CACHE CREEK STREAMWAY STUDY

3.6 CHANNEL DYNAMICS

3.6 CHANNEL DYNAMICS

General

Lower Cache Creek is a seasonal stream with flows ranging from zero to more than 50,000 cubic feet per second at Capay. The active stream channel varies in width from over 2,300 feet to less than 300 feet, with a gradient that varies from 5 to 20 feet per mile (0.0009 to 0.004) through the study area. Because of the variation in width and slope, there is considerable variability in local depth and velocity resulting in different sediment transport capacities from subreach to subreach. Historically (see Section 3.4), the overall buildup of streambed materials has affected the flood carrying capacity of the creek as well as its course. Since the early 1950s, aggregate removal has lowered the streambed by 15 to 25 feet in many places and narrowed the active channel to a fraction of its historical width. These morphological changes have increased the flood flow carrying capacity of the 14.5 mile long study reach to the point that in most areas above Road 94B, the channel can now contain more than the 50-year flow within its banks. In the subreaches downstream from Road 94B, the channel capacity is significantly less than this. Prior to extensive aggregate mining, channel deepening, and levee construction, overbank flow during floods was more common in most reaches of this study area. Excess flow left the channel and flowed as shallow, low energy flow in the broad floodplain areas adjacent to the channel. Construction of water diversions and bridges initially contributed to the alteration of the creek's plan and profile, and its ultimate flow characteristics. Today's creek from the Capay Dam to Yolo is dramatically different in its geomorphic, hydraulic and sediment carrying characteristics compared to pre-1930 conditions. The following sections briefly summarize changes in key geomorphic, hydraulic and sediment transport characteristics that have occurred and summarize in more detail the present day channel dynamic conditions found in the Cache Creek study area. The objectives of this chapter are to summarize the relative hydraulic capacity, sediment transport, and channel and bridge stability characteristics in each subreach of the study area.

Methods

As presented in Section 3.1, the study area extends from I-5 at the downstream end to the Capay Dam at the upstream end (see Figures 3.1-1 and 3.3-1 in Section 3.1). Present day channel and floodplain geometry and cross sectional data were taken from topographic mapping prepared from aerial photos flown on October 31, 1994. The Corps of Engineers' West Side Tributary Study (USACE, 1994) HEC-2 hydraulics model, obtained from the Yolo County Planning Department, was used as the basis for the NHC river hydraulics (HEC-2) and sediment transport (HEC-6) models. Cross sectional data from the 1994 mappings were used to update the Corps of Engineers HEC-2 model in the study area (a fixed bed hydraulics model, formerly based on 1992 channel geometry). The HEC-2 model from I-5 to the settling basin was not modified. The downstream hydraulic boundary condition was set as a stage-discharge relationship at the settling basin outlet weir. HEC-2 channel and overbank roughness ("n" values) were initially estimated from the October 1994 aerial photographs and field work, and further adjusted to reproduce observed highwater mark information for the March 1995 event.[1] For the purpose of comparing historical channel hydraulic conditions to the present, an HEC-2 model was also developed using geometry taken from 1905 USGS quadrangle maps.[2]

An HEC-6 model (mobile boundary, sediment transport model) was developed from about 1.5 miles downstream from I-5 to the Capay Dam using the modified HEC-2 model as a base. Effective bed roughness for HEC-6 was computed at each cross section using Limerino's equation.[3] Bank and overbank roughness were estimated from October 1994 aerial photographs. The HEC-6 model was initially run in fixed bed mode for comparison to HEC-2 results to verify channel hydraulic characteristics.

Single event hydrographs for use in HEC-6 were developed for the 2-, 10-, 100-, and 500-year flood events, as well as the March 1995 flood event (estimated to be slightly greater than a 50-year magnitude flood at Capay) as described in Section 3.3. Sediment input data for HEC-6 were based on the suspended and bed load rating curves and relationships described in Section 3.6. Inflowing sediment grain size distribution was developed from measured USGS data. Bed material grain sizes were developed for each reach from existing data[4] and data collected by NHC during this study (see Section 3.3).

Reach-averaged channel hydraulic conditions were computed for approximate 1905 and 1994 conditions using the subreach delineations defined in Section 3.2. Reach averaged values represent the arithmetic average of a particular hydraulic parameter averaged for all of the cross sections within a subreach. Reach averaged values are intended to show spatial trends for a particular parameter on a subreach by subreach basis rather than the local value at a given cross section.

Present Hydraulic Conditions

Sections 3.2 through 3.5 describe the changes that have occurred in the study area due to changes in the morphologic and hydraulic conditions of the creek. Figure 3.6-1 compares computed 100-year water surface profiles for approximate 1905 and 1994 channel conditions. Flows are generally shallower and cover a much wider area on the floodplain for 1905 conditions. Figure 3.6-2 compares the reach-averaged velocity, bed shear and depth of flow (in subreach areas not directly influenced by bridge constrictions) for approximate 1905 and 1994 conditions. Note that the 1905 reach averaged channel velocities are all about 10 to 30 percent lower than the 1994 velocities, except in the Hoppin Subreach (refer to Figure 3.2-13 in Section 3.2 for the definition and location of subreaches). Under present conditions the channel slope flattens through the Hoppin subreach, and a significant backwater condition occurs due to channel constrictions downstream. Similar characteristics are observed for computed bed shear which is an indication of sediment carrying capacity and an indirect indicator of potential channel bed and bank erosion problems. All subreaches, except Hoppin show approximately 30 to 40 percent increases in bed shear from the 1905 estimates. Hydraulic depth is also shown to increase significantly from 1905 to 1994, resulting in flow depths doubling in the downstream-most subreaches. As discussed in previous chapters, these hydraulic changes have greatly altered the magnitude and location of flood threats to the City of Woodland and community of Yolo in last 20 years.

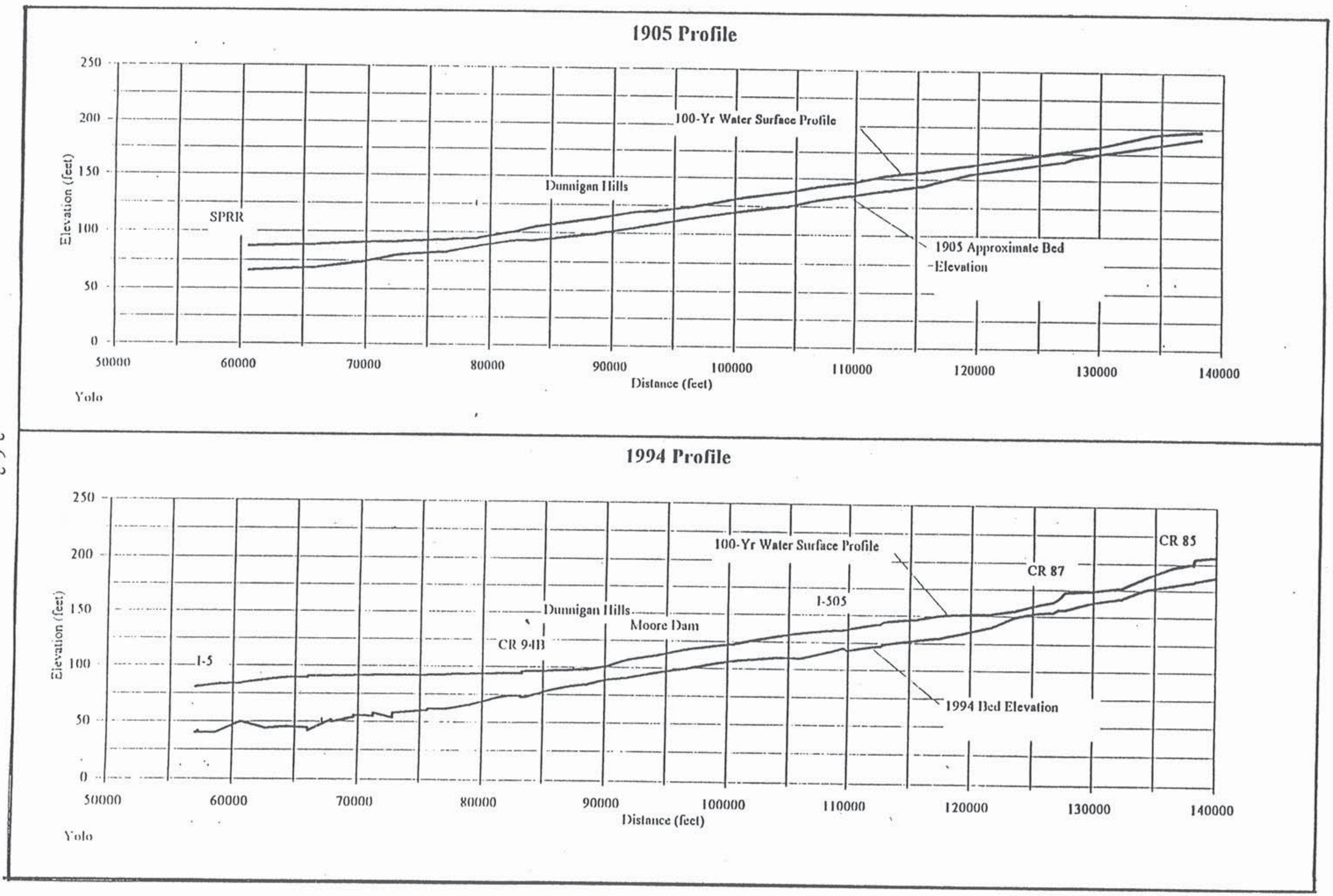


Figure 6-1: 1905 & 1994 Stream Profiles

Figure 3.6-2 · Reach Average Flow Characteristics for 100-yr Flood

1905 and 1994

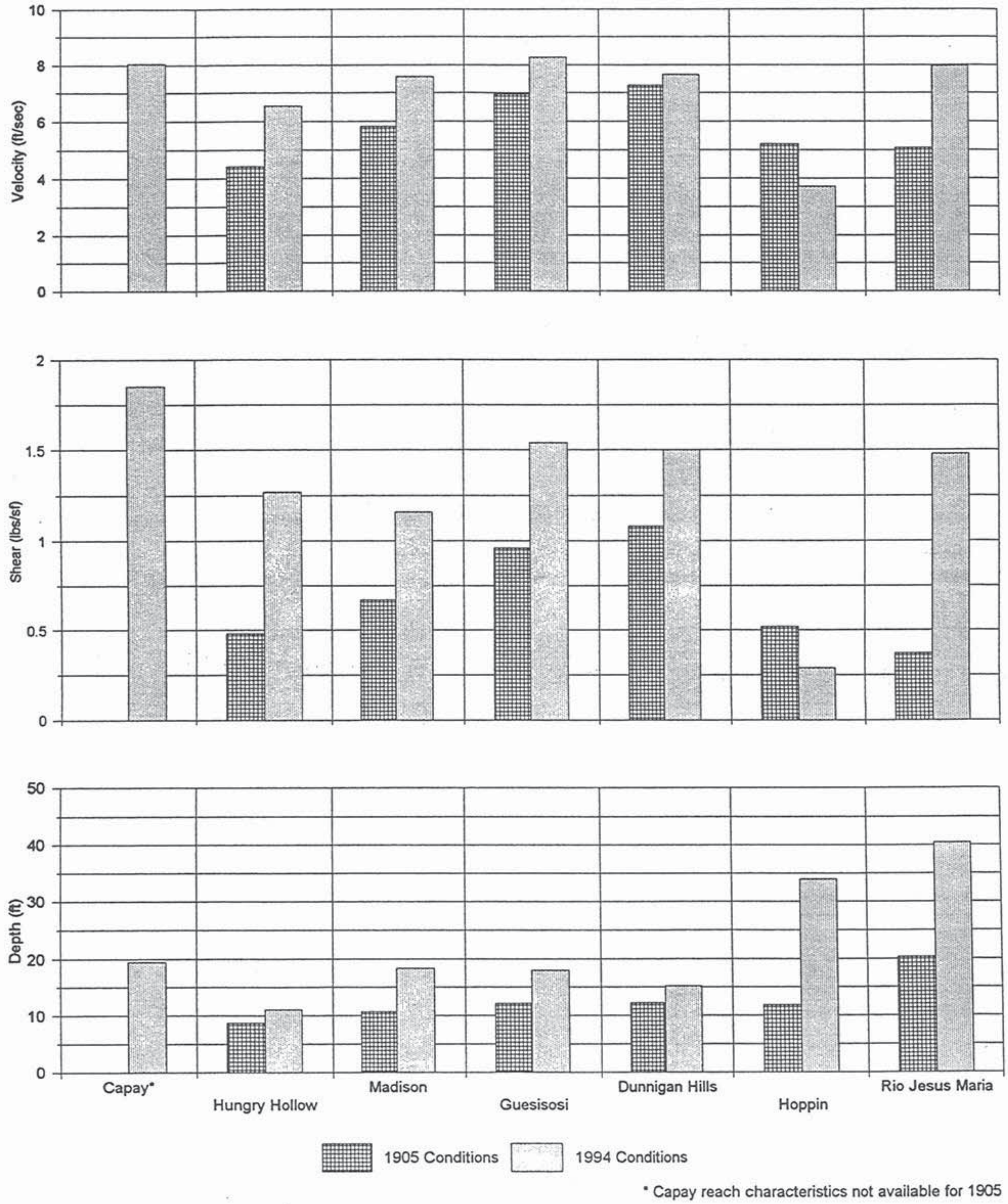
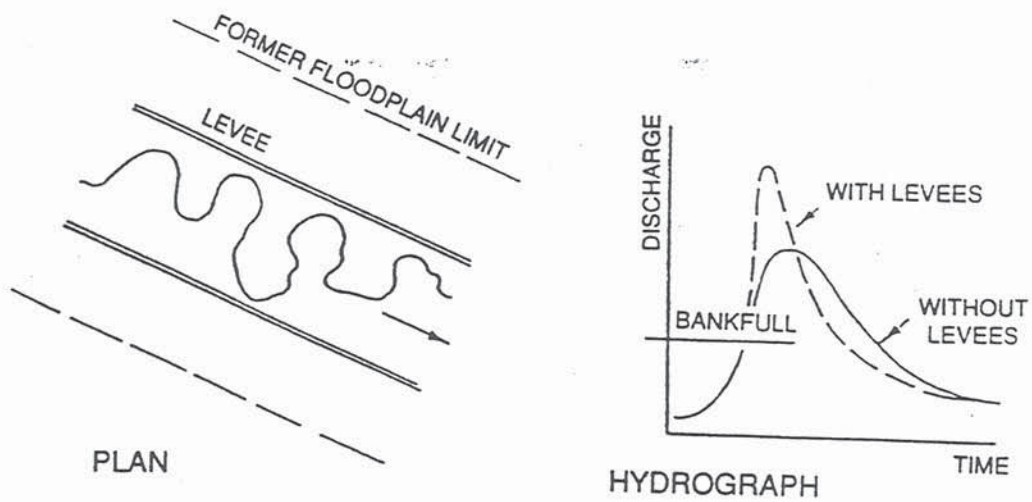


Table 3.6-1 summarizes present day reach-averaged hydraulic conditions in the study area for flows ranging from the 2-year flood to the 500-year flood. These estimates are not intended for use in design or prediction of flooding potential. They do, however, show the dramatic spatial differences in hydraulic properties (depth, velocity and top width) from upstream to downstream in the study area. It should be noted from Table 3.6-1 that the capacity of the present-day channel capacity is between the 10- and 50-year flow downstream from Road 94B. Present day This conclusion was also reached in the Corps of Engineers (1994) Westside Tributaries Study.

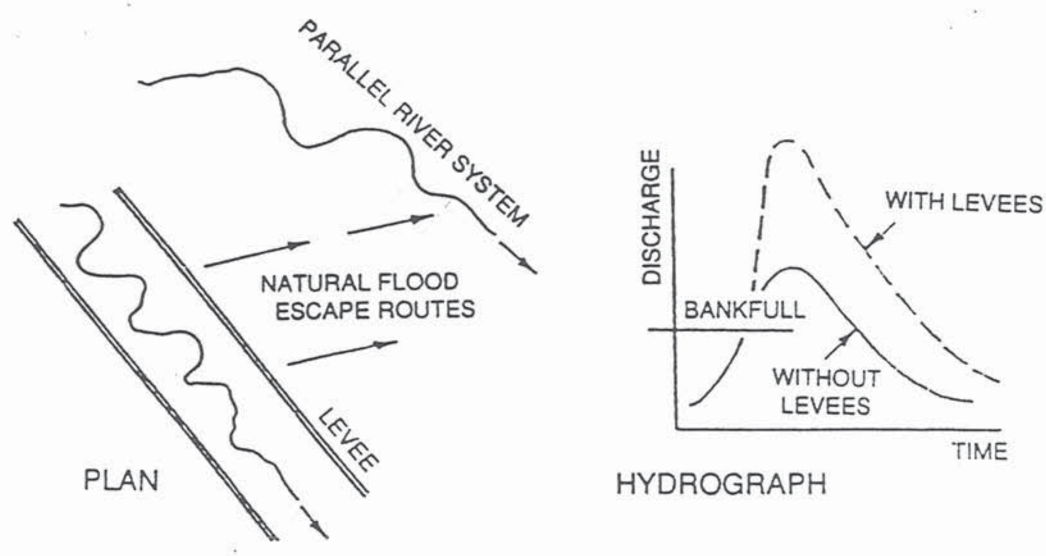
Figure 3.6-3 presents sketches of the effects of reducing floodplain storage and conveyance by either levee building or channel excavation, and the effects of blocking flood flow escape routes by building levees or channel excavation. In both cases the net effect is to increase the peak flow in the vicinity of the change. This can have obvious detrimental effects on flood protection if it is not planned for properly. Channel narrowing and deepening due to local channelization and levee building projects and more regional aggregate extraction over the past 50 years may have had effects on channel hydraulic conditions similar to those shown in Figure 3.6-3.

Bridge Hydraulics

Federal Highway Administration (FHWA) procedures were used to compute hydraulic conditions at each of the six major bridge crossings on Cache Creek. FHWA procedures were also used to compute potential depths of scour at each of the bridges due to combined contraction and pier scour (see FHWA, HEC-18, 1990). Table 3.6-2 summarizes the bridge hydraulics parameters for the six bridges crossing in the study area. Notice that the average velocity through some of the bridge openings is in excess of ten feet per second. Velocities exceeding five to six feet per second are capable of moving significant sediment loads. Flows moving at ten feet per second can cause measurable local scour near fixed structures. Hydraulic conditions summarized in Table 3.6-2 were used with the FHWA, HEC-18 bridge scour procedures to compute estimated scour depths for 10-, 50-, and 100-year flow conditions. Tables 3.6-3, 3.6-4 and 3.6-5 present the computed contraction and pier scour results at each of the bridges. When known, general channel degradation (amount of general channel bed lowering in the main channel due to sediment scouring) should also added to the local scour depth estimates listed in Tables 3.6-3, 3.6-4 and 3.6-5. For Cache Creek general scour is on the order of one to five feet at most of the bridges for events greater than the two-year event, thus adding significantly to the scour susceptibility of all the bridges. According to these estimates the most scour susceptible bridge in the study area is the Capay Bridge, followed by I-5, Stevens, the Highway 505 bridges, and Esparto Bridge respectively. Stevens, 505, and Esparto bridges may experience more total scour than I-5, because they are in reaches where the general channel scour can be as much as three to five feet for the 100-year event. Estimated 100-year total scour depths (general scour plus pier scour) for all of the bridges (except Capay) are on the order of 5.5 to 14 feet. Capay Bridge shows significantly more (41.6 feet of pier scour, plus two feet of general channel scour) scour potential for 100-year conditions. This is due to (1) the present (1994) channel alignment problems (there is significant channel skew to the Capay Bridge); (2) the channel slope steepens in the vicinity of the bridge; (3) the number, shape and diameter of bridge piers and (4) the amount of floating debris that can block a portion of the bridge opening during a 100-year event.[5] Forty-one and a half feet of scour (see last column, 41.6' of total potential scour at Capay Bridge, in Table 3.6-3) is unlikely to happen during a 100-year event at the Capay Bridge, but the FHWA relationships show a very high scour potential.



a. Effect of reducing floodplain storage and conveyance (flood volume unchanged)



b. Effect of blocking escape routes (flood volume increased)

revised from COE EM 1418, 1994

Figure 3.6-3 Effects of Levees on Flood Hydrographs

Table 3.6-1 Table of Reach Averaged Hydraulic Characteristics

Reach	2-Year Event				5-Year Event				10-Year Event			
	Flow (cfs)	Depth (feet)	Velocity (fps)	Top Width (feet)	Flow (cfs)	Depth (feet)	Velocity (fps)	Top Width (feet)	Flow (cfs)	Depth (feet)	Velocity (fps)	Top Width (feet)
I-5 / RR Bridges	14500	28.93	4.36	170.64	28000	37.63	5.81	193.45	36106	39.50	7.01	199.19
Rio Jesus Maria	14500	27.62	4.94	210.30	28000	36.77	5.65	290.41	36106	39.63	6.40	356.56
Hoppin	14500	18.37	2.89	672.30	28000	25.74	2.67	1161.01	37000	29.00	2.72	1422.51
CR94B Bridge	14500	8.82	6.09	305.02	28000	14.15	6.89	322.90	37000	17.03	7.39	332.05
Dunnigan Hills	14500	7.38	4.49	689.79	28000	10.36	5.54	728.76	37000	12.22	5.97	761.03
Guesisosi	14500	9.10	4.78	490.00	28000	12.44	5.99	532.64	37000	14.24	6.62	545.74
I-505 Bridge	14500	8.17	6.02	373.32	28000	11.31	7.66	412.06	37000	13.06	8.45	418.40
Madison	14500	9.25	5.74	489.30	28000	12.83	6.77	576.31	37000	14.68	7.35	592.21
Lower Hungry Hollow	14500	4.98	4.01	1454.91	28000	6.59	4.82	1497.36	37000	7.61	5.13	1558.99
CR87 Bridge	14500	6.21	8.08	451.80	28000	8.22	10.23	496.62	37000	9.30	11.28	514.22
Upper Hungry Hollow	14500	6.87	4.57	864.50	28000	9.13	5.50	987.39	37000	10.25	6.01	1038.03
CR85 Bridge	14500	9.54	5.78	530.39	28000	12.69	6.80	702.68	37000	13.85	7.69	769.87
Capay	14500	11.44	6.32	328.11	28000	15.08	7.49	1052.30	37000	16.54	7.99	1433.63

Reach	50-Year Event				100-Year Event				500-Year Event			
	Flow (cfs)	Depth (feet)	Velocity (fps)	Top Width (feet)	Flow (cfs)	Depth (feet)	Velocity (fps)	Top Width (feet)	Flow (cfs)	Depth (feet)	Velocity (fps)	Top Width (feet)
I-5 / RR Bridges	40758	40.42	7.66	203.07	41197	40.50	7.72	203.42	41994	40.65	7.83	203.94
Rio Jesus Maria	40758	41.36	7.31	381.67	41197	41.55	7.47	383.85	41994	41.90	7.77	385.60
Hoppin	52697	32.04	3.33	1516.54	56183	32.57	3.55	1584.42	63710	33.71	4.10	1674.60
CR94B Bridge	57000	20.22	9.38	338.15	63500	20.87	10.09	339.39	80900	22.31	11.93	342.14
Dunnigan Hills	57000	15.25	6.79	866.03	63500	16.10	7.07	879.15	80900	18.17	7.71	913.27
Guesisosi	57000	17.64	7.71	607.15	63500	18.59	8.02	614.27	80900	20.88	8.77	642.72
I-505 Bridge	57000	16.56	9.72	431.11	63500	17.59	10.07	434.81	80900	20.09	10.92	443.89
Madison	57000	18.24	8.23	659.26	63500	19.26	8.44	692.03	80900	21.84	8.92	735.70
Lower Hungry Hollow	57000	9.63	5.72	1611.20	63500	10.09	5.71	1859.49	80900	11.69	5.95	2042.80
CR87 Bridge	57000	11.21	13.15	569.13	63500	11.73	13.73	574.98	80900	12.94	15.19	595.46
Upper Hungry Hollow	57000	12.40	6.76	1211.68	63500	12.94	6.96	1246.60	80900	14.43	7.46	1324.01
CR85 Bridge	57000	16.03	9.12	938.73	63500	16.60	9.52	982.59	80900	17.89	10.36	1217.74
Capay	57000	18.98	8.66	1740.17	63500	19.65	8.81	1758.70	80900	21.44	9.03	1792.86

Flows provided represent the discharge passing the downstream boundry of the averaged subreach.
Top width is the cross-section width at the calculated water surface elevation.

Table 3.6-2 Hydraulic Parameters for Bridge Scour Evaluations

Bridge	Discharge (cfs)	Frequency (years)	Flow velocity (feet/second)	Water depth (feet)	Top width (feet)
I-5 North	41.197	100	7.17	39.71	236.10
	40,578	50	7.11	39.66	235.44
	36,106	10	6.53	39.43	228.24
I-5 South	41,197	100	7.38	38.75	229.51
	40,758	50	7.33	38.70	228.96
	36,106	10	6.72	38.47	222.91
Stevens	63,500	100	10.29	20.85	316.51
	57,000	50	9.55	20.25	315.94
	37,000	10	7.45	17.38	313.11
I-505	63,500	100	9.94	16.85	436.99
	57,000	50	9.60	15.77	432.88
	37,000	10	8.34	12.19	418.76
Esparto	63,500	100	12.55	15.21	524.24
	57,000	50	11.90	14.69	520.33
	37,000	10	9.63	10.87	506.54
Capay	63,500	100	9.47	23.60	441.36
	57,000	50	9.1	22.64	440.29
	37,000	10	7.64	15.90	433.84

d:\projects\report\tab3_6.2

Table 3.6-3. Scour Evaluation for Cache Creek Bridges (10-year Flood)

Bridge Name	Bridge #	Highway #	Water Depth (feet)	Flow Velocity (feet/second)	Contraction Scour (feet)	Pier Scour (feet)	Total Scour (feet)
Capay	22C-05	CR 85	14.01	11.58	3.0	32.4	35.4[1]
Esparto	22C-01	CR 87	10.87	9.63	0.1	4.1	4.2
I-505	22-101	I-505	12.19	8.34	0.1	5.2	5.3
Stevens	22c-04	CR 94B	17.38	7.45	0.1	5.7	5.8
I-5 South	22-07L	I-5	38.47	6.72	1.3	7.2	8.5
I-5 North	22-07R	I-5	39.43	6.53	1.0	7.1	8.1

(1) Scour depth of Capay Bridge was calculated with exposed footing and debris accumulation at 45 degree of attack angle.

f:\projects\50012\bridges\scour\scour.wpd

Table 3.6-4. Scour Evaluation for Cache Creek Bridges (50-year Flood)

Bridge Name	Bridge #	Highway #	Water Depth (feet)	Flow Velocity (feet/second)	Contraction Scour (feet)	Pier Scour (feet)	Total Scour (feet)
Capay	22C-05	CR 85	15.78	15.0	3.8	36.8	40.6[1]
Esparto	22C-01	CR 87	12.71	11.9	0.4	4.7	5.1
I-505	22-101	I-505	15.72	9.6	0.1	5.7	5.8
Stevens	22c-04	CR 94B	20.56	9.55	0.3	6.5	6.8
I-5 South	22-07L	I-5	39.43	7.33	1.1	7.5	8.6
I-5 North	22-07R	I-5	40.39	7.11	1.9	7.4	9.3

(1) Scour depth of Capay Bridge was calculated with exposed footing and debris accumulation at 45 degree of attack angle.

f:\projects\50012\bridges\scour\scour.wpd

Table 3.6-5. Scour Evaluation for Cache Creek Bridges (100-year Flood)

Bridge Name	Bridge #	Highway #	Water Depth (feet)	Flow Velocity (feet/second)	Contraction Scour (feet)	Pier Scour (feet)	Total Scour (feet)
Capay	22C-05	CR 85	16.21	16.09	3.5	38.1	41.6[1]
Esparto	22C-01	CR 87	13.23	12.55	0.5	4.8	5.3
I-505	22-101	I-505	16.75	9.94	0.1	5.8	5.9
Stevens	22c-04	CR 94B	21.2	10.29	0.3	6.7	7.0
I-5 South	22-07L	I-5	39.52	7.38	1.2	7.5	8.7
I-5 North	22-07R	I-5	40.48	7.17	2.0	7.4	9.4

(1) Scour depth of Capay Bridge was calculated with exposed footing and debris accumulation at 45 degree of attack angle.

f:\projects\50012\bridges\scour\scour.wpd

The Capay Bridge experienced significant channel and bed scour problems during the January and March 1995 flood events. Figures 3.6-4 and 3.6-5 show Cache Creek channel conditions immediately upstream from the Capay Bridge in May, 1994 and June, 1995, respectively. The high flows of January and March, 1995 greatly altered the morphology of the approach channel upstream, under and downstream from the bridge. By the end of the March event, approximately 1,600 lineal feet of the left bank had been seriously eroded upstream from the bridge. The bank retreated back distances of approximately 300 feet almost flanking the north abutment. The bed and bank erosion forced county crews and contractors to perform emergency bank repair activities during the March event. In February 1995, the Natural Resource Conservation Service (NRCS) performed similar emergency bank repairs to protect the West Adams Canal upstream from the Capay Bridge that had occurred during the January high flows. Figure 3.6-6 shows emergency bank and bridge abutment repairs occurring at the Capay Bridge during the March 1995 storm event.

Other bridges experienced local scour in 1995, but not to the extent that the Capay Bridge did. During the extended high flow period following the March 9th peak, the channel bottom under the Capay Bridge continued to erode, resulting in noticeable settlement of the number 7 pier under the bridge. This forced the permanent closure of the bridge to pedestrian and automobile traffic. Recent feasibility investigations estimate that bridge repair or replacement will cost several million dollars. Similar episodes at other bridges, including the Capay Bridge have occurred many times. Table 3.4-4 lists the number of scour related repairs, failures and bridge replacements the county has experienced since 1940.

Factors Affecting Bridge Scour

As just discussed, local scour at a bridge crossing can be substantial. The depth and area of scour at a given bridge at a given time may be affected by any or all of the following factors:

- slope, natural alignment and shifting of the channel;
- type and amount of bed-material in transport;
- history of former and recent floods;
- accumulation of debris, trees or other obstructions to flow;
- constriction and/or realignment of flow due to the bridge and its approaches;
- layout and geometry of training works;
- geometry and alignment of bridge piers;
- classification, stratification, and consolidation of bed and sub-bed materials;
- placement or loss of rip-rap and other protective materials; and
- natural or man-made changes in flow or sediment loading.

The order in which the factors are listed above is not intended to indicate their relative importance, which varies from case to case. The erosive power of flowing water on a channel boundary is determined primarily by the local shear stress or drag exerted by the flow on the boundary, and by the associated velocities and turbulent fluctuations of velocity near the boundary. Rate and extent of scour depends on the relationship of erosive power to erosional resistance, and on the balance between material eroded and material deposited. Under steady flow conditions, most scour situations eventually reach a final or equilibrium (balanced) condition. Under natural unsteady flow conditions a final scour topography is not necessarily attained in a single flood event, but it may develop progressively over a series of events. During

Figure 3.6-4 Cache Creek looking upstream from Capay Bridge May 31, 1994

Figure 3.6-5 Cache Creek looking upstream from Capay Bridge June 6, 1995

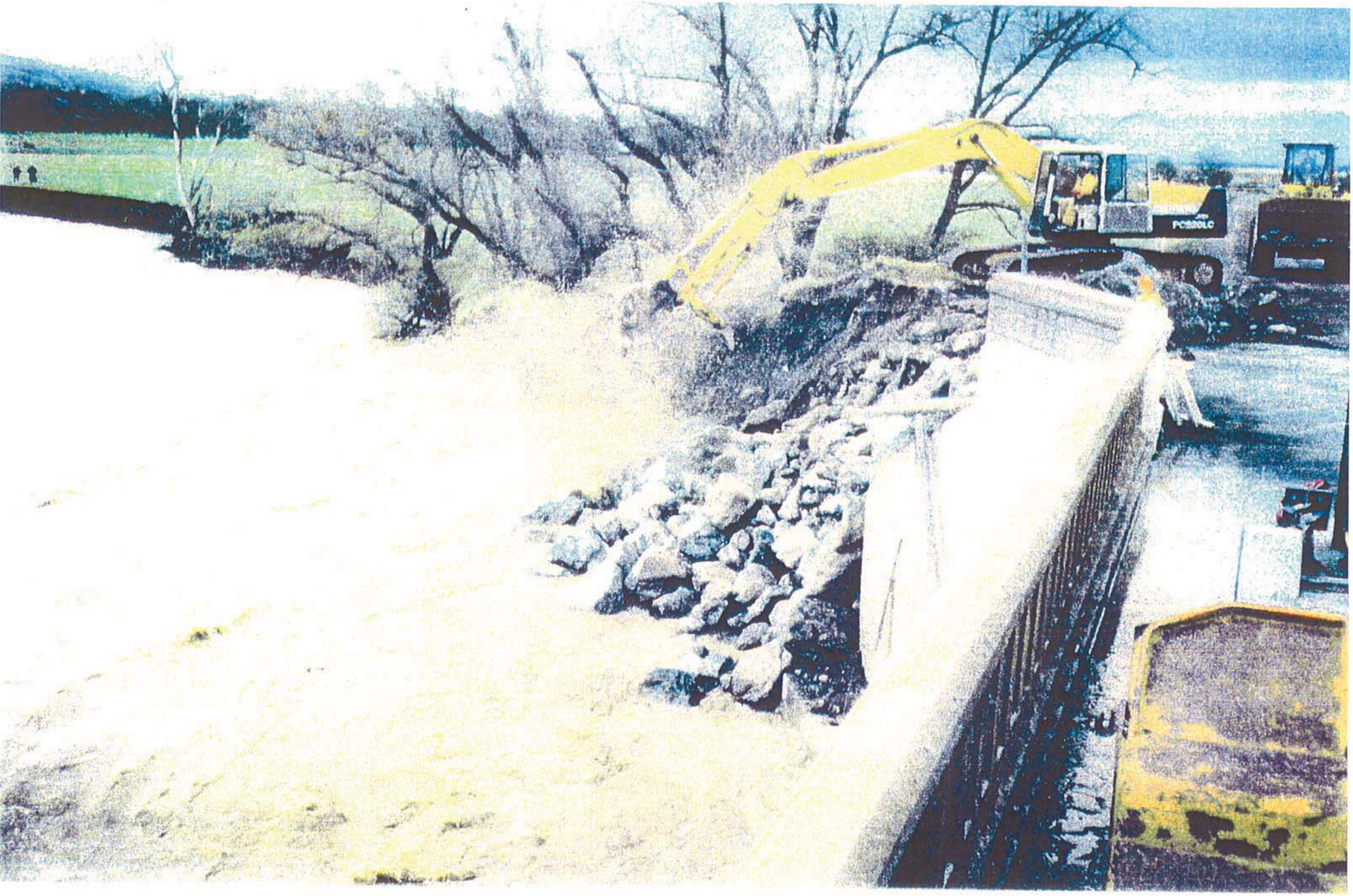


Figure 3.6-6 Emergency Repair of North Abutment of Capay Bridge During the March 1995 Flood Event

a typical flood rise and recession, scour tends to lag behind discharge, so that maximum scour may occur after the flood peak passes, but it may recover by backfilling and not be as severe by the end of the event. FHWA guidelines (HEC-18 and HEC-20) describe in more detail the factors affecting stream stability near bridge crossings and measures for controlling scour, called countermeasures.

Countermeasures

Numerous measures are available to counteract the actions of man and nature which contribute to the instability of alluvial streams. These include measures installed in or near the stream to protect roads and bridges by stabilizing a local reach of the river. Other measures may be incorporated into the highway design to ensure the structural integrity of the highway in an unstable stream environment. Selection, location and design of countermeasures (protective, stream stabilization measures) are dependent on hydraulic and geomorphic factors that contribute to instability as well as costs and construction and maintenance considerations. Countermeasures may be different for each bridge, depending on the bed material characteristics, flow alignment, depth, velocity and bridge pier and abutment characteristics.

Most bridges in the study area tend to be in sediment deficit and are, therefore, more scour-prone. Countermeasures may include upstream and downstream flow realignment structures such as guide banks, rock groins or jetties to improve the inflow and outflow characteristics at the bridge. As observed this year at the Capay and Rumsey Bridges, flows oriented obliquely to the bridge opening are very inefficient and exacerbate local scour under the bridge. In reaches where channel lowering upstream or downstream from the bridge occurs annually because of in-channel aggregate mining, the bridge may become sediment starved resulting in accelerated scour and channel incision beneath the bridge. Local channel bed stabilization works, such as placement of scour resistant rock donuts around the bridge piers and abutments may be necessary. In severe cases where prolonged channel incision exceeds the ability of local countermeasures to protect the piers from scour and exposure, the pier footings and pile caps may require extension and repairs. This has occurred previously at the Stevens Bridge, Highway 505 and Esparto Bridges. Erosion resistant rock mattresses and rock or concrete sills (grade control structures) may also help to reduce local channel incision in the vicinity of bridges. Such countermeasures can be costly, but are a preferred alternative to bridge replacement.

Bridge Recommendations

Regular bridge inspections (including the structure and the channel areas upstream,beneath, and downstream from the bridge), especially following large runoff events are important. Regular inspections can monitor changes occurring near the bridge and anticipate the effects of upstream or downstream channel adjustments that may result in localized bridge scour problems. Most bridge openings are much narrower than the stream channel upstream or downstream from them. Widening the bridge openings and reducing the number of bridge piers in the flow by lengthening individual spans would reduce flow velocities and decrease scouring potential. This would be costly. If bridge openings cannot be widened, then it is best to develop and maintain a smooth, non-abrupt, entrance and exit alignment for flows through all bridges. If velocities through the bridge openings are still too high for a stable bottom, then channel bottom stabilization measures can be installed, such as rock or concrete mattresses and sills. Debris should be removed as soon as possible from piers and abutments. Extensive encroachment from invasive plants and trees in the main channel immediately upstream or downstream of bridges should also be discouraged through maintenance. Vegetation can and should be encouraged to grow along the channel banks to reduce bank erosion and potential channel realignment problems.

Sediment Transport

Flood Event Analyses

Similar to the spatial variability found with channel hydraulic conditions (see Table 3.6-1), sediment transport characteristics also change from subreach to subreach in the study area. Computer program HEC-6 was used to estimate the temporal and spatial sediment variability throughout the study reach for a wide range of flood events. Estimated changes in reach averaged sediment loads during the 2-, 10-, 100-, and 500-year events were computed. NHC also computed sediment transport conditions for the March 1995 flood event. Figures 3.6-7 through 3.6-11 present the reach-averaged sediment load changes occurring throughout the study reach for the 2-, 10-, March 1995 event (approximately a 50-year event), 100-, and 500-year events, respectively. In each case the complete single event hydrograph was routed through the entire study reach. Results are presented as dimensionless vertical bar graphs referred to as "disequilibrium" plots. The amount of sediment load in disequilibrium is indicated as a percent above or below zero. A subreach is in sediment equilibrium with its inflowing load if the disequilibrium bars are close to zero. These analyses were intended to show large-scale reach-averaged trends, not site specific local conditions.

Each bar graph indicates on a reach-averaged basis the percent differences between the cumulative inflowing sediment load entering a subreach at its upstream end and the average outflowing sediment load leaving each subreach normalized by the total cumulative sediment load supplied to the modeled reach. Positive percentages indicate that more sediment stays within a subreach up to that particular time than had left the subreach in the same time period. Therefore, positive percentages indicate a general trend for sediment deposition within the subreach for that time period. Negative values indicate more sediment left than entered the subreach. Negative percentages indicate a general "sediment hungry" trend within the subreach and bed or bank scouring is likely to occur within that subreach for that time period, thus positive (deposition) or negative (scour) disequilibrium conditions can be compared on a reach by reach basis. In each of the Figures 3.6-7 through 3.6-11, three different time periods are represented by the families of three vertical bar plots from left to right as follows: (1) cumulative conditions up to and including the **peak** of each event; (2) cumulative conditions up to and including the time past the peak and **half way through the recession period of flow**; and (3) the cumulative conditions at the **end of each event**. Disequilibrium conditions are indicated for sixteen different subreaches from the Capay Dam to I-5 at Yolo. Reach numbers correspond to those shown in Figures 3.2-13 in Section 3.2 and 3.5-1 in Section 3.5. An A, B, or C indicates the subreach was divided into two or more segments. Examination of results from the five different single event analyses shows the spatial and temporal variation in sediment transport conditions throughout the study area.

As presented in the previous section, all of the bridges upstream from the Rio Jesus Maria subreach (subreach 2) are sediment starved near the peak of the events. Then, depending on the magnitude of the event and the duration of the receding flows, the channel in the vicinity of the bridges attempts to recover (fill back in) toward the end of the event. In general the wide and

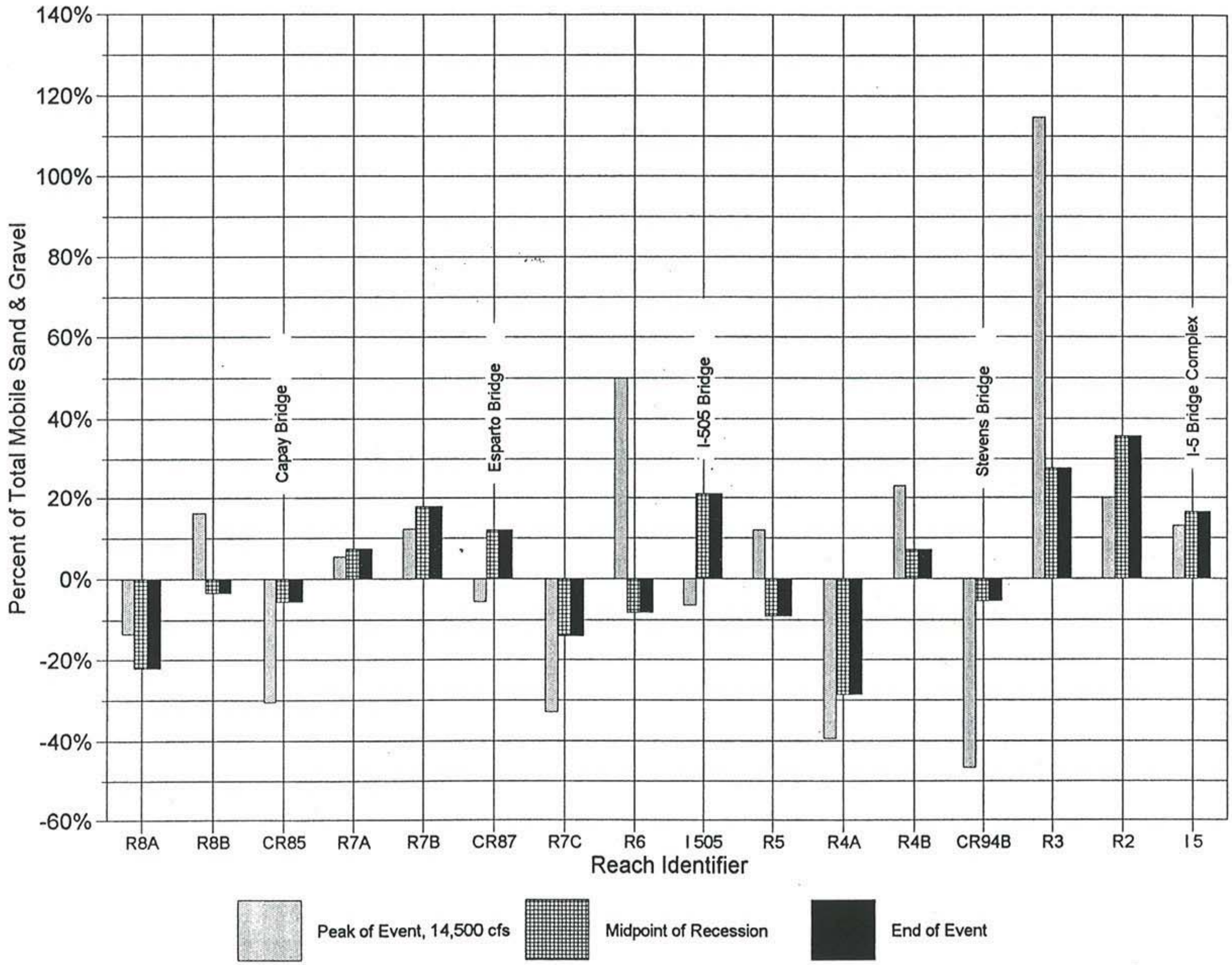


FIGURE 3.6-7 Reach Averaged Sediment Load Changes for the 2-Year Event

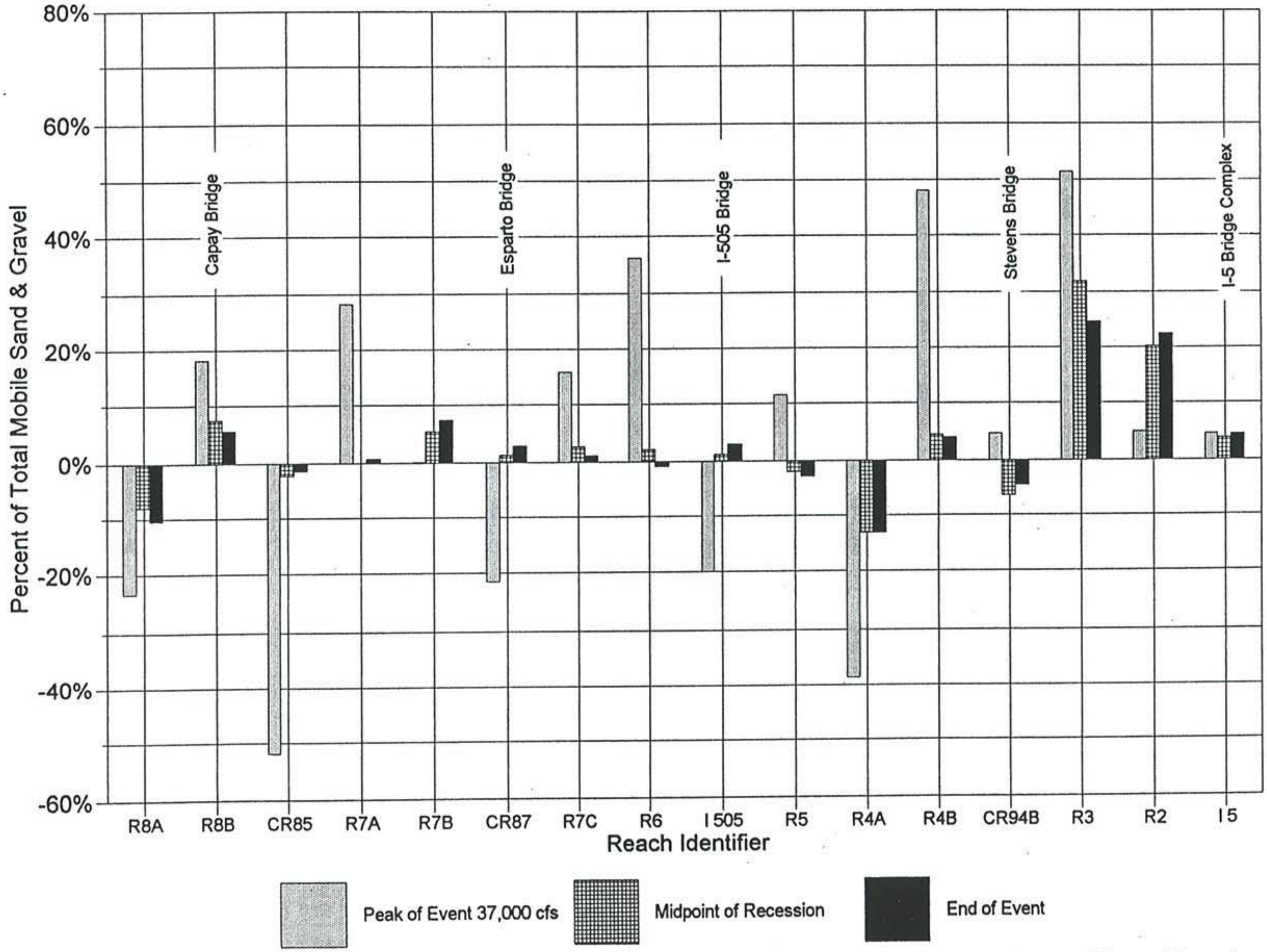


FIGURE 3.6-8 Reach Averaged Sediment Load Changes for the 10-Year Event

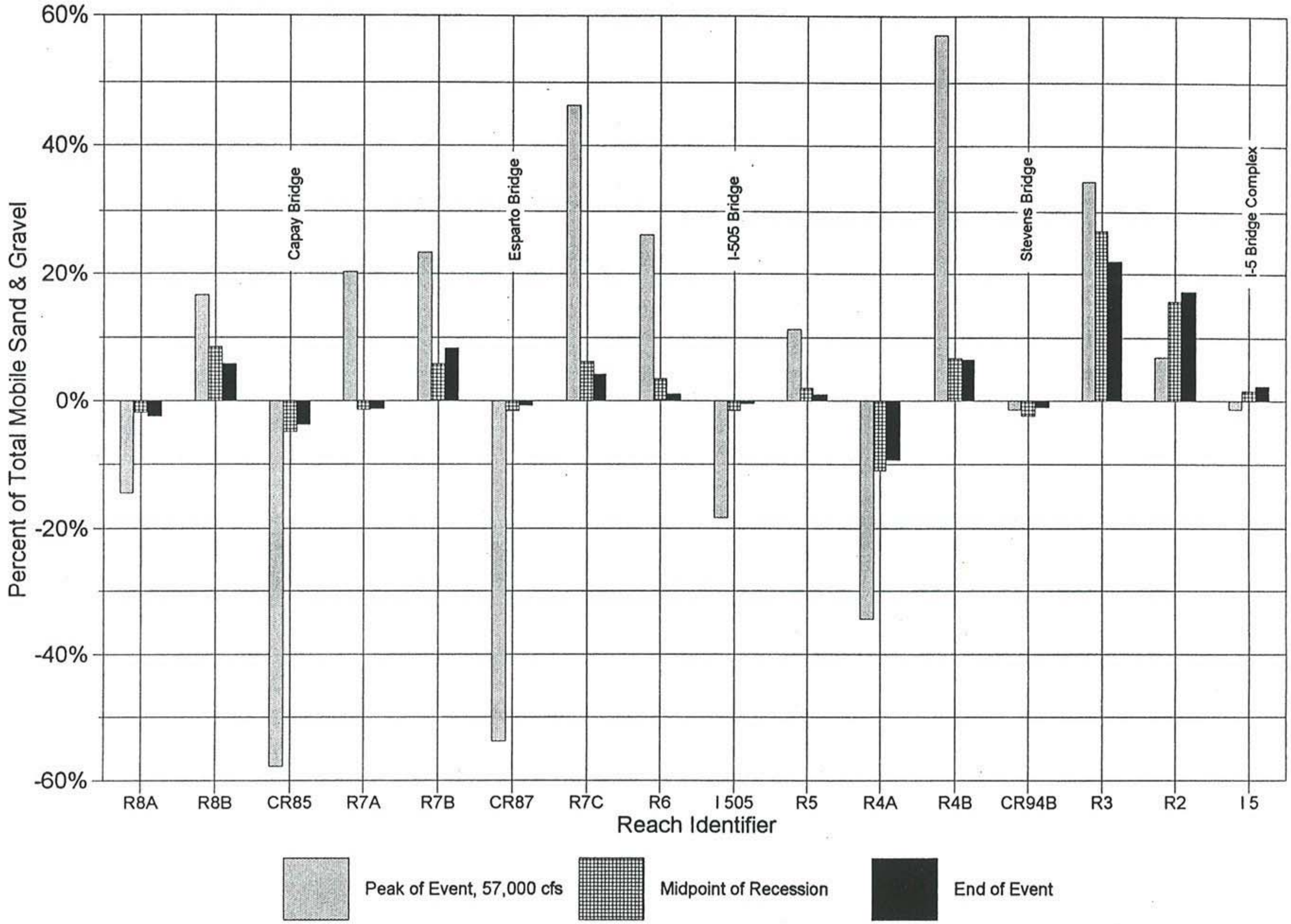


FIGURE 3.6-9 Reach Averaged Sediment Load Changes for the March 1995 Event

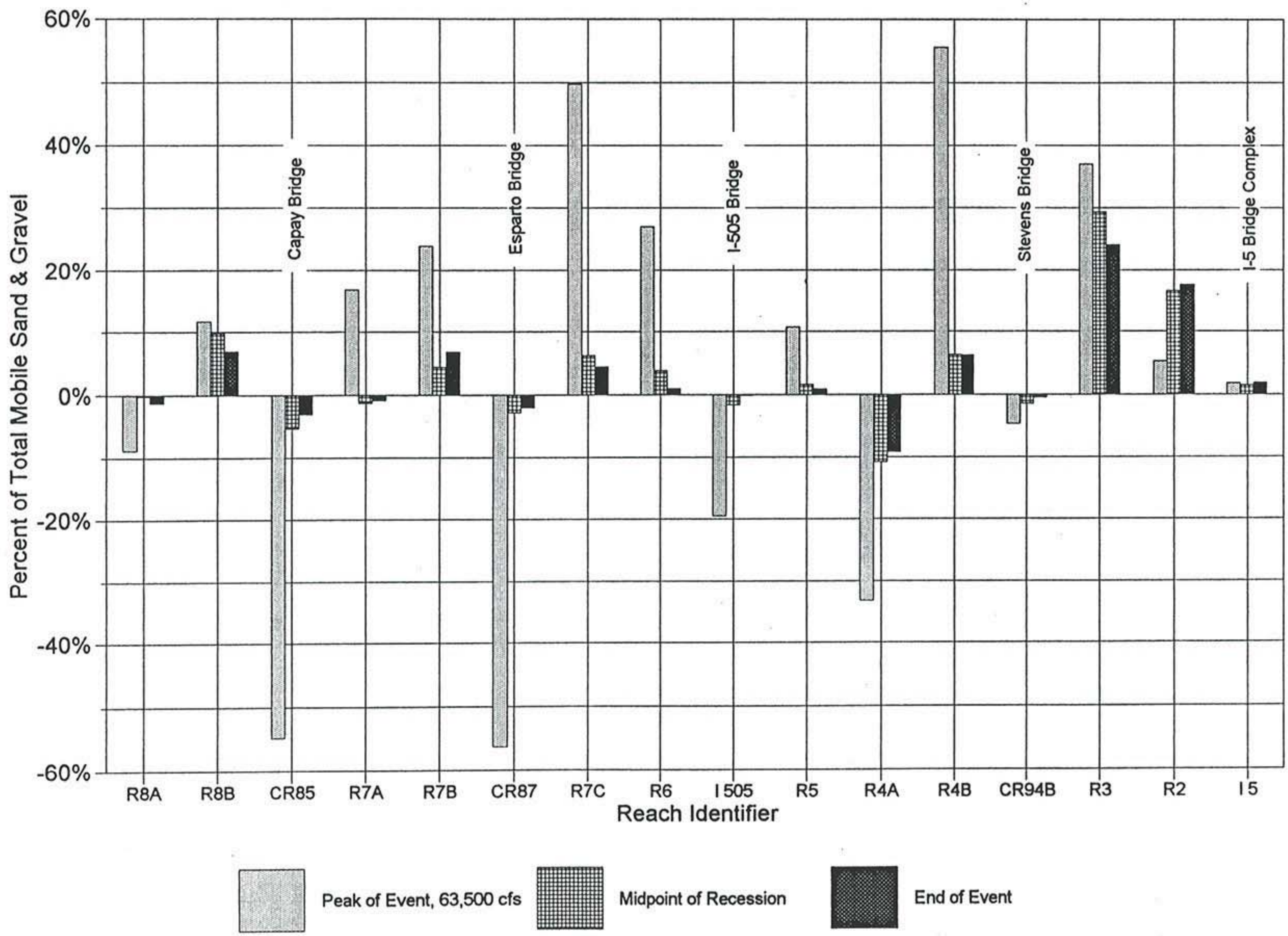


FIGURE 3.6-10 Reach Averaged Sediment Load Changes for the 100-Year Event

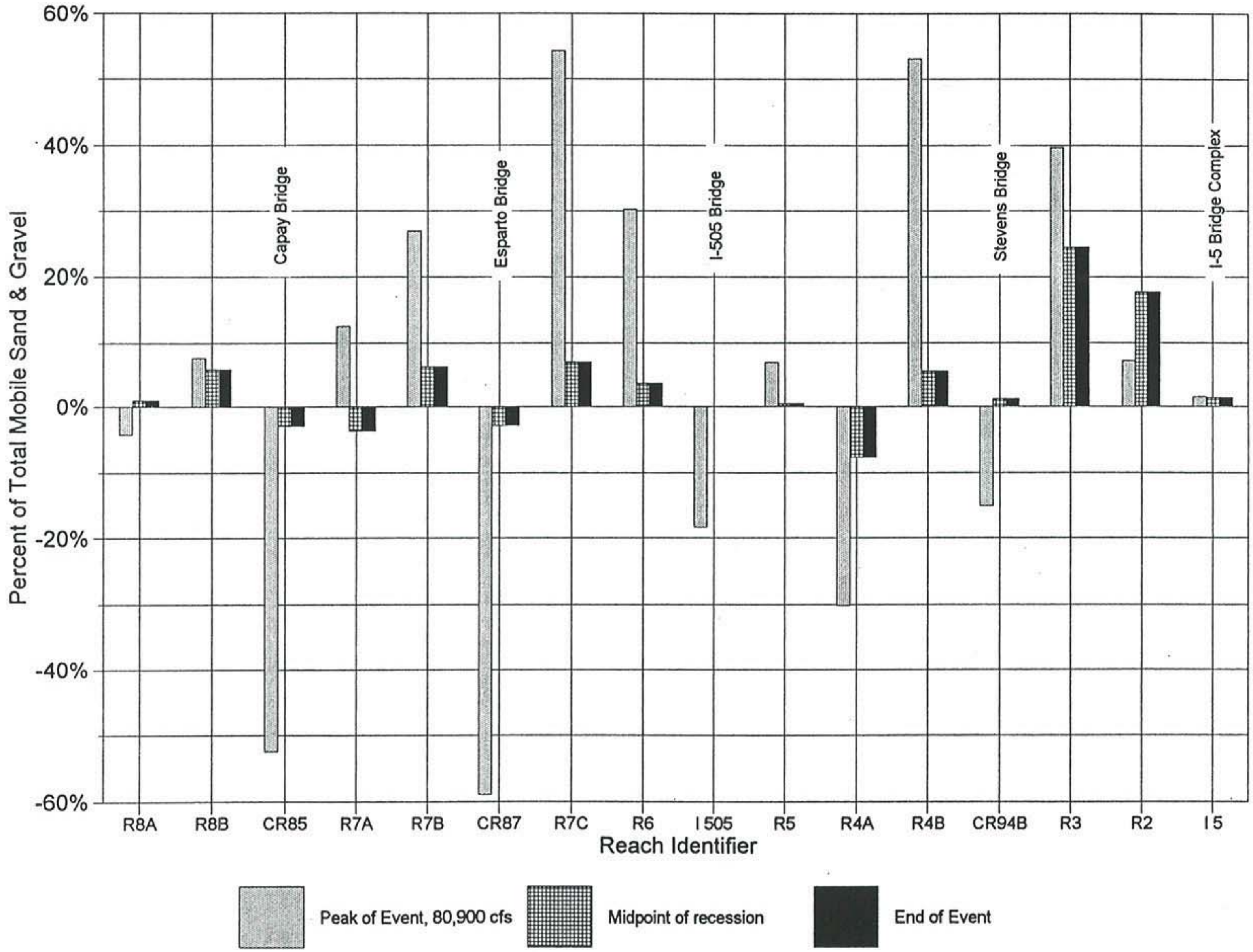


FIGURE 3.6-11 Reach Averaged Sediment Load Changes for the 500-Year Event

flat areas (Hungry Hollow and Madison subreaches) tend to be aggradational while the narrow and confined subreaches, such as the Dunnigan Hills subreach continue to be sediment starved and degrading (see subreach R4A). Degradation is the morphologic expression of a sediment transport imbalance, since sediment transport capacity exceeds the supply. Examination of the detailed HEC-6 output also showed that the total sediment load moving through the system dramatically drops off downstream from subreach R4A, resulting in net deposition in the downstream-most subreaches, especially in subreach R3, the Hoppin subreach. Because of significant channel constrictions through the Rio Jesus Maria subreach (subreach 2), water surface profiles flatten upstream from I-5 and can extend up past the Road 94-B bridge for the large magnitude, less frequent events. This backwater condition results in slower, deeper flows and greatly reduces the sediment carrying capacity through the downstream subreaches. Figures 3.6-7 through 3.6-11 support this and show substantial sediment accumulation potential in subreaches R2 and R3. The results also show that more deposition potential occurs near the peak of large events followed by removal of a portion of the deposited materials during receding flows. Field inspection of subreaches R2 and R3 following the 1995 events indicated this same trend, where observed in-channel deposits on the order of 8 to 12 feet deep had occurred during the event, only to be reworked by incision during receding flows.

The hydraulic effects of downstream control on subreaches 2 and 3 are dramatically evident from the series of disequilibrium plots presented in Figures 3.6-7 to 3.6-11. For the 2-year event, the Stevens Bridge experiences net sediment loss (scouring) for the entire event. However, for the 10-year event or greater, the Stevens Bridge is less in deficit because of deeper, slower flow conditions resulting from backwater control downstream. The I-505, Esparto and Capay bridges all show increasing sediment deficits (scour potential) with increasing flow. This is especially evident near the peak of flood events.

It's important to understand how much sediment disequilibrium may occur at various times during an event. Figures 3.6-7 to 3.6-11 show how sediment movement from subreach to subreach may be out of phase compared to the flow occurring at a given time. (The bars may change their orientation from below zero to above zero after the peak passes.) This occurs because bed material movement (transport of coarse materials) proceeds downstream at a slower rate of speed than the flow velocity. Therefore, significant localized channel scouring and bridge pier scouring can occur near the peak of an event only to be back filled during the long recession flow period after the peak has passed. What is left behind after the flood may not tell the most important story with respect to maximum scour potential occurring near the peak of an event. Conversely, the development of large gravel bar deposits upstream or downstream from a bridge after an event does not mean the bridge crossing is stable or sediment overabundant in that subreach.

Important points to observe from these results are that sediment transport capacities changes from subreach to subreach, and some narrow subreaches (especially near bridges) tend to be sediment starved. The relative amount of imbalance, or disequilibrium, (where equilibrium means sediment inflow is approximately equal to outflow within each subreach) is much more amplified at the peak of an event than at the end of the event after recession flows have reworked the channel materials. Such an imbalance sets in motion a complex series of interactions and channel adjustments through which the system attempts to adjust to a new state of dynamic equilibrium. The large amount of imbalance (the longer vertical bars) near the peak of the events may result from confinement of the flows into a channel that is much narrower than it wants to be for the magnitude of flow and availability of sediment being supplied to the study reach (refer to Figure 3.6-3). After the peak passes, recession flows are more evenly distributed within the channel cross section and the transport rates become more balanced with the sediment availability. The net result is that, under present-day channel conditions, there is an imbalance between water discharge and sediment load for most of the subreaches in the study area. In addition, the channel wants to be much wider than it is presently, especially at the peak of each flood event.

Stream Bank Stability

Stream banks erode by fluvial entrainment and by mass failure mechanisms, which are controlled by combinations of various geomorphic and geotechnical (soil mechanics) processes. Fluvial entrainment causes banks to retreat in two ways. First, sediment may be directly entrained from the bank and transported downstream. Second, the flow may scour the bed at the base or toe of the bank, thus increasing the bank angle and height, resulting in gravitational failure of the bank (Thorne et al., 1988). The later failure mechanism is of greatest importance when the banks are located on the outside portion of a bend. In streams that are degrading (sediment hungry), bank failure usually results from channel scour along the base of the bank, which leads to increasing bank heights beyond a critical height and bank angle near vertical. Once these unstable conditions occur, gravitational forces (the weight of the steep bank materials) may exceed the resistant forces of material cohesion and the bank fails abruptly. In the Cache Creek study area, both sediment entrainment and gravitational bank failure processes occur, with the most pronounced mechanism being the mass failure (gravitational) mechanism. Many locations where observed this January and March where this type of bank failure occurred. Streambank stability is often directly related to channel bed stability or instability. According to Schumm[6] base level lowering (channel bed lowering at the downstream end) will cause progressive downcutting and readjustment of the stream gradients until a new graded or equilibrium situation has been developed. Downcutting is often followed by bank failure of over steepened banks and some degree of channel widening, thus initiating a sequence of negative feedback responses throughout the channel reach in an attempt to reestablish balance.

Therefore, bank failure (bank instability) is likely in those subreaches of the creek where ongoing channel degradation is occurring. This is typically in the vicinity of the bridges (including the former Madison Bridge area) and in subreach R4A, where the channel passes through the relatively narrow and steep channel section in the Dunnigan Hills subreach and where channel base level lowering has occurred downstream. Localized bank erosion can occur anywhere in the system, however. It can result from localized obstructions such as a large gravel bar or tree redirecting high energy flows at the toe of an unprotected bank. Removal of the toe materials over steepens the bank, increases its height and may lead to mass failure. Once a section of bank falls into the creek there is usually sufficient energy to carry all or most of the failed bank materials away so the toe of the bank may not stabilize itself against further failures. This describes the mechanism where a very localized, yet severe cusp (lateral excavation by the creek) can attach a bank. If flows remain high for days or weeks, a cusp in the bank becomes susceptible to further erosion, often leading to a progressive downstream movement of the failing bank as well an expanding lateral migration of the creek into the failed area. This process of rapidly accelerating lateral bank failure occurred upstream from the Capay Bridge in January and again in March of this year. On Cache Creek, the lateral extent of bank loss has been observed to be 200 to 800 feet during severe flood events. Therefore, it is prudent to set back structures, off-channel mining pits and other valuable facilities far enough from the edge of the present channel bank to avoid damage and provide sufficient room for flood and bank erosion fighting during large events. In reaches where engineered or stabilized bank sections exist, narrower set backs could be accepted.

Stream Bank Stabilization

Traditional methods of bank protection involve placement of a facing or covering of erosion resistant material, such as large rocks, riprap, concrete rubble, articulated concrete mattresses, sacked concrete, gabions, vegetation, regrading steep banks back to a lessor slope, and/or combinations of all of these on the face of the exposed bank. Other, bio-engineered procedures may combine structural and vegetative procedures together to obtain stabilization. Some bio-technical methods may work in the lower subreaches (Hoppin or Rio Jesus Maria) of the study area where perennial water is available to sustain vegetation. Bio-technical methods may not work in the upper subreaches (Hungry Hollow or Madison) because of the lack of water to sustain vegetation. There are many reference manuals and design guidelines for designing bank stabilization and revetment (protective covering and armoring against erosion) works. Selection of the most appropriate measure must weigh such factors as severity of the problem, local hydraulic properties (depth, velocity and sediment load), aesthetics, durability of the protective materials, resistance of components to movement by the flow, flexibility of the stabilization system to future changes in the channel system, maintenance requirements, susceptibility to vandalism and environmental considerations.

For most applications in Cache Creek, bed and bank stabilization can be achieved by developing a smoothed channel alignment so as to remove abrupt changes in width or changes in flow direction. This can be accomplished through integrated channel reclamation plans organized to resculpt the present channel plan form to a smoother, more uniform width and depth with a compound channel shape (eg., a stable low to intermediate flow channel, confined within compound terraces). Installation of channel training structures called groins or spur dikes can aid in developing a smoothed more uniform channel plan form rather than filling in excessively wide sections. This is especially important in the vicinity of bridges, where present mining regulations allow abrupt changes in channel width upstream and downstream from bridges. Near bridges, upstream oriented guide banks through the bridges, in combination with groin fields into and out of the bridge opening may provide the best treatment. Areas near bridges are very scour susceptible under present conditions and show a continuing history of local bank repair and maintenance. Other areas typically occur where the channel changes course or gradient, such as through the Dunnigan Hills and Rio Jesus Maria subreaches. In general, bank erosion can occur anywhere along the creek. Prediction of the next site and its degree of erosion are not possible, however, through regular annual monitoring and channel maintenance to remove significant flow obstructions, stream bank erosion can be greatly reduced.

Other Means for Improving Channel Stability

Other methods of reducing bank erosion include installation of slightly elevated and vegetated berms along the toe of eroding banks. Placement of all gravel haul roads on compacted elevated berms next to the toe of high banks will provide toe buttressing and erosion protection. Avoid placement of haul roads in center of creek. Haul roads that cross the channel should be removed after each mining season so as not to orient flows in the direction of the haul road remnants left in the channel. Abrupt channel gradient changes should be avoided. As an example, those reaches where significant channel incisions have been made by the end of the mining season (such as at the upstream end of the mining reach in subreach 7) are artificially "oversteepened" by mining in the downstream reach. Even though the present ordinance calls for a 1 on 10 transition slope from a mined reach the next reach, this may be too steep if the mined reach is downstream from a high energy reach. Also at the end of a mining season in a reach where bank to bank flat bottom skimming has occurred, the creek will have had no time to establish a seasoned and stable low flow channel. In these locations where the creek is over steepened and the flow is confused as to where the main channel is, an upstream migrating gully, called a nick point can progressively move upstream, resulting in further channel incision, and potential bank instability. If these conditions occur downstream from a bridge, the nick point could migrate to the bridge and contribute to local bridge scour and instability problems. Concrete or rock nick point control sills across the creek, normal the flow direction can reduce these potential problems. Again, regular monitoring and maintenance can control such problems as they arise.

Long-term Simulations

Results in a previous section showed an imbalance between present sediment load and water discharge in most subreaches for single event storms. As a test to see how long it might take the creek to reestablish a more stable channel profile if it were left alone to do so, we simulated 100-years of continuous flow conditions through the study area with no in-channel mining. One hundred years of flows were developed by taking mean daily flow records since 1943 to the present and duplicating the same flows and storm patterns out for 100-years. We substituted 2-year, 10-year and 50-year hourly runoff event hydrographs for the mean daily flows where the hydrologic records indicated storms of similar magnitude had occurred in the historical record. Initial channel cross sections and profiles were from the 1994 channel geometry. Inflowing sediment load was computed from the sediment discharge relationships that were developed and discussed in Section 3.3.

Figure 3.6-12 shows the results from 50 and 100-years of continuous simulations using actual daily flows and sediment loads. After 50 and 100-years the upstream subreaches have adjusted to become slightly depositional (0.5-4.2 percent) rather than sediment starved as seen today. The Highway 505 bridge area remains in sediment deficient as does the R4A (upper Dunnigan Hills) subreach. Subreaches between Stevens Bridge and I-5 continue to be much more aggradational than other subreaches, resulting in the potential long-term reduction in channel capacity through those reaches. This could have long-term impacts on channel flood carrying capacity in the areas where channel capacity is already limited. This emphasizes the need for annual monitoring and maintenance.

Figure 3.6-13 shows the beginning and ending cross section changes that occur after 100-years of flows at a typical cross section located in subreach 7, downstream from the Esparto Bridge. Thick deposits occur in the wide floodplain areas while a reasonably stable low flow channel forms within the floodway.

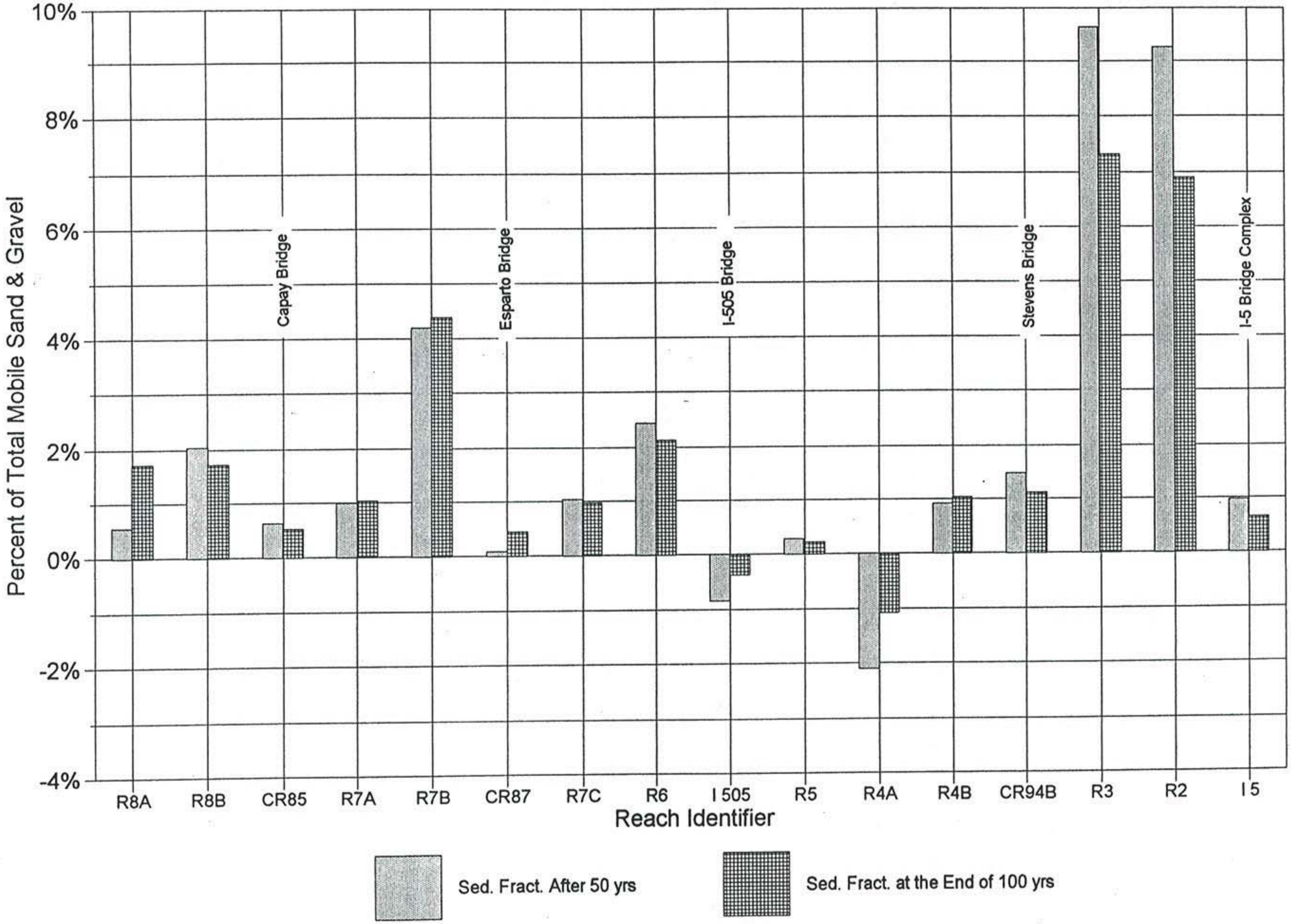


FIGURE 3.6-12 Reach Averaged Sediment Load Changes after 50 and 100 Years of Continuous Simulation

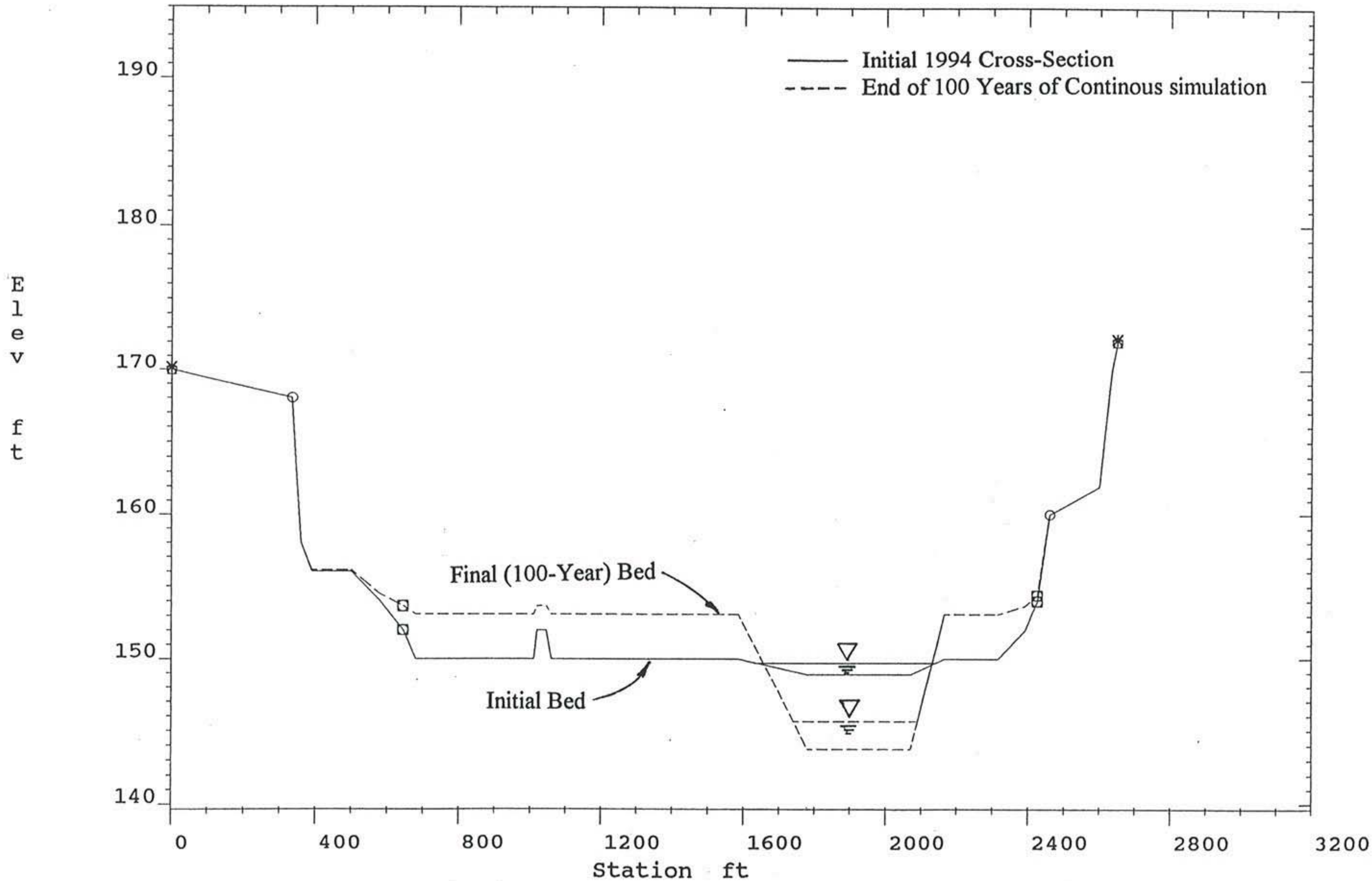


Figure 3.6-13
Typical Channel Adjustments After 100-Years of Continous Flow Simulation

To test of the creek's ability to adjust itself to a more stable condition if left alone, we simulated 100-years of continuous flows with no annual mining occurring in the channel (results are shown in Figure 3.6-12), followed by a 50-year flood event similar to the event which occurred in March 1995. Figure 3.6-14 shows the sediment load is more balanced from reach to reach at the peak of the 50-year event and at the end of the event if it were to occur after 100-years of system recovery with no mining. Figure 3.6-14 is plotted at the same scale as was Figure 3.6-9 for a 50-year event starting with today's (1994) channel conditions. At the peak of the future conditions 50-year event, we see that the Capay, Esparto and I-505 bridges are still in sediment deficit, however, they are much closer to being stable than for today's conditions. The degree of disequilibrium at the bridges is reduced by more than 50 percent. Also noticed are reductions of the imbalances observed at the peak and end of the event for subreaches R7C, R4A, R4B, R3 and R2. These results demonstrate that the river system has adjusted itself to a more balanced configuration after 100-years of uninterrupted channel adjustment with no mining. These results are not intended to say that it will take 100-years to achieve a more stable configuration, or that the 100-year configuration will be the most stable form the creek can attain. The following section explains why attainment of dynamic equilibrium is difficult. Results from this exercise, however, demonstrate that there are opportunities through alternate river management practices to improve today's channel conditions.

Testing of Conceptual Channel Improvement Scenarios

Previous sections discussed results from modeling single event storms and long-term simulations of the creek under present (1994) conditions. The present channel configuration confines the flow energy for large flood events to a much narrower channel than had occurred historically. Basically the 5-year channel width is not much different from the 50- or 100-year width because it's confined. This causes the creek to incise and erode its banks. Realizing this, a series of sensitivity tests were conducted using the HEC-6 sediment transport model to test the effects of widening and smoothing the channel. Three conceptual tests were evaluated: (1) all bridge openings were widened by 30 to 240 feet with no other modifications to the stream, (2) the entire study reach was slightly widened and smoothed so no abrupt width or slope changes occurred from upstream to I-5; this included lengthening all bridges by as much as 300 to 900 feet and (3) the study reach was smoothed to remove abrupt width and slope changes and the channel sections upstream and downstream from bridges were modified to allow smooth flow transitions into and out of the narrow bridge openings; bridge lengths were not changed; channel bottoms beneath the bridges were assumed to be protected against scour during high flows.

For the first test scenario (limited bridge widening alone), there is a measurable improvement at the bridges. Disequilibrium plots, similar to Figures 3.6-7 through 3.6-11 show that merely widening present bridge openings another 10 to 75 percent reduces scour potential, eases backwater conditions caused by the bridges and improves sediment transport between subreaches containing bridges.

For Test 2, the overall channel smoothing and bridge widening scenario, the creek's hydraulic and sediment transport characteristics are markedly improved. Where the bridges used to be 50 to 60 percent in sediment load deficit (scouring) at the peak of a 10-, 50- or 100-year event, they

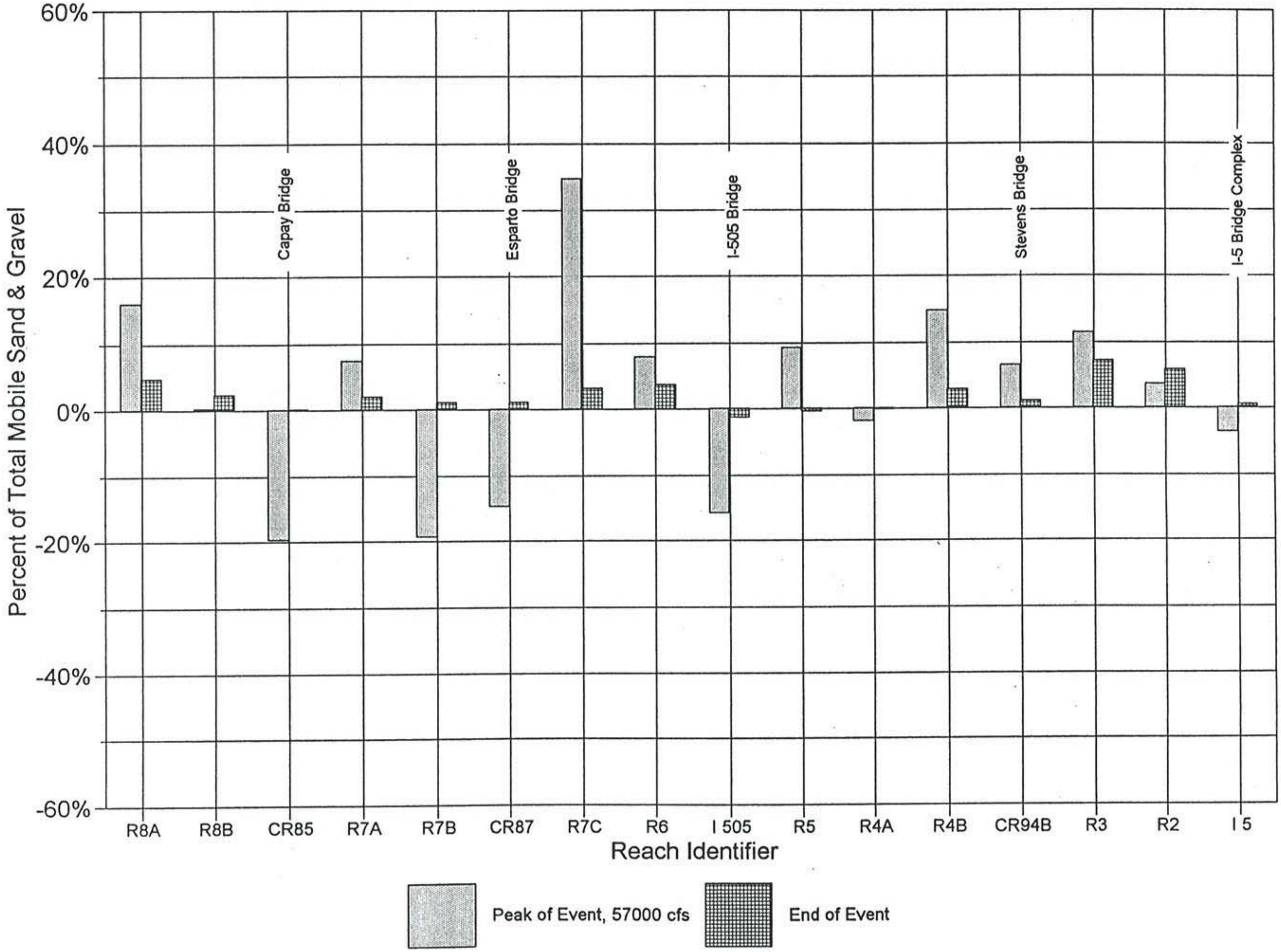


FIGURE 6-14 System Response to a 50-Year Event Following 100 Years of Continuous Simulation and No Mining

are now less than 5 percent in scour disequilibrium. This is an improvement of approximately 92 percent. Disequilibrium plots for the 10-year, March 1995 (50-year), and 100-year events for Test 2 conditions were compared to Figures 3.6-8, 3.6-9 and 3.6-10 and show significant improvement throughout the study reach for all times during the events. Reaches 2 and 3 continue to trap sediment due to limited channel capacity of subreach 2.

Realizing that widening all of the bridges openings may not be feasible, Test 3 evaluates the effects of channel smoothing and protecting the channel bottoms beneath the bridges with no bridge widening. Results from Test 3 are similar to Test 2, however, the magnitude of the disequilibrium bars are slightly increased, with the largest increases near the bridges. All of the Test 3 results are still much improved over present conditions. Figure 3.6-15 presents the disequilibrium plots for Test 3 for a 100-year event.

Figure 3.6-16 presents a map of the study reach showing the approximate channel shape (smoothed to reduce constrictions) used for the sensitivity tests of the Test 3 scenario. The channel was smoothed and slightly widened in some locations; smooth transitions were provided through the bridges; no additional bridge widening occurred and scour control sills were located beneath the bridges to arrest channel incision. Table 3.6-6 summarizes the computed disequilibrium amounts for (i) existing (1994) channel conditions, (ii) Test 1 changes, (iii) Test 2 changes and Test 3 changes. All values shown in Table 3.6-6 reflect system responses to the 100-year flood. It is apparent that bridge widening alone results in measurable improvement to channel stability. Combining channel smoothing with bridge widening in Test 2 produces the most significant net improvements. However, Test 3 results show that channel smoothing with smooth transitions through the bridges (and no bridge widening) produces large benefits as well. These test results are not intended for design purposes. They do, however, support the recommendations proposed in Chapter 6 of this report.

As a final test of system sensitivity to channel smoothing and widening, 50 and 100 years of continuous simulations were run for the Test 3 scenario. The results show a much improved equilibrium condition for the channel system, with a more uniform distribution of sediment load, depth and velocity throughout the study reach. The downstream-most reaches 2 and 3 continue to trap sediment. Unless the downstream control is lessened or removed, these subreaches may require periodic maintenance and sediment removal.

Because today's channel contains more flow and greater depths of flow than the historic channel, it is not possible to return to similar hydraulic conditions as existed in the early 1900s. Figure 3.6-17 compares reach averaged channel velocities and flow depths for the 100-year flood for 1905, 1994 and Test 3 channel conditions. In general, Test 3 conditions reduce average channel depth and velocity, thus improving channel stability and the return of riparian habitat features.

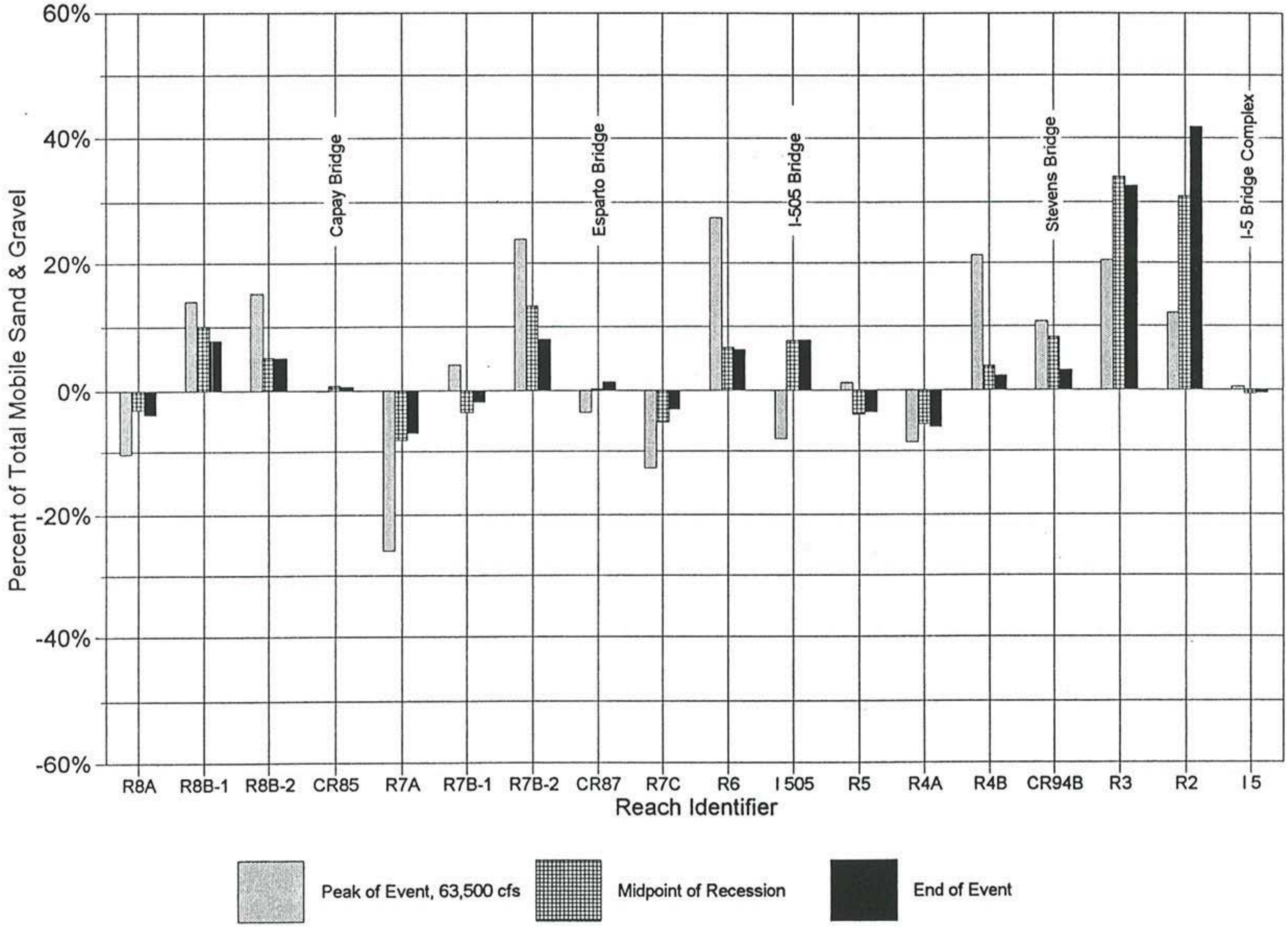


Figure 3.6-15 Test 3 - Reach Averaged Sediment Load Changes for the 100-Year Event

Sensitivity Test Number 3

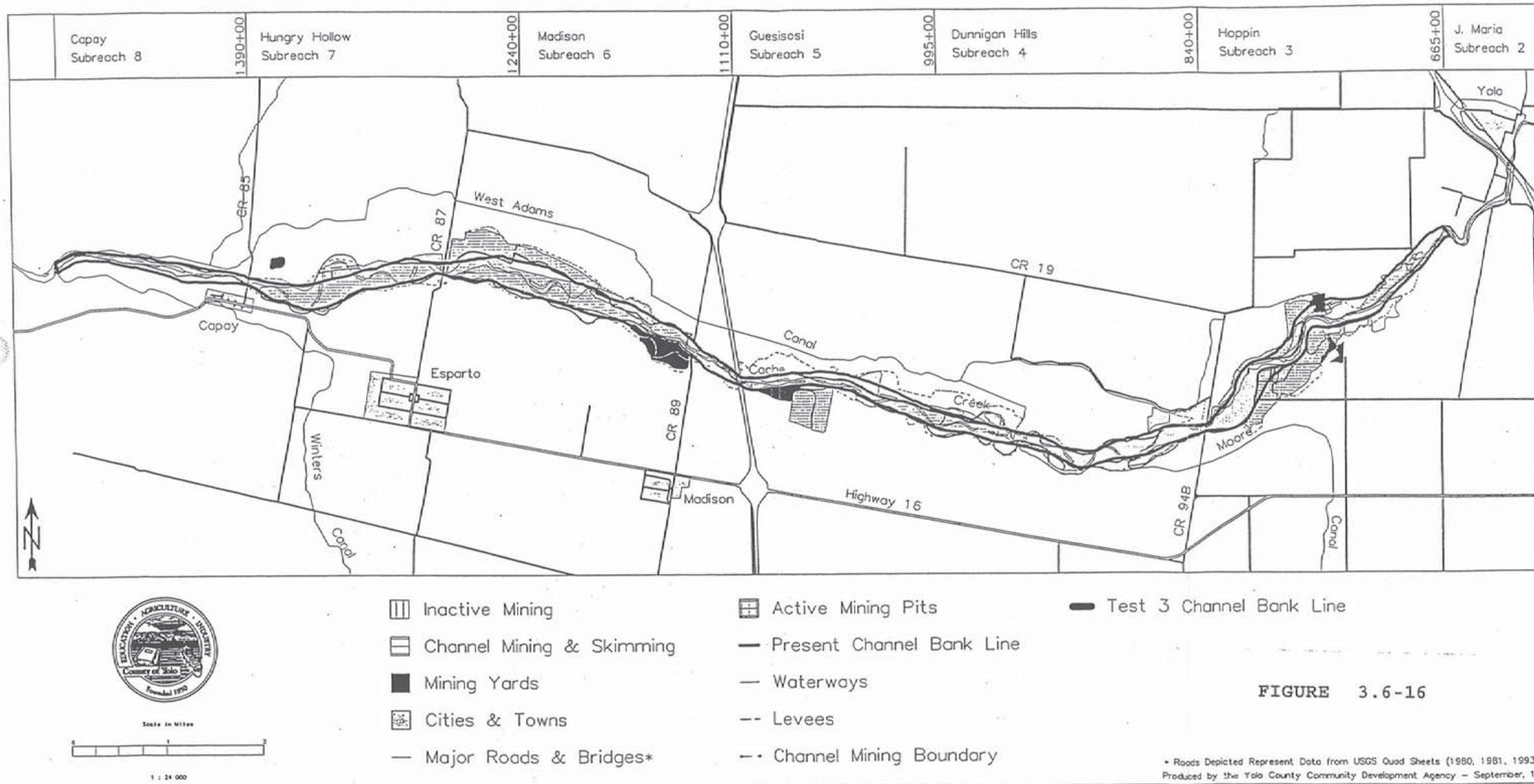


FIGURE 3.6-16

* Roads Depicted Represent Data from USGS Quad Sheets (1980, 1981, 1993)
Produced by the Yolo County Community Development Agency – September, 1995

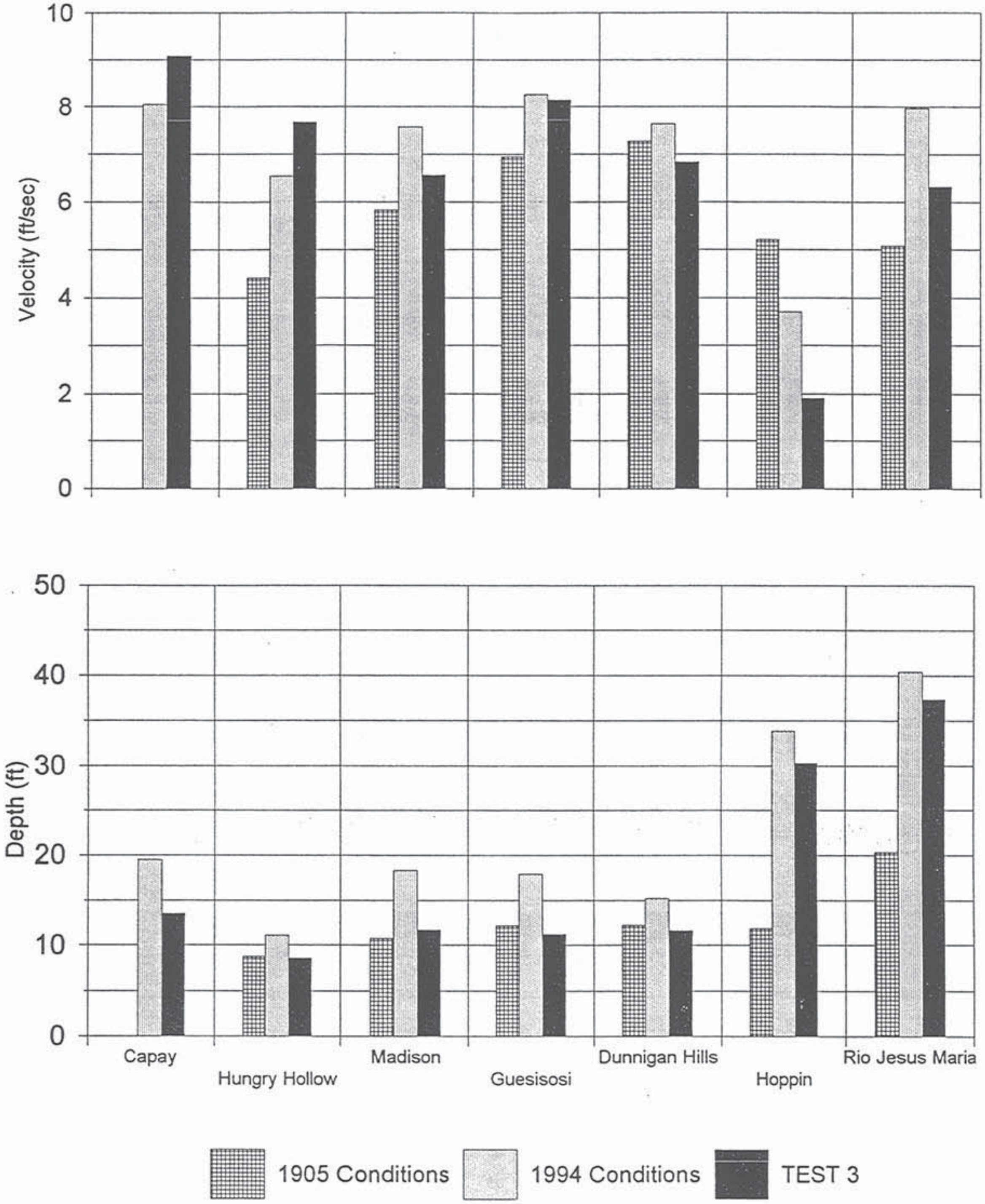


Figure 3.6-17 Reach Average Flow Characteristics for 100-yr Event for 1905, 1994, and Test 3 Conditions

* Capay reach characteristics not available for 1905

Table 3.6-6

Comparison of Sediment Disequilibrium Values for Existing (1994) Conditions versus Three Test Cases, During a 100-year Flood

	Existing			Test 1			Test 2			Test 3		
Reach	8 day	32 day	61 day	8 day	32 day	61 day	8 day	32 day	61 day	8 day	32 day	61 day
R8A	-8.7%	-0.2%	-1.2%	-8.7%	-0.4%	-1.0%	-12.2%	-3.5%	-4.2%	-10.4%	-3.0%	-3.8%
R8B-1	-23.1%	-6.2%	-4.1%	-45.7%	-8.1%	-5.7%	-1.3%	-1.1%	-1.0%	13.9%	10.1%	7.9%
R8B-2	17.5%	11.3%	7.9%	17.3%	10.8%	7.4%	7.0%	1.5%	1.4%	15.2%	5.3%	5.2%
Capay Bridge	-37.4%	-0.5%	0.1%	-0.8%	2.3%	2.1%	-2.6%	0.5%	0.0%	-0.1%	0.8%	0.6%
R7A	16.8%	-1.2%	-0.8%	6.0%	-1.0%	-0.9%	-9.8%	-2.9%	-2.5%	-25.9%	-8.1%	-6.9%
R7B-1	20.2%	4.3%	6.9%	12.7%	3.5%	5.3%	12.7%	4.5%	6.1%	4.0%	-3.6%	-1.8%
R7B-2	11.3%	0.2%	-0.2%	16.1%	8.8%	6.7%	-16.1%	5.5%	6.8%	24.0%	13.1%	8.0%
Esparto Bridge	-63.8%	-2.9%	-1.8%	-31.8%	-3.5%	-0.6%	18.3%	3.0%	1.7%	-3.6%	0.3%	1.3%
R7C	49.8%	6.2%	4.4%	17.9%	0.0%	-0.7%	4.0%	-1.8%	-2.6%	-12.6%	-5.2%	-3.0%
R6	27.0%	3.8%	1.0%	27.0%	3.1%	0.5%	23.3%	5.9%	3.5%	27.5%	6.7%	6.4%
I 505 Bridge	-19.5%	-1.5%	-0.1%	-6.2%	1.8%	2.0%	-3.3%	0.6%	1.3%	-7.9%	7.8%	7.8%
R5	10.7%	1.6%	0.9%	-4.5%	-2.4%	-2.0%	1.1%	-0.2%	-0.3%	1.2%	-4.0%	-3.5%
R4A	-33.2%	-10.8%	-9.1%	-29.1%	-10.2%	-8.0%	-6.6%	-3.7%	-4.3%	-8.5%	-5.5%	-6.0%
R4B	55.6%	6.2%	6.2%	44.5%	4.0%	3.8%	12.9%	-0.2%	-0.3%	21.3%	3.8%	2.4%
Stevens Bridge	-4.6%	-1.4%	-0.4%	20.2%	6.0%	4.9%	21.1%	3.1%	2.5%	10.8%	8.3%	3.1%
R3	36.4%	28.6%	23.3%	21.8%	22.7%	19.3%	19.7%	44.5%	37.0%	20.5%	34.0%	32.6%
R2	5.8%	17.4%	18.5%	7.3%	14.9%	17.4%	13.3%	36.0%	46.6%	12.1%	31.0%	41.7%
I 5 Bridge Complex	1.8%	1.5%	1.9%	-0.3%	0.4%	1.9%	0.5%	-0.7%	-0.5%	0.5%	-0.7%	-0.5%

* Positive values indicate net accumulation (potential aggradation) of sediment to that point in time; negative values indicate sediment loss from a subreach (potential scour) to that point in time.

Channel Response to Altered Conditions

Channel instability and sedimentation have two aspects with respect to managed (project) channels: (1) the impact of existing fluvial system processes on the project channel, and (2) the impact of project changes (channel management, or mining practices) on the stream system both within and beyond the immediate project area. If controlling variables or boundary properties are altered, the stream or channel system may respond by altering its cross section, slope or platform. Figure 3.6-18 shows the generalized relationship for equilibrium and channel response processes in erodible channels. It is often difficult to determine where and to what extent observed post-channel alteration (e.g. mining, levee building or bridge construction) instabilities represent a response to the alternation or whether it might have occurred anyway as a result of pre-alteration processes. This is especially difficult in systems where there is a long history of successive alterations over time. Frequently, it is extremely difficult to distinguish the long term responses of a system to a historical sequence of interferences. In almost every case natural river systems will continue to attempt to adjust to a new state of dynamic equilibrium following an imbalance imposed on the system. The final state of equilibrium for the new set of channel conditions may be far from what the historic channel system looked like. This is most likely the case for Cache Creek. Depending on the spatial extent and degree of instability that has occurred, it may take significant counter measures to re-establish system wide equilibrium.

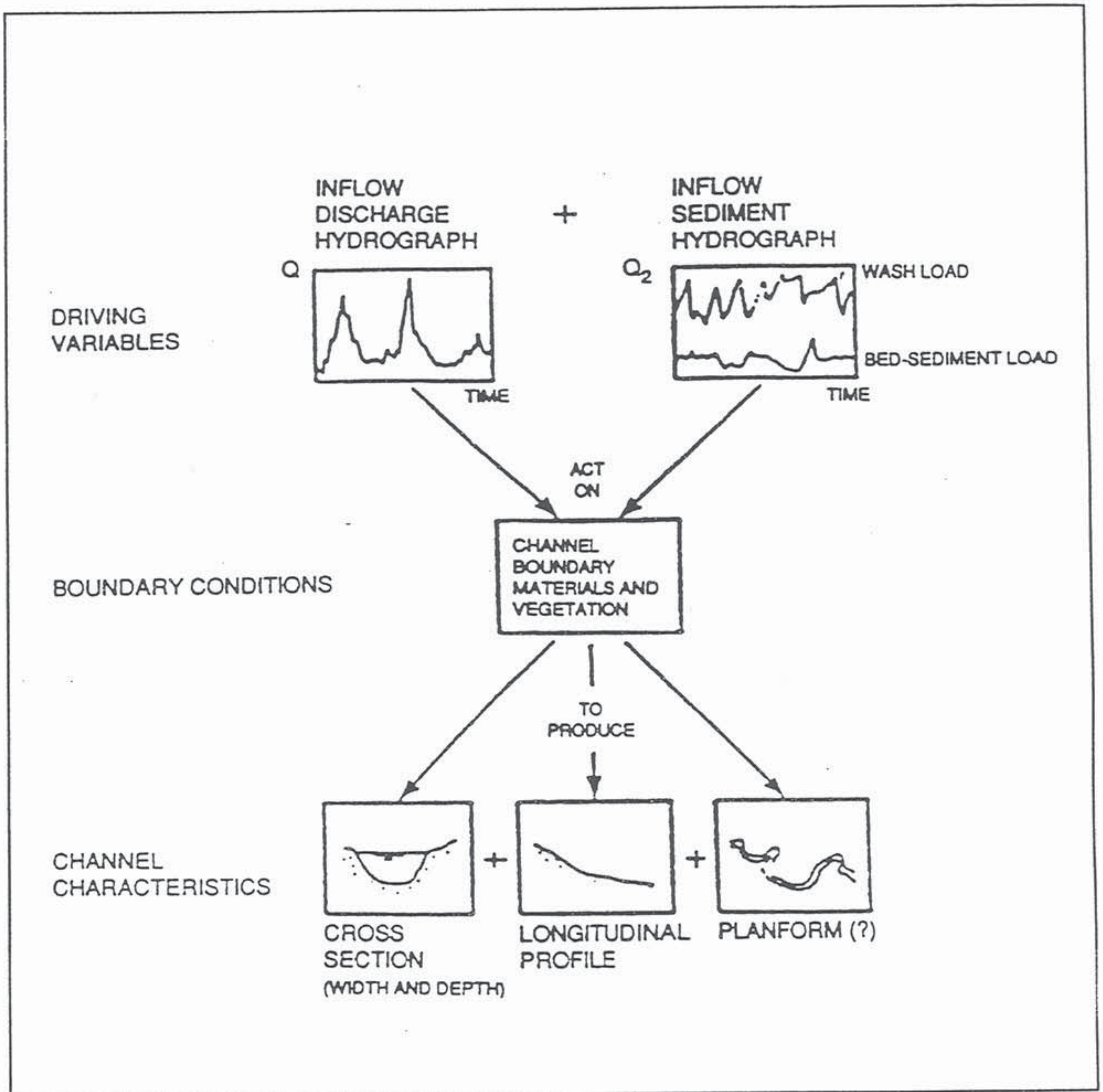
The concept of dynamic equilibrium is best expressed through the simple relationship introduced by Lane[7] in 1955.

$$Q\,S \sim Q_s\,D_{50} \qquad \text{(Eq. 6-1)}$$

where Q = water discharge
S = energy slope
Q_s = sediment discharge
D_{50} = median sediment size

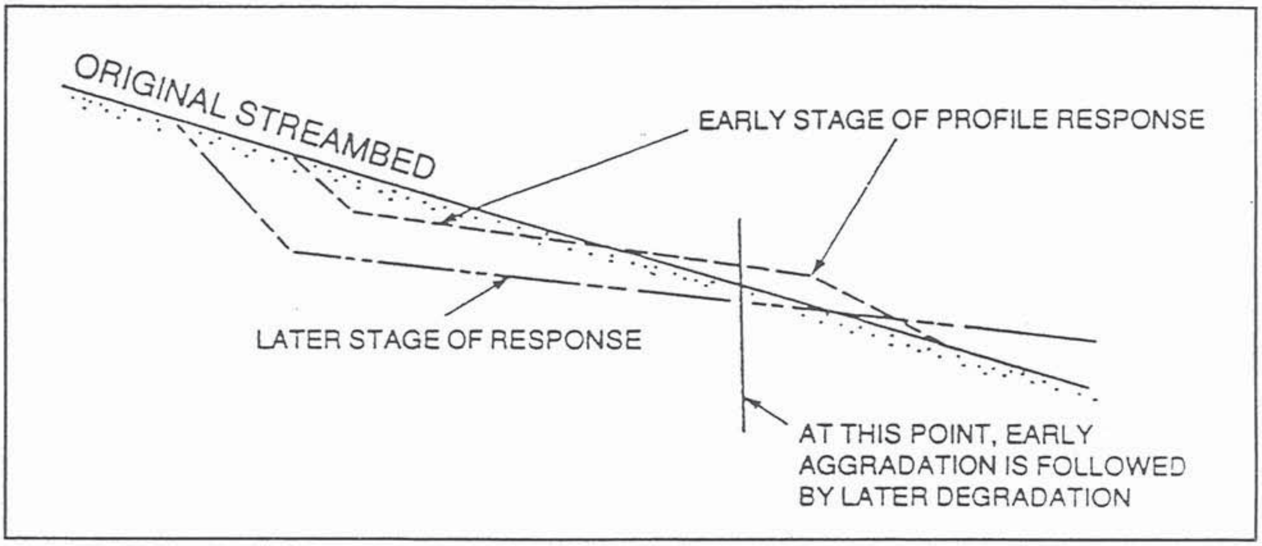
This qualitative relationship means that alluvial channels tend toward a state of equilibrium in which the dominant discharge and slope are in balance with the sediment transport capacity and bed material size. If, for example the sediment supply is increased, all other factors remaining the same, the channel will attempt to steepen (aggrade) to achieve a new state of equilibrium. If, the channel is artificially steepened (perhaps due to mining), all other conditions remaining the same the channel will attempt to flatten its gradient (degrade or scour) itself to achieve a new state of equilibrium. This relationship (Eq. 6-1) is useful for qualitatively assessing the response of an alluvial channel to changed conditions associated with natural or man-induced changes.

Anticipation of channel response to imposed change is difficult. The sequence of responses to certain imposed changes, for example channelization and deepening, can be quite complex. An initial profile response may involve temporary aggradation while later or final conditions may involve degradation below original bed elevations as shown in Figure 3.6-19. Bed scour (degradation) may undercut high over-steepened banks and deliver increased sediment loads that temporarily stop or reverse degradation at downstream points. The equilibrium concept generally



revised from COE EM 1418, 1994

Figure 3.6-18 Generalized Equilibrium Concept for Long-term Formation and Response of Erodible Channels



revised from COE EM 1418, 1994

Figure 3.6-19 Example of Complex Profile Response To Channelization refers to a supposed final condition after the system has responded to a change. In many instances, however, intermediate conditions during the evolution to an eventual equilibrium may be of equal interest and often times the most difficult to predict. In flashy, episodic systems, such as Cache Creek, the system is constantly lurching from one imbalance to the next after large flood events. This long-term imbalance and reduced ability to reestablish equilibrium conditions, including those assisted by riparian vegetation, may be exacerbated by the seasonal diversion of essentially all runoff below Capay Dam during the irrigation season. When the water is shut off the system is left in a static mode until flow energies return to adjust the channel properties again through sediment transport processes. Under the present interim mining ordinance, however, in-channel bar skimming down to the theoretical thalweg is allowed annually, thus artificially imposing a new channel profile on the creek prior to the next runoff season. Therefore, the channel is never given the opportunity to adjust itself to any state of dynamic equilibrium.

Summary

The present Cache Creek channel system is out of balance with the flow and sediment loads entering it. Bridges create significant hydraulic controls (constrictions) in the system. The bridges upstream from subreach 2 are, in general, sediment starved (degradational). Subreach 8 is sediment starved and experienced significant incision during the 1995 floods. Channels typically respond to incision by trying to widen. Therefore, in future years we may see increased bank erosion in those reaches that experienced general bed lowering this year. Subreaches 6 and 7 tend to be depositional. Annual bar skimming to an arbitrary elevation (the theoretical thalweg) eliminates the long term accumulation of sediment, thus affecting the gradient through the reach and the availability of sediment loads to subreaches downstream. Annual bar skimming removes vegetation and disrupts the formation of armor materials and renders the bed surface more erosion prone during subsequent flood events. Reduction of floodplain storage and blockage of natural flood water escape routes has altered the local hydrology (flood peaks and travel time) in the study area. In most subreaches, the channel wants to be much wider than it is presently. Channel flows, depths and velocities have increased through the study area since the early 1900s. Increased hydraulic stresses within the channel may limit the type and survivability of some vegetative species formerly found in the creek. Continuous long-term (100-years) sediment transport simulations indicate that the creek will work on its own toward a more stable configuration (channel slope and compound-shaped cross section). The long-term simulations demonstrated that there are opportunities through alternate river management practices to improve today's channel conditions. These opportunities will be presented in Chapter 6.

ENDNOTES

1. USACE, 1995.

2. USGS, 1915.

3. US Army Corps of Engineers. *HEC-6 Scour and Deposition in Rivers and Reservoirs.* US Army Corps of Engineers, Sacramento District. User's Manual. 1993.

4. Klein and Goldman, 1958; US Army Corps of Engineers, Sediment Engineering Investigation, Cache Creek Basin, California. Clear Lake Channel. US Army Corps of Engineers, Sacramento District. 1988. USGS, 1967 and Harmon, 1989.

5. For more detailed discussions of the Capay Bridge, refer to the Draft Capay Bridge Hydraulics Report prepared for Yolo County Department of Public Works (NHC, June 1995).

6. Chorky, Richard J., Schumm, Stanley A., and Sugden, David E. *Geomorphology.* Metheun & Co., Ltd, New York. 1984.

7. Lane, E.W. "Design of Stable Channels," *Transactions of the American Society of Civil Engineers,* Vol 120, pp 1234-1279. 1955.