Chapter 3. Hydrology and Hydraulic Conditions

Hydrology

Friant Dam and the associated storage at Millerton Lake as well as the distribution of upper San Joaquin River flows via the Madera and Kern canals have radically altered the flows that enter the study area. At the Friant gage (subreach 1A) for the period between 1908 and 1948, the peak flow–frequency analysis indicates that the 2-year peak flow was about 13,000 cfs, the 5-year peak flow was 24,000 cfs, the 10-year peak flow was 35,000 cfs, and the 100-year peak flow was 83,000 cfs (Figure 3.1). Cain (1997), using the period from 1908 to 1940, reports similar values up to the 10-year event, but his 100-year peak discharge estimate of 192,500 cfs is 2.3 times higher. The Corps (1993) estimate for the 100-year unregulated peak discharge based on a 1989 analysis was 137,300 cfs, but this estimate has been revised recently (Corps 1997) to account for the high flows in 1995 and 1997. The revised 100-year unregulated peak discharge is 145,800 cfs.

For the period from 1949 to 1993, the peak flow values for the 2-, 5-, 10- and 100-year events at the Friant gage were 1,200 cfs, 4,200 cfs, 9,000 cfs and 55,000 cfs (Figure 3.2). Cain's (1997) values are a little lower for the 2-, 5- and 10-year events, but his 100-year estimate is higher (77,000 cfs), which is close to the revised Corps (1997) 100-year peak discharge estimate of 75,500 cfs.

In the downstream portion of the study area below the confluence of the mainstem San Joaquin River and the Eastside Bypass, the best gage record upstream of the Merced River confluence is that from the Fremont Ford gage located at the Highway 140 crossing (subreach 5). For the period from 1937 to 1948, a very short period of record, the peak flow discharges for the 2-, 5-, 10- and 100-year events were 3,600 cfs, 5,000 cfs, 6,100 cfs, and 9,200 cfs (Figure 3.3). For the period from 1949 to 1989, the corresponding peak flow discharge estimates are 1,800 cfs, 4,500 cfs, 7,000 cfs, and 23,000 cfs (Figure 3.4).

For the period from 1940 to 1954, the Mendota (subreach 2) and Dos Palos (subreach 4A) gages have very similar peak flow–frequency distributions. The 2-year peak flow estimates for the two gages are 3,000 cfs and 2,700 cfs, respectively. The 5-year estimates are 8,000 cfs and 7,300 cfs, respectively, and the 10-year estimates are 14,000 cfs and 12,000 cfs, respectively. The 100-year peak discharge estimates are 38,000 cfs and 35,000 cfs, respectively. The significance of these peak flow estimates is unclear. The period of record spans the period between construction of Friant Dam and the construction of the flood control project. However, if the record is not too confounded, these peak flow estimates provide an indication of the historic flow regime in that part of the river whose hydrology has been most affected by the flood control project. Peak flow frequency estimates for the four gages are summarized in Table 3.1.

Based on the hydrologic analyses and limited hydraulic modeling of the system (Section 3.2), it appears that the bankfull discharge of the channel (i.e., the threshold of flow rate that causes river stage to spill out of the normal banks of the channel) of the San Joaquin River within the study area had a frequency of between 2 and 5 years, which is within the range of expected values (Leopold et al. 1964, Williams 1978).

Flow duration analysis (a way to assess the probability that flows will equal or exceed a particular discharge for longer durations than just the instantaneous peak) was conducted for a number of gages that span the study area of the San Joaquin River as well as Mud Slough and Salt Slough (Table 3.2). The duration analyses were conducted on an annual basis as well as for the April–May period. At the Friant gage, order-of-magnitude changes in the median (Q_{50}; 50th percentile) discharges are evident in the pre- and post- Friant periods. In the pre-Friant period, the Q_{50} (median discharge) on an annual basis (1,400 cfs) was about 30% of that of the April–May period (4,500 cfs), but in the post-Friant period the Q_{50} values are very similar (140 vs 160 cfs). Annual and April–May flow duration curves for the pre- and post-Friant periods are shown on Figures 3.5 through 3.8.

At the Fremont Ford gage in the pre-Friant period (1938-1948), the annual Q_{50} was 480 cfs and that of the April–May period was 2,400 cfs. In the post-Friant period (1949-1989), the Q_{50} values are very similar (200 and 280 cfs), but in the April–May period, the Q_{50} value is an order of magnitude lower than it was historically (280 vs. 2,400 cfs). Annual and April–May flow-duration curves for the pre- and post-Friant periods are shown on Figures 3.9 through 3.12.

The Mendota and Dos Palos gage records provide some insight into flow durations before the introduction of Delta-Mendota Canal water (1940-1954) and during the filling of Millerton Lake. At the Mendota gage, the annual Q_{50} was 330 cfs and that for the April–May period was about 400 cfs. Downstream of Sack Dam at the Dos Palos gage, the corresponding Q_{50} values were 3.8 and 4.5 cfs, suggesting that the bulk of the flows were being diverted from the channel even before the Delta-Mendota Canal project came on line. The Sack Dam and Arroyo canal were in operation as early as the 1914 CDC survey.

The gage at Mud Slough near Gustine and the gage at Salt Slough near Highway 165 provide some insight into the flow regime of the sloughs from 1986 to 1996 (Table 3.2). The annual and April–May Q_{50} values at the Mud Slough gage are 28 cfs and 33 cfs, respectively. Salt Slough annual and April–May Q_{50} values are 220 cfs and 250 cfs, respectively. The Salt Slough discharges probably reflect the increased discharge of agricultural tailwater since 1985 (Saiki et al. 1993). In normal water years, irrigation tailwater in Mud and Salt Sloughs (about 255,000 acre-feet) accounts for about 44% of the flow in the San Joaquin River above its confluence with the Merced River. In a dry water year, the combined discharges account for about 70% of the San Joaquin River flows. Historically, the combined flows accounted for less than 1% (Moore et al. 1990).

The Gravelly Ford gage record (1987–1997) (Table 3.2) shows that the Q_{50} is about 17 cfs on an annual basis and 24 cfs in the April–May period. These discharges reflect the USBR releases required to satisfy riparian water rights. In contrast, the gage on the San Joaquin River below the Chowchilla Bypass shows that, on an annual basis, there are no flows, but in the April–May period, the Q_{50} is only about 1 cfs.

Hydraulics

Existing hydraulic models (HEC-2 or HEC-RAS) within the study area cover the following areas: RM 136–152 (Bear Creek), RM 191–198 (Firebaugh), RM 243–244 (Highway 99), and RM 245-270 (FMFCD). The models were developed to evaluate flooding limits, and, therefore, they are not well suited to evaluate in-channel flows. A number of cross sections were selected from each model to evaluate existing channel capacities (Table 3.3). Water-surface elevations for the design discharge and other discharge levels that overtop various geomorphic surfaces on the cross section are shown on the individual cross section plots, as are the “n” values that were used for the channel and overbank areas (Figures 3.13 through 3.32). (Manning’s coefficients of friction, or “n” values, are used in hydraulic models and equations to quantify the resistance to flow in a channel segment caused by the combined effects of vegetation, texture of the bank and bed, turbulent flow, and other factors.) Estimated bankfull discharge values for each of the cross sections are listed in Table 3.3. These values are not within-levee capacities, but rather reflect the channel and channel-margin geometries where riparian restoration may be possible.

Within the boundaries of the Bear Creek model (RM 136–152), the design discharge of 10,000 cfs is contained within levees (Figures 3.13 through 3.18). The bankfull discharge of the channel varies from 2,300 cfs at cross section 26 to 5,500 cfs at cross section 24 (Table 3.3). At discharges above the bankfull stage, each of the cross sections shows that there are channel margin areas that can be inundated. However, under existing criteria, attainment of discharge in the required range to achieve inundation of the channel margins depends on routing of flood flows from the Eastside Bypass to the San Joaquin River mainstem via the Mariposa Bypass.

Within the boundaries of the Firebaugh model (RM 191–198), the design discharge of 4,500 cfs is contained within the non-project levees (Figures 3.19 through 3.24). The bankfull discharge of the channel varies from 8,500 cfs at cross section 4150 to about 13,000 cfs at cross section 19180. At all of the cross sections, removal of a local levee or berm on one of the adjacent banks would permit inundation of channel margin areas. Flows in the overbank area would be confined by a secondary non-project levee system that is formed by the Helm, Poso, and Columbia canals. On the west side of the river, the canals are perched on a terrace that also confines the overbank flows (Figure 1.3).

The Highway 99 model (RM 243–244) shows the confinement of flood flows by the terraces that flank the river. The design discharge of 8,000 cfs is confined by the lower terrace at both cross sections (Figures 3.25 and 3.26). The intermediate terrace confines flows up to 15,500 cfs at cross section 125 and up to 29,000 cfs at cross section 300.

Within the boundaries of the FMFCD model (RM 245–270), the design discharge of 8,000 cfs is contained by the lower terrace (Figures 3.27 through 3.32). The intermediate and high terrace confine a much higher range of discharges. The intermediate terrace appears to contain flows up to about 50,000 cfs. Flows in excess of the 100-year peak discharge (75,500 cfs) are contained by the high terrace.

Normal-depth computation routines in HEC-RAS were used to evaluate the hydraulic characteristics of the 12 selected CDC 1914 cross sections. (Locations of cross sections are shown on map in Appendix A.) Table 3.4 summarizes the estimated bankfull discharges for each of the cross sections. As expected, the bankfull discharge values diminish in the downstream direction. From the terrace-bounded reach at cross section 9 (RM 233.3) to the upstream end of the sloughs at about cross section 48 (RM 178.8), the bankfull discharge of the channel varied from about 8,000 cfs to 13,800 cfs (Figures 3.33 through 3.38), which represents a range of frequencies between the 2- and 5-year peak discharges (Table 3.1). The capacity of the San Joaquin River channel diminished in the anabranched reach where the sloughs were conveying a significant portion of the flood flows. From cross section 53 (RM 171) to cross section 81 (RM 125.8), the bankfull capacities of the channel ranged from about 5,000 cfs to 1,200 cfs (Figures 3.39 through 3.43), which again appears to represent a range of frequencies between the 2- and 5-year events (Table 3.1). At the Merced River confluence at cross section 85 (RM 118), the bankfull discharge increased to about 24,000 cfs as the flows from the various sloughs were forced back into a single channel (Figure 3.44).

At the bankfull discharges shown in Table 3.4, average channel velocities range from 2.0 to 2.9 feet per second (ft/sec) between cross section 9 (RM 233.3) and cross section 53 (RM 171). In the anabranched reach between cross section 58 (RM 162.6) and cross section 81 (RM 125.8), the bankfull discharge average velocities are lower and range from 1.3 to 1.9 ft/sec. The average velocity at cross section 85 (RM 118.2) is 3.8 ft/sec.

Normal-depth computation routines in HEC-RAS were used to evaluate the hydraulic characteristics of the 12 selected 1914 cross sections that were resurveyed in 1998 (locations of cross sections are shown on map in Appendix A). Estimated bankfull capacities for each of the cross sections are shown in Table 3.5, as are the computed average velocities at the bankfull stage. The bankfull designation was assigned on the basis of the individual cross section characteristics and was primarily intended to identify the boundary between the within-channel and channel-margin features that could support riparian vegetation. A range of discharges up to and including the design discharge for the individual reaches of the flood control system was run in the model.

The design discharges are contained within the cross sections at all of the locations. In the reach between cross section 9 at RM 233.3 and cross section 19 at RM 222.6, which is located upstream of the Chowchilla Bypass, the bankfull capacity of the channel is around 4,000 cfs (Figures 3.45 through 3.47). In the reach between cross section 29 at RM 201.6 and the head of the sloughs at cross section 53 (RM 171), the bankfull capacity is between 2,000 cfs and 3,000 cfs (Figures 3.48 through 3.51). Downstream of the Sand Slough Control Structure, the bankfull capacity at cross section 58 (RM 162.6) is about 1,200 cfs (Figure 3.52). At cross section 70 (RM 142.7), located downstream of the Mariposa Bypass, the bankfull capacity is about 4,000 cfs (Figure 3.53). The bankfull capacity at cross section 78 (RM 130.1) is about 18,000 cfs, which is abnormally high for this reach of the river (Figure 3.54). The bankfull capacity at cross section 81 (RM 125.8) is about 5,700 cfs (Figure 3.55) and at cross section 85 (RM 118.2) is 14,500 cfs (Figure 3.56).

At the bankfull discharges shown in Table 3.5, average channel flow velocities range from 1.6 ft/sec to 2.8 ft/sec between cross section 9 at RM 233.3 and cross section 19 at RM 222.6. Between Mendota and the Sand Slough Control Structure (cross sections 29, 36, 48, and 53), average velocities range from 1.6 ft/sec to 3.4 ft/sec. At cross sections 58 and 70, the average velocities are about 1.4 ft/sec at the bankfull stage. At cross section 78 at RM 130.1, the average velocity at the bankfull stage is 3.1 ft/sec. At cross section 81 and cross section 85, the average bankfull velocities are 1.2 ft/sec and 2.4 ft/sec, respectively.

In summary, the results of the hydraulic modeling show that, on the whole, average velocities in the study area at the bankfull stage are fairly low, rarely exceeding 3 ft/sec in either the historical (1914) or present (1998) condition. Average velocities are somewhat higher in the upper reaches (> 2 ft/sec) where the slopes are a little steeper than in the lower reaches (< 2 ft/sec). Channel velocity and bankfull capacities are quite similar at some cross sections for the two time periods (Tables 3.4 and 3.5), whereas most other cross sections indicate a trend of decreasing bankfull capacity since 1914, but increasing in sections 78, 81, and 85 at the downstream end of the study area. Channel capacities generally decrease in the downstream direction and then increase at the Merced River confluence and downstream of the Eastside Bypass confluences (Bear Creek and Mariposa Bypass) in the modern survey cross sections.