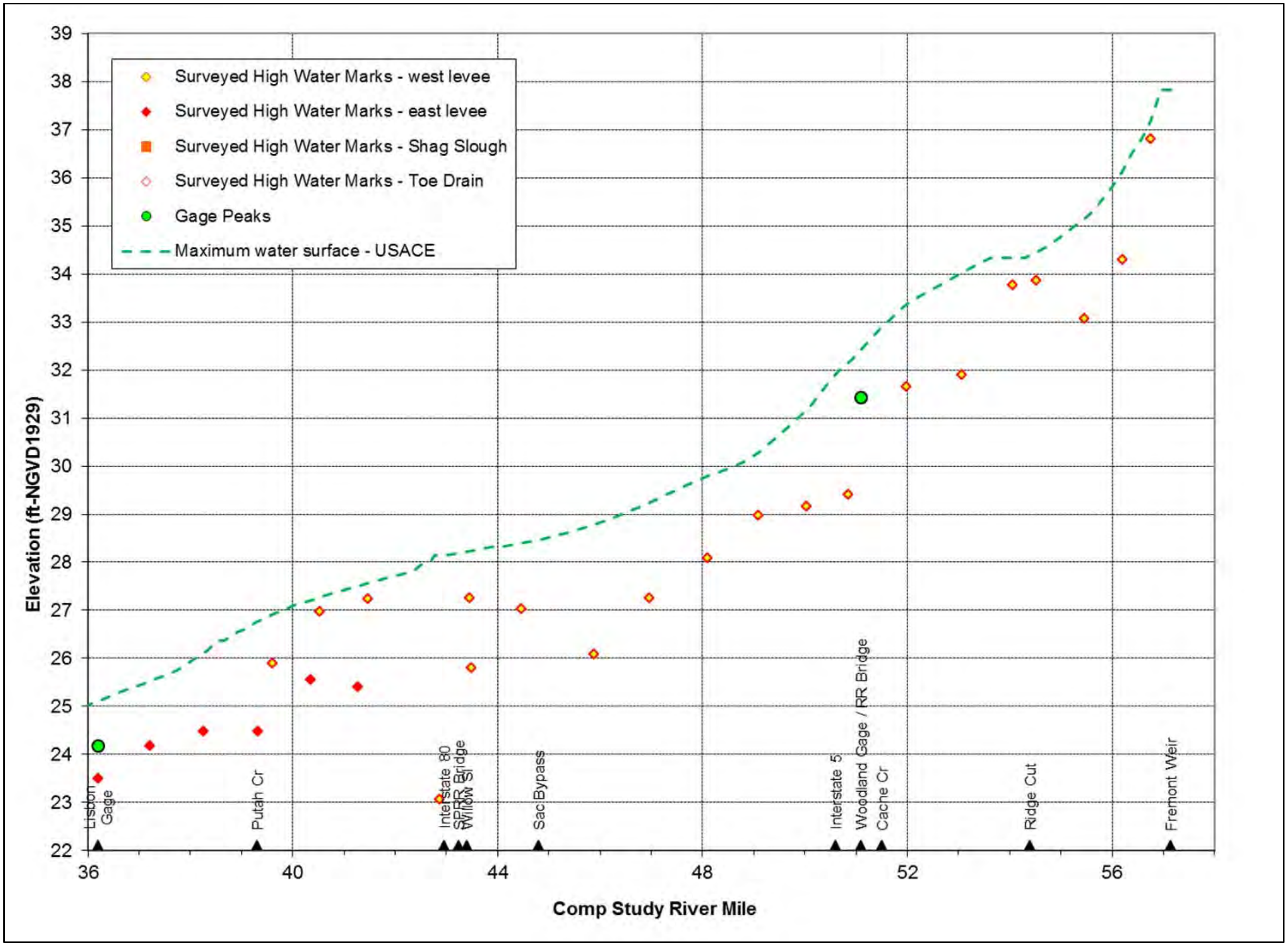

Figure 3. Yolo Bypass Maximum Water Surface Profile, January 1997 Calibration Simulation, USACE

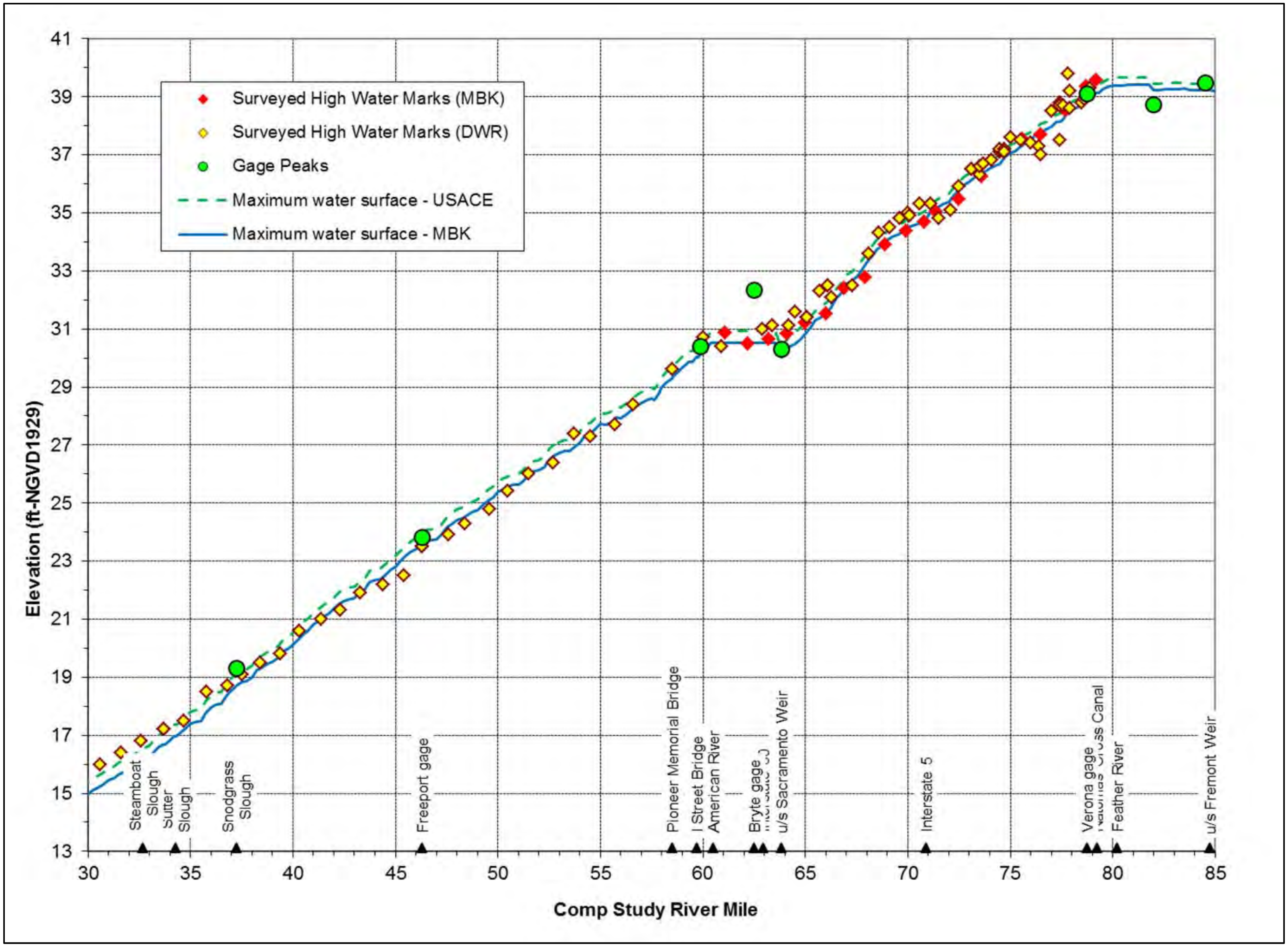

Figure 4. Sacramento River Maximum Water Surface Profile, January 1997 Calibration Simulation

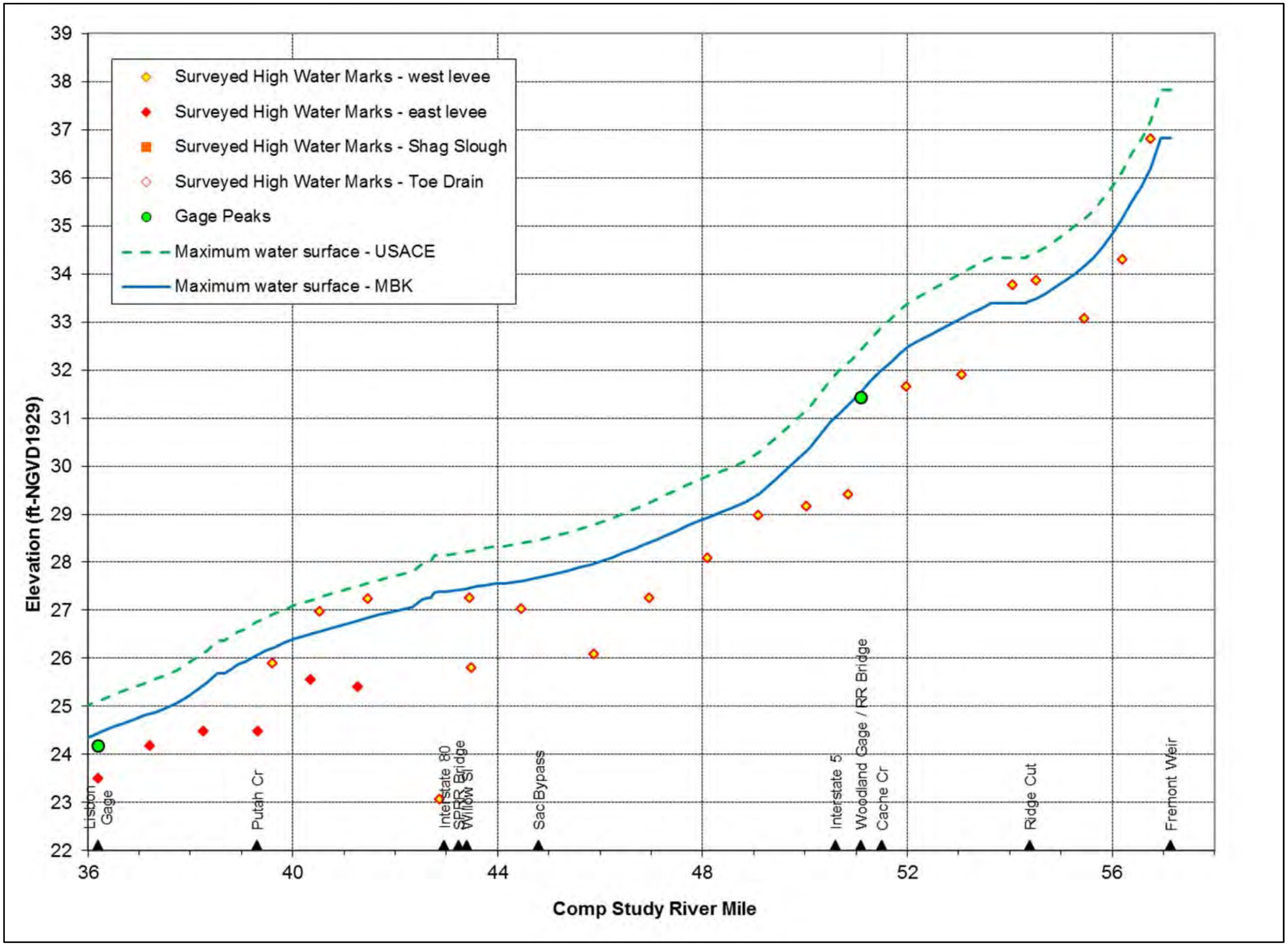

Figure 5. Yolo Bypass Maximum Water Surface Profile, January 1997 Calibration Simulation

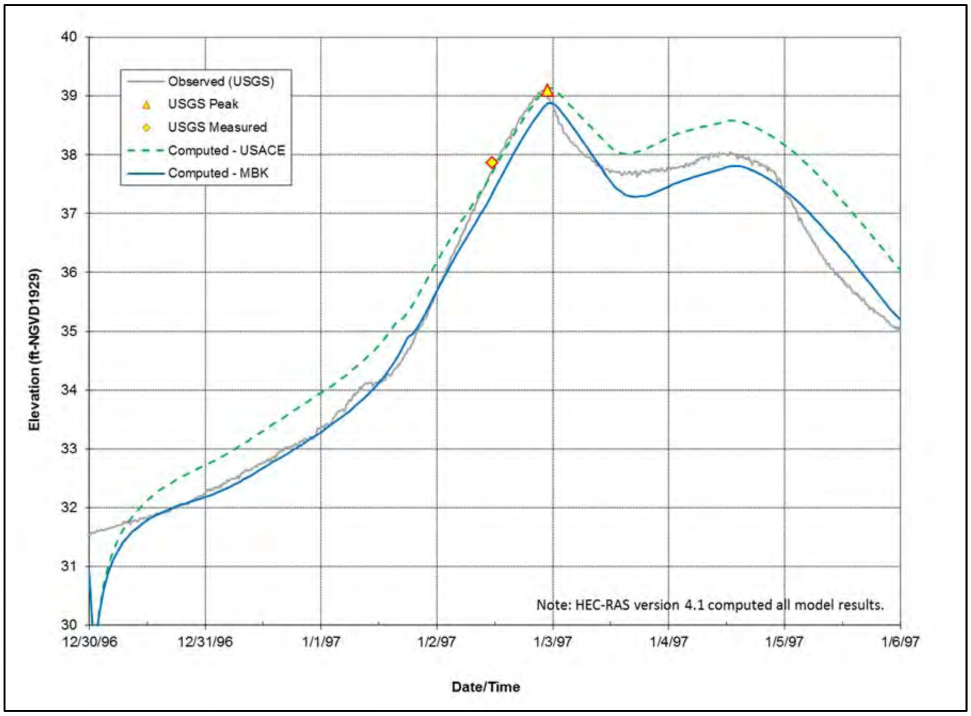

Figure 6. Sacramento River at Verona Gage (11425500 USGS) Water Surface Elevation, January 1997 Calibration Simulation

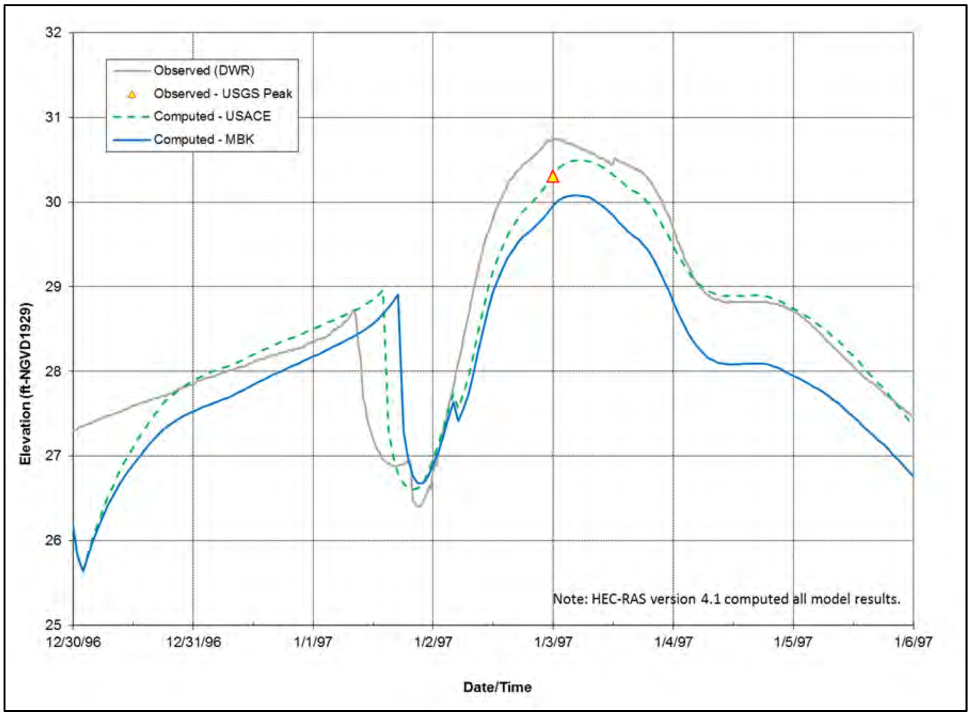

Figure 7. Sacramento River at Sacramento Weir (A02108 DWR) Water Surface Elevation, January 1997 Calibration Simulation

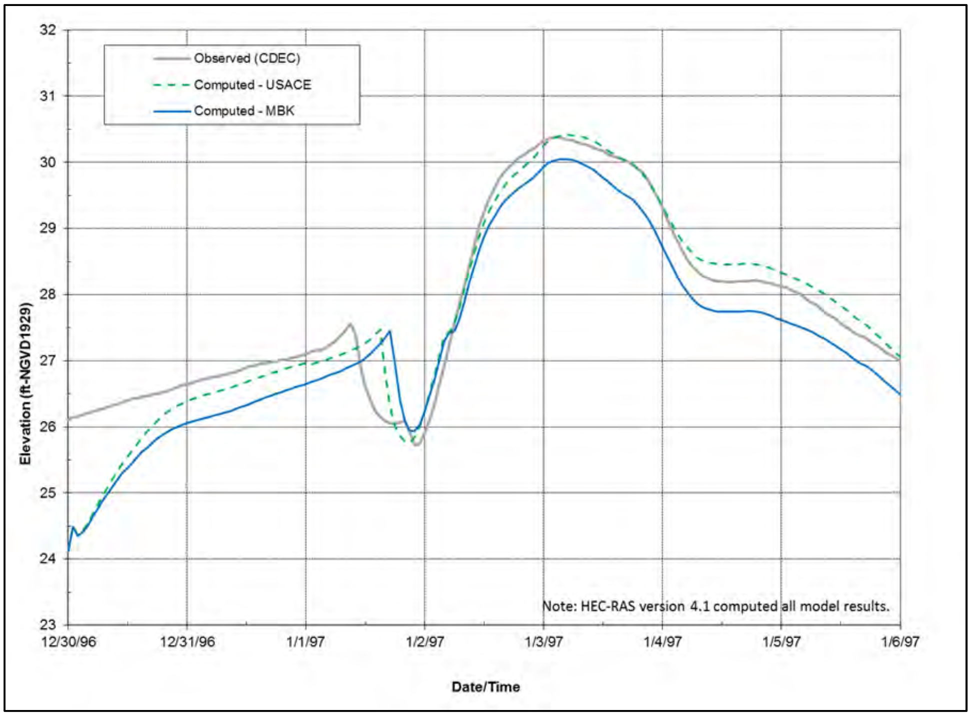

Figure 8. Sacramento River at I Street Gage (A02100 DWR) Water Surface Elevation, January 1997 Calibration Simulation

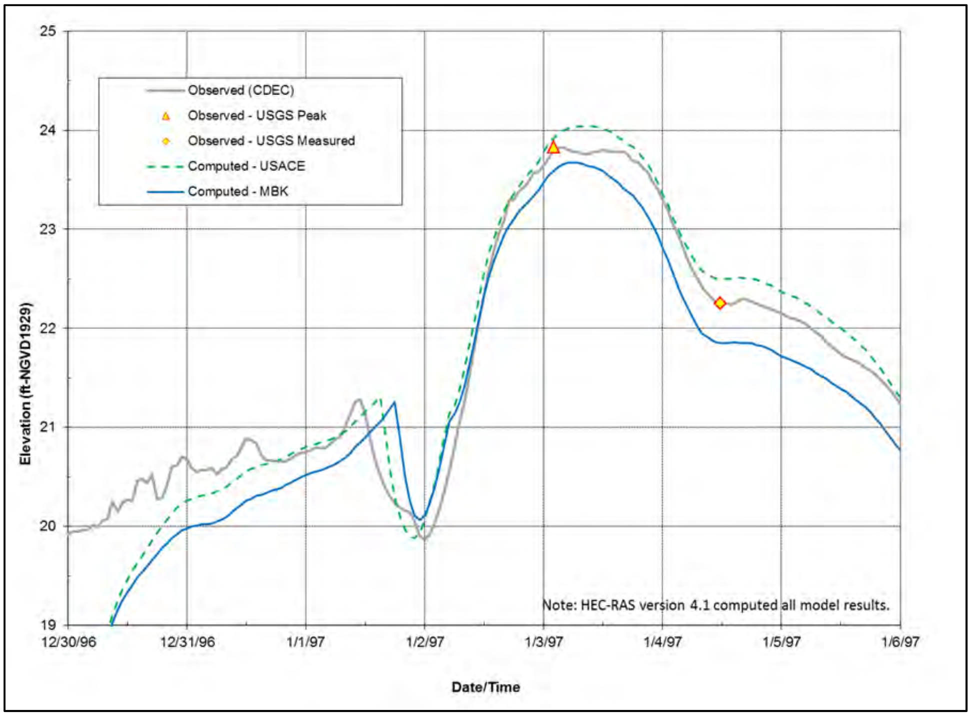

Figure 9. Sacramento River at Freeport Gage (11447650 USGS) Water Surface Elevation Figure 4, January 1997 Calibration Simulation

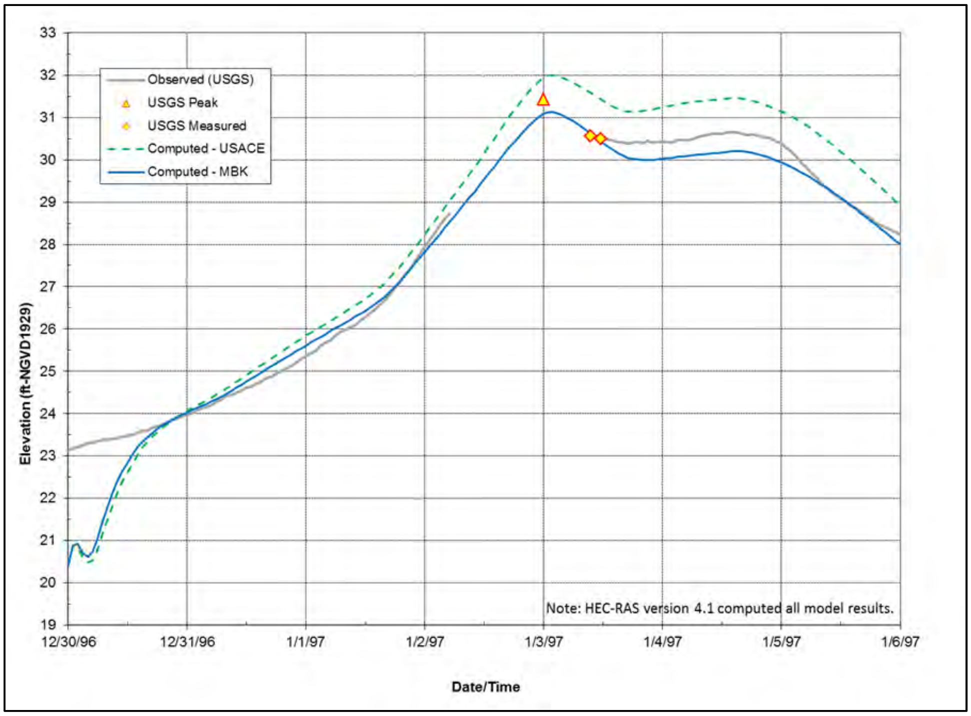

Figure 10. Yolo Bypass near Woodland Gage (11453000 USGS) Water Surface Elevation, January 1997 Calibration Simulation

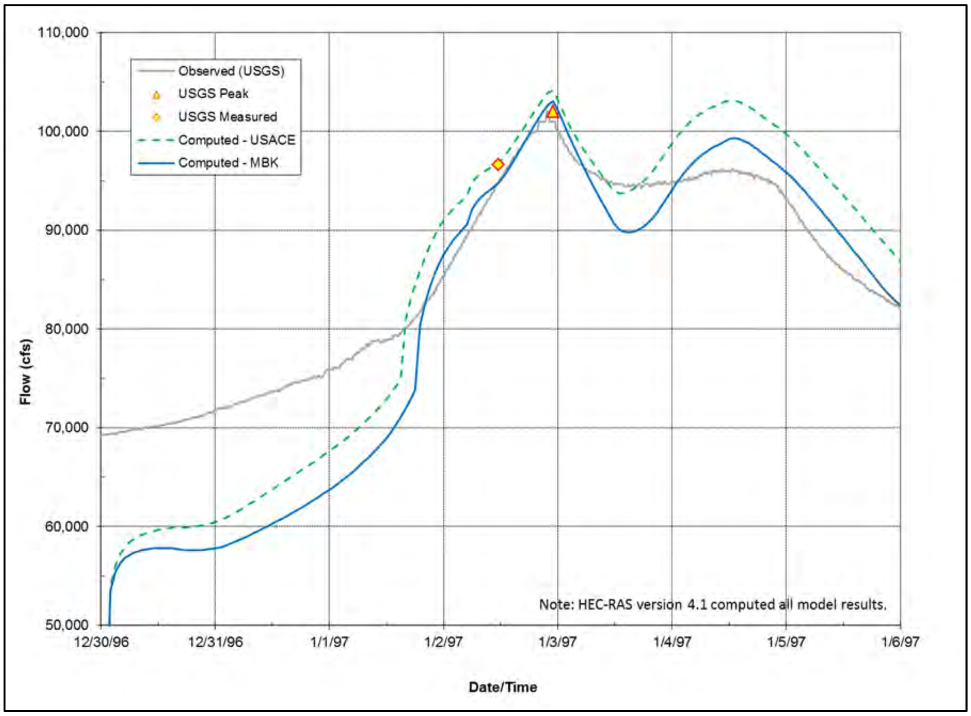

Figure 11. Sacramento River at Verona Gage (11425500 USGS) Flow, January 1997 Calibration Simulation

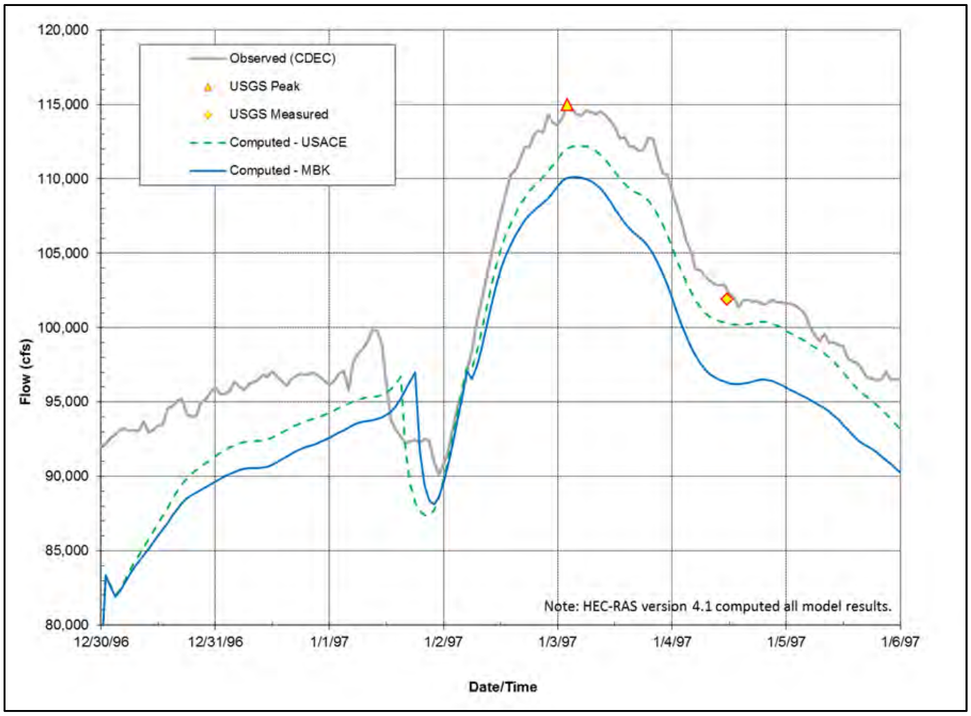

Figure 12. Sacramento River at Freeport Gage (11447650 USGS) Flow, January 1997 Calibration Simulation

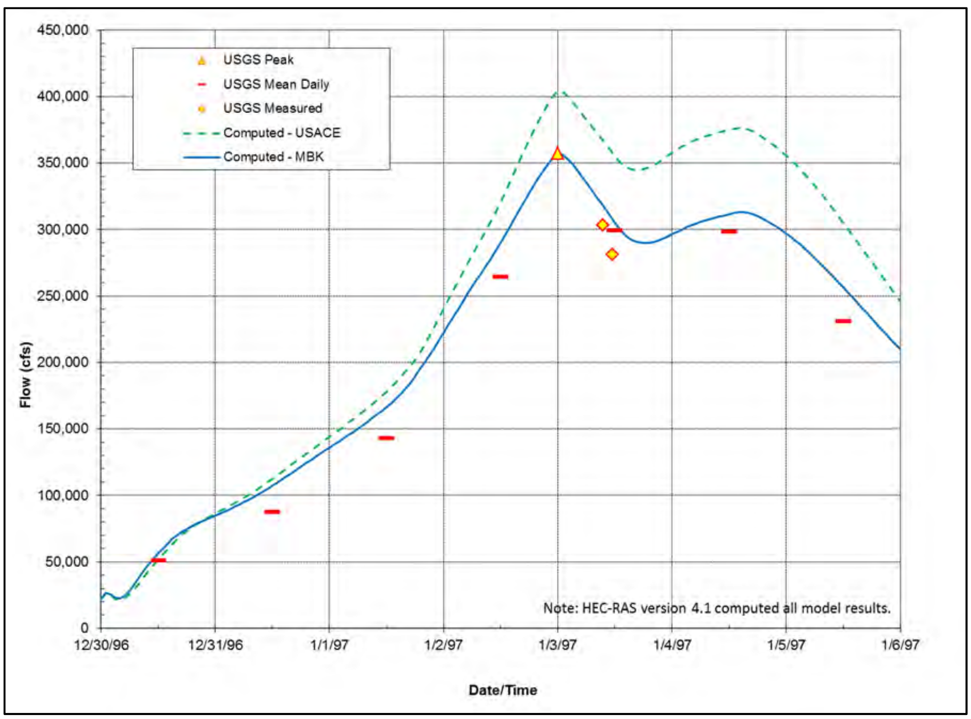

Figure 13. Yolo Bypass near Woodland Gage (11453000 USGS) Flow, January 1997 Calibration Simulation

Attachment 1

CESPK-ED-HD 13 December 2010

MEMORANDUM FOR RECORD

SUBJECT: Sacramento River Basin HEC-RAS Model Release 2

1. REFERENCES

   a. Memorandum for Record, CESPK-ED-DH. Subject: Sacramento River Basin HEC-RAS Model Release, 24 March 2009.
   b. Hydraulic Technical Documentation, Post-Authorization Change Report and Interim General Reevaluation Report, American River Watershed, Common Features Project, Natomas Basin, Sacramento and Sutter Counties, California, USACE, Sacramento District, August 2010.
   c. Hydrologic Engineering Center, HEC-RAS River Analysis Program Version 4.0.0, March 2008.
   d. Hydrologic Engineering Center, HEC-RAS User's Manual Version 4.0.0, March 2008.

2. PURPOSE. This memo documents a second release of the Sacramento Basin HEC-RAS model and accompanying documentation by the Sacramento District, U.S. Army Corps of Engineers (the Corps). An initial release of the model was completed in March 2009 with a Memorandum for Record, see Reference 1.a.

The Corps makes no warranty regarding use of the model beyond that described in the documentation accompanying the model - namely to support flood damage reduction feasibility study efforts of the Sacramento metropolitan areas along the Sacramento and lower American Rivers per applicable Corps guidance.

Those planning on using the model are responsible for applying it appropriately for their purpose(s). This includes ensuring that the boundary conditions included in the model are applicable.

3. MODEL DEVELOPMENT. The Sacramento River Basin HEC-RAS model was generated from previous modeling efforts in the Sacramento Basin. Much of the model geometry originated from the Sacramento Basin Comprehensive Study UNET model. The extent of the model is basically the same as the UNET model, except it does not include the Butte Basin and Sacramento River upstream of Colusa. The model was further enhanced by adding in top of levee profiles generated as part of the National Levee Database. The model was calibrated to the 1997 flood event and validated using the 2006 flood event. Model runs were completed using HEC-RAS version 4.0. The

model has been developed to varying levels of certainty on a reach by reach basis. The synthetic hydrology is the same as that used in the Comprehensive Study with some changes to flood routings through Folsom Dam (the objective release is assumed to be 160,000 cfs). It should be cautioned that much of the Hydrology for many of the smaller tributaries was not derived from detailed hydrologic analyses and only acts as a placeholder in the model. In addition, a simplified approach was used in generating the downstream boundary conditions for the n-yr frequencies. Further details regarding development of the model are contained in the documentation, *Draft Hydraulic Technical Documentation F3 Milestone, Appendix D, American River Watershed Common Features General Reevaluation Report (GRR).*

4. FUTURE MODEL DEVELOPMENT. The model and documentation are considered an interim product. The model is under continued development and therefore subject to change. Of particular note, the model with this release is based on the NGVD'29 vertical datum. The conversion of the model to NAVD'88 vertical datum is in process. As part of this effort, significant effort is being made to check and correct inaccuracies in the historical data used in calibration. Also, new modeling was generated for the east side of the Natomas Basin and is being incorporated into the basin model. A future release of the model will include these changes and perhaps others not mentioned here.

5. For questions or comments on the model and/or documentation contact Ethan Thompson, P.E., (916) 557-7142 or ethan.a.thompson@usace.army.mil.

Lea Adams, P.E.
Chief, Hydraulic Design Section
Sacramento District
U.S. Army Corps of Engineers

ATTACHMENT B

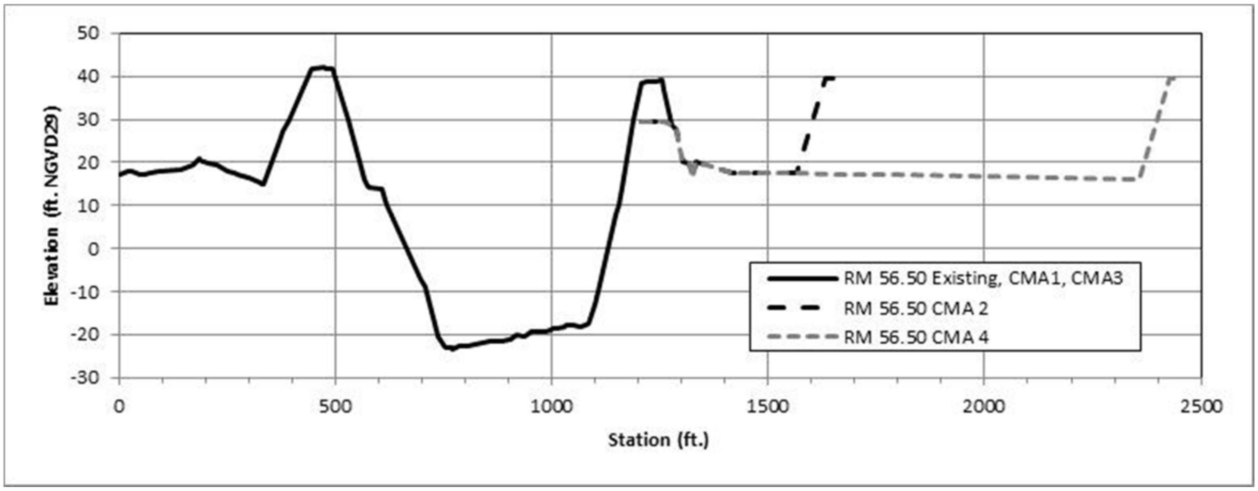

Exhibit B-1. Cross-section 56.50 (looking downstream)

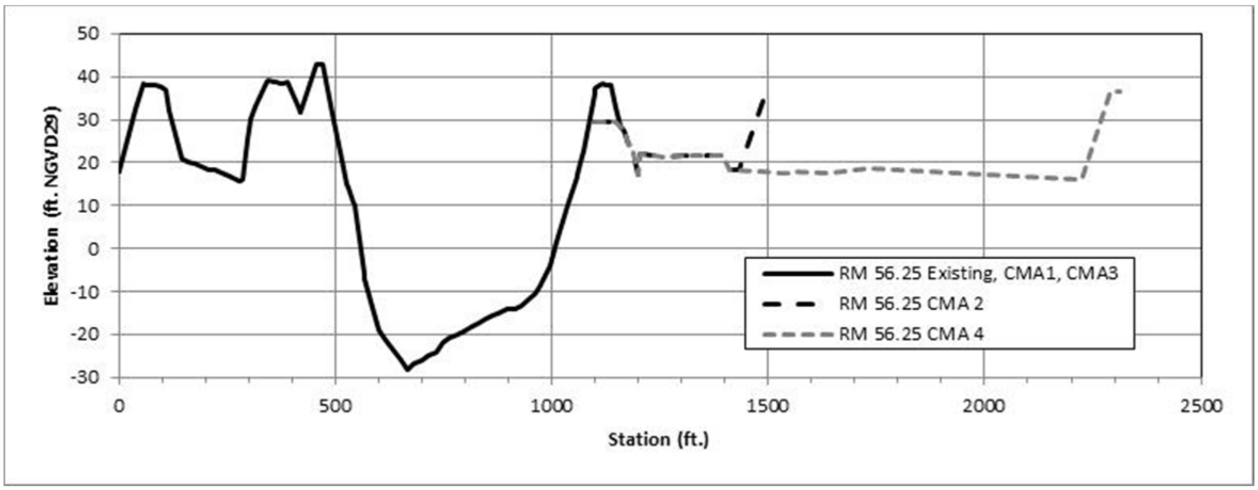

Exhibit B-2. Cross-section 56.25 (looking downstream)

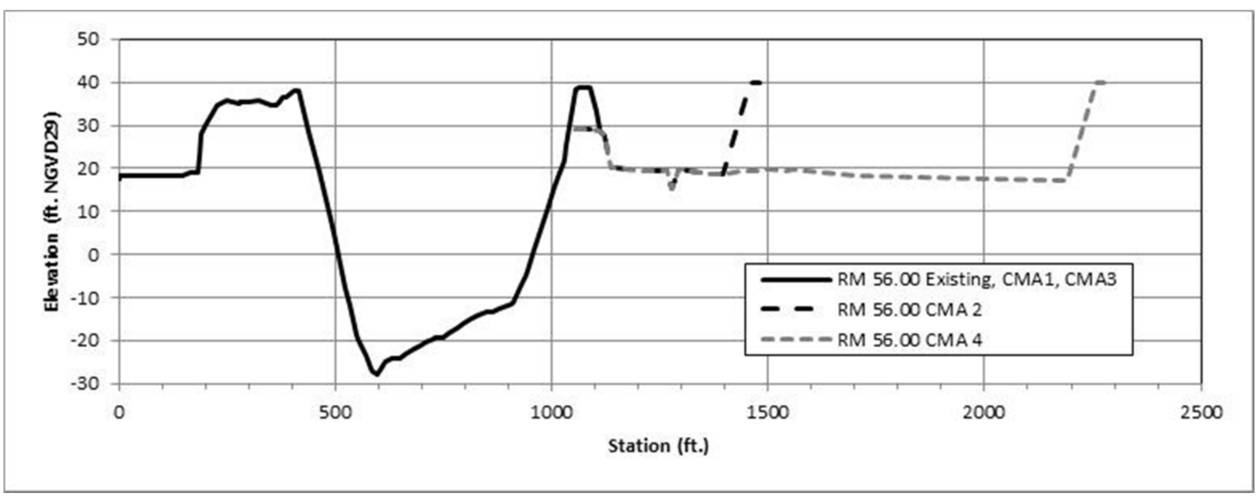

Exhibit B-3. Cross-section 56.00 (looking downstream)

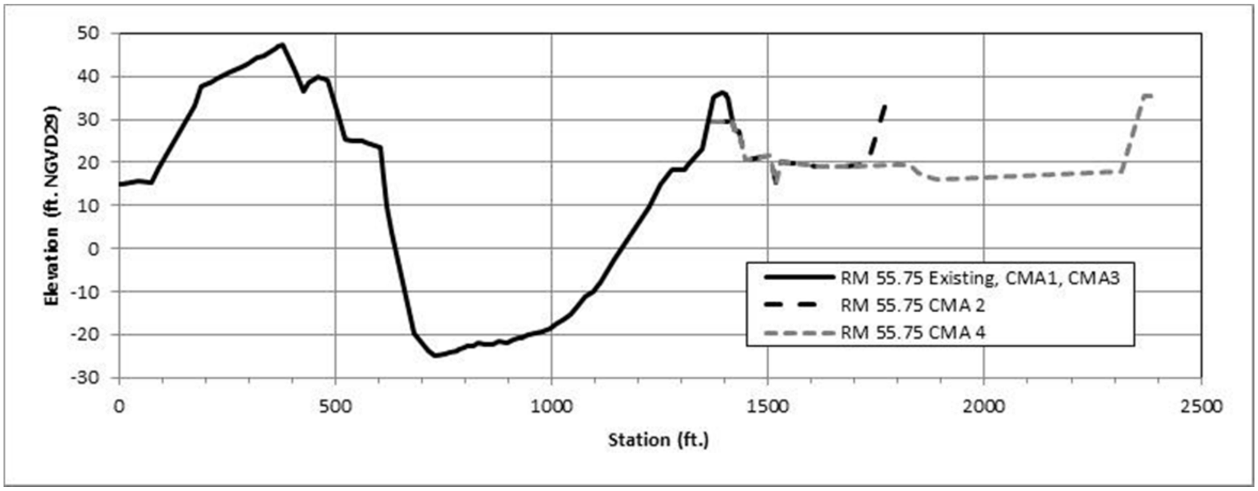

Exhibit B-4. Cross-section 55.75 (looking downstream)

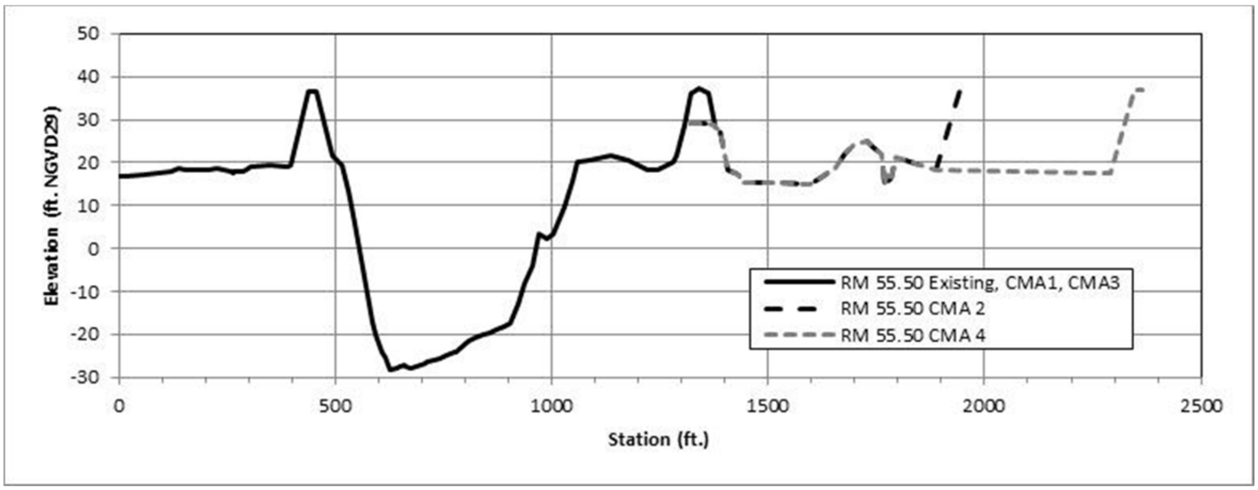

Exhibit B-5. Cross-section 55.50 (looking downstream)

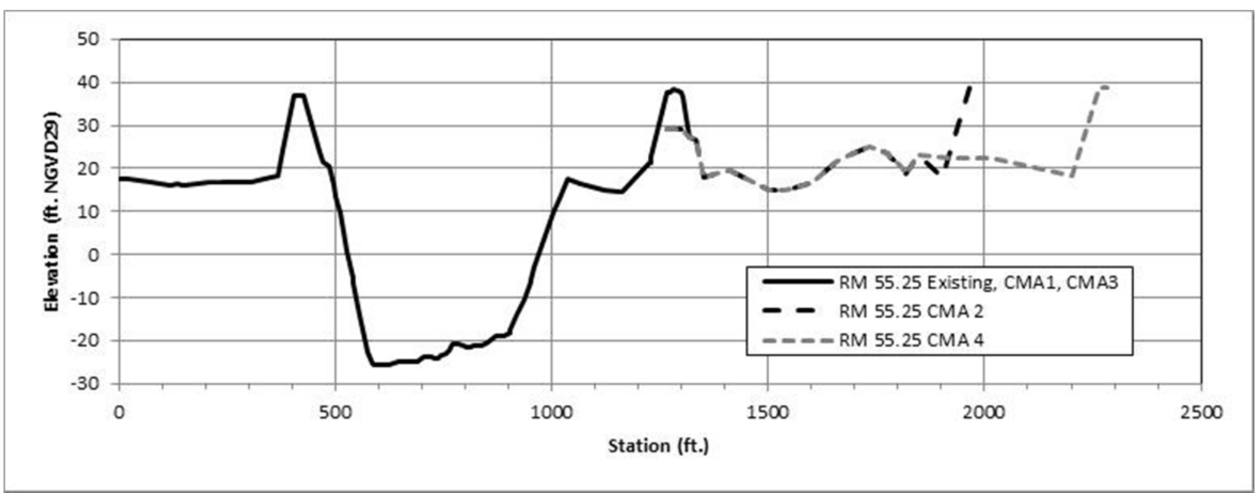

Exhibit B-6. Cross-section 55.25 (looking downstream)

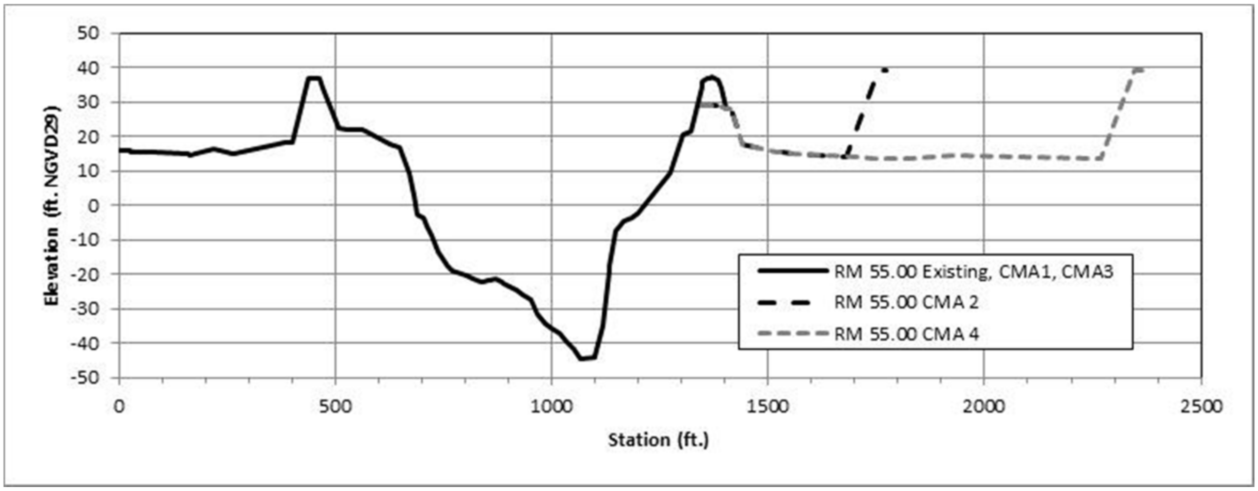

Exhibit B-7. Cross-section 55.00 (looking downstream)

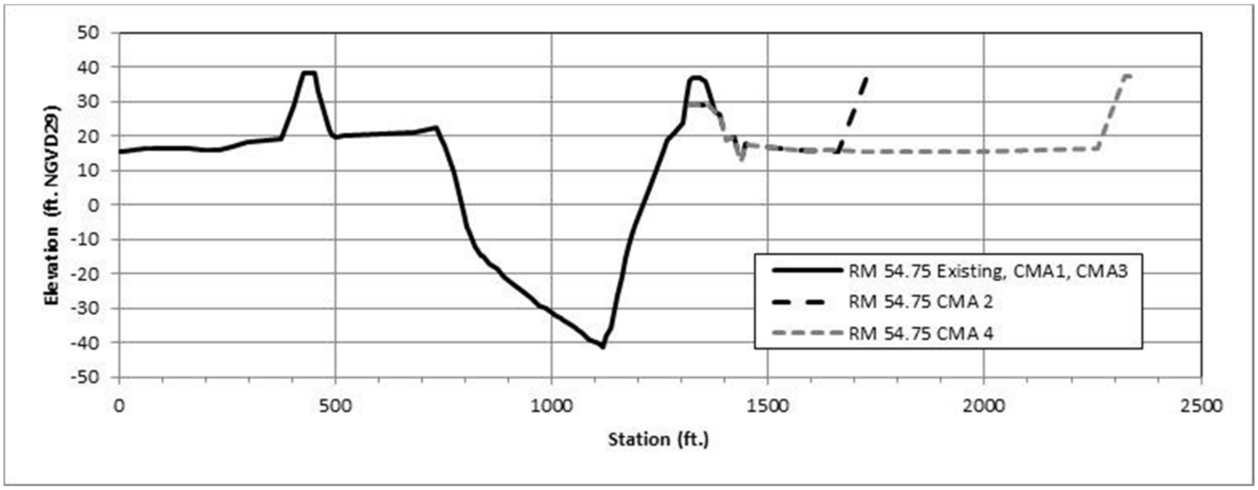

Exhibit B-8. Cross-section 54.75 (looking downstream)

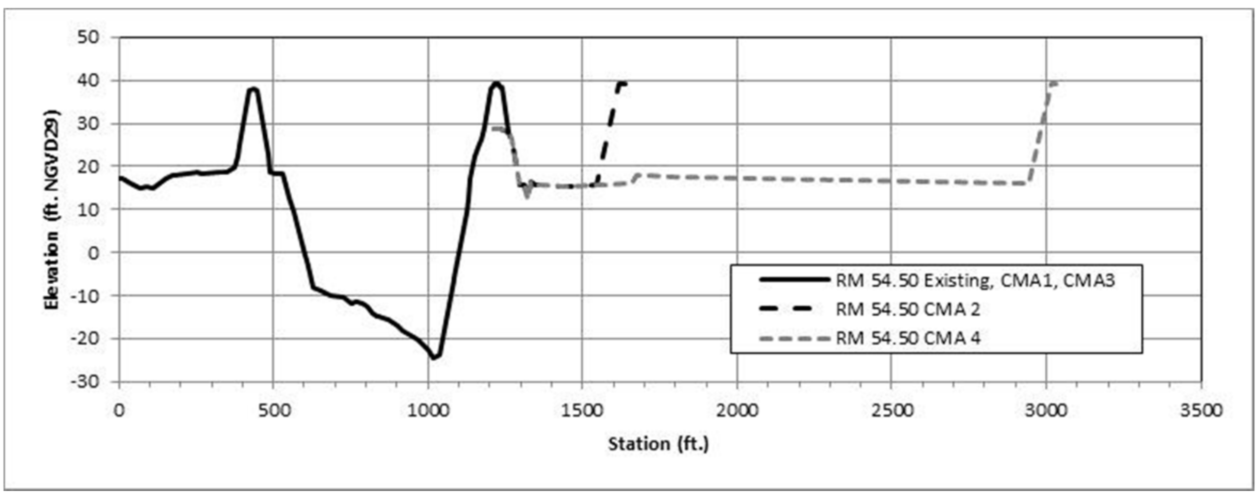

Exhibit B-9. Cross-section 54.50 (looking downstream)

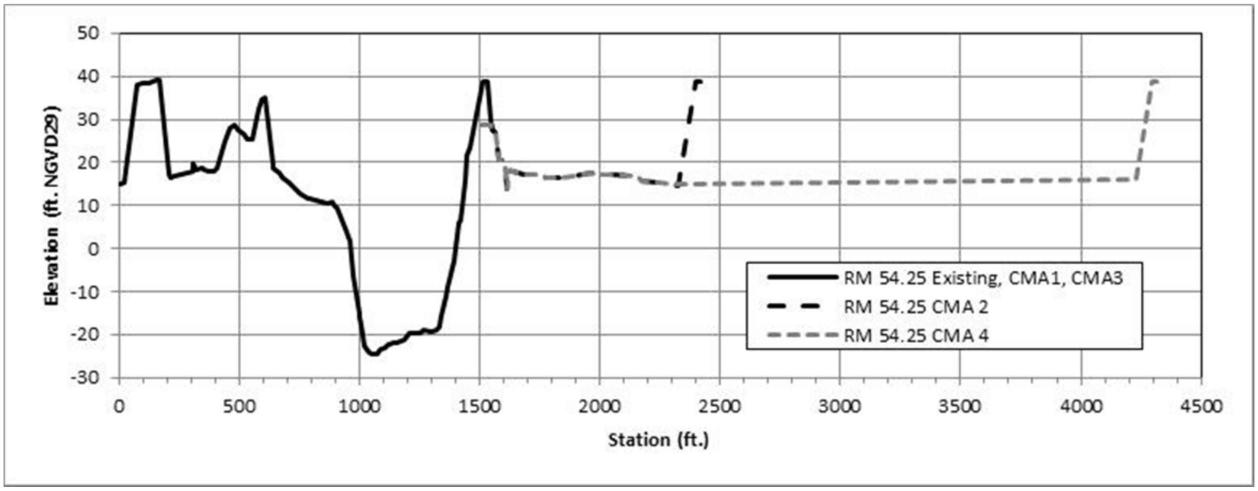

Exhibit B-10. Cross-section 54.25 (looking downstream)

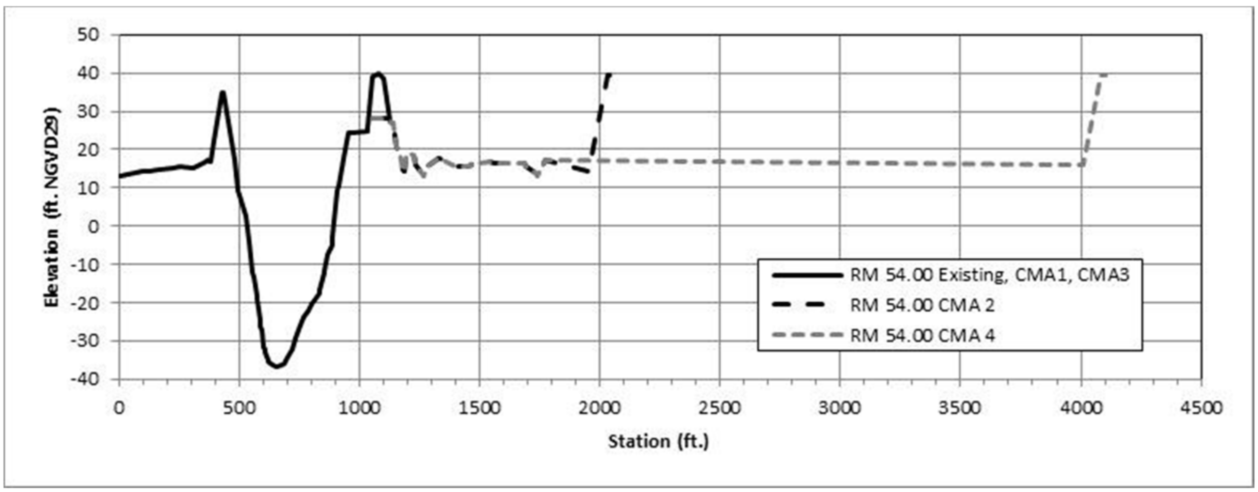

Exhibit B-11. Cross-section 54.00 (looking downstream)

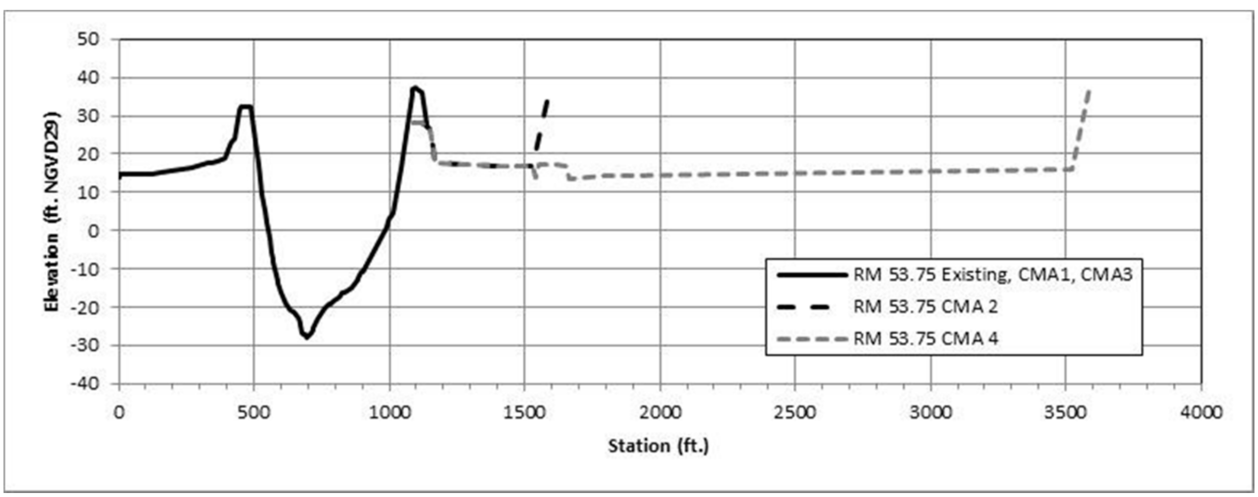

Exhibit B-12. Cross-section 53.75 (looking downstream)

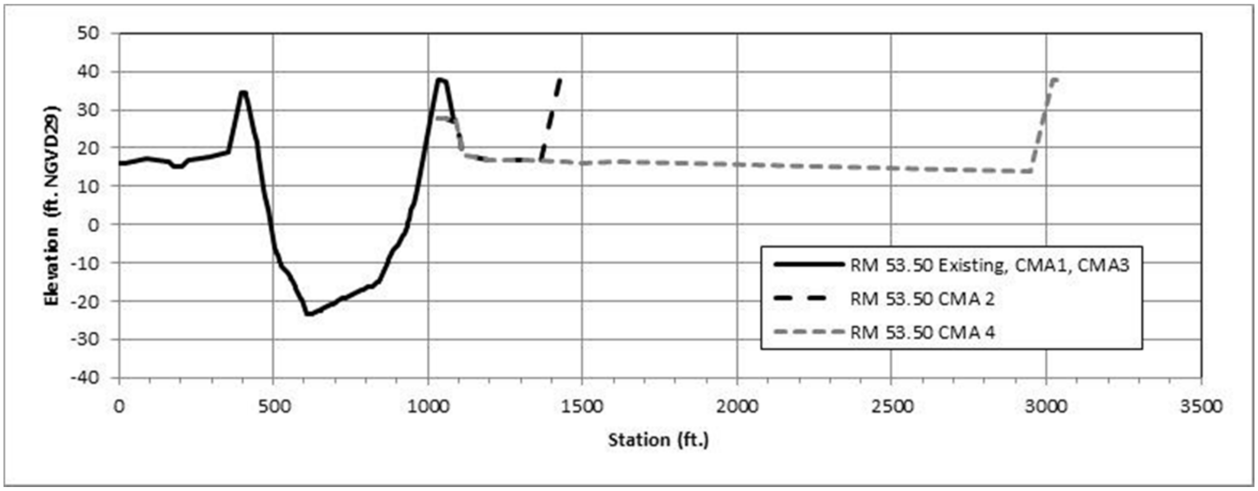

Exhibit B-13. Cross-section 53.50 (looking downstream)

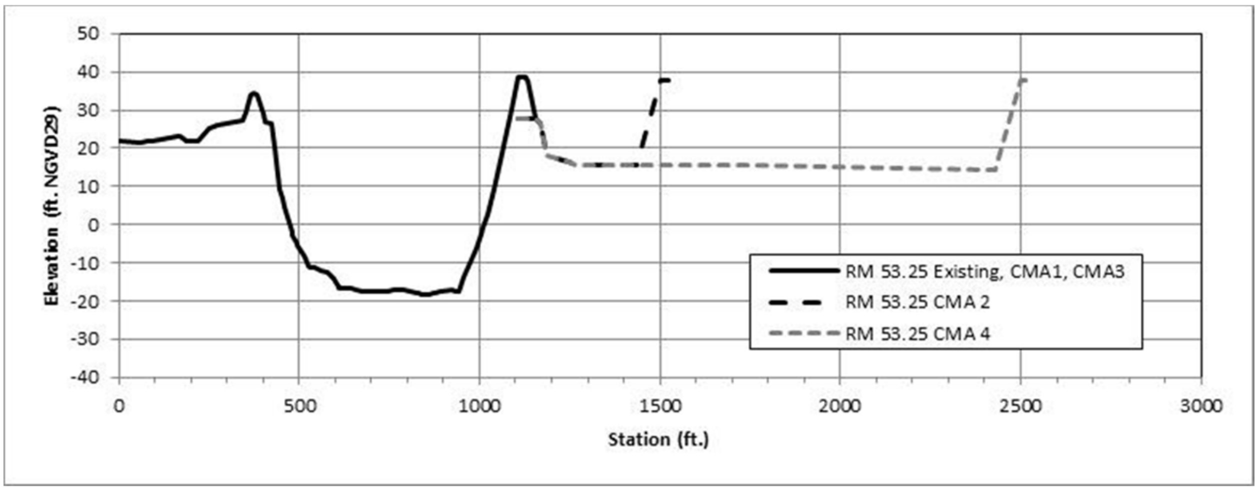

Exhibit B-14. Cross-section 53.25 (looking downstream)

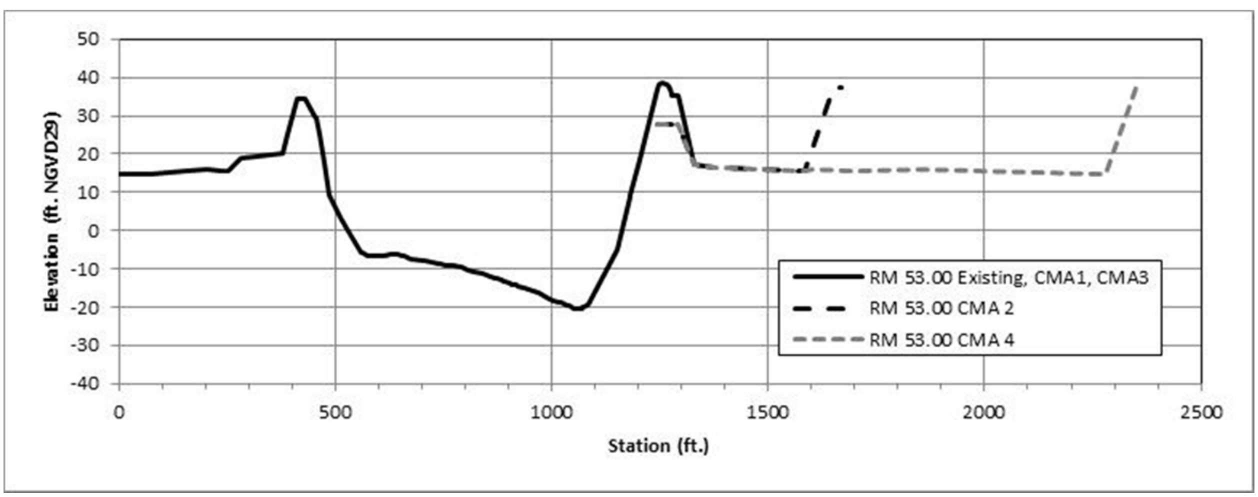

Exhibit B-15. Cross-section 53.00 (looking downstream)

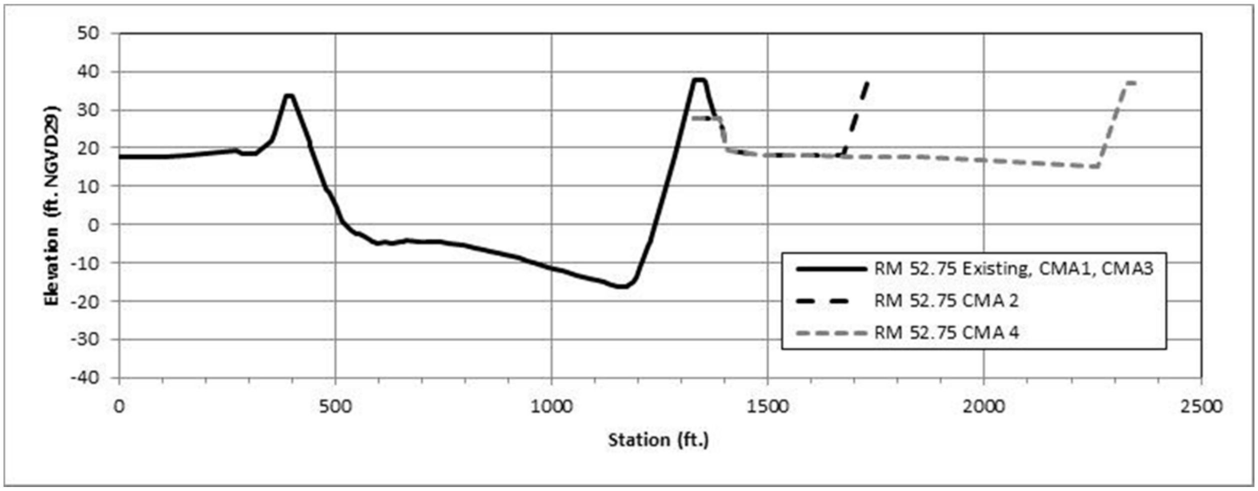

Exhibit B-16. Cross-section 52.75 (looking downstream)

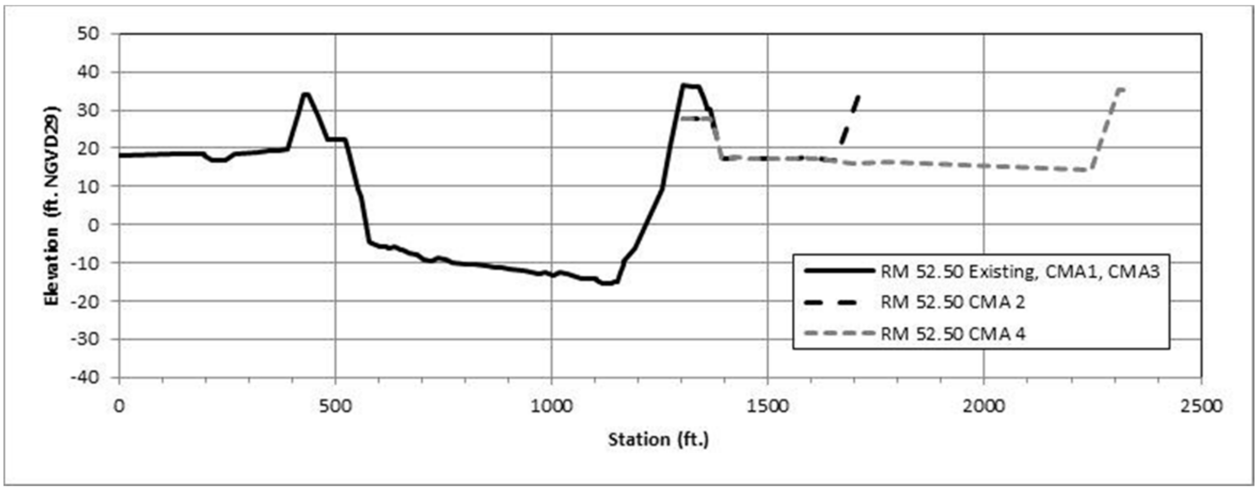

Exhibit B-17. Cross-section 52.50 (looking downstream)

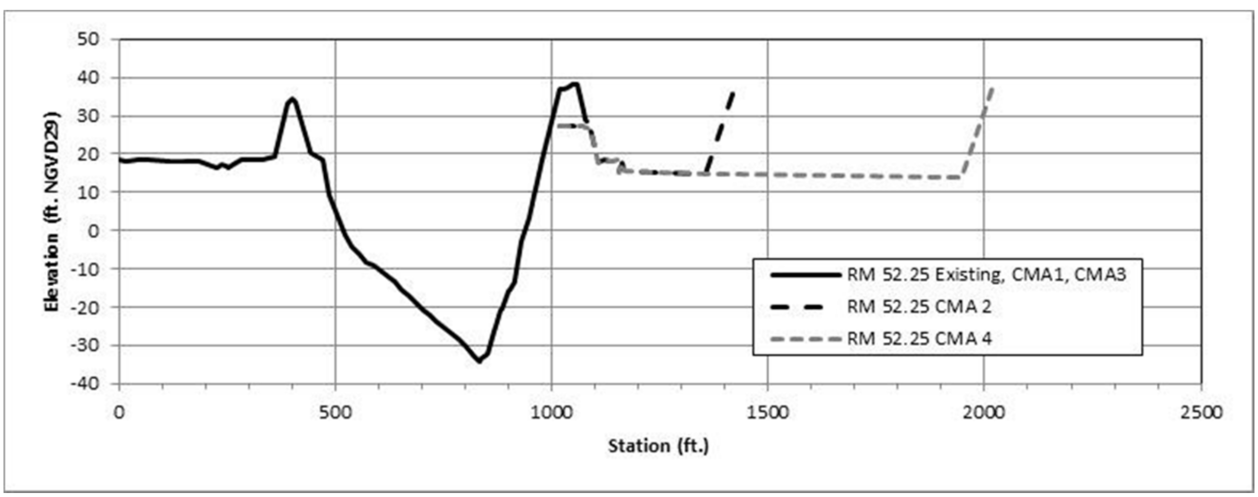

Exhibit B-18. Cross-section 52.25 (looking downstream)

Appendix B.5

Preliminary Existing Condition 2-Dimenstional Hydraulic Simulation Model—MBK Engineers, December 23, 2011

PRELIMINARY EXISTING CONDITION 2-DIMENSIONAL HYDRAULIC SIMULATION MODEL

Southport Early Implementation Project **December 23, 2011**

Prepared by: Mike Archer, P.E.

Reviewed by: Don Trieu, P.E.

1.0 Background

The West Sacramento Area Flood Control Agency (WSAFCA) is studying a potential levee setback as part of the Southport Early Implementation Project (EIP). A preliminary hydraulic analysis of four "Combined Measure Alternative" (CMA) projects was performed by MBK Engineers (MBK) with a 1-dimensional (1-D) HEC-RAS hydraulic simulation model as documented in a technical memorandum entitled Hydraulic Impact Analysis – Southport Early Implementation Project, June 29, 2011. The project location is shown in Exhibit H-1. A more detailed site map of the project area showing river miles and reference locations is provided in Exhibit H-2. To better understand and analyze the hydraulic impacts of the various CMA's and aide in design of the preferred CMA, a 2-dimensional (2-D) hydraulic simulation model is being developed. The development of the 2-D model is a being performed over two phases. The first phase consists of developing and calibrating the 2-D model to simulate the existing condition while the second phase consists of simulating the preferred CMA, to be determined at a later date. This Technical Memorandum discusses and presents phase one of the model development. Also included are results of preliminary simulations of the 100-year (1% annual chance of exceedence) and 200-year (model 0.5% annual chance of exceedence) flood events for existing conditions.

The model and analysis are considered preliminary because it is likely that the model mesh will be modified and refined to improve model efficiency and stability. Modifications may also be necessary to the existing condition model when the preferred CMA is developed to ensure consistency between the models for purposes of comparison.

2.0 Purpose

The 2-D hydraulic model is being developed for the Southport EIP to better analyze the localized hydraulics of the various CMA's for use in hydraulic impact analysis and design. Some of the CMA's consist of setting back the existing levee. The sharing of water between the Sacramento River main channel and the levee setback area is significantly 2-dimensional in nature. The 2-D hydraulic model will allow for improved analysis of the effects of partial levee removal and degrading, setback area grading and habitat enhancement. The 1-D model documented in the previously noted June 29, 2011 technical memorandum will still be needed for analyzing regional impacts and the results of the 2-D model will be used to refine the 1-D model for future regional impact studies.

3.0 Hydraulic Model

The 2-D model was developed using the Aquaveo SMS program, version 10.1. SMS is a pre- and post-processor for surface water modeling and analysis. The numerical hydraulic analysis was performed with RMA2 version 4.58. RMA2 is a 2-D depth averaged finite element hydrodynamic numerical model that computes water surface elevations and horizontal velocity components for subcritical, free-surface 2-D flow fields. RMA2 is on the FEMA list of numerical models that are acceptable for use for National Flood Insurance Program (NFIP) purposes.

4.0 Mesh Development

The 2-D model extends from the I Street stream-gage located about 900 feet upstream of the I Street Bridge (Comp Study[1] river mile 59.9) downstream to the Freeport stream-gage located at the Freeport Bridge (Comp Study river mile 46.4). The boundaries were selected such that they corresponded with streamgage locations and were sufficient distance from the project site to eliminate the potential of the boundary conditions influencing the results at and near the project site. The upstream boundary is approximately 3.1 miles upstream of the Project site and the downstream boundary is approximately 5.3 miles downstream of the Project site. Elevation contours from the topographic data described in Section 5.0 along with Comp Study elevation contour maps were used during the mesh construction to ensure that the mesh shape corresponded to the elevation contours as much as practicable. This initial model development was for existing conditions, however mesh elements were included at the project site in anticipation of model development of levee setback alternatives. Additionally, the model mesh was designed so that a recently constructed levee setback on the Sacramento River right bank downstream of Stone Lock, which did not exist at the time of the calibration and verification flood events, could be included in the existing conditions simulations and not included in the calibration and verification simulations. The 2-D model mesh is shown in Exhibit H-3.

Structures such as bridge piers and marinas were not explicitly included in this initial model, but were implicitly accounted for with increased roughness coefficients.

[1] Sacramento and San Joaquin River Basins Comprehensive Study. The United States Army Corps of Engineers (USACE) digitized stream centerlines and produced the Comp Study river mile stationing for hydraulic model development for the Comp Study. The Comp Study river mile stationing differs from the United States Geologic Survey (USGS) river mile stationing by 0.28 miles at the I Street Bridge (USGS river mile 60.00 = Comp Study river mile 60.28).

5.0 Topography

The topographic data used for the model was provided by HDR Engineering, Inc. The topographic data was a combination of 2007 LiDAR data from the city of West Sacramento and 2008 bathymetry data from the California Department of Water Resources (DWR). The DWR bathymetry was supplemented, where needed, with bathymetry data from the Comp Study. The topographic data is referenced to the North American Vertical Datum of 1988 (NAVD88).

6.0 Boundary Conditions

The 2-D model analysis will be steady state. The boundary conditions for the 2-D model are flow at the upstream boundary and corresponding stage at the downstream boundary. The 2-D model boundaries are defined at streamgage locations so that observed values can be used for the model calibration and verification. The 100-year and 200-year boundary conditions will be from the 1-D model simulations.

7.0 Calibration and Verification

The January 1997 flood event (1997 flood) was used for model calibration. The 1997 flood is commonly used for Sacramento River basin hydraulic model calibration because it is one of the largest flood events in recent history in the study area, with an estimated 1-day duration return period of about 90 years (1.1% chance of exceedence)[2], and there are abundant surveyed high water mark data. The 1997 flood peak flow reported at the USGS Freeport stream-gage (11447650) was 115,000 cubic feet per second (cfs). The January 2006 flood event (2006 flood) was used for model verification. The 2006 flood was much smaller than the 1997 flood, probably on the order of a 5 to 10 year flood, but was large enough that numerous high water marks were collected.

For the upstream boundary condition (I Street) of the 2-D model, a flow is required for input into the model. Since flow data is not published for the I Street gage (Sacramento River at Sacramento, DWR A02100), flow data at the USGS Freeport gage (11447650) will be used for model calibration and verification. The California Data Exchange Center (CDEC) reports real-time flow data for the I Street gage (IST), but the quality of the rating curve is not known. Flow data is published for the USGS Freeport gage. The USGS makes regular streamflow measurements and rating curve adjustments for the Freeport gage, therefore the reported flow data at the Freeport gage is more reliable than that at the I Street gage. Due to the relatively uniform channel between I Street and Freeport the peak flow during large flood events does not vary significantly between those locations. Hydraulic simulations of the 1997 and 2006 floods with the 1-D model indicate a difference of less than 1%. Based on this, the peak flow reported at the Freeport gage was used as the upstream boundary condition for the 2-D model calibration and verification simulations. Review of observed hydrographs at I Street and Freeport indicate

[2] Sacramento and San Joaquin River Basins Comprehensive Study, Technical Studies Documentation, Appendix B, Synthetic Hydrology Technical Documentation, USACE, December 2002.

that the peaks occurred at essentially the same time at both locations, therefore the peak stage reported at the Freeport gage was used as the downstream boundary condition for the 2-D model calibration and verification simulations. The calibration and verification boundary conditions are summarized in Table 1. The datum of the Freeport gage is 100 feet below NGVD29. USACE determined as a result of a datum analysis and survey of the gage reference as part of the American River Common Features Feasibility Study that the gage height can be converted to NAVD88 by subtracting 97.84 feet.

Table 1. Calibration and Verification Boundary Conditions

Study	Flood Event	Upstream Boundary – Flow, Sacramento River at I Street	Downstream Boundary – Stage, Sacramento River at Freeport
Calibration	1997	115,000 cfs	25.99 ft. NAVD88
Verification	2006	97,200 cfs	23.30 ft. NAVD88

The locations of the available surveyed high water marks in the study area for the 1997 and 2006 floods are shown in Exhibits H-4 and H-5. The high water marks for both the 1997 flood and 2006 flood were staked and surveyed by the California Department of Water Resources. The 1997 and 2006 high water mark elevations are summarized in Tables 2 and 3. The recorded peak stage data for the I Street and Freeport streamgages shown in Table 4 will also be used for the calibration and verification.

Table 2. January 1997 Flood Event Surveyed High Water Marks (Source: DWR)

ID/Description	Coordinates		Comp Study River Mile	High Water Mark Elevation (ft. NAVD88)
	Easting	Northing		
80' upstream of L.M. 0.4 post	6703000	1977680	59.95	33.0 [a]
175' downstream of Hwy. 80 bridge	6700130	1969810	58.45	31.9 [a]
TBM MF 1	6701760	1960880	56.50	30.6 [b]
TBM MF 2	6699510	1957030	55.68	29.9 [b]
TBM MF 3	6697680	1952950	54.35	29.5 [b]
TBM MF 4	6695990	1950320	53.70	29.6 [b]
TBM MF 5	6690860	1949170	52.70	28.6 [b]
TBM MF 6	6688160	1943790	51.40	28.2 [b]
TBM MF 7	6690640	1939160	50.38	27.6 [b]
TBM MF 8	6693550	1935460	49.43	27.0 [b]
TBM MF 9	6699830	1935310	48.20	26.5 [b]
TBM MF 10	6703510	1933740	47.47	26.1 [b]
Upstream @ Freeport bridge	6704560	1928290	46.43	25.7 [b]

[a] Original HWM referenced to NGVD29 vertical datum. Converted to NAVD88 by adding 2.3 ft. as per USACE conversion of Sacramento River HEC-RAS at this location.
[a] Original HWM referenced to NGVD29 vertical datum. Converted to NAVD88 by adding 2.2 ft. as per USACE conversion of Sacramento River HEC-RAS at this location.

Table 3. January 2006 Flood Event Surveyed High Water Marks (Source: DWR)

ID	Coordinates		Comp Study River Mile	High Water Mark Elevation (ft. NAVD88)
	Easting	Northing		
3264	6702350	1976050	59.74	29.9
3262	6702320	1975680	59.67	29.4
3258	6702130	1974560	59.45	29.4
3260	6701940	1973930	59.32	29.3
70002	6701370	1959260	56.21	27.8
70006	6696140	1950330	53.74	27.3
70008	6693760	1950090	53.28	26.5
70010	6691190	1949260	52.76	26.1
70012	6689230	1947730	52.22	25.6
70014	6688390	1945250	51.71	25.2
70016	6688510	1942650	51.16	24.8
70018	6689910	1940440	50.66	25.3
70020	6691220	1938150	50.16	25.1
70021	6692880	1935960	49.6	24.6
70023	6695200	1935040	49.07	24.2
70025	6697820	1935210	48.56	24.1
3051	6700390	1935320	48.08	23.6
3053	6702860	1934450	47.62	24.4
3055	6704190	1929980	46.76	23.8

Table 4. January 1997 and January 2006 Peak Stages at Streamgages

Gage	Agency/ ID	Comp Study River Mile	Flood Event	Peak Gage Height (ft.)	Gage Height to NAVD88 [a]	Peak Stage (ft. NAVD88)
Sacramento River at Sacramento (I Street)	DWR A02100	59.86	1997	30.38[b]	+2.54	32.92
			2006	27.70[b]	+2.54	30.24
Sacramento River at Freeport	USGS 11447650	46.43	1997	23.83[b]	+2.16	25.99
			2006	121.14[c]	-97.84	23.30

[a] Conversion to NAVD88 based on survey of gage reference by USACE for the American River Common Features Feasibility Study.
[b] Gage datum = 0 ft. NGVD29
[c] Gage datum = 100 ft. NGVD29

The model was calibrated by adjusting Manning's n roughness coefficients until the model reasonably reproduced the 1997 flood observed peak water surfaces. Each 2-d model mesh element is assigned a "material" which has an associated Manning's n roughness coefficient.

Mesh element materials were assigned based on review of aerial photography. Exhibit H-6 shows the material map for the calibrated model. The final roughness coefficients used in the model calibration are listed in Table 5. The HEC-RAS Hydraulic Reference Manual, Version 4.1, January 2010, provides a summary table of typical Manning's n value ranges from "Open-Channel Hydraulics" by V.T. Chow, 1959. The applicable values for the 2-d model are provided in Table 6. For the most part, the calibrated n values are at or below low end of the typical range documented by Chow.

The calibrated 1997 flood water surface elevation profile with observed high water data is shown in Exhibit H-7. A tabulation comparing the observed high water elevations and the computed water surface elevations from the calibration simulation is provided in Table 7. The differences of the computed calibration water surface elevations from the observed elevations range from -0.52 ft. to +0.75, with an average difference of +0.14 ft.

After completion of the model calibration, the 2006 flood event was simulated with the calibrated model to check the model's ability to reproduce a different flood event, that is, to verify the model. The 2006 flood verification simulation water surface profile with observed high water data is shown in Exhibit H-8. A tabulation comparing the observed high water elevations and the computed water surface elevations from the calibration simulation is provided in Table 8. The differences of the computed verification water surface elevations from the observed elevations range from -1.17 ft. to +0.63 ft., with an average difference of -0.15 ft.

Table 5.Manning's n Roughness Coefficient

Material	Manning's n Roughness Coefficient
Channel	0.024
No brush	0.030
Scattered trees	0.045
Dense trees	0.065
Marina	0.180

Table 6. Typical Manning's n Roughness Coefficients

Type of Channel and Description	Manning's n Roughness Coefficient		
	Minimum	Normal	Maximum
Main Channel: clean, straight, full, no rifts or deep pools	0.025	0.030	0.033
Scattered brush, heavy weeds	0.035	0.050	0.060
Light brush and trees, in winter	0.035	0.050	0.070
Heavy stand of timber, few down trees, little undergrowth, flow below branches	0.080	0.100	0.120
Same as above, but with flow into branches	0.100	0.120	0.160

Table 7. 1997 Flood Event Calibration Results

ID/Description	Comp Study River Mile	Maximum Water Surface Elevation (ft. NAVD88)		Difference
		Observed	Computed	
I St gage	59.86	32.92	33.06	0.14
175' DS of Hwy 80 bridge	58.45	31.9	31.62	-0.29
TBM MF 1	56.50	30.6	30.85	0.25
TBM MF 2	55.68	29.9	30.13	0.23
TBM MF 3	54.35	29.5	29.49	-0.01
TBM MF 4	53.70	29.6	29.08	-0.52
TBM MF 5	52.70	28.6	28.86	0.26
TBM MF 6	51.40	28.2	28.12	-0.08
TBM MF 7	50.38	27.6	27.84	0.24
TBM MF 8	49.43	27.0	27.36	0.36
TBM MF 9	48.20	26.5	27.04	0.54
TBM MF 10	47.47	26.1	26.85	0.75
US of Freeport Bridge	46.43	25.7	26.00	0.30
Freeport gage	46.43	25.99	26.00	0.01

Table 8. 2006 Flood Event Verification Results

ID/Description	Comp Study River Mile	Maximum Water Surface Elevation (ft. NAVD88)		Difference
		Observed	Computed	
I St gage	59.86	30.24	29.70	-0.54
3264	59.74	29.9	29.46	-0.44
3262	59.67	29.4	29.47	0.07
3258	59.45	29.4	29.23	-0.17
3260	59.32	29.3	28.94	-0.37
70002	56.21	27.8	27.55	-0.25
70006	53.74	27.3	26.13	-1.17
70008	53.28	26.5	25.97	-0.53
70010	52.76	26.1	25.90	-0.20
70012	52.22	25.6	25.52	-0.08
70014	51.71	25.2	25.44	0.24
70016	51.16	24.8	25.14	0.34
70018	50.66	25.3	25.09	-0.21
70020	50.16	25.1	24.86	-0.24
70021	49.60	24.6	24.56	-0.04
3051	48.08	23.6	24.23	0.63
70025	48.56	24.1	24.28	0.18
70023	49.07	24.2	24.37	0.17
3053	47.62	24.4	24.10	-0.30
3055	46.76	23.8	23.59	-0.21
Freeport gage	46.43	23.3	23.30	0.00

8.0 Preliminary Simulation of 100-year and 200-year Floods

The 2-D model boundary conditions for the 100-year and 200-year flood events were obtained from the 1-D model simulations of the existing conditions documented in the June 29, 2011 Technical Memorandum. The 100-year and 200-year flood event simulations assumed that levees do not fail, but rather act as weirs if overtopped, and that the Folsom Joint Federal Project (JFP) was completed. The 100-year and 200-year 2-D model boundary conditions are summarized in Table 9. Contour maps of the computed velocities and water surface elevations for the 100-year and 200-year simulations are provided in Exhibits H-9 through H-12. The water surface elevations computed by the 2-D model compare well with those computed by the 1-D model as shown in Exhibits H-13 and H-14.

Table 9. 100-year and 200-year Flood Event Boundary Conditions

Flood Event	Upstream Boundary – Flow, Sacramento River at I Street	Downstream Boundary – Stage, Sacramento River at Freeport
100-year	118,600 cfs	27.3 ft. NAVD88
200-year	131,100 cfs	29.0 ft. NAVD88

9.0 Future Hydraulic Analysis

This technical memorandum presents the initial development, calibration and verification of the Southport EIP 2-D flood hydraulic analysis model, along with the results of preliminary 100-year and 200-year simulations of the existing conditions.

The model will be used to simulate the preferred CMA levee setback alternatives when those are available. During incorporation of preferred CMA into the 2-D model it is possible that further refinements will be made to the existing condition model. If this occurs, the calibration, verification and existing conditions simulations will be updated.

The results of the 2-D model runs will also be used to refine the 1-D model representation of the CMA's if necessary.

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EXHIBITS

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Levee Improvement Program Southport Early Implementation Project

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Checked
B. JONES, HDR

Drawn
A. ARNOLD, HDR

PROJECT_BOUNDARY
COUNTIES
INTERSTATE
US HIGHWAY
STATE HIGHWAY
WATERWAYS AND WATER BODIES

Location Map

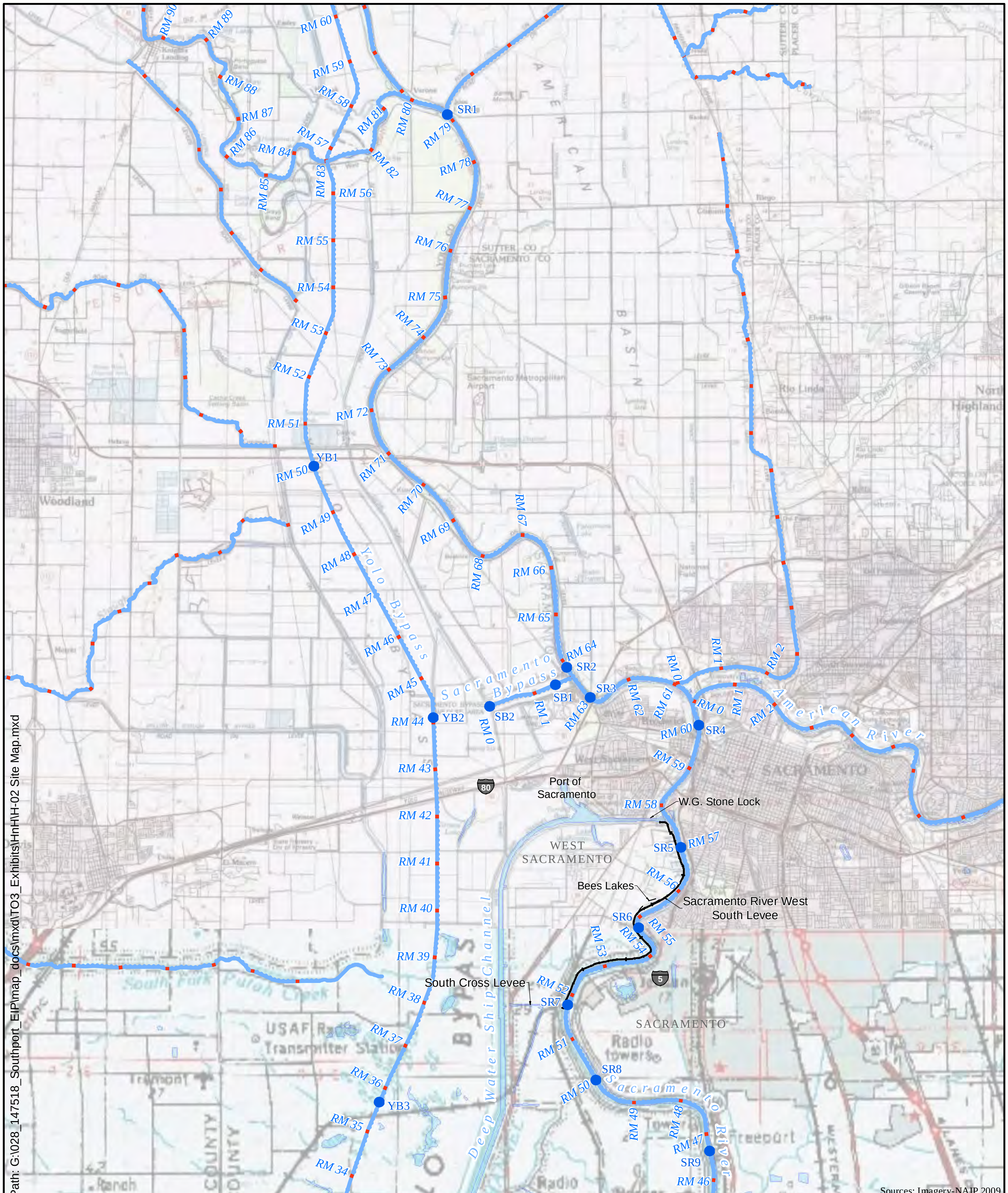
Exhibit H-1

Project Num: 453318-166505-028

Dec 2011

N
W E
S

0 1 2 3 4
Miles
1 in = 4 miles



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SB1 INDEX POINT LOCATION

WATERWAY CENTERLINE

RM 57 RIVER MILES

PROJECT STATIONING

Site Map

Exhibit H-2

Project Num: 453318-166505-028

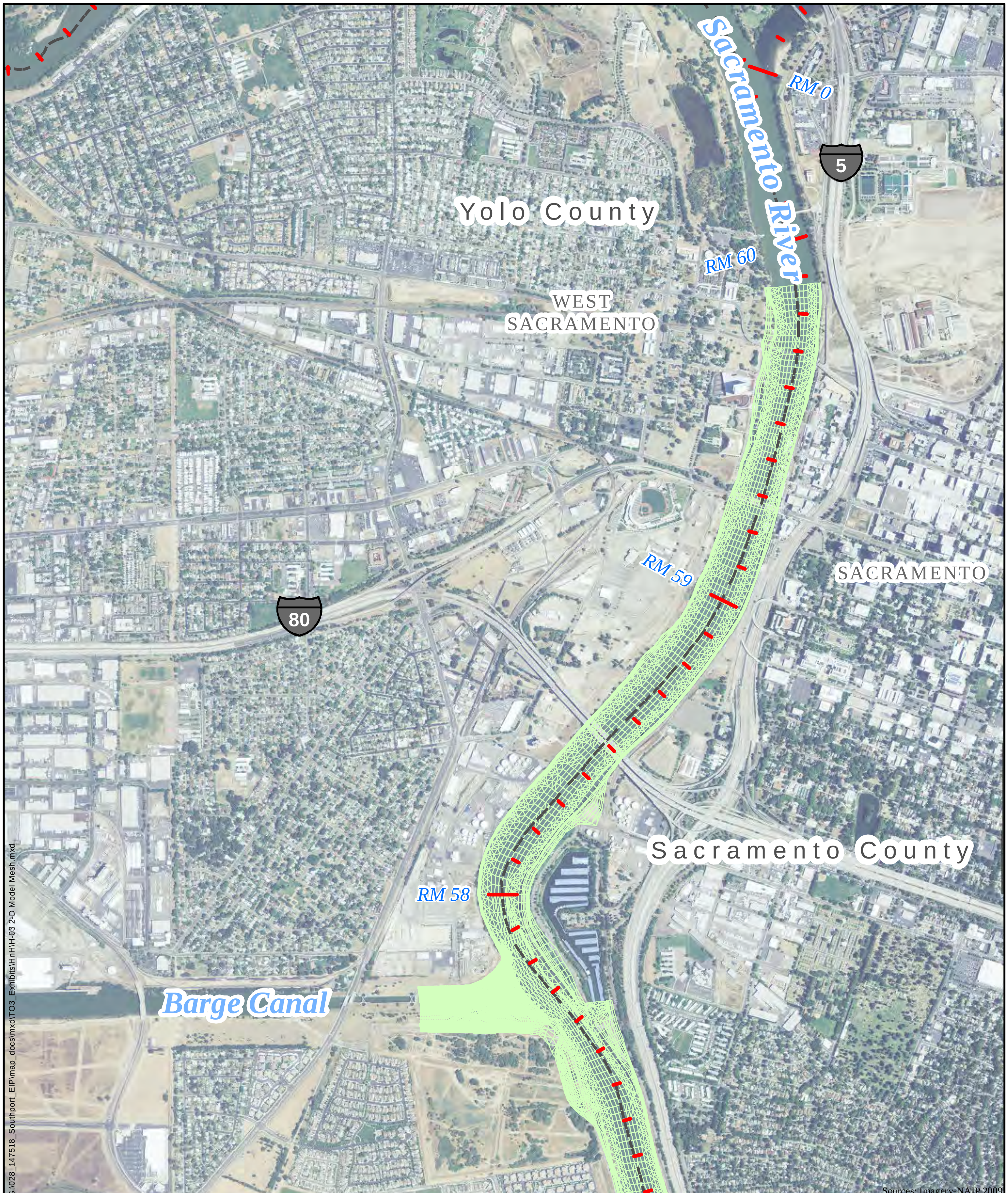
Dec 2011

N W E S

0 1 2 3

Miles

1 in = 3 miles



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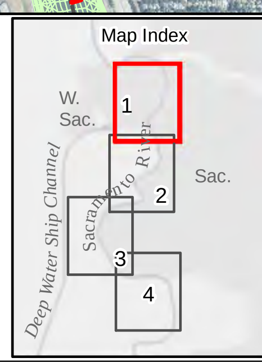
RM 45 River Miles Tics
Counties
Mesh



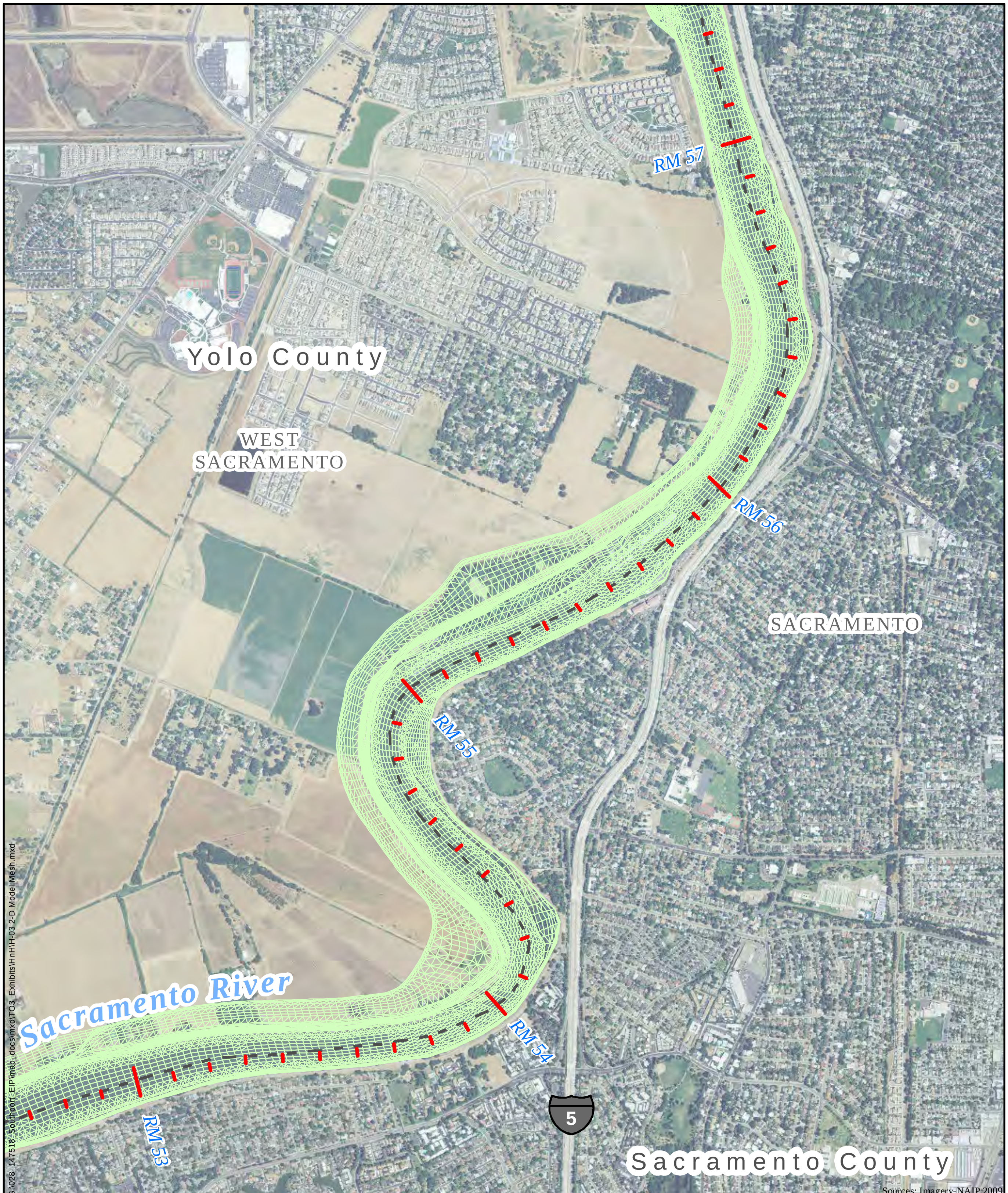
2-D Model Mesh

Exhibit H-3

Project Num: 453318-166505-028

Dec 2011

0 1,000 2,000
Feet
1 inch = 2,000 feet



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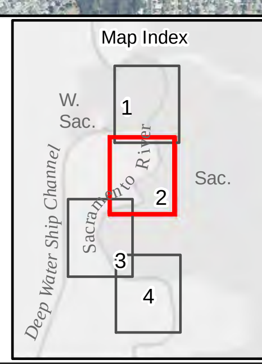
RM 45 — River Miles Tics
Counties
Mesh



2-D Model Mesh

Exhibit H-3

Project Num: 453318-166505-028

Dec 2011

N
W E
S

0 1,000 2,000
Feet
1 inch = 2,000 feet

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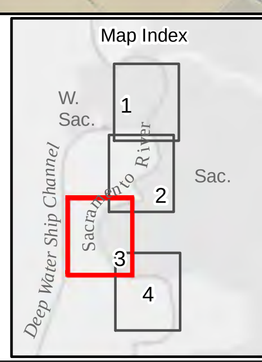
Counties
Mesh



2-D Model Mesh

Exhibit H-3

Project Num: 453318-166505-028

Dec 2011

0 1,000 2,000
Feet
1 inch = 2,000 feet

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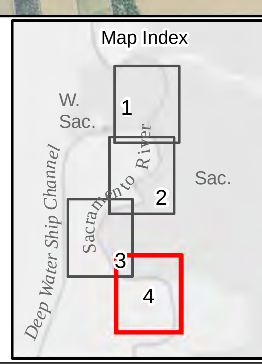


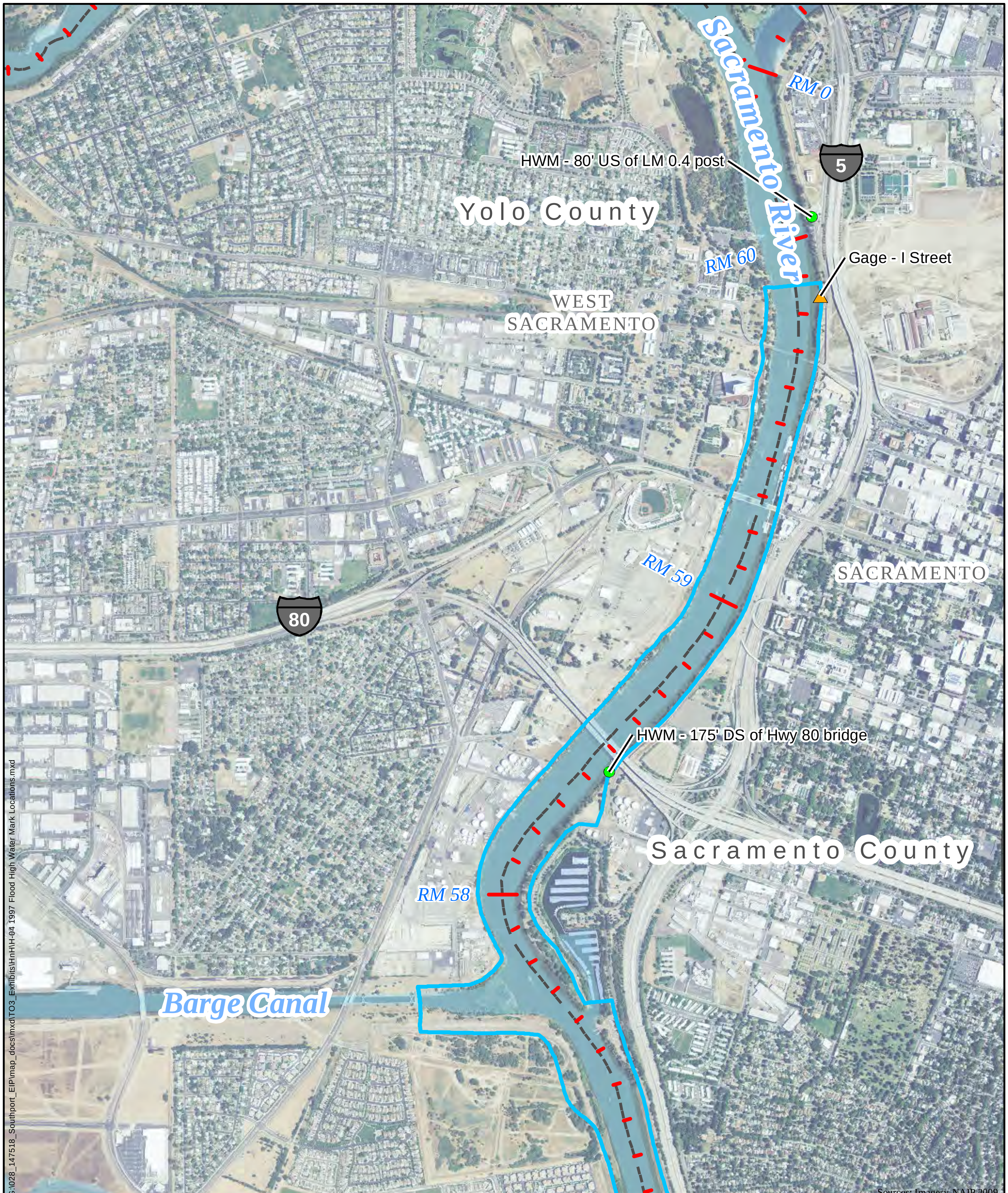
2-D Model Mesh

Exhibit H-3

Project Num: 453318-166505-028

Dec 2011

0 1,000 2,000
Feet
1 inch = 2,000 feet



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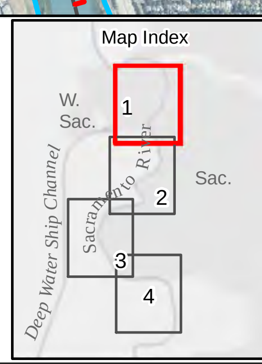
RM 45 River Mile Tics
Mesh Boundary
Counties
Waterways and Water Bodies

Flood1997

Gage
Surveyed High Water Mark

Map Index



1997 Flood High Water Mark Locations

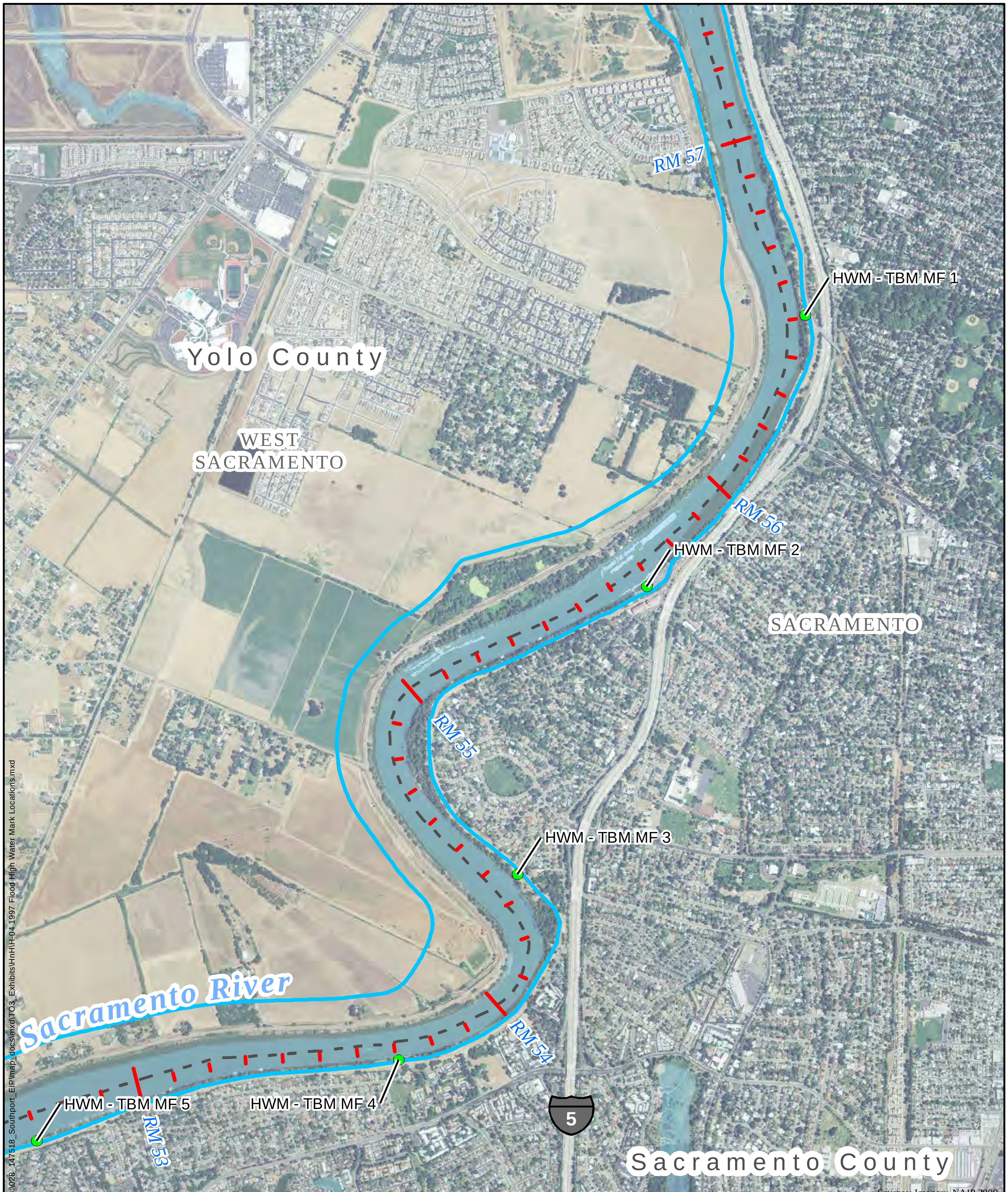
Exhibit H-4

Project Num: 453318-166505-028

Dec 2011

N W E S

0 1,000 2,000
Feet
1 inch = 2,000 feet



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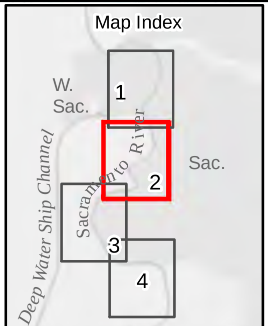
RM 45 River Mile Tics
Mesh Boundary
Counties
Waterways and Water Bodies

Flood1997

Gage
Surveyed High Water Mark

Map Index



1997 Flood High Water Mark Locations

Exhibit H-4

Project Num: 453318-166505-028

Dec 2011

0 1,000 2,000
Feet
1 inch = 2,000 feet

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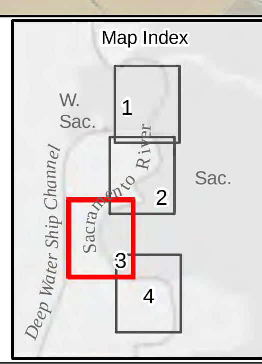
RM 45 River Mile Tics
Mesh Boundary
Counties
Waterways and Water Bodies

Flood1997

Gage
Surveyed High Water Mark



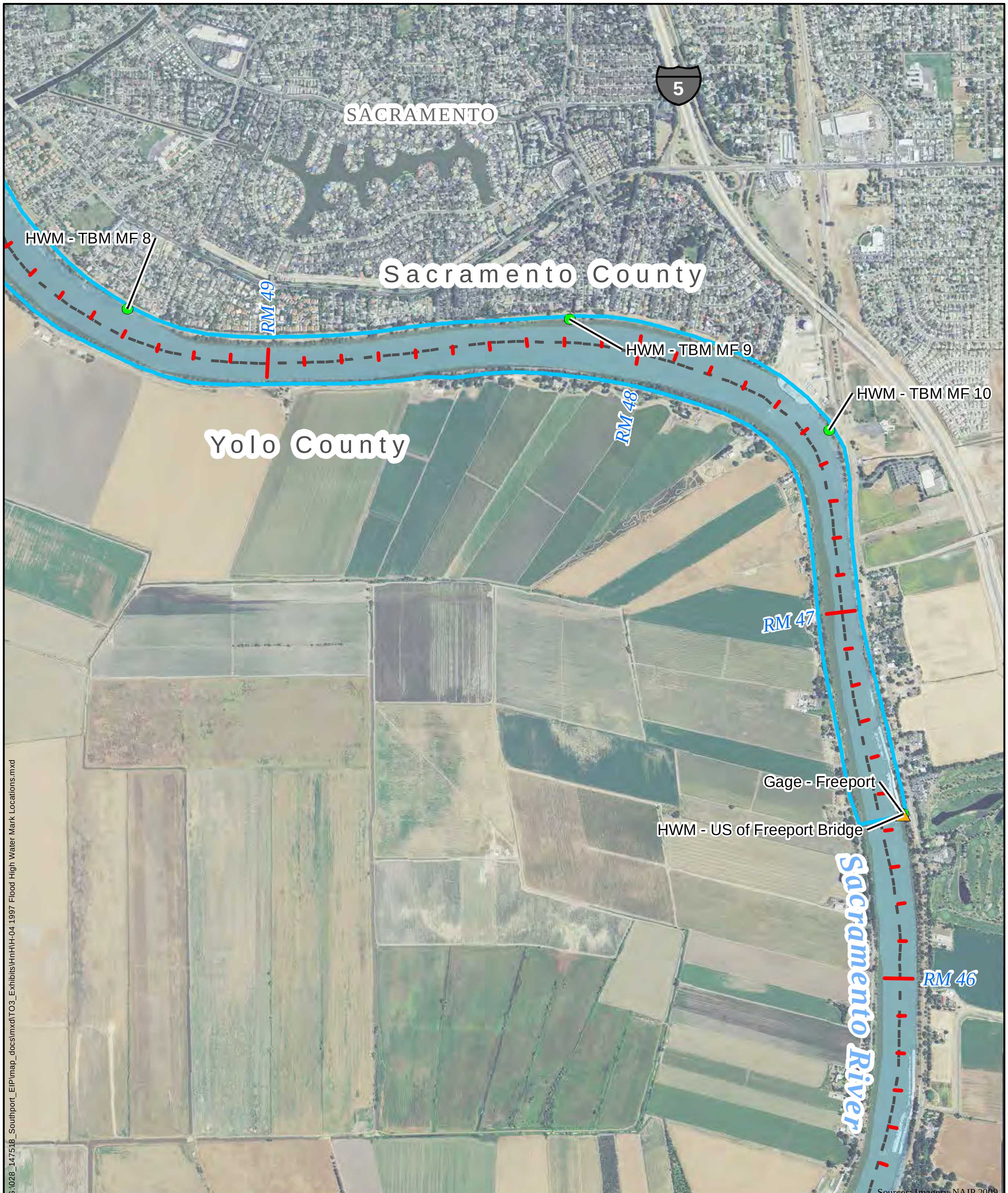
1997 Flood High Water Mark Locations

Exhibit H-4

Project Num: 453318-166505-028

Dec 2011

0 1,000 2,000
Feet
1 inch = 2,000 feet



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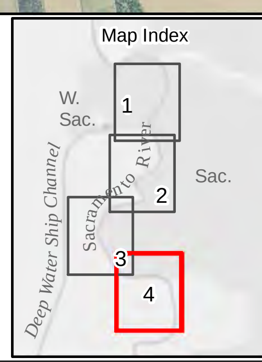
RM 45 River Mile Tics
Mesh Boundary
Counties
Waterways and Water Bodies

Flood1997

Gage
Surveyed High Water Mark



1997 Flood High Water Mark Locations

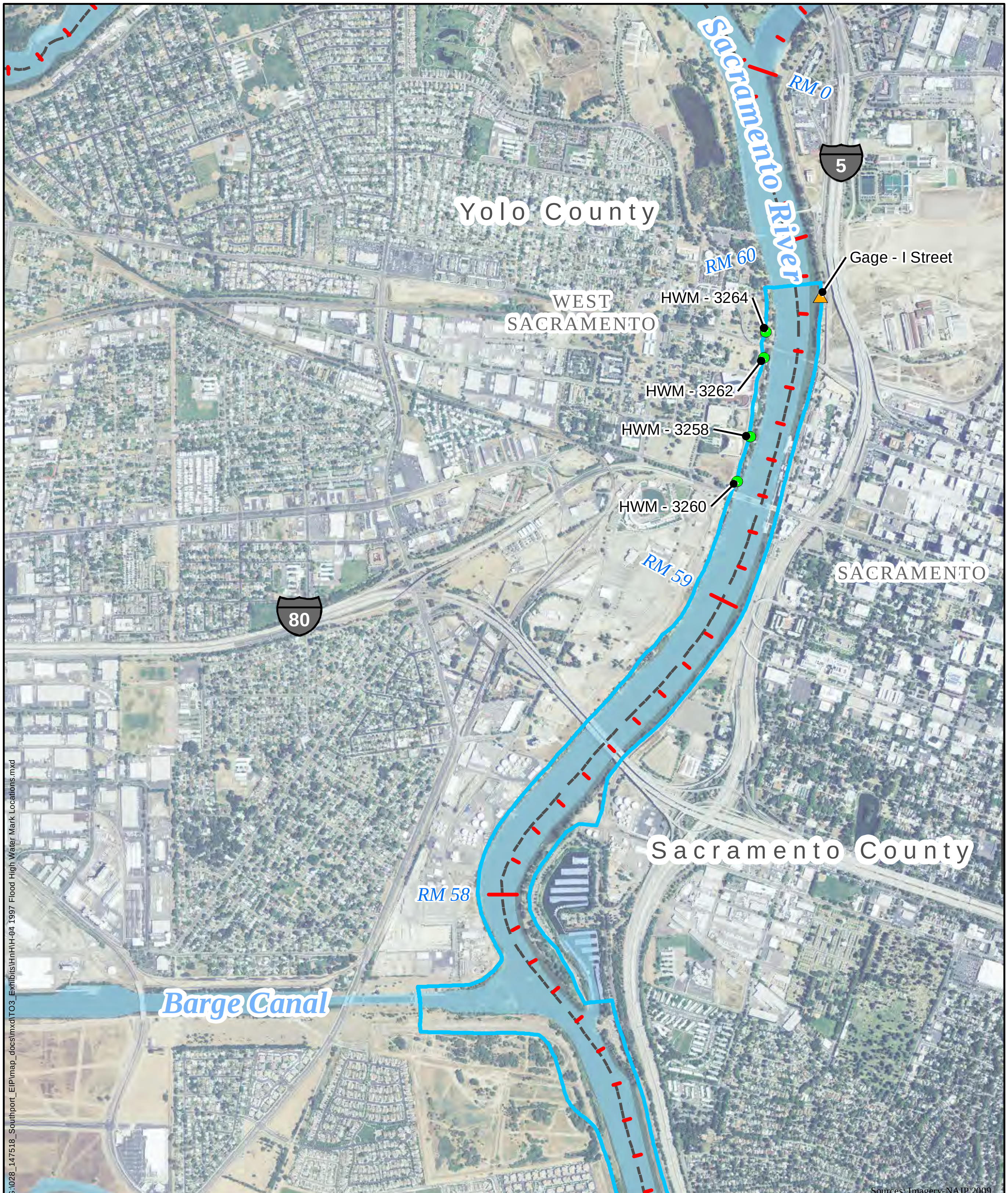
Exhibit H-4

Project Num: 453318-166505-028

Dec 2011

N
W E
S

0 1,000 2,000
Feet
1 inch = 2,000 feet



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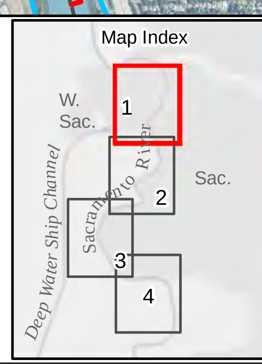
RM 45 River Miles Tics
Counties
Waterways and Water Bodies
Mesh Boundary

Flood2006

Gage
Surveyed High Water Mark



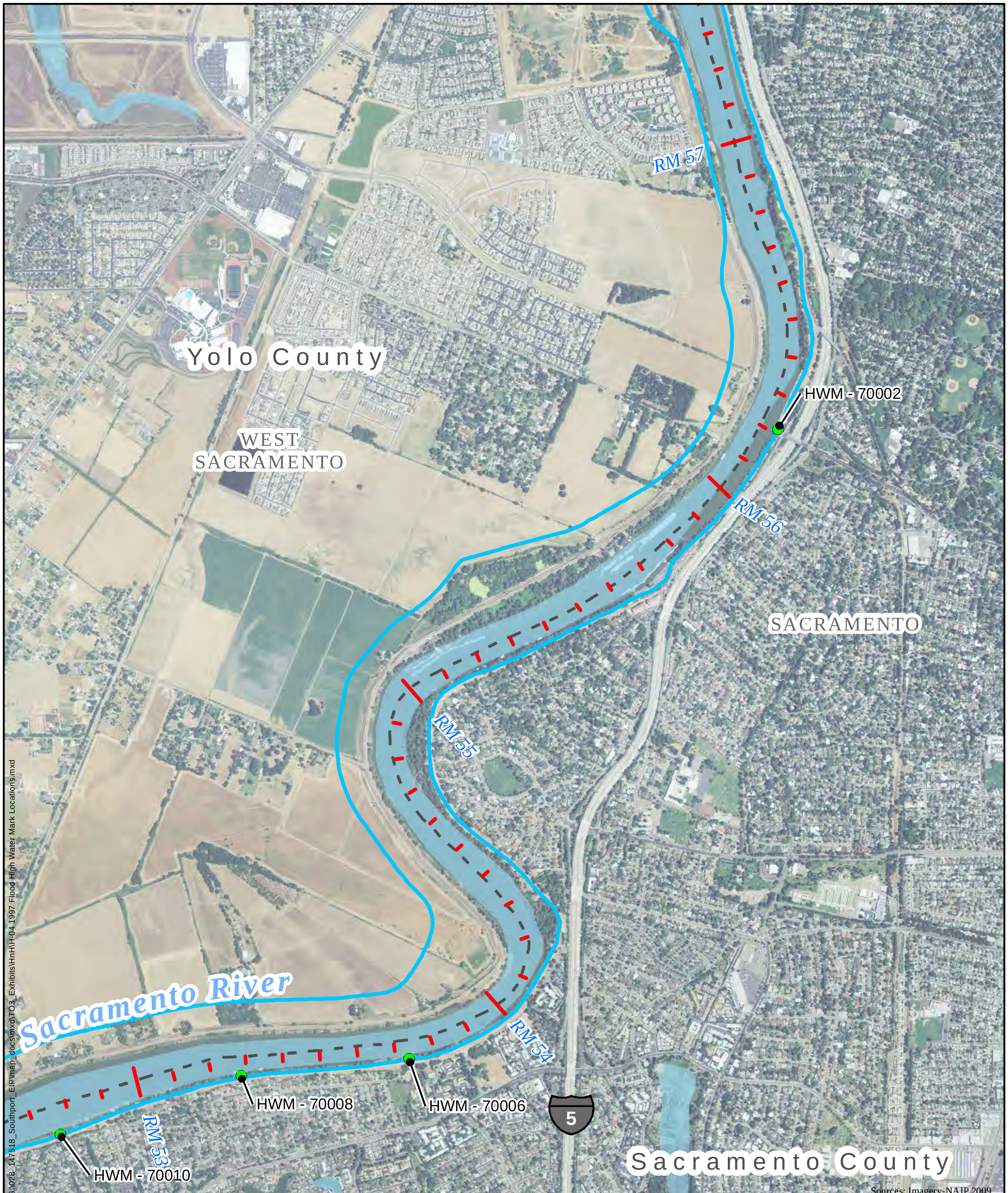
2006 Flood High Water Mark Locations

Exhibit H-5

Project Num: 453318-166505-028

Dec 2011

0 1,000 2,000
Feet
1 inch = 2,000 feet



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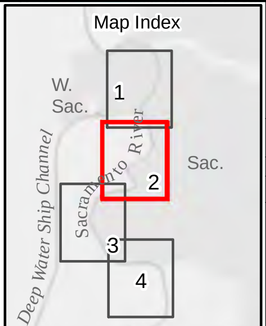
RM 45 — River Miles Tics
Counties
Waterways and Water Bodies
Mesh Boundary

Flood2006

Gage
Surveyed High Water Mark

Map Index



2006 Flood High Water Mark Locations

Exhibit H-5

Project Num: 453318-166505-028

Dec 2011

N
W
E
S

0 1,000 2,000

Feet

1 inch = 2,000 feet

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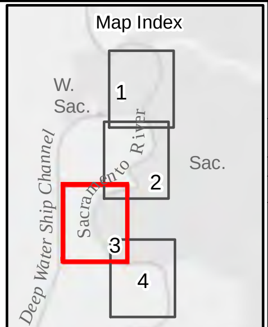
RM 45 River Miles Tics
Counties
Waterways and Water Bodies
Mesh Boundary

Flood2006

Gage
Surveyed High Water Mark

Map Index



2006 Flood High Water Mark Locations

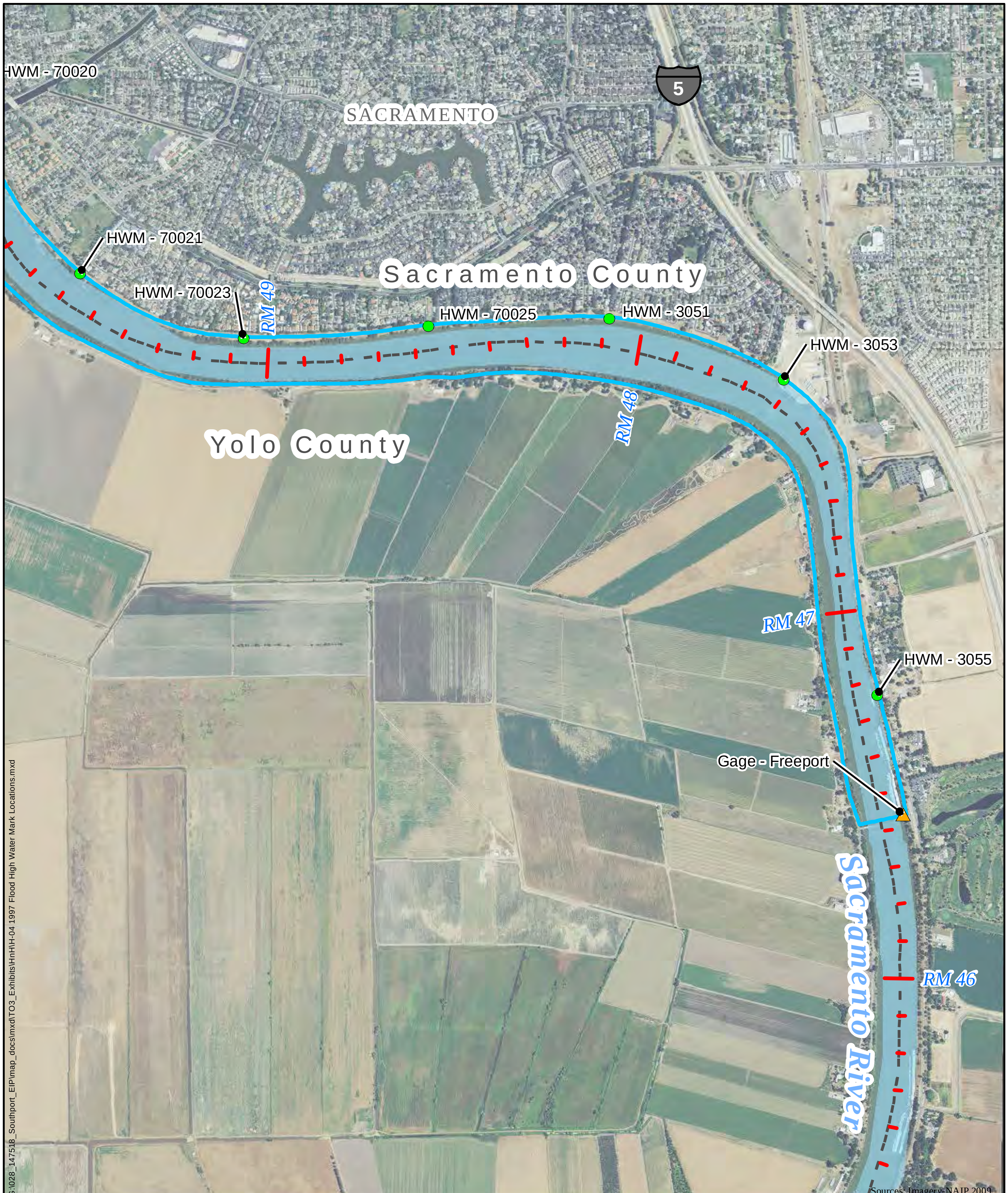
Exhibit H-5

Project Num: 453318-166505-028

Dec 2011

N W E S

0 1,000 2,000
Feet
1 inch = 2,000 feet



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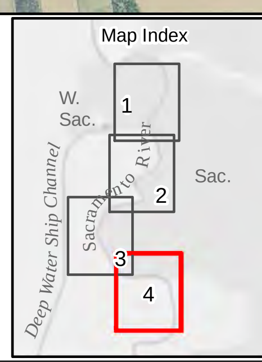
RM 45 River Miles Tics
Counties
Waterways and Water Bodies
Mesh Boundary

Flood2006

Gage
Surveyed High Water Mark

Map Index



2006 Flood High Water Mark Locations

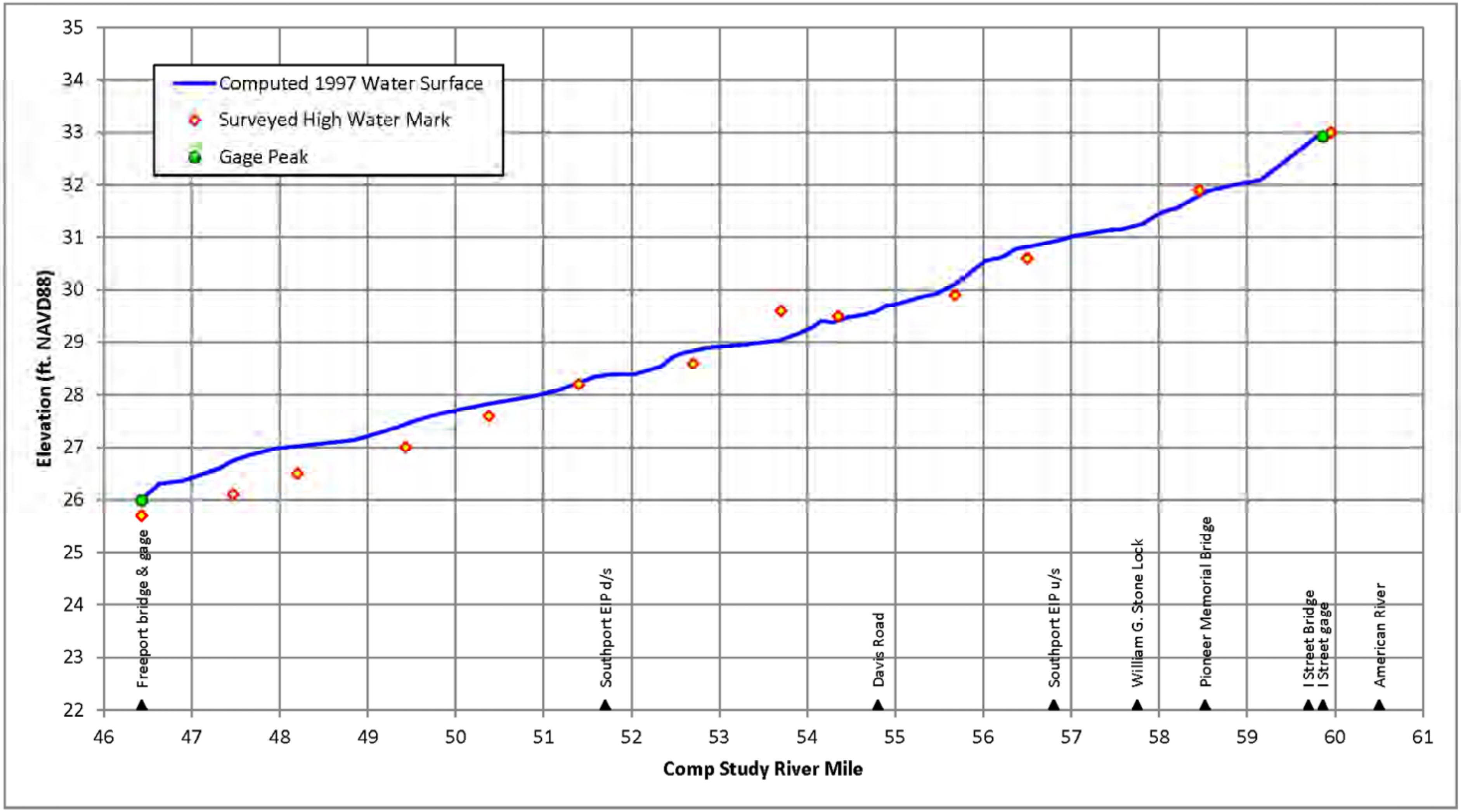
Exhibit H-5

Project Num: 453318-166505-028

Dec 2011

N
W E
S

0 1,000 2,000
Feet
1 inch = 2,000 feet



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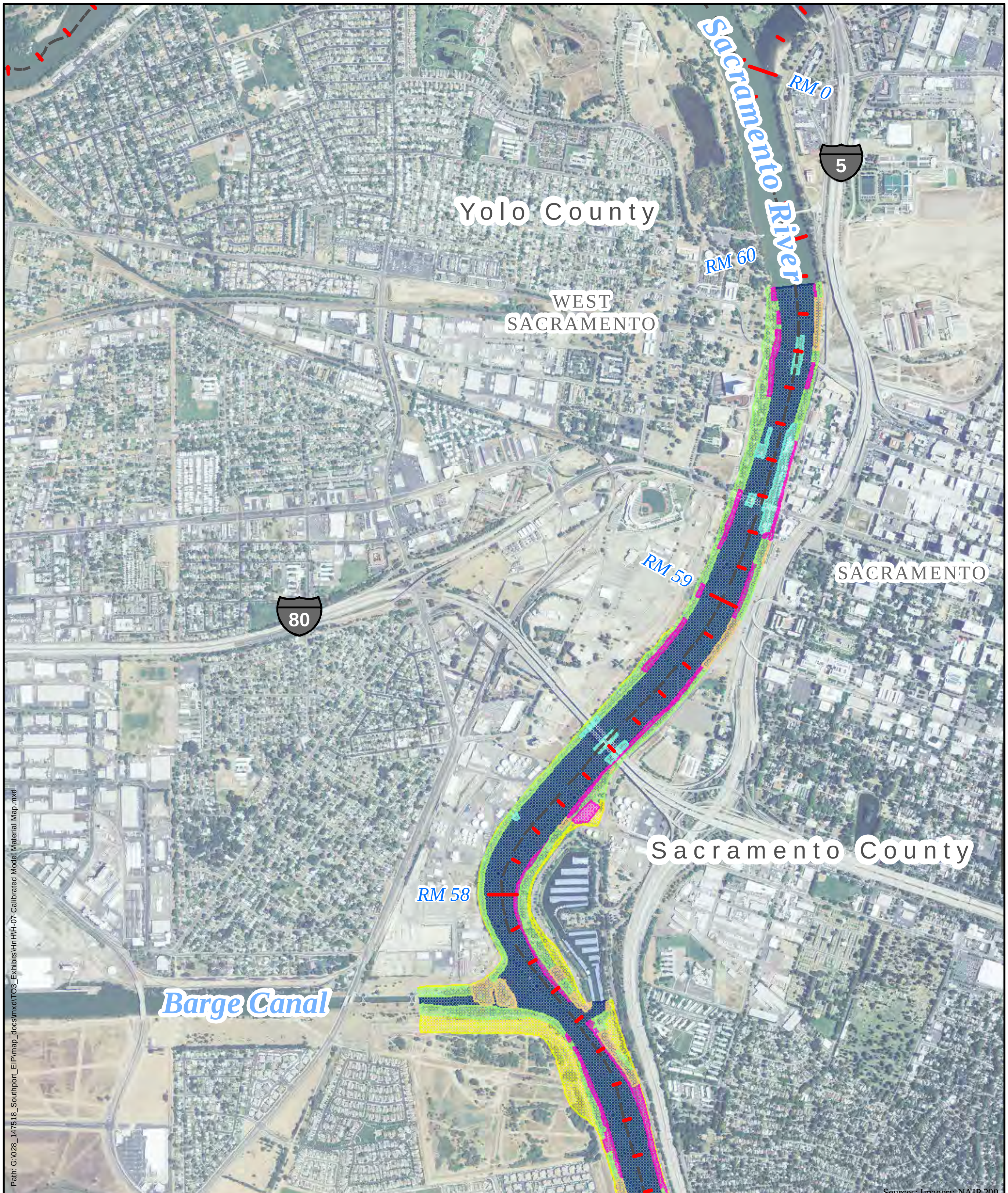
Calibration Water Surface Elevation Profile January 1997 Flood

Exhibit H-6

Project Num: 453318-166505-028

Dec 2011



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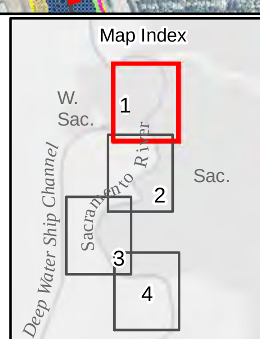
RM 45 — River Mile Tics
Counties

Materials

- Disable
- Channel (n = 0.024)
- No brush (n = 0.03)
- Scattered trees (n = 0.045)
- Dense trees (n = 0.065)
- Marina (n = 0.18)

Map Index



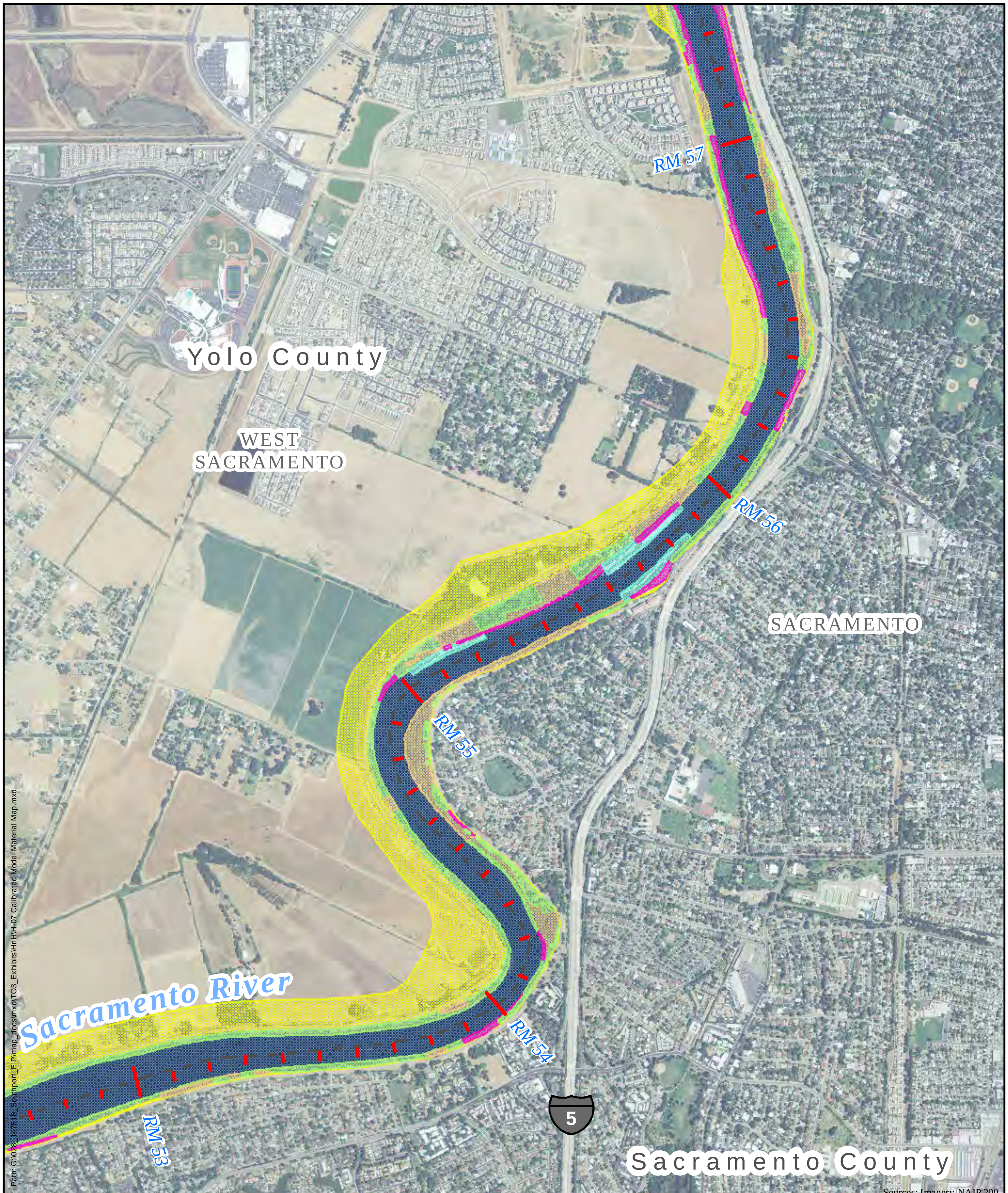
Calibrated Model Material Map

Exhibit H-7

Project Num: 453318-166505-028

Dec 2011

0 1,000 2,000
Feet
1 inch = 2,000 feet



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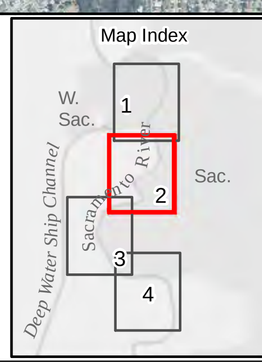
RM 45 River Mile Tics
Counties

Materials

- Disable
- Channel (n = 0.024)
- No brush (n = 0.03)
- Scattered trees (n = 0.045)
- Dense trees (n = 0.065)
- Marina (n = 0.18)



Calibrated Model Material Map

Exhibit H-7

Project Num: 453318-166505-028

Dec 2011

0 1,000 2,000
Feet
1 inch = 2,000 feet

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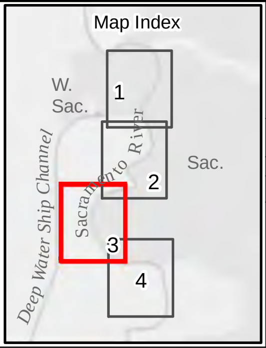
RM 45 River Mile Tics
Counties

Materials

- Disable
- Channel (n = 0.024)
- No brush (n = 0.03)
- Scattered trees (n = 0.045)
- Dense trees (n = 0.065)
- Marina (n = 0.18)

Map Index



Calibrated Model Material Map

Exhibit H-7

Project Num: 453318-166505-028

Dec 2011

0 1,000 2,000
Feet
1 inch = 2,000 feet

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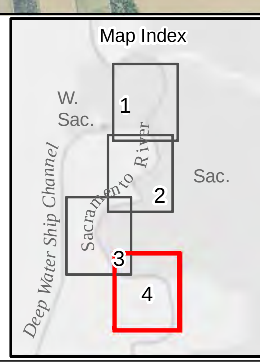
RM 45 — River Mile Tics

Counties

Materials

- Disable
- Channel (n = 0.024)
- No brush (n = 0.03)
- Scattered trees (n = 0.045)
- Dense trees (n = 0.065)
- Marina (n = 0.18)

Map Index



Calibrated Model Material Map

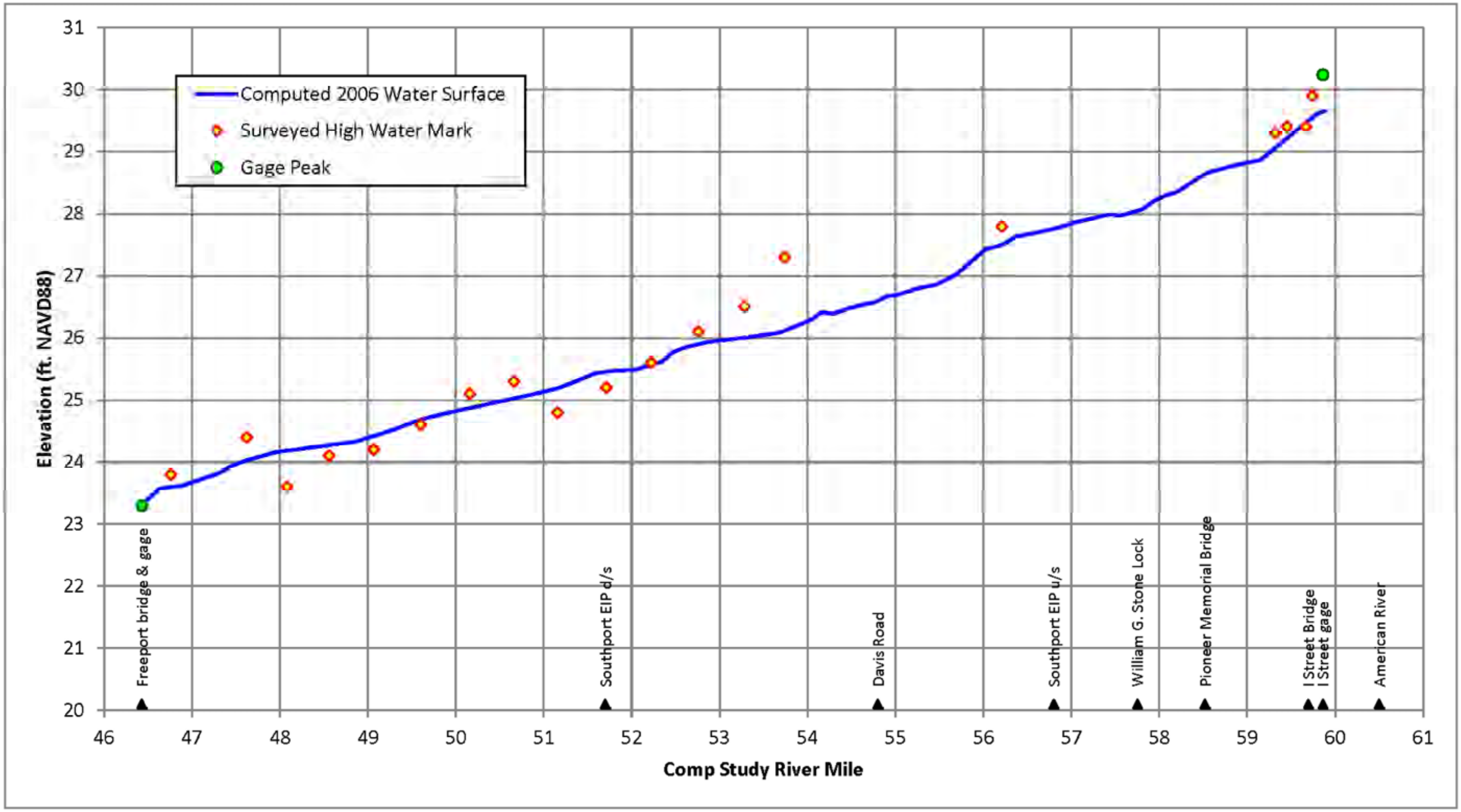
Exhibit H-7

Project Num: 453318-166505-028

Dec 2011

N
W E
S

0 1,000 2,000
Feet
1 inch = 2,000 feet



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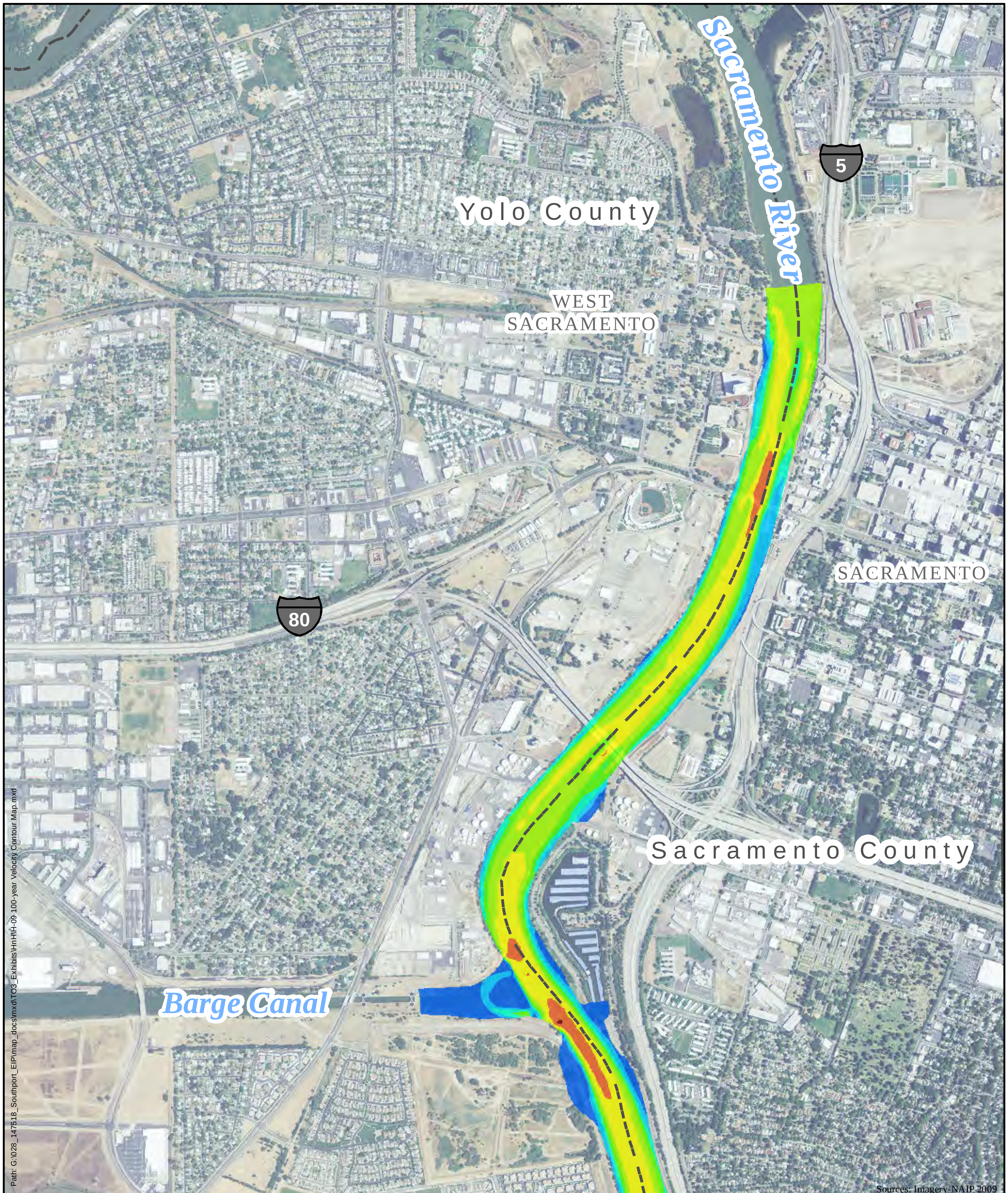
Verification Water Surface Elevation Profile January 2006 Flood

Exhibit H-8

Project Num: 453318-166505-028

Dec 2011



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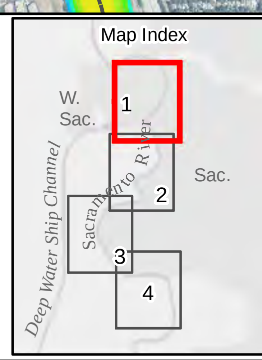
C. PLOTTS, HDR

Counties

Velocity (fps)

- 0 to 1.0
- 1.0 to 2.0
- 2.0 to 3.0
- 3.0 to 4.0
- 4.0 to 5.0
- 5.0 to 6.0
- 6.0 to 7.0
- 7.0 to 8.0
- Greater than 7.0



Velocity Contour map 100-year Flood Event

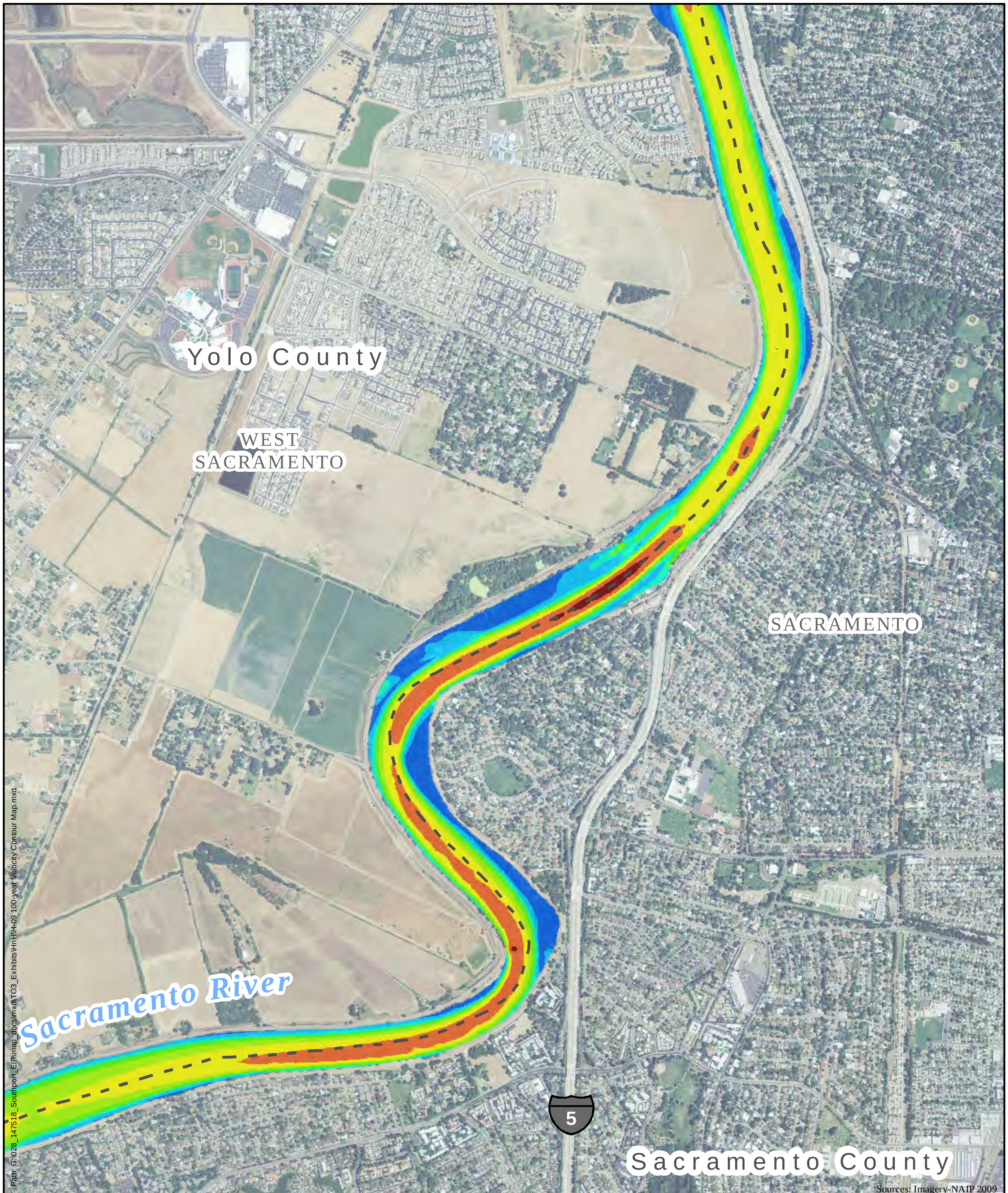
Exhibit H-9

Project Num: 453318-166505-028

Dec 2011

N
W E
S

0 1,000 2,000
Feet
1 inch = 2,000 feet



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Levee Improvement Program Southport Early Implementation Project

Project Manager
M. VECCHIO, HDR

Designed
M. ARCHER, MBK

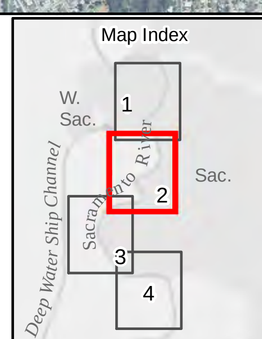
Checked
B. JONES, HDR

Drawn
C. PLOTTS, HDR

Counties

Velocity (fps)

- 0 to 1.0
- 1.0 to 2.0
- 2.0 to 3.0
- 3.0 to 4.0
- 4.0 to 5.0
- 5.0 to 6.0
- 6.0 to 7.0
- 7.0 to 8.0
- Greater than 7.0



Velocity Contour map 100-year Flood Event

Exhibit H-9

Project Num: 453318-166505-028

Dec 2011

0 1,000 2,000
Feet
1 inch = 2,000 feet

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- 0 to 1.0
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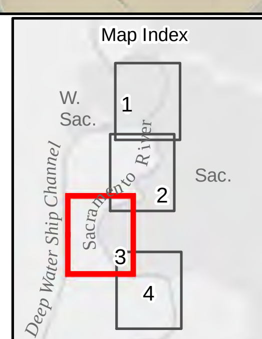
- 4.0 to 5.0
- 5.0 to 6.0
- 6.0 to 7.0
- 7.0 to 8.0
- Greater than 7.0



Velocity Contour map 100-year Flood Event

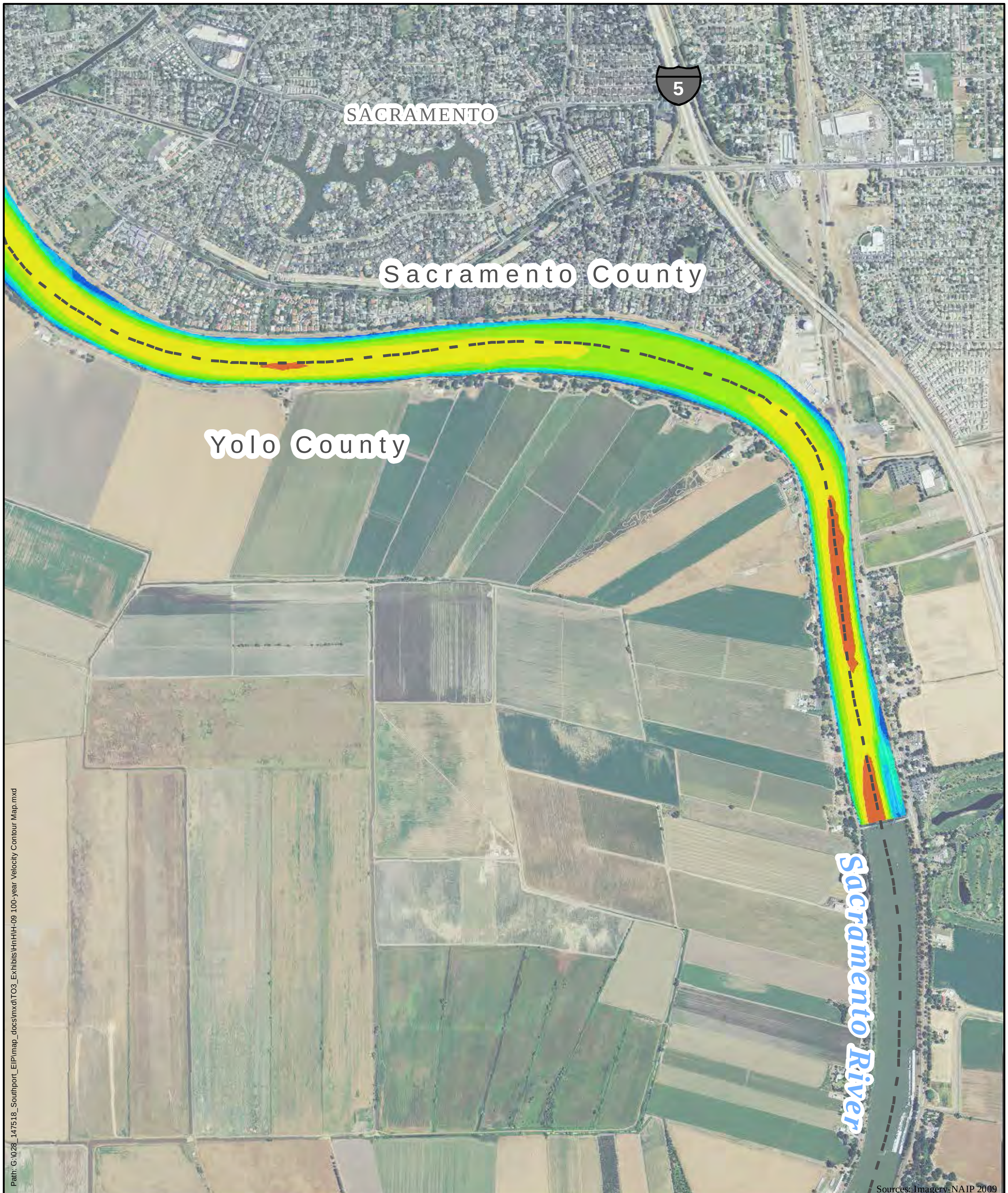
Exhibit H-9

Project Num: 453318-166505-028

Dec 2011

N
W E
S

0 1,000 2,000
Feet
1 inch = 2,000 feet



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- 1.0 to 2.0
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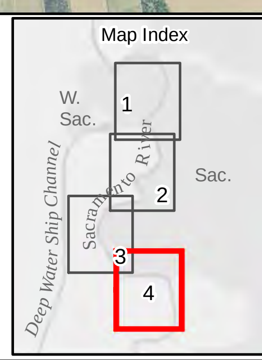


Velocity Contour map 100-year Flood Event

Exhibit H-9

Project Num: 453318-166505-028

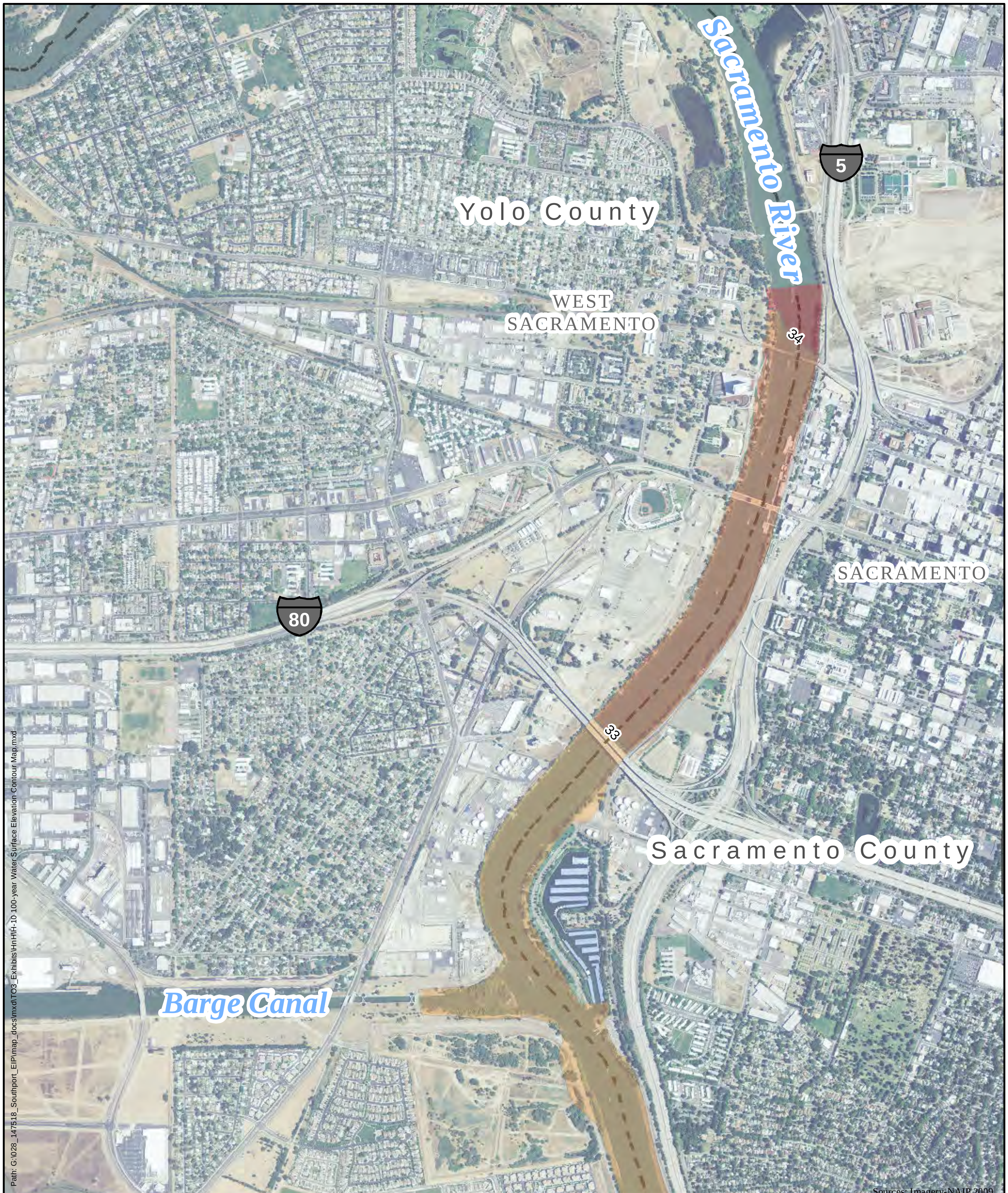
Dec 2011

N W E S

0 1,000 2,000

Feet

1 inch = 2,000 feet



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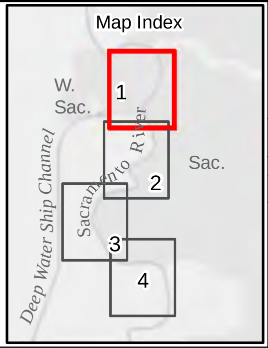
Counties

Vertical Datum feet NAVD88

Map Index



Water Surface Elevation Contour map 100-year Flood Event

Exhibit H-10

Project Num: 453318-166505-028

Dec 2011

0 1,000 2,000
Feet
1 inch = 2,000 feet

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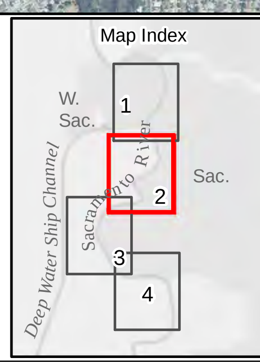
Counties

Vertical Datum feet NAVD88



Water Surface Elevation Contour map 100-year Flood Event

Exhibit H-10

Project Num: 453318-166505-028

Dec 2011

N W E S

0 1,000 2,000
Feet
1 inch = 2,000 feet

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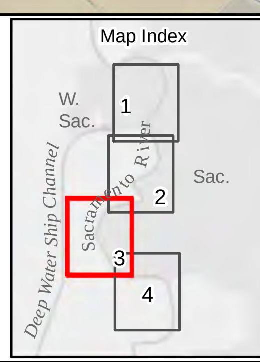
Counties

Vertical Datum feet NAVD88



Water Surface Elevation Contour map 100-year Flood Event

Exhibit H-10

Project Num: 453318-166505-028

Dec 2011

0 1,000 2,000
Feet
1 inch = 2,000 feet

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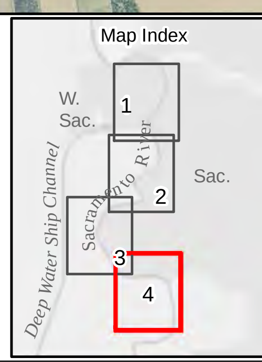


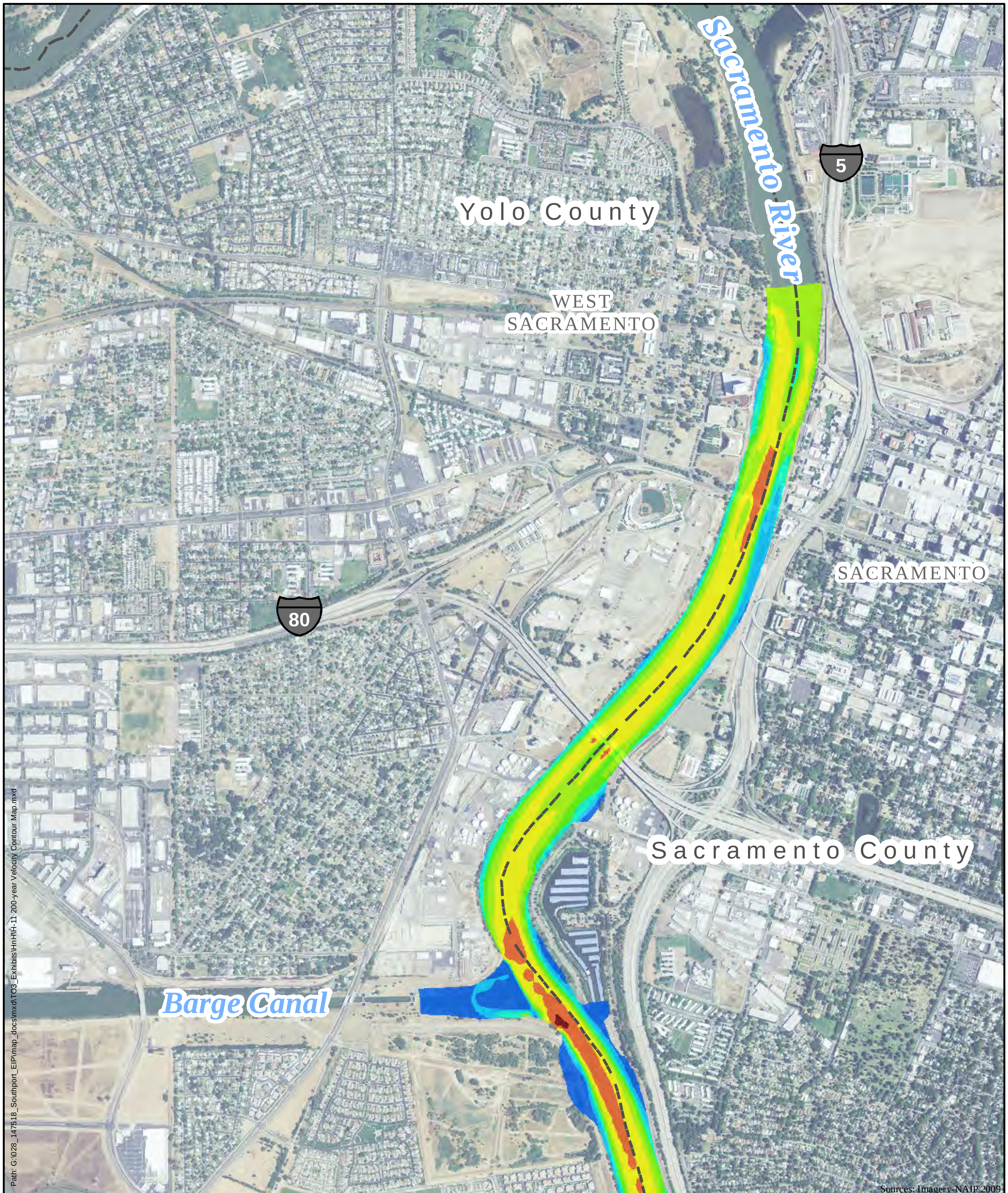
Water Surface Elevation Contour map 100-year Flood Event

Exhibit H-10

Project Num: 453318-166505-028

Dec 2011

0 1,000 2,000
Feet
1 inch = 2,000 feet



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- 0 to 1.0
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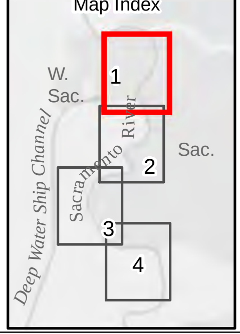
Map Index



Velocity Contour map 200-year Flood Event

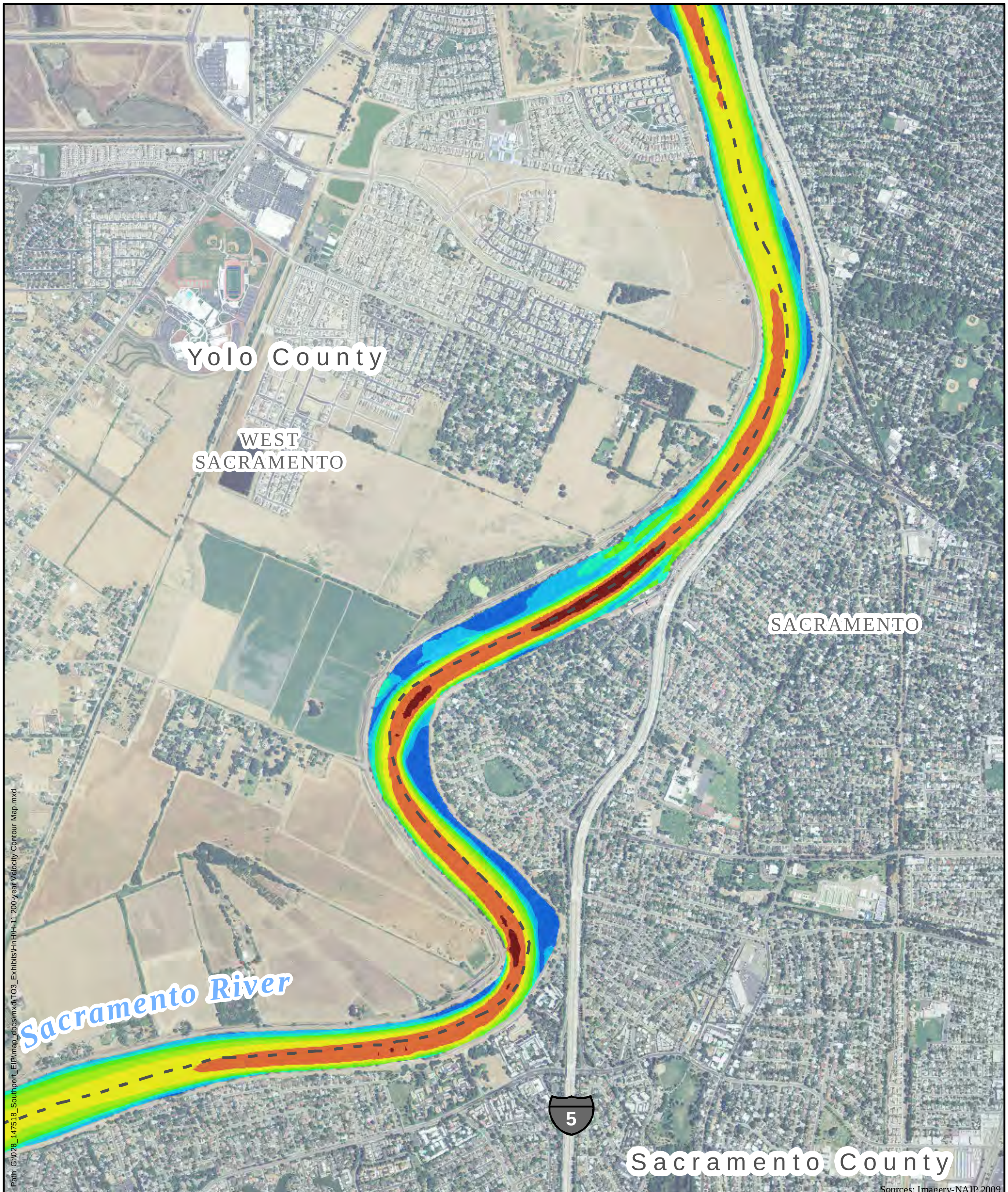
Exhibit H-11

Project Num: 453318-166505-028

Dec 2011

N
W E
S

0 1,000 2,000
Feet
1 inch = 2,000 feet



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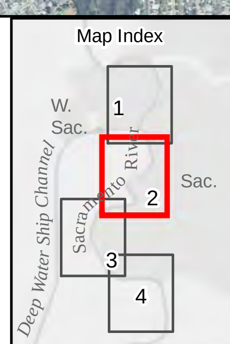

Velocity Contour map 200-year Flood Event

Exhibit H-11

Project Num: 453318-166505-028

Dec 2011

0 1,000 2,000
Feet
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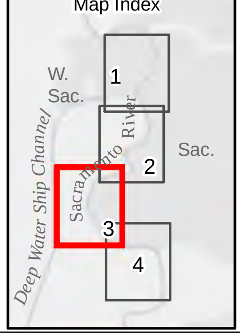
Map Index



Velocity Contour map 200-year Flood Event

Exhibit H-11

Project Num: 453318-166505-028

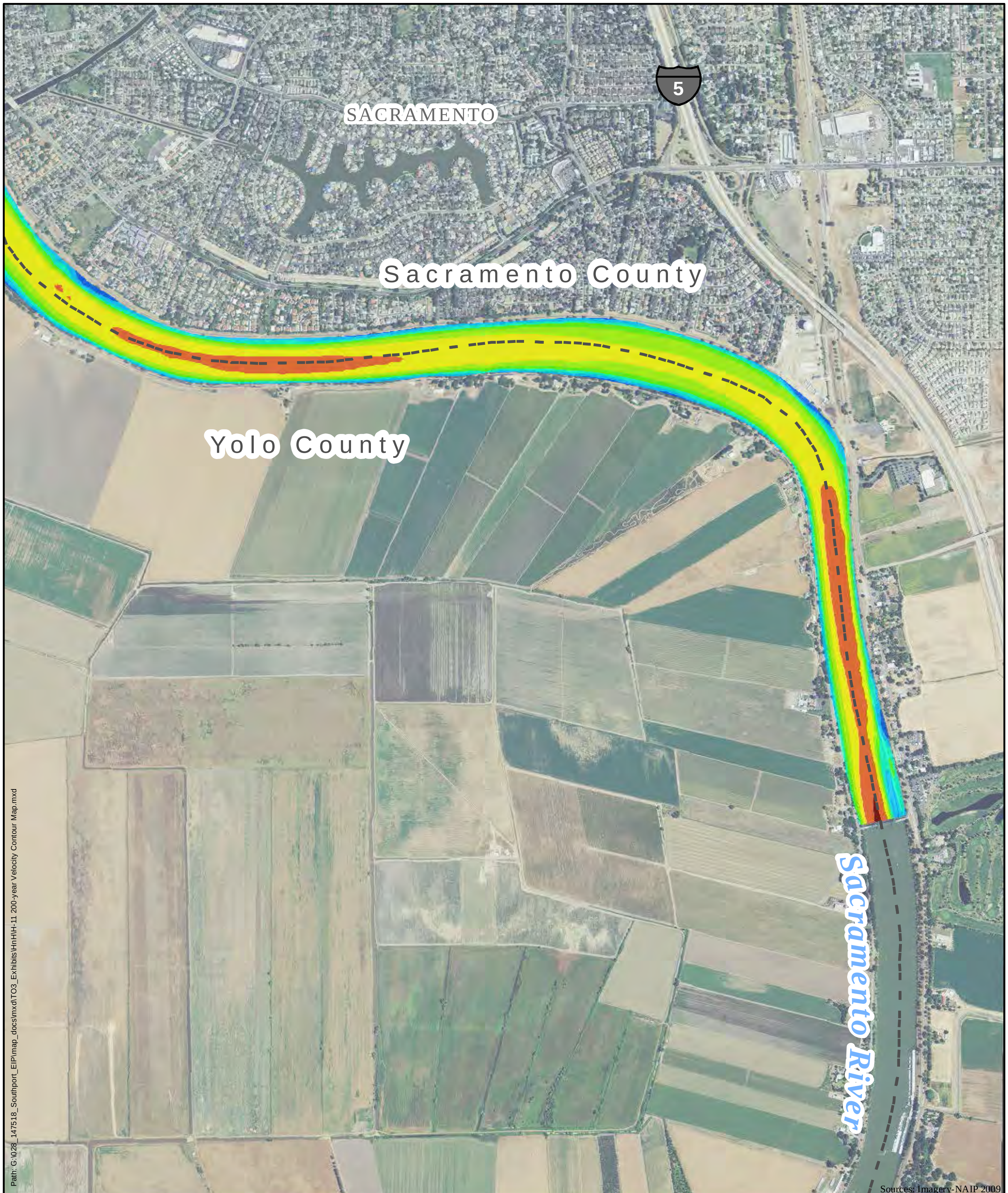
Dec 2011

N W E S

0 1,000 2,000

Feet

1 inch = 2,000 feet



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- 5.0 to 6.0
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- 7.0 to 8.0
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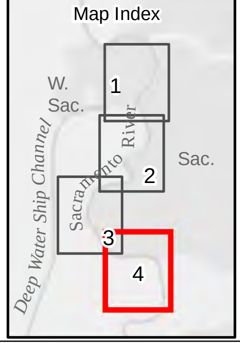
Map Index



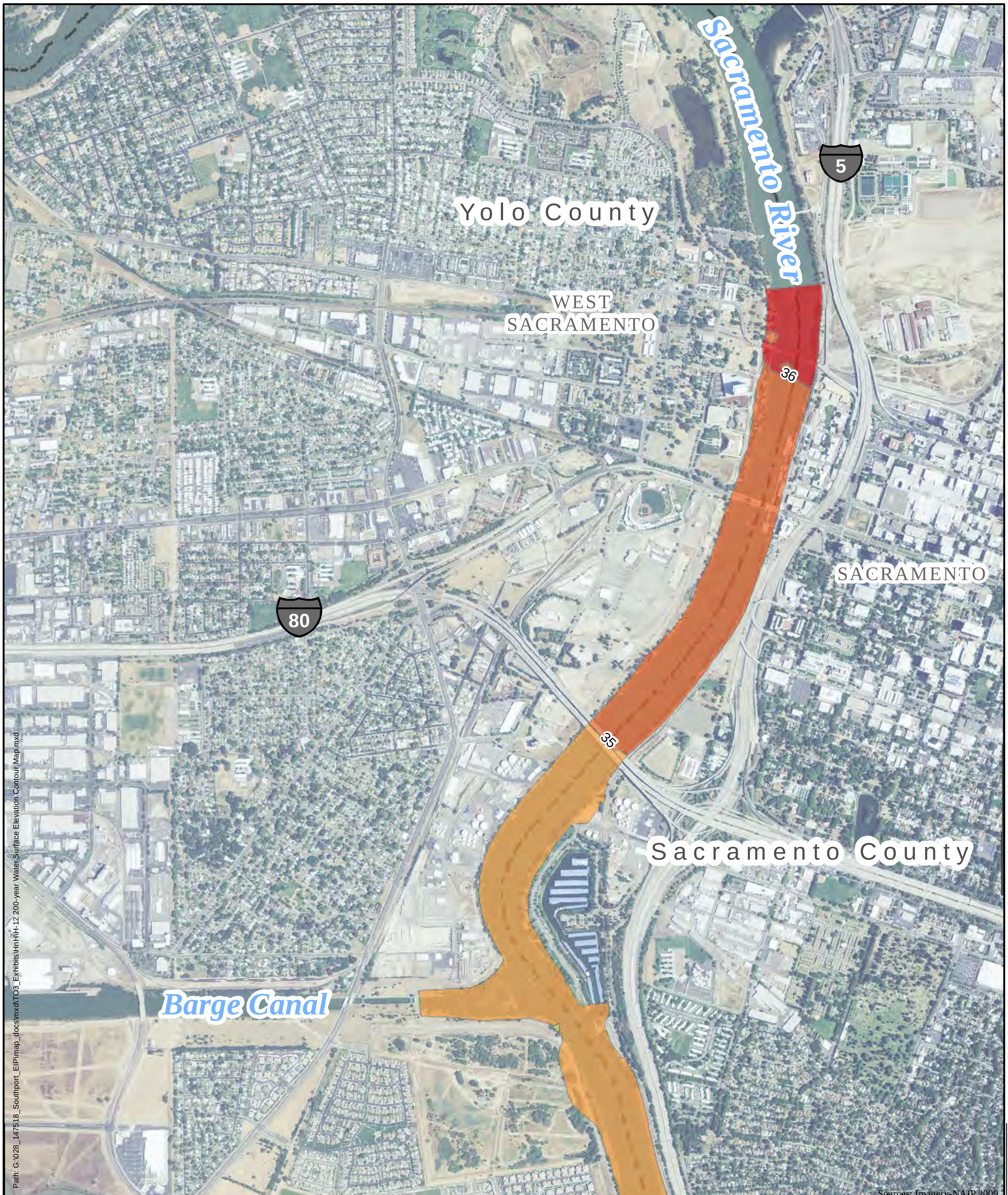
Velocity Contour map 200-year Flood Event

Exhibit H-11

Project Num: 453318-166505-028

Dec 2011

0 1,000 2,000
Feet
1 inch = 2,000 feet



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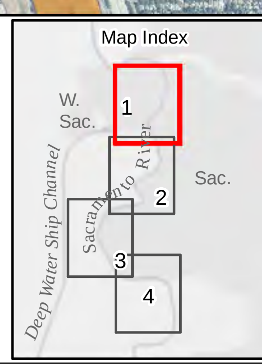
Counties

Vertical Datum feet NAVD88



Water Surface Elevation Contour map 200-year Flood Event

Exhibit H-12

Project Num: 453318-166505-028

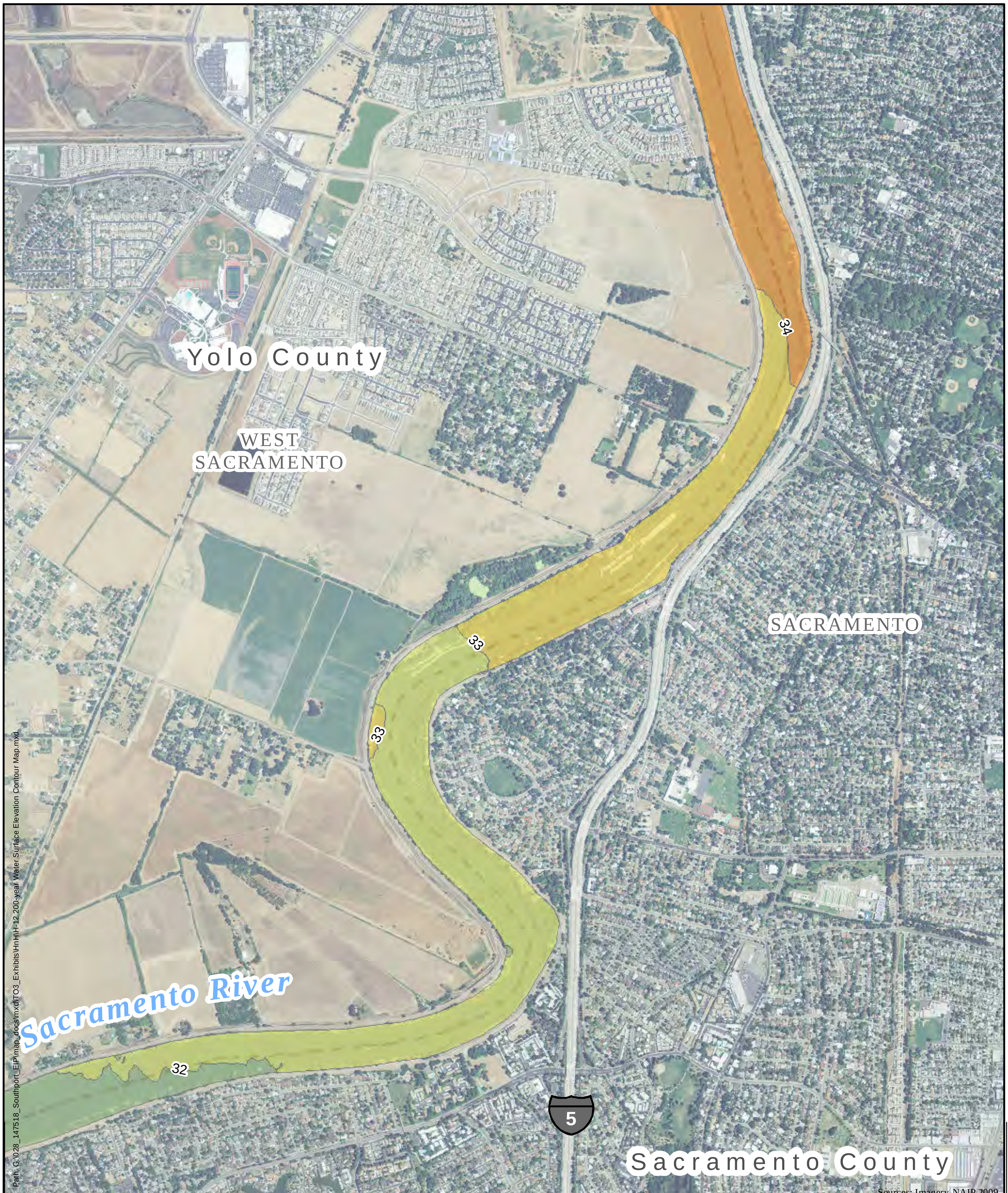
Dec 2011

N W E S

0 1,000 2,000

Feet

1 inch = 2,000 feet



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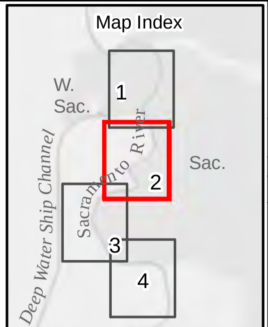
Counties

Vertical Datum feet NAVD88

Map Index



Water Surface Elevation Contour map 200-year Flood Event

Exhibit H-12

Project Num: 453318-166505-028

Dec 2011

N
W E
S

0 1,000 2,000
Feet
1 inch = 2,000 feet

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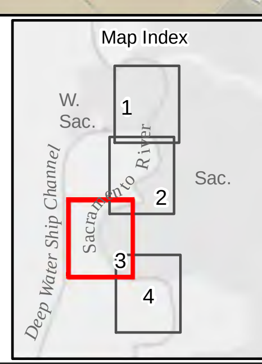
Counties

Vertical Datum feet NAVD88



Water Surface Elevation Contour map 200-year Flood Event

Exhibit H-12

Project Num: 453318-166505-028

Dec 2011

N
W E
S

0 1,000 2,000

Feet

1 inch = 2,000 feet

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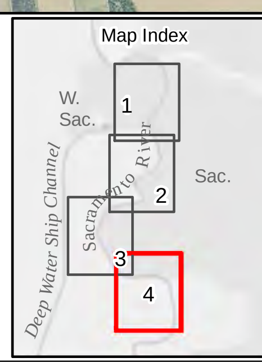
Vertical Datum feet NAVD88



Water Surface Elevation Contour map 200-year Flood Event

Exhibit H-12

Project Num: 453318-166505-028

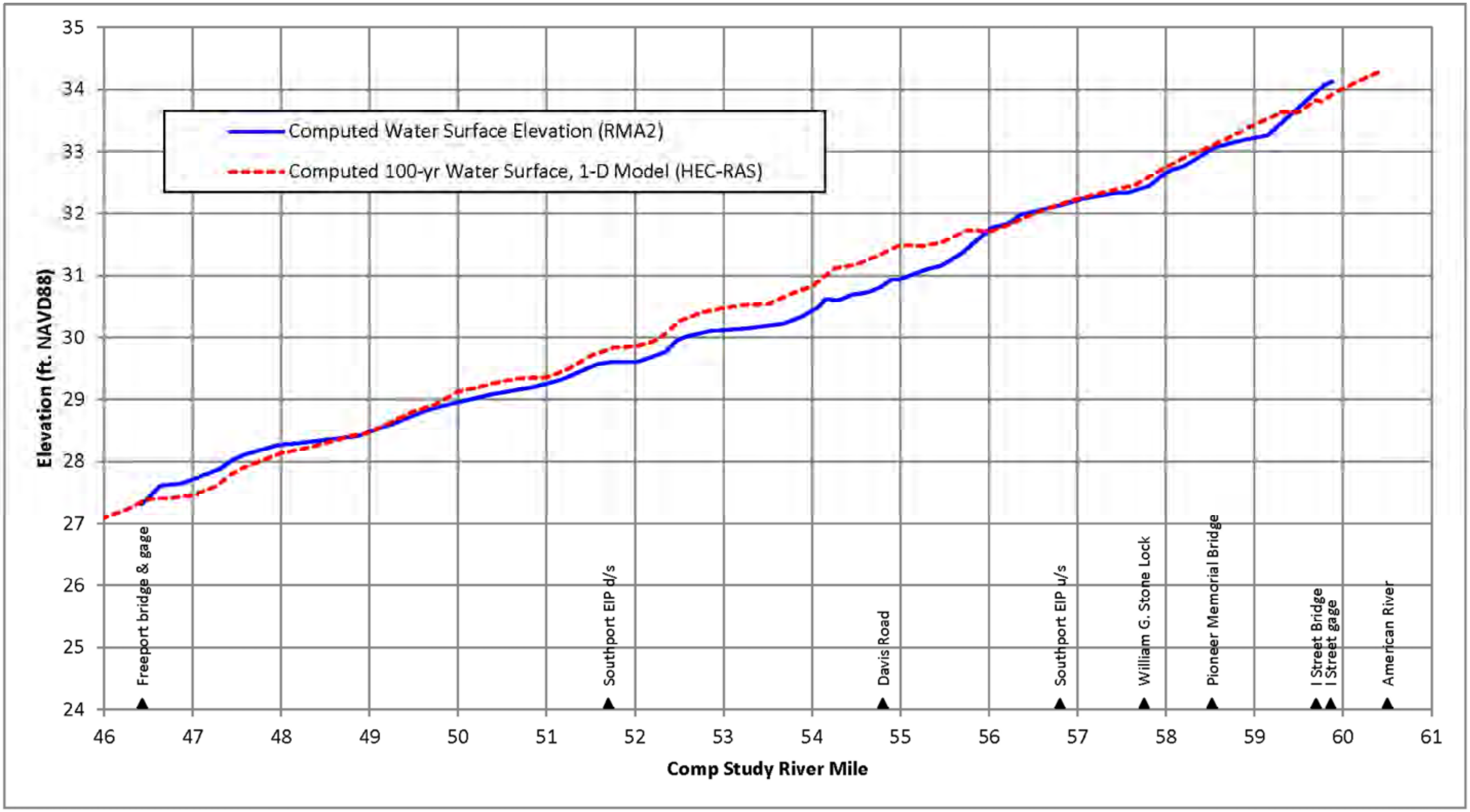
Dec 2011

N
W
E
S

0 1,000 2,000

Feet

1 inch = 2,000 feet



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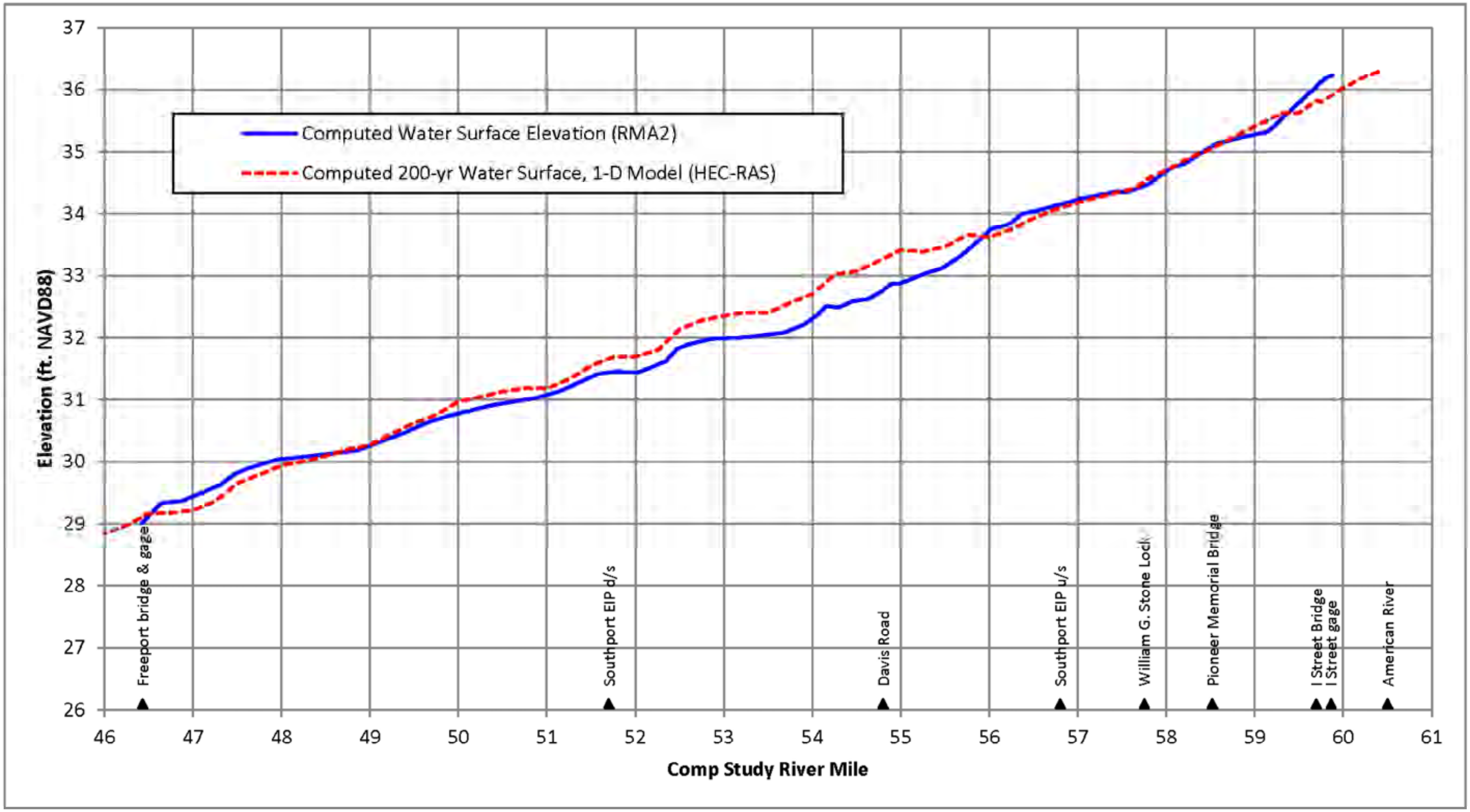
100-year Water Surface Elevation Profile

Exhibit H-13

Project Num: 453318-166505-028

Dec 2011



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200-year Water Surface Elevation Profile

Exhibit H-14

Project Num: 453318-166505-028

Dec 2011

Appendix B.6

Memorandum—Average Annual Inundation Duration of the Offset's Lower Floodplain—cbec inc., eco engineering, September 6, 2013

MEMORANDUM

Date:	09/06/13
To:	Michael Vecchio (HDR), Sergio Jimenez (HDR), Carl Jensen (ICF)
From:	John Stofleth, M.S., P.E., Chris Bowles, Ph.D., P.E., Poyom Riles, M.S., P.E.
Project:	12-1001 Southport EIP
Subject:	Average annual inundation duration of the offset's lower floodplain (draft)

cbec has been requested by program and project management to estimate the average annual duration of inundation for the lower floodplain terrace (elevation 10 ft NAVD 88) within the proposed offset areas. The frequency and duration of inundation of this area has implications for the viability of new plantings during the establishment period as well as informing construction staging design.

This task was accomplished by first determining the approximate discharge that would inundate the offset's lower floodplain terrace (10 ft NAVD 88). To estimate this discharge that correlates to a water surface elevation of 10 ft NAVD88 at a midpoint between the offset areas, cbec linearly interpolated between computed (MIKE 21C) water surface profiles associated with Sacramento River discharges of 18,099 cfs and 33,501 cfs (cbec, 2012). This interpolation indicates that a Sacramento River discharge of approximately 29,000 cfs will correlate to water surface elevation of 10 ft NAVD 88 and would allow the offset area's lower floodplain terrace to inundate.

In order to determine the average number of days per year that a 29,000 cfs discharge was exceeded, cbec analyzed the historic flow record (1970 – 2010) from the USGS gauge at Freeport immediately downstream of the project reach. By analyzing the average daily flows from this dataset, cbec calculated that:

1. The 29,000 cfs discharge was exceeded 77 days per year on average between 1970 and 2010 (Table 1). As indicated in Table 1, this annual average varies considerably from year to year with the standard deviation of 65 days and a maximum of 239 days. The months with the highest average flow occurs between January and March each year (Table 2).

Assuming the offset had been constructed prior to this period of record the following statistics are interesting to note:

2. In 10 years out 40 years (or 25% of the years on record) the offset floodplain would have been inundated for at least 5 months consecutively between November and May.
3. In 40 years of record, the offset floodplain would have not been inundated in October.
4. In 40 years of record, the offset floodplain would have not been inundated in August, September and October.
5. The latest in to the calendar year the floodplain would have ever been inundated would have been July 1983 and 1995 (or 5% of the July's on record).
6. 50% of the January's on record the offset floodplain would have been inundated.
7. 55% of the February's on record the offset floodplain would have been inundated.
8. 53% of the March's on record the offset floodplain would have been inundated.
9. 30% of the April's on record the offset floodplain would have been inundated.
10. 28% of the May's on record the offset floodplain would have been inundated.
11. 28% of the December's on record the offset floodplain would have been inundated.

.

Table 1. Number of days exceeding 29,000 cfs annually. Flow records derived from the USGS gauge at Freeport.

Year	Number of days flow exceeded 29,000 cfs
1970	115
1971	113
1972	28
1973	130
1974	151
1975	100
1976	0
1977	0
1978	113
1979	42
1980	85
1981	73
1982	199
1983	239
1984	110
1985	3
1986	75
1987	10
1988	13
1989	27
1990	3
1991	11
1992	21
1993	112
1994	3
1995	197
1996	146
1997	68
1998	209
1999	88
2000	73
2001	25
2002	35
2003	95
2004	78
2005	91
2006	161
2007	10
2008	19
2009	20
2010	56
Average	**77**
Standard Deviation	**65**
Minimum	**0**
Maximum	**239**

Table 2. Mean monthly discharge of the Sacramento River at Freeport. (Shaded green cells denote flows that would inundate the offset floodplain at 10 feet NAVD 88).

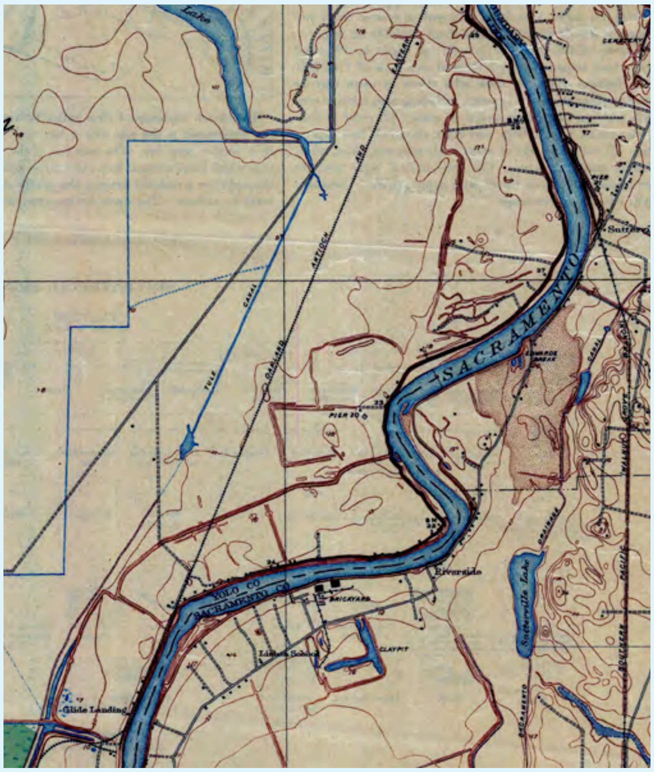
YEAR	Mean Monthly Discharge (ft^3/s) at Freeport # 11447650 (Calculation Period: 1970-01-01 to 2010-12-31)											
	Jan	**Feb**	**Mar**	**Apr**	**May**	**Jun**	**Jul**	**Aug**	**Sep**	**Oct**	**Nov**	**Dec**
1970	70,260	66,060	44,210	14,620	14,260	11,790	13,170	14,980	18,510	15,260	22,520	63,970
1971	52,320	31,200	30,480	38,270	29,190	27,550	20,980	22,460	24,390	16,070	15,850	21,760
1972	20,000	22,120	23,900	13,120	12,850	13,840	15,000	15,660	16,820	16,080	23,200	27,420
1973	60,130	65,260	51,640	20,670	16,420	14,940	15,170	16,120	17,490	16,720	48,040	61,630
1974	74,830	52,390	64,680	66,280	29,180	24,410	21,750	23,950	25,060	20,120	22,000	25,650
1975	19,430	47,520	50,940	33,170	30,260	23,710	18,280	19,500	20,380	19,170	22,250	25,550
1976	15,130	12,770	14,570	12,720	10,910	10,930	12,080	13,350	12,510	8,103	7,823	7,743
1977	9,802	8,003	6,573	5,961	7,597	6,865	8,248	7,687	6,838	4,494	6,687	11,750
1978	45,490	44,700	55,570	38,880	25,190	12,660	14,300	15,970	17,930	12,490	12,440	13,200
1979	23,190	32,440	29,160	16,550	17,980	12,210	16,410	15,680	14,570	12,580	15,200	20,320
1980	58,640	52,580	55,340	22,590	15,890	17,810	17,730	14,920	15,890	11,340	10,870	16,690
1981	18,510	24,240	24,510	17,220	13,780	10,730	15,300	14,850	12,800	9,895	32,940	62,060
1982	64,610	59,430	62,810	76,580	42,360	25,810	17,630	20,610	24,860	19,230	31,520	57,710
1983	47,510	79,040	78,290	60,510	62,280	48,380	31,000	25,040	24,620	21,150	48,820	74,510
1984	56,800	32,370	31,430	17,930	15,410	14,990	21,630	18,780	17,690	13,240	26,280	32,560
1985	16,790	18,270	14,310	12,500	13,430	13,310	16,040	13,450	12,190	9,711	10,420	16,110
1986	19,960	68,890	74,980	25,830	12,760	11,820	16,880	15,110	18,140	15,450	12,680	13,110
1987	13,170	17,400	21,580	11,830	9,996	10,070	15,140	14,440	11,630	9,509	8,129	15,740
1988	25,400	12,190	11,350	16,890	10,970	10,580	14,640	13,290	11,540	9,314	11,360	12,390
1989	12,830	12,060	43,370	21,270	13,800	13,290	18,770	18,320	16,460	14,270	14,830	15,400
1990	18,910	13,800	12,870	15,270	10,400	10,520	13,510	13,840	10,030	7,620	7,723	10,820
1991	8,984	8,133	25,750	10,880	7,332	8,930	9,514	9,515	9,948	9,398	6,958	9,259
1992	10,440	26,060	20,340	9,448	6,414	8,510	8,309	8,718	9,815	6,645	6,380	12,440
1993	48,260	48,600	49,340	43,210	24,950	30,470	19,860	21,080	15,830	13,820	12,090	20,340
1994	14,190	20,200	13,460	8,435	8,848	8,091	11,860	12,150	14,410	8,255	9,489	16,370
1995	62,210	58,180	71,920	61,440	63,180	38,960	29,230	18,720	23,270	14,150	12,610	24,570
1996	32,870	75,270	56,240	35,980	40,110	23,530	20,680	21,300	17,600	12,690	15,500	58,420
1997	87,110	57,330	24,470	13,490	11,410	15,220	20,840	18,720	14,000	12,010	14,790	22,010
1998	51,780	81,370	63,830	57,680	48,250	55,690	26,800	25,180	25,320	15,760	20,920	44,370
1999	34,500	67,150	56,840	30,680	19,740	17,240	22,240	18,030	15,830	12,380	13,840	16,550
2000	24,340	62,370	58,560	26,640	20,450	16,090	20,850	17,700	15,160	11,680	12,280	13,670
2001	17,190	20,870	24,700	12,310	9,060	12,380	14,940	13,220	12,360	8,370	12,300	27,380
2002	38,270	18,170	21,320	14,480	12,970	13,890	18,900	17,020	13,560	9,891	11,750	29,130
2003	51,940	36,090	22,920	21,590	40,540	22,280	22,430	19,580	15,350	11,000	12,450	27,790
2004	36,770	44,420	46,710	23,790	12,530	15,130	20,440	17,920	14,610	12,610	12,250	17,750
2005	33,680	24,870	30,370	22,130	40,220	28,650	19,670	17,250	17,930	14,070	13,390	35,460
2006	66,150	48,920	67,410	77,650	52,150	27,210	18,590	18,860	18,010	11,720	12,150	16,950
2007	13,820	22,700	18,320	13,630	9,363	12,290	19,060	17,120	15,200	10,540	10,010	12,120
2008	22,480	26,310	13,700	10,190	8,788	11,310	12,520	10,820	10,330	7,767	9,740	8,873
2009	9,143	20,470	22,620	13,600	16,370	11,950	18,620	15,090	11,450	9,781	9,008	10,610
2010	26,810	28,990	19,750	18,830	17,280	20,940	17,420	16,710	16,580	12,010	13,040	45,700
Average	35,000	38,300	37,300	26,500	21,600	18,200	17,700	16,700	16,000	12,400	15,900	26,200

Appendix B.7

West Sacramento / Southport EIP: Task Order 2: Historic and Current Preliminary Geomorphic Assessment—cbec inc., eco engineering, September 12, 2011

cbec eco engineering

Hydrology | Hydraulics | Geomorphology | Design | Field Services



West Sacramento / Southport EIP: Task Order 2: Historic and Current Preliminary Geomorphic Assessment

WSAFCA
West Sacramento Area Flood Control Agency

West Sacramento Flood Control Agency and HDR Engineering

Final Report

September 12, 2011

Project Number 11-1003

Historic and Current Preliminary Geomorphic Assessment
Southport EIP
Task Order 2

Prepared for
West Sacramento Flood Control Agency
And
HDR Engineering

Final Report
Prepared by
cbec, inc.

{ŜLJűŜƳōŜǊ 2011

cbec Project #: 11-1003

This report is intended solely for the use and benefit of the West Sacramento Flood Control Agency and HDR Engineering. No other person or entity shall be entitled to rely on the details contained herein without the express written consent of cbec, inc., eco engineering, 1255 Starboard Drive, Suite B, West Sacramento, CA 95691.

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1 INTRODUCTION

1.1 BACKGROUND

The West Sacramento Area Flood Control Agency (WSAFCA) is in the final phase for Southport Early Implementation Project (EIP). The EIP will identify improvements, or combinations of improvements, to be used to attain the level of flood protection desired for West Sacramento Flood Control Agency (WSAFCA). Four combined measure alternatives (CMAs) are currently under development by HDR Engineering (HDR), one of which will be forwarded as the preferred alternative. The CMAs include options for setting back levees from their current alignment parallel to the right bank (west side) of the Sacramento River, to strengthen the existing levee in-place, and to construct a new levee parallel to the existing alignment. Implementation of setback levees has the potential to change the existing hydraulics, and hence geomorphology, of the Sacramento River and floodplain, both through the project reach and regionally. Implementation of any of the CMAs will result in impacts to regulated land cover types and suitable habitat for special-status species.

1.2 STUDY OBJECTIVES

The purpose of this technical memorandum (TM) is to document the findings of various studies undertaken by cbec, inc., eco engineering (cbec) in support of the final phase of the EIP:

1. To investigate historic planform changes and geomorphic influences on the river over recent history.
2. To describe the current geomorphology of the Sacramento River through the project reach.
3. To investigate the potential geomorphic impacts to the Sacramento River, both region-wide, and locally, at a preliminary level, as a result of potential implementation of various CMAs.
4. To provide recommendations for additional geomorphic studies to be undertaken in support of regulatory processes and design of the project.

2 HISTORIC PLANFORM CHANGES AND GEOMORPHOLOGY

This section provides an historic perspective on how the land use changes, and evolution of the Sacramento River has affected the floodplain and geomorphic processes within the river channel. The perspectives described here are based on earlier studies (Kleinfelder, 2007; William Lettis Associates (WLA), 2009) and research based on historic maps dating back to the earliest available maps in 1850.

The present-day Sacramento River system has been shaped by thousands of years of complex river processes. These processes include channel migration, erosion and flood-stage deposition. During most of Holocene time (since the last ice age, generally defined as the last 11,000 years), sediments from the Sierra Nevada and Klamath Mountains were carried by the Sacramento River and deposited into the Great Valley. Natural levees were built up along the river banks that frequently overflowed during flood stages, depositing sediments into low-lying basins and wide floodplains. The natural river migrated through a wide active area comprised of ponds, abandoned channels, meander cutoffs, oxbow lakes and dendritic channels.

Hydraulic mining in the Sierra foothills during the late 1800’s resulted in high volumes of sediment discharged to the Sacramento and American Rivers. During this time, attempts were made to control flooding and reclaim low basin lands for agriculture by levee construction. Most of the early attempts at flood control were unsuccessful and resulted in multiple breaks within the levee system.

Mapping by Helly & Harwood (1985) shows a variety of alluvial deposits placed by the river within meandering channels. Within the project limits, some of these channels have been eroded/incised, backfilled and overlain by younger deposits. A review of historic air photos from 1932-2007 by Kleinfelder (2007) identified numerous drainage features and depressions that may be remnants of abandoned river channels and other drainage features.

Areas of historic levee breaks along old natural levee are identified by WLA (2009) as “crevasse splays”, characterized by coarse sediments deposited in a fan-shaped or dendritic pattern away from the river. WLA also mapped substantial areas of “overbank deposits” consisting of sand, silt and clay under and adjacent to the existing levees along much of the project alignment. These soils were deposited during high-water events as water overtopped the old natural levee. Relatively deep (greater than 100 feet at some locations) deposits of sand and gravel are located under and land-side of the existing levee along most of the project alignment. These deposits are identified as “meander scrolls” in the WLA mapping and are remnants of riverbanks and natural levees created as the Sacramento River migrated southeast.

The WLA geomorphology mapping of the West Sacramento Area is presented by Figure 3-1.

Historic maps and aerial images of the project area were also collected and studied. Maps from 1850, 1880, 1895, 1908, and 1916 were obtained from various sources as referenced in Figure 2-1 to Figure 2-5. Aerial images from 1937, 1964, and 1984 were obtained from various sources as referenced in Figure 2-6 to Figure 2-8. The map from 1850, and the aerial images from 1934 were compared with the current river alignment to understand bank line migration through the past 160 years, as shown by

Figure 2-9. Historic research on levee development and failure was also undertaken to gain a full understanding of the geomorphic changes that have occurred in the project region. Historic channel and floodplain geomorphology interpretation prior to the establishment of records, based on geologic investigations, is covered in greater detail in Section 3.1.

The following sections describe the results of this analysis.

2.1 HISTORIC INFLUENCES

- The 1850 Ranchero maps (also known as land grant maps or land case maps) were originally developed as part of the private land claims adjudicated by the U.S. District Courts of California (Northern and Southern Districts) and the U.S Circuit Court (9th Circuit) from ca. 1850 to 1860. Since their intent was to map private land and ownership, not geomorphic features or landmarks, these maps provide anecdotal evidence only. They do, however, approximately portray the land use and habitat prior to any major anthropogenic influences. The area that the City of West Sacramento currently occupies was essentially a tidal backwater area, at the fringe of the Sacramento and San Joaquin River Delta (Delta) consisting of predominantly tule marsh in the 1850, as shown by Figure 2-1. This figure seems to indicate that the alignment of the Sacramento River changed dramatically between 1850 and 1880. However, it is unclear the level of accuracy to which the 1850 maps were produced and therefore it is not possible to assume that the river was actually realigned during this period. From 1864 to 1868, the Lower American River was rechannelized, in an effort to create higher velocity flows that might scour out mining debris, Sacramento officials straightened the last two miles of the Lower American River above the confluence with the Sacramento River. When the project was completed in 1868, the Lower American River joined the Sacramento River about a mile upstream of its original location.
- The 1880 map shows the river approximately in a similar alignment to today, as shown by Figure 2-2. The project area was divided into 32 properties and bought by different landowners. Some of the larger landowners are C.H. Crum (acreage unknown), D. McGoeman (220 acres), T.A. Snider (325 acres), and H. Hyster (217 acres). No levees had been built at this time, according to this map. The first comprehensive flood control plan in this area was in response to the 1878 flood. State Engineer William Hammond Hall developed an integrated, comprehensive flood control plan for the Sacramento Valley. The plan subsequently came to include a system of levees, weirs and bypass channels to protect existing population centers. This plan was authorized and federally funded by congress in 1917. This map also shows the development of the Tule Canal, which runs from Lake Washington, north of the project reach, to Big Lake, south of the project reach.
- The 1895 map, shown by Figure 2-3, shows levee installations on both banks of the Sacramento River for the project reach. This is the first instance of levee construction. This map also shows high and low water marks, presumably in the Mean Sea Level datum (2.53 feet below NAVD 88). It is interesting to note the high water marks are between 28.3 and 26.0 feet NAVD 88.
- The 1908 map, shown by Figure 2-4, provides early details of the project area. It shows the project reach with measured cross sections, marshlands, regional lakes, sand bars, levees, land ownership, buildings, channel meander areas, water surface elevations, topographic lines, land

uses, and historic flood plains. The planform of the river had not significantly changed from the 1895 map, but this is the first map were depositional and erosional patters can be seen. Levees are shown on this map and were built on a parcel-by-parcel basis to protect land from flooding by the Sacramento River.

- The 1916 map, shown by Figure 2-5, is the earliest US Geological Survey (USGS) map of the area. It shows one lake on the west side of the Sacramento River near Glide Landing, in the southern most section of the project area. It is interesting to note that Bees Lake is not shown on this map, nor older maps.
- Aerial images from 1937, shown by Figure 2-6, indicate land use on the floodplain and depositional areas in the channel at the time. These aerials show the first indication that the land through the project reach was primarily used for row crops, and irrigation from the river was prominent. Extensive sand bars appear to be present at this time, likely natural channel features exacerbated by legacy sediment deposits as a result of the hydraulic mining era of the late 1800s. This 1937 photograph pre-dates flow gauging on the Sacramento River, however it appears with comparison to later photographs, that the presence of channel sand bars diminishes over time.
- Aerial images from 1964 (flow of 11,900 cfs) and 1984 (flow of 38,900 cfs), shown by Figure 2-7 and Figure 2-8, respectively, show the gradually increasing urbanization in the area, particularly the east side (City of Sacramento) of the Sacramento River. The project reach was still essentially pastureland and row crop agriculture, with progressively increasing signs of urbanization.

2.2 HISTORIC SEDIMENT REGIME, EROSION AND MIGRATION

Since levees of some form have been in place since at least 1895, augmented by rip-rap protection in recent history, there has been very little river migration since that period. As mentioned previously, differences in the perceived river alignment between the 1850 and 1880 maps are clear, but the accuracy of the 1850 maps should be taken with caution. Within the main channel of the Sacramento River however, which has been constrained by levees, agriculture, and urbanization since the early 1900s, the alignment of the river has not changed substantially in over 100 years, as shown by Figure 2-9. However, erosion and deposition patterns have changed over the last 100 years, which are shown by Figure 2-2 to Figure 2-8 and summarized by Figure 2-9. Typically, erosion occurs on the outer banks of meanders or bends where the velocity of the flow is the highest. These areas have been the most problematic for levee maintaining authorities over the last 100 years. Depositional gravel or sand bar features typically form on the inner banks for meanders where the velocity of flow is the lowest, and these deposits can be seen in the earlier maps dating back to 1908. Observations made through this assessment appear to match comparable observations made in a recent unpublished US Army Corps of Engineers study of the region (Northwest Hydraulic Consultants, personal communication).

The morphology of the Sacramento River through the project reach has been largely affected by the sediment regime and budget of the watershed over the last approximately 130 years. As a result of hydraulic gold mining in the late 1800s, vast amounts of sediment were transported from Foothill rivers and streams into tributaries and hence into the Sacramento River. Lower energy reaches of the

Sacramento River, such as through the project reach, would have had vast amounts of these sediments deposited on the bed of the river. With the cessation of hydraulic mining in the early 1900s, and the construction of dams (sediment, flood and water supply) through the middle of the 20^{th} century, the supply of sediment to the system was dramatically reduced. As a result a period of rapid erosion occurred causing incision of the bed and widening of the banks of the Sacramento River. Construction of levees through the mid- to late- 20^{th} century, reduced the amount of river widening, but exacerbated the rate of bed incision. While bed incision has likely reduced in the last 20 to 30 years, bank erosion is an ongoing issue and is a major contributor to finer sediments transported through the Sacramento River. In addition, agricultural runoff erosion also contributes to fine sediment supply.

Closer analysis of the historic sediment regime through the project reach, including morphologic changes, was undertaken through a historical analysis of channel slope through the reach. Figure 2-10 shows a longitudinal comparison of the channel bed between 1908 and the current day. The longitudinal profile from 1908 was digitized from the 1908 map. The longitudinal profile from 2011 was surveyed in the spring of 2011 by cbec as described in Appendix B. Slopes of 0.008% and 0.006% corresponding to 1908 and 2011, respectively, were calculated, based on the slope of the water surface profile. The water surface profile was used since the bed elevations obtained were too variable to allow a reasonable comparison. These values are consistent with the modeled results from MBK's 1D model and show no significant change in slope between 1908 and 2011. This to be expected since the planform of the reach has not substantially changed over this time and any degradation/aggradation has occurred uniformly across the reach.

Finally, the hydrologic changes in the Sacramento River watershed have also had an impact on the morphology of the river through the project reach. The regional hydrology has substantially changed since dramatic flow regulation occurred on the Sacramento River, with construction of Shasta Dam completed in 1945, and other large tributary dams such as Oroville Dam on the Feather River, completed in 1968, and Folsom Dam on the American River, completed in 1955. In combination with urbanization in the Sacramento Region and the resulting hydromodification, and the changes in the sediment regime described previously, have resulted in the morphologic conditions observed today. Further discussion of these morphologic changes is provided in Section 3.4.

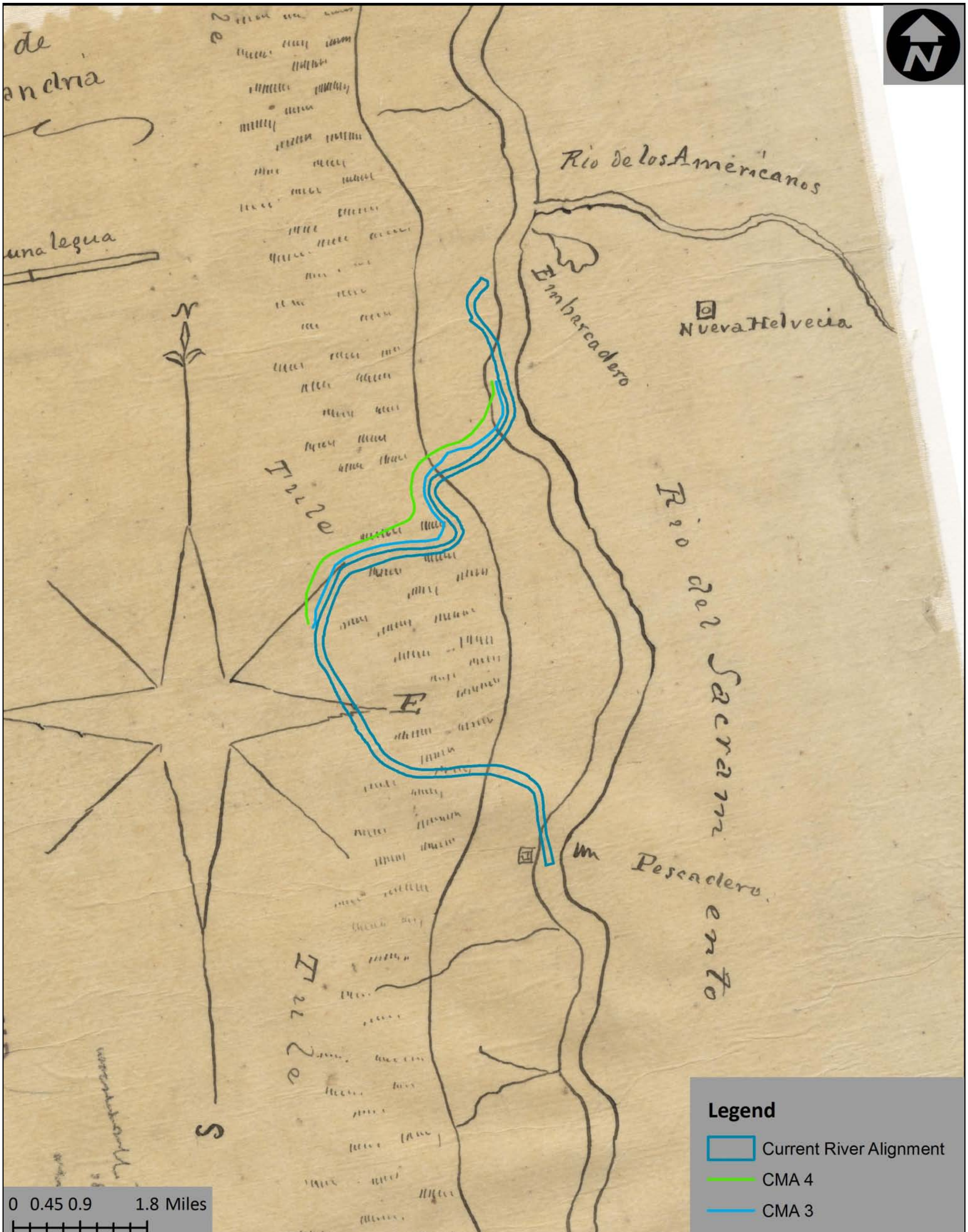
2.3 HISTORIC FLOODPLAIN GEOMORPHIC CHANGES

The natural levees of the Sacramento River were estimated to be between 5 and 20ft high prior to constructed levees being built (Tompson, 1960). During bankfull flows, water often overtopped these natural levees and entered the floodplain. During extended periods of high floods, the floodplains acted as a persistent marsh. Some of the early aboriginal people, the Wintun, considered this region "the half-drowned region" and did not inhabit these floodplain areas. Approximately 40,000 acres of tules remained in Yolo County in the 1870's during colonization from Spaniards. Due to the frequent inundation of this area, the land was almost exclusively used for livestock. 1878 sparked the creation of the Sacramento River Drainage District, which undertook the task diverting floodwaters into canals and expanding the main channel in order to scour out the debris from hydraulic mining. The main drainage canal in the vicinity of the project reach is the Tule canal seen in Figure 2-2. The District assumed that the poorly drained tule basin would be sacrificed to spring floods in order to save the more valuable land near the river.

In 1907 uncontrolled flows of 525,000 cfs passed through the Yolo basin, creating a commerce barrier between the Sacramento area and San Francisco. Flood events like this helped the regional districts gain funding for studies and help construct levees. Figure 2-4 illustrates levees constructed as the product of this funding.

Lake Washington has undergone few changes since it is first seen on the 1880 map (Figure 2-2). From 1880 to 1937, as shown by Figure 2-2 through Figure 2-6, there are very few changes to the planform shape of the river. As population grew, the lake has been altered to accommodate the changing needs. In 1963 it became directly connected to the Sacramento River by means of the Sacramento Deep Water Ship Canal, which can be seen in Figure 2-7.

Bees Lake is not observed until 1937, as shown by Figure 2-6. Very few changes can be seen in the planform view between its first occurrence and today. Since Bees Lake is only first observed in 1937 it is unlikely to be a relic feature of the floodplain. It may have been exacerbated by anthropogenic disturbance, such as construction of levees, and associated loss of free surface water, or shallow groundwater drainage to the Sacramento River.



Notes: Notes: Map Courtesy of Shields Library at the University of California Davis.

Historic and Current Preliminary Geomorphic Assessment- Southport EIP

1850 Map

Project No. 11-1003 | Created By: LA | **Figure 2-1**

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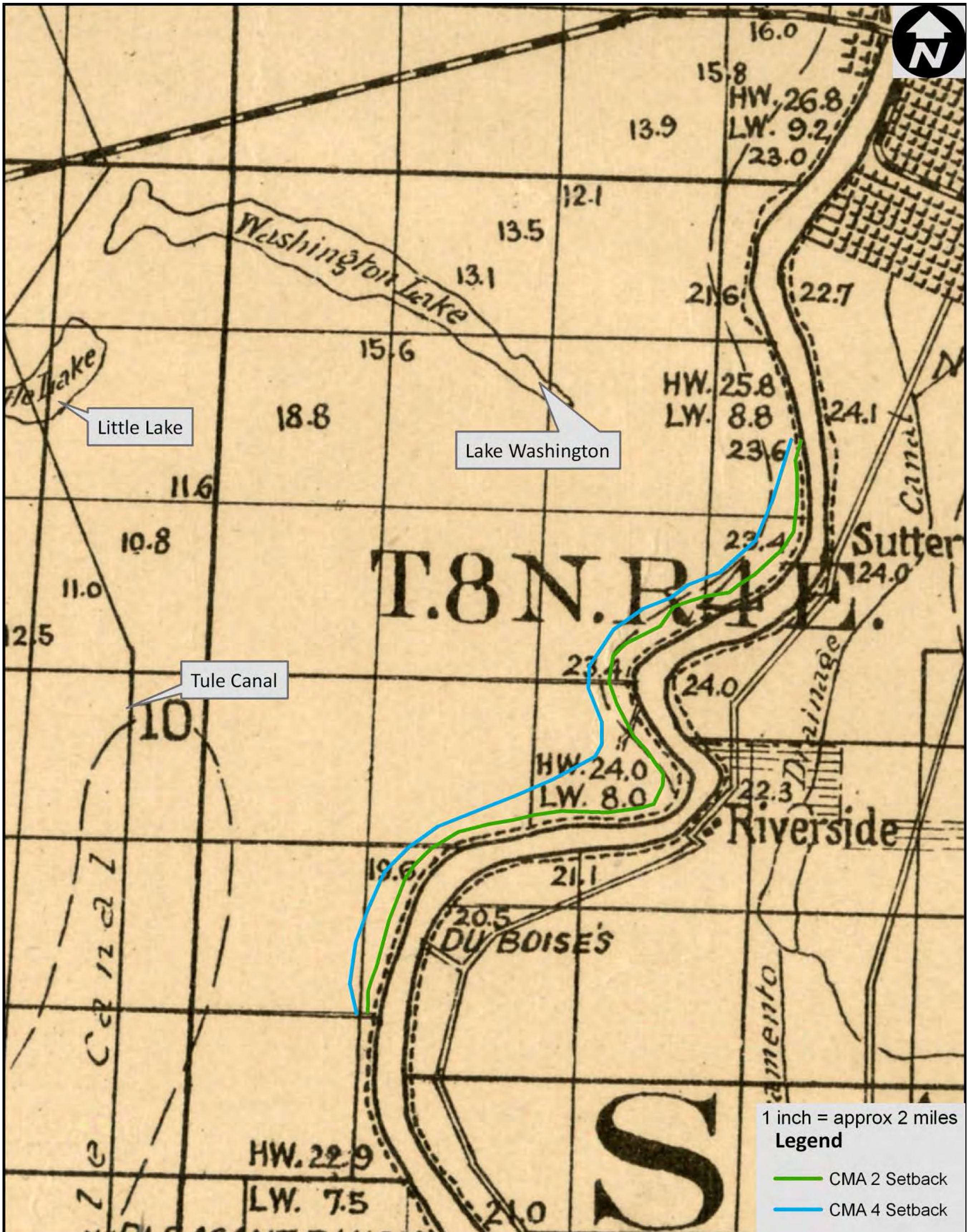
5/13/2011

Notes: Map courtesy of Shields Library at the University of California, Davis,	cbec eco engineering	*Historic and Current Preliminary Geomorphic Assessment Southport EIP* **1880 Map**		
		Project No. 11-1003	Created By: LA	**Figure 2-2**



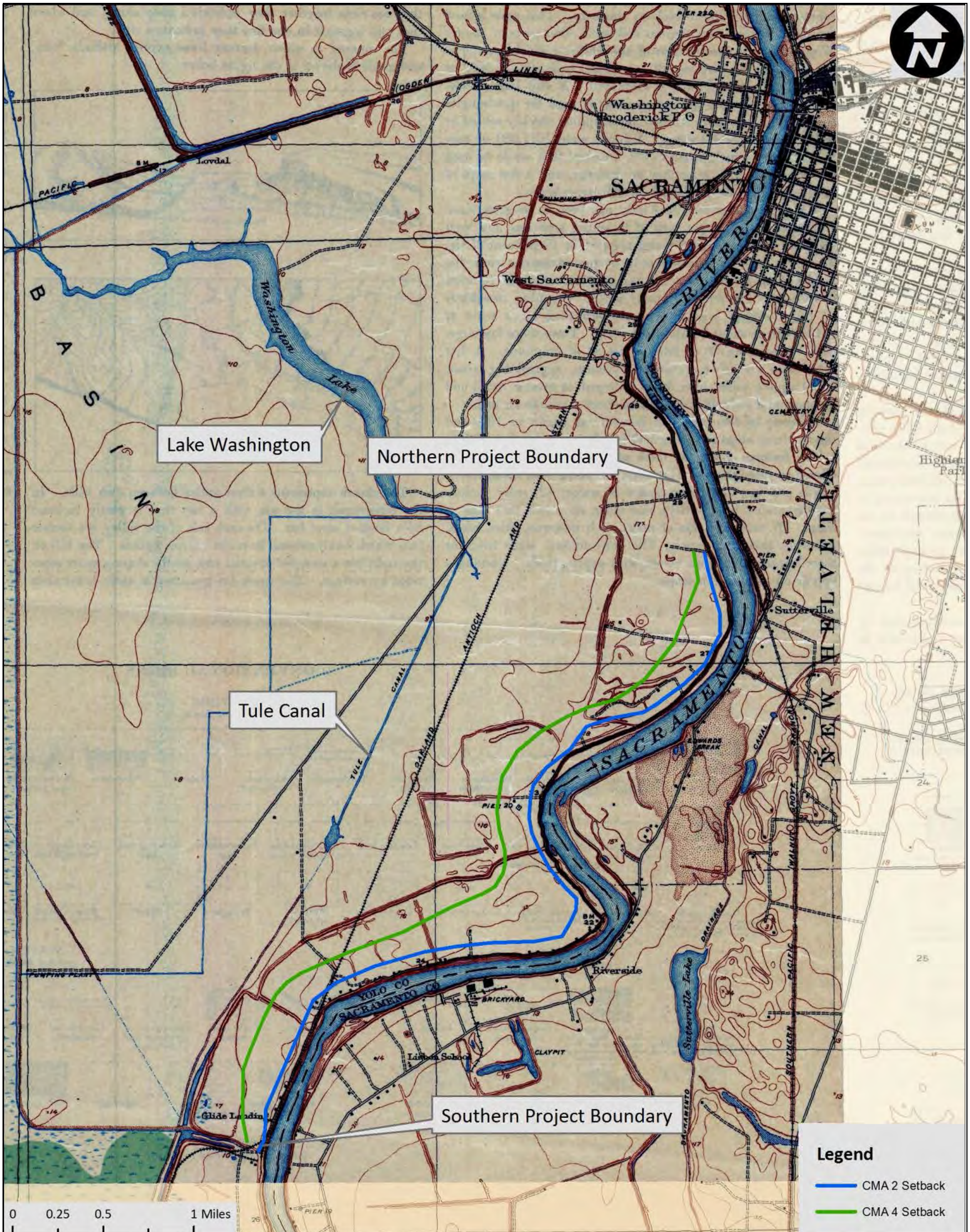
Notes: Map courtesy of Shields Library at the University of California, Davis,	cbec eco engineering	*Historic and Current Preliminary Geomorphic Assessment Southport EIP* **1895 Map**		
		Project No. 11-1003	Created By: LA	**Figure 2-3**

Notes: Map courtesy of Shields Library at the University of California, Davis. CMAs for reference only.

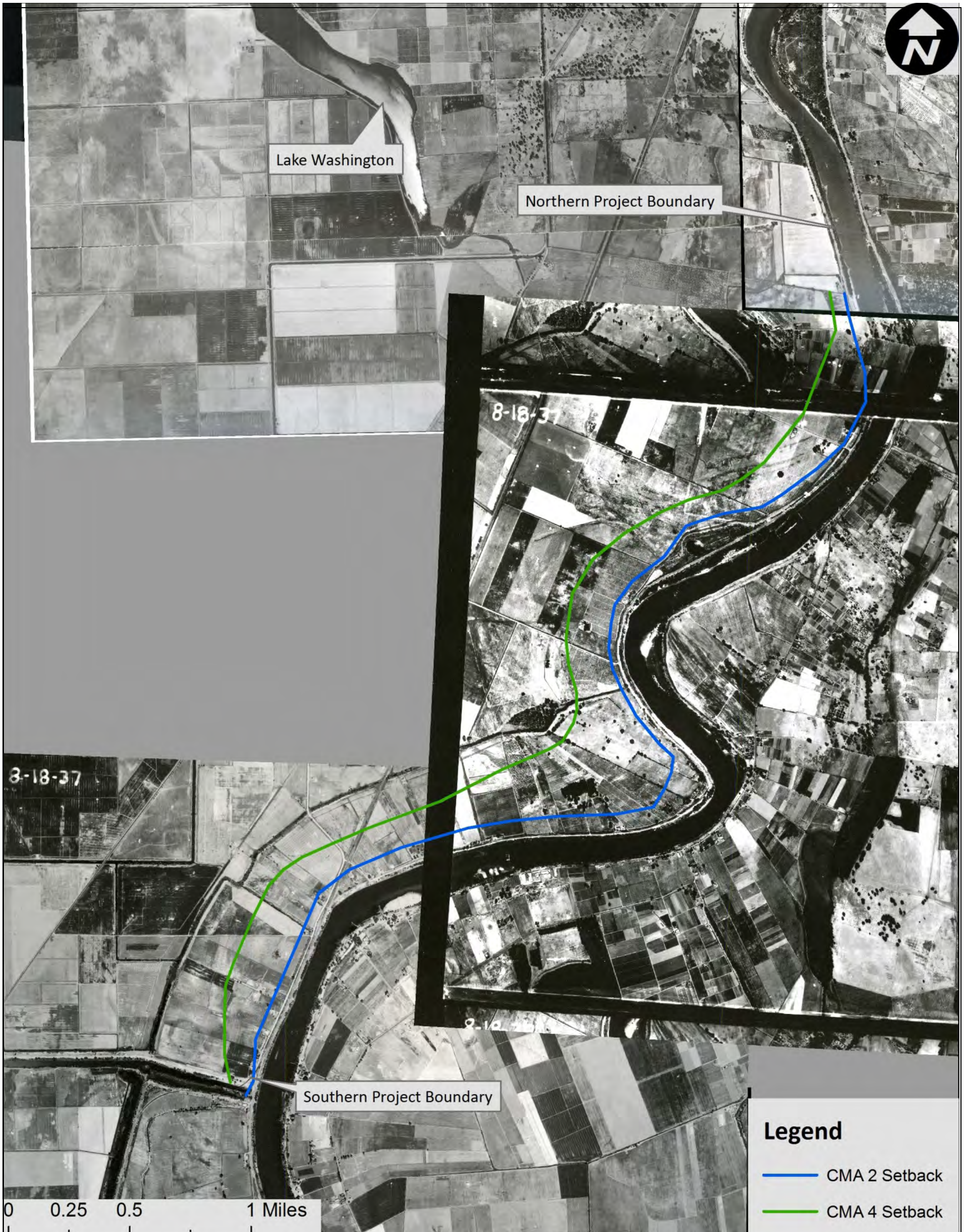
Historic and Current Preliminary Geomorphic Assessment Southport EIP

1908 Map

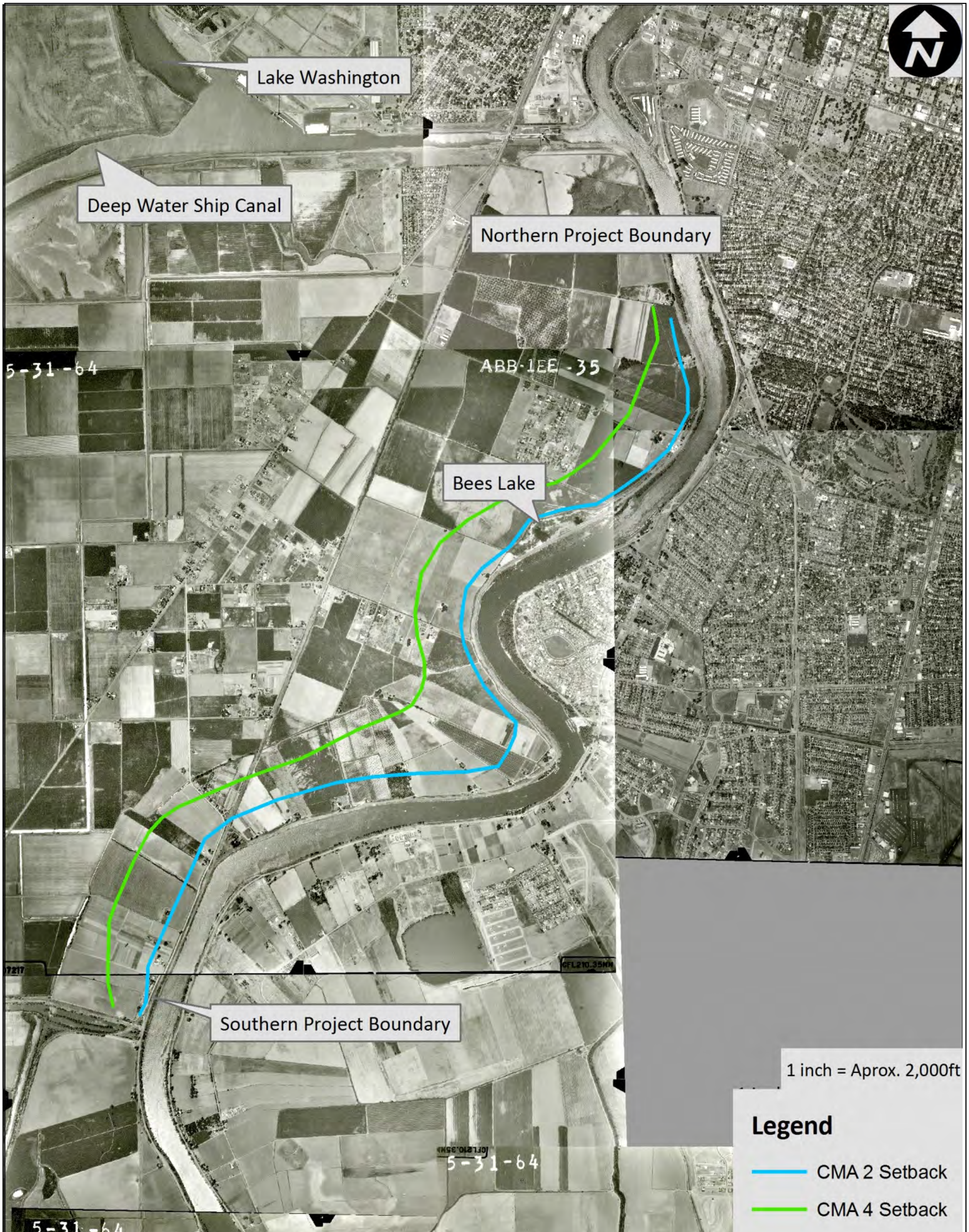
Project No. 11-1003	Created By: LA	**Figure 2-4**



Notes: Map courtesy of USGS.	cbec eco engineering	*Historic and Current Preliminary Geomorphic Assessment Southport EIP* **1916 Map**		
		Project No. 11-1003	Created By: LA	**Figure 2-5**



Notes: Aerial image courtesy of the University of California, Davis.	cbec eco engineering	*Historic and Current Preliminary Geomorphic Assessment Southport EIP* **1937 Aerial Image**	
		Project No. 11-1003 / Created By: LA	**Figure 2-6**



Notes: Aerial image courtesy of the University of California, Davis.

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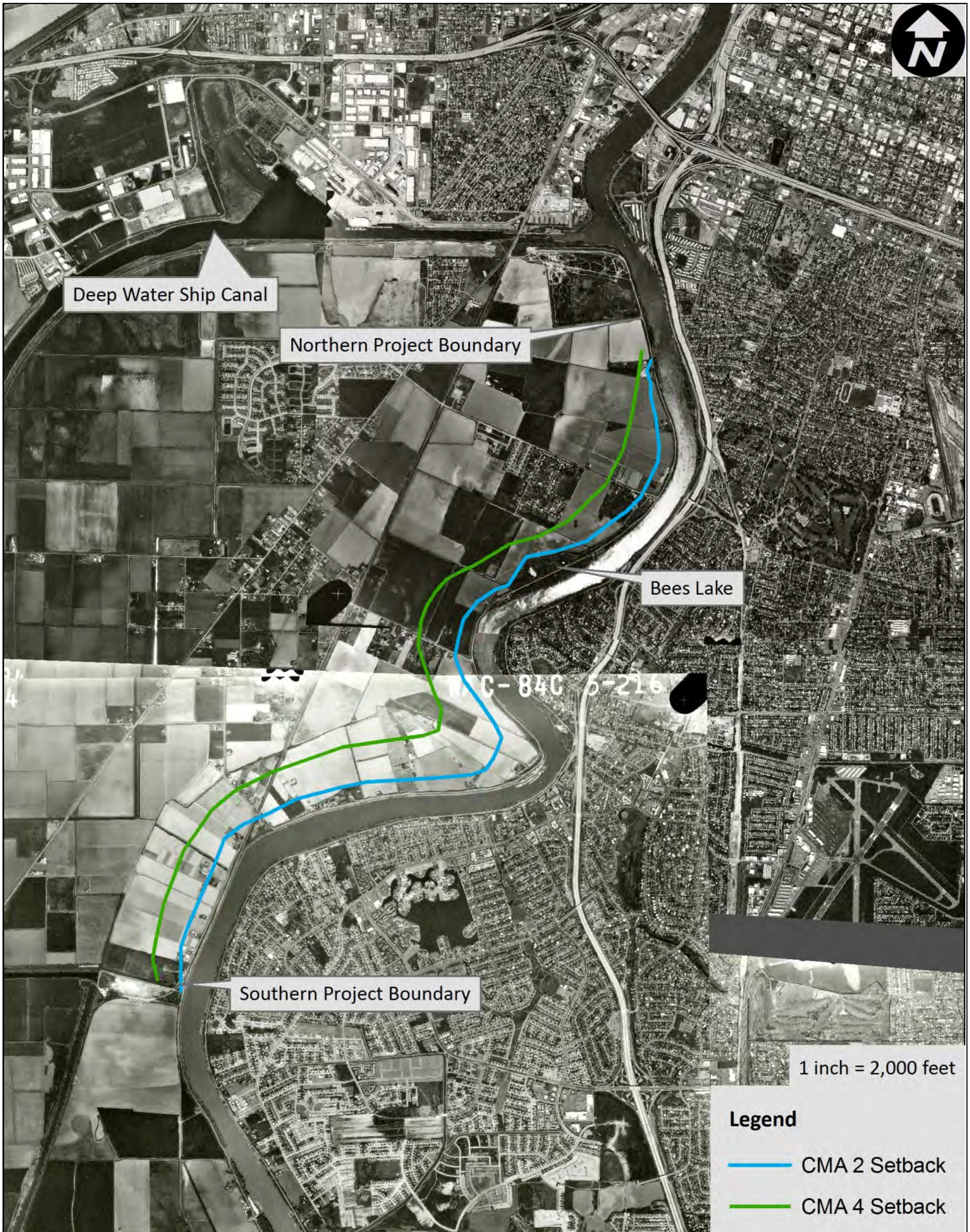
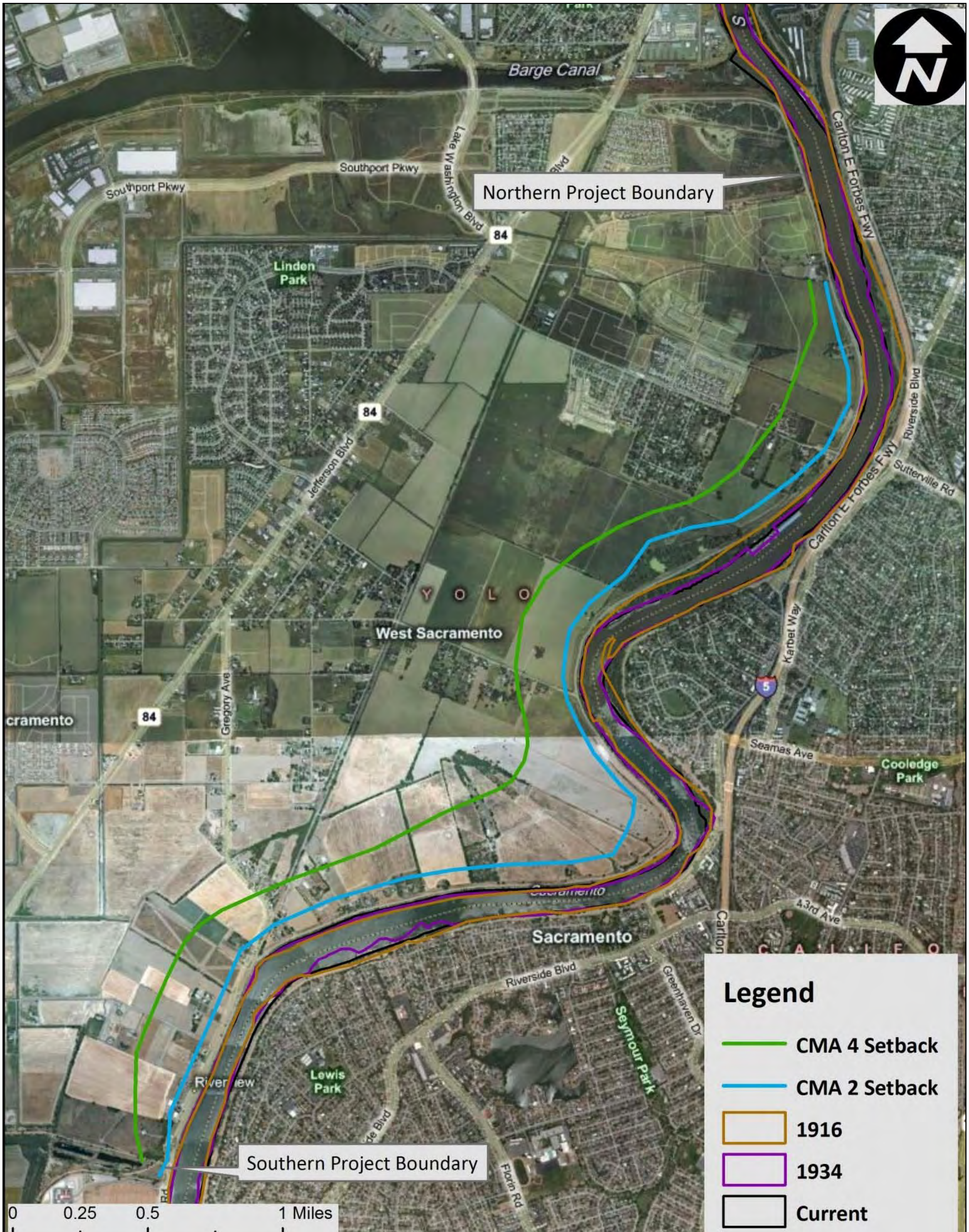
Historic and Current Preliminary Geomorphic Assessment Southport EIP

1964 Aerial Image

Project No. 11-1003	Created By: LA	**Figure 2-7**



Notes: Aerial image courtesy of the University of California, Davis.	cbec eco engineering	*Historic and Current Preliminary Geomorphic Assessment Southport EIP* **1984 Aerial Image**		
		Project No. 11-1003	Created By: LA	**Figure 2-8**



Notes: Orothoimagery courtesy of Bing Maps.

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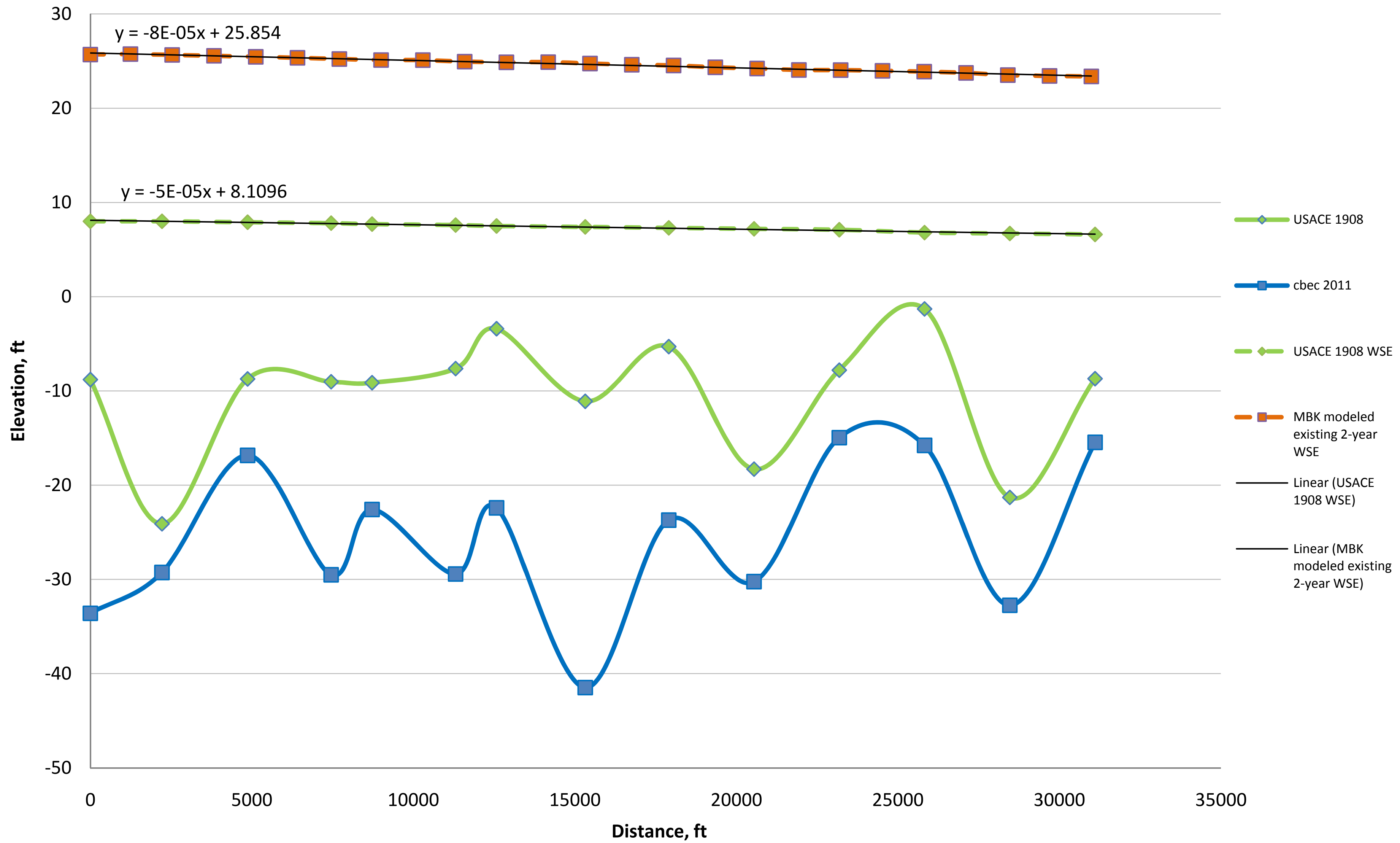
Historic and Current Preliminary Geomorphic Assessment- Southport EIP

Historic Changes in Bank Lines

Project No. 11-1003	Created By: LA	**Figure 2-9**



Notes:	cbec eco engineering	*Historic and Current Preliminary Geomorphic Assessment Southport EIP* **Longitudinal Profile**		
		Project No. 11-1003	Created By: LA	**Figure 2-10**

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9/12/2011

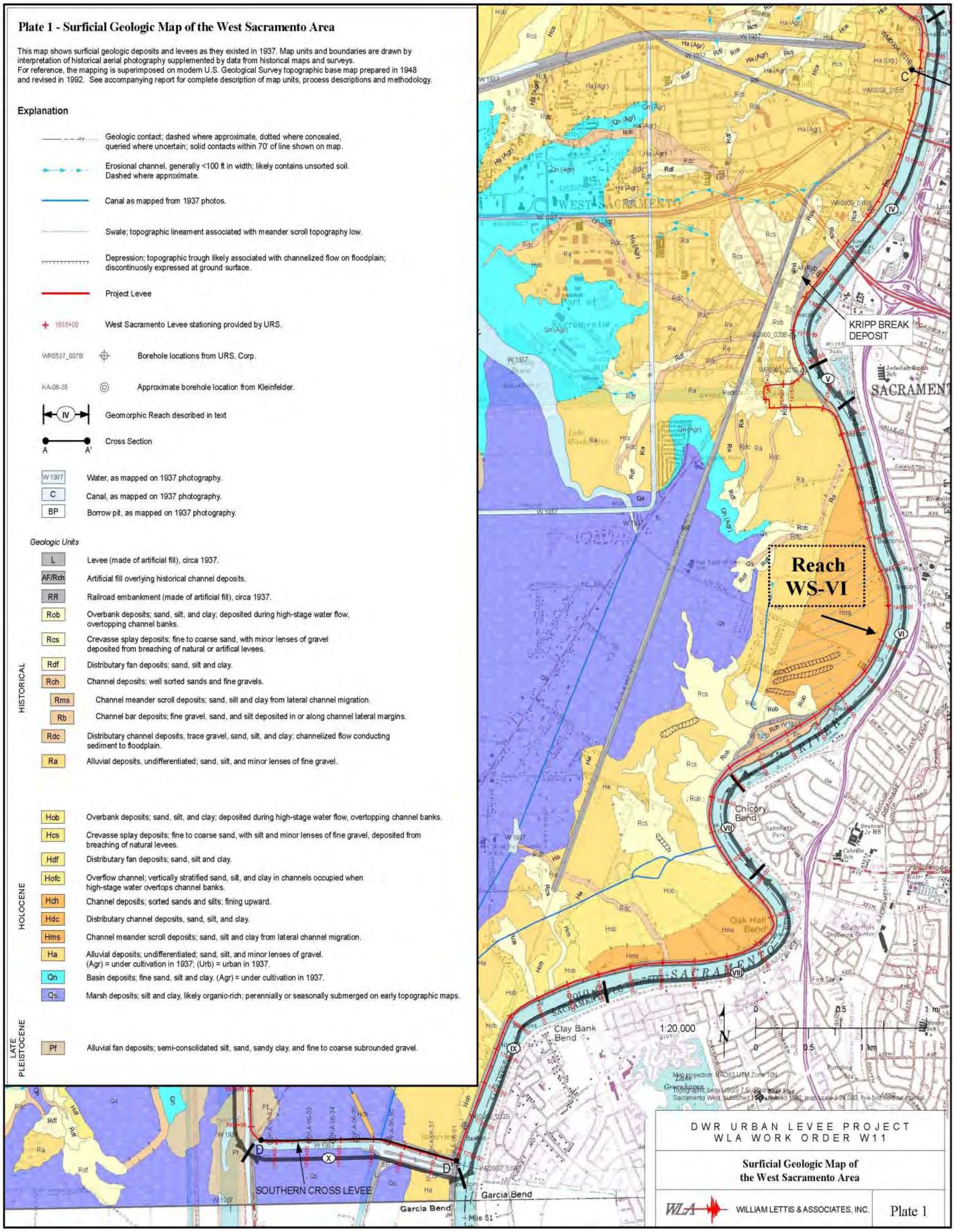
3 CURRENT GEOMORPHOLOGY

This section describes the current geomorphology of the Sacramento River through the project reach, and the characteristics of the floodplain.

3.1 GEOLOGY AND SOILS

The northern segments of the project reach have been largely covered with artificial fill associated with urban development in the City of West Sacramento. The 1916 USGS topographic map, as shown by Figure 2-5, as well as geologic research, suggest extensive fluvial deposits were present in this segment, that were covered by development by 1937 (Figure 2-6). Holocene Alluvium beneath the levee consists of loose, brown, silty fine sand and sandy clay. Beneath this formation is about 50 feet of poorly-graded compact brown fine to coarse sand, with some occasional gravel and silt. Sand found about 20 feet beneath the base of the levee may be representative of the last Pleistocene upper Modesto Formation deposits. The sand layer is uniform from east to west on both sides of the levee. This lies on top of about 20 feet of gravel with silt and sand, which lies on hard clay and silt.

The river has displayed historical overbank flooding and deposition along with levee performance issues as well across from Miller Park, where on the outside of a river bend, it is not rare to see deposition and flooding. In March 1907, a flood event caused a break in the levee on Kripp farm, which deposited many acres of sand, about 4 to 5 feet thick. Further South, in reach WS-VI (as shown by Figure 3-1,WLA Geologic map) most deposits are made up of Holocene meander scroll, historical channel, and historical alluvial deposits. Meander scrolls are a series of ridges and troughs that are formed from the stacking of point bar deposits along the inner bank of a stream meander as the channel migrates laterally and down-valley towards the outer bank (Saucier, 1994). They are recognized as being prone to levee under seepage, being unconsolidated and very loose, horizontally stratified with vertically interbedded sand, silt and clay. Through the reach encompassing Chicory Bend, the sediment beneath the levee consists of overbank deposits that are made up of beds of silt, clay, sandy silt, and sand. Below this section is a 30-foot thick composite of granular fluvial sediment deposits, and underneath that is hard clay. Further south, towards the Pocket Area and Garcia Bend, there are additional Holocene meander scroll deposits. This reach consists of loose silty sand, and sandy or clay-like silt directly beneath the levee. The reach near Garcia Bend has similar features to the reach near Chicory bend, but the meander bend is not as sharp. The deposits beneath the levee are fine-grained, soft silt, sandy silty clay, and clay for about 20 to 40 feet in depth beneath the levee. The proportion of sand is likely greatest in the upper 10 to 15 feet beneath the levee and likely evidence of the latest Holocene overbank sedimentation and natural levee construction. At greater depths the sediment is made up of coarser-grained sands that overlie gravel. In summary, there is a general upward trend of sediment becoming finer in the subsurface stratification, going from gravel to sand to silt to clay throughout about 100 vertical feet beneath the levee.



Notes: Map courtesy of WLK Consulting.

cbec eco engineering

Historic and Preliminary Geomorphic Assessment- Southport EIP

Surficial Geologic Map

Project No. 11-1003	Created By: LA	**Figure 3-1**

3.2 FLOODPLAIN HYDRAULIC CONNECTIVITY

An initial estimate of the connectivity of the Sacramento River to its floodplain through the project reach was undertaken to inform separate, but associated, ecosystem planning activities, supported by cbec. Potential setback of the levee, as proposed through CMA 2 and 4, will expose floodplain to frequent inundation to varying degrees. The potential for, and frequency of, floodplain inundation is an important metric for habitat design since species such as riparian vegetation and aquatic biota require specific floodplain inundation characteristics to thrive. Specifically, the amount of floodplain inundation that could be expected during a 2-year recurrence interval flood was investigated, using modeled results by MBK, and water surface elevation measurements recorded by cbec during the high flow event of December 2010.

3.2.1 Modeled Water Surface Elevations

Modeling results were obtained from MBK who conducted region-wide 1-dimensional modeling of the potential impacts of CMAs 1 through 4. Figure 3-2 shows the water surface elevations of the 2- and 10-year recurrence interval events modeled through the project reach for CMA-2. This figure shows water surface elevations in the 2-year recurrence interval event varying from 26.03 feet (NAVD 88) at the upper extents of the project reach to 23.67 feet at the lower extents of the project reach. The corresponding magnitude of the modeled 2-year event was approximately 83,000 cfs.

3.2.2 Measured Water Surface Elevations

cbec installed two continuously recording water level recorders in November 2010, the locations of which are shown by Figure 3-3. A flood frequency analysis (FFA) was conducted on data downloaded from California Data Exchange Center (CDEC) at the Freeport gauge on the Sacramento River, as shown by Figure Figure 3-4. The Freeport gauge is approximately five miles downstream of the downstream extent of the project reach. The water level recorders through the project reach captured a high flow event in December 2010, that peaked around the 2-year recurrence interval, as shown by Figure 3-5, and based on the FFA conducted at Freeport. This 2-year recurrence interval event corresponded to a flow of approximately 72,300 cfs at the Freeport gauge. Further details of these field measurement can be obtained through an interim field data collection memo produced by cbec under Task Order 1, and included here as Appendix B.

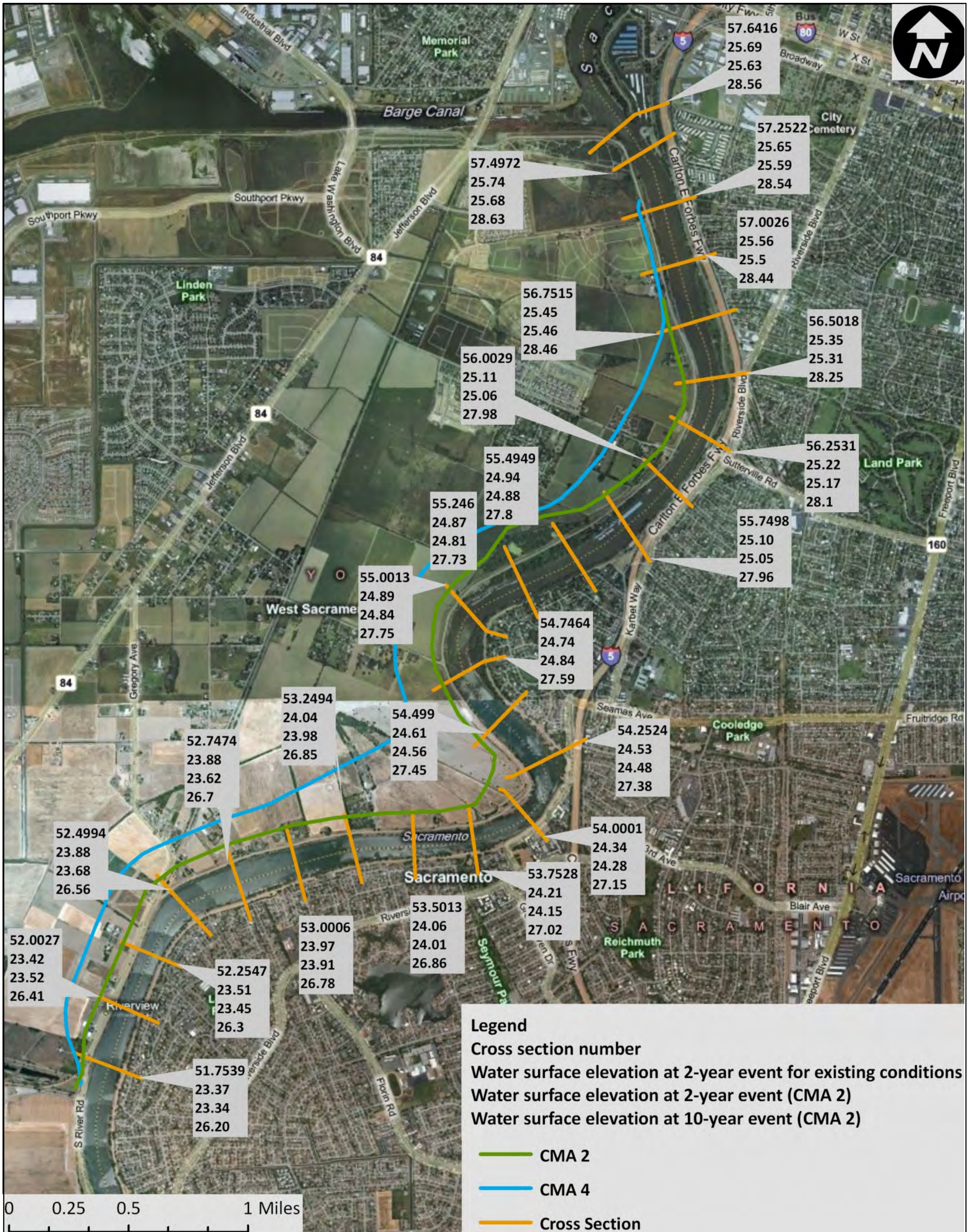
3.2.3 Frequency and Extent of Inundation

The water surface elevation of the 2-year recurrence interval event obtained from modeling and from field data were compared to topographic elevations on the floodplain obtained from mapping provided by HDR. Through visual observation, floodplain depths under CMA 2 and CMA 4 (setback alternatives) could vary from between 2 to 6 feet. During an event of this magnitude practically the whole of the setback area could be inundated. The duration of inundation has not been determined at this time but will be investigated through Task Order 3 activities.

It is interesting to note that the 2-year recurrence interval water surface elevations obtained through modeling and through field measurements varied by up to 1.8 feet. There are several possible reasons for this:

1. The FFA is based on a period of record of 97 years, and inherently the 2-year recurrence interval event may vary from 78,400 cfs to 67,340 cfs as shown by the upper and lower confidence limits, as shown by Figure 3-4. This results in a corresponding uncertainty of measured water surface elevation.
2. The modeled water surface elevations at the 2-year recurrence interval event are based on hydrology developed theoretically through a specific combination of storm centering, which is not necessarily reflected through the FFA of the measured data at Freeport.
3. The corresponding modeled 2-year recurrence interval flow is on average 83,000 cfs through the project reach, compared with a measured 2-year recurrence interval flow of approximately 72,000 cfs measured in December 2010. This directly explains why the modeled water surface elevations are up to 1.8 feet higher than the measured equivalent.
4. The measured water surface elevation was conducted through the current configuration of the levees bounding the Sacramento River. The modeled water surface elevations were obtained through specific levee setback conditions. Hence, differences in water surface elevation should be expected. Further observation of the modeled existing conditions and CMA 2 setback conditions for the 2-year recurrence interval flow shows that the water level may drop by 0.05 to 0.06 feet as a result of the setback (see Figure 3-2).

With consideration of the above factors, it is reasonable to assume at this preliminary stage that floodplain inundation will occur approximately at the 2-year recurrence interval event for CMA 2 and CMA 4, at depths between 2 to 6 feet. A more detailed investigation of floodplain frequency, duration, extent, depth, timing and rate of inundation will be conducted through 2-dimensional modeling under Task Order 3.



Notes: 2- and 10-year water surface elevation modeling done by MBK

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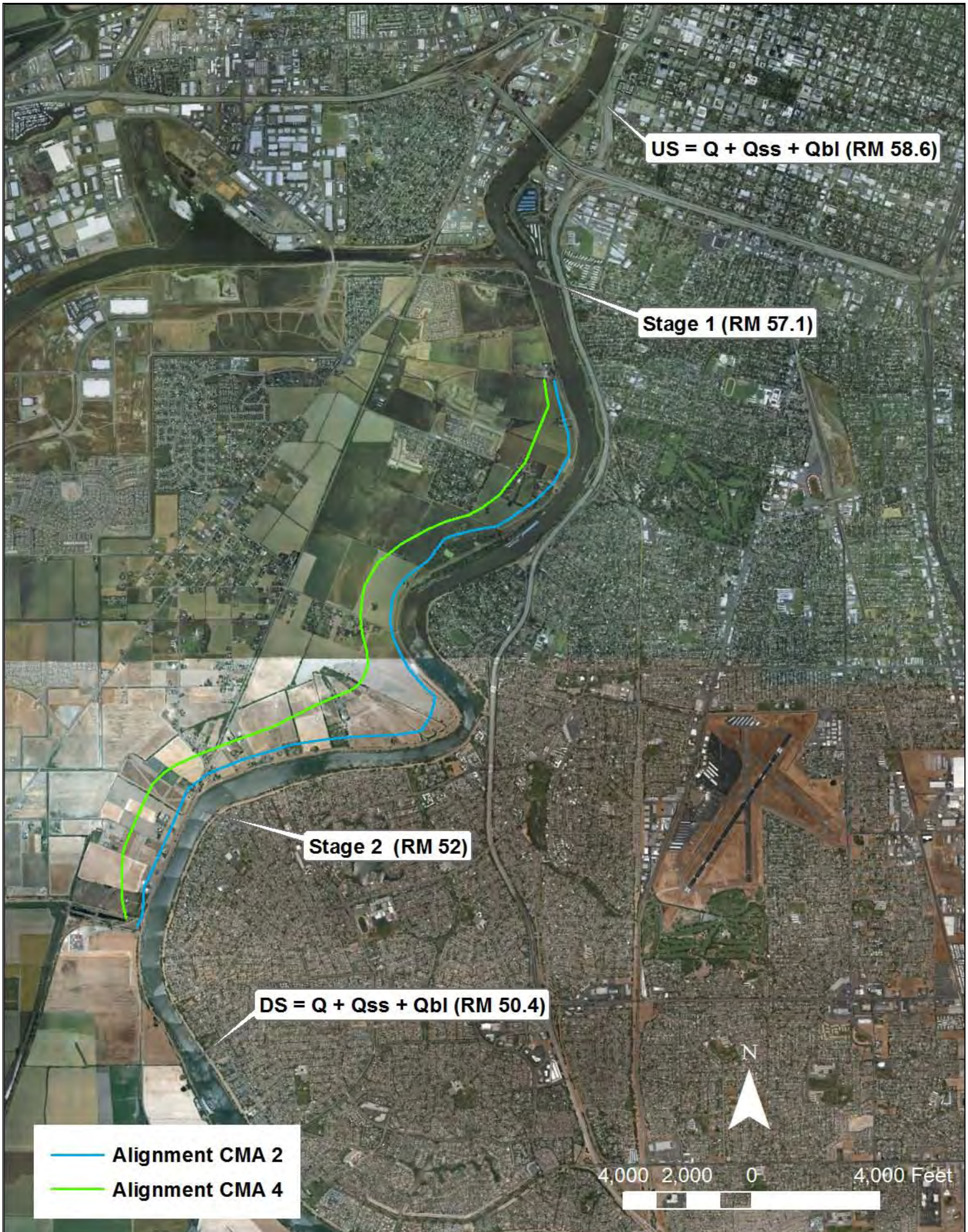
Modeled 2- and 10-Year Water Surface Elevations

Project No. 11-1003	Created By: LA	**Figure 3-2**



Notes: Q = Discharge (cfs) Qss = Suspended Load Qbl = Bedload	cbec eco engineering	*Historic and Current Preliminary Geomorphic Assessment- Southport EIP* **Monitoring Locations**		
		Project No. 11-1003	Created By: JS	**Figure 3-3**

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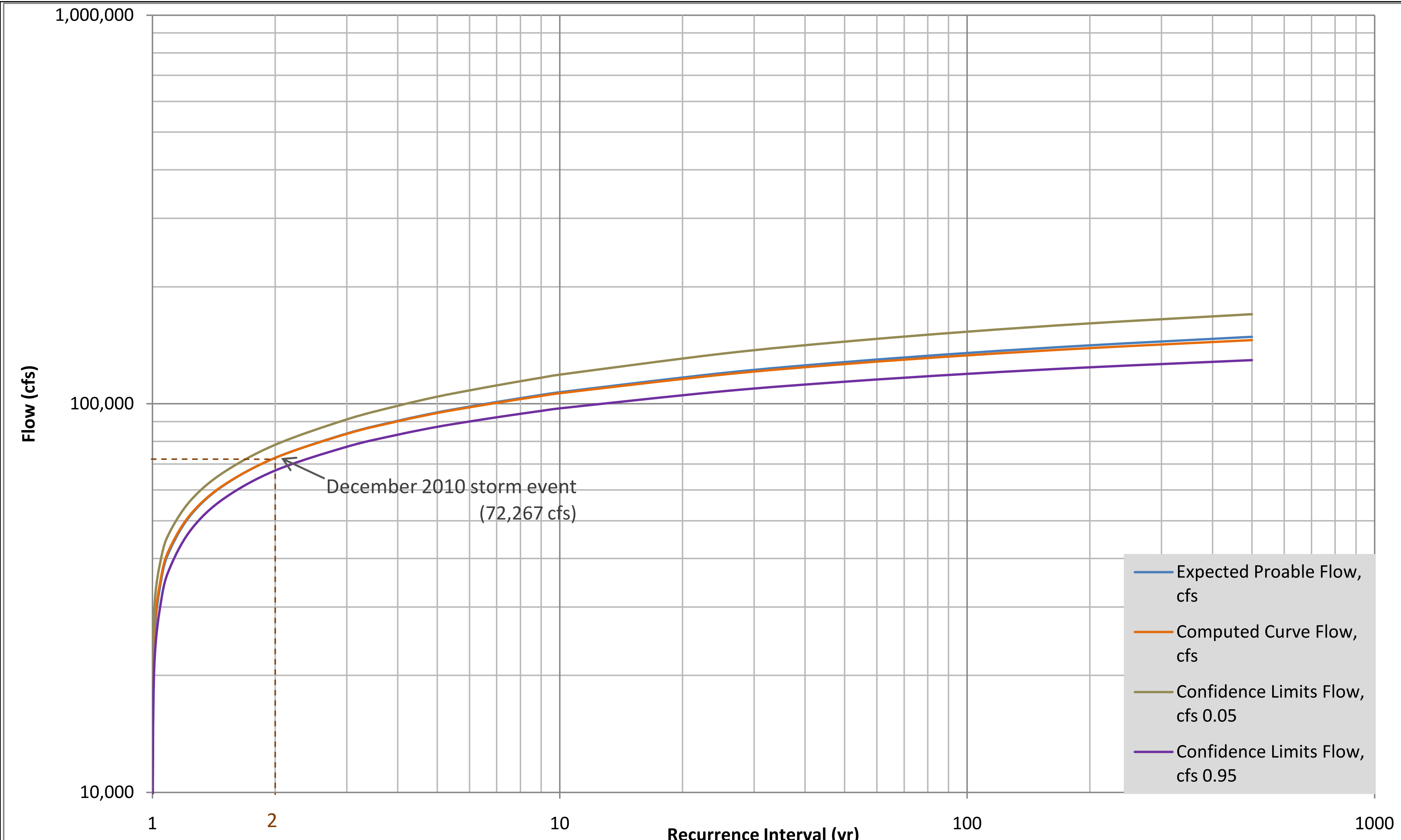

Notes: Flood frequency analysis for a 2-year event yields an expected probable flow of 72,592cfs. Peak discharge data for December storm event (12/21/2010) was 72,267cfs. Datum NAVD 88

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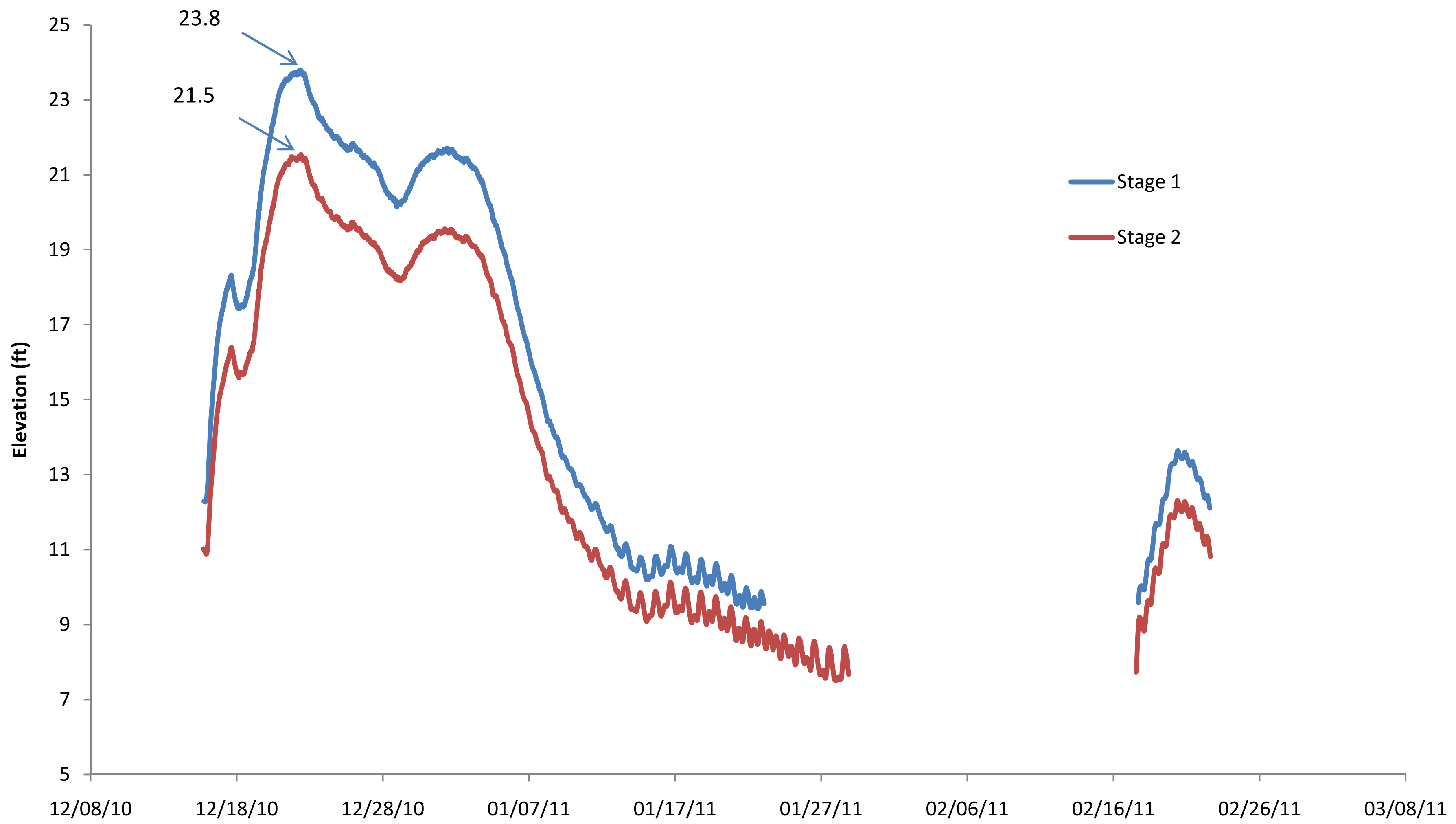
Flood Frequency Analysis of Flows at Freeport Gage

Project No. 11-1003	Created By: LA	**Figure 3-4**



Notes: Elevation references the NAVD 88 vertical datum. Missing data is a result of the water level dropping below the elevation of stage recorder

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Measured State within Project Reach

Project No. 11-1003 | Created By: LA | **Figure 3-5**

3.3 FIELD GEOMORPHIC ASSESSMENT

3.3.1 Methods

cbec staff assessed the existing geomorphic conditions of the project reach on August 10, 2011. Field staff made observations on the current state of the channel banks and levees from one mile upstream to one mile downstream of the proposed setback project. Staff used handheld GPS to map specific geomorphic features including areas of existing erosion and deposition.

3.3.2 Overview

The levees that exist throughout the study reach are in close proximity to the channel banks and the integrity of these banks and levees vary considerably. Downstream of Chicory bend, a majority of the levees and banks are reinforced with rip rap. Upstream of Chicory bend, about half of the levees are protected with rip rap (Figure 3-6). Since 2005, California Department of Water Resources (DWR), Sacramento Area Flood Control Agency (SAFCA), and the United States Army Corps of Engineers (USACE) implemented a number of levee repair and enhancement projects. cbec staff observed six constructed restoration projects consisting of riparian benches through the study reach (Figure 3-6). Two of these sites are on the right bank and ongoing vegetation management at these two sites will likely be affected by the construction of the setback

3.3.3 Bank and Levee Material

cbec staff observed the bank stratigraphy at an exposed cut bank on the right bank upstream of the project reach (Figure 3-6). A photograph of the stratigraphic section is presented in Figure 3-7. Surficial geologic maps (Figure 3-1) indicate that the stratigraphy is mix of overbank and crevasse splay deposits. The 10 ft. of visible stratigraphy at this site consist of predominantly unconsolidated medium to fine sand and silt. These deposits are very likely attributable to hydraulic mining, but determining the precise timing and provenance of these deposits could not be determined during this reconnaissance level effort. Other erosion sites throughout the study reach revealed a similar stratigraphic sequence. The levees throughout the study reach are sourced from dredged sands, local overbank, crevasse splay and relict channel deposits. The levees are generally composed of unconsolidated, relatively fine grained sediments that are porous and prone to seepage and erosion. Where not protected by revetments, placed levee material and bank material has the potential to erode rapidly during high discharge events and more progressively as a result of wind wave and boat wake induced erosion.

3.3.4 Erosional Features

cbec staff observed five areas of bank erosion through the study reach where unprotected channel banks are actively eroding (Figure 3-6). On the right bank immediately upstream of the proposed upstream breach, the levee is unprotected and eroding (Figure 3-6, Location 3). Figure 3-8 depicts areas of erosion along Location 3. Cross-section 3 (Appendix A) indicates that the geometry of the channel has changed very little at this location since 2008. However, because there have been no significant runoff events since the winter of 2006, we cannot confidently define a trend of erosion by evaluating the differences between the 2008 and 2011 survey data.

On the left bank, adjacent to the proposed downstream breach, another small portion of unprotected levee appears to be eroding (Figure 3-6, Location 5). However, cross-section 14 indicates the bed and bank have accreted in the vicinity of this location since 2008. Figure 3-9 depicts the eroding levee across from the proposed downstream breach.

Erosion observed on the left bank, downstream of Chicory Bend (Figure 3-6, Location 4) appears to be eroding material deposited inboard of the levee since its construction; however the bend downstream of location 4 appears to focus a significant amount energy/shear at the toe of the levee. Downstream of this point, the toe of the levee on the left bank is armored with riprap, but upstream of the bend the levee toe is lacking armoring. Cross-section 9 (Appendix A), surveyed just upstream of Location 4, indicates very little change to the bank and bed at this location.

cbec Staff observed two additional areas of levee erosion, just upstream of the project reach, between the entrance to the deep water ship canal and the proposed breach (Figure 3-6, Locations 1 and 2). These areas of erosion occur along unprotected sections of levee adjacent to levee sections protected by riprap.

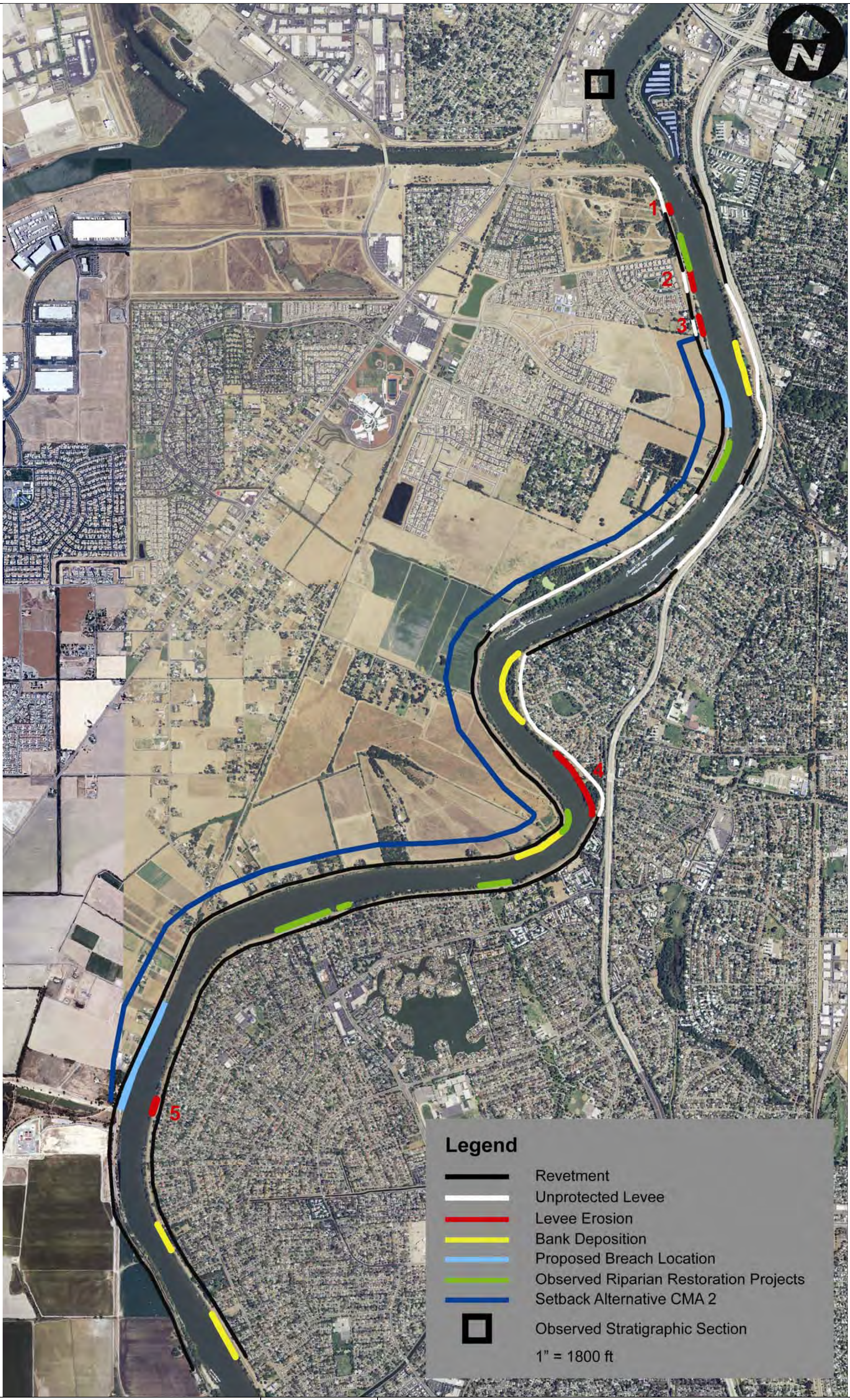
MBK's existing 1D model indicates a minimal increase in shear associated with the proposed setback alternatives. Since erosion exists in the majority of areas that lack armoring, even at locations where erosion typically wouldn't occur (inside of bends), it is hypothesized that the majority of the erosion at these sites are induced by boat wake / wave generated erosion due to the high level of recreational boat traffic in the project reach.

3.3.5 Depositional Features

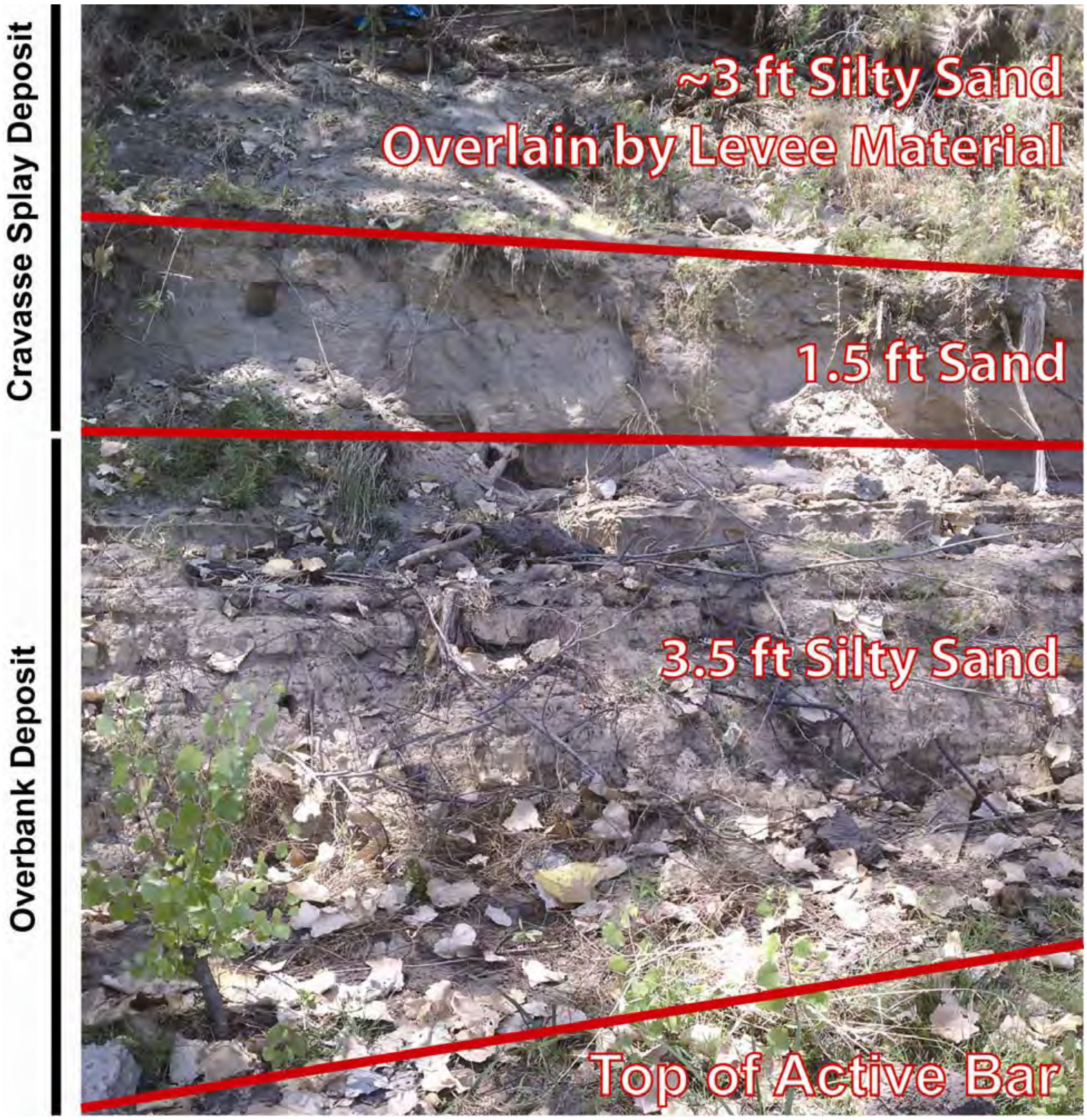
Remnants of natural bar features exist within the project reach on the right bank between the Sacramento Yacht Club and Sherwood Harbor and on the left bank at Chicory Bend. Both of these bars support mature riparian vegetation including willow and cottonwood. Cross-section 6 and 7 (Appendix A) indicate minimal change in bed geometry between the Sacramento Yacht Club and Sherwood Harbor. Cross-section 8 (Appendix A) indicates that there has been erosion of this bar since 2008. Historical surveys and aerial photographs (Appendix A and Section 2-3, respectively) indicated that these bars were less vegetated and likely inundated more frequently. As more dams have been constructed on tributaries upstream of the study reach, large flow events have become more rare and attenuated. cbec Staff observed active deposition of sediment along the banks at other locations (Figure3-6), but deposition is limited to narrow un-vegetated bars at the toe of the levees. cbec does not believe that the proposed levee setback will not significantly affect the location and size of these depositional features; however cbec recommends post construction monitoring of these features.

3.3.6 Potential Morphologic Impacts of Setback

Section 4 presents the modeled (1D) changes in channel shear stress that may result from the proposed setback alternatives. The model indicates overall reduction in channel bed shear in the Sacramento River adjacent to the proposed breaches at all modeled flood levels for both the CMA-2 and CMA-4 alternatives. It is possible that the local reduction in shear may result in an increase deposition on the channel bed and on the banks for typical conditions. However, predicted accretion will likely be minimal and will likely not increase flood stage at these locations. In order to develop a more detailed picture of potential erosion and deposition, both in the existing main channel and within the floodplain created by the proposed setback alternatives, cbec is currently under contract to the develop and run a 2-dimensional hydrodynamic sediment transport model. That modeling effort is currently under contract to develop a 2-dimensional hydrodynamic sediment transport model to characterize existing and project conditions.



Notes:	cbec eco engineering	*Historic and Current Preliminary Geomorphic Assessment – Southport EIP* **Geomorphic Assessment Map**	
		Project No. 11-1003	Created By: ED	**Figure 3-6**



Notes:	cbec eco engineering	*Historic and Current Preliminary Geomorphic Assessment – Southport EIP*		
				Stratigraphic Cross-section
		Project No. 11-1003	Created By: ED	**Figure 3-7**

Notes:	cbec eco engineering	*Historic and Current Preliminary Geomorphic Assessment – Southport EIP* **Levee Erosion Location 3**		
		Project No. 11-1003	Created By: ED	**Figure 3-8**

Notes:	cbec eco engineering	*Historic and Current Preliminary Geomorphic Assessment – Southport EIP* **Levee Erosion Location 5**		
		Project No. 11-1003	Created By: ED	**Figure 3-9**

3.4 CROSS SECTIONAL ANALYSIS

cbec surveyed 14 cross sections bathymetrically through the project reach in April 2011. An Ohmex Sonarmite single beam echosounder, dynamically coupled to a survey grade Trimble RTK GPS was deployed to a North River 21’ jet boat. Bathymetric cross sections obtained through this effort were compared to two other sources of bathymetric data:

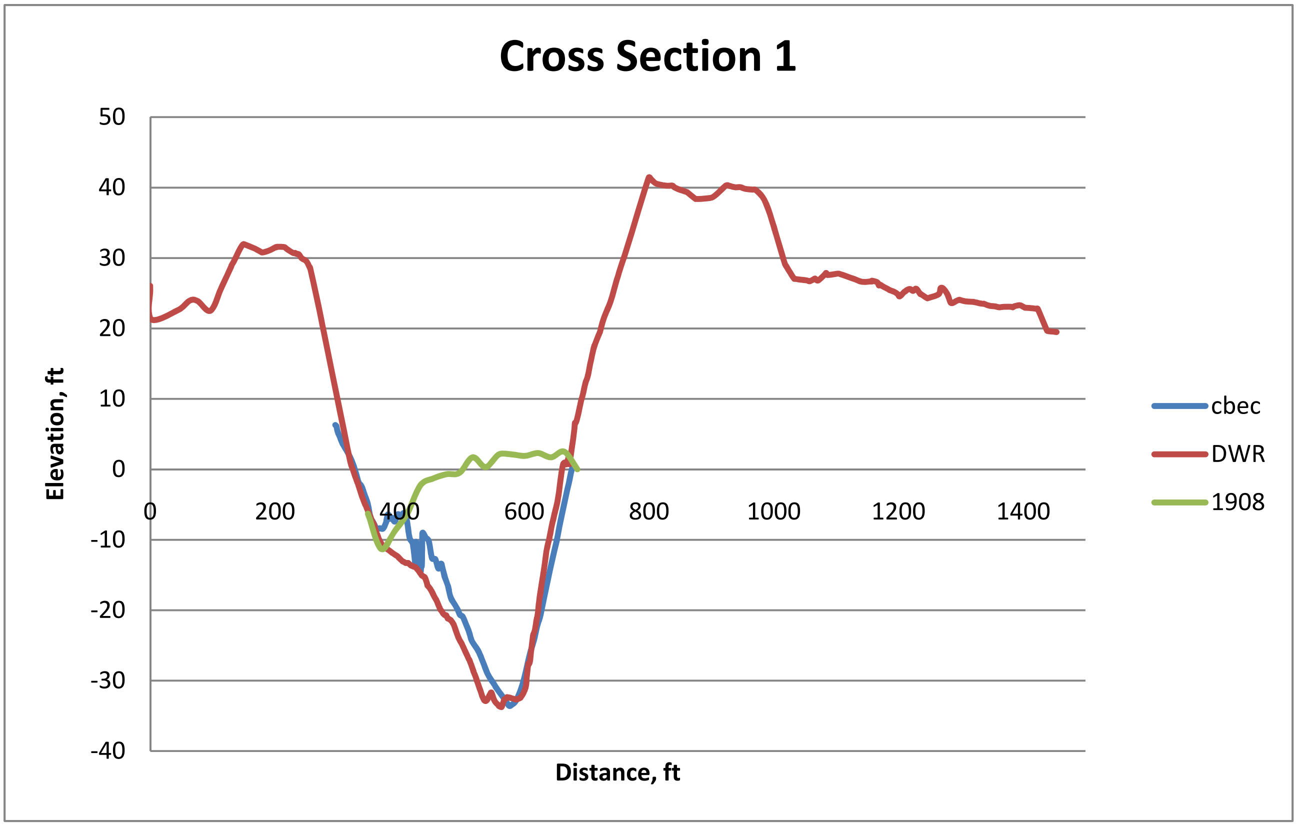
1. Cross sections extracted from a bathymetric digital terrain surface provided by HDR, and based off the Urban Levee Evaluation Program conducted by DWR in May 2008 (ULE).
2. Cross sectional elevation digitized from data obtained from the 1908 mapping shown by Figure 2-4.

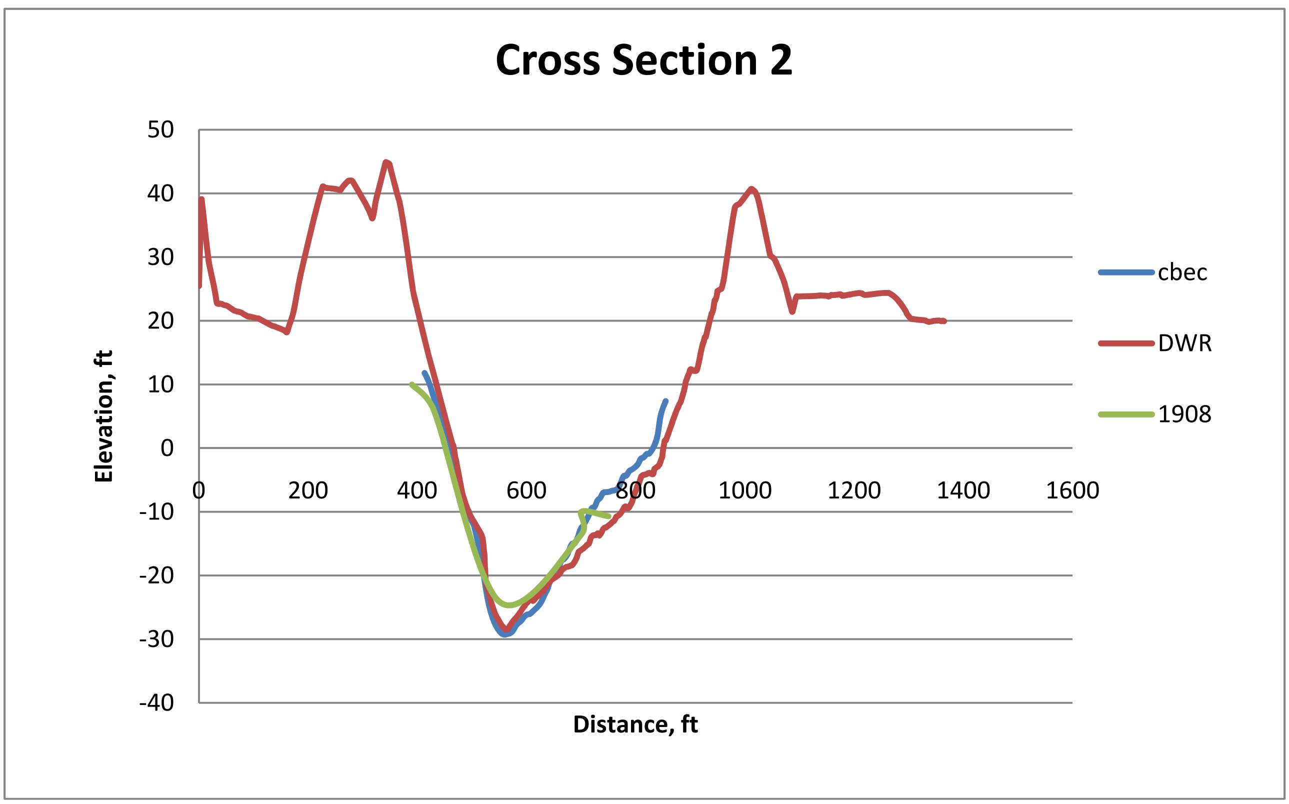
An example of the comparison of these data sources is shown by

Figure 3-10. Further cross sectional comparison is shown in Appendix A. Various observations can be made based on these analyses:

1. Bed elevations in the Sacramento River were substantially higher in elevation in 1908 (on average by 15 to 20 feet but up to 35 feet higher at one cross section) than present day.
2. The channel cross sections at present day are generally wider than in 1908.
3. Some of the bed elevations measured by cbec in April 2011 are slightly higher in elevation than when surveyed in 2008 as shown by the ULE data. It is inconclusive whether this is due to some small amount of sediment deposition or due to debris in the river providing false returns to sonar equipment during the time of surveying.

It is not surprising to observe that the bed elevations in the Sacramento River were higher in elevation in 1908. As a result of hydraulic gold mining in the later 1800s, vast amounts of sediment were transported from Foothill rivers and streams into tributaries and hence into the Sacramento River. Lower energy reaches of the Sacramento River, such as through the project reach, would have had vast amounts of these sediments deposited on the bed of the river. With the cessation of hydraulic mining in the early 1900s, and the construction of dams (sediment, flood and water supply) through the middle of the 20th century, the supply of sediment to the system was dramatically reduced. As a result a period of rapid erosion occurred causing incision of the bed and widening of the banks of the Sacramento River. Construction of levees through the mid- to late- 20th century, reduced the amount of river widening, but exacerbated the rate of bed incision. While bed incision has likely reduced in the last 20 to 30 years, bank erosion is an ongoing issue.





Notes: Veridical datum referenced NAVD 88	cbec eco engineering	*Historic and Current Preliminary Geomorphic Assessment- Southport EIP* **Sacramento River Cross Sectional Analysis**	
		Project No. 11-1003	Created By: LA | **Figure 3-10**

4 POTENTIAL GEOMORPHIC IMPACTS OF PROJECT

This section provides a preliminary assessment of the potential geomorphic impacts of possible implementation of a levee setback alignment. The potential impacts of possible implementation of alternatives adopting a strengthen-in-place alignment (CMA 1 and 3) were not analyzed since the conveyance capacity of the channel and floodplain through the project reach should not change, and hence the sediment transport capacity should not change. Currently, CMA 2 and CMA 4 include proposals for setback alignments of varying acreages, as shown by Figure 4-1 and Figure 4-2, respectively. The general effect of implementing either CMA 2 or CMA 4 will be to increase the available cross-sectional area for flow, or conveyance area during flood flows in excess of approximately a 2-year return period event. As a result of an increase in conveyance area, the flow velocity generally reduces. However, in certain locations, such as the entry or exit to a setback area, localized velocities may also increase.

4.1 REGION WIDE SEDIMENT TRANSPORT ASSEMENT

An analysis was undertaken in order to assess whether the corresponding reduction or increase in velocity, and hence bed and bank shear stresses, could present geomorphic problems in terms of sediment transport capacity, or excessive erosion, both through the project reach, and region wide. In terms of region wide, the geographic coverage of the assessment matches the hydraulic analysis undertaken by MBK Engineers, and upon which the results of this assessment are based.

4.1.1 Data Sources

Data on channel shear stress for the 2-, 10-, 25-, 50-, 100- and 200-year return period events under 'No Setback', 'CMA 2', and 'CMA 4' scenarios were obtained, along with river centerline and cross section shapefiles, using a 1-dimensional hydrodynamic model (HEC-RAS) developed by MBK Engineers. The shear stress data obtained represent the maximum values simulated.

4.1.2 Methods

Percent change in channel shear stress was calculated between 'No Setback', or existing conditions and the CMA 2 and CMA 4 scenarios. Percent change in channel shear stress, along with actual shear stress from 'No Setback' conditions and CMA 4 scenario, were imported into ArcMap and referenced to the corresponding cross section based on river station. Percent change in channel shear stress and the actual channel shear stress was symbolized for all flood events. The symbology for actual shear stress is based off the critical shear stress values as described by Fischenich (2001), which gives the upper limit thresholds needed to move sediment of various sizes. Table 4-1 provides these values in pounds per square foot (lb/sf).

Table 4-1 Critical Shear Stress Values and Symbols

Symbol	Class Name	Critical Shear Stress (lb/ft^2)
● (red)	Very Coarse Gravel	0.54
● (orange)	Coarse Gravel	0.25
● (yellow)	Medium Gravel	0.12
● (light green)	Fine Gravel	0.06
● (cyan)	Very Fine Gravel	0.03
● (light blue)	Very Coarse Sand	0.01
● (blue)	Coarse Sand	0.006

4.1.3 Results

Table 4-2 and Table 4-3 summarizes the maximum and minimum percent change in shear stress over the reach (calculated on a cross-sectional basis) between the existing conditions and setback alignments proposed under CMA 2 and CMA 4, respectively. Table 4-4 shows the values of absolute shear stress through the project reach for existing conditions and for the setback alignments proposed under CMA 4.

Table 4-2 shows minimum and maximum values for percent change in channel shear stress ranging from a percent reduction in shear stress of just over 32% in the 200-year event for CMA 2, to a percent reduction in shear stress of just over 45% in the 200-year event for CMA 4, as shown by Table 4-3. Relative increases in shear stress for both CMA 2 and 4 are well below 10%, and are considered insignificant.

Table 4-2 Percent Change in Channel Shear Stress Compared to Existing Conditions in CMA 2

	Max % Change			Min % Change		
Return Period of Flood Events	Change in Shear Stress (lb/ft^2)	Existing Shear Stress (lb/ft^2)	CMA 2 Shear Stress (lb/ft^2)	Change in Shear Stress (lb/ft^2)	Existing Shear Stress (lb/ft^2)	CMA 2 Shear Stress (lb/ft^2)
2	6%	0.15	0.16	-21%	0.19	0.15
10	6%	0.16	0.17	-27%	0.22	0.16
25	5%	0.17	0.18	-26%	0.19	0.14
50	6%	0.18	0.19	-30%	0.20	0.14
100	6%	0.18	0.19	-30%	0.20	0.14
200	7%	0.14	0.15	-32%	0.20	0.14

Table 4-3 Percent Change in Channel Shear Stress Compared to Existing Conditions in CMA 4

	Max % Change			Min % Change		
Return Period of Flood Events	Change in Shear Stress (lb/ft^2)	Existing Shear Stress (lb/ft^2)	CMA 4 Shear Stress (lb/ft^2)	Change in Shear Stress (lb/ft^2)	Existing Shear Stress (lb/ft^2)	CMA 4Shear Stress (lb/ft^2)
2	7%	0.13	0.14	-21%	0.19	0.15
10	6%	0.16	0.17	-31%	0.22	0.15
25	5%	0.18	0.19	-37%	0.24	0.15
50	5%	0.18	0.19	-40%	0.25	0.15
100	6%	0.15	0.16	-40%	0.25	0.15
200	7%	0.28	0.30	-45%	0.31	0.17

Generally, region-wide impacts of the potential implementation of CMA 2 and 4 are negligible. However, some localized features appeared to be significant and required further assessment.

Figure 4-3 and Figure 4-4 show the percent change in shear stress between existing and with the setback alignment proposed under CMA 2, for the 2-, 5- and 25-year and the 50-, 100- and 200-year return period events, respectively. In the 2- and 5-year return period event, changes in shear stress observed are negligible, with less than a 10% change. In the 25-year event, one small reach may experience greater than a 20% reduction in shear stress. This reduction in shear stress is observed to increase slightly with increasing return period interval up to the 200-year event. No increases in shear stress greater than 5% are observed, based on the model results. These increases are determined to be negligible.

Figure 4-5 and Figure 4-6 show the percent change in shear stress between existing and with the setback alignment proposed under CMA 4, for the 2-, 5- and 25-year and the 50-, 100- and 200-year return period events, respectively. In the 2- and 5-year return period event, changes in shear stress observed are negligible, with less than a 10% change. In the 50-, 100- and 200-year return period events, shear stress is generally predicted to reduce with increasing return period events, but generally only up to a reduction of 10%. In one small reach, similar to the results obtained from CMA 2, shear stress may reduce by in excess of 20%. No increases in shear stress greater than 5% are observed, based on the model results. These increases are determined to be negligible.

Generally, localized increases in shear stress could be problematic in terms of the potential for increased erosion as a result of the potential setback alignments. Shear stress typically increases upstream a levee setback because of the local reduction in water surface elevation within setback reach. The localized reduction in elevation provides for a increase in the water surface slope immediately upstream of the setback reach as the upstream water surface elevation remains unchanged. Regionally, increases in shear stresses for all scenarios does not exceed 5%. The potential "noise" and variability of these types of analysis could potentially be 10% or more, and therefore, the increases in shear stresses regionally were considered to be negligible. However, reductions in shear stresses may indicate the potential for sediment deposition. Significant deposition could be an issue for local marinas or navigation generally in the region. Since reduction in shear stress of up to 45% were observed for the 200-year return period event between existing conditions and the proposed setback alignment of CMA 4, additional analysis was undertaken in order to assess the sediment transport capacity under CMA 4.

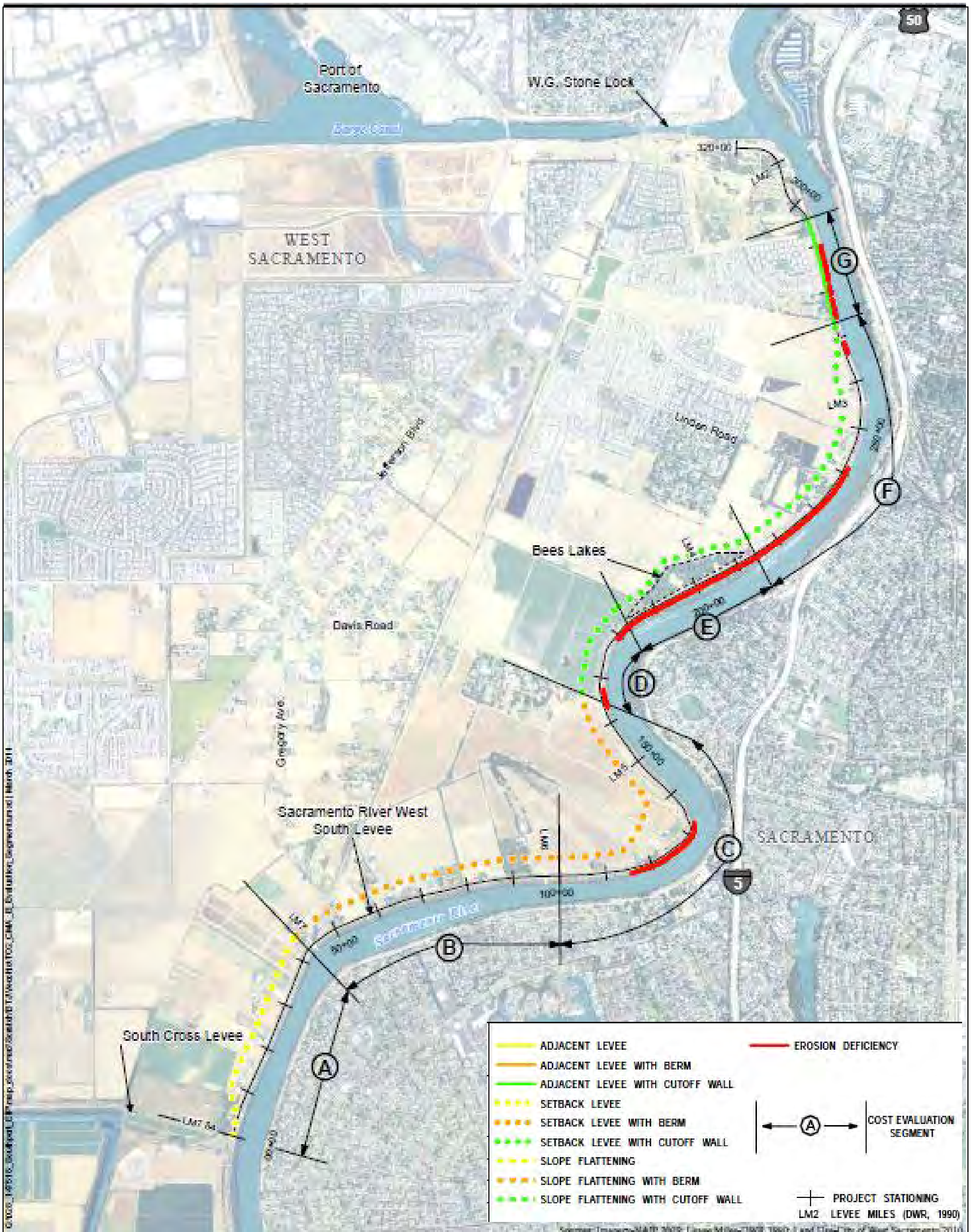
Table 4-4 shows the absolute shear stress maximum values for current conditions, compared to conditions under CMA 4.

Table 4-4 Absolute Shear Stress, Maximum Values for Current Conditions and CMA 4

Flood Year	No Setback Max (lb/sf)	CMA 4 Max (lb/sf)
2	0.29	0.29
10	0.34	0.35
25	0.37	0.40
50	0.38	0.43
100	0.33	0.46
200	0.35	0.45

Table 4-4 shows the maximum shear stress values observed among all cross sections for CMA 4. The shear stress values do not represent individual cross sections, as they do in Table 4-2 and Table 4-3. The intention of Table 4-4 is to demonstrate that sediment will likely still be transported under CMA 2 or 4 conditions, rather than be deposited.

Further, Figure 4-7 and Figure 4-8 show the absolute shear stress under existing conditions for the 2-, 10-, 25-, 50-, 100- and 200-year return period events. In can be observed that the shear stresses under these conditions are of sufficient magnitude to transport coarse to very coarse gravels. Figure 4-9 and Figure 4-10 show the absolute shear stress under the proposed setback alignment conditions for the 2-. 10-, 25-. 50-, 100- and 200-year return period events. It can be observed that the shear stresses under these conditions, while the shear stresses are generally slightly lower than under existing conditions, still have sufficient magnitude to transport coarse to very coarse gravels. Therefore, it is likely that sediment deposition will not be increased between the 2- and 200-year events as a result of proposed setback alignments.



Notes: Map courtesy of HDR.

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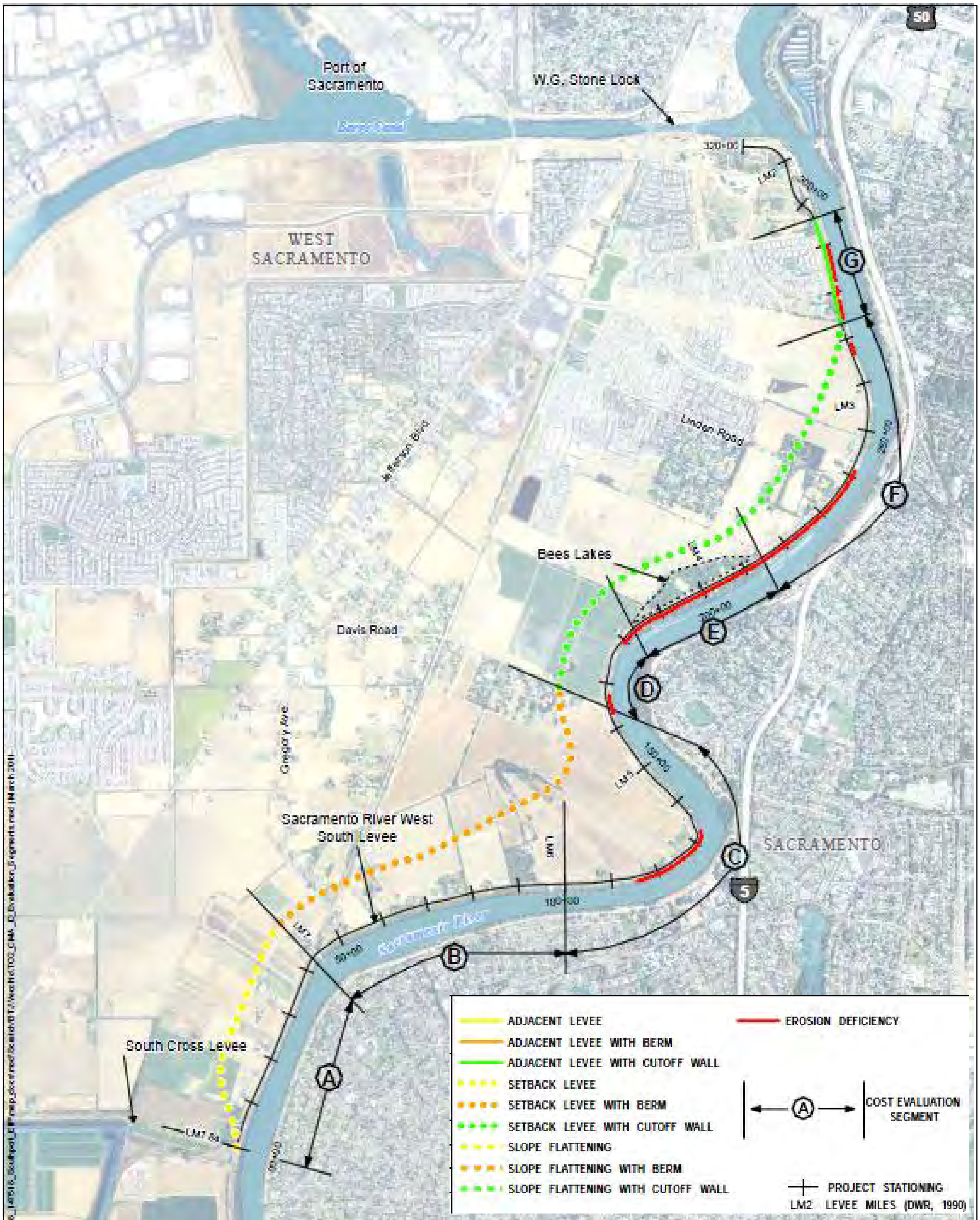
Proposed Levee Setback Alignment- CMA 2

Project No. 11-1003	Created By: LA	**Figure 4-1**

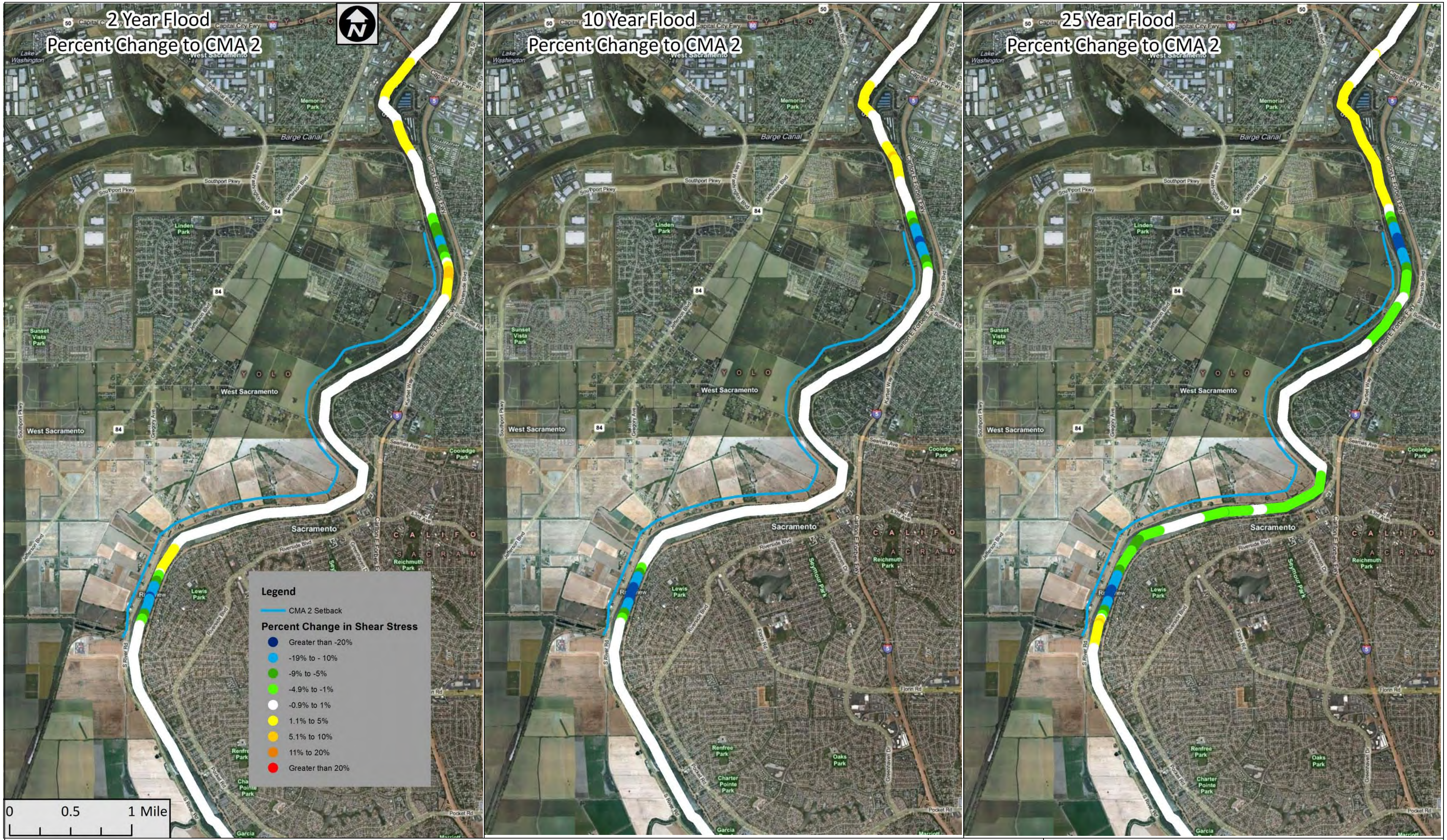


Notes: Map courtesy of HDR.

Historic and Current Preliminary Geomorphic Assessment- Southport EIP

Proposed Levee Setback Alignment- CMA 4

Project No. 11-1003	Created By: LA	**Figure 4-2**



Notes: Orthoimagery courtesy of Bing Maps.

Historic and Current Preliminary Geomorphic Assessment- Southport EIP	
Channel Shear Stress Percent Changes CMA2- I	
Project No. 11-1003	Created By: LA
Figure 4-3	

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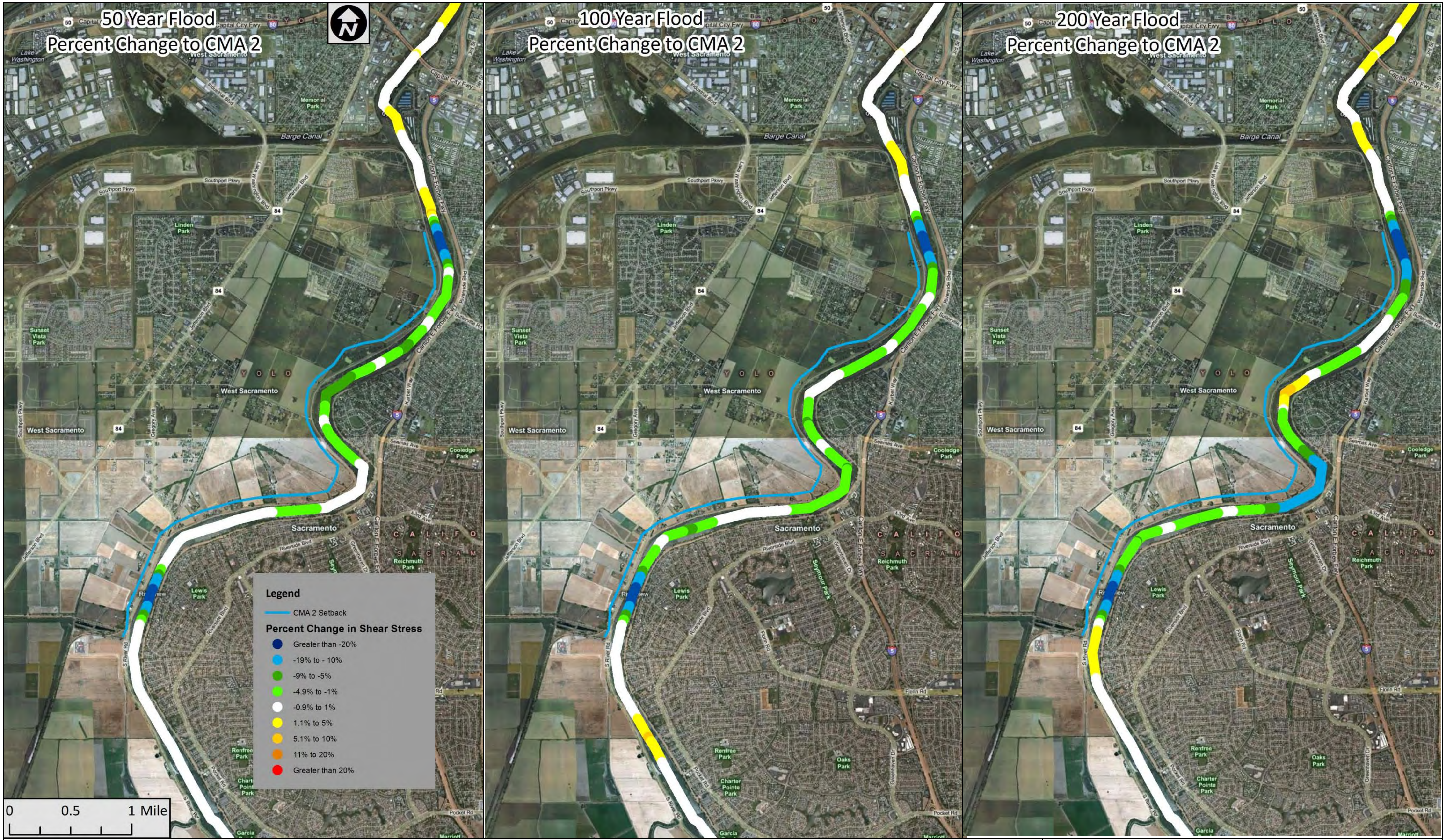

Notes: Orthoimagery courtesy of Bing Maps.

Historic and Current Preliminary Geomorphic Assessment- Southport EIP

Channel Shear Stress Percent Changes CMA2 II

Project No. 11-1003	Created By: LA	**Figure 4-4**

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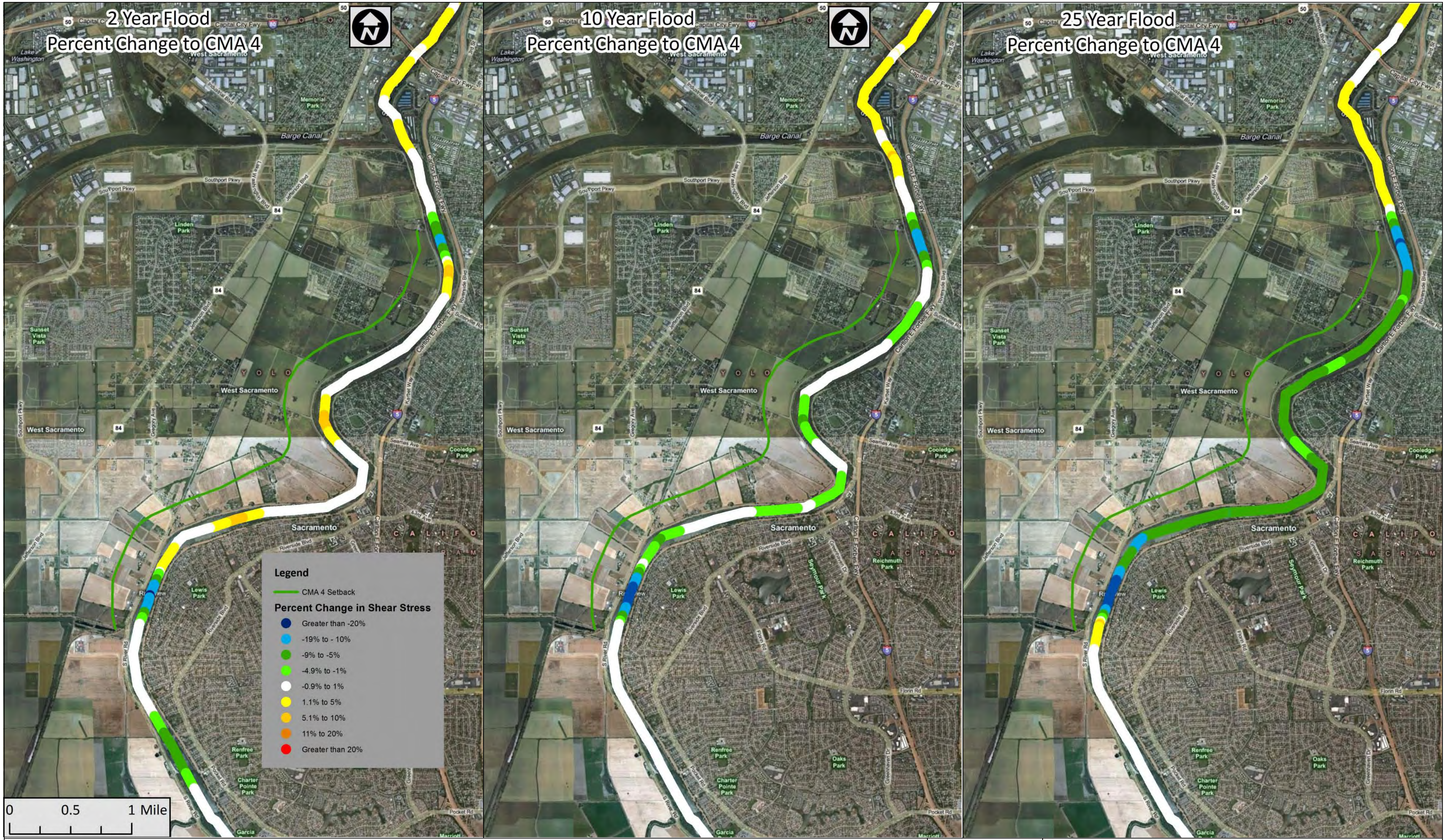

Notes: Orthoimagery courtesy of Bing Maps.

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Channel Shear Stress Percent Change CMA4

Project No. 11-1003 | Created By: LA | **Figure 4-5**

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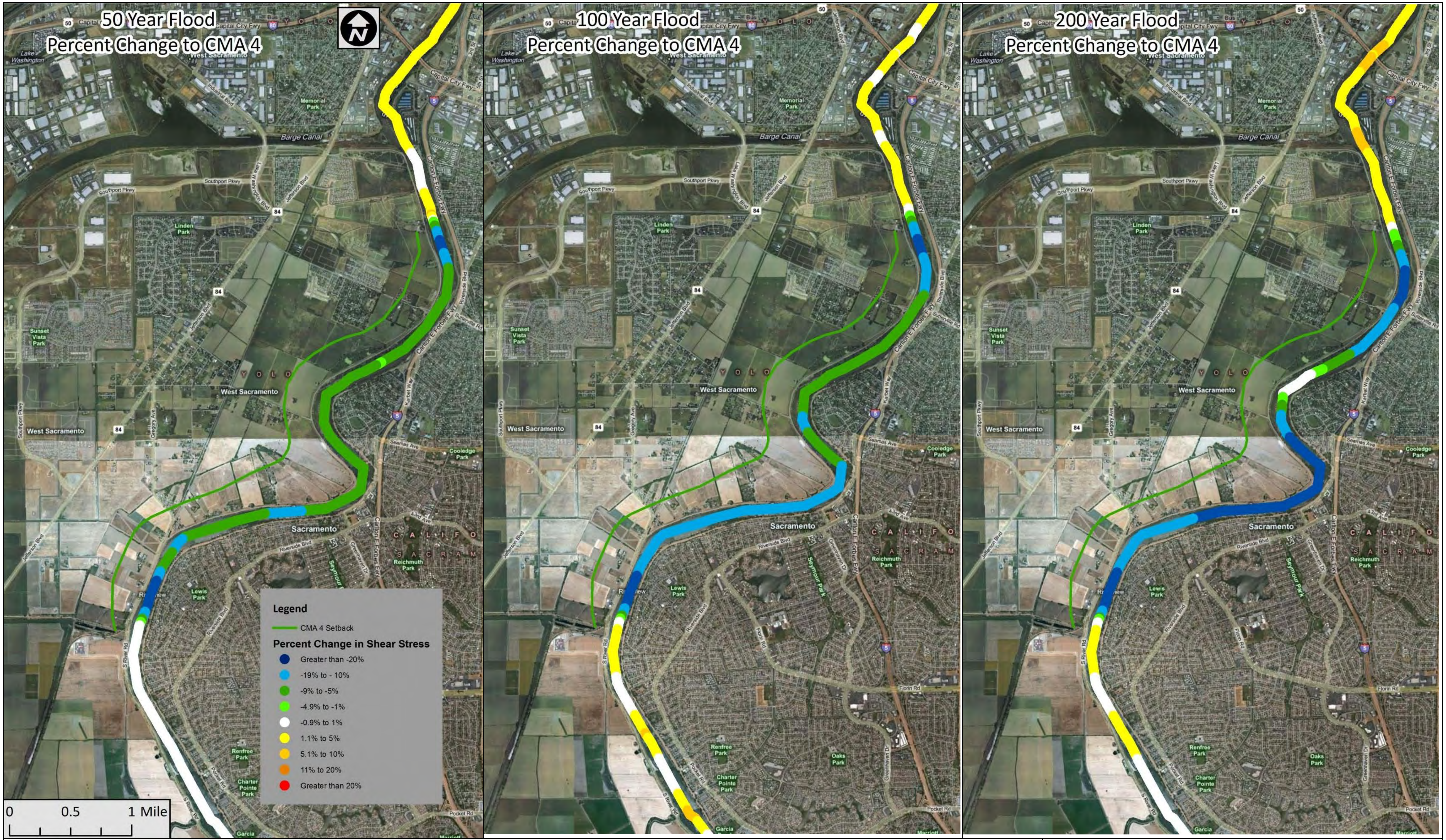

Notes: Orthoimagery courtesy of Bing Maps.

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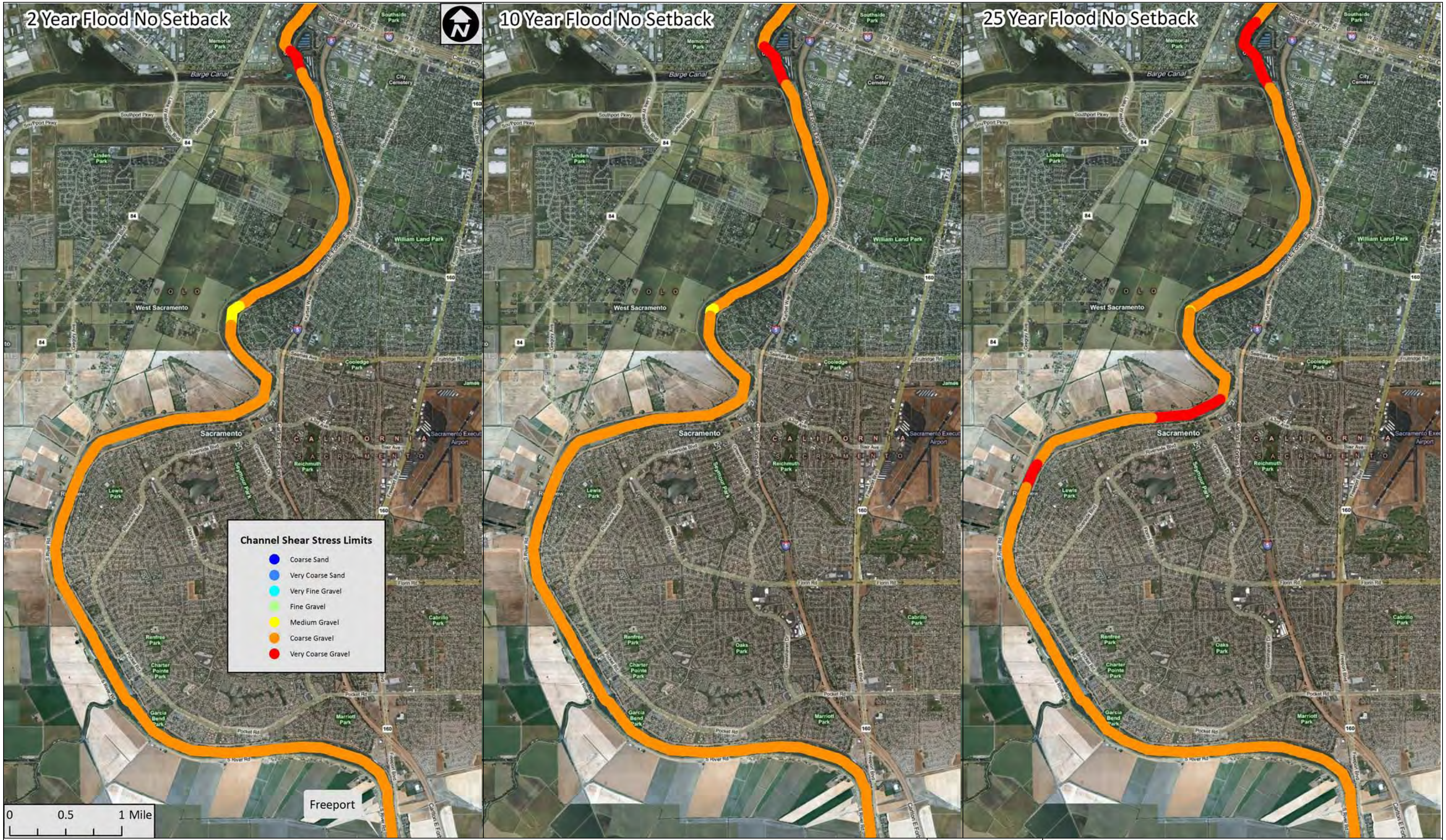
Channel Shear Stress Percent Changes CMA4 II

Project No. 11-1003 | Created By: LA | **Figure 4-6**



Shear stress limits based on Fischenich, 2001. Colors represent upper limit of shear stress power. Orthoimagery courtesy of Bing Maps.

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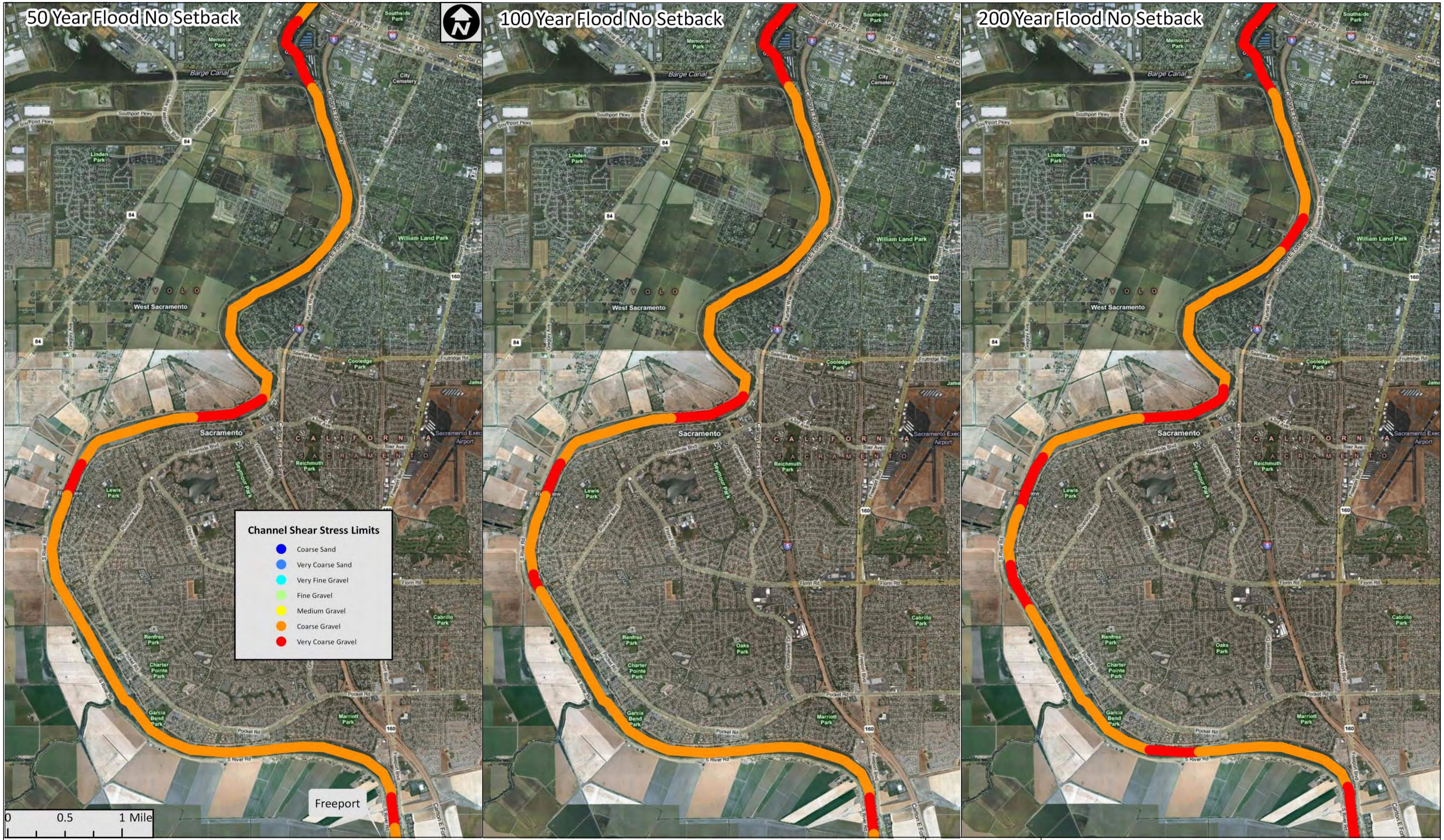
Historic and Current Preliminary Geomorphic Assessment- Southport EIP

Absolute Channel Shear Stress Existing- I

Project No. 11-1003	Created By: WGW	**Figure 4-7**

R:\Projects\11-1003_West_Sac_Levee_TO2\Report\Shear Stress Figs WW\Finished fig boxes\NoSetback 1.docx
5/13/2011



Notes: Shear stress limits based on Fischenich, 2001. Colors represent upper limit of shear stress power. Orthoimagery courtesy of Bing Maps.

cbec

Historic and Current Preliminary Geomorphic Assessment- Southport EIP

Absolute Channel Shear Stress Existing-II

Project No. 11-1003	Created By: WGW	**Figure 4-8**

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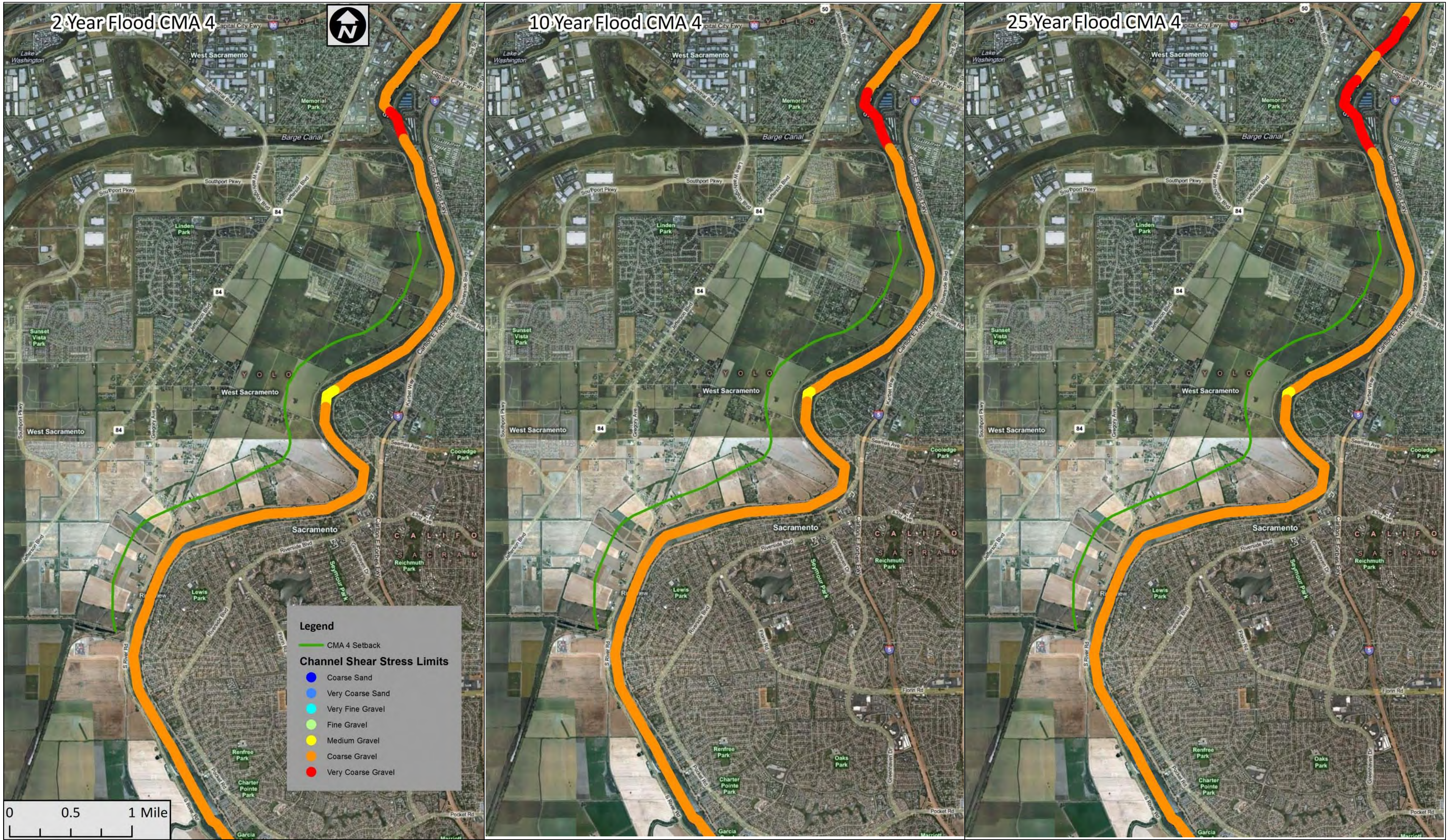
5/13/2011



Notes: Orthoimagery courtesy of Bing Maps. Colors represent upper limit of shear stress based on Fishenice, 2001.

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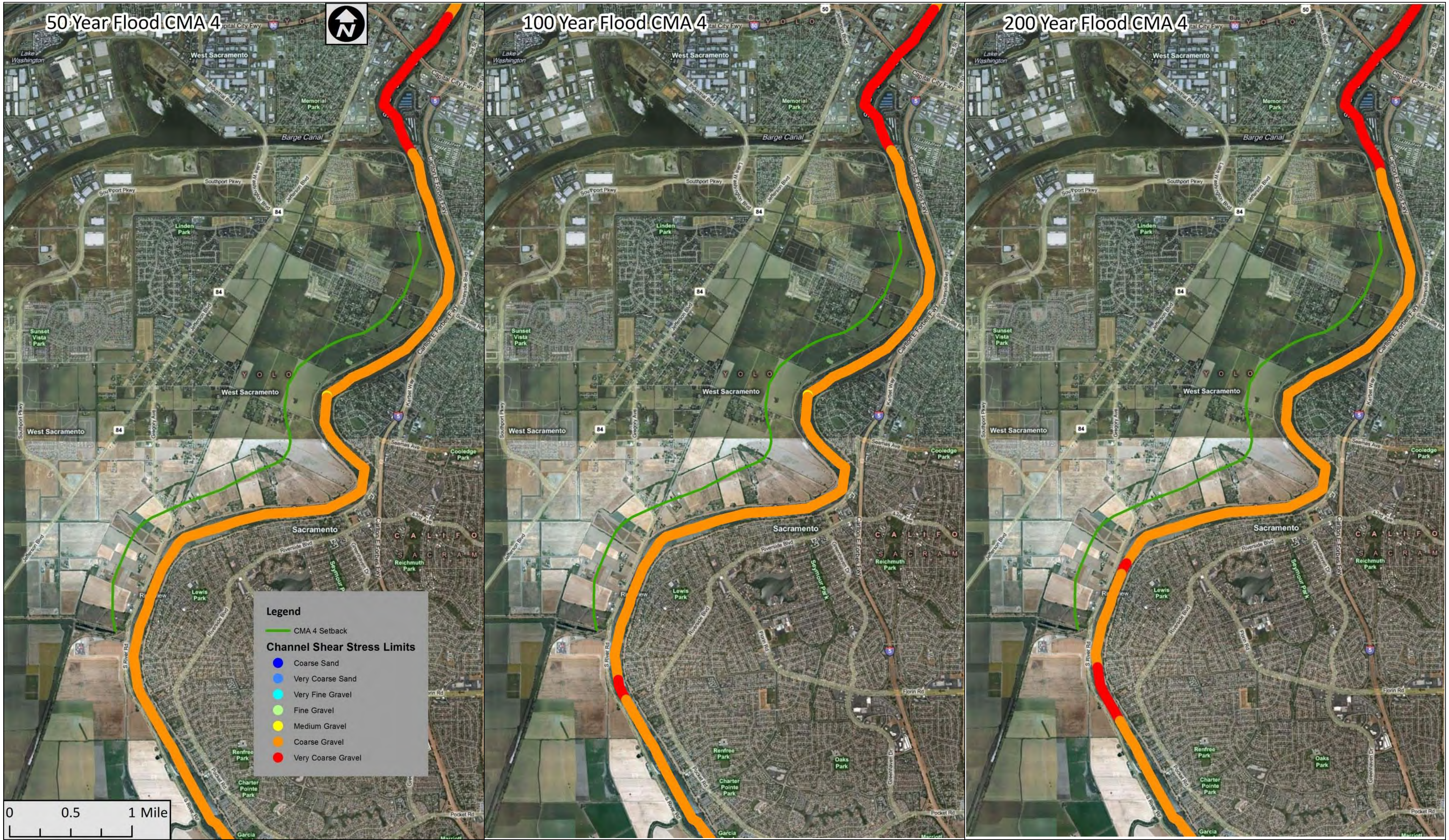
Historic and Current Preliminary Geomorphic Assessment- Southport EIP

Absolute Channel Shear Stress CMA4- I

Project No. 11-1003	Created By: LA	**Figure 4-9**

R:\Projects\11-1003_West_Sac_Levee_TO2\Report\Shear Stress Figs WW\Act 2-25yr CMA4.docx
5/13/2011



Notes: Shear stress limits based on Fischenich, 2001. Colors represent upper limit of shear stress power. Orthoimagery courtesy of Bing Maps.

cbec eco engineering

Historic and Current Preliminary Geomorphic Assessment- Southport EIP

Absolute Channel Shear Stress CMA4- II

Project No. 11-1003	Created By: LA	**Figure 4-10**

R:\Projects\11-1003_West_Sac_Levee_TO2\Report\Shear Stress Figs WW\Act 50-200yr CMA4.docx
5/13/2011

5 CONCLUSIONS

The general conclusions that can be made as a result of investigations undertaken through this study include:

1. Since the late 1800s the planform geometry of the Sacramento River through the project reach has been “locked” in place by levees and rip-rap and has not changed significantly to date. Localized changes in depositional bars and other in-channel sedimentation features have been observed over time.
2. In the early 1900s large amounts of sediment were deposited in the Sacramento River as a result of hydraulic mining practices in Sierra Foothill rivers and streams. This raised the bed substantially. Subsequently, the channel incised and widened leading to its current form, as a result of upstream anthropogenic impacts, such as reservoir and dam construction, and urbanization. Today, the thalweg elevation of the river is up to up to 20 feet lower than in the early 1900s.
3. The geology of the floodplain through the project reach is complex and is being closely studied for potential levee impacts by Blackburn Consulting.
4. The floodplain of the Sacramento River through the project reach should be inundated in at least the 2-year recurrence interval event, or more frequently, from depths ranging from 2 to 6 feet, under levee setback conditions.
5. Few relic floodplain features remain today, upon which to base restoration strategies. Bees Lake is a notable exception, although it is unclear if this is a relic feature, and is more likely a product of anthropogenic influences and levee construction.
6. Regionally, sediment transport impacts of the proposed levee setback are negligible. Frequent bankfull events primarily define channel morphology in river systems such as the Sacramento River. Out-of-bank flows under levee setback conditions will affect the frequency of bankfull events to a negligible extent, and therefore will likely not influence channel morphology over time. Locally, shear stresses through the project reach should be substantially reduced, which may benefit existing bank erosion issues. Reduction in shear stresses should not increase deposition through the project reach.

5.1 RECOMMENDATIONS FOR FUTURE ANALYSIS

This study represents a preliminary assessment only. Additional field reconnaissance and data collection will be required leading to final design. In addition, detailed 2-dimensional sediment transport analysis will be conducted under a subsequent task order to identify potential impacts to specific river and floodplain features, such as areas of potential erosion or deposition.

6 REFERENCES

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7 LIST OF PREPARERS

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8 ACKNOWLEDGMENTS

Don Trieu, P.E., MBK Engineers
Michael Vecchio, P.E., HDR Engineering
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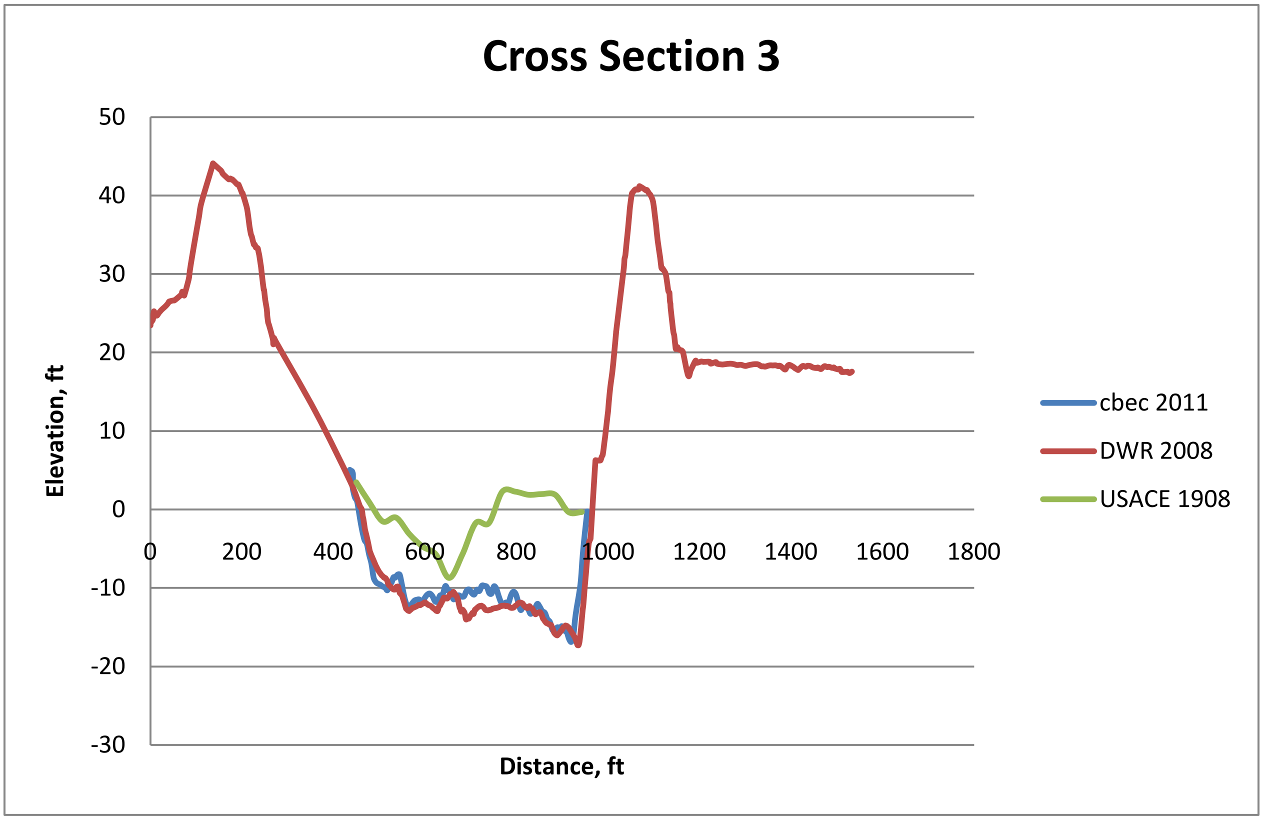
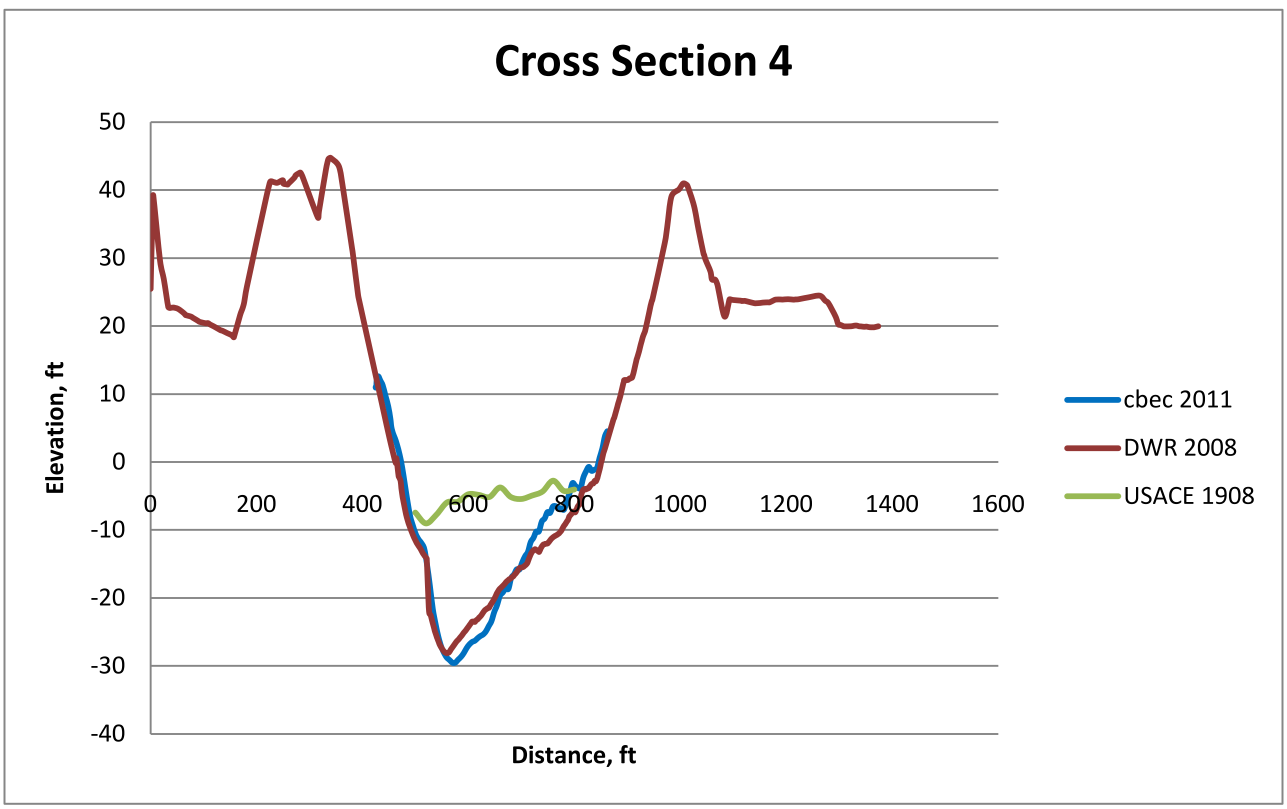
APPENDIX A – CROSS SECTIONAL ANALYSIS

Notes: Orthoimagery courtesy of Bing Maps.	cbec eco engineering	*Historic and Current Preliminary Geomorphic Assessment- Southport EIP* **cbec Cross Section Locations**		
		Project No. 11-1003	Created By: LA	**Appendix A-1**

R:\Projects\11-1003_West_Sac_Levee_TO2\Report\Map Figs\Cross Section Locations.docx
5/13/2011





Notes: Veridical datum referenced NAVD 88	cbec eco engineering	*Historic and Current Preliminary Geomorphic Assessment- Southport EIP* **Sacramento River Cross Sectional Analysis**	
		Project No. 11-1003	Created By: LA	Appendix A-2

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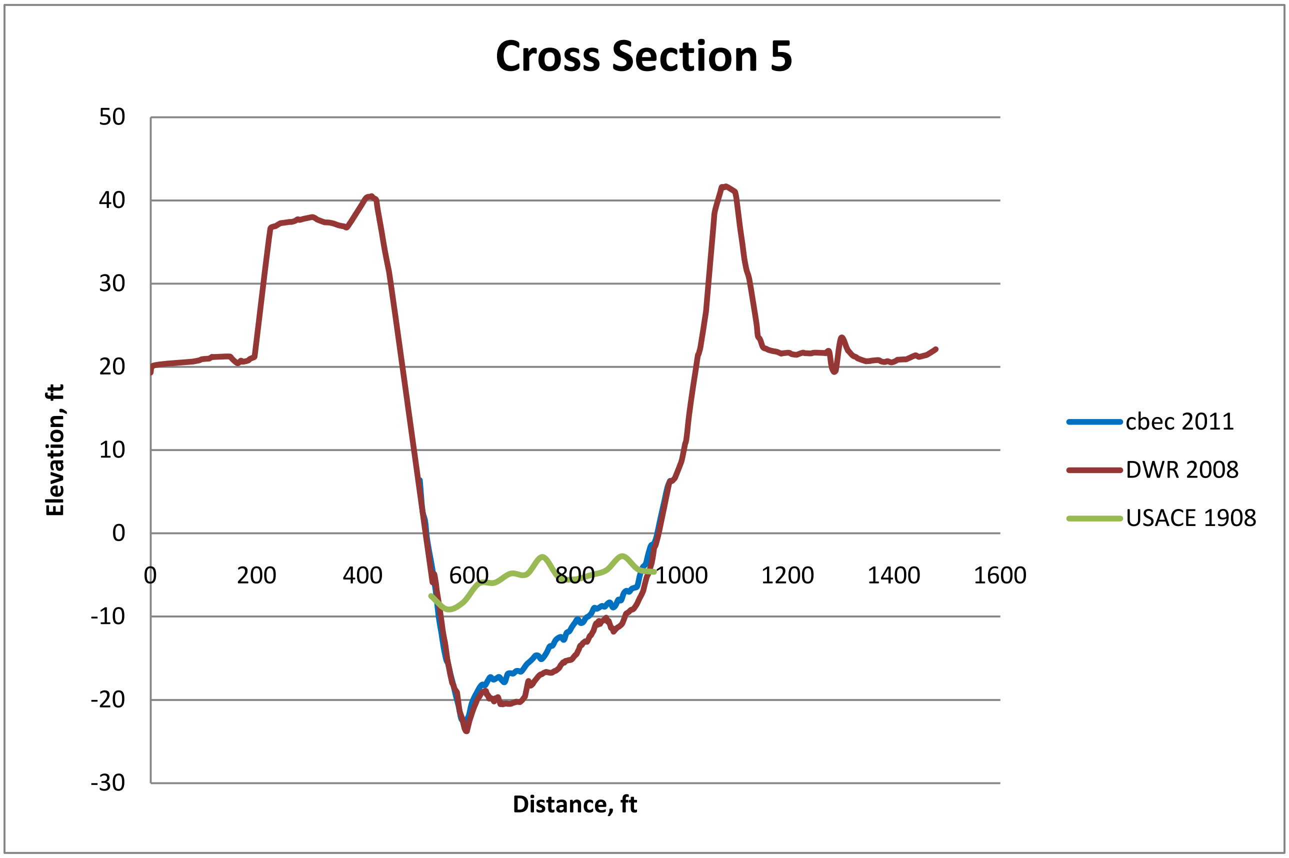
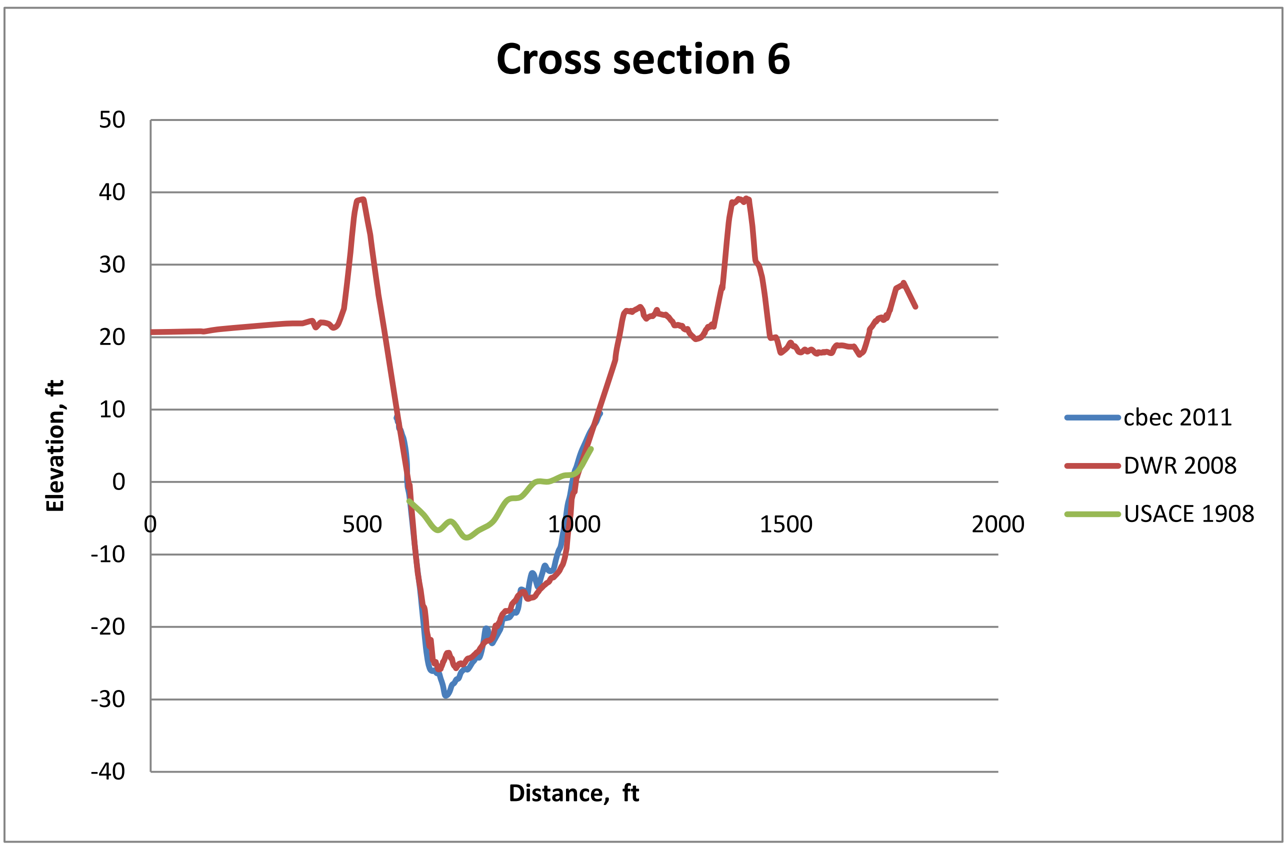
5/13/2011





Notes: Veridical datum referenced NAVD 88	cbec eco engineering	*Historic and Current Preliminary Geomorphic Assessment- Southport EIP* **Sacramento River Cross Sectional Analysis**
		Project No. 11-1003 | Created By: LA | Appendix A-3

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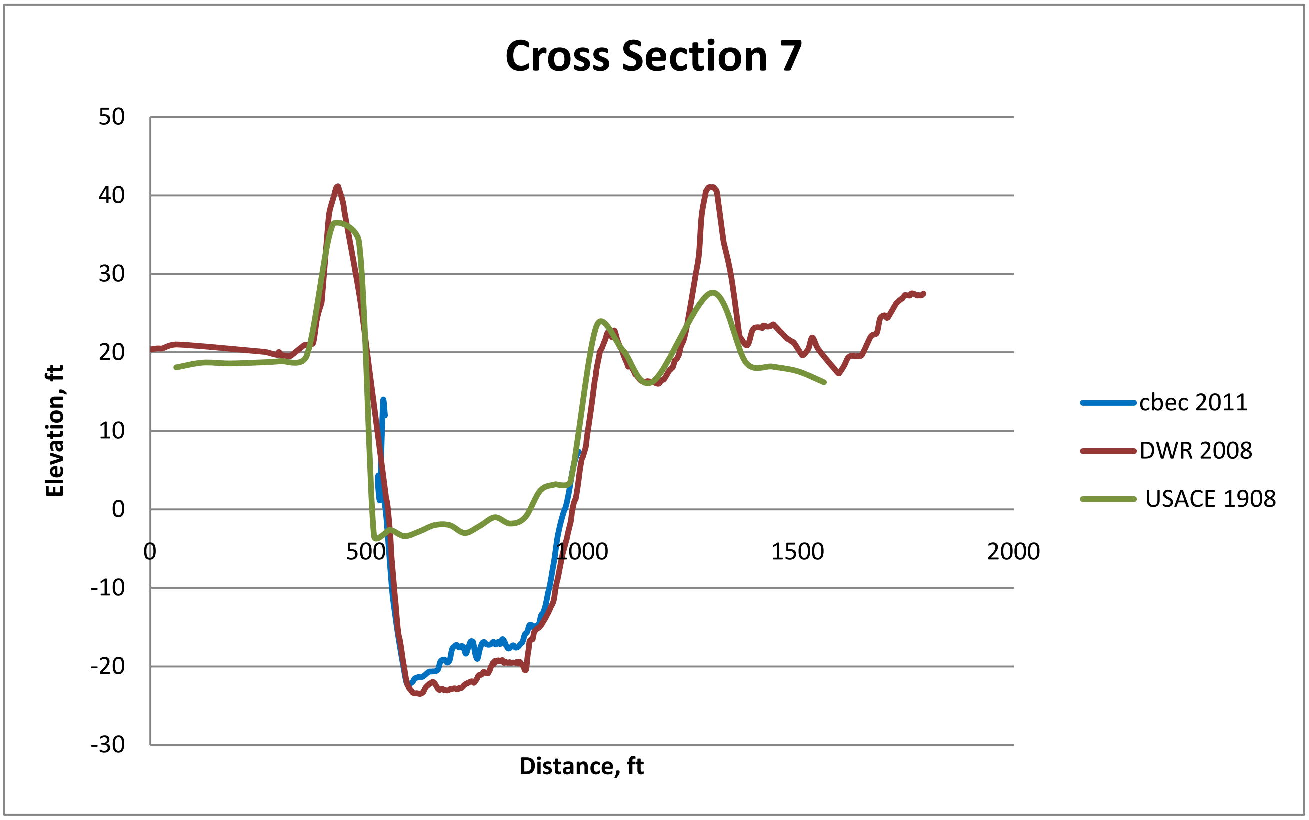
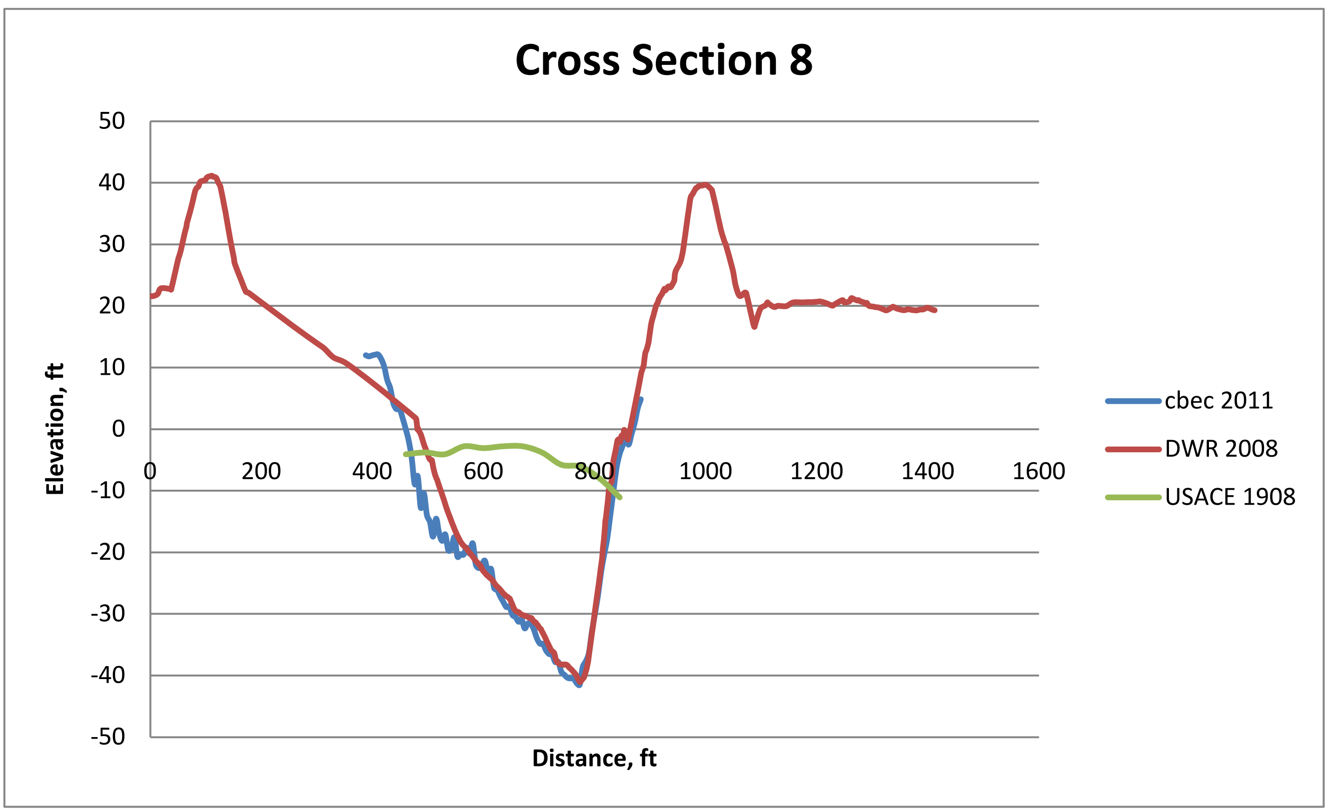
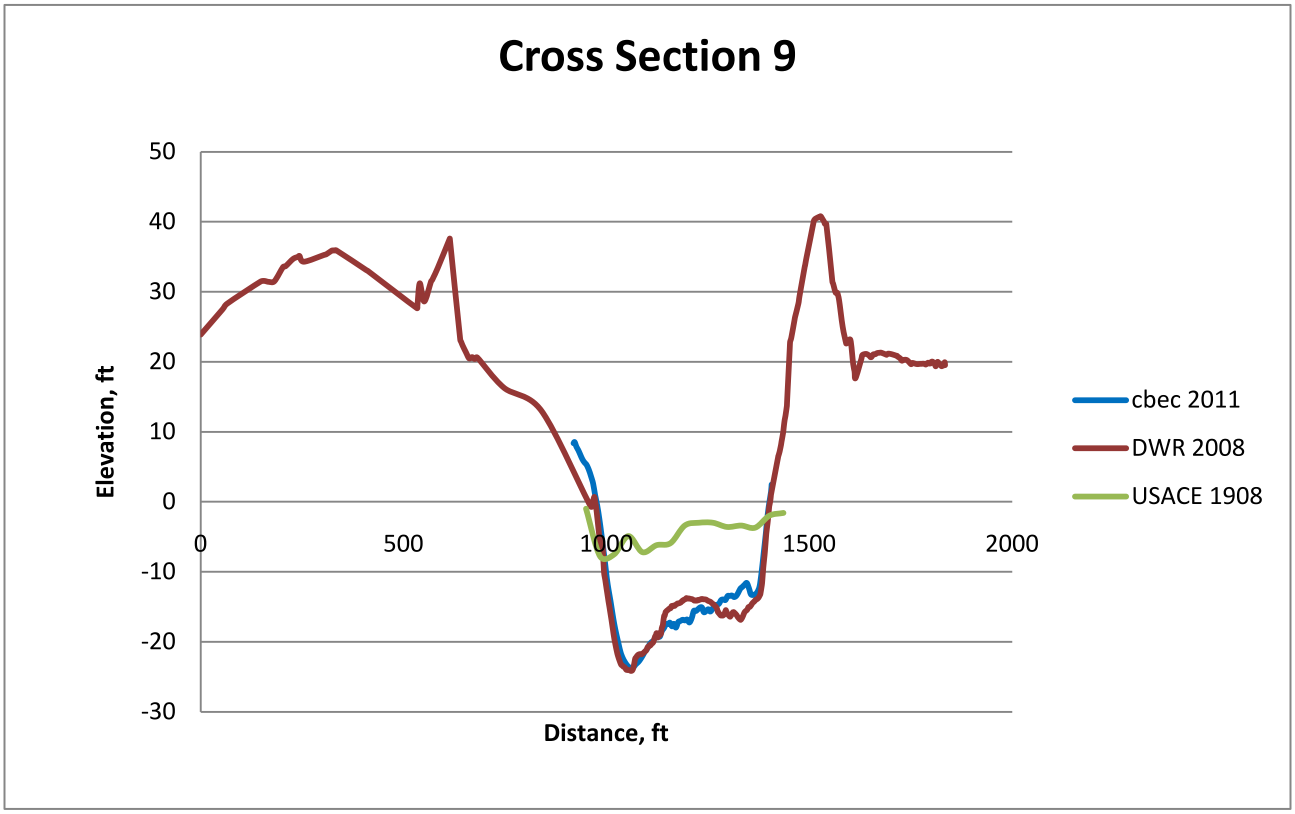
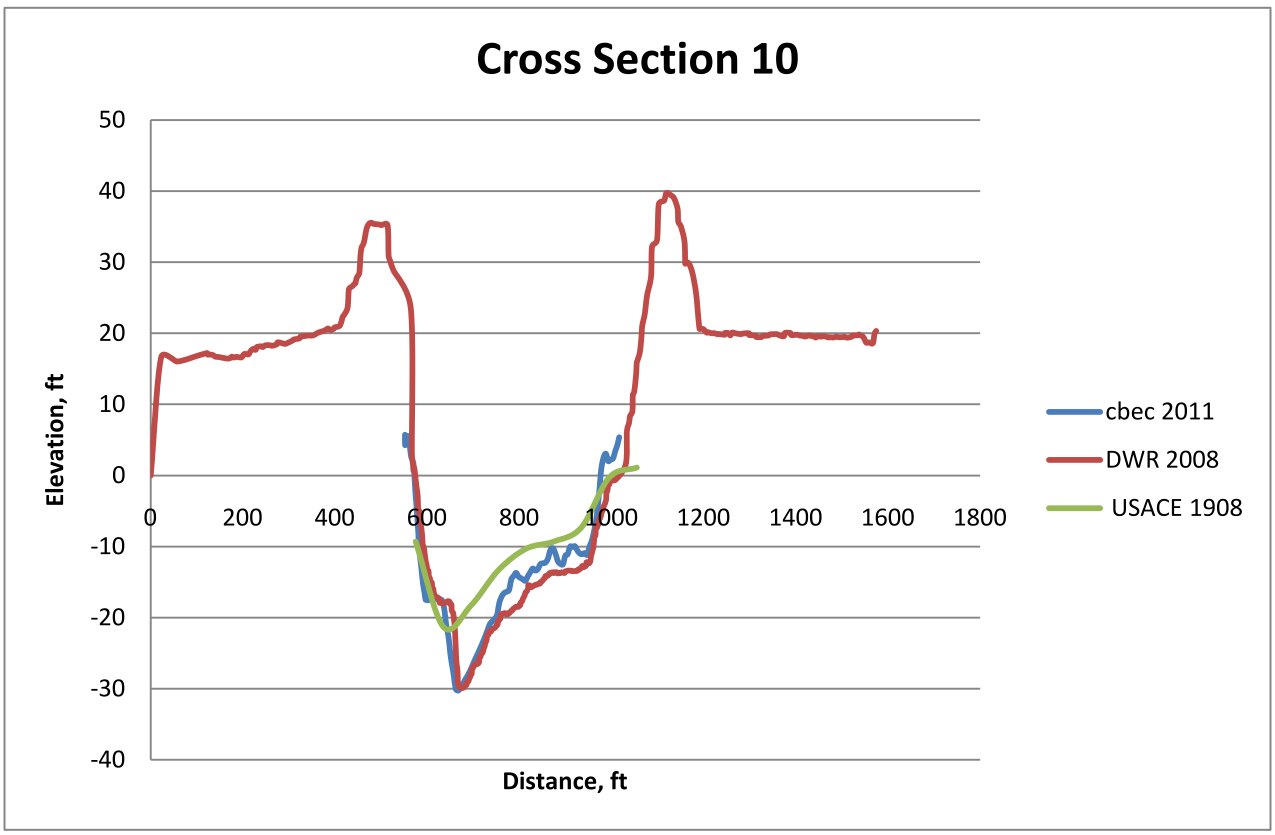
5/13/2011





Notes: Veridical datum referenced NAVD 88	cbec eco engineering	*Historic and Current Preliminary Geomorphic Assessment- Southport EIP* **Sacramento River Cross Sectional Analysis**		
		Project No. 11-1003	Created By: LA	Appendix A-4





Notes: Veridical datum referenced NAVD 88

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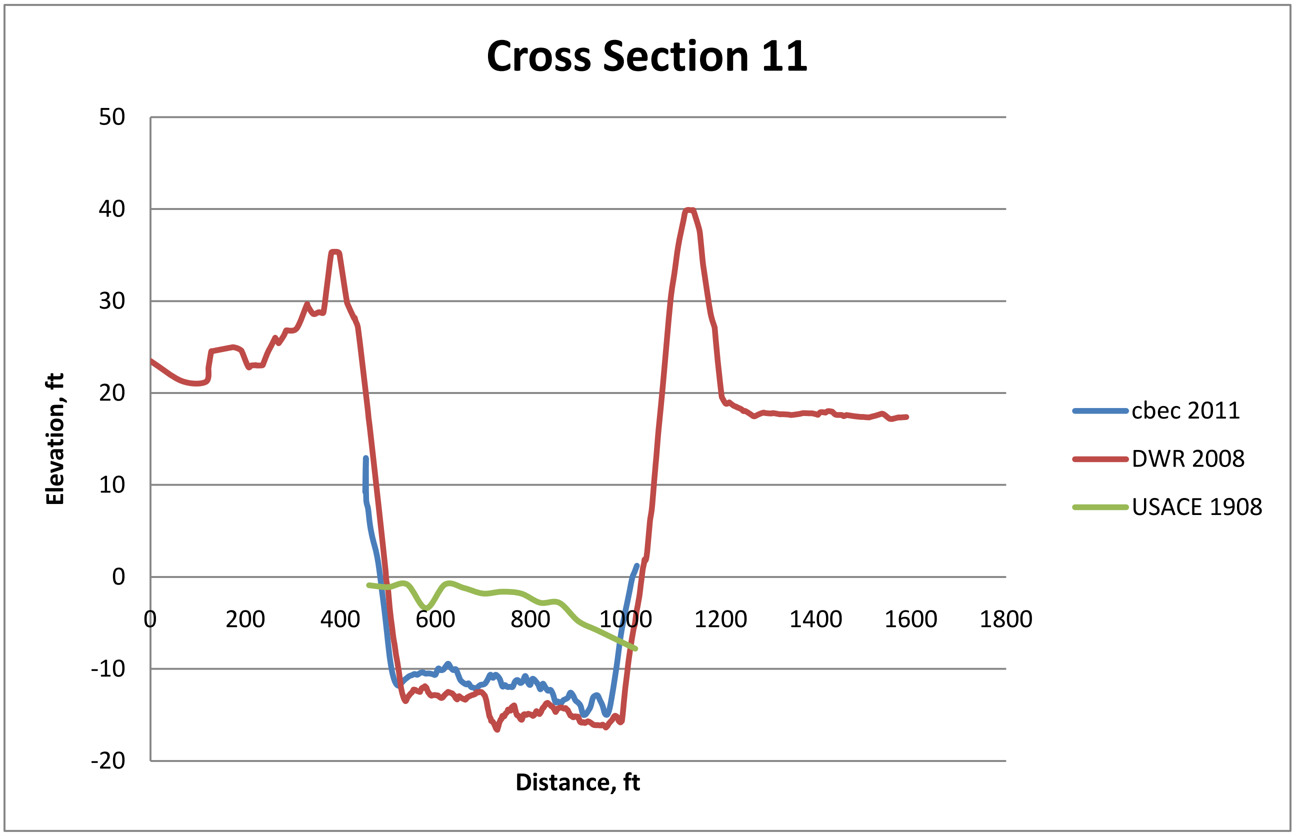
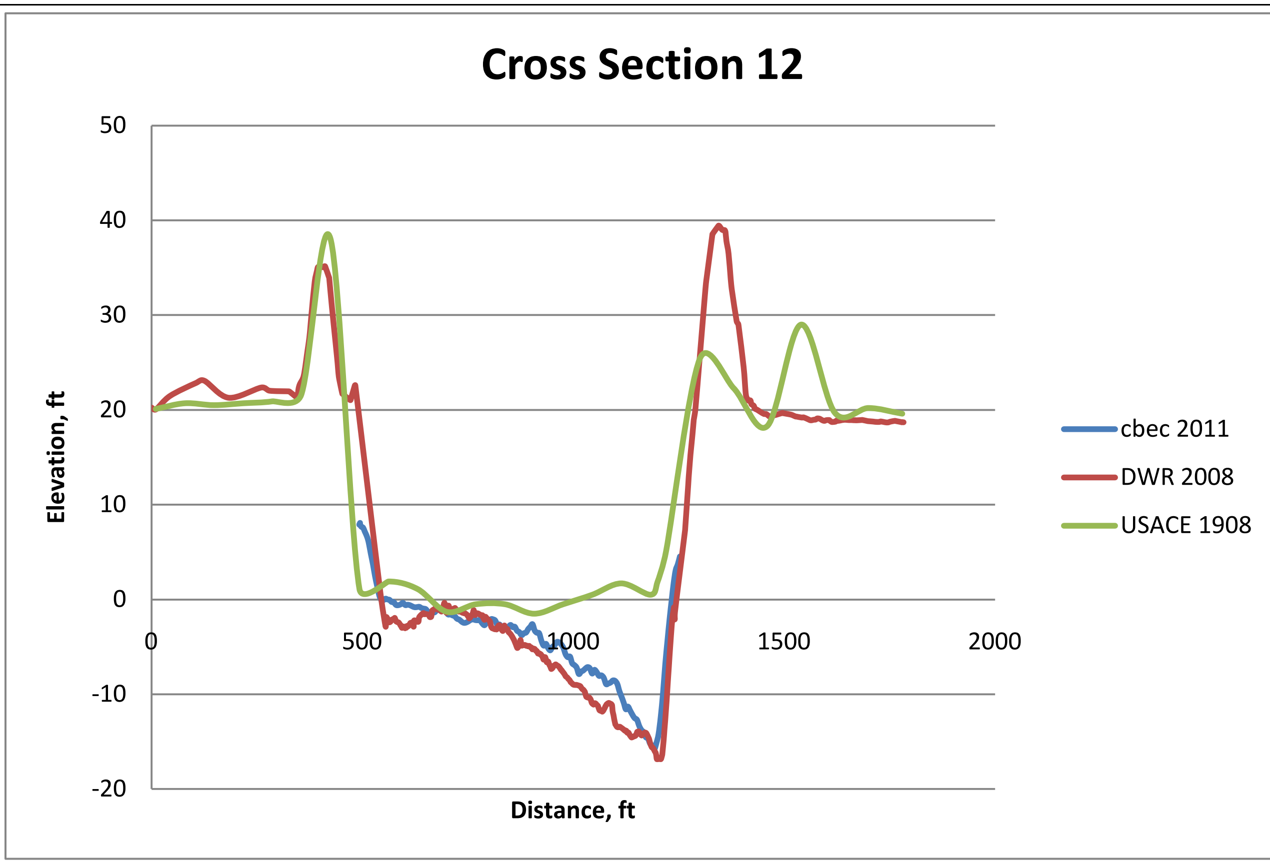
Historic and Current Preliminary Geomorphic Assessment- Southport EIP

Sacramento River Cross Sectional Analysis

Project No. 11-1003	Created By: LA	Appendix A-5



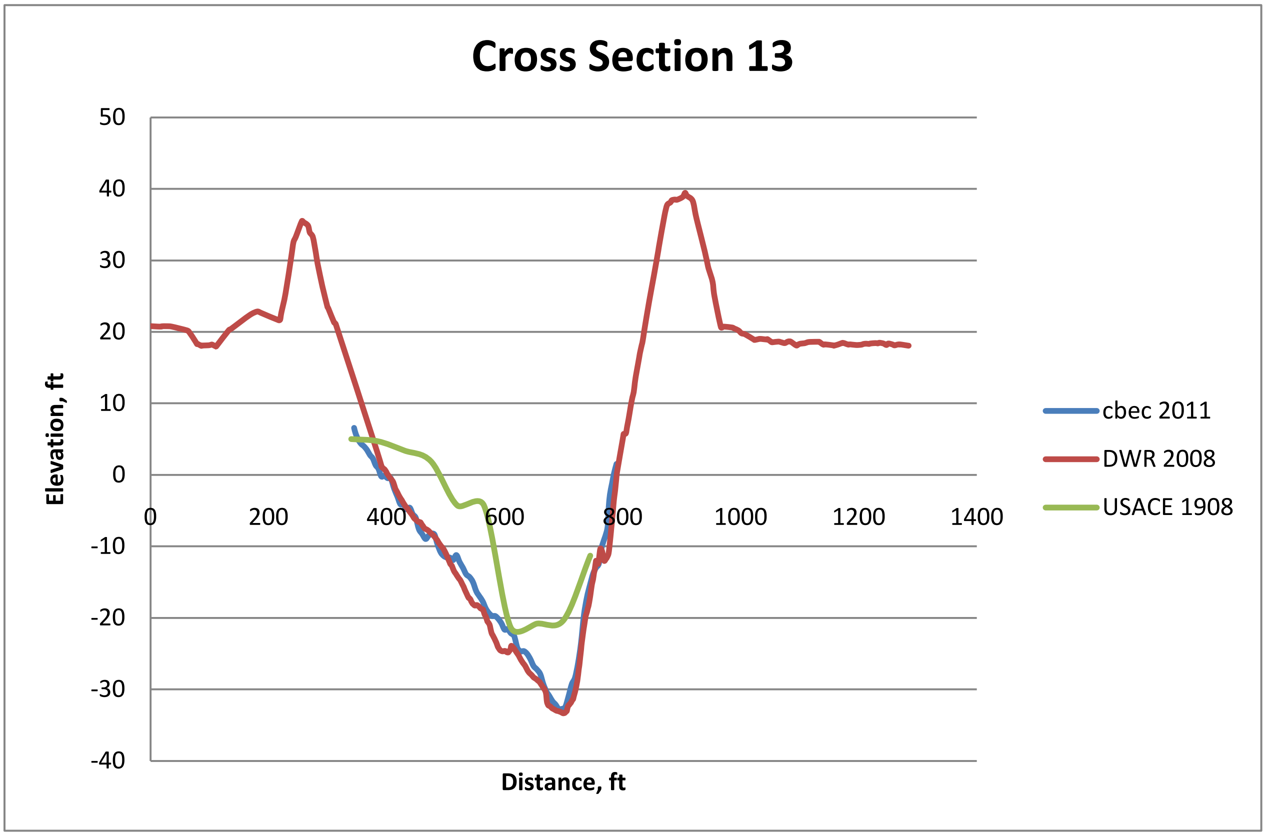


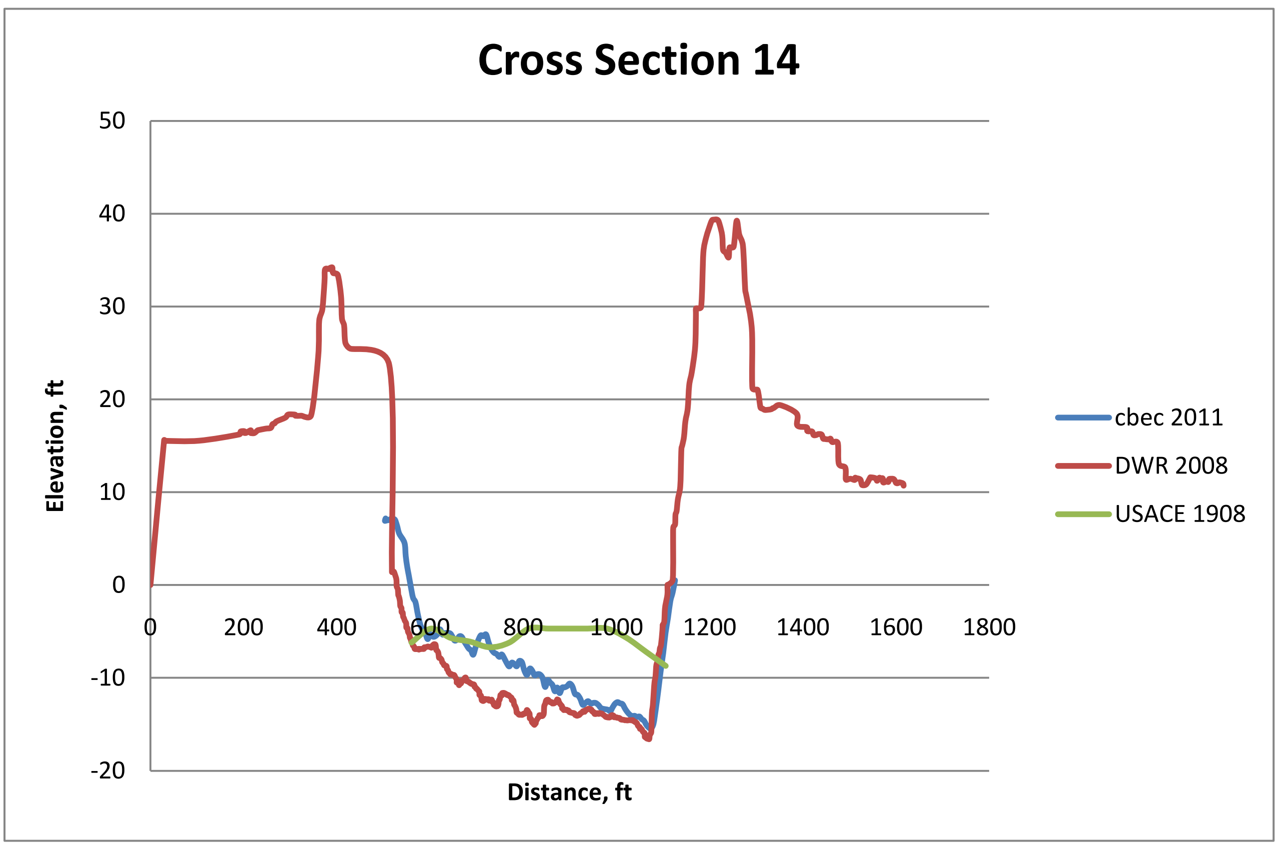
Notes: Veridical datum referenced NAVD 88

Historic and Current Preliminary Geomorphic Assessment- Southport EIP

Sacramento River Cross Sectional Analysis

Project No. 11-1003	Created By: LA	Appendix A-6





Notes: Veridical datum referenced NAVD 88

Historic and Current Preliminary Geomorphic Assessment- Southport EIP

Sacramento River Cross Sectional Analysis

Project No. 11-1003	Created By: LA	Appendix A-7

APPENDIX B – INTERIM DATA COLLECTION MEMORANDUM

Hydraulics | Hydrology | Geomorphology | Design

MEMORANDUM

Date:	05/02/11
To:	Michael Vecchio (HDR Engineering)
From:	John Stofleth, M.S., EIT, Chris Bowles, Ph.D., P.E., cbec inc., eco engineering
Project:	10-1043 – West Sacramento Southport EIP Task Order 1
Subject:	Preliminary Hydraulic and Sediment Transport Data Collection

Introduction

cbec, inc., eco engineering (cbec) has collected water level data, measured flows, and measured suspended and bedload sediment transport during geomorphically and ecologically significant events on the Sacramento River during the 2010-2011 flood season.

Water Level Monitoring

cbec staff deployed 3 stage (water level) recorders (pressure transducers) within the project reach (RM 57.1, 55.6, 52) to continuously measure water level data at 15 minute intervals. Figure 1 displays the locations of the deployed stage recorders. Note that the stage recorder at RM 55.6 failed shortly after installation and no useful data was generated at this location. The stage recorders, manufactured by Solinst, model 3001 Gold, are of the non-vented type. Therefore, a barometric recorder was also deployed. The hydraulic pressure measured by the recorders was post-processed using measured barometric pressure to provide a depth of water above the recorder. The stage recorders were deployed on 12/15/2011 and remain in place collecting data to date. The recorders were surveyed to the NAVD 88 vertical datum and were last downloaded on 02/25/11. Figure 2 displays the data collected during this period. Note that a gap existed that is as a result of the water level dropping below the stage recorder. However, the data not captured is not critical to future studies since the corresponding stages occurred during a period of lower flow, not critical to geomorphic processes or floodplain inundation. Data will be downloaded again this spring after water levels recede so that equipment can be accessed.

A flood frequency analysis of flow data at the Freeport gauge on the Sacramento River, downstream of the Southport EIP project reach, indicated that the flow at the peak of the December 2010 event shown by Figure 2, corresponded approximately to a 2-year return interval flow, as shown by Figure 3. Examination of this peak water surface elevation, which ranged from 23.8 at Stage 1 (upstream), to 21.5 at Stage 2 (downstream), and comparison with typical floodplain elevations through the potential area of the levee setback alignments (CMA 2 and 4), indicated that under a levee setback condition, the exposed floodplain would be inundated to a depth of approximately 2 to 6 feet. This has significant implications for the proposed design of floodplain enhancement and ecosystem mitigation of the setback area.

Water level data collected through this and ongoing efforts will be valuable for calibration and validation of future hydrodynamic modeling, particularly the 2-dimensional modeling that will be developed for ecological and geomorphic design purposes.

Velocity and Flow Measurements

cbec staff measured flows in the Sacramento River using an Acoustic Doppler Current Profiler (ADCP) (RDI RiverRay 600 kHz) at two transects (RM 58.6, 50.4) during ecologically and geomorphically significant flow events in December and January 2010, using methods prescribed by the USGS (see appendix - Mueller and Wagner 2009). Figure 1 displays the locations where the measurements were made. The results of these flow measurements are summarized in Table 1. ADCP velocity cross section contour plots are shown by Figure 4, 5 and 6. Velocities in these sections ranged from approximately 0.2 to 6.5 ft/s during the events measured. cbec staff were unable to conduct measurements at both locations on 12/23/11 due to the large quantity of floating debris (wheat stubble) which clogged the field vessel motor intake.

Flow and velocity collected through this and ongoing efforts will be valuable for calibration and validation of future hydrodynamic and sediment transport modeling, particularly the 2-dimensional modeling that will be developed for ecological and geomorphic design purposes.

Sediment Load Measurements

cbec staff measured bed and suspended sediment loads in the Sacramento River at two transects through the project reach (RM 58.6, 50.4) during ecologically and geomorphically significant flow events using methods prescribed by the USGS (Edwards and Glysson 1999). Figure 1 displays the locations where the measurements were made. Figure 7 shows photographs of the equipment used to measure bed and suspended sediment loads.

Bed load measurements were taken using a Helley Smith Model 8035 bedload sampler. 20 bedload samples were taken at each transect on 12/23/11 and 01/5/11 per USGS guidelines. Samples were analyzed for total dry weight and the particle size distribution by Blackburn Consulting. For additional information on sampling techniques and the method for calculating bedload please refer to the appendix (Edwards and Glysson 1999).

Depth integrated suspended sediment samples were taken using a DH 2A sampler. 10 samples were taken at each transect per USGS guidelines (Edwards and Glysson 1999). Samples were analyzed by Cooper Laboratories for total suspended solids (TSS – mg/L). Using the average concentration of suspend sediment (TSS – mg/L) and the measured discharge at each transect, an instantaneous load of suspended sediment was calculated. A daily load (tons/day) was calculated assuming the measured discharge would be sustained for a 24 hour period.

The results of the bed and suspended load measurements are summarized in Table 1. The particle size distributions of the bedload measurements are included in Figures 8, 9 and 10. cbec staff were unable to conduct measurements at both locations during on 12/23/11 due to the large quantity of floating debris (wheat stubble) which clogged the field vessel motor intake.

Table 1: Summary of measured discharge (Q), suspended load (Qss) and bedload (Qbl).

Date	River Mile (RM)	Q (cfs)	RI (cfs)	Min Velocity (ft/s)	Max Velocity (ft/s)	Q_{bl} (tons/day)	TSS (mg/L)	Q_{ss} (tons/day)
12/23/2010	58.6	73,670	2-yr	0.2	6.4	6,977	84	16,618
1/5/2011	50.4	58,275	1.5	0.2	6.2	1,501	51	7,990
1/5/2011	58.6	57,847	1.5	0.1	5.6	1,748	49	7,682

Bed and suspended sediment loads collected through this and ongoing efforts will be valuable for calibration and validation of future hydrodynamic and sediment transport modeling, particularly the 2-dimensional modeling that will be developed for ecological and geomorphic design purposes.

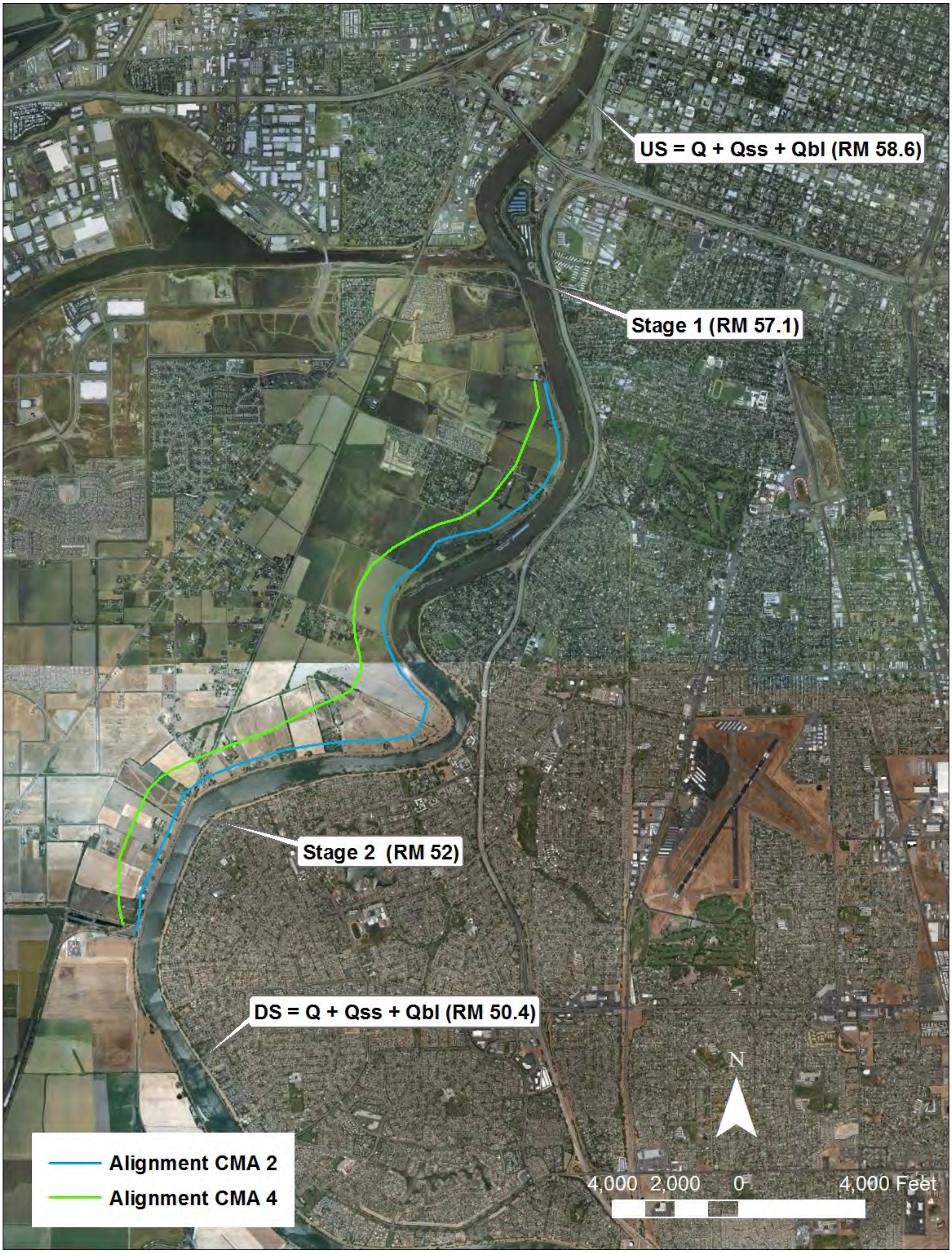
Conclusions and Recommendations

The results of the field data collection effort initiated through this task indicate that high quality data were collected that will be valuable for future project feasibility and design stages. We highly recommend that the water level (stage) recorders continue to be deployed through project construction and monitoring. In addition, further ADCP measurements should be taken in the main channel of the Sacramento River to provide additional baseline data, and on the floodplain after construction of the levee setback alignment (if it is implemented) for verification purposes. Finally, the bed and suspended sediment load measurements taken through this effort represent just an initial "snapshot in time" and additional data should be collected through the 2011-12 winter season for project design purposes.

References

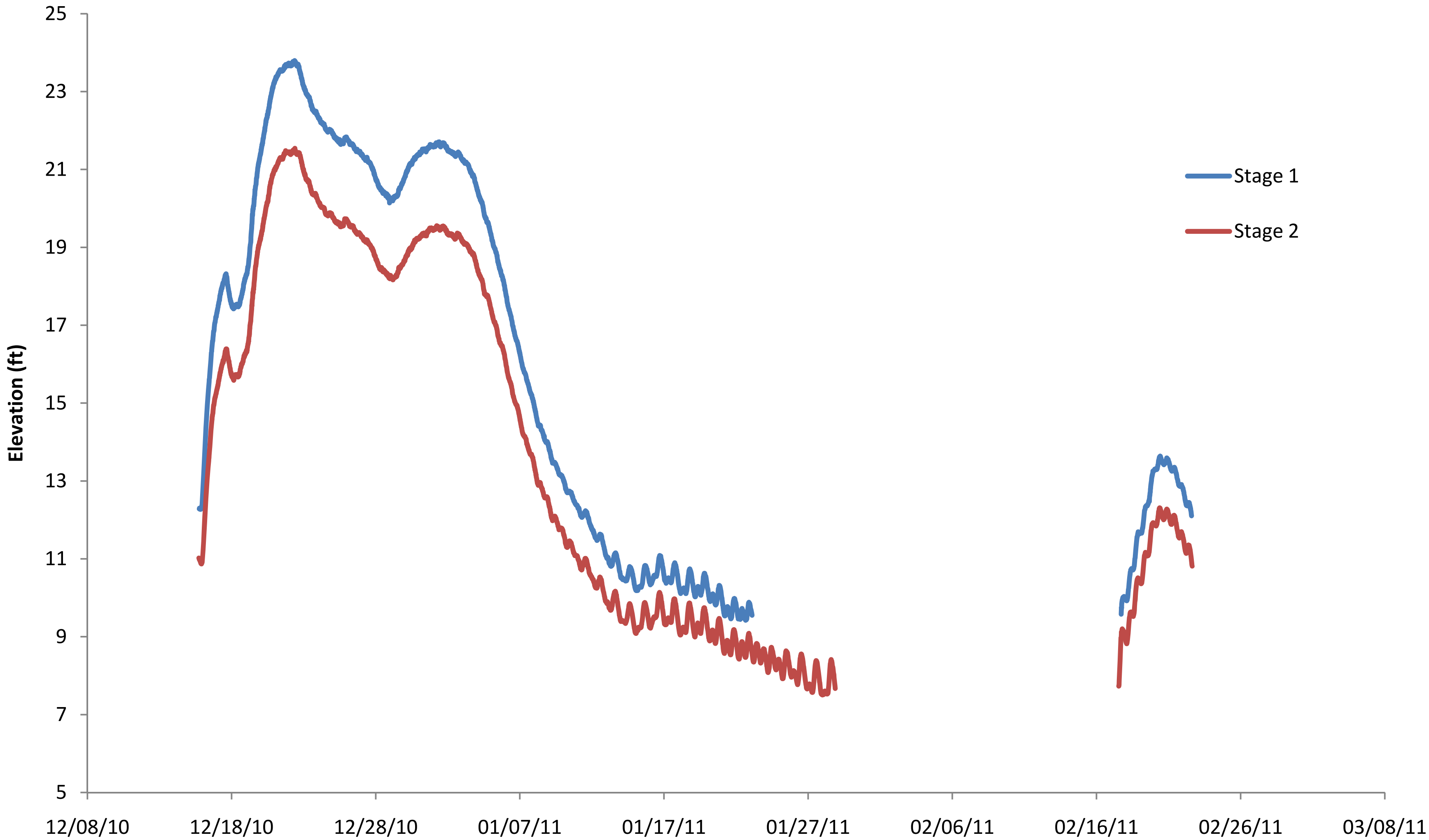
Edwards T.K. and Glysson G.D., 1999. Field methods for measurement of fluvial sediment**.** USGS—TWRI Book 3, Chapter C2. 1999.

Mueller, D.S., and Wagner, C.R., 2009, Measuring discharge with acoustic Doppler current profilers from a moving boat: U.S. Geological Survey Techniques and Methods 3A–22, 72 p.



Notes: Q = Discharge (cfs) Qss = Suspended Load Qbl = Bedload	cbec eco engineering	*West Sacramento Southport EIP Task Order 1* **Monitoring Locations**

Project No. 10-1043	Created By: JS	**Figure 1**



Notes: Elevation references the NAVD 88 vertical datum. Missing data is a result of the water level dropping below the elevation of stage recorder.

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West Sacramento Southport EIP Task Order 1

Measured Stage within Project Reach

Project No. 10-1043	Created By: JS	**Figure 2**

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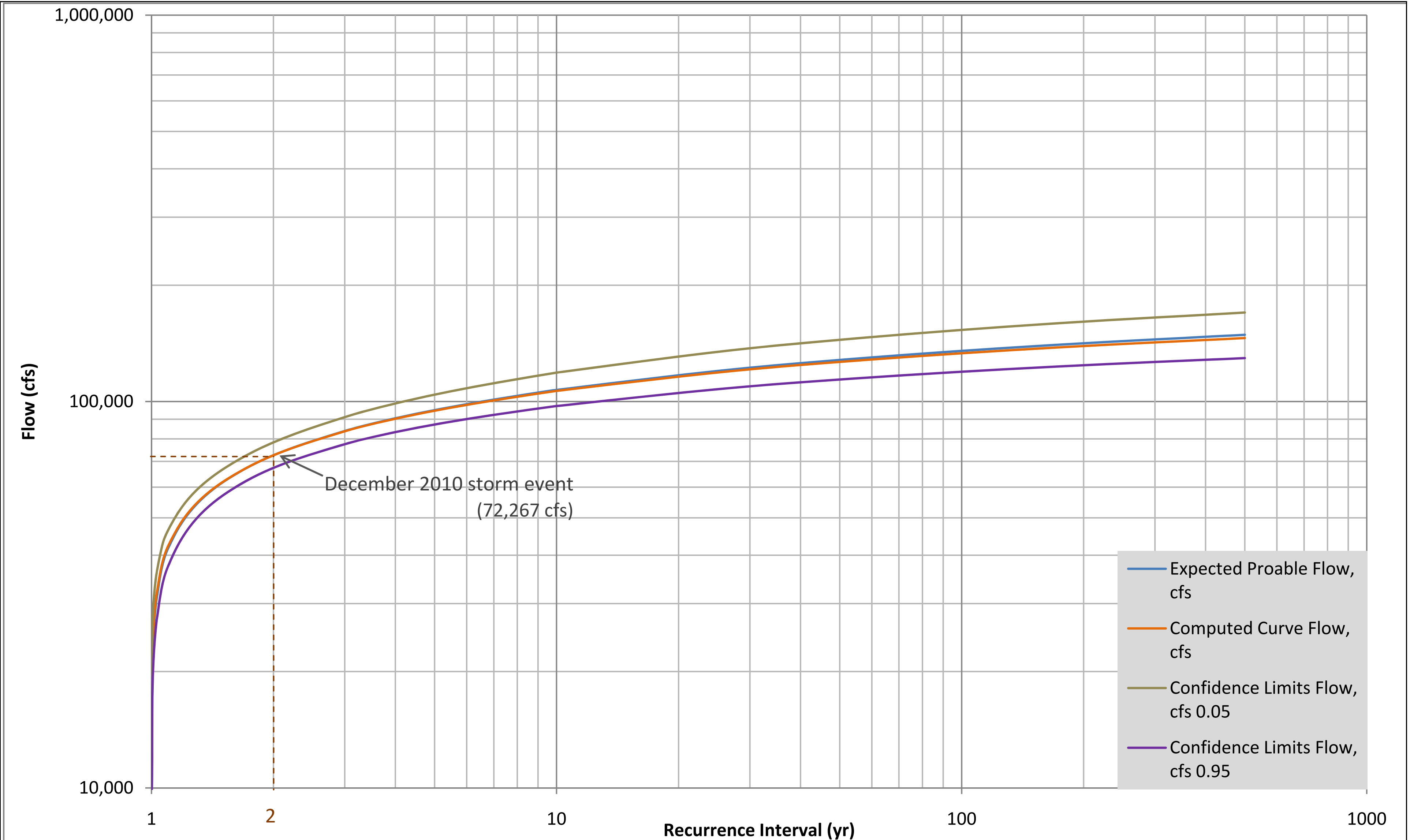
5/2/2011



Notes: Flood frequency analysis for a 2-year event yields an expected probable flow of 72,592cfs. Peak discharge data for December storm event (12/21/2010) was 72,267cfs. Datum NAVD 88

Historic and Current Preliminary Geomorphic Assessment - Southport EIP

December 2010 event stage data

Project No. 11-1003	Created By: LA	**Figure 3**

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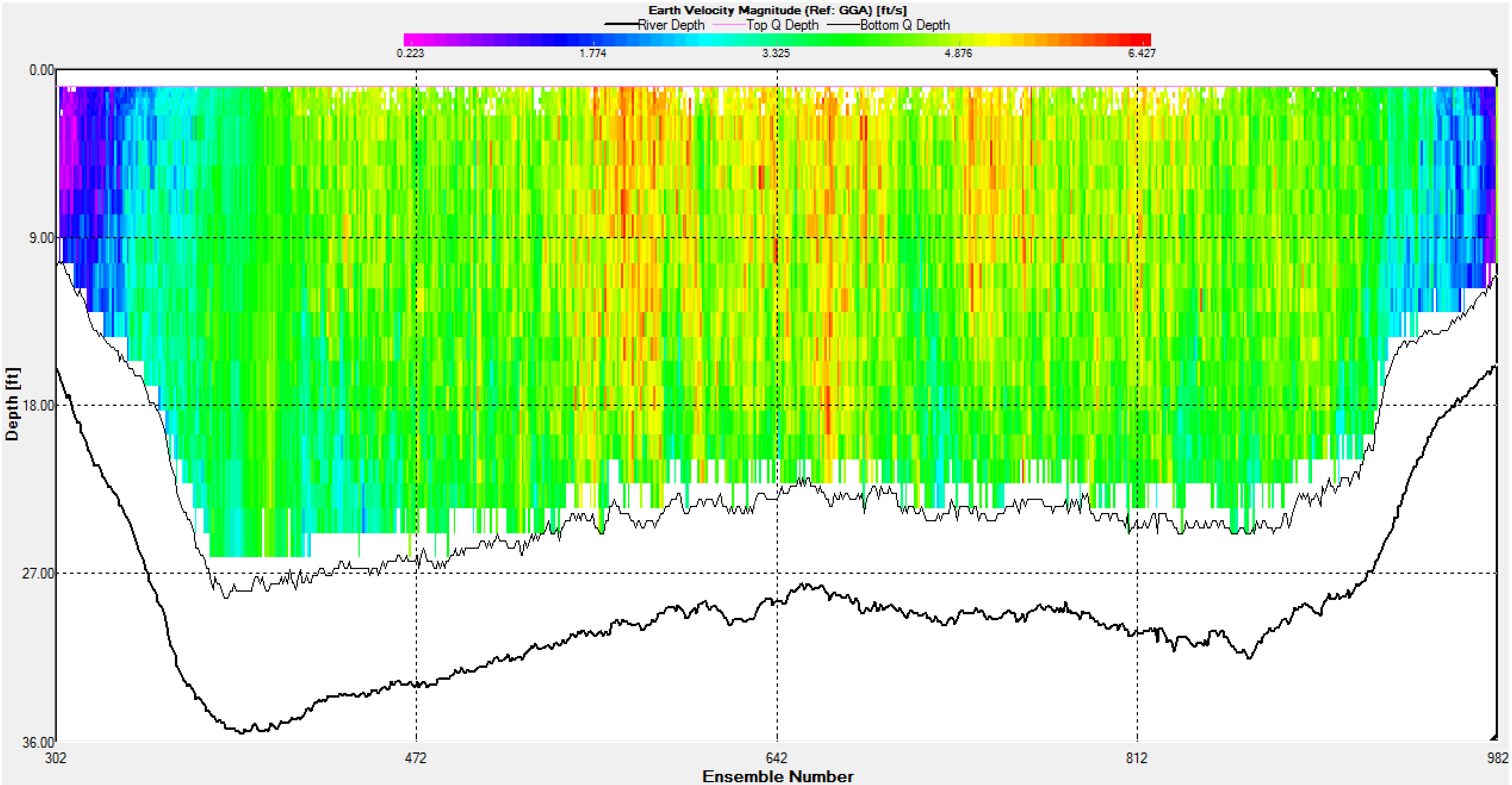
5/2/2011



Notes: Measurement taken at upstream monitoring site.

West Sacramento Southport EIP Task Order 1

ADCP Velocity Magnitude - 12/23/11

Project No. 10-1043	Created By: JS	**Figure 4**

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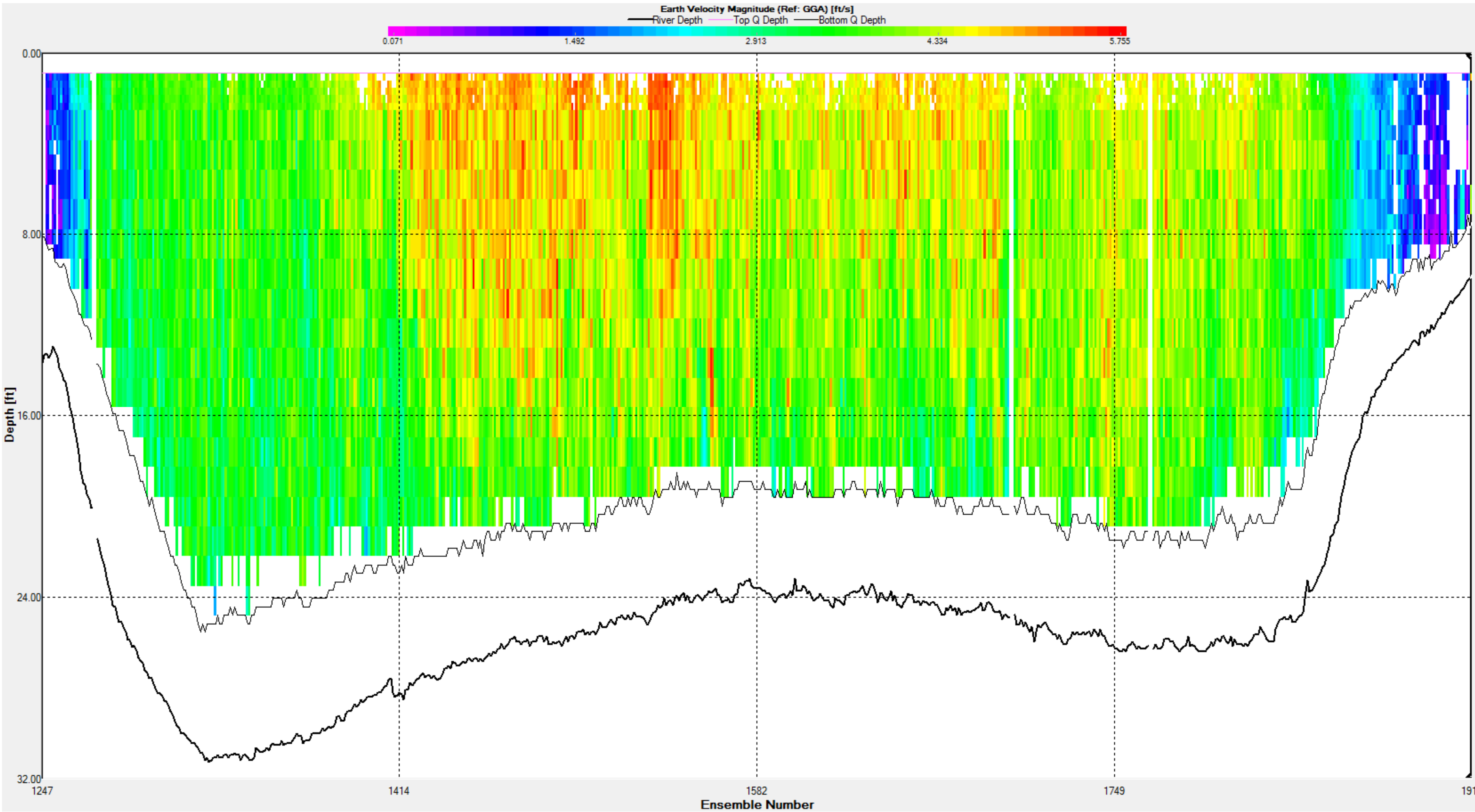
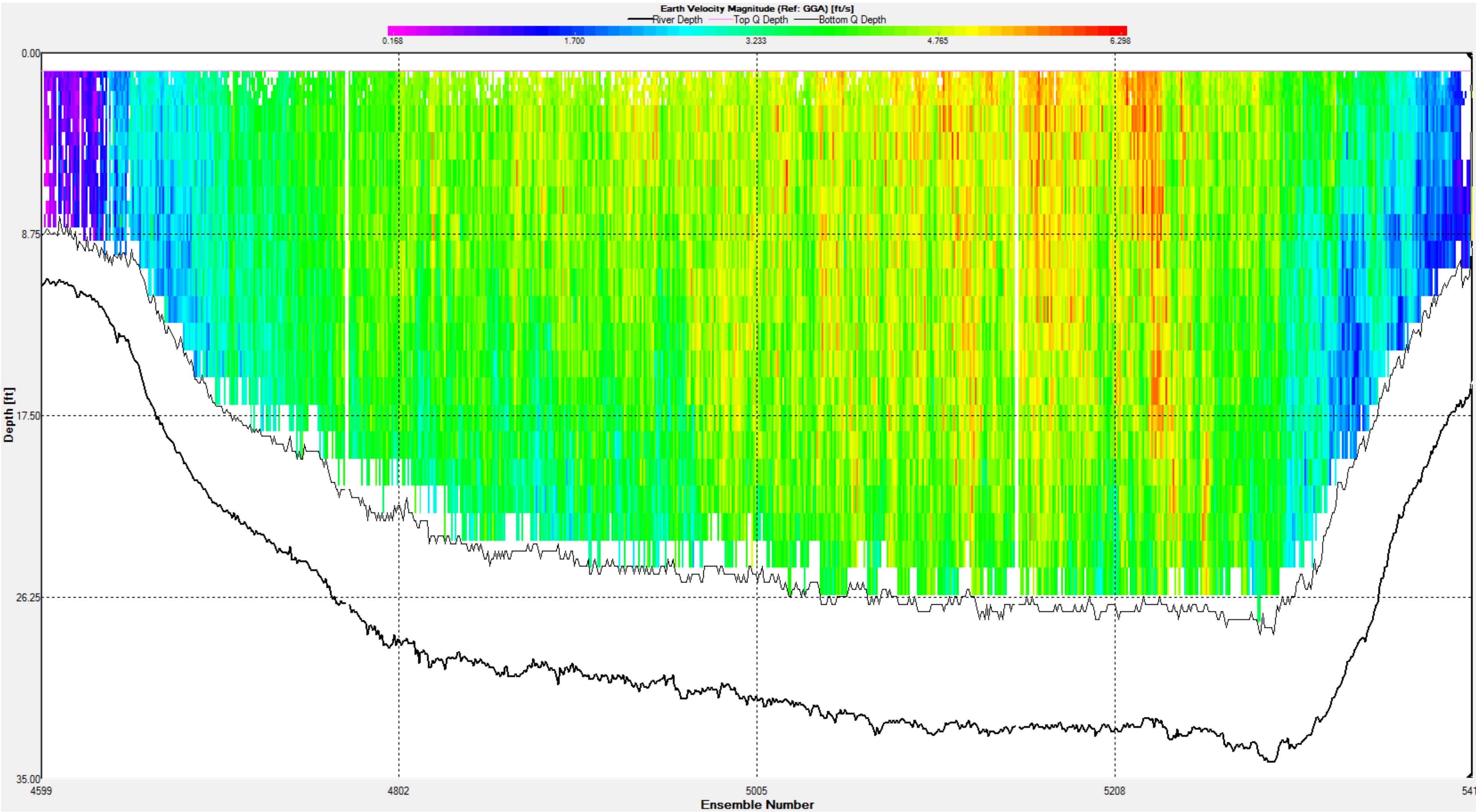
5/2/2011



Notes: Measurement taken at upstream monitoring site.	cbec eco engineering	*West Sacramento Southport EIP Task Order 1* **ADCP Velocity Magnitude - 01/05/11**
		Project No. 10-1043 | Created By: JS | **Figure 5**



Notes: Measurement taken at downstream monitoring site.	cbec eco engineering	*West Sacramento Southport EIP Task Order 1* **ADCP Velocity Magnitude - 01/05/11**		
		Project No. 10-1043	Created By: JS	**Figure 6**

R:\Projects\10-1043_West_Sac_Levee\Reporting\TO1_Data_Collection_Memo\Figs\Fig6_ADCP_DS_010511.docx
5/2/2011

Notes: Above: DH 2A suspended sediment sampler. Below: Helly Smith bedload sampler	cbec eco engineering	*West Sacramento Southport EIP Task Order 1* **Bedload and Suspended Load Sampling Equipment**		
		Project No. 10-1043	Created By: JS	**Figure 7**

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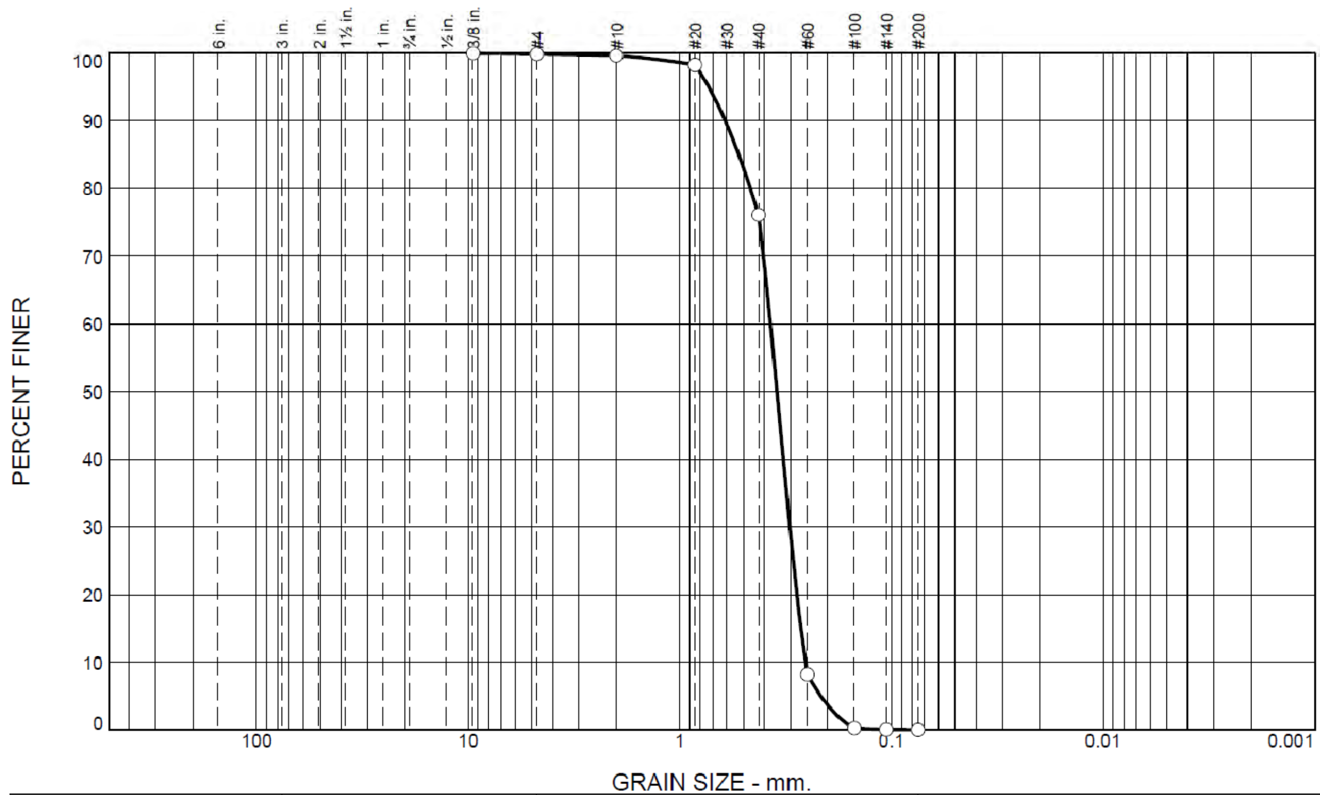
5/2/2011

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.1	0.3	23.5	76.1	0.0	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/8"	100.0		
#4	99.9		
#10	99.6		
#20	98.3		
#40	76.1		
#60	8.2		
#100	0.3		
#140	0.1		
#200	0.0		

* (no specification provided)

Soil Description

Poorly-graded SAND, olive brown

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 0.6103 D_{85}= 0.5275 D_{60}= 0.3735
D_{50}= 0.3481 D_{30}= 0.3028 D_{15}= 0.2684
D_{10}= 0.2553 C_u= 1.46 C_c= 0.96

Classification

USCS= SP AASHTO=

Remarks

Shells and organics on No. 4 sieve
Organics, some coarse sand on No. 10 sieve
Some organics, sand on No. 20 sieve

Sample Number: DS-BL

Date: 2-15-11

Notes: Sample collected at downstream monitoring site on 1/5/11.

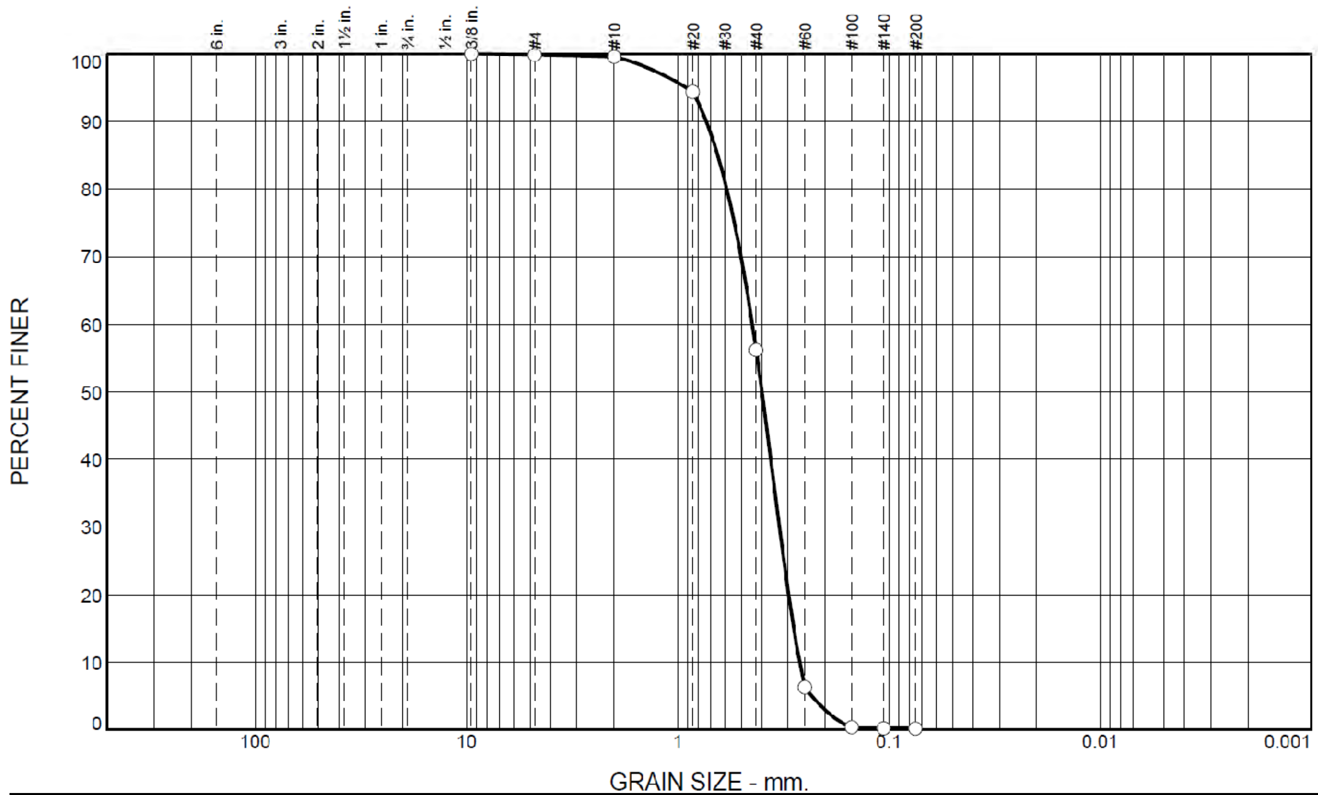
cbec eco engineering

West Sacramento Southport EIP Task Order 1

Particle Size Distribution – Bedload

Project No. 10-1043 | Created By: JS | **Figure 8**

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.2	0.3	43.3	56.0	0.2	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/8"	100.0		
#4	99.8		
#10	99.5		
#20	94.4		
#40	56.2		
#60	6.3		
#100	0.4		
#140	0.3		
#200	0.2		

* (no specification provided)

Soil Description

Poorly-graded SAND, olive brown

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 0.7317 D_{85}= 0.6462 D_{60}= 0.4430
D_{50}= 0.3991 D_{30}= 0.3301 D_{15}= 0.2824
D_{10}= 0.2648 C_u= 1.67 C_c= 0.93

Classification

USCS= SP AASHTO=

Remarks

Sample Number: US-BL

Date: 2-15-11

Notes: Sample collected at upstream monitoring site on 12/23/10.

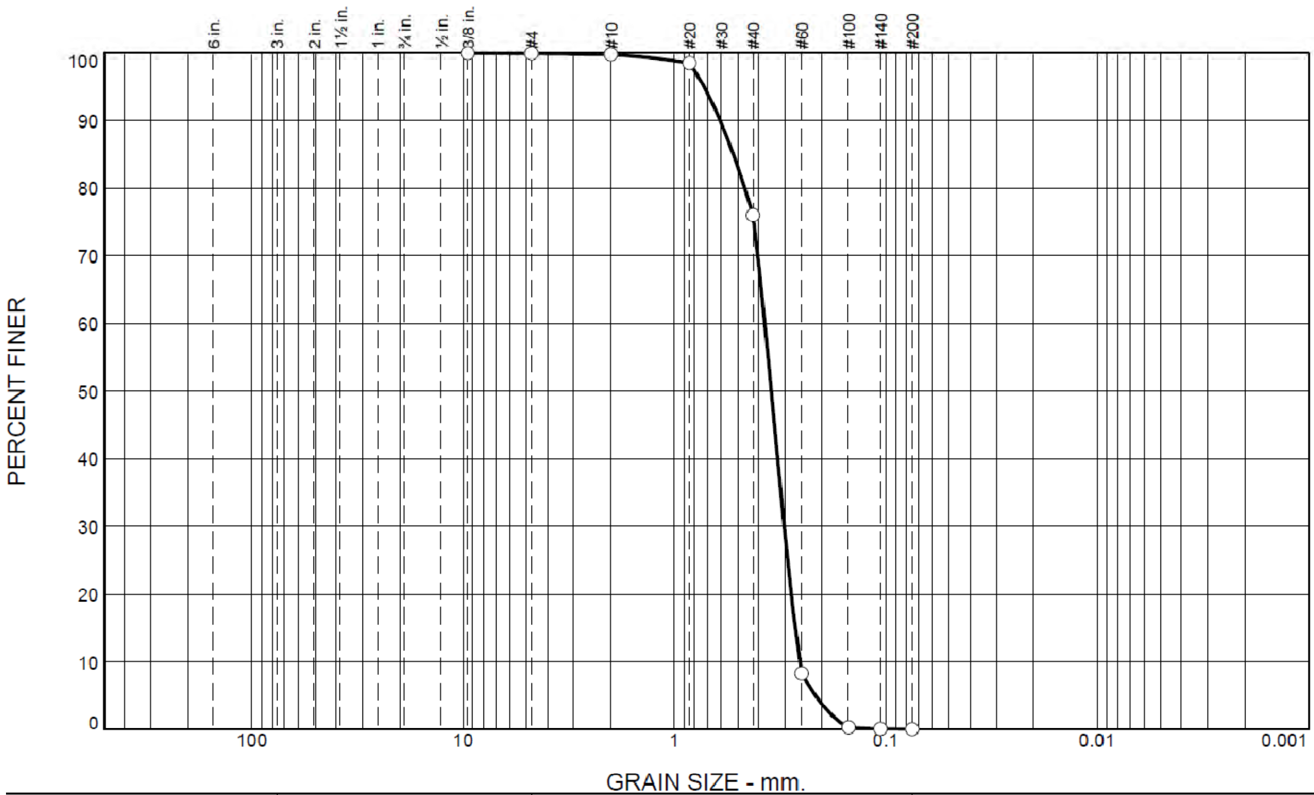
cbec eco engineering

West Sacramento Southport EIP Task Order 1

Particle Size Distribution – Bedload

Project No. 10-1043 | Created By: JS | **Figure 9**

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.0	0.2	23.8	75.9	0.1	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
.375 in.	100.0		
#4	100.0		
#10	99.8		
#20	98.5		
#40	76.0		
#60	8.3		
#100	0.3		
#140	0.1		
#200	0.1		

* (no specification provided)

Soil Description

Poorly-graded SAND, olive brown

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 0.6072 D_{85}= 0.5262 D_{60}= 0.3736
D_{50}= 0.3481 D_{30}= 0.3026 D_{15}= 0.2681
D_{10}= 0.2549 C_u= 1.47 C_c= 0.96

Classification

USCS= SP AASHTO=

Remarks

All organics (0.22g) retained on the No. 4 sieve

Sample Number: US-BL2

Date: 2-15-11

Notes: Sample collected at upstream monitoring site on 1/5/10.

cbec eco engineering

West Sacramento Southport EIP Task Order 1

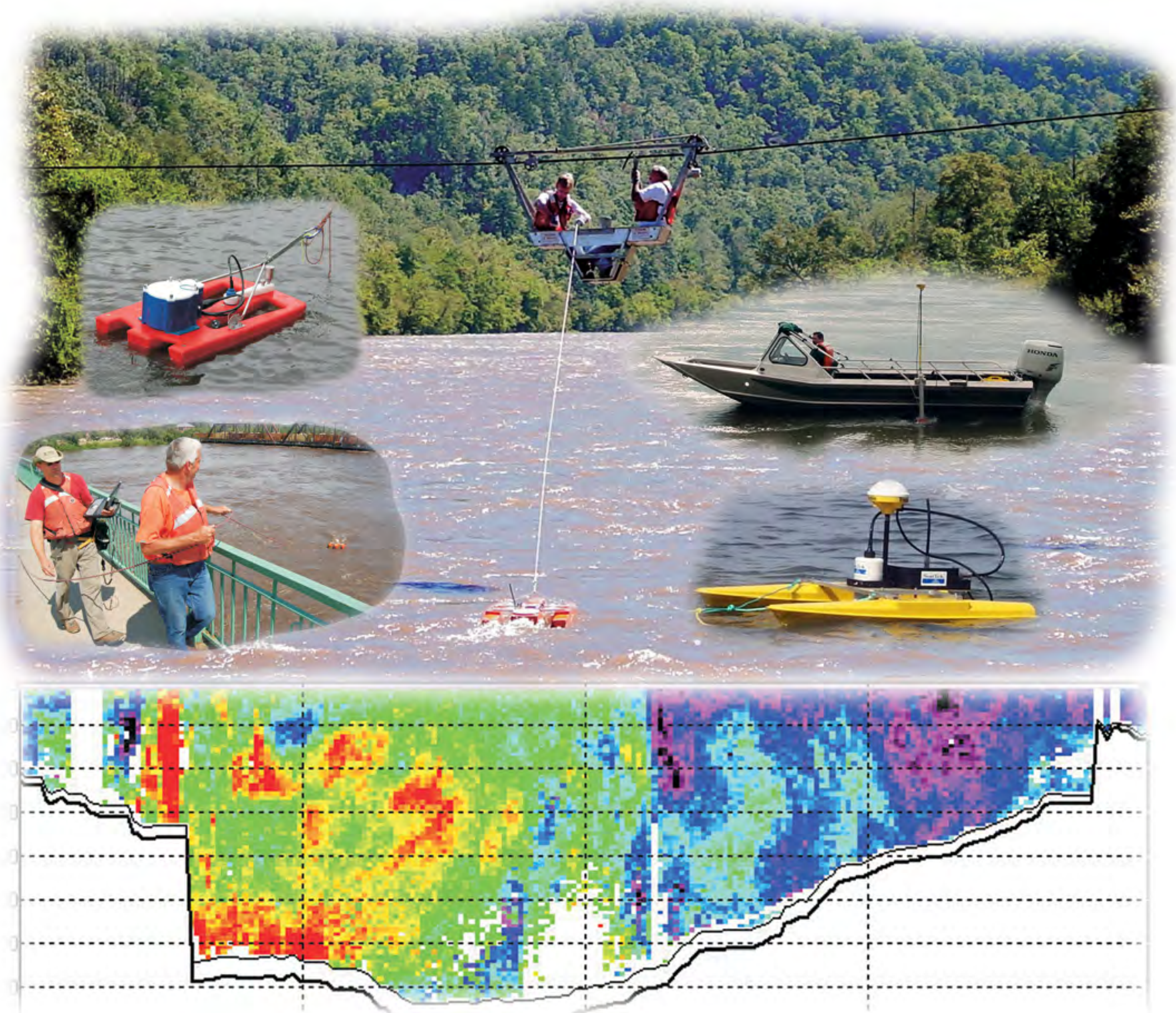
Particle Size Distribution – Bedload

Project No. 10-1043 | Created By: JS | **Figure 10**

science for a changing world

Measuring Discharge with Acoustic Doppler Current Profilers from a Moving Boat

Chapter 22 of Book 3, Section A



Techniques and Methods 3–A22

U.S. Department of the Interior
U.S. Geological Survey

Measuring Discharge with Acoustic Doppler Current Profilers from a Moving Boat

By David S. Mueller and Chad R. Wagner

Chapter 22 of Book 3, Section A

Techniques and Methods 3–A22

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manned boat, or a manned boat where little movement of the boat was ensured, a moving bed is determined to be present when the measured moving-bed velocity is greater than 1 percent of the mean water velocity at the test location. If the moving-bed test was conducted using a manned boat that was not anchored and may have moved either upstream or downstream, a criteria of 2 percent instead of 1 percent is used because uncertainty has been introduced into the test by the boat's movement. Discharge-measurement techniques that are not affected by a moving bed, or that correct for the effect of a moving bed, should be used if a moving bed has been detected (Appendix B).

A more accurate method for estimating the errors introduced by a moving bed can be determined if a GPS is available for use and is interfaced with the ADCP and the data-collection software. This second method also requires that the ADCP boat be held in a stationary position and a data file recorded for at least 5 minutes, if quality GPS data are being recorded. The error caused by the moving bed can be computed in the same manner as previously described for the first method, except that the distance in the upstream direction indicated by bottom tracking should be corrected by the distance actually traveled in that direction, as indicated by GPS (Oberg and others, 2005). In the WinRiver software, this distance can be found in the "compass calibration" tabular window and is labeled "BMG-GMG mag," and the direction of the "BMG-GMG dir" should be in the upstream direction. If the measured moving-bed velocity is greater than 1 percent of the mean water velocity at the test location, discharge-measurement techniques that are not affected by a moving bed, or that correct for the effect of a moving bed, should be used (Appendix B).

If the ADCP can be held stationary, stationary moving-bed tests are a good measure of the magnitude of an apparent moving streambed; however, these tests represent moving-bed conditions for only one location in the cross section. An alternative to the stationary moving-bed test is the loop method, which is based on the fact that as an ADCP is moved across the stream, a moving bed will cause the bottom-track-based ship track to be distorted in the upstream direction. Therefore, if an ADCP makes a two-way crossing of a stream (loop) with a moving bed and returns to the exact starting position, the bottom-track-based ship track will show that the ADCP appears to have returned to a position upstream from the original starting position (fig. 10). The mean moving-bed velocity can be computed from the distance the ADCP appeared to have moved upstream from the starting position (loop-closure error) and the time required to complete the loop. If the moving-bed velocity measured by the loop method is greater than 0.04 ft/s and greater than 1 percent of the mean water velocity, a moving bed is present. Discharge-measurement techniques that are not affected by a moving bed, or that correct for the effect of a moving bed, should be used if a moving bed has been detected (Appendix B). The loop method must be applied properly, or it may produce incorrect results. Anyone planning to use the loop method should read and follow USGS Scientific Investigations Report 2006–5079 (Mueller and Wagner, 2006), which describes the procedures, limitations, and uncertainties associated with the loop method. A detailed description of the loop method also is presented in Appendix B.

Discharge-Measurement Procedures

The procedures to be followed to make quality discharge measurements vary depending on the flow conditions being measured. The procedures for measuring steady-flow conditions are different from the procedures used to measure unsteady-flow conditions. Although the procedures may be different for the various flow conditions, the data-quality indicators for both conditions are consistent. The following sections provide details on the recommended procedures for measuring discharge in steady- and unsteady-flow conditions as well as data-quality problems to monitor in the field when making discharge measurements.

Steady-Flow Conditions

A discharge measurement in steady-flow conditions is obtained from the measurement of a minimum of four transects (two in each direction). The measured discharge is the average of the discharges from the four transects. If the discharge for any of the four transects differs by more than 5 percent from the mean measured discharge and no critical data-quality problem can be identified and documented, a minimum of four additional transects should be obtained, and the mean of all eight transects will be the measured discharge (U.S. Geological Survey, 2002b). If the discharge for one or more transects is not within 5 percent of the mean measured discharge and a critical data-quality problem can be identified

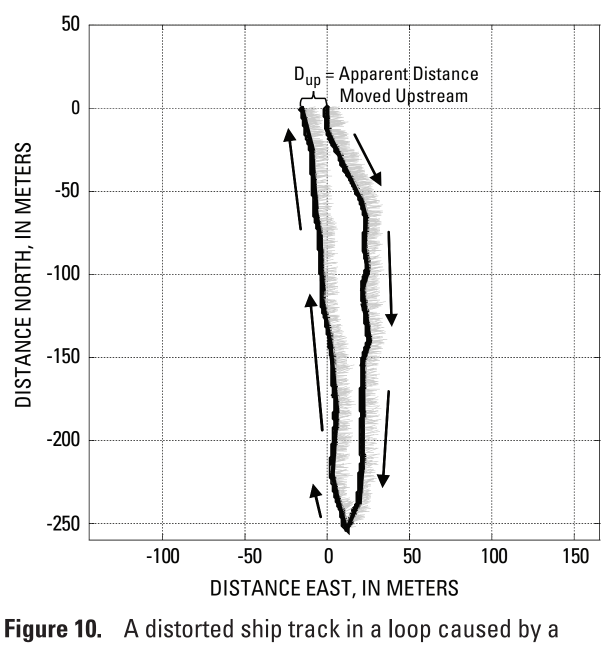


Figure 10. A distorted ship track in a loop caused by a moving bed.

and documented (for example, a tow boat approaching the section, a sudden change in discharge because of a lockage, communication problems between the computer and ADCP, or other factors), the transect deviating from the mean may be replaced with an additional transect collected in the same direction. Reciprocal transects should always be measured to reduce potential directional biases. Directional biases occur when the discharges measured for transects from the left bank to the right bank are consistently either greater than or less than discharges measured for transects made from the right bank to the left bank.

When the mean channel velocity is less than 0.8 ft/s, the TRDI StreamPro ADCP discharge measurements for individual transects have much greater variability than those StreamPro measurements made when the mean channel velocity is greater than 0.8 ft/s. Discharge measurements made when mean velocities were less than 0.8 ft/s had an average coefficient of variation for individual transect discharges of 12 percent, whereas measurements with mean velocities greater than 0.8 ft/s had an average coefficient of variation of 2.5 percent. Despite this larger variation, the measured discharges (the mean discharge for all transects) do not seem to be biased, provided that enough transects (potentially more than eight) are included in the mean discharge. When using a StreamPro ADCP in these slow conditions, a slow, steady boat speed is critical, and water mode 13 (WM13) should be used if the site conditions meet the criteria for maximum water speed (less than 0.8 ft/s) and depth (less than 3.3 ft). Additional details on the StreamPro ADCP testing results can be found in OSW Technical Memorandum 2005.05 (U.S. Geological Survey, 2005b).

Unsteady-Flow Conditions

At times, flow changes rapidly enough that discharge measurements within 5 percent of the mean cannot be collected from four transects. Unsteady flows can be caused by upstream dam or lock regulation, tidal effects, downstream backwater effects, flood waves, or other conditions. It may be necessary to use measurements from individual transects as discrete measurements of discharge if the flow is changing rapidly. If possible, however, pairs of reciprocal transects should be averaged together as one measurement of discharge to reduce the potential of directional bias (U.S. Geological Survey, 2002b). The justification for using a single transect or pairs of transects for discharge measurements should be documented in the field notes and stored with the discharge measurement or applicable station analysis files. Another consideration for unsteady flows, specifically bi-directional flows, is the assignment of a positive or negative sign to the discharge measurement. The ADCP software may or may not assign flow direction correctly, and the positive or negative sign also can change depending on which edge is designated "left" or "right." Thus, the operator should note the direction of flow during measurement for each transect, according to accepted convention for a particular site.

Critical Data-Quality Problems

When making ADCP discharge measurements, the ADCP operator should continuously monitor the data through the ADCP software. If a critical data-quality problem is observed during measurement at a transect, the use of that transect may be terminated. If a transect is not used, the reason should be documented on the ADCP discharge-measurement field note form, and that transect should not be used in the computation of measurement discharge. If the problem was related to undesirable measurement-section characteristics, a new measurement section should be located and noted on the measurement field note form. If the terminated transect was not the first transect in a measurement series, the boat should be returned to the initial starting point to ensure the transects are measured in reciprocal pairs (Oberg and others, 2005). Potential critical data-quality problems can include, but are not limited to the following:

a. inappropriate or improperly configured water or bottom mode;

b. configuration errors, such as an insufficient number of depth cells to profile down to the channel bed;

c. appreciable or consistent portion of the section with invalid or missing data (invalid data failed to meet internal and user-specified data-quality criteria, and missing data are a result of communication problems between the computer and the ADCP);

d. appreciable invalid bottom tracking;

e. erroneous boat or water velocities, such as ambiguity errors (Appendix A);

f. excessive boat speed;

g. poor GPS data attributed to multipath, satellite changes, or high dilution of precision (DOP);

h. excessive pitch-and-roll or erratic motion of boat and ADCP; and,

i. inadvertent early termination of the transect.

Boat Operation

Average boat speed during each transect normally should be less than or equal to the average water speed. At some sites, it may be necessary to move the boat across the channel using a non-ferrous tag line in order to meet this requirement. Other methods for moving the boat slow enough to be equal to or less than the water speed include the use of push poles, paddles, low-speed trolling motors, or tethered boats, which can be moved slowly across the channel when deployed from a hand-operated cableway or a bridge. In certain conditions, it may not be possible to keep the boat speed less than the water speed. If it is not practical or safe to keep the boat speed less than or equal to the average water speed, additional transects may be measured to obtain a good average discharge. The reason that the boat speed was higher than the average water speed should be documented on the ADCP discharge-measurement field note form. Ongoing research (Oberg and Mueller, 2007a) indicates that the number of transects and the boat speed are not as important as the cumulative time in which data are collected and averaged. A cumulative time for data collection of at least 720 seconds should result in a good mean discharge in steady-flow conditions. When using GPS, keeping the boat speed as low as practical is especially important because errors in the compass readings are additive and increase with boat speed. Rapid course changes should be avoided; the key element in boat operation during the measurement is to do everything slowly and smoothly. Simpson (2002) discusses proper boat operation for ADCP measurements in detail, and his remarks on boat operation should be heeded (Simpson, 2002, p. 122):

> "Be a smooth operator! The BB [broadband]-ADCP discharge-measurement system will give more consistent results if rapid movements and course changes are kept to a minimum. Smooth boat motion is more important than a straight-line course."

Estimating Edge Discharge

Because depths will eventually get too shallow for valid data collection as the ADCP approaches a bank, it is necessary to estimate discharge in the near-shore unmeasured zones using the ADCP discharge-measurement software. To ensure the accuracy of near-shore discharge estimates, the distances from the edge of water to the starting and stopping points of each transect must be measured using a distance-measurement device (such as a laser or optical rangefinder), tagline, or some other accurate measurement device. Placing marker buoys at the start and end points of transects is advantageous for keeping consistent edges. Use of marker buoys enhances the data collection by ensuring more consistent edge estimates and by measuring in approximately the same section for all passes. When measuring in channels with vertical walls at the edges, start and stop points for transects should be no closer to the wall than the depth of water at the wall to prevent acoustic interference from the main beam or side lobes impinging on the wall. For example, if the depth at a vertical wall is 10 ft, transects should start or stop at least 10 ft away from the wall. In order to obtain an accurate mean velocity for estimating the discharge in the near-shore zones, the boat should be kept nearly stationary from 5 to 10 seconds at the beginning and end of each transect. Accurate edge-discharge estimates also require the ADCP operator to select the correct edge-shape coefficient for the type of edge (sloping or vertical). The edge shapes should be recorded in the ADCP discharge-measurement notes (Oberg and others, 2005).

When using a tethered boat, special methods are required to measure edge distances. Distance marks on the bridge handrail or guardrail may be used to measure edge distances (fig. 11). If the tethered boat is too far away from the bridge to accurately use distance marks for measuring edge distances, laser rangefinders having a compass, an inclinometer, and a "missing-line mode" capability may be used. Missing-line mode calculates a horizontal distance between two points, given a range, heading, and vertical angle measured for each point. Edge distance may be measured by selecting the shore and the transect start or end point while using this mode (Rehmel and others, 2002).

When using a remote-controlled boat at some sites, edge distances may be measured using the same techniques as with tethered boats. At other sites where edge distances cannot be measured using these techniques, it may be necessary to have someone in line with the measurement section to measure the distance from the near-shore edge of water to the starting point and the distance from the ending point to the edge of water on the far shore.

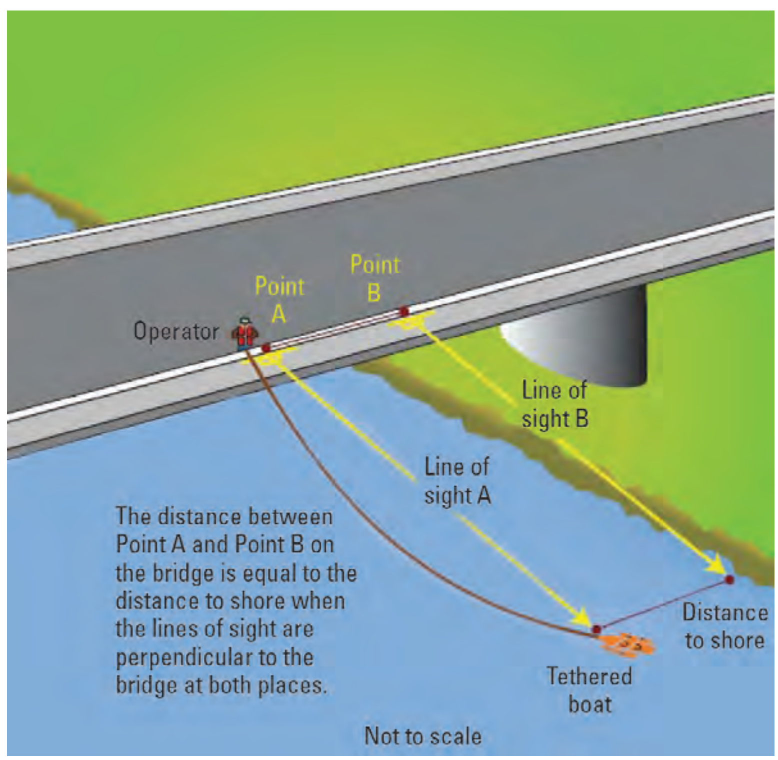


Figure 11. Edge distances needed when using a tethered acoustic Doppler current profiler boat for discharge measurements (modified from Environment Canada, 2004).

Field Notes

All information on an ADCP measurement field note form should be filled out during the course of the measurement. The ADCP operator should note any conditions that potentially could affect the measurement, including estimated wind speed and direction, bi-directional or unusual flow patterns, excessive waves, or passing boats. Use of an ADCP does not negate long-standing, agency guidelines and policies regarding measurement documentation, such as recording reference gage heights before, after, and, if needed, during the discharge measurement. An example of a completed USGS ADCP discharge-measurement field note form is shown in figure 12 (Oberg and others, 2005).

Step-by-Step Procedure

The steps for making a discharge measurement using an ADCP are not complex, but each step must be completed to ensure quality data. To assist the field hydrographer, quick-reference guides that detail the step-by-step procedure for making ADCP discharge measurements, along with other pertinent information, are presented in Appendix E, figures E-3–E-6. These guides can be printed, laminated, and kept with the ADCP for reference.

Post-Measurement Field Procedures

An assessment of the discharge measurement should be made after completion of the transects composing the measurement. A thorough review of all measurement data may not be practical in the field, but a cursory review of the measurement should be made in order to assign a preliminary quality rating to the measurement and to ensure that specific transects do not have critical data-quality problems. If all data were collected at the same measurement section, the transect widths and discharges in the measured (middle) and unmeasured (top, bottom, and edge) sections should be consistent. If transect widths or discharges are not consistent with those of the other transects, the transect data should be scrutinized to determine if a critical data-quality problem occurred (examples of critical data-quality problems are listed in the Discharge-Measurement Procedures section of this report). If a critical data-quality problem is identified, the data from the affected transect should not be used in the computation of discharge. A new transect should be measured, starting from the same side as the discarded transect, if flow conditions have remained steady. If the flow has changed, a new transect series should be collected. A minimum of four transects should be measured if the flow is stable when the new discharge data are collected. A transect should be discarded only if a critical data-quality problem is identified and documented on the field note form. Site-specific conditions, such as turbulence, eddies, reverse flows, surface waves, moving bed, high sediment concentration, and proximity of the instrument to ferrous objects, should be noted under the appropriate sections on the ADCP measurement field note form and used in assigning a quality rating for the measurement (Lipscomb, 1995).

If the discharge measurement was collected at a site with a rating curve, the measured discharge should be plotted on the rating curve for that streamgaging station, and the percentage of difference from the stage-discharge rating should be computed. Rantz and others (1982, p. 346) state: *"If the discharge measurement does not check a defined segment of the rating curve by 5 percent or less, or if the discharge measurement does not check the trend of departures shown by recent measurements, the hydrographer is normally expected to make a second discharge measurement to check his original measurement."* Rantz (1982, p. 346–347) then describes procedures for making check discharge measurements with mechanical current meters. For ADCPs, power off all equipment and begin with step 1c in figure E-3 of Appendix E and proceed through the remainder of the procedures. If practical, choose a new measurement cross section for the check measurement. The measured discharge from the check measurement should then be plotted on the rating curve, and the percentage of difference from the discharge rating should be computed in the field.

Immediately after completion of a measurement, all files, including raw data files, configuration files, instrument test files, compass calibration files, and any electronic measurement forms, should be backed up on nonvolatile media, such as CD-ROMs, flash-memory cards, or USB drives, and stored separately from the field computer. The purpose of this backup is to preserve the data in the event of loss or failure of the field computer.

The ADCP should be dried after use and stored in its protective case for transport. When working in estuaries and other saltwater environments, the ADCP must be rinsed off with freshwater and dried prior to storing for transport. Failure to dry the ADCP may result in corrosion of the ADCP connectors, mounting brackets, and any ADCP accessories stored in the protective case (Oberg and others, 2005).

Office Procedures

Upon returning to the office from field data collection, routine maintenance of equipment should be completed, all data files and notes should be stored properly, data should be reviewed, and measurements should be finalized and archived. Adherence to these procedures will ensure the equipment is ready for the next deployment and that data are reviewed and processed in the most efficient manner.

Preventive Maintenance

The ADCP and associated accessories, such as GPS, vertical depth sounders, and electronic rangefinders, should be inspected upon returning from the field to determine their

Acoustic Profiler Discharge Measurement Notes							Filename Prefix: foxmon_ds1200_
Left Bank:	Sloping Vertical Other:__________					Right Bank:	Sloping Vertical Other:__________
Transect No.	Starting			Ending		Total Discharge	Notes
	Bank	Time	Distance	Distance	Time		
0	L R						Moving bed test in center of channel
1	L R	1249	16	69	1255	1,321	Simultaneous comparison discharge meas.
2	L R	1256	69	16	1301	1,358	Upstream of dam
3	L R	1301	69				Transect aborted due to debris in river
4	L R	1303	69	16	1308	1,327	
5	L R	1309	16	69	1315	1,356	
	L R						
	L R						
	L R						
	L R						
	L R						
	L R						
	L R						

Notes: All times are CST. Measurement was made using a temporary rope-and-pulley cableway. Edge distances were measured with laser rangefinder by marking the start and ending positions on the rope and measuring the distance from edge of water to the center of the tethered boat.

9-275-I	10/24/08	U.S. Department of The Interior U.S. Geological Survey	Meas. No.	57
Station Number			Processed by	BLL
05551540		ADCP Discharge Measurement Notes	Checked by	KAO

Station Name: Fox River at Montgomery, IL
Date: July 6, 2004 Party: B. L. Loving, S.E. Anderson

Width	Area / Rated Area	Velocity	Index Vel.	Gage Height	Discharge
235	707	1.9		11.74	1,350

Gage Height Change	Meas. plots	From rating	Shift	ADCP Sync'd to WT
0.00 in 0.4 hrs.	1.1 % diff	No.: 4	0	Y at 1207 or N

ADCP Mfr / Model / Frequency	Serial No.	Firmware	Software
TRDI/Rio Grande/1200	1636	10.14	WinRiver 10.06

Boat/Motors Used	GPS Used	ADCP Depth	Diag. Test / Errors?
OceanScience Tethered	Trimble AgGPS	0.27 ft	☒ Y or N

Compass Calib. & Total Error	Mag. Var	MagVar Method	Moving Bed?
Y or N	-2.4	On-site Model Previous	Y or N

Meas. Water Temp	ADCP Water Temp	Weather / Air Temp	Wind Speed / Dir.
26.5 °F C at 1210	26 °F C at 1210	Sunny & clear 83 F C	Southerly @ 5-10

Gage Readings							Site Conditions	
Time			ETG	CR10	Inside	Outside	Max Water Depth	10 f
1100			11.74	11.70		11.70	Max Water Speed	2.5fps
1230				11.70		±0.05	Max Boat Speed	1 fps
1249	(s)			11.70			Water Mode	12
1300				11.70			Bottom Mode	5
1315	(f)			11.70			Streambed material	
							Gravel	
1400			11.74	11.70		11.70	Salinity	
						±0.05	-- ppt at --	
Weighted MGH			11.74				Checkbar found	22.41
GH corrections							Checkbar changed to:	
Correct MGH			11.74				at	

Wading cable ice, boat, upstr., downstr., side bridge 1,500 ft mi. upstr. downstr. of gage
Measurement rated: excellent (2%) good (5%), fair (8%), poor (>8%) based on following conditions
Flow: Steady & uniform. Flow at edges appears to be moving in ds direction.
Cross section: Sand and gravel with some mud
Control: Dam is clear of debris
Gage operating: Y or N Record removed: Y or N Filename: Telephone telemetry
Battery voltage 12.5 V Intakes/Orifice cleaned/purged: No
Bubble-gage psi: Tank -- Line -- Bubble rate -- / min
Extreme-GH indicators: Max -- Min -- CSG Checked Y or N
HWM on stick none Ref elev. 12.65 HWM elevation none
GH of zero flow = GH -- - depth at control -- = --ft, Rated= 10.48
Sheet No. 1 of 1 sheets

Figure 12. Example of completed acoustic Doppler current profiler discharge-measurement field note form (Oberg and others, 2005).

condition. Deployment platforms and mounts also should be inspected. Damage or undue wear to any instrument components, deployment platforms, or mounts should be corrected as soon as possible. The ADCP, all accessories, platforms, mounts, and field computers should be prepared for redeployment and stored in an appropriate location. All batteries should be recharged immediately to facilitate rapid reuse.

Data Storage

All measurement data should be moved from the field computer or field backup media to a permanent storage location for archival and backup. Field computers used to collect ADCP data should have local area network (LAN) capability to facilitate the process of transferring the measurement data to an office server.

Measurement Review Procedures

Discharge measurements should be reviewed in detail by the person who made the measurements as soon as practical after completion of ADCP field measurements. ADCP discharge measurements should be routinely checked by someone other than the person who made the measurement, in accordance to specific agency policies.

Important aspects of reviewing ADCP discharge measurements both in the office and in the field as soon as the data are collected are listed below.

1. The discharge-measurement field note forms should be complete, understandable, and legible.
2. All electronic data files associated with the measurement should be backed up in the field and archived on an office server.
3. The number of transects measured should be appropriate for the flow conditions and satisfy agency policy. Transects should be measured in reciprocal pairs.
4. Configuration files should be checked for errors, appropriateness for the hydrologic conditions, and consistency with field notes. ADCP depth, salinity, edge distances, edge shapes, extrapolation methods, and ADCP configuration parameters listed on the field notes should match those in the configuration file.
5. The temperature measured by the ADCP thermistor should be reasonable for the site and time of year and match the water temperature measured and noted on the field form. Speed-of-sound calculations that are not corrected for temperature can cause velocity-measurement errors and depth errors as great as 7 percent. An error in temperature caused by a faulty ADCP thermistor results in an erroneous calculation of water density and introduces uncertainty into the speed-of-sound calculations (Simpson, 2002).
6. The salinity of the water at the measurement site should be measured and noted on the field form and entered into the ADCP software for use in the speed-of-sound calculations. If the hydrographer has entered an incorrect salinity value or has forgotten to enter the proper value, depths and velocities will be calculated incorrectly. Errors in excess of 3 percent can be caused by speed-of-sound calculations that are not corrected for salinity (Simpson, 2002).
7. A moving-bed test using proper technique should be performed prior to the discharge measurement, recorded, archived, and noted on the ADCP measurement field note forms. If a moving bed was detected, GPS should be used. If GPS was not used, the measured discharges should be adjusted for the moving bed (Appendix B).
8. The average boat speed for the measurement should not have exceeded the average water speed unless it was impractical or unsafe to do so. The reason for any exceedance should be documented in the field notes or station file. Boat pitch-and-roll should not be excessive. Excessive boat speed or pitch-and-roll may justify downgrading the measurement quality.
9. The measured edge distances recorded on the ADCP measurement field form should match those electronically logged with each transect. The correct edge shape should be selected and 5–10 seconds of data collected at transect stop and start points while the boat is held stationary. If subsectioning was used to correct problems with edges, then the reason for subsectioning should be clearly documented on the field forms. If a vertical wall is present, then the start and end points for the transect should be located such that the distance from the wall is equivalent to or greater than the water depth at the wall.
10. The number of missing or invalid ensembles should not be excessive. (An ensemble is a single profile of the water velocity through the water column consisting of one or the mean of multiple pings.) The number of missing or invalid ensembles that will result in a poor measurement is difficult to establish because the location and clustering of the missing or invalid ensembles is important. If 50 percent of the ensembles were missing or invalid, but every other ensemble was valid, the measurement could still be a good measurement. However, if 10 percent of the ensembles were missing or invalid, but they all occurred in one location where the neighboring valid data would be a poor representation of what was unmeasured, the measurement would be poor. When the missing or invalid ensembles always occur in

in the cross section. The time required to collect a proper sample can vary from 5 seconds or less to several hours or more. Generally, a sampling time that does not exceed 60 seconds is preferred. Because of the temporal variations in bedload transport rates, there is no easy way to determine the appropriate sampling time. Several test samples (as many as 10 or more collected sequentially at a vertical with a suspected high transport rate) may be needed in order to estimate the proper sampling-time interval to be used. The sample time should be short enough to allow for the collection of a sample from the section with the highest transport rate, without filling the sample bag more than about 40 percent full. The sample bag may be filled to 40 percent full with sediment coarser than the mesh size of the bag without reducing the hydraulic efficiency of the sampler (Druffel and others, 1976). Sediment that is approximately equal to the mesh size may clog the bag and cause a change in the sampling efficiency of the sampler.

(3) One sample should be collected at each vertical, starting at one bank and proceeding to the other. It is recommended that, during this initial data gathering stage, a minimum of one transect using the SEWI method be used. The samples should be placed in separate bags for individual analysis and labeled with the vertical's station number. They may be composited into one or several sample bags for a composite analysis, but if composited, no information on cross-sectional variability can be obtained from the data.

(4) A second sample should be collected using the UWI or MEWI methods. Four or five verticals should be sampled four or five times each, obtaining a total of 20 samples. Samples should be collected using the same procedure as described in number 2 above, except that the sample time for each sample need not be the same. All samples should be bagged and tagged for separate analysis.

(5) The following data must be recorded on a field note sheet for each cross-section sample:

Station name/number
Date
Cross-section sample starting and ending times
Gage height at the start and end of sample collection
Total width of the cross section, including stations on both banks
Width between verticals (SEWI method)
Number of verticals sampled (SEWI method)
Station of verticals sampled (UWI or MEWI method)
Time sampler was on the bottom at each vertical
Type sampler used
Name of person collecting sample

In addition, the following information should be recorded on each sample container:

Station name
Date
Designation of cross-section sample to which the container belongs (that is, if two cross-section samples were collected, one would be "A" and the other "B")
Number of containers for that cross section (for example, "1 of 2" or "2 of 2")
Stations(s) of the vertical(s) the sample was collected from
Time sampler was on the bottom and at the vertical station
Clock time the sample was collected (start and finish if composite)
Collector's initials

Analysis of the first transect (SEWI method) will give some indication of the cross-sectional variability if individual verticals are analyzed separately. Analysis of the second set of transects (UWI or MEWI method) will give some indication of temporal variability. As stated before, the procedure described above should be considered the minimum to be followed when first collecting bedload data at a site. Additional samples and transects will help define the temporal and spatial variation at the site for all flow ranges. After a cross section has been sampled several times at different flow ranges using the above procedure, it should be possible to develop a sampling protocol that fits the site better.

Computation of Bedload-Discharge Measurements

The bedload transport rate at a sample vertical may be computed by the equation

$$R_i = \frac{KM_i}{t_i} \tag{1}$$

where

R_i = bedload transport rate, as measured by bedload sampler, at vertical i, in tons per day per foot;

M_i = mass of the sample collected at vertical i, in grams;

t_i = time the sampler was on the bottom at vertical i, in seconds; and

K = a conversion factor used to convert grams per second per foot into tons per day per foot. It is computed as

$$K = (86{,}400 \text{ seconds/day}) \frac{1 \text{ ton}}{(907{,}200 \text{ grams})} \frac{1 \text{ foot}}{(N_w)} \qquad (2)$$

where

N_w is the width of sampler nozzle in feet. (For a 3-inch nozzle, $K = 0.381$; for a 6-inch nozzle, $K = 0.190$.)

The cross-sectional bedload discharge measured by the Helley-Smith sampler may be computed using the total cross-section, midsection, or mean-section method. The simplest method of calculating bedload discharge from a sample collected with a Helley-Smith type bedload sampler is the total cross-section method (fig. 54). This method should only be used if the following three conditions are met:

1. The sample times (t_i) at each vertical are equal.

2. The verticals were evenly spaced across the cross section (that is, SEWI or MEWI method used).

3. The first sample was collected at one-half the sample width from the starting bank.

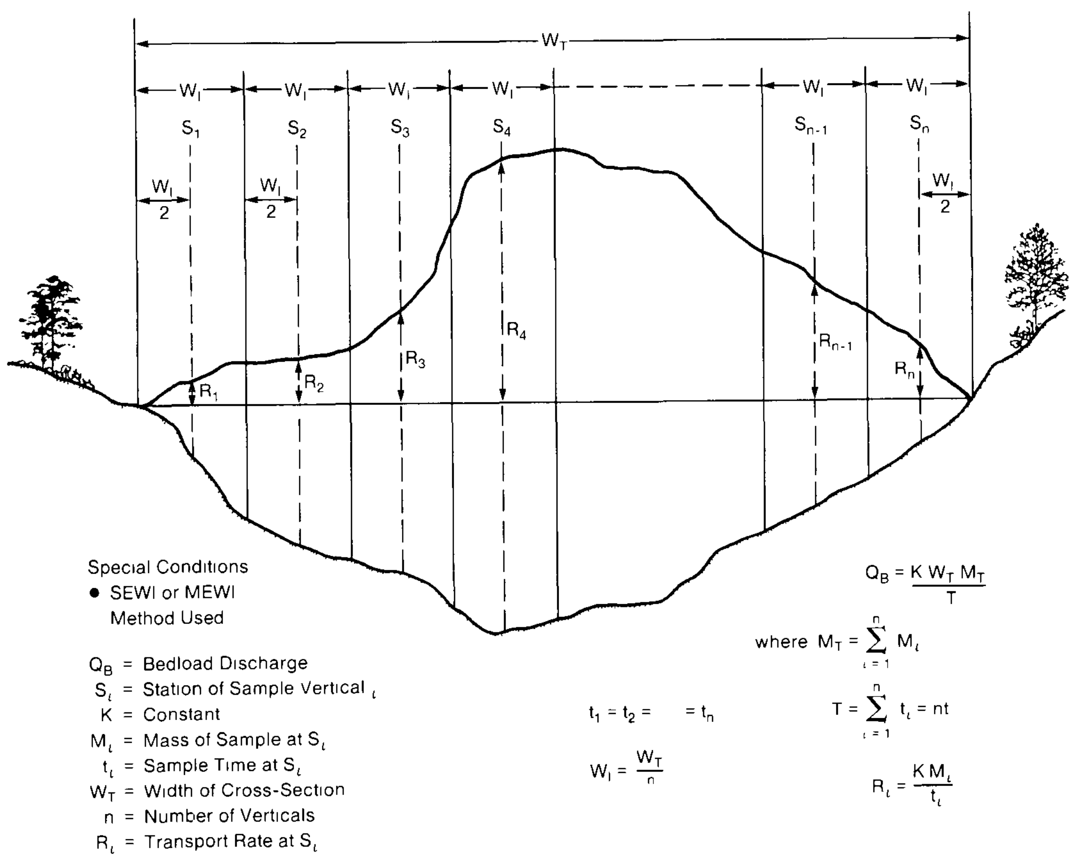


Figure 54. Total cross-section method for computing bedload discharge from samples collected with a Helley-Smith bedload sampler.

If these conditions are met, then

$$Q_B = K\frac{W_T}{t_T}M_T \qquad (3)$$

where

Q_B = bedload discharge, as measured by bedload sampler, in tons per day;

W_T = total width of steam from which samples were collected, in feet, and is equal to the increment width (W_i) times n (n = total number of vertical samples);

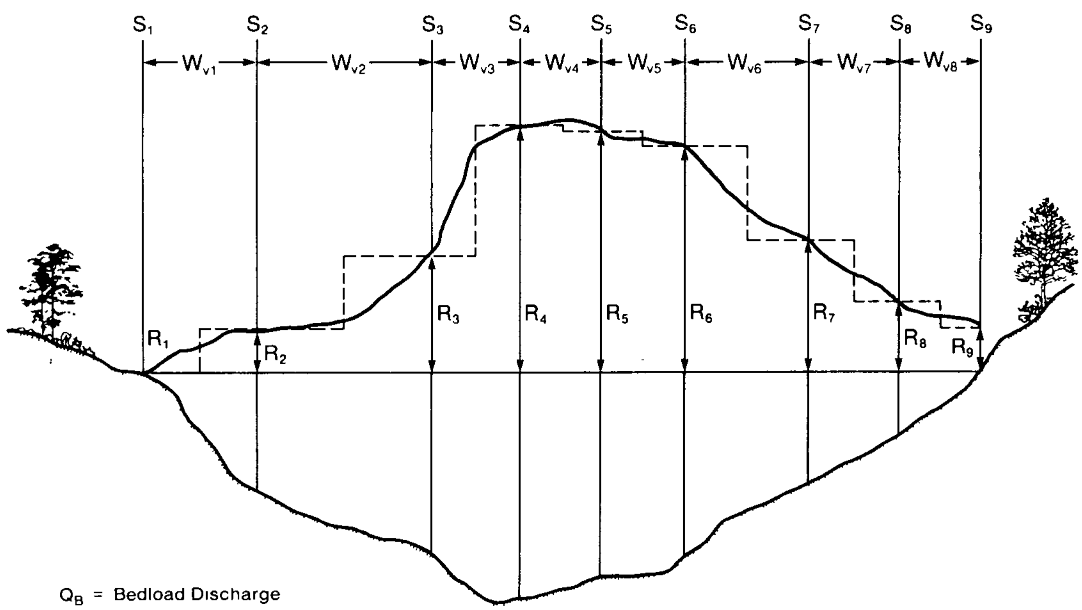
t_T = total time the sampler was on the bed, in seconds, computed by multiplying the individual sample time by n;

M_T = total mass of sample collected from all verticals sampled in the cross section, in grams; and

K = conversion factor as described in equation 2 above.

If any of the three conditions stated above are not met, then either the midsection or mean-section method should be used. Mathematically, the two methods, if used with no modifications, will produce identical answers. However, as indicated under the discussion of the UWI method, the placement of the sampling verticals with respect to breaks in the lateral cross-sectional distribution curve of mean bedload transport rate will somewhat dictate which method should be used. The midsection method (fig. 55) is computed using the following equation:

$$Q_B = \frac{R_1W_1}{2} + \sum_{i=2}^{n-1} R_i\left[\frac{(S_i - S_{i-1})}{2} + \frac{(S_{i+1} - S_i)}{2}\right] + \frac{R_nW_{n-1}}{2} \qquad (4)$$



Q_B = Bedload Discharge
S_i = Station of Sample Vertical $_i$
R_i = Transport Rate at S_i
K = Constant
M_i = Mass of Sample Collected at S_i
t_i = Sample Time at S_i
n = Number of Verticals
W_{vi} = Width Between Verticals $_i$ and $_{i+1}$

$$Q_B = \frac{R_1W_{v1}}{2} + \sum_{i=2}^{n-1} R_i\left[\frac{(S_i - S_{i-1})}{2} + \frac{(S_{i+1} - S_i)}{2}\right] + \frac{R_n W_{vn-1}}{2}$$

$$= \frac{K}{2}\left[\frac{M_1}{t_1}W_{v1} + \frac{M_n}{t_n}W_{vn-1} + \sum_{i=2}^{n-1}\frac{M_i}{t_i}(S_{i+1} - S_{i-1})\right]$$

Figure 55. Midsection method for computing bedload discharge from samples collected with a Helley-Smith bedload sampler.

where

W_i = width between sampling verticals i and $i+1$, in feet;

S_i = stations of the vertical (i) in the cross section measured from some arbitrary starting point, in feet; and

Q_B, n, R, and K have previously been defined,

You will note that equation 3 is very similar to the equation used to compute a surface-water discharge measurement. This method corresponds to the midpoint method currently used to compute surface-water discharge measurements (Buchanan and Somers, 1969). By combining equations 1 and 4 and rearranging terms:

$$Q_B = \frac{K}{2}\left[\frac{M_1W_1}{t_1} + \frac{M_nW_{n-1}}{t_n} + \sum_{i=2}^{n-1}\frac{M_1}{t_i}(S_{i+1}-S_{i-1})\right] \qquad (5)$$

One advantage to using the midsection method is that the distance W_1 need not necessarily be equal to the distance between sampling verticals. At times, it may become apparent, due to local conditions, that a particular R_1 should not be applied over a width equal to halfway back to the last station and halfway forward to the next, but applied to some other width. This width, sometimes referred to as the effective width, is decided on by the user. Bridge piers, large boulders, abrupt changes in velocity or lateral bed topography, or other conditions that may obstruct or cause sudden changes to bedload transport rate will affect the selection of the effective width.

The third method, the mean-section method (fig. 56), is computed using the following equation:

$$Q_B = \sum_{i=1}^{n-1} W_i \frac{(R_i + R_{i+1})}{2}, \qquad (6)$$

which is equivalent to:

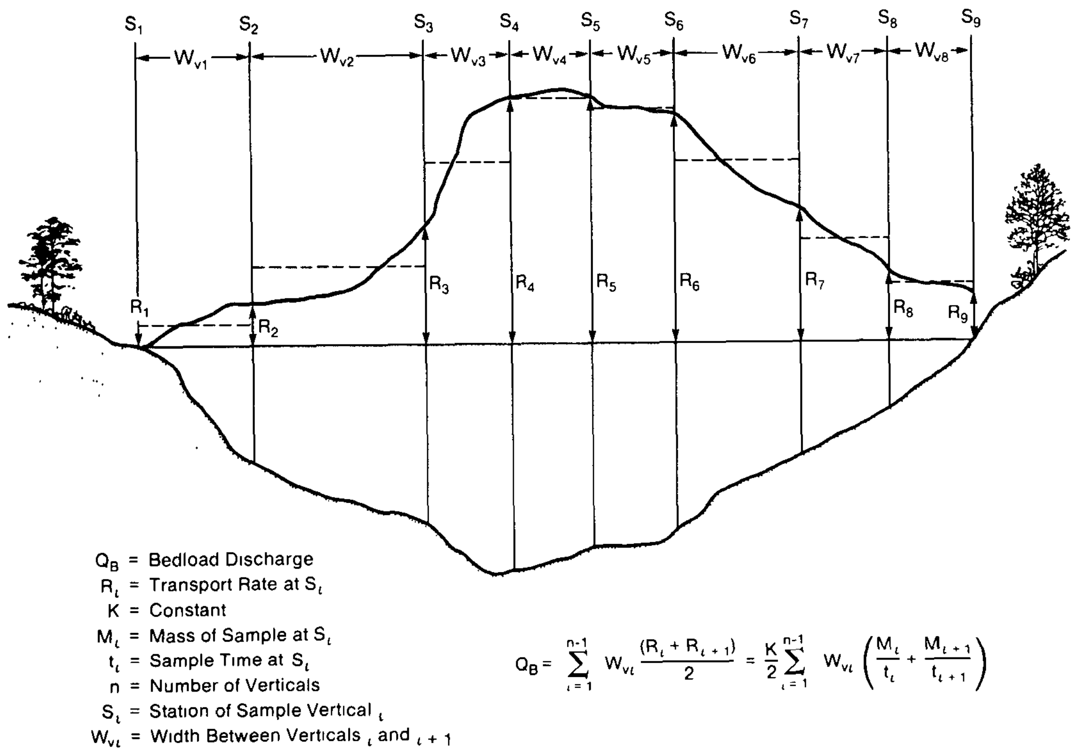


Figure 56. Mean-section method for computing bedload discharge from samples collected with a Helley-Smith bedload sampler.

$$Q_B = \frac{K}{2}\sum_{i=1}^{n-1} W_1\left(\frac{M_i}{t_i} + \frac{M_{i+1}}{t_{i+1}}\right) \tag{7}$$

All the above terms are the same as used in the midsection method. This method averages the two adjoining rates and applies the average rate over the distance between them. For this reason, it is important to try to place the sampling verticals at points where the trends in lateral mean bedload transport rate change. Under most field conditions, this might be difficult.

For situations where the total cross-section method cannot be used, it is recommended that the midsection method be used. This recommendation is made because of its similarity to the surface-water discharge-measurement method, which most field personnel are familiar with, and because of the flexibility in using the effective width concept.

Collecting bedload samples will generate 40 or more samples, creating a potential problem regarding transportation and analyses of so many samples. Carey (1984) adapted a procedure for measuring the submerged weight of bedload samples in the field and converting that measurement to dry weight from a laboratory procedure used by Hubbell and others (1981). The method uses the basic equation

$$W_{ds} = \frac{SG_s}{SG_s - 1} W_{ss} \tag{8}$$

where

W_{ds} = dry weight of the sediment;
SG_s = specific gravity of the sediment; and
W_{ss} = submerged weight of the sediment.

Measurements for Total Sediment Discharge

Total sediment discharge is the mass of all sediment moving past a given cross section in a unit of time. It can be defined as the sum of the (1) measured and unmeasured sediment discharges, (2) suspended-sediment discharge and bedload discharge, or (3) fine-material discharge (sometimes referred to as the washload) and coarse-material or bed-material discharge.

There are some sand-bed streams with sections so turbulent that nearly all sediment particles moving through the reach are in suspension. Sampling the suspended sediment in such sections with a standard suspended-sediment sampler represents very nearly the total load. Several streams with turbulent reaches are described in Benedict and Matejka (1953). Further discussion concerning total-load measurement also can be found in Inter-Agency Report 14 (Federal Inter-Agency Sedimentation Project, 1963b, p. 105–115). Turbulence flumes or special weirs can be used to bring the total load into suspension. Total load can usually be sampled with suspended-sediment samplers to a high degree of accuracy where the streambed consists of an erosion resisting material such as bedrock or a very cohesive clay. In such situations, most, if not all, the sediment being discharged is in suspension (or the bed would contain a deposit of sand).

Benedict and Matejka (1953) and Gonzales and others (1969) have described some structures used for artificial suspension of sediment to enable total-load sampling. However, most total-load sampling is usually accomplished at the crest of a small weir, dam, culvert outlet, or other place where the sampler nozzle integrates throughout the full depth of flow from the surface to the top of the weir.

Where such conditions or structures are not present, the unmeasured load must be computed by various formulas. The unmeasured load can be approximated by use of a bedload formula such as that of Meyer-Peter and Muller (1948), Einstein (1950), Colby and Hembree (1955), or Chang and others (1965). However, these computational procedures can give widely varying answers. The Colby and Hembree (1955) method [modified from Einstein (1950)] determines the total load in terms of the amount transported for different particle-size ranges. Colby and Hubbell (1961) later simplified the modified Einstein method to include the use of four nomographs in lieu of a major computational step. The essential data required for the Colby and Hubbell technique at a particular time and location are listed here:

1. Stream width, average depth, and mean velocity.
2. Average concentration of suspended sediment from depth-integrated samples.
3. Size analyses of the suspended sediment included in the average concentration.
4. Average depth of the verticals where the suspended-sediment samples were collected.

5. Size analyses of the bed material.
6. Water temperature.

Stevens (1985) has developed two computer programs for the computation of total sediment discharge by the modified Einstein procedure. One program is written in FORTRAN 77 for use on the PRIME computer; the other is in BASIC and can be used on most microcomputers.

Hubbell (1964) gives the following formula for determining the total sediment discharge of a given size range from the measured suspended-sediment discharge and the discharge measured with any type of bedload apparatus (see fig. 57).

$$Q_T = \frac{Q_D}{eff} + Q_{sm} + Q_{usm1} - FQ_{sm} + (1 - E/e)Q_{ts2} \quad (9)$$

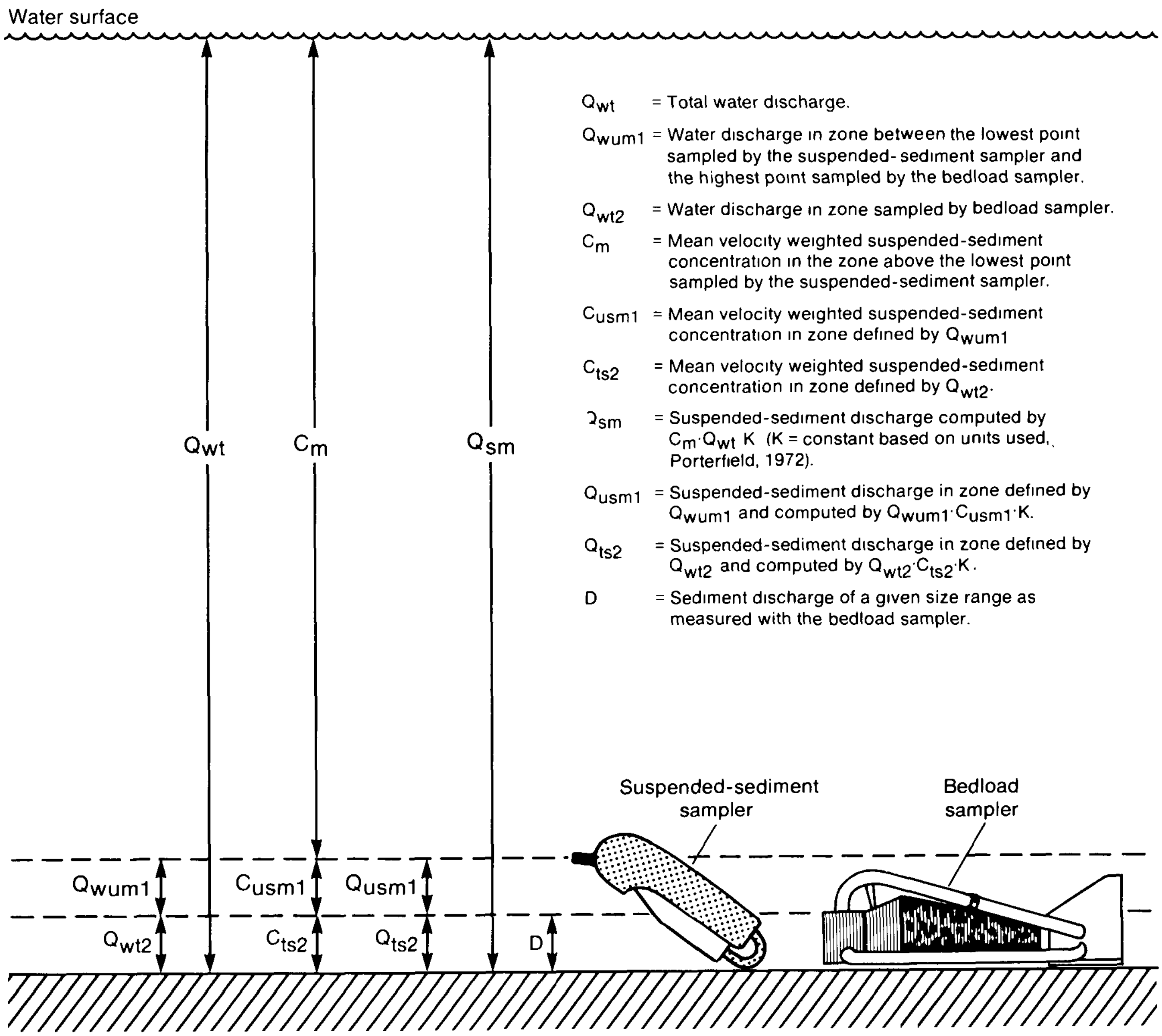


Figure 57. Zones sampled by suspended-sediment and bedload samplers and the unmeasured zone.

Appendix B.8

Technical Memorandum—
Existing Conditions Sediment Transport Assessment—
cbec inc., eco engineering, November 7, 2011

TECHNICAL MEMORANDUM

Date:	11/07/2011
To:	Michael Vecchio (HDR Engineering)
From:	John Stofleth, M.S., EIT, Chris Campbell, M.S., EIT, Chris Bowles, Ph.D., PE
Project:	11-1013 – West Sacramento Southport EIP Task Order 3
Subject:	Existing Conditions Sediment Transport Assessment

INTRODUCTION

cbec inc., eco engineering (cbec), is providing hydrodynamic and geomorphic assessment and design services to the Southport EIP project team. Prior assessments were based on field data collection (Task Order 1), ongoing field data collection efforts, preliminary sediment transport assessment of 1-dimensional (HEC-RAS) modeling results provided by MBK Engineers (MBK), and a historic geomorphic analysis (all Task Order 2). Task Order 3 includes a detailed 2-dimensional hydrodynamic and sediment transport assessment of the existing conditions through the project reach as described in this memorandum. The modeling described herein provides the foundation for detailed hydrodynamic and sediment transport analysis of project alternatives and the preferred design alternative, including levee setback alignments, and corresponding erosion assessments to be conducted under future efforts (partially in Task Order 3 and ultimately in Task Order 4). The modeling will also be used to inform habitat mitigation and enhancement solutions for the setback project.

A series of hydrologic events were simulated using a hydrodynamic and sediment transport model, MIKE 21C. MIKE 21C is a 2–dimensional, curvilinear computation mesh, hydrodynamic model that calculates depth-averaged flow velocity, flow direction, flow depth and energy gradient for elements that represent the channel and floodplain (http://www.mikebydhi.com/). The preliminary results presented in this technical memorandum include existing conditions for the Southport EIP project and are intended to serve an interim deliverable. The ultimate product will include simulations of the preferred design alternative with analysis that will examine the relative impacts on hydraulics and erosion/depositional trends associated with the selected design.

MODEL DEVELOPEMENT

MIKE 21C is a two-dimensional curvilinear grid model developed by DHI Water & Environment (DHI) to simulate changes in river morphology. The hydrodynamic model solves the vertically-integrated equations of continuity and conservation of momentum (the Saint Venant equations) in two directions and includes descriptions for helical flow and vertical velocity profiles. These descriptions are important for simulating the physical processes associated with secondary flow in meandering systems. The morphological model, following calculation of bed material transport (bed load and suspended load), solves the equation for sediment continuity and simulates the development of the river bed due to erosion (bed and bank), deposition, and shoaling.

Two-dimensional models like MIKE 21C are applicable to sediment transport modeling of the Sacramento River. The available formulas in MIKE 21C can successfully describe transport of Sacramento River sediments ranging from fine to medium sands.

MODEL DOMAIN

The study reach, as shown by Figure 1, extends from River Mile[1] 56.7 near William Land Park downstream to RM 51.2 near Garcia Bend on the Sacramento River. However, the upstream boundary of the model was extended upstream of the Pioneer Memorial Bridge to RM 58.9 and the downstream boundary of the model was extended to the Freeport Bridge at RM 46.4 to move the model boundaries away from the study reach and minimize boundary-forcing effects, This corresponds to a 66,000 foot (or 12.5 mile) long reach over which channel and floodplain sedimentation were investigated. cbec developed the MIKE 21C model mesh to capture the complexity of the study reach (see Figure 2), resulting in a highly-detailed grid with typical element dimensions of 3 m wide by 9 m long (or 10 feet wide by 30 feet long). The model domain is represented by 450,260 computation cells.

BATHYMETRY AND TOPOGRAPHY

All bathymetric and topographic data for the project was provided to cbec by HDR. Bathymetric data for the river bed was derived from the California Department of Water Resources (DWR) Urban Levee Evaluation Program. This dataset was collected with multibeam technology in January of 2008. Topographic data for the levees and landward floodplains was derived from West Sacramento LiDAR data collected on February 7, 2007. Both of these datasets reference the horizontal projection of CA State Plane Zone 2 NAD83 feet and the vertical datum of NAVD 88 feet. Both of these datasets were reprojected / converted from feet to meters for use in the MIKE 21C model.

HYDRODYNAMIC BOUNDARY CONDITIONS

Hydrodynamic boundary conditions for the model domain for existing conditions were supplied to cbec by MBK for the 2-, 10-, 25-, 50-, 100-, and 200- year flood events using Feather River at Shanghai Bend centered hydrology derived from the Comprehensive Study (USACE, 2002). The boundary conditions were generated by MBK by routing the Feather River at Shanghai Bend centered hydrology through their HEC-RAS (RAS) model of the Sacramento River Flood Control Project (SRFCP) and extracting the

[1] River Mile (RM) as used in this study is based on stationing derived from the Comprehensive Study (USACE, 2002).

appropriate model output. The peak discharges and water surface elevations for these events are summarized in Table 1.

Table 1. Peak discharge and stage for modeled flood events

Recurrence Interval	Discharge at RM 58.9	Discharge at RM 58.9	Stage at RM 46.4	Stage at RM 46.4
(years)	(cms)	(cfs)	(m, NAVD 88)	(ft, NAVD 88)
2-year	2,335	82,448	6.31	20.7
10-year	2,768	97,762	7.13	23.4
25-year	3,142	110,970	7.9	25.9
50-year	3,236	114,294	8.1	26.6
100-year	3,338	117,876	8.28	27.2
200-year	3,728	131,658	8.87	29.1

HYDRAULIC ROUGHNESS

Existing conditions Manning's roughness coefficients were initially provided to cbec by MBK (see Table 2). MBK calibrated the roughness coefficients to the 100- and 200-year flood events using their RMA2 hydraulic model for the study reach. Initial roughness coefficients used in MIKE 21C ranged from 0.02 for the river bed to 0.05 for levee slopes to 0.1 for piers at boat docks and marinas. Bridge piers were not assigned roughness coefficients for the MIKE 21C model, which differed from the RMA2 model approach, but rather were treated as obstructions (see Figure 3). cbec further adjusted roughness elements associated with piers at boat docks and marinas to a refined scale appropriate for the higher resolution mesh in MIKE 21C. cbec also adjusted (or calibrated) the RMA2 roughness coefficients by globally scaling them for use in the MIKE 21C model such that the steady-state water surface profiles for the 100-year and 2-year flood events computed in MIKE 21C closely matched the 100-year water surface profile computed in RMA2 and observed 2-year water surface elevations.

Table 2. RMA2 Manning's roughness coefficients

Roughness Element	Manning's Roughness Coefficient (n)
Channel Bed	0.020
Bank Slope - Low Density	0.030
Bank Slope - Medium Density	0.035
Bank Slope / Floodplain - High Density	0.040
Bridge Piers / Marinas	0.100

SEDIMENT TRANSPORT THEORY

The Yang total load sediment transport formula was used to simulate sand transport in the Sacramento River. Selection of the Yang formula was based on the guidance provided in Yang and Huang (2001), which is more robust and accurate compared with other available sediment transport formulations. The Yang formula was used to simulate the transport of four (4) representative grain size classes in MIKE

21C. These grain size classes included very fine sand (d_{gm} = 0.09 mm; 0.062 to 0.125 mm), fine sand (d_{gm} = 0.17 mm; 0.125 to 0.25 mm), medium sand (d_{gm} = 0.31 mm; 0.25 to 0.5 mm), and coarse sand (d_{gm} = 0.51 mm; 0.5 to 1 mm). A limited number of size classes were selected based on the fairly uniform distribution of grain sizes within the system. Grain sizes less than 0.062 mm, which are typically considered to be washload and to not interact with the bed, were excluded from the sediment transport analyses.

SEDIMENT BOUNDARY CONDITIONS

Sediment transport boundary conditions for the MIKE 21C model for the 2-, 10-, 25-, 50-, 100-, 200-year flood events were derived from a suspended load rating curve developed from the USGS gage at Freeport (11447650) and from bed load and suspended load data collected during the 2010-2011 storm season (cbec, 2011). Figure 4 shows the suspended load rating curve derived from the Freeport gage as well as the data collected by cbec. Since sediment regimes tend to change over time due to changes in watershed land use and dam operations, only the last 20 years of data from the Freeport gage was selected as the best representation of existing conditions.

As stated prior, grain sizes less than 0.062 mm are considered to be washload and were excluded from the sediment transport calculations. As such, the proportion of the total suspended load attributed to this size class was subtracted from the incoming suspended load. This proportion was determined to be approximately 60 percent of the suspended loaded based on the particle size distribution analysis (see Figure 5) conducted under the 2010-2011 data collection effort (cbec, 2011).

Bed load measurements were also conducted during the 2010-2011 storm season (cbec, 2011). These data indicate that on average, the bed load is approximately 12 percent of the total load when compared to the USGS rating curve developed for suspended load. Using this relationship, bed load discharge was calculated for the n-year flood events.

BED MATERIAL AND REPRESENTATIVE GRAIN SIZE

The grainsize distributions for the suspended and bed load were defined from measurements taken during the 2010-2011 field study (cbec, 2011). Based on the results of this study, the representative grain size of the bed material and the relative distribution between the incoming bed load and suspended load are provided in Table 3.

Table 3. Representative grain size and relative distribution used as model input in MIKE 21C

Fraction	Grain Size (mm)	Bed Material (% finer)	Bed Load	Suspended Load
1	0.09	1%	0%	100%
2	0.17	3%	10%	90%
3	0.31	30%	80%	20%
4	0.51	80%	90%	10%

BED THICKNESS AND BANK ERODIBILITY

The active bed material thickness was arbitrarily set at 15 meters (50 feet) for the channel as this was considered an ample scour limit. Geomorphic field reconnaissance indicated that a majority of the banks within the project reach are armored. The extent of the armoring was estimated to be approximately 1.5 to 2.4 meters (5 to 8 feet) above the bank toe based on a limited number of field observations. Based on this assumption, bank erodibility was limited to the lower 1.5 to 2.4 meters (5 to 8 feet) of the bank above the toe, thereby preventing any erosion and/or migration of the bank or levee in the model. The bed material thickness at the toe was also set to 0.5 meters (1.6 feet) as a transition from the moveable bed to the armored banks.

MODEL ASSUMPTIONS AND LIMITATIONS

The preparation and use of the MIKE 21C model and results for existing conditions include the following assumptions and limitations:

1. All simulations utilize synthetic hydrographs derived from Comprehensive Study (USACE, 2002) hydrology as provided by MBK based on Feather River at Shanghai Bend storm centering.
2. There is the potential that the channel bathymetry may have changed since the time of the survey in 2008.
3. Detailed mapping of rip rap extents was not available to classify bank erodibility at the toe for every segment of the river. However, a majority of the river banks are armored, which was used to globally describe the moveable bed conditions at the toes.
4. MIKE 21C is being used as a tool for assessing potential geomorphic change, sediment transport results are not intended to be taken as absolute and conclusions drawn are based on short-duration synthetic flood hydrographs.
5. All simulations rely on the Yang sediment transport formula for sand. Equation selection was based on the findings in Yang and Huang (2001) that Yang's formula for sand transport is more robust and accurate compared with other available sediment transport formulations.
6. MIKE 21C was calibrated for hydrodynamics by adjusting Manning's roughness coefficients such that the predicted water surface profile closely matched the water surface profile of the calibrated RMA2 model. MIKE 21C was not calibrated for sediment transport and geomorphic change. However, the MIKE 21C model did rely upon measured bed material and sediment transport data collected by the USGS and cbec.
7. The USGS suspended load data was adjusted for washload. Washload was excluded from the sediment transport analyses.

RESULTS

HYDRAULIC MODEL CALIBRATION

Calibration of the hydrodynamic model was performed for both the 100-year (3,358 cms [118,587 cfs])[2] flood event and an observed 2-year (2,086 cms [73,670 cfs]) flood event using standard techniques. Calibration was conducted for the 100-year flood event by comparing the water surface profile between MIKE 21C model predictions and those computed using the RMA2 model. The MIKE 21C model was run for two days at a constant discharge to approximate the steady-state approach utilized within RMA2. With this approach, it was necessary to increase Manning's roughness coefficients uniformly by 33.3% from the initial values to compensate for differences in model resolution, discretization, and compounded model assumptions. This increase corresponds to a change in bed roughness from the base value of 0.02 to 0.027, both of which are within acceptable ranges for meandering sand bed channels (Chow, 1959). Calibration results for the 100-year flood event are shown in Figure 6 and demonstrate that the match to the RMA2 and RAS water surface profiles.

Calibration was also performed for an observed 2-year flood event by comparing water surface elevations between MIKE 21C model predictions and the corresponding stages observed during the 2010-2011 flood season. On December 23, 2011, cbec measured a discharge of 2,086 cms (73,670 cfs) at RM 58.9, while stage recorders measured the corresponding water surface elevations of 22.5 and 20.5 feet at RM 57.6 and RM 52.5, respectively (cbec, 2011). With these data and the corresponding stage recorded by the USGS gage at Freeport, a constant discharge was simulated in MIKE 21C for a duration of 2 days. In order to match the observed water surface elevations, it was necessary to increase Manning's roughness coefficients uniformly by 39.8% from the initial values provide by MBK. This increase corresponds to a change in bed roughness from the base value of 0.02 to 0.028, both of which are within acceptable ranges for meandering sand bed channels (Chow, 1959). This would suggest that the Manning's roughness coefficients for a 2-year flood event are approximately 6.5% higher than that of the 100-year flood event. The calibration results for the 2-year flood event are shown in Figure 7 and demonstrate that the absolute errors between predicted and observed water surface elevations are less than 0.02 meters (0.07 feet). This magnitude of calibration error is within acceptable limits considering the accuracy of model predictions and the water level data collection.

EXISTING CONDITIONS

Under existing conditions (i.e., no levee setbacks), the bed of the Sacramento River through the study reach is effectively vertically-stable during the 2- through 200-year recurrence interval flood events. Figures 8 through 13 display bed level change for these events. Erosional and depositional features exist within meander bends, in reaches where there is a significant change in cross sectional area (constricted reaches), and around structures that induce local scour. These features represent a net movement of

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[2] This discharge corresponds to the hydraulic impact analysis (HIA) condition simulated by MBK for the 100-year event. Design discharges with Shanghai centered hydrology were utilized for the unsteady simulations as presented in Table 1.

bed material from one location to another over the course of given flood event. For example, sediment is typically eroded from the outside of meander bend where bed shear stress is high and is then re-deposited on the inside of that bend within the bar area due to secondary or helical flow. By contrast, within straight reaches, where fairly uniform flow fields exist, there is minimal erosion or deposition. Over longer periods of time, and with the occurrence of additional lower magnitude events, these features are typically redistributed resulting in an equilibrium condition. This equilibrium condition is further corroborated by the results shown in Table 4, which demonstrate that there is minimal change in the average bed level when the net change in bed volume is distributed over entire area of the study reach. Also, due to the existing revetment within the project reach, the channel is laterally confined and lateral migration has been largely arrested. This is correspondingly represented in the model per the model input parameters.

Table 4. Net bed change within the model domain

Recurrence Interval	Net Change in Bed Volume	Net Change in Bed Volume	Average Change in Bed Level	Average Change in Bed Level
(years)	(m^3)	(yd^3)	(m)	(ft)
2-year	-108,543	-141,969	-0.03	-0.1
10-year	-115,263	-150,759	-0.03	-0.1
25-year	-129,704	-169,646	-0.04	-0.1
50-year	-480,315	-628,229	-0.13	-0.4
100-year	-270,779	-354,166	-0.10	-0.3
200-year	-285,401	-373,291	-0.10	-0.3

Bed shear stress is a measure of erosive force described as a function of velocity, depth, and gravitational forces and is often used as a measure of a river's ability to entrain bed material. When the shear stress equals the critical shear stress of the riverbed material or floodplain soil characteristics, the channel or floodplain will likely be in equilibrium. When shear stress is excessively greater than the critical shear stress, channel degradation may result. The maximum bed shear stress over the course the hydrograph has been calculated for the 2- and 100-year flood events and is displayed in Figures 15 and 16. These values range from 0 to 63 N/m^2 for the 2-year flood event and 0 to 115 N/m^2 for the 100-year flood event. These values are positively correlated with areas of high velocity and high bed roughness. Shear stresses can be compared with the list of allowable shear stresses for stream channel materials shown in Table 5 to aid in the understanding of mobility thresholds for this system. These results generally show that the levee slopes and rip rap are not erodible (notwithstanding toe failure).

Table 5. Allowable velocities and shear stresses for streambank materials (from Fischenich, 2001)

Boundary Category	Boundary Type	Permissible Shear Stress (lb/sq ft)	Permissible Shear Stress (N/M^2)	Permissible Velocity (ft/sec)	Permissible Velocity (m/sec)	Citations
Soils	Fine colloidal sand	0.02 - 0.03	0.96 - 1.44	1.5	0.4572	A
	Sandy loam (noncolloidal)	0.03 - 0.04	1.44 - 1.91	1.75	0.5334	A
	Alluvial silt (noncolloidal)	0.045 - 0.05	2.2 - 2.4	2	0.6096	A
	Silty loam (noncolloidal)	0.045 - 0.05	2.2 - 2.5	1.75 - 2.25	0.5334 - 0.6858	A
	Firm loam	0.075	3.6	2.5	0.7620	A
	Fine gravels	0.075	3.6	2.5	0.7620	A
	Stiff clay	0.26	12	3 - 4.5	0.9144 - 1.3716	A,F
	Alluvial silt (colloidal)	0.26	12	3.8	1.143	A
	Graded loam to cobbles	0.38	18	3.8	1.143	A
	Graded silts to cobbles	0.43	21	4	1.2192	A
	Shales and hardpan	0.67	32	6	1.8288	A
Gravel/Cobble	1 - in	0.33	16	2.5 - 5	0.762 - 1.524	A
	2 - in	0.67	32	3.0 - 6.0	0.9144 - 1.8288	A
	6 - in	2	96	4.0 - 7.5	1.2192 - 2.286	A
	12 - in	4	191	5.5 - 12.0	1.6764 - 3.6576	A
Vegetation	Class A turf	3.7	177	6.0 - 8.0	1.8288 - 2.4384	E,N
	Class B turf	2	96	4.7 - 7.0	1.2192 - 2.1336	E,N
	Class C turf	1	48	3.5	1.0668	E,N
	Long native grasses	1.2 - 1.7	81	4.0 - 6.0	1.2192 - 1.8288	G,H,L,N
	Short native and bunch grasses	0.7 - 0.95	33 - 45	3.0 - 4.0	0.9144 - 1.2192	G,H,L,N
	Reed plantings	0.1 - 0.6	5.0 - 29	N/A	N/A	E,N
	Hardwood tree plantings	0.41 - 2.5	20 - 120	N/A	N/A	E,N
Temporary Degradable RECP's	Jute net	0.45	22	1 - 2.5	0.3048 - 0.762	E,H,M
	Straw with net	1.5 - 1.65	72 - 79	1.0 - 3.0	0.3048 - 0.9144	E,H,M
	Coconut fiber with net	2.25	108	3.0 - 4.0	0.9144 - 1.2192	E,M
	Fiberglass roving	2	96	2.5 - 7.0	0.762 - 2.1336	E,H,M
Non - Degradable RECP's	Unvegetated	3	144	5.0 - 7.0	1.524 - 2.1336	E,G,M
	Partially established	4.0 - 6.0	191 - 287	7.5 - 15.0	2.286 - 4.572	E,G,M
	Fully vegetated	8	383	8.0 - 21.0	2.4384 - 6.4008	F,L,M
Riprap	6 - in d50	2.5	120	5.0 - 10.0	1.524 - 3.048	H
	9 - in d50	3.8	182	7.0 - 11.0	2.1336 - 3.3528	H
	12 - in d50	5.1	244	10.0 - 13.0	3.048 - 3.9624	H
	18 - in d50	7.6	364	12.0 - 16.0	3.6576 - 4.8768	H
	24 - in d50	10.1	483	14.0 - 18.0	4.2672 - 5.4864	E
Soil Bioengineering	Wattles	0.2 - 1.0	10.0 - 48	3	0.9144	C,I,J,N
	Reed fascine	0.6 - 1.25	29 - 60	5	1.524	E
	Coir roll	3.0 - 5.0	144 -4239	8	2.4384	E,M,N
	Vegetated coir mat	4.0 - 8.0	191 - 383	9.5	2.8956	E,M,N
	Live brush mattress (original)	0.4 - 4.1	19 - 196	4	1.2192	B,E,I
	Live brush mattress (grown)	3.90 - 8.2	187 - 392	12	3.6576	B,C,E,I,N
	Brush Layering (initial/grown)	0.4 - 6.25	19 - 299	12	3.6576	E,I,N
	Live fascine	1.25 - 3.10	60 - 148	6.0 - 8.0	1.8288 - 2.4384	C,E,I,J
	Live willow stakes	2.10 - 3.10	100 - 148	3.0 - 10.0	0.9144 - 3.048	E,N,O
Hard Surfacing	Gabions	10	478	14.0 - 19.0	4.2672 - 5.7912	D
	Concrete	12.5	598	>18	>5.4864	H

The maximum velocity over the course of the hydrograph has been calculated for the 2- and 100-year flood events and is displayed in Figures 16 and 17. These values typically range from 0.5 m/s to 1.6 m/s (1.6 to 5.2 ft/s) for the 2-year flood event and 0.5 to 2.1 m/s (1.6 to 6.9 ft/s) for the 100-year flood event.

CONCLUSIONS

A detailed MIKE 21C morphological model of the Sacramento River from RM 58.9 upstream of Pioneer Memorial Bridge to RM 46.4 at the Freeport Bridge was developed for existing conditions and successfully calibrated for hydrodynamics. The sediment transport model was not calibrated, but was developed based on measured data for bed material, bed load, and suspended load. Preliminary model results for existing conditions indicate that the bed of the Sacramento River within the project reach is for the most part vertically stable. Erosion and depositional patterns shown in the model do exist within areas where such patterns are typically expected to occur (i.e., meander bends) following significant flood events. These results are consistent with long-term trends, which suggest that the Sacramento River in the vicinity of the project reach is overall in equilibrium with the existing sediment regime. A recent study (Hall, et. al. 2010) showed that this reach of the Sacramento River is slightly degradational based on long terms simulations showing 0.02 feet of erosion over 50 years and 0.1 feet of erosion over 100 years. However, since these depths of erosion are so small for the time horizons predicted, the predictions are considered to be within accepted uncertainties for sediment transport modeling. The preliminary findings presented here are intended to serve as an interim deliverable. The final deliverable will include simulations of the preferred setback design alternative with analyses that will examine the relative project impacts on channel hydraulics and erosional/depositional trends associated with the selected design.

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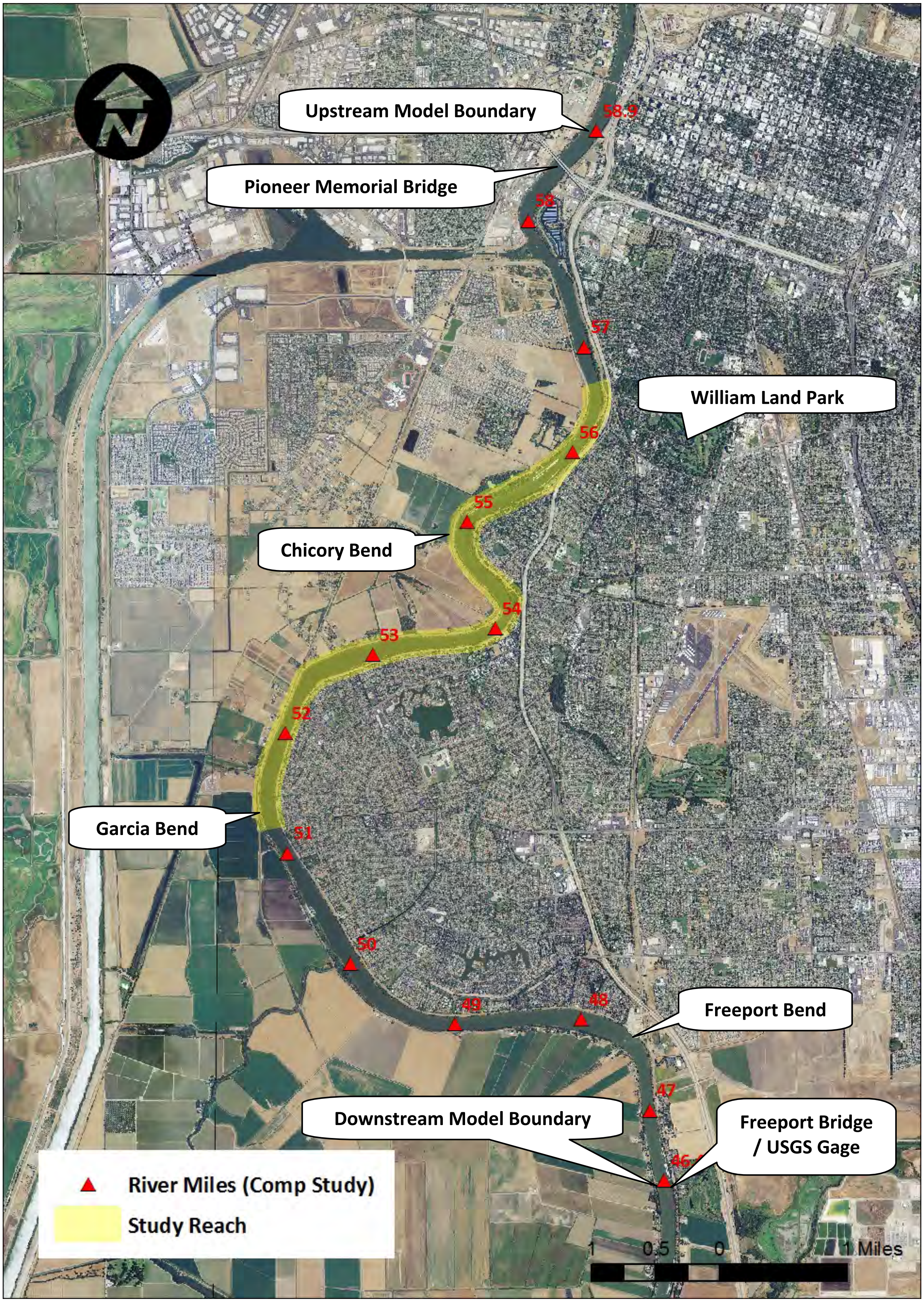
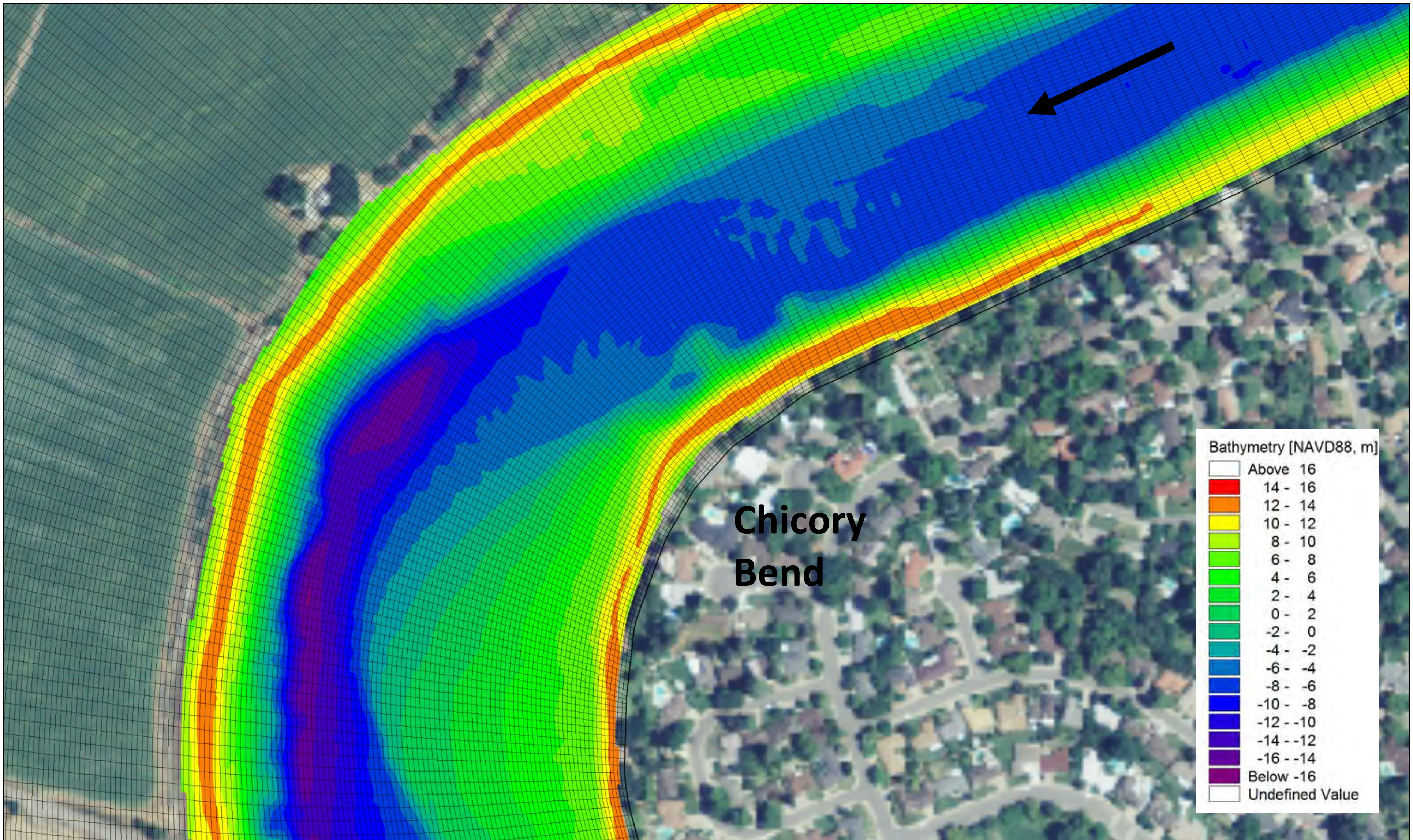


Notes:

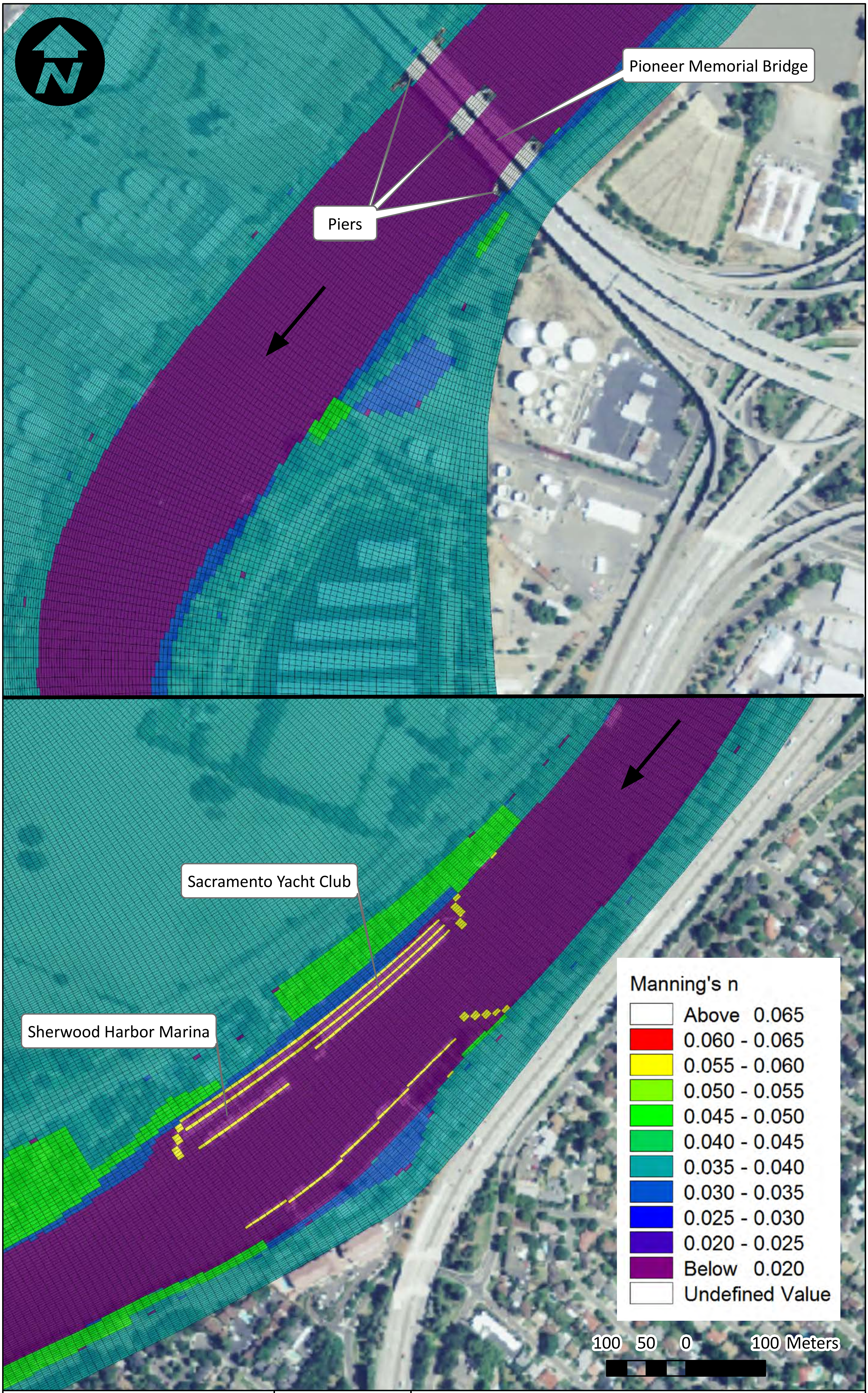
West Sacramento Southport EIP Task Order 3

MIKE 21C existing condition model domain

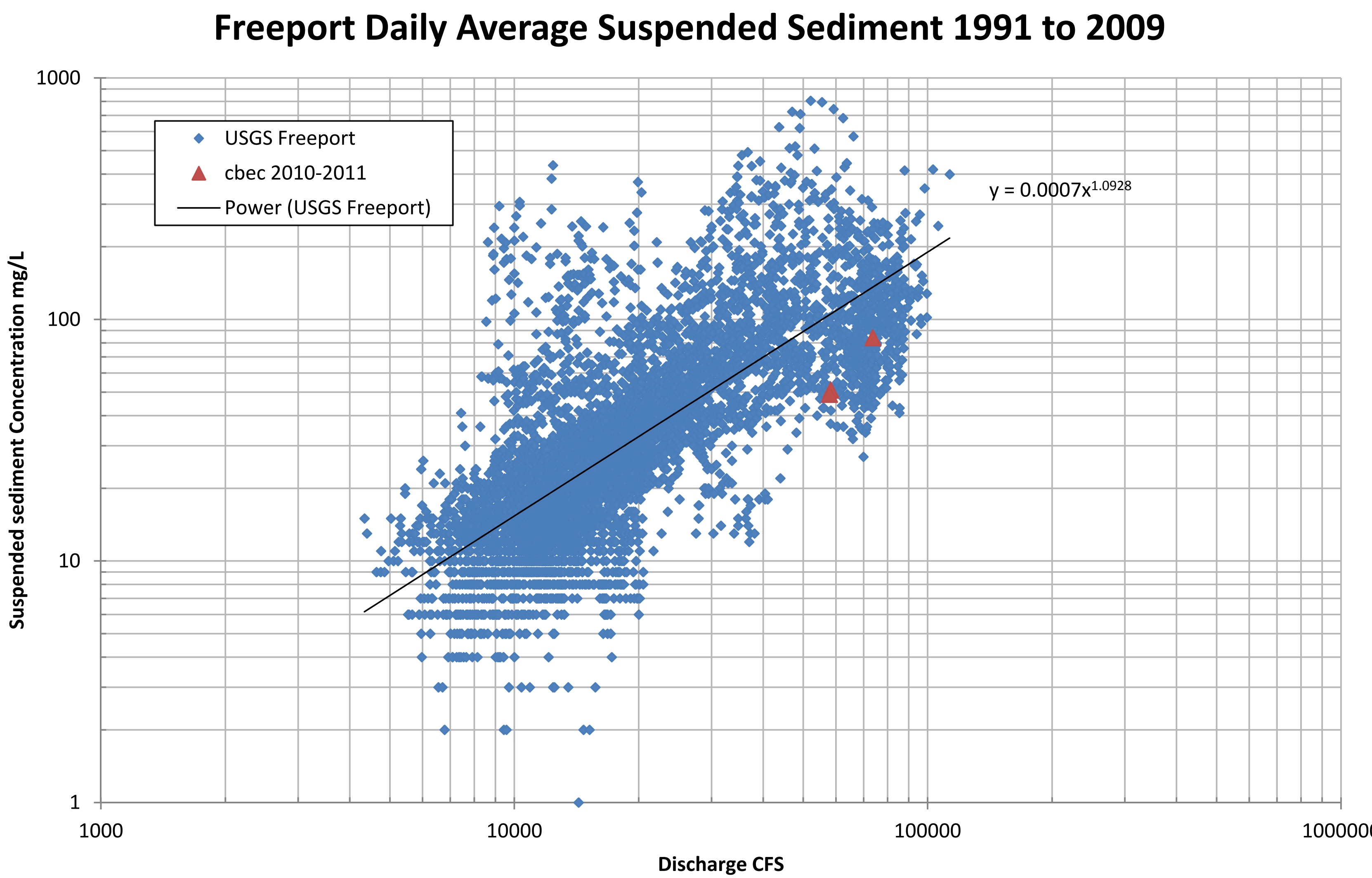
Project No. 11-11013	Created By: JS	**Figure 1**



Notes:	cbec eco engineering	*West Sacramento Southport EIP Task Order 3* **Example of model grid and bathymetry**		
		Project No. 11-1013	Created By: AMS	**Figure 2**



Notes:	cbec eco engineering	*West Sacramento Southport EIP Task Order 3* **Pre-calibrated Manning's n values**		
		Project No. 11-1013	Created By: AMS	**Figure 3**



Notes:	cbec eco engineering	*West Sacramento Southport EIP Task Order 3* **Suspended Sediment Rating Curve at Freeport**		
		Project No. 11-1013	Created By: JS	**Figure 4**

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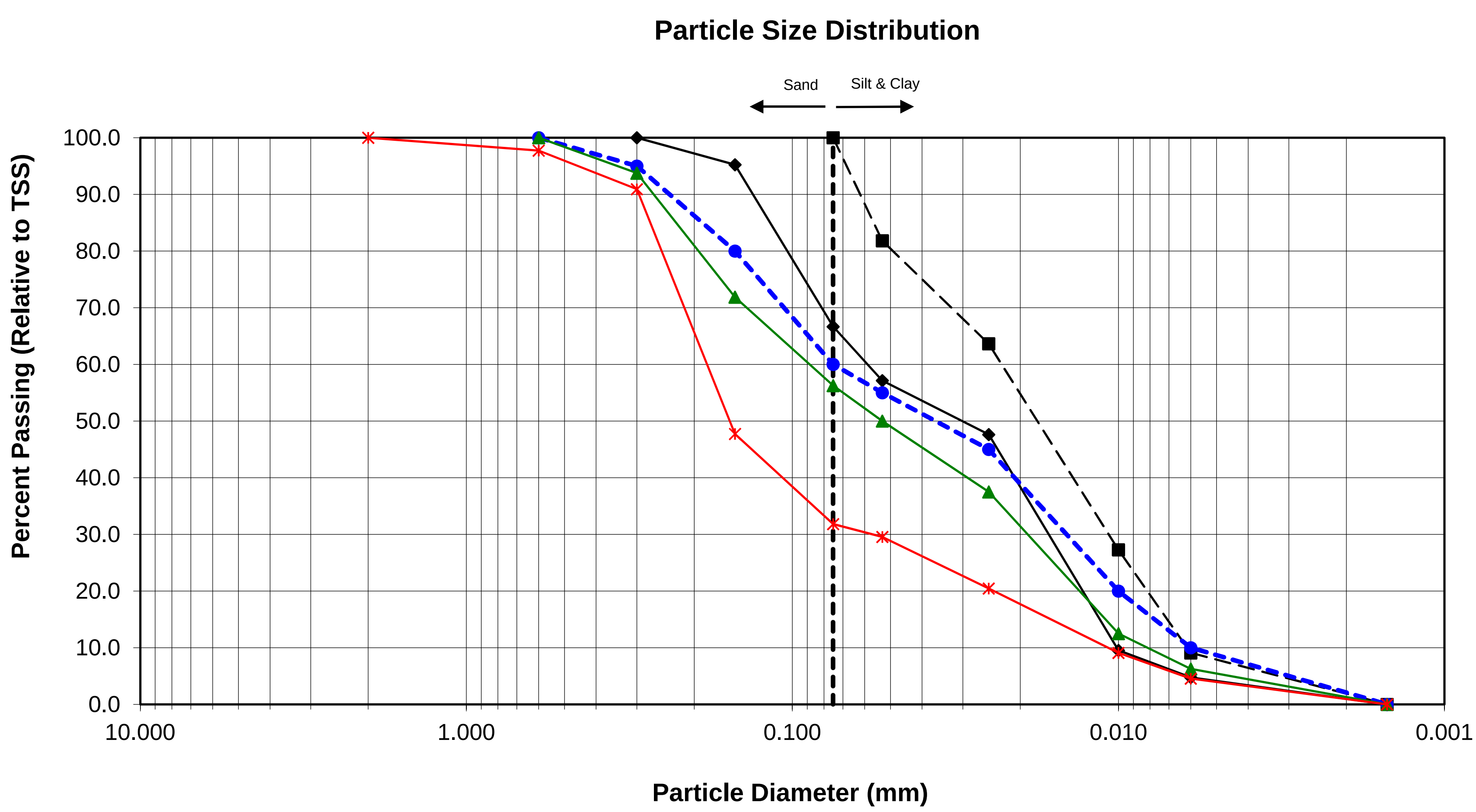
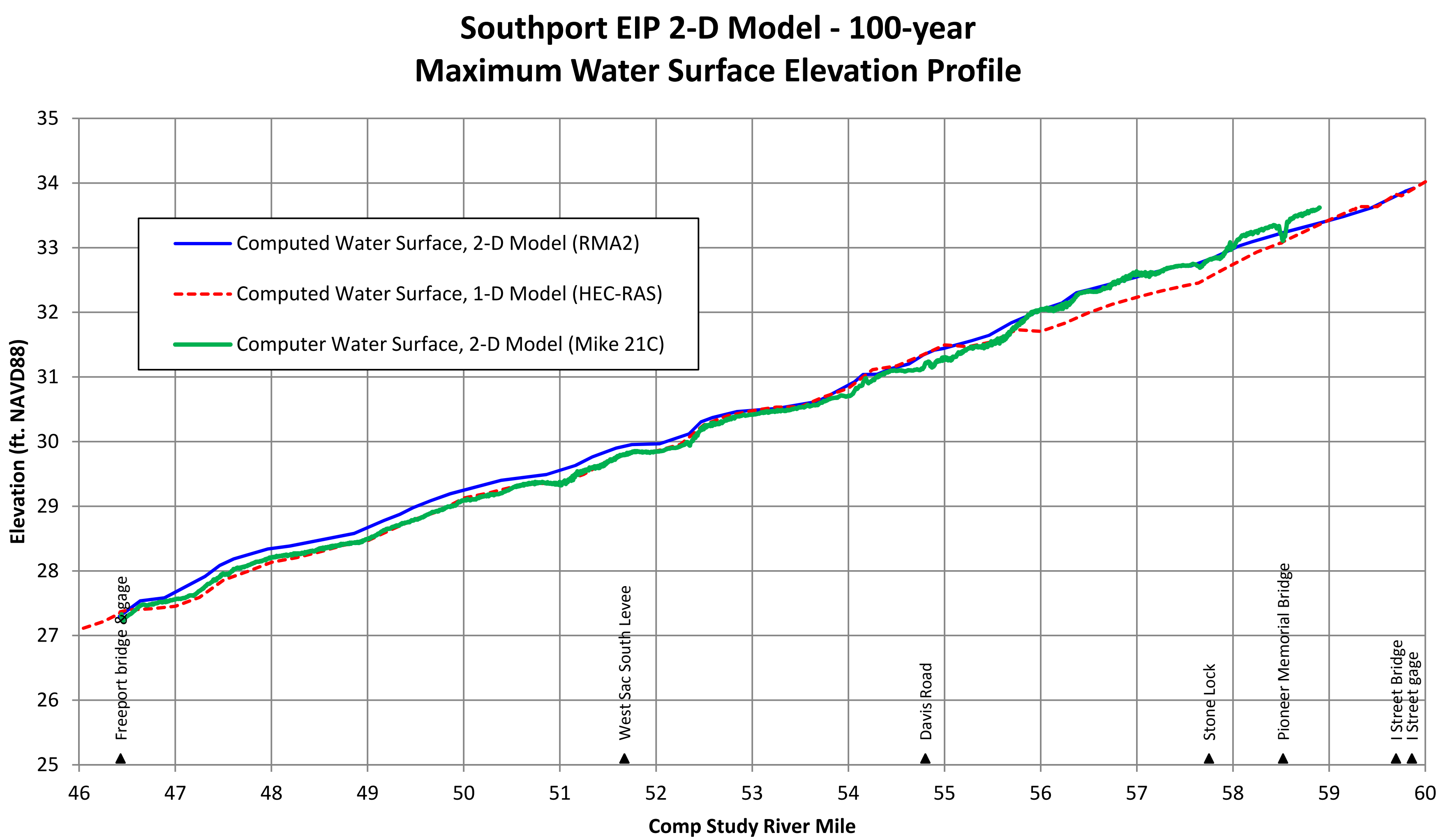
11/7/2011



Notes:	cbec	*West Sacramento Southport EIP Task Order 3* **Suspended Sediment Particle Size Distribution**		
		Project No. 11-1013	Created By: JS	**Figure 5**



Notes:		*West Sacramento Southport EIP Task Order 3*		
		Calibration Results: 100-year event		
		Project No. 11-1013	Created By: JS	**Figure 6**

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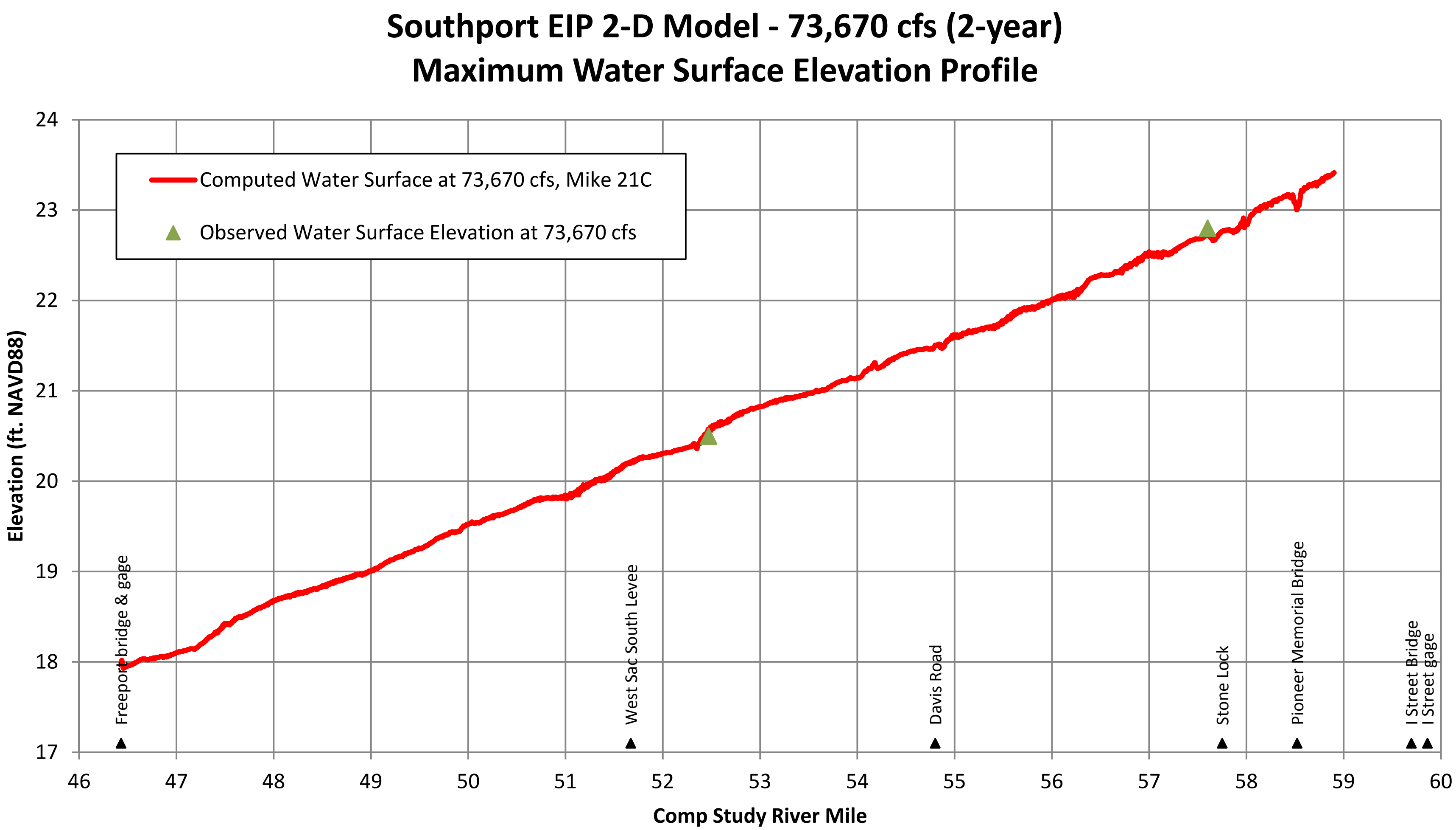
11/7/2011



Notes:	cbec	*West Sacramento Southport EIP Task Order 3* **Calibration results: 73,670 cfs (2-year)**		
		Project No. 11-1013	Created By: JS	**Figure 7**

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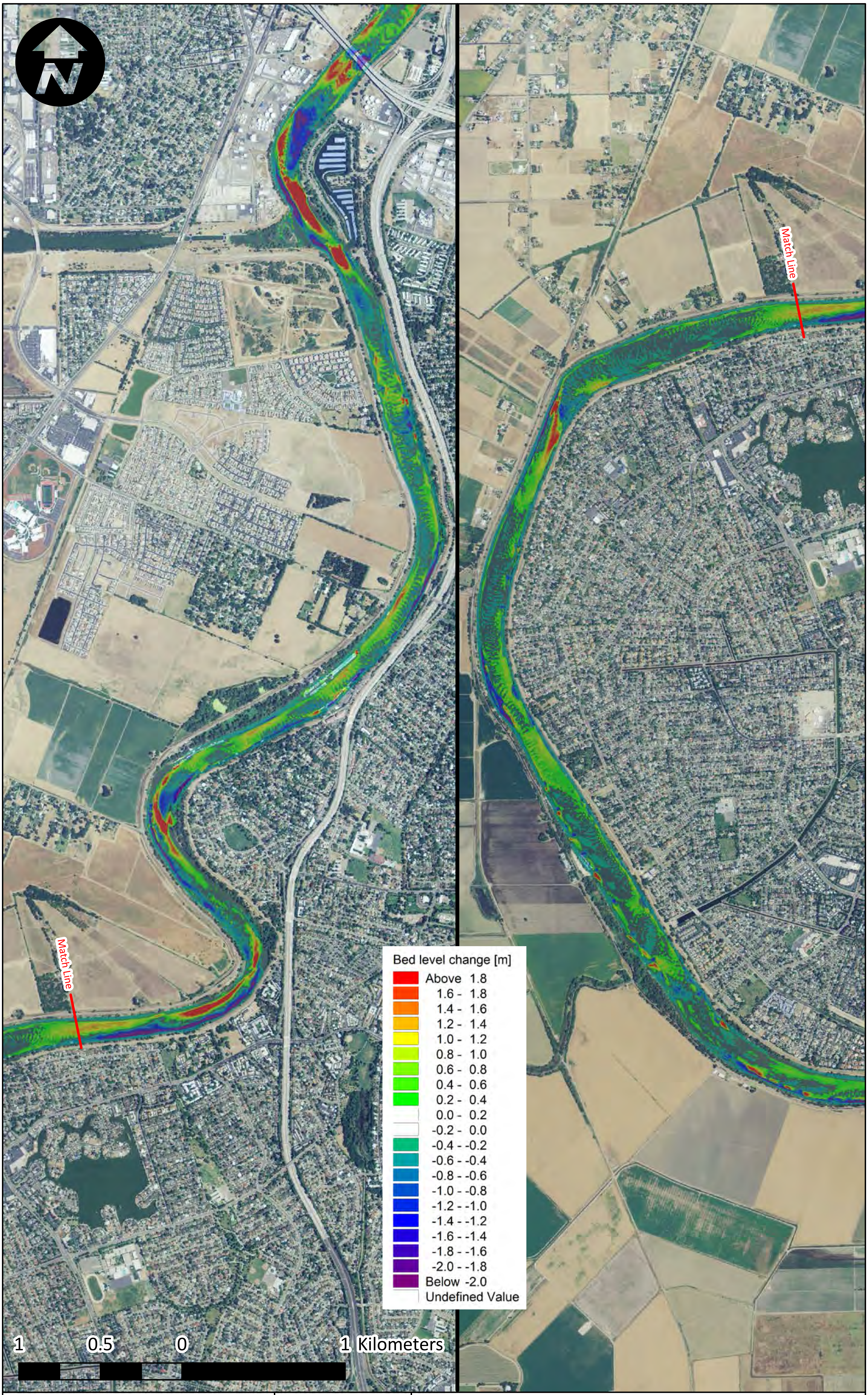
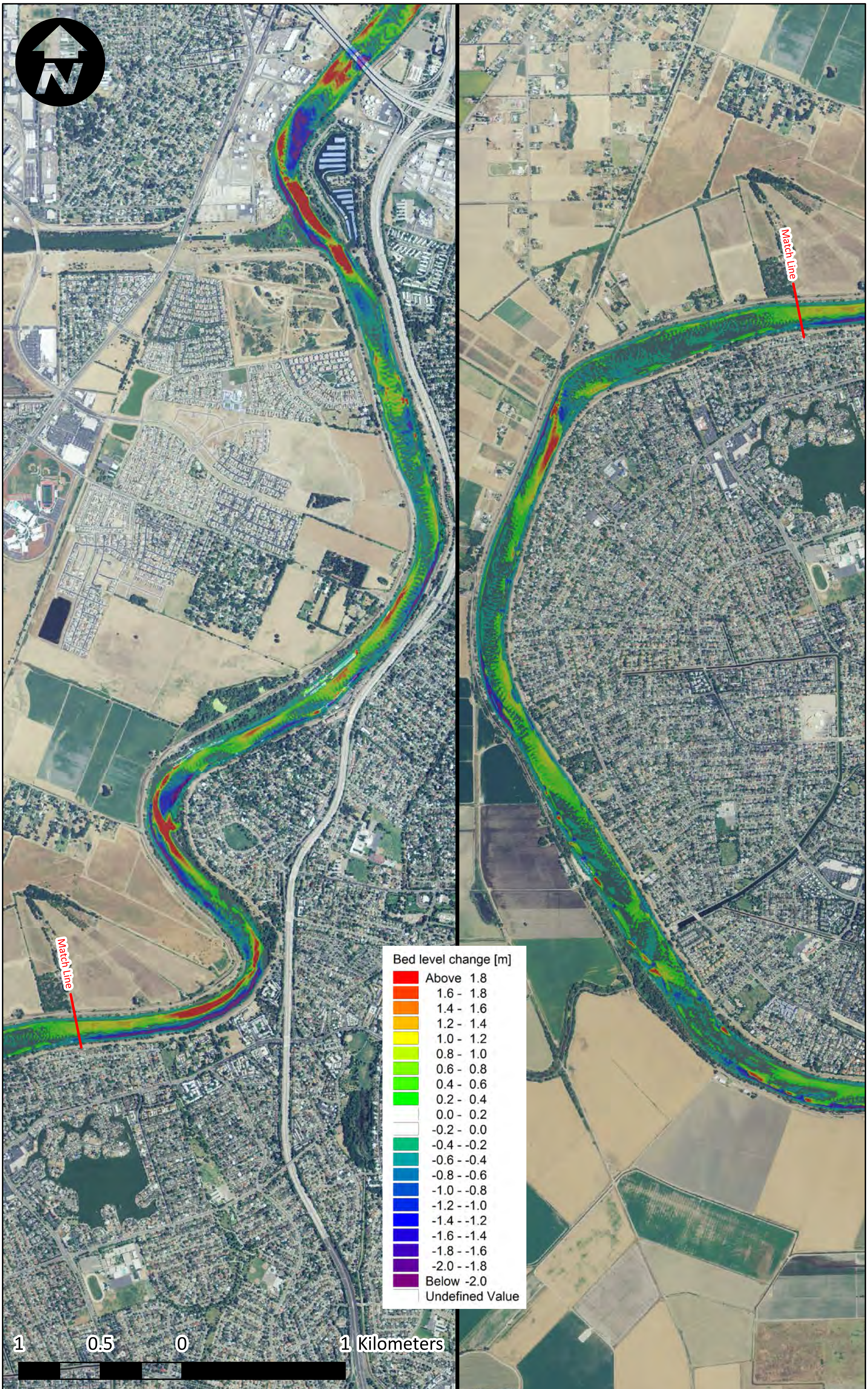
11/7/2011



Notes:	cbec eco engineering	West Sacramento Southport EIP Task Order 3 **2-year event bed level change**	
		Project No. 11-1013	Created By: AMS	**Figure 8**



Notes:

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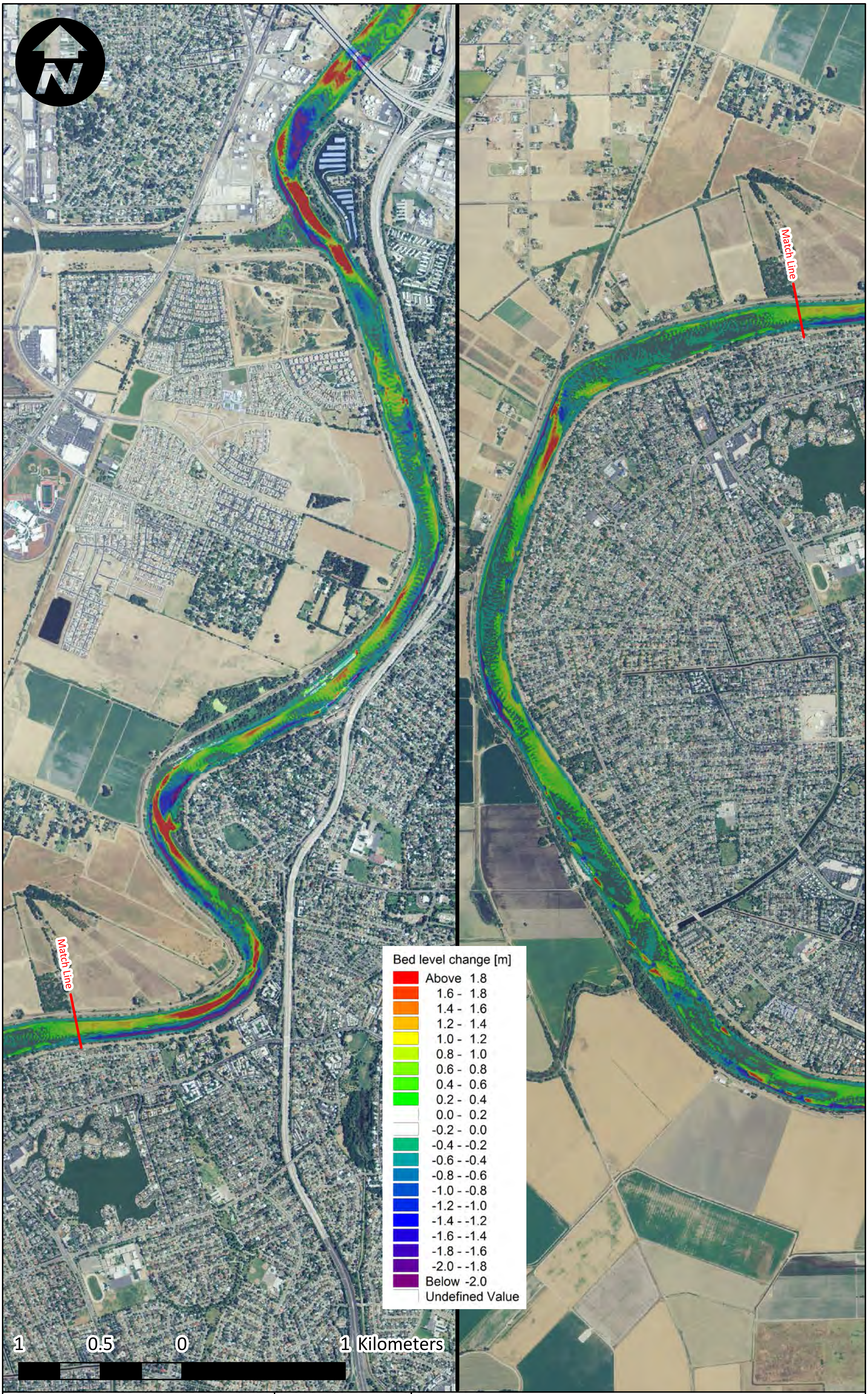
West Sacramento Southport EIP Task Order 3

10-year event bed level change

Project No. 11-1013 | Created By: AMS | **Figure 9**



Notes:	cbec eco engineering	*West Sacramento Southport EIP Task Order 3* **25-year event bed level change**		
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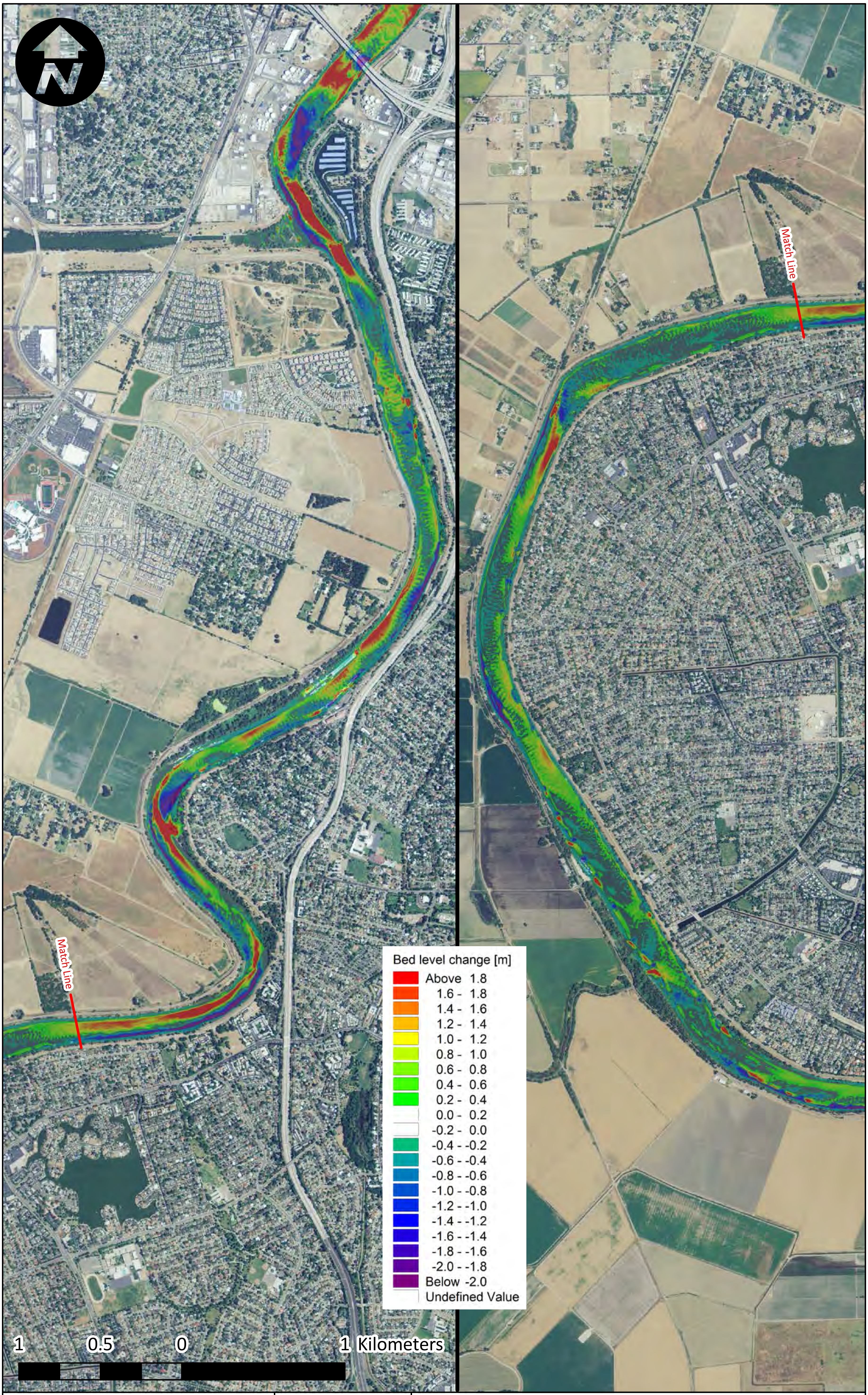

Notes:	cbec eco engineering	*West Sacramento Southport EIP Task Order 3*		
		50-year event bed level change		
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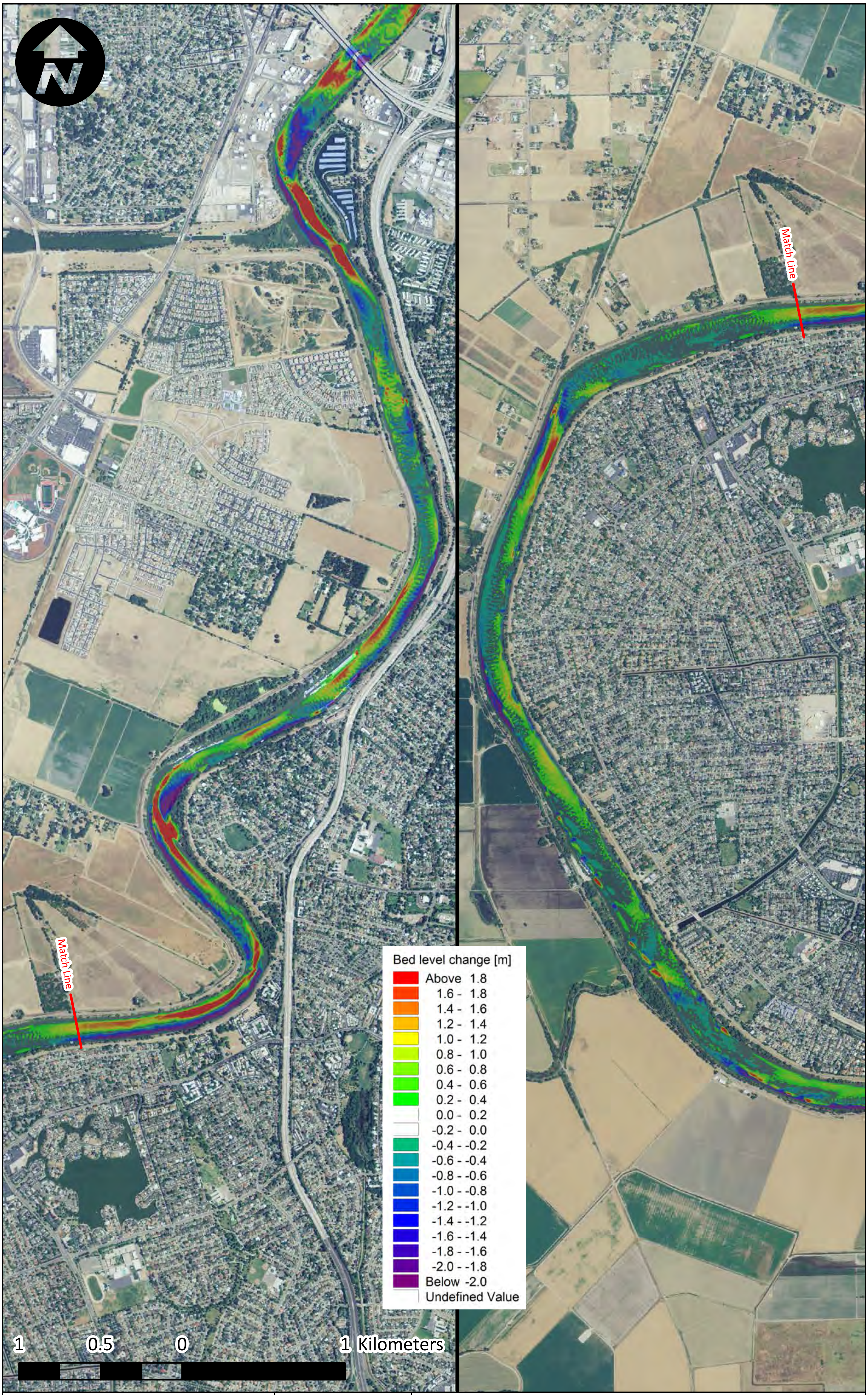
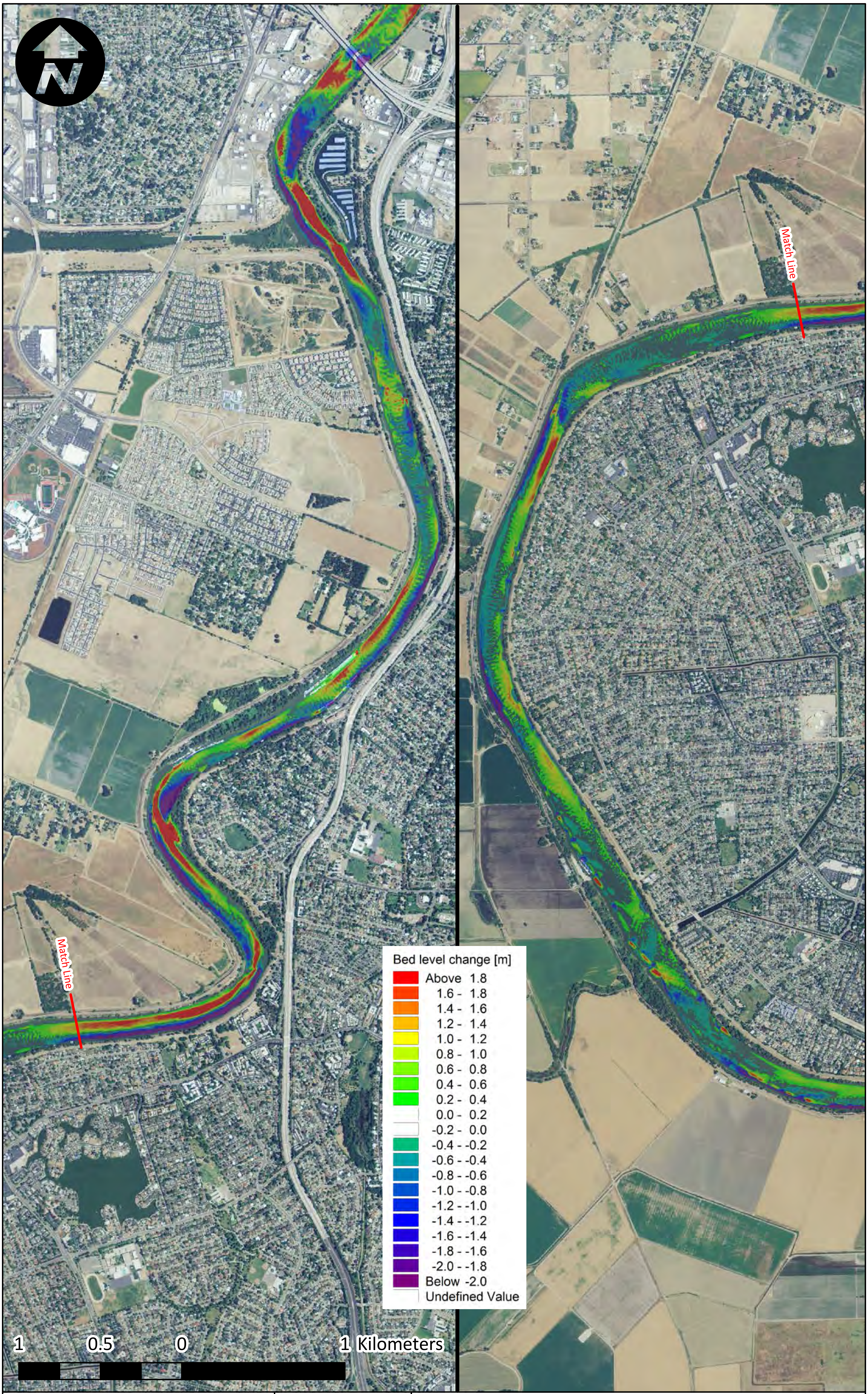


Notes:	cbec eco engineering	*West Sacramento Southport EIP Task Order 3* **100-year event bed level change**		
		Project No. 11-1013	Created By: AMS	**Figure 12**



Notes:	cbec eco engineering	West Sacramento Southport EIP Task Order 3 **200-year event bed level change**		
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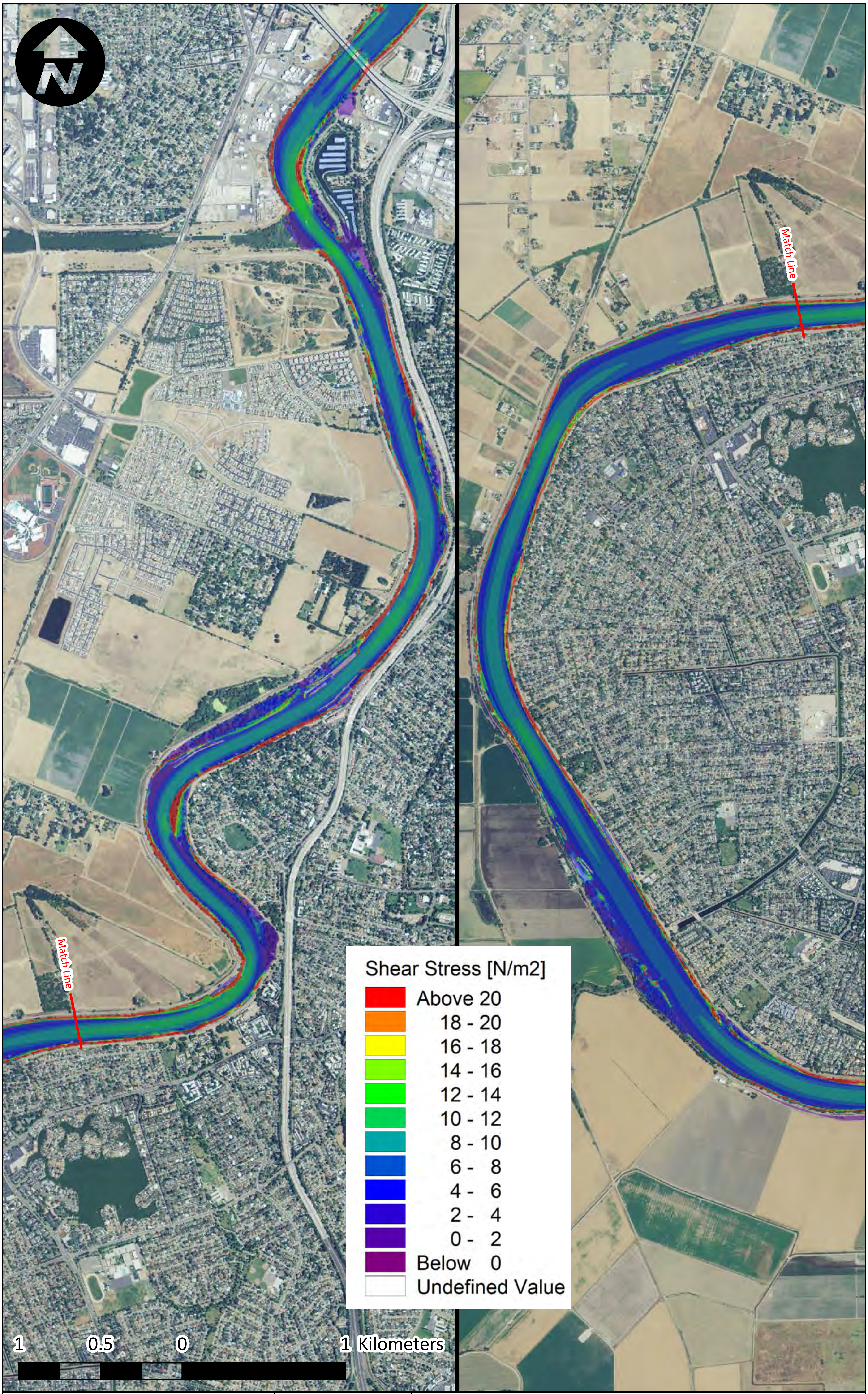

Notes:	cbec eco engineering	West Sacramento Southport EIP Task Order 3 **100-year event maximum bed shear stress**		
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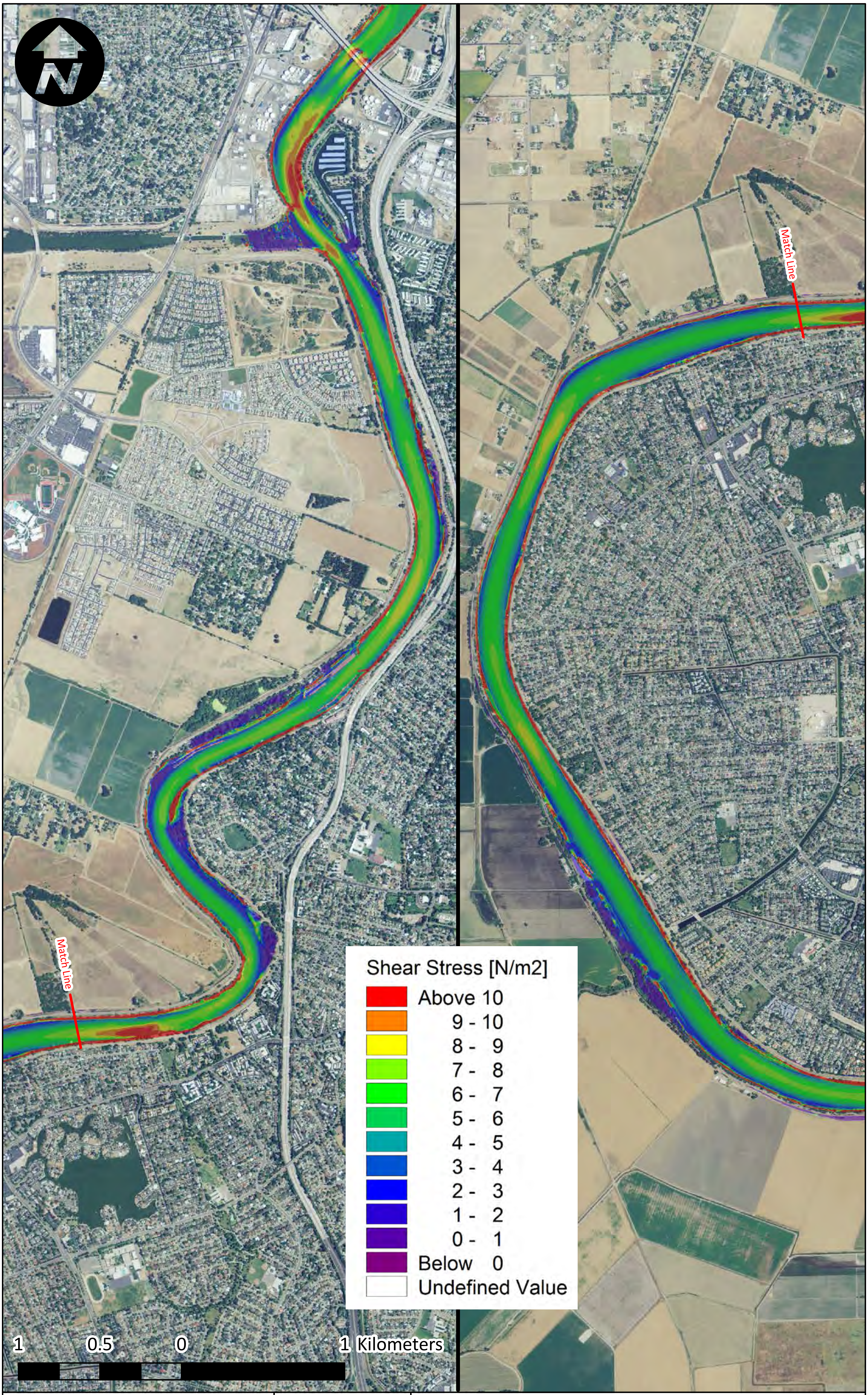

Notes:

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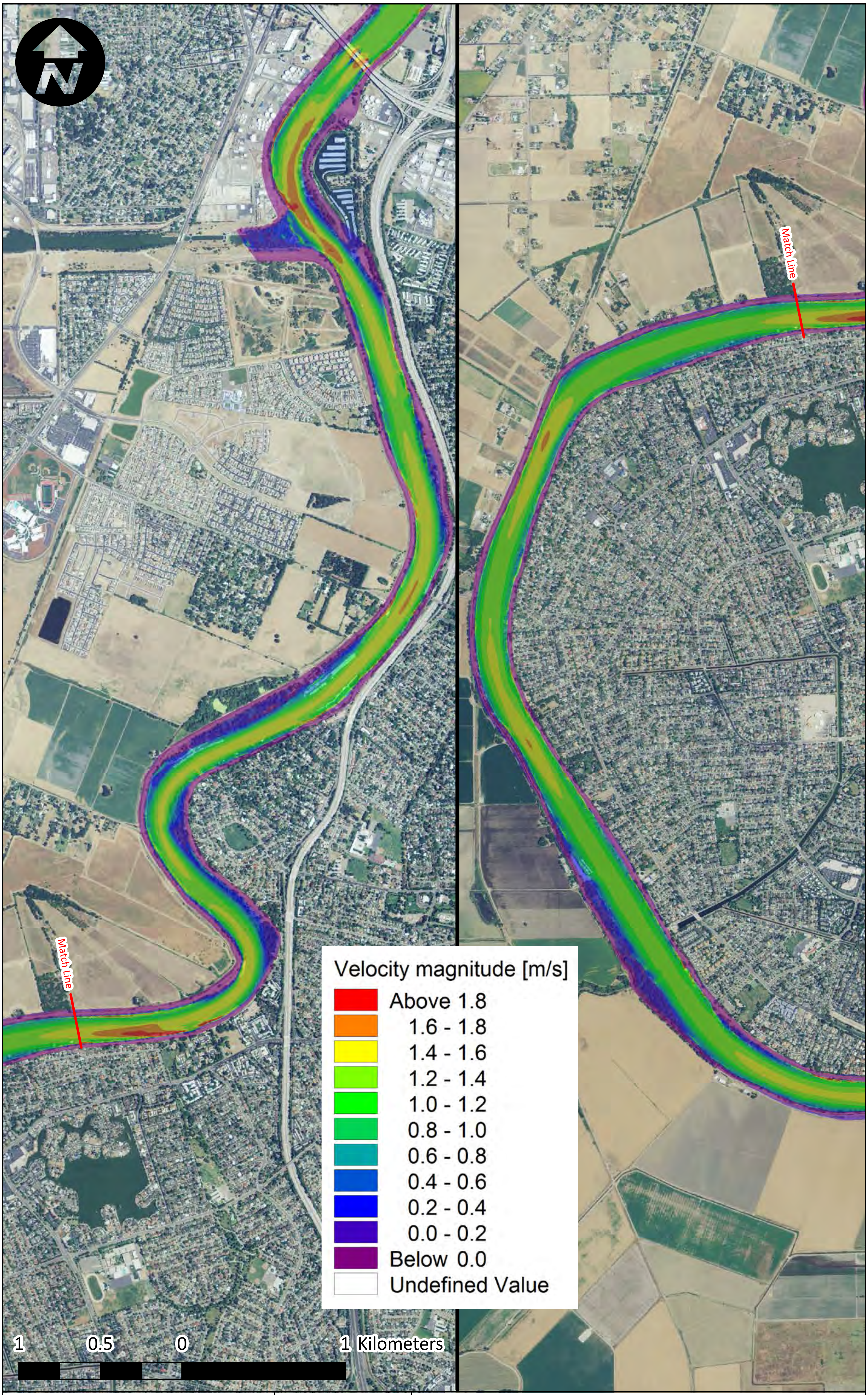
West Sacramento Southport EIP Task Order 3

2-year event maximum bed shear stress

Project No. 11-1013 | Created By: AMS | **Figure 15**



Notes:	cbec eco engineering	*West Sacramento Southport EIP Task Order 3* **2-year event maximum velocity magnitude**		
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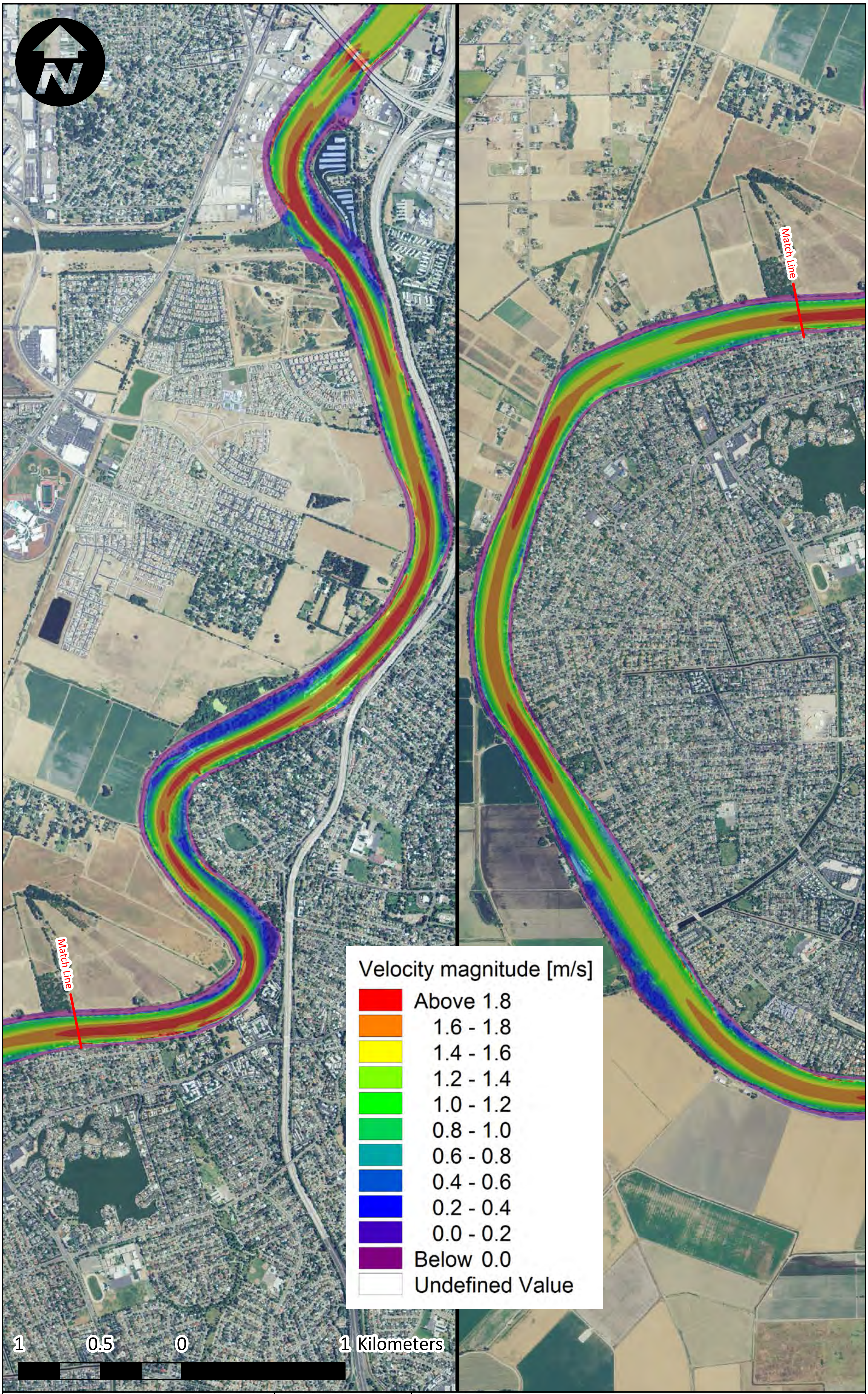

Notes:

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West Sacramento Southport EIP Task Order 3

100-year event maximum velocity magnitude

Project No. 11-1013	Created By: AMS	**Figure 17**