LOWER AMERICAN RIVER, EROSION SUSCEPTIBILITY ANALYSIS FOR INFREQUENT FLOOD EVENTS

MAIN REPORT
JULY 9, 2004

H Street Bridge During 1997 High Flow

Bank Erosion RM 7.3R After 1986 High Flow
LOWER AMERICAN RIVER – EROSION SUSCEPTABILITY ANALYSIS FOR INFREQUENT FLOOD EVENTS

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1.0 INTRODUCTION
1.1 General
1.2 Authorization and Work Requirements
1.3 Purpose and Scope of Project
1.4 Report Format
2.0 BACKGROUND
2.1 Man-Made Modifications of Lower American River
2.2 Lower American River Thalweg Profiles
3.0 HYDRAULIC MODELING
3.1 Model Background and Development
3.2 Hydraulic Modeling Results
3.3 Hydraulic Modeling Limitations
4.0 COMPUTATION OF CRITICAL SHEAR STRESS
5.0 REVIEW OF HISTORIC AERIAL PHOTOGRAPHS AND CHANNEL BEND MIGRATION PLOTS
6.0 GUIDELINES USED FOR THE EVALUATION OF LOWER AMERICAN RIVER LEVEES
7.0 FIELD INVESTIGATION
8.0 PREDICTED EROSION LOCATIONS FOR A FLOW OF 145,000 CFS
9.0 DISCUSSION OF PREDICTED EROSION SITES
9.1 RM 1.8 – Left
9.2 RM 2.5 – Left
9.3 RM 4.2 – Left
9.4 RM 6.4 – Left
9.5 RM 6.9 - Left
9.6 RM 7.0 - Right
9.7 RM 7.3 – Right
9.8 RM 8.0 – Right
9.9 RM 9.0 – Right
9.10 RM 9.7 – Left
9.11 RM 10.0- Left
9.12 RM 10.2 – Right
10.0 CONCLUSIONS
11.0 REFERENCES
COMMENTS AND RESPONSES
ITRT CERTIFICATION
1.0 INTRODUCTION

1.1 General

The Lower American River (LAR) is a 23 mile stretch of the American River which flows downstream of Folsom and Nimbus Dams into the Sacramento River. Flood control for the City of Sacramento is provided by Folsom Dam, and levees on the American River, from the confluence with the Sacramento River at River Mile 0 (RM) to RM 11 on the south bank and to approximately RM 14 on the north bank. A location map is shown in Figure 1.

Prior to the 1986 flood event, the estimated 100-year discharge from Folsom Dam was 115,000 cubic feet per second (cfs). The flow of 115,000 cfs was also the design flow for the Lower American River Levees. During the flood of 1986, the peak release from Folsom Dam was 134,000 cfs.

The peak flow during the 1986 event caused significant damage to the levees at several locations. Since that time, bank protection projects have been installed at five locations within the leveed reach.

After the 1986 flood event, hydrology for the American River watershed was recomputed. The new estimated 100-year discharge from Folsom Dam was 180,000 cfs. In the middle part of the 1990’s, the American River Common Features Project was authorized by Congress. One component (stream flow gages upstream of Folsom Dam – allowing more informed reservoir operation) resulted in reducing the estimated 100-year discharge to 163,000 cfs. After adoption of a Folsom Dam Reoperation Plan (currently being evaluated), the estimated 100-year discharge should further reduce to 145,000 cfs. Congress also authorized the Folsom Dam Modifications Project (scheduled for completion in 2012). Once constructed, this project will reduce the 100-year flow to 115,000 cfs (therefore, the estimated 100-year discharge of 145,000 cfs will be an interim condition).
The overall goal of the American River Watershed Project (which the American River Common Features Project and the Folsom Dam Modifications Project are components to) is to provide a higher level of flood protection to the City and County of Sacramento. It was estimated that by improving the Lower American River levees (American River Common Features Project), by enlarging the outlet capacity of Folsom Dam (Folsom Dam Modifications Project), and by raising Folsom Dam (Folsom Dam Mini-Raise Project), that the City and County of Sacramento would have a 1 in 213 chance of being flooded in any one year from the American River. The estimated flow rate for the 200-year event is 160,000 cfs. The Lower American River levees were improved under the American River Common Features Project to counter seepage and stability for the estimated 200-year flow of 160,000 cfs.

The purpose of this study is to evaluate the potential for erosion of grass-lined levees and overbanks associated with flows of 145,000 cfs and 160,000 cfs. Hydraulic modeling was developed for the American River for flows of 115,000 cfs, 130,000 cfs, 145,000 cfs, and 160,000 cfs. The flows of 145,000 cfs and 160,000 cfs are described above. The flow of 115,000 cfs is a flow that has been seen two times since Folsom Dam was constructed (1997 and 1965) and was modeled for comparative purposes. The flow of 130,000 cfs is a flow that has been seen one time since Folsom Dam was constructed (1986, peak flow 134,000 cfs) and was modeled for comparative purposes. In addition, there was a possibility that the Folsom Dam Reoperation Plan could have reduced the 100-year flow down to 130,000 cfs (instead of 145,000 cfs as described above). It was later determined however that this possibility is not achievable with the current Folsom Dam configuration. This analysis focused on evaluation of applied velocities and shear stresses for the four flow scenarios, comparing the applied velocity and shear stress values to allowable values, and comparing applied velocity and shear stress values for flows of 130,000 cfs, 145,000 cfs, and 160,000 cfs to values for the flow of 115,000 cfs.

As stated above, the purpose of this study was to evaluate the potential for erosion of grass-lined levees and overbanks associated with flows of 145,000 cfs and 160,000 cfs. Whether or not a levee failure is likely to occur at any site identified in this report is beyond the scope of this study. Sites identified in this study need to be evaluated in more detail to determine if a levee failure is likely to occur.

1.2 Authorization and Work Requirements

This study was authorized by the US Army, Corps of Engineers (USACE), Sacramento District, in a scope of work dated April 23, 2003 and a supplemental scope of work dated June 10, 2003, under Contract Number DACW05-02-0002. The Technical Manager for the Sacramento District was Mr. Daniel P. Tibbitts, P.E. of the Hydraulic Design Section.

The Project Manager for Ayres Associates was Mr. Thomas W. Smith, P.E. Subcontractors to Ayres Associates were Mr. Michael D. Harvey, Ph.D., P.G. of Mussetter Engineering, Fort Collins, CO. and Mr. Ronald R. Copeland, Ph.D., P.E., of Mobile Boundary Hydraulics, Clinton, Mississippi, both of whom were primarily responsible for technical review of this study.
1.3 Purpose and Scope of Project

The primary purpose of this project was to evaluate the potential for erosion of grass-lined levees and overbanks as a result of increasing the Folsom Dam peak release rate above 115,000 cfs. An existing RMA-2V hydraulic model was modified and utilized for this study. Flows modeled include 115,000, 130,000, 145,000, and 160,000 cfs. Products from the modeling effort include the following and are included in the Appendix to this report.

- Water Depth Plots; 115K, 130K, 145K and 160K cfs.
- Critical Shear Stress Plot
- Ratio of Critical Shear Stress to Applied Shear Stress; 130k, 145K and 160K cfs.
- Ratio of 0.5 psf Shear Stress to Applied Shear Stress; 145K.

This study also included the following scoped items.
- Review of Historic Aerial Photographs
- Channel Bend Lateral Migration Plots
- Field Investigation; 2-day field review of the entire levee system with Corps technical manager, Ayres Associates and Subconsultants.
- Technical Review by Subconsultants.
- Draft and Final Project Reports

1.4 Report Format

This main report contains general background information, project purpose, discussion of findings and conclusions as to whether the existing protection adequately prevents significant erosion from occurring at peak flows of 130,000 cfs, 145,000 cfs, and 160,000 cfs. The appendix to this report contains a technical report and all of the output plots from the hydraulic modeling and aerial photograph comparison. The technical report includes additional details on the hydraulic model and procedures used for the historic aerial photograph review.

2.0 BACKGROUND

2.1 Man-Made Modifications of Lower American River

A Geomorphic, Sediment Engineering, and Channel Stability Analysis (Ayres 1997) was conducted by the USACE in the early 1990’s and represents the most recent study to investigate sediment transport and channel stability issues in the lower river. The following background information is excerpted from that report.

*The present day characteristics of the Lower American River have been significantly affected by historical mining activities and upstream dam construction. Hydraulic mining caused approximately 15 to 20 ft of aggradation through the project reach. Dredge*
Lower American River mining for gold as far downstream as Goethe Park (RM 13.5) (See Figure 1) caused reworking of the floodplain and bars, and it significantly altered the out-of-bank topography. Sand and gravel mining both in the river and on the floodplain has resulted in the development of numerous split flow reaches. Construction of Folsom Dam and Nimbus Dam has effectively eliminated any sediment supply from the upper watershed to the Lower American River. The combined effect of base level lowering resulting from recovery of channel bed elevations (in the Sacramento River by about 1930 (Meade 1982)), sand and gravel mining, and dam construction has been channel degradation and possibly exhumation of pre-existing bedrock topography within the reach. Channel degradation has required reinforcement of numerous bridge piers especially in the lower reaches where the oldest bridges are located.

Shlemon (1972) identified the spatial distribution of the Pleistocene and Holocene-age formations that represent the paleo-American River sediments that are currently affecting the vertical and lateral stability of the Lower American River (LAR) between the Sacramento River confluence (RM 0) and Nimbus Dam (RM 22). The formations include Modesto (youngest and least erosion resistant), Riverbank (consolidated and erosion resistant) and Turlock Lake (cemented and most erosion resistant). In general, the youngest formation crops out downstream and the older ones farther upstream. Previous (Ayres, 1997) and current field inspection of the LAR indicate that Modesto Formation cropped out in the bed of the river at about RM 4.5 and 5.8, but the river was capable of eroding through the outcrop at these locations. Modesto Formation outcrop currently forms the right bank of the river at RM 5.5, and it appears to be retarding river migration at this location. The more erosion resistant Riverbank Formation is currently exposed in the bed of the LAR at RM 7 to 7.3, RM 9.4 to 10, RM 10.9, RM 11.6 and from RM 13.8 to 14. Both Riverbank Formation and Turlock Lake Formations crop out in the bed of the river at numerous locations between Goethe Park and Nimbus Dam.

![Figure 1. Location map of the Lower American River.](image-url)
2.2 Lower American River Thalweg Profiles

A plot of the thalweg profiles of the Lower American River is shown in Figure 2 and includes profiles from 1906, 1955, 1987, 1997 and 2002. This plot demonstrates the history of degradation in the river due primarily to the combined efforts of base level lowering at the Sacramento River and the construction of Folsom and Nimbus Dams. The more recent profiles show that bed materials are continuing to move through the system (since some deposition is seen in the downstream reach) and that certain locations are not moving indicating that bedrock is controlling further degradation.

The thalweg profiles (Figure 2), and the general lack of significant change in the thalweg elevations since 1987 indicate that the Lower American River is currently stable vertically. However, the presence of extensive outcrop of the erosion resistant Riverbank and Turlock Lake Formations (Shlemon, 1972) in the bed of the river at a number of locations is responsible for the apparent vertical stability. Given that there is effectively zero bed material inflow to the river downstream of the dams, the expected response of the river would be to further degrade to balance the sediment transport capacity with the sediment supply (Schumm, 1977; Williams and Wolman (1984). Since the erosion resistant outcrop is controlling the bed profile, and hence slope, balancing the sediment transport capacity and supply has to occur through either slope reduction or channel widening. Slope reduction could occur as a result of increasing the river sinuosity, but that is unlikely to occur because of the extensive distribution of existing bank revetments. Sediment transport capacity can also be reduced through channel widening that reduces the unit transport rates for a given slope. Since it is unlikely that the bed slope will be reduced significantly (assuming that the Riverbank and Turlock Lake outcrops will remain erosion resistant), the expected longterm response of the river will be general channel widening that will in turn remove or narrow the existing berms, thereby increasing the erosional threat to the levees. If further bed degradation does occur, the erosion potential for the berms and levees will be increased because of toe scour, and depending on the magnitude of the general degradation, the integrity of existing revetments could also be threatened.

3.0 HYDRAULIC MODELING

3.1 Model Background and Development

Extensive two-dimensional modeling of the lower 12 miles of the LAR has been conducted in recent years by SAFCA and the USACE using the RMA-2V computer model (Ayres 1999). The model has been used as a planning and management tool for addressing issues related to the American River parkway, such as floodway capacity, vegetation management, protection and enhancement of environmental and recreational resources, and erosion prevention.

The model of the lower 12 miles was initially constructed to represent conditions present during the 1997 flood season and was calibrated to the peak flow that occurred during January 1 and 2 of that year (108,000 cfs at the Fair Oaks Gage). A subsequent calibration was made to water surface elevations surveyed during a 1999 runoff event of 22,900 cfs to check calibration for a bankfull event. Some slight modifications were made to the initial roughness coefficients as a result of this bankfull calibration run.
Figure 2. Lower American River Thalweg Profiles

Lower American River Erosion Susceptibility Analysis for Infrequent Flood Events
Engineers/Scientists/Surveyors
July 9, 2004
Sacramento, CA

Vertical Datum Reference
The model has since been used for several purposes, including:
  o Incorporating bank protection and mitigation measures constructed at Sac Bank Sites 1-5 in the last few years in order to model "existing conditions"
  o Modeling the impacts of the proposed extension to the E.A. Fairbairn Water Treatment Plant intake structure
  o Simulating flow conditions related to proposed overbank modifications in the Howe to Watt reach of the river
  o Modeling of water tower impacts on the Sacramento River immediately downstream of the confluence with the Lower American River.

For this particular application, the Lower American River hydraulic model was modified to include the reach of river from RM 0 to RM 14 (a portion of the upper model was added to the lower model) and changes in the mesh were made to provide finer definition at locations of newly constructed revetments at Sac Bank Sites 1 through 5 (RM 2.1L, 3.75L, 4.8L, 6.8L, and 8.75R) and potential erosion sites based on an initial assessment.

A more complete technical discussion of the hydraulic model is included in the Appendix to this report.

3.2 Hydraulic Modeling Results

The two-dimensional model computes hydraulic values such as depth of flow, water surface elevation, and velocity. These results can be viewed spatially in the form of color contour plots. From the initial variables computed by the model, additional variables, including applied shear stress can be computed and plotted.

The appendix to this report contains the following color contour plots derived from the hydraulic modeling.
  o Applied Velocity Plots; 115K, 130K, 145K and 160K cfs.
  o Water Depth Plots; 115K, 130K, 145K and 160K cfs.
  o Applied Shear Stress Plots; 115K, 130K, 145K and 160K cfs.

The plots are shown on 9 plates with an aerial photograph background for all 13 miles of the lower river. These plots provide an "aerial atlas" of hydraulic conditions within the system.

3.3 Hydraulic Modeling Limitations

The two-dimensional model performed for this study and the associated output plots provide valuable data concerning the hydraulic conditions in the Lower American River for the modeled flows. However, the following cautions are given in using and applying the results of this modeling effort as presented in the Plates in the Appendix.
While the model is well suited as a tool for determining the likelihood of certain material sizes to mobilize under certain flow conditions, it cannot be used to determine where those materials will be transported to or any associated adjustments in channel planform or elevation. The results presented in this report are based on an incipient motion analysis, not detailed sediment transport modeling.

The results reflect steady state conditions for each of the flows modeled. The duration over which a flow condition occurs will impact the extent of mobilization and erosion that would occur at that flow. The time required for erosion to breach a levee cannot be estimated from this analysis.

The results provide detailed hydraulic information for quantifying conditions in the river channel for the range of flows modeled. The model refinement is not such that it can be used to micro-analyze conditions around an individual site location.

The model is based upon 1997 topography and does not account for changes in topography that may have occurred since the date of the mapping, except for the inclusion of bank protection sites 1-5, which have been constructed in recent years.

4.0 COMPUTATION OF CRITICAL SHEAR STRESS

The applied shear stress is the result of the actual hydraulic forces acting on a soil particle in the hydraulic environment, whereas the critical shear stress is the theoretical shear stress at which that particle will mobilize. Critical shear stress is dependent on two variables: (1) Shield’s parameter and (2) median grain size of the soil material. A technical discussion on how the critical shear stress was computed for this analysis is included in the Appendix to this report. Grain size for soil materials for this analysis were taken from soil boring information provided by the Corps of Engineers of the Lower American River Levees and available channel bed material samples (Ayres, 2001).

The following color contour plots are included in the Appendix.

- Critical Shear Stress Contours
- Ratio of Critical Shear Stress to Applied Shear Stress; 130K, 145K and 160K
- Ratio of 0.5 psf Shear Stress to Applied Shear Stress; 145K

The plots of the ratio of the critical shear stress to the applied shear stress show that most all of the soil particles in the overbanks and the levees are susceptible to mobilization if they lack cohesion, vegetative cover or other armor. The plots showing the comparison of 0.5 psf shear stress to the applied shear stress at 145,000 cfs were provided to demonstrate the effect that well managed vegetation can have in reducing the threat of erosion. However, the 0.5 psf value is believed to be the upper limit of shear stress for well managed vegetation to resist erosion (FHWA, 1988). This plot when compared to the previous set, provides a general overview of how vegetation can play a role in preventing soil erosion.

Due to the limited information in soils available, and the fact that vegetation is not considered in the equation to determine critical shear stress, the Critical Shear Stress plates should not be used for the evaluation of erosion potential on the levee.
5.0 REVIEW OF HISTORIC AERIAL PHOTOGRAPHS AND CHANNEL BEND MIGRATION PLOTS

An assessment of the historic bankline migration for the Lower American River between River Mile (RM) 0.0 and RM 14.5 was conducted as part of this study and was accomplished by delineating the bankline positions of the river on available time sequential aerial photography. Historic bank erosion has been limited on much of the Lower American River by bank protection.

Time sequential aerial photographs were obtained at various scales from a variety of sources. Aerial photographs taken in 1957, 1964, and 1972 were acquired from the Aerial Photo Field Office (APFO) of the USDA Farm Service Agency (FSA) in Salt Lake City, Utah. Aerial photos of the river taken in 1986 were acquired from the California Department of Water Resources. The 1998 aerial photos were acquired digitally from the MSN TerraServer world wide web site.

After creating the mosaics for each year and registering the various years together, the channel bend migration banklines were delineated and incorporated as a layer overlain on the 1998 aerial photo mosaic. The banklines are delineated with different colors on the 1998 aerial photos and are shown on plates included in the Appendix to this report.

An examination of the bankline overlays reveals that the banklines of the Lower American River have not naturally migrated to any significant degree. Based on our analysis of changes in river bankline position using these historic aerial photography for the Lower American River, it is concluded that the significant changes in bankline position are attributable to extensive sand and gravel mining operations that have taken place along the river since 1957.

A more detailed discussion of methodologies, site specific analyses and radius of curvature/channel width ratios are included in the Technical Report section of the Appendix to this report.

6.0 GUIDELINES USED FOR THE EVALUATION OF LOWER AMERICAN RIVER LEVEES

Guidelines for the evaluation of earthen riverine levees are provided in Chapter 7 of the FEMA “Flood Insurance Study, Guidelines and Specifications for Study Contractors” (FEMA, 1995). Of particular application to this analysis are the following two requirements summarized below.

- **Structural Design Analyses.** The Study Contractor must review the structural analyses, which address closures, embankment protection, embankment and foundation stability and settlement.
- **Operations.** In general, levee evaluation shall not consider human intervention (e.g., capping of levees by sandbagging, earthfill, or flashboards) for the purpose of increasing a levee’s design level of protection during an imminent flood.

As these requirements apply to this study, the results of the hydraulic analysis will be used to indicate if the effects of flows of 130,000 cfs, 145,000 cfs, and 160,000 cfs could cause erosion of the existing banks and levees.
Additional guidelines (and references) used in this study for documenting the initiation of erosion are as follows:

- Erosion of Bare, Fine Grained Sandy Soils: Velocity exceeding 2 fps (SCS, 1977) and (Corps, 1970)
- Erosion with Annual Grass Cover: Velocity exceeding 3.5 fps (SCS, 1954)
- Erosion with Grass-Lined Earth, Kentucky Blue Grass: Velocity exceeding 5 fps (Corps, 1970)
- Erosion of Dense Vegetation: Velocity exceeding 5 fps (FHWA, 1988)

An additional factor taken into consideration is the existing berm width between the top of the bank and the toe of the levee and the erodeability of the soil type making up the berm. This is primarily a judgment call, however berm erosion at RM 7.3 (opposite the Fairbairn Water Tower) exceeded 100 feet in the 1986 runoff event. For the purposes of this report, berms of less than 50 feet in width where there is active toe erosion are considered to be precarious (Ayres, 1997).

Given the limited geotechnical information of the levees and channel, which is used in the analysis of critical shear stress, the critical shear stress does not have enough detailed data to be a defining criterion. The critical shear stress, and ratios between critical and actual are provided for information and were not used as a guideline for evaluation.

7.0 FIELD INVESTIGATION

As part of this project a field investigation was performed that viewed the entire river reach of all the levees from both the water and landside. The waterside review (by boat) was completed on July 8th and a landside review (by driving the top of both levees) was performed on July 9th.

In attendance for both days of this review were:

- Thomas W. Smith Ayres Associates
- Michael D. Harvey Mussetter Engineering
- Ronald R. Copeland Mobile Boundary Hydraulics
- Daniel P. Tibbitts Corps of Engineers
- Henri Mulder Corps of Engineers

For the riverside review the following were also in attendance:

- Ric Reinhardt MBK Engineers
- Ray Costa Kleinfelder

For the landside review, we were joined by Paul Deveroux, American River Flood Control District, in the afternoon, which included the review of the north levee.

The primary purpose of the trip was to physically review each site where the hydraulic modeling showed velocities and shear stresses higher than allowable and to determine if that erosion would cause significant damage to the levee to the point where a failure is possible. As a result of the field review, 12 sites have been identified as possibilities and are described and discussed in the following section.
8.0 PREDICTED EROSION LOCATIONS FOR A FLOW OF 145,000 CFS

The locations of predicted erosion sites that are deemed to present a threat to the levees at a flow of 145,000 cfs are summarized in Table 1. The sites that are listed are usually of concern for flows of 130,000 cfs and 160,000 cfs, and some are of concern at 115,000 cfs. The table lists each site by river mile and identifies the location of the erosion and potential failure mechanism. These locations are shown in the Appendix, “Bank Material Type and Cross Section Location Plots”.

<table>
<thead>
<tr>
<th>River Mile</th>
<th>Erosion Location</th>
<th>Potential Failure Mechanism</th>
</tr>
</thead>
<tbody>
<tr>
<td>RM 1.8 - Left</td>
<td>Levee slope, velocity of 5 fps, toe erosion on lower riverbank, velocity of 8 fps</td>
<td>Erosion of levee waterside slope, toe erosion followed by upper slope failure into levee top width</td>
</tr>
<tr>
<td>RM 2.5 - Left</td>
<td>Levee slope, velocity of 4 fps</td>
<td>Erosion of levee waterside slope</td>
</tr>
<tr>
<td>RM 4.2 – Left</td>
<td>Levee slope, velocity of 5 fps</td>
<td>Erosion of levee waterside slope</td>
</tr>
<tr>
<td>RM 6.4 - Left</td>
<td>Levee and bank slope, velocity of 4 fps</td>
<td>Erosion of waterside levee and bank slope</td>
</tr>
<tr>
<td>RM 6.9 - Left</td>
<td>Levee slope, velocity of 6 fps</td>
<td>Erosion of waterside levee slope</td>
</tr>
<tr>
<td>RM 7.0 - Right</td>
<td>Overbank area at toe of levee slope, velocity of 6 fps</td>
<td>Erosion of overbank at toe of levee and levee waterside slope</td>
</tr>
<tr>
<td>RM 7.3 – Right</td>
<td>Overbank area, velocities 8 to 10 fps</td>
<td>Potential for large overbank scour hole and levee failure</td>
</tr>
<tr>
<td>RM 8.0 – Right</td>
<td>Levee slope, velocity of 4 fps</td>
<td>Erosion of waterside levee slope</td>
</tr>
<tr>
<td>RM 9.0 – Right</td>
<td>Existing erosion at toe of old cobble site, Velocity of 7 fps</td>
<td>Toe erosion, followed by upper bank slope failure</td>
</tr>
<tr>
<td>RM 9.7- Left</td>
<td>Levee slope, velocity of 7 fps</td>
<td>Erosion of waterside levee slope</td>
</tr>
<tr>
<td>RM 10.0 – Left</td>
<td>Riverbank slope and toe, velocity of 5 fps</td>
<td>Slope and toe erosion followed by slope failure into levee section</td>
</tr>
<tr>
<td>RM 10.2 - Right</td>
<td>Overbank and levee slope, velocity of 6 fps</td>
<td>Erosion of the overbank and waterside levee slope</td>
</tr>
</tbody>
</table>

9.0 DISCUSSION OF PREDICTED EROSION SITES

This section describes each of the predicted erosion sites and includes a channel cross-section plot showing the water surface elevation, velocity and shear stress for flows of 115,000 cfs, 130,000 cfs, 145,000 cfs, and 160,000 cfs. A brief discussion of each site as it was viewed on July 8th and 9th and a discussion of predicted mode of erosion and
potential failure mechanism is also provided. The following discussion of each site is focused on a flow of 145,000 cfs, however, most of these sites are of concern for flows of 130,000 cfs and 160,000 cfs, and some are of concern at 115,000 cfs.

9.1 RM 1.8 – Left

This site is located immediately downstream of the Sac Bank Site 1 repair project and is also the location of a slurry wall leak through the levee into the river. This site was also classified by SAFCA as an anticipatory erosion site in 1999 prior to the installation of the slurry wall and erosion was documented at the toe of the slope at that time.

Since the slurry wall leak, the levee above the water line (at the time of the repair) has been rebuilt and the outer face covered with quarry rock down to the waterline. No rock was placed below the waterline.

There is no waterside berm remaining at this location and the waterside slope is estimated to be steeper than 2:1 from the top of the levee down to the water. The condition of the toe is unknown but since this is an old cobble revetment site, it is likely that there is no toe protection or toe trench.

Velocities at the toe of the riverbank are approximately 8 fps for the 145,000 cfs flow and shear stresses are about 0.4 psf. (See Figure 3). This is an increase of 1.0 to 1.5 fps over the velocity for the 115,000 cfs flow. Velocities against the unrocked portion levee slope have also increased to 5 fps. Flow depth increases by about 2 feet at this location for the 145K over the 115K flow.

It is our opinion that the steep configuration of the bank makes this slope very close to a safety factor of 1.0 for the existing slope stability. The most likely failure mechanism at this site is seen as toe erosion followed by a large upper slope failure that will reach into the top width of the levee. Photograph 1 shows the left bank with bare toe and quarry rock up the slope.

![Photograph 1. Left Bank of RM 1.8 showing the Quarry Rock and Bare Toe.](image-url)
Figure 3 - Lower American River
Cross Sections Showing Elevation, Velocity, and Shear Stress
RM 1.8 (looking downstream)
9.2  RM 2.5 – Left

The lower riverbank at this site is protected by an older cobble layer and shows no signs of distress. It appears to have been stable since it was installed. The levee slope at this location is protected by mowed annual grasses. There is no evidence of erosion at this site, however the hydraulic model indicates a computed velocity of 4 fps, which is above the allowable velocity of 3.5 fps for annual grasses. The increase in velocity at this site for the 145K flow over the 115K flow varies from 0.5 to 1.5 fps and the increase in water depth is 2 feet.

Figure 4 shows a river cross section at this location that includes water surface, velocity and shear stress for the 145K flow. The erosion mechanism at this site will start with flows eroding away the vegetative cover followed by a more rapid erosion of the fine grained sandy soils of the levee.

9.3  RM 4.2 – Left

This site is located upstream of the Capitol City Freeway Bridge and is at the location of an emergency bank repair following the 1986 flood event. The riverbank below the levee is protected with quarry rock (post 1986 emergency repair). Protection on the levee slope is mowed annual grasses. Figure 5 shows velocities of approximately 5 fps on the levee slope, which exceed the allowable of 3.5 fps. The increase in velocity in this area from the 115K to the 145K flow is 0.5 to 1.0 fps and the increase in depth is 2 feet.

The erosion mechanism at this site will start with flows eroding away the vegetative cover followed by a more rapid erosion of the fine grained sandy soils of the levee. Photograph 2 shows this reach of river, just upstream of the Capitol City Freeway Bridge.

Photograph 2. Looking Downstream at the Left Bank of RM 4.2, near the Capitol City Freeway Bridge.
Figure 4 - Lower American River
Cross Sections Showing Elevation, Velocity, and Shear Stress
RM 2.5 (looking downstream)
Figure 5 - Lower American River
Cross Sections Showing Elevation, Velocity, and Shear Stress
RM 4.2 (looking downstream)

Legend
- 180,000 cfs
- 145,000 cfs
- 130,000 cfs
- 115,000 cfs
9.4  RM 6.4 – Left

This site is located in a high energy reach of the river downstream of H Street and is at the location of a 1964 flood fight. The riverbank below the levee is protected by cobbles. The levee proper is protected by mowed annual grasses. Figure 6 shows a cross section of the river at this site and a velocity on the levee and bank slope of approximately 4 fps, which is slightly above the allowable of 3.5 fps. The increase in velocity for the 145K flow over the 115K flow varies from 1 to 2 fps and the increase in water depth is 2.5 feet.

The erosion mechanism at this site will start with flows eroding away the vegetative cover followed by a more rapid erosion of the fine grained sandy soils of the levee. Photograph 3 shows this reach of river, showing new vegetation growth since the last flood event.

![Photograph 3. Looking Downstream along the Channel Reach at RM 6.4](image-url)
Figure 6 - Lower American River
Cross Sections Showing Elevation, Velocity, and Shear Stress
RM 6.4 (looking downstream)
9.5  RM 6.9 - Left

High velocities at this site are due to the location on an outside of a bend and from narrowing of the channel width. The cross section in Figure 7 shows a velocity of approximately 6 fps against the left levee slope. Photograph 4 shows this reach of the river in a view taken from the Guy West Bridge, which is immediately upstream.

Photograph 4. Looking Downstream at the Channel Reach at RM 6.9 - Left

The increase in velocity for the 145K flow over the 115K is 1.0 to 2.5 fps and the increase in water depth is 3 feet.

The erosion mechanism at this site will be similar to the other levee locations except this site has even higher velocities on the levee and erosion will proceed more quickly.
Figure 7 - Lower American River
Cross Sections Showing Elevation, Velocity, and Shear Stress
RM 6.9 (looking downstream)
9.6 RM 7.0 - Right

This site has high velocities and shear stress in the right over bank and on the right side levee slope for the 145K flow. The river cross section shown in Figure 8 indicates velocities of 6 fps for most of the overbank and the levee slope. Photograph 5 shows this area as viewed from the Guy West Bridge. Ground cover consists of mowed annual grasses and scattered trees.

Photograph 5. Looking Downstream from Guy West Bridge at Right Levee and Overbank Area

The velocity increase for the 145K flow over 115K varies from 1 to 2 fps and the increase in depth is 3 feet.

The erosion mechanism at this site will be a combination of overbank erosion and erosion along the lower levee slope. Erosion will proceed much quicker after the vegetation is eroded away.
Figure 8 - Lower American River
Cross Sections Showing Elevation, Velocity, and Shear Stress
RM 7.0 (looking downstream)

Legend

- 180,000 cfs
- 145,000 cfs
- 130,000 cfs
- 115,000 cfs
9.7   RM 7.3 – Right

This site is at the location of the force main sewer line crossing and was an emergency repair site following the 1986 runoff event. Immediately upstream of this site, more than 100 feet of the bank line was eroded. Subsequent repairs consisted of a transverse buried rock dike in the right overbank to protect the sewer line. Photograph 6 shows a view looking upstream from the end of the transverse rock dike towards the 1986 upstream erosion site (the intake structure is to the right of the photograph). Photograph 7 shows the hardpan after the 1986 high flows.

Photograph 6. Looking Upstream along the Right Bank from RM 7.3

Photograph 7. Looking at the Bank Erosion along the Right Bank from RM 7.3 after the 1986 Storm Event
The velocities in the overbank areas just downstream of the buried transverse rock dike are in the 8 to 10 fps range with shear stress of 1.0 to 2.5 psf as can be seen in Figure 9. Velocity increases over the 115K flow are 1 to 1.5 fps and shear stress increases vary from 0.1 to 0.2 psf. Flow depths increase by 3 feet. Also, the velocities against the levee slope are approximately 4 fps.

The exact failure mechanism is difficult to predict at this location due to the complicated hydraulic conditions. Our best judgment is that overbank erosion will occur, creating a large scour hole on the downstream side of the transverse rock dike. How large the scour hole becomes and how close it gets to the levee will depend on the duration of the peak flow.

9.8 RM 8.0 – Right

The river cross section at this site is shown in Figure 10 and the erosion concern is for velocities of 4 fps against the right levee. Erosion mechanism and mode of failure is similar to the previous sites with levee erosion. Velocity increases over the 115k flow are approximately 1 fps and depth increases by about 3 feet. Photograph 8 shows the right bank of RM 8 with the Howe Ave. Bridge in the background.
Figure 9 - Lower American River
Cross Sections Showing Elevation, Velocity, and Shear Stress
RM 7.3 (looking downstream)
Figure 10 - Lower American River
Cross Sections Showing Elevation, Velocity, and Shear Stress
RM 8.0 (looking downstream)

Legend

- 160,000 cfs
- 145,000 cfs
- 130,000 cfs
- 115,000 cfs
9.9 RM 9.0 – Right

This location is typical of the reach of river upstream of the existing Site 5 repair and downstream of Watt Avenue. This site was an old cobble site and shows the worst location with active erosion from this past year. The peak flow from the 2002-2003 runoff year was 7,000 cfs (USGS website) and has caused new erosion to occur and at least one tree to overturn. Photograph 8 shows the river bank site at RM 9.0-R (July 8, 2003).

Photograph 8. Looking Downstream at the Right Bank at RM 9.0

The soils in this reach are fine grained sands and silts and highly susceptible to erosion. Velocities along the toe and lower bank for the 145K flow are in the 5 to 7 fps range as can be seen from the cross section plot in Figure 11. Some of the remaining trees are unstable and will overturn with further erosion leaving an unprotected soil surface on the bank. The increase in velocity from the 115K flow to 145K varies from 0.5 to 1.0 fps and the increase in depth is 3.5 feet.

The most likely failure mechanism at this site will be the continuation of toe erosion, overturning of the remaining large trees followed by a large slump failure or quick erosion of the fine grained overbank soils. There is approximately 80 feet of berm remaining at this site and while one single high flow event may not reach the levee, a large erosion scar will remain and require an immediate repair before the next flow.
Figure 11 - Lower American River
Cross Sections Showing Elevation, Velocity, and Shear Stress
RM 9.0 (looking downstream)
9.10  RM 9.7 – Left

The river cross section shown in Figure 12 indicates velocities of 7 fps on the left bank along the levee face. Cover on the levee consists of mowed annual grasses. Photograph 9 shows a view of the site, looking in the upstream direction taken on the day of our field investigation (July 9, 2003).

Photograph 9.  Looking Upstream at the Levee Slope at RM 9.7 – Left

The site currently does not show any signs of distress but modeling shows that the velocity against the levee slope will be increased by from 0.5 to 1.5 fps in this area for a flow of 145K over a flow of 115K. Water depths will increase by approximately 3.5 feet.

The most probable mechanism of failure at this site will be erosion and removal of the grass cover followed by more rapid erosion of the fine grained sandy levee soils. Water depth for the 145K flow is approximately 8 feet at the toe of the levee.
Figure 12 - Lower American River
Cross Sections Showing Elevation, Velocity, and Shear Stress
RM 9.7 (looking downstream)
9.11 RM 10.0- Left

The site at RM 10.0 – L is characterized by a steep riverbank, a very small berm and evidence of active erosion on the toe and lower riverbank. Photograph 10 shows a view of the site on the day of our field investigation (July 8, 2003).

Photograph 10. Looking Downstream at the Erosion Site at RM 10.0 - Left

Velocities at the toe and lower riverbank are approximately 4 fps for the 145K flow, which is 1 to 2 fps higher than for the 115K flow. Flow depth increases by approximately 3.5 feet for 145K.

The lower bank slope is steep (steeper than 2:1) and is vegetated with large trees and vines for the most part. However there is evidence of human usage, which leaves areas bare with no cover, which will be susceptible to erosion. There is a small waterside berm remaining, which was estimated to be about 20 feet wide.

The most likely failure mechanism at this site will be toe erosion followed by a large upper slope failure that may reach into the levee section.
Figure 13 - Lower American River
Cross Sections Showing Elevation, Velocity, and Shear Stress
RM 10.0 (looking downstream)

Legend
- 160,000 cfs
- 145,000 cfs
- 130,000 cfs
- 115,000 cfs
9.12 RM 10.2 – Right

The velocities in the overbank and at the toe of the levee are approximately 6 fps as can be seen from the river cross section in Figure 14. The high velocities extend over most of the overbank, which is covered by mowed annual grasses with isolated trees as shown in Photograph 11 (July 9, 2003).

Photograph 11. Looking Downstream at RM 10.2 showing Levee and Overbank Area

Velocity increases for the 145K over the 115K flow are approximately 0.5 to 1.0 fps for the overbank area and 1.0 to 1.5 at the toe of the levee. Depth is increased by approximately 3.5 feet.

This location is also characterized by a narrowing down of the total floodplain width between the levees and some of the increase in velocity may be the result of the access road that slopes down into the overbank area as shown in the photograph.

The mechanism of failure at this site will be overbank and levee toe erosion of the annual grass followed by more rapid erosion of the levee soils.
Figure 14 - Lower American River RM 10.2 (looking downstream)

Cross Sections Showing Elevation, Velocity, and Shear Stress
10.0 CONCLUSIONS

Based upon our modeling efforts, field review and overall experience with the Lower American River system, we offer the following conclusions:

1. Geomorphic principles, the thalweg profile, and the field review all agree that the river system is degradational under present operating conditions.

2. The Lower American River is starved of sediments by Folsom and Nimbus Dams. Bedrock has been reached in the channel bottom as far downstream as Guy West Bridge, and this bedrock is slowing further degradation. With the river starved for sediments and without significant bed slope reduction, it will now tend to erode laterally to satisfy the need for sediment.

3. The hydraulic modeling shows areas of riverbank and levees where allowable velocities for vegetative cover and soil materials are exceeded. Twelve (12) priority sites have been identified. These sites need to be evaluated in more detail to determine if a levee failure is likely to occur.

4. The field review verified that erosion of the riverbank is occurring (RM 9.0R) even at low flow conditions of 7,000 cfs, which was the peak flow from the 2003 runoff season. Erosion on the American River is continually occurring. This condition is leaving the channel banks scarred and susceptible to further erosion, especially during a high flow event. In addition, this condition is further reducing the amount of berm separating the main channel from the levee. The loss of underlying vegetation is leaving bare soil, which is susceptible to erosion at a lower velocity.

5. It is our opinion that a flow of 145,000 cfs could cause some damage at most of the 12 designated sites and possibly have a levee failure occur for at least one of the sites.
11.0 REFERENCES


Soil Conservation Service (SCS), June 1987, WNTC Technology Transfer Note, Portland, OR.

COMMENTS AND RESPONSES

Mobile Boundary Hydraulics’ Comments with Ayres Associates’ Responses

Mussetter Engineer Inc. Comments with Ayres Associates’ Responses
August 7, 2003

Mr. Thomas Smith
Ayes Associates
2151 River Plaza Drive
Sacramento, CA 95833

SUBJECT: Lower American River FEMA Certification, Comments on Draft Report
Dated July 21, 2003

Ayres Associates’ responses are shown in bold and italic. Dated February 12, 2004

Backchecked responses by Mobile Boundary Hydraulic to Ayres Associates’ responses are in red. Dated March 30, 2004

Ayres Associates’ responses to the backcheck responses are shown in blue. Dated April 1, 2004

General

1. The Draft Main Report and Draft Appendix of the Lower American River Certification Study dated July 2003 have been reviewed by Dr. Ronald R. Copeland of Mobile Boundary Hydraulics, PLLC. The study was conducted by Ayres Associates for the Sacramento District of the U.S. Army Corps of Engineers. The following comments are based on the contents of the Draft Main Report, Draft Appendix and a field trip conducted with the study team on July 8 and 9, 2003.

Response: Comment noted. No response needed.

2. The numerical model calculations themselves were not reviewed as part of this technical review. However, based on information provided in the draft report, I conclude that the methodology used in the study for calculating velocities and shear stresses in the lower 14 miles of the American River is appropriate for the intended study purpose, which was to evaluate the potential for erosion and levee failures at a discharge of 145,000 cfs. The allowable velocity of 3.5 fps assigned in the Draft Report for the annual-grass-lined levee is reasonable. With the following minor adjustments, it is my judgment that technical details in the draft report support the study conclusions.

Response: Comment noted. No response needed.
Conclusions

3. Draft Report, page 31, Conclusion 1: It is not clear how the steady state hydraulic model results are used to indicate that the river is degrading. This should be explained in the body of the report.

Response: We agree and have revised this conclusion to remove the words “modeling results” – it is the geomorphologic principles, the thalweg, and field review that are the indicators of further degradation.

4. Draft Report, page 31, Conclusion 2 - Last sentence: I would argue that lateral erosion will not occur because the river has a "need" for sediments. Lateral erosion is also characteristic of braided streams that have an abundance of sediment. In the case of the American River there may be a lateral erosion trend because widening is the only unrestrained adjustment process available to reduce excess stream energy.

Response: We agree and have added a more explanation in a new paragraph to section 2.2 “Lower American River Thalweg Profiles” that goes into further details on lateral erosion of the river. We have also reworded conclusion 2 for clarification.


Response: We reviewed a cross section plot showing velocities at this location and found the maximum against the levee and on the berm for the 145k flow to be 3 fps, which is lower than the 3.5 ft/s maximum mentioned in Section 6 “Guidelines used for the Evaluation of Lower American River Levees.” As a result, this site will not be added.

Response: We recommend that this site (RM 4.6-4.9) be included as a priority site. It may be true that the calculated velocities are slightly less than 3.5 ft/sec, however, based on field observations and channel geometry we feel that this site has potential for levee erosion. The site is located on the outside of a bend. The channel narrows at this location and a point bar is forming on the opposite bank. These factors will tend to force flow to the right side of the channel. There are no trees between the channel and the levee at RM 4.6 (from aerial photo in the appendix). The berm between the levee and channel is narrow. Calculated velocities are very useful in determining potential erosion sites, but one must bear in mind the uncertainties associated with the numerical calculations, especially at the boundaries. We suggest not relying solely on calculated results for determining erosion potential.

Response: The lower bank has been repaired (LAR Site 3) and an extensive re-vegetation planting has been done at this site, which when mature will further reduce velocities against the levee. We acknowledge that this site is a close call but it does not meet the minimum criteria defined in the scope.

Draft Main Report

6. On page 4 of the Draft Report, the Lower American River thalweg profiles are
discussed. It is not intuitive how the profiles show that bed materials are continuing to move through the system. Please explain.

**Response:** This section has been re-written and includes additional discussion and explanation.

Response: What we were looking for is a statement something like “Since the thalweg plots show deposition at some cross sections between surveys, it can be concluded that bed materials continue to move through the system.”

**Response:** Agree, we have added to the section to say that deposition is shown from the recent thalwegs in the downstream reach.

7. At the top of the page 9 of the Draft Report, in the second paragraph, it is stated that 13 potential erosion locations were identified during the field trip. Twelve sites are listed in Table 1. The site at River Mile 4.8(4.6 to 4.9) on the left (south) side of the river is missing from the table. The concern here is that the annual-grass-lined levee is vulnerable to erosion by overbank velocities. Calculated velocities against the levee are between 3 and 4 fps but could be greater due to the lack of a vegetation buffer between the river and the levee and the location of this site on the outside of a bend. A gravel bar on the opposite bank indicates that flow is concentrated against this side of the river at high flow.

**Response:** The site at RM 4.8 was removed because it did not fit the criteria for a predicted erosion site. The number of sites has been changed to 12. Also see response to comment 5 above.

Response: Same response as response to comment 5

**Response:** See response to comment 5.

8. The discussion of erosion potential at River Mile 1.8 is discussed on page 10 of the Draft Report. The primary concerns of toe and slope failure are stated. It is suggested that the following be added to include a secondary concern at this site: Calculated velocities against the levee itself are between 4 and 5 fps, exceeding the allowable velocity of 3.5 fps for annual-grass-lined levee.

**Response:** We have reviewed the plot and agree with the comment. A revision has been made to this section to include the high velocities against the levee.

9. In the discussion of erosion potential at River Mile 6.4 on page 12 of the Draft Report, I would add that there is no vegetation between the river and levee to slow impinging flows, and the berm at this location is very narrow.

**Response:** Our photo documentation of this site shows trees within the riverbank armor, however this site is still considered a predicted erosion site. A photo has been added to this section of the report.

Response: Look at aerial photo on Plate 5 “Bank Material Types and Cross Section Locations.” It looks like there are no trees along the bank for most of this reach.
Response: The aerial photographs are from the year 1997; we have included a photograph from 2003 showing that vegetation has re-established in this area since the time of the aerial photography.

10. There are two potential problems at River Mile 7.3 (Page 20 of the Draft Report). Discussed in the draft report is the local scour problem predicted by the numerical model downstream from the transverse rock dike. Suggest adding that overbank velocities of 4-6 fps are predicted against the right annual-grass-lined levee downstream from this location. These exceed the allowable velocity of 3.5 fps.

Response: We have reviewed the velocity plot and agree. A statement has been added.

Draft Appendix

11. Critical Shear Stress is discussed in the Appendix on pages 4-6. Critical shear stress was determined for this study using the Shields diagram from ASCE (1977) and by assuming $d_{50}$ as the characteristic grain size. The Shields diagram was developed using flume data with uniform sands. Subsequent work using sediment mixtures has shown that the Shields parameter is significantly less in sediment mixtures when $d_{50}$ is used as the characteristic grain size. Instead of a Shields parameter of 0.060 for gravels, the Shields parameter should be closer to 0.047 (Meyer-Peter and Muller 1948) or 0.030 (Neill 1968). This suggests that the critical shear stresses calculated in the draft report are probably too high.

Response: Comment noted, however in the calculation of critical shear stress, we acknowledge that there are different approaches to the methodologies. However, these plots are provided for information and not used as evaluation criteria.

Response: Section 6.0 of the main report describes the guidelines used for evaluation of the Lower American River Levees. It is not clear from this section exactly what the evaluation criteria are. If the critical shear stresses are not used as evaluation criteria, then that fact and the reasons should be clearly stated in Section 6.0 and in the Appendix.

Response: We have added a sentence mentioning that critical stress is not a guideline and only provided for information.

12. Table 2 of the Appendix provides a summary of median grain sizes used to calculate critical shear stress. The median grain size alone may not adequately define the critical shear stress of a sediment. Some of the median grain sizes shown in the table are silts. Silts have cohesive properties that increase the critical shear stress. Even if the median grain size is in the sand range, it is possible for there to be sufficient cohesive material in the sediment mixture to increase critical shear stress. For example a mixture with only 10 percent clay will produce a very erosion resistant material. An adequate analysis requires that the entire gradation be considered. Criteria for several soil types are available in EM 1110-2-1418 page 5-4, Figure 5-4.

Response: We agree that clay content can make a difference in erosion resistance. However the largest problem with this approach was the limited amount of soils data. Also, this approach does not account for the vegetation
cover, which can also make a difference. Because of these limitations, we do not recommend pursuing this methodology as a defining criterion.

Response: We concur with the conclusion that this methodology should not be pursued. This fact should be stated in the Appendix and in Section 6.0.

Response: We agree, and have added a sentence about the limited geotechnical information and that the critical shear stress should not be a defining criteria for the integrity of the system.

13. Plates are provided in the Appendix that compare applied and critical shear stresses. When the ratio of applied to critical shear stress is less than unity the bed is said to be immobile. These plates show the applied shear stress to be less than critical in the entire river channel upstream from river mile 2.0, even for the high flows modeled in this study. This implies that the entire channel upstream from river mile 2.0 is armored. Observations during the field trip and geomorphic evidence presented elsewhere in the report contradict this conclusion. The apparent contradiction may be attributed somewhat to the high Shields parameter used in the study, but more likely is due to the assignment of a bed grain size that is too large to be representative of a long reach of river. Better definition of bed material gradations is necessary to make these plates useful. Since these plates are not necessary to support report conclusions and because they may lead to incorrect interpretations, I suggest they be removed from the report.

Response: The thought when these were put in the scope was that they would provide more valuable data, however due to a limited source of raw data, the results are not conclusive and should not be used for final analysis. Since they are in the scope they will remain in the report as a deliverable.

Response: Clearly state in the report that these plates were not used in the final analysis.

Response: We agree, and a sentence has been added in section 4 about them not being used.

14. The Appendix plates that show existing bank protection are very useful, but difficult to read. Suggest using symbols in addition to colors. Especially differentiate between red and magenta.

Response: Comment noted, we agree that symbols would be a nice touch but it is beyond what we negotiated for this task.

Comments on Items Not Covered in the Draft Report

15. During the field investigation several locations were noted where the channel bed consisted almost entirely of what the report identified as Modesto formation. The stability of the American River at these locations and upstream from these locations is highly dependant on the strength and thickness of these formations. Henri Mulder from Sacramento District recollected that the formation may be 10-15 ft thick at Guy West Bridge. I suggest that the report recommend that the stability of this formation be evaluated in a future study by determining erosion rates and thicknesses of the formation.
Response: We agree with this comment, but it is outside of the scope of this report. The time frame for erosion of the Modesto formation is outside the realm of the FEMA certification process.

16. The Draft Report does not provide technical information addressing the issue of riprap stability. The Plates entitled "Bank Material Types and Cross Section Locations" provide valuable information relative to location of bank protection. According to the legend on those plates, rock-lined reaches have rock greater than 20 inches or between 12 and 20 inches. It is not clear if this dimension is the blanket thickness or the median stone size. Table 2 of the Appendix lists riprap $d_{50}$ as 208 mm (8.2 inches) so it is assumed that the legend is referring to thickness. I made calculations for riprap size for the design discharge of 145,000 cfs at River Mile 6.9 using the procedure outlined in EM 1110-2-1601. Input variables were: a) the average velocity and depth from Figure 7 of the Draft Report (8.5 fps and 36 ft), b) the radius of curvature to width ratio from Table 4 of the Appendix (7.1), c) a safety factor of 1.1, d) angular rock Cs = 0.3, Ct= 1.0, $S_0 = 165$ #/ft^2 and $K_1 = 0.7$. Some of these input parameters were assumed due to lack of actual design information. My calculations indicate a minimum $d_{30}$ of 8 inches. This puts the existing riprap right at design size if Table 2 in the Appendix is correct. Cobble-lined banks would require a $d_{30}$ of 10 inches at River Mile 6.9. No information is provided in the Draft Report relative to riprap toe depth or how scour adjacent to the banks was estimated. I am especially concerned about the size and toe protection at cobble-lined sites. Perhaps this issue has been addressed in other studies or design reports are available to confirm the stability of the toe and side slope protection works. Technical work that establishes the stability of bank protection works should be referenced in this report. Without this backup information it is not possible to comment on the reliability of existing bank protection.

Response: We agree, however riprap stability is outside of the scope of work.

Response: Riprap stability is a key element in the evaluation of potential for erosion and levee failures. Since riprap analysis is outside the scope of work for this report, Section 1.3 “Purpose and Scope of Project” should be revised to reflect this fact. A possible restatement of the purpose could be - The primary purpose of this project was to evaluate the potential for erosion of grass-lined levees and overbanks as a result of increasing …….. The project purpose should be revised in paragraphs 2 and 3 on page 2 and paragraph 1 on page 3. If a study has been done that confirms riprap stability it can referenced.

Response: We agree, and have edited the report to clarify.

17. Erosion resistance along the levees could be increased by planting the slopes with a resistant grass and irrigating. Some of the potential levee erosion sites could be eliminated with this improvement. The report may want to make this recommendation.

Response: Good point, but recommendations for fixes are outside our scope of work.
Minor Corrections

18. A few grammatical corrections are suggested.

a. Draft Report, page 1, third line - add by between provided and these, so that the sentence reads Flood control for the City of Sacramento is provided by these dams
Response: Agree, change made.

b. Draft Report, page 3, first paragraph- the two sentences "The combined effect of base level lowering resulted from recovery of channel bed elevations in the Sacramento River by about 1930 (Meade 1982). Sand, gravel mining, and dam construction has been channel degradation and possibly exhumation of pre-existing bedrock topography within the reach." are unclear. Have these been misprinted from the original?
Response: This is not a misprint, it is a direct quote that includes a poorly written sentence, we talked to the author and parenthesis have been added to try and clarify this sentence.

c. Draft Report, page 4, first paragraph- Replace plots with profiles in the last sentence, so that it reads: The more modern profiles show that
Response: Agree, change made.

d. Draft Report, page 20, line 2: Replace that with than.
Response: Agree, change made.

e. Draft Report, page 20, first sentence in paragraph 2: add just before downstream so that the sentence reads "The velocities in the overbank area, just downstream of the buried transverse rock dike, are in the…"
Response: Agree, change made.

f. Draft Report, page 31, Conclusion 2 second sentence: replace which is with". These outcrops are". It is the bedrock that is slowing degradation, not the Guy West Bridge.
Response: Agree, change made, but we used the word “bedrock” rather than “outcrop”.

g. Appendix, page 6, fourth line: Capitalize Shields
Response: Agree, change made.

Sincerely,

Ronald R. Copeland, P.E. PhD

19. In Section 3.1 on page 5, it would be useful to include the calibration discharge from January 1997.

Response: We agree, the discharge has been added.
Ayres Associates’ responses are shown in bold and italic. Dated February 12, 2004

Remarks:

As requested in our Scope of Work, I conducted the field inspection of the Lower American River on July 8 and 9, 2003 (Task k – Field Investigation), and I have reviewed the Draft Project Report (Task m – Technical Review). The following are my technical review comments on the draft report. I have also made a number of editorial suggestions in the report itself, and I am providing you with my marked up copy of the report.

1. Background Section (p.3)

I suggest you replace the second paragraph on page 3 with the following text that emphasizes the locations of the Pleistocene-age outcrop within the project reach. Shlemon (1972) identified the spatial distribution of the Pleistocene and Holocene-age formations that represent the paleo-American River sediments that are currently affecting the vertical and lateral stability of the Lower American River (LAR) between the Sacramento River confluence (RM 0) and Nimbus Dam (RM 22). The formations include Modesto (youngest and least erosion resistant), Riverbank (consolidated and erosion resistant) and Turlock Lake (cemented and most erosion resistant). In general, the youngest formation crops out downstream and the older ones farther upstream. Previous (Ayres, 1997) and current field inspection of the LAR indicate that Modesto Formation cropped out in the bed of the river at about RM 4.5 and 5.8, but the river was capable of eroding through the outcrop at these locations. Modesto Formation outcrop...
currently forms the right bank of the river at RM 5.5. and it appears to be retarding river migration at this location. The more erosion resistant Riverbank Formation is currently exposed in the bed of the LAR at RM 7 to 7.3, RM 9.4 to 10, RM 10.9, RM 11.6 and from RM 13.8 to 14. Both Riverbank Formation and Turlock Lake Formations crop out in the bed of the river at numerous locations between Goethe Park and Nimbus Dam.

Response: Agree, paragraph added, old paragraph removed.

2. Lower American River Thalweg Profiles (p. 4)

I suggest that you add the following paragraph to this section of the report.

The thalweg profiles (Figure 2), and the general lack of significant change in the thalweg elevations since 1987 indicate that the Lower American River is currently stable vertically. However, the presence of extensive outcrop of the erosion resistant Riverbank and Turlock Lake Formations (Shlemon, 1972) in the bed of the river at a number of locations is responsible for the apparent vertical stability. Given that there is effectively zero bed material inflow to the river downstream of the dams, the expected response of the river would be to further degrade to balance the sediment transport capacity with the sediment supply (Schumm, 1977; Williams and Wolman (1984). Because the erosion resistant outcrop is controlling the bed profile, and hence slope, balancing the sediment transport capacity and supply has to occur through either slope reduction or channel widening. Slope reduction could occur as a result of increasing the river sinuosity, but that is unlikely to occur because of the extensive distribution of existing bank revetments. Sediment transport capacity can also be reduced by channel widening that reduces the unit transport rates for a given slope. Since it is unlikely that the bed slope will be reduced significantly (assuming that the Riverbank and Turlock Lake outcrops will remain erosion resistant), the expected longterm response of the river will be general channel widening that will in turn remove or narrow the existing berms, thereby increasing the erosional threat to the levees. If further bed degradation does occur, the erosion potential for the berms and levees will be increased because of toe scour, and depending on the magnitude of the general degradation, the integrity of existing revetments could also be threatened.

Response: Agree, paragraph added

3. Computation of Critical Shear Stress (p.6,7)

I suggest that you make reference to the bed material samples that were used for determining the critical shear stress in the channel. Rather than using the borings to determine the soil characteristics for the overbank areas, I suggest you use the engineering properties for the mapped soil series. The spatial distribution of the mapped series could be used to improve the resolution of your model results.


4. Guidelines Used for the Evaluation of Lower American River Levees (p.8)

I suggest you reference the Ayres (1997) report to support your assumption of a 50 foot berm width. The same width was used as one of the criteria for prioritizing erosion sites
on the Lower American River in a prioritization approach that was agreed to by the Corps and SAFCA, and was used to identify the 5 sites that have been recently revetted.

Response: Agree, reference added.

5. Predicted Erosion Locations for a Flow of 145,000 cfs (p.9)

I suggest you add a reference to Table 1 that indicates that the potential site at RM 9R is an old cobble revetment with significant loss of toe protection.

Response: Changed the description from “existing erosion at toe of riverbank” to “existing erosion at toe of old cobble site”
ITRT CERTIFICATION
June 27, 2004

Mr. Thomas Smith
Ayres Associates
2151 River Plaza Drive
Sacramento, CA 95833

SUBJECT: Lower American River FEMA Certification, Certification statement for Report

1. The Draft Main Report and Draft Appendix of the Lower American River Certification Study dated July 2003 have been reviewed by Dr. Ronald R. Copeland of Mobile Boundary Hydraulics, PLLC. The study was conducted by Ayres Associates for the Sacramento District of the U.S. Army Corps of Engineers. Review comments were provided by letter on August 7, 2003. Responses by Ayres Associates were dated February 12, 2004. Mobile Boundary Hydraulics provided backchecked responses on March 30, 2004. Ayres Associates provided responses to the backchecked responses on April 1, 2004. This correspondence is provided as an attachment.

2. The technical review by Mobile Boundary Hydraulics was based on a content review of the Draft Main Report, the Draft Appendix, and observations and discussions from a field trip conducted with the study team on July 8 and 9, 2003. The numerical model calculations themselves were not reviewed as part of this technical review. However, based on information provided in the draft report, and the responses to review comments, I conclude that the methodology used in the study for calculating velocities and shear stresses in the lower 14 miles of the American River is appropriate for the intended study purpose, which was to evaluate the potential for erosion of grass-lined levees and overbanks at discharges of 115,000, 130,000, 145,000, and 160,000 cfs. The allowable velocity of 3.5 fps assigned in the Draft Report for the annual-grass-lined levee is reasonable. It is my judgment that technical details in the draft report support the study conclusions.

3. It is acknowledged that there is a difference in judgment related to the inclusion of River Mile 4.8 as a priority site for potential levee failure. Based on the model results at this location, Ayres Associates determined that the maximum velocity against the levee and on the berm for the 145k flow was 3 fps, which is lower than the 3.5 ft/s maximum mentioned in Section 6 “Guidelines used for the Evaluation of Lower American River Levees.” In addition, the lower bank at this site has been repaired, and vegetation has been planted. As a result, this site was not included as a priority site. However, based on field observations and channel geometry, it is my judgment that this site has potential for levee erosion. The site is located on the outside of a bend. The channel narrows at this location and a point bar is forming on the opposite bank. These factors...
will tend to force flow to the right side of the channel. Currently, there are no mature trees between the channel and the levee at RM 4.6. The berm between the levee and channel is narrow. Calculated velocities are very useful in determining potential erosion sites, but one must bear in mind the uncertainties associated with the numerical calculations, especially at the boundaries.

4. Recommendations for additional study to increase confidence in the reliability of the American River levees that are outside the scope of work for this project and therefore not included in the report are as follows:

   a. The stability of the American River at several locations where the Modesto formation forms the bed of the river is highly dependant on the strength and thickness of the formation. I recommend that the stability of this formation be evaluated in a future study by determining erosion rates and thicknesses of the formation.

   b. The scope of work for this study did not address the issue of riprap stability. No information is provided in the Draft Report relative to riprap toe depth or how much scour is to be expected adjacent to the banks. I am especially concerned about the size and toe protection at cobble-lined sites. Perhaps this issue has been addressed in other studies or perhaps design reports are available to confirm the stability of the toe and side slope protection works, however, without this backup information it is not possible to comment on the reliability of existing bank protection.

   c. Erosion resistance along the levees could be increased by planting the slopes with a resistant grass and irrigating. Some of the potential levee erosion sites could be eliminated with this improvement.

Sincerely,

Ronald R. Copeland, P.E. PhD