Effect of Bridge Piers on Streamflow and Channel Geometry

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ABSTRACT

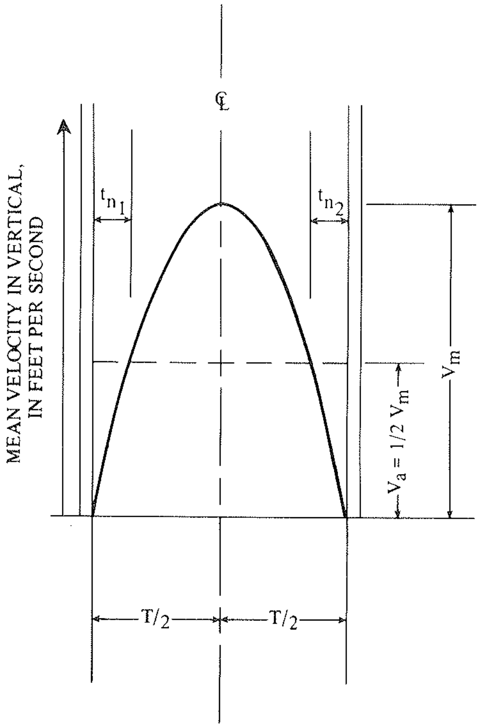
Piers in the waterway affect the velocity distribution across the channel and may act as barriers to floating debris. In addition, they also affect channel geometry by causing general and local scour in the vicinity of the bridge. The level of hydraulic efficiency of an unobstructed channel may be reduced by several percent if piers are placed in the waterway. Reductions of up to 11 percent were observed at sites used in the study. Field studies of several channels indicate that depths of general scour may be greater than local scour. Maximum depths of general scour usually occur midway between piers. General scour may extend several hundred feet upstream and downstream from the bridge. Large piers in alluvial channels may initiate long-term general scour that may continue for decades. General scour at a bridge has been observed to cause enlargement of the channel by as much as 23 percent to compensate for reduced flow area and flow inefficiency associated with bridge piers. Pier design should consider the probability that the channel alignment and geometry will change with time. To reduce the potential for pier damage by streamflow; pier shape, size, spacing, location in the channel, and footing elevation relative to the lowest point in the channel bed should be considered. Pier size should individually or collectively occupy not more than about 5 percent of the original waterway at bankfull discharge.

Bridges across waterways are usually located in response to alignment and traffic considerations, and the length of the bridge is minimized by extending the approach embankments to reduce costs. The bridge designer needs to be aware that the stream will maintain its capacity for the full range of flows that occur at the site. Site alterations as part of bridge construction can cause stream changes such as channel-bed scour, alignment, bank erosion, backwater, and modified flow distribution. In the process, the stream may damage or destroy the bridge and approach embankment.

The purpose of this paper is to describe the effect of bridge piers on streamflow and channel geometry. Two types of scour occur near a bridge: general scour which is related to the flow constriction caused by the bridge piers, abutments, and approach fill, and local scour which is caused by turbulence around the pier. Most of this paper is concerned with general scour caused by piers and abutments. The analysis is based on a comparison of data for unlined canals; streams with cobble, gravel, or sand beds; and streams with bridges. All the data obtained from streamflow measurements of canals, streams, and bridge sites were assembled as part of the annual data collection program or from site-specific studies done by the U.S. Geological Survey. A discussion of the hydraulic characteristics of bridge piers at 10 sites is presented, followed by suggestions to minimize the impact of bridge abutment and piers on a stream.

VELOCITY DISTRIBUTION ACROSS CHANNELS AND CHANNEL EFFICIENCY

The velocity distribution across an open channel that is unobstructed by piers is related to channel shape, alignment, depth of flow, and boundary roughness. The highest velocities at the cross section normally occur midway between the banks where the influence of the boundaries is minimal. For laminar flow in pipes, it has been determined (1) that the rate of velocity change across one-half of a cross section is nonlinear and is proportional to the inverse of the distance from the boundary (Figure 1). Accordingly, the shape of the (lateral) velocity



EXPLANATION

t_n = Width of pipe in which the mean velocity is less than mean for cross section

$V_a = \frac{Q}{A}$ = Mean velocity for cross section

V_m = Maximum velocity for cross section

T = Width of pipe

FIGURE 1 Definition sketch of velocity distribution across a pipe.

profile approximates a parabola and the maximum velocity may be estimated as

$$V_m = 2V_a \qquad (1)$$

where

V_m = maximum velocity in the cross section, and
V_a = average velocity in the cross section.

The velocity distribution curve in a circular pipe is similar along any diameter, with the maximum velocity (V_m) at the center. In open channels, the velocity distribution is usually considered in the vertical or horizontal direction. The velocity distribution among verticals in the cross section is described in this paper, and point velocities or the distribution within any vertical are not considered. The velocity distribution on the horizontal is defined by the average velocity in a vertical for several locations in the cross section. The highest average velocity measured in any of the verticals is designated as V_m.

The most efficient velocity pattern in a channel would have a velocity distribution that is uniform throughout the cross section. This implies no boundary friction, and the ratio of the maximum velocity in any given vertical (V_m) to mean velocity in the cross section (V_a) would be 1.0. The velocity distribution does not reach this condition in pipes and open channels because of the effects of boundary roughness. Velocity distributions for canals usually have the highest level of hydraulic efficiency because bank roughness is at a minimum, as shown in Figure 2. The average ratio of V_m/V_a for the canal

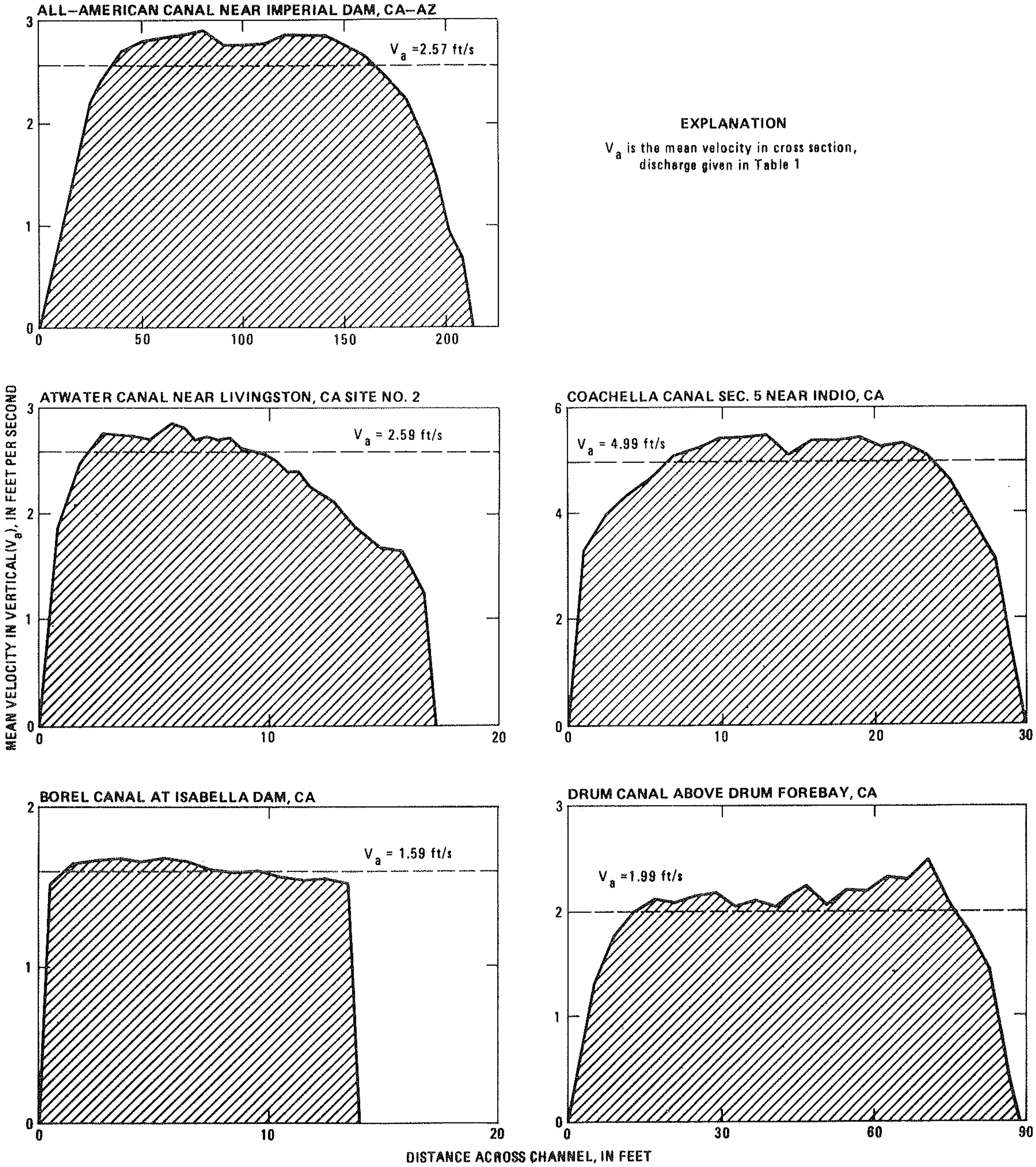


FIGURE 2 Typical velocity distribution across canals.

TABLE 1 Channel and Velocity Properties of Canals

Site	Discharge (ft^3/s)	Mean Depth, d_a (ft)	Mean Velocity, V_a (ft/s)	Ratio of Maximum to Mean		Channel Efficiency, C_e (%)
				Depth, d_m/d_a	Velocity, V_m/V_a	
California						
All-American Canal near Imperial	7,640	13.9	2.57	1.24	1.13	60.5
Arena Canal near Livingston						
Site 1	124	2.7	2.64	1.47	1.08	47.8
Site 2	122	2.6	2.65	1.51	1.15	48.7
Site 3	126	2.7	2.65	1.48	1.12	46.8
Atwater Canal near Atwater						
Site 1	123	2.7	2.66	1.54	1.17	51.4
Site 2	115	2.6	2.61	1.59	1.10	43.1
Site 3	117	2.3	3.12	1.57	1.17	46.8
Site 4	115	2.3	3.11	1.54	1.12	41.4
Borel Canal at Isabella Dam	108	4.9	1.59	1.00	1.05	61.9
Bowman Spaulding Canal at Jordan	292	4.5	4.35	1.25	1.12	47.9
Chicago Park Plume near Dutch Flat	442	4.8	5.10	1.00	1.13	56.9
Coachella Canal near Indio						
Section 1	911	5.2	2.20	1.12	1.13	67.7
Section 2	908	5.3	2.21	1.15	1.10	67.2
Section 3	884	4.7	2.33	1.17	1.20	53.9
Section 4	898	5.1	2.16	1.12	1.12	69.0
Section 5	923	5.2	1.99	1.12	1.25	70.4
Drum Canal at Tunnel Outlet	569	6.5	3.90	1.17	1.07	56.3
Drum Canal above Drum Forebay	669	4.5	4.99	1.14	1.10	58.7
Dutch Flat Flume no. 2	636	6.1	7.45	1.01	1.08	62.9
South San Joaquin Canal	266	3.1	5.87	1.21	1.07	59.2
Indiana						
King Hall Canal near Hagerman	342	4.3	6.42	1.00	1.09	65.2
Salmon River Canal near Rogerson	485	6.1	2.58	1.15	1.29	50.5
Utah						
Westside Canal near Collinston	660	5.9	6.23	1.00	1.09	59.5

sites listed in Table 1 was 1.13. Velocity distributions for selected natural channels, which are usually affected by more bank roughness than canals, are shown in Figure 3. For the natural channels, given in Table 2, the average ratio of V_m/V_a was 1.34. In general, increasing boundary roughness translates to a channel with lesser hydraulic efficiency, and a corresponding increase in the ratio of maximum to mean velocity.

The hydraulic efficiency of a channel can also be measured by the amount of channel width in which the mean velocity in the vertical is less than the mean velocity for the cross section, as illustrated in Figure 1. That part of the channel near the boundary with velocities less than the mean for the cross section is designated t_n. The channel efficiency ratio (C_e), in percent, is computed by the equation

$$C_e = [1 - (\Sigma t_n/T)]\ 100 \qquad (2)$$

where

t_n = the width of each flow area (including piers) where the mean velocity in one or more verticals is less than the cross section average, and
T = width of channel.

At bridge sites, the value of T is determined as the gross width between abutments.

The channel efficiency ratio (C_e), which indicates the degree of contraction caused by the bridge, has been developed in preference to the channel contraction ratio (m). The channel contraction ratio (m) as discussed by Matthai (2) is a measure of the proportion of total flow that enters the contraction from upstream of the embankments. The ratio m is based on the geometry of the main channel and flood plain, and may be computed by the equation

$$m \cong [1 - (q/Q_t)] \qquad (3)$$

where

q = discharge that could pass through the opening without contraction, and
Q_t = total discharge.

In comparison, the channel efficiency ratio (C_e) is a measure of the impact of the bridge structure on flows by evaluating the reduction in channel efficiency at the bridge opening. Any flow contraction created by the return of overbank flow into the channel at the bridge is incorporated in the estimate of channel efficiency (C_e). Velocity is a measure of the potential for channel scour; therefore, the channel efficiency (C_e) has been selected as an indicator of the impact that a bridge structure has on the channel geometry. At many bridges, problems of scour and bank erosion occur when flows are confined to the main channel and there is no overbank flow.

To develop a comparison of the hydraulic efficiency of various open channels, flow and geometry data were assembled for selected canals (Table 1), natural streams (Table 2), and streams affected by bridge piers and abutments (Table 3). The sites were selected to provide a sample of diverse hydraulic and channel conditions. If the velocity ratio (V_m/V_a) is used as a measure of efficiency, the natural channels that are obstructed by bridge piers or abutments are, on an average, 35 percent less efficient than unobstructed canals (Table 4). For streams with bridges, the mean velocity (V_a) is computed as the mean velocity for the net opening of the bridge. On the other hand, if channel efficiency (C_e) is used, the unobstructed natural channels are 17 percent less efficient than canals (Table 5).

A test of the variation in the means of the two measures (Tables 4 and 5) used to indicate that the hydraulic efficiency of various channels was made by analysis of variance. The test consisted of analyzing for significant differences of the mean velocity ratio (V_m/V_a) and the channel efficiency ratio (C_e).

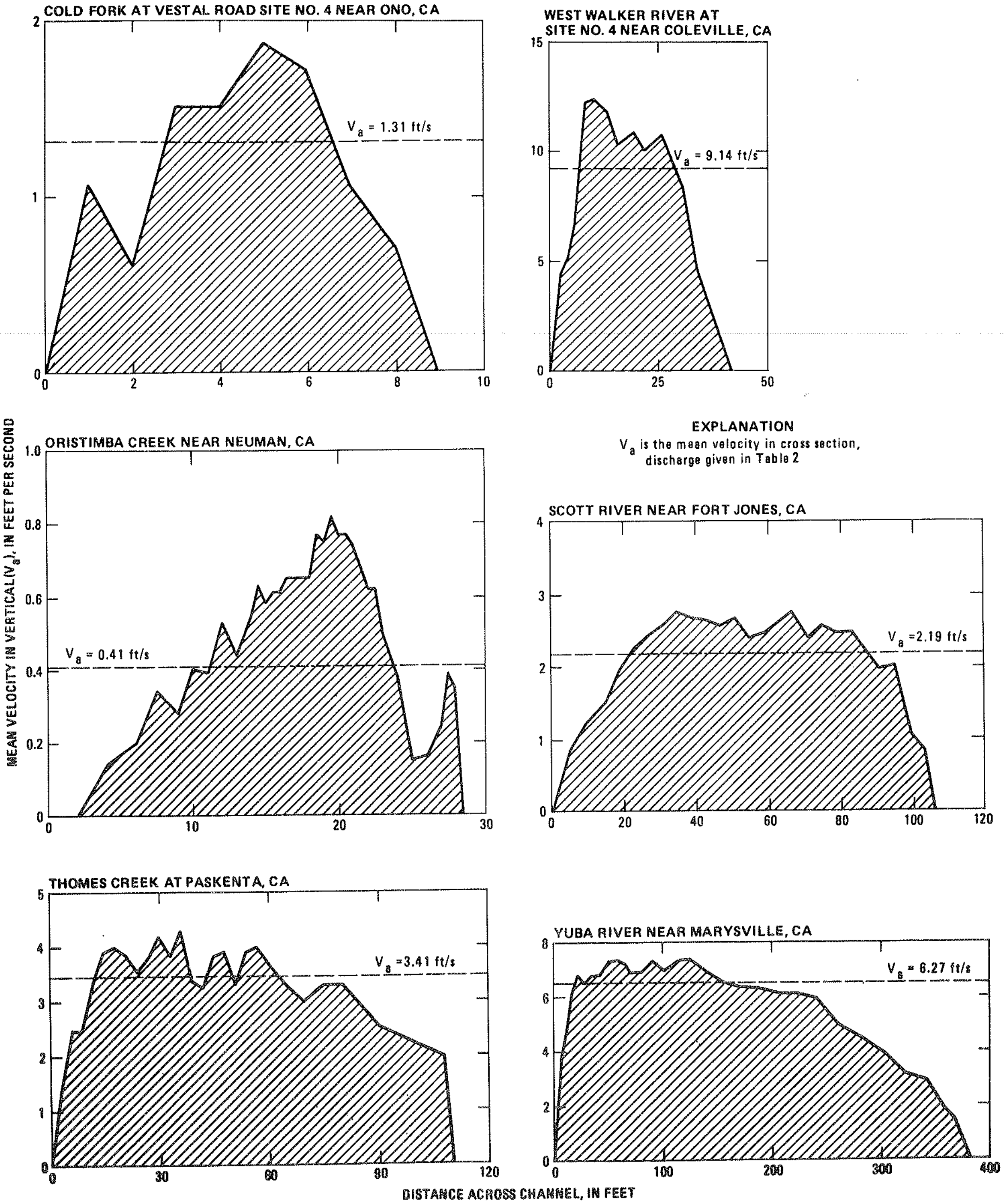


FIGURE 3 Typical velocity distributions across stream channels.

Results of the test indicated that the means of the two measures were significantly different for each of the channel conditions (canals, streams, and streams with bridges). For example, the means 1.13, 1.34, and 1.53 given in Table 4 are statistically different. The probability is less than 1 percent that the observed difference in the means of the velocity ratio or channel efficiency ratio would have occurred by chance.

EFFECT OF PIERS ON STREAMFLOW

Velocity Distribution Across Channels

The placement of piers and abutments in a previously unobstructed channel reduces the flow area in proportion to the size of the piers and footings. The velocity distribution across the channel is now affected by a new set of boundaries imposed by the channel bed, the sides of each pier, and possibly the abutments. Although boundary shear along pier faces may be minimal, piers can cause localized turbulence that may be more significant than the original bank boundary roughness. A definition sketch showing the velocity distribution in a channel with piers is shown in Figure 4.

The effect of piers on streamflow as indicated by the ratio A_p/A_g, where A_p is area of piers and A_g is gross area of cross section, is given in Table 3. For

TABLE 2 Channel and Velocity Properties of Streams

Site	Discharge (ft³/s)	Mean Depth, d_a (ft)	Mean Velocity, V_a (ft/s)	Ratio of Maximum to Mean		Channel Efficiency, C_e (%)
				Depth, d_m/d_a	Velocity, V_m/V_a	
Arizona						
Hassayampa River at Bos Damsite near Wickenburg	316	0.82	5.02	1.40	1.30	49.8
Santa Cruz River at Nogales	638	0.99	3.69	1.52	1.34	61.0
Santa Maria River near Bagdad	301	0.68	2.26	2.35	1.47	40.7
California						
Cold Fork at Vestal Road near Ono	4.89	0.42	1.31	1.52	1.44	43.7
Klamath River near Seiad Valley	15,100	4.3	7.06	1.29	1.37	47.7
Mokelumne River near Mokelumne Hill	782	3.09	2.53	1.42	1.25	50.0
North Fork Cottonwood Creek near Ono	29.7	0.59	1.21	1.90	1.36	50.0
Oristimba Creek near Newman	12.1	0.99	0.41	1.76	2.00	47.8
Sacramento River above Bend Bridge near Red Bluff	35,200	14.93	5.75	1.15	1.33	54.4
Sacramento River at						
Colusa	45,900	20.54	4.47	1.84	1.30	55.7
Verona	20,800	15.93	2.48	1.31	1.35	62.2
Butte City	12,600	10.61	2.50	1.14	1.22	67.0
Hamilton City	9,680	4.66	3.19	1.18	1.19	55.9
Scott River near Fort Jones	640	2.75	2.19	1.27	1.25	61.6
Thomas Creek at Paskenta	529	1.11	3.41	2.27	1.25	41.2
West Walker River no. 4 near Coleville	1,280	3.33	9.14	1.44	1.37	45.2
Yuba River near Marysville	11,600	4.96	6.27	2.14	1.18	53.3

TABLE 3 Channel and Velocity Properties of Streams with Bridge Piers

Site	Discharge (1000 ft³/s)	Mean Depth, d_a (ft)	Mean Velocity, V_a (ft/s)	Ratio of Maximum to Mean		Channel Efficiency, C_e (%)	Ratio A_p/A_g (%)
				Depth, d_m/d_a	Velocity, V_m/V_a		
California							
Angel Slough at Ord Ferry Road near Ord Bend	12.4	6.39	5.79	1.49	1.73	41.5	4.6
	3.34	3.77	2.65	1.91	2.00	35.3	4.6
	0.442	3.19	0.92	1.63	2.33	41.0	5.3
Butte Creek at Gridley-Colusa Road near Gridley[a]	3.60	9.41	2.68	2.39	1.39	28.2	1.4
Butte Creek Overflow at SR-162 at Butte City (BR11-26)	1.86	5.00	2.10	1.48	1.36	53.1	5.0
Dry Creek at County Road at Galt	6.11	11.7	3.49	1.50	2.37	43.0	4.1
Honcut Creek at SR-70 near Live Oak							
Bridge 1	0.698	5.18	2.17	1.39	1.43	50.0	3.2
Bridge 2	2.77	5.70	3.24	1.75	1.61	47.1	2.0
Bridge 3	3.85	6.31	2.82	2.57	1.53	54.1	4.5
Sacramento River at							
Delta	12.5	6.60	9.47	1.79	1.34	36.9	7.2
	15.3	7.76	9.62	1.61	1.50	35.9	7.2
SR-32 at Hamilton City	155	27.7	9.74	1.49	1.31	46.6	7.6
	144	24.9	10.02	1.35	1.27	55.2	7.9
	134	25.0	9.35	1.45	1.19	55.1	7.8
	119	24.9	8.32	1.32	1.27	50.9	7.5
	87.7	20.3	7.57	1.29	1.31	52.3	7.5
	78.8	19.1	7.25	1.33	1.17	50.9	7.0
	57.6	16.8	5.91	1.48	1.33	52.9	5.3
	10.1	6.71	2.63	1.82	1.32	50.8	1.1
Ord Ferry Road near Ord Bend	107	22.4	3.79	1.71	2.15	45.2	4.2
SR-162 at Butte City	92.3	30.3	6.18	1.52	1.57	40.2	15.0
County Road at Colusa (Old Br.)	40.2	25.2	4.38	1.67	1.36	40.0	10.5
County Road at Colusa (New Br.)	33.4	24.1	3.83	1.49	1.28	65.1	2.0

[a] Wall-type piers subject to 19 degree angle of flow.

most bridges, the proportion of the waterway occupied by piers is less than 5 percent. Extra bridge maintenance problems associated with general scour and lateral erosion have been observed at sites where bridge piers occupy more than 5 percent of the waterway. In general, the following flow and channel changes will occur after bridge construction:

1. Local increase in flow velocity,
2. General scour in vicinity of bridge,
3. Local scour around piers and abutments,
4. Backwater from flow constriction or debris lodged against the piers, and
5. Increased flow turbulence near boundaries.

The magnitude of these changes is dependent on the occurrence of floods and amount of flow constriction imposed by the bridge.

Measurements of the reduction in channel size and scour caused by the construction of a bridge are

TABLE 4 Comparison of Channel Hydraulics for Different Stream Types on the Basis of Maximum and Mean Velocity

Type of Stream	Sample Size	Ratio of Maximum to Mean Velocity at a Cross Section			Average Increase in Velocity Ratio in Relation to Canals (%)
		Minimum	Maximum	Mean	
Canal	26	1.05	1.29	1.13	0
Stream channels	18	1.17	2.00	1.34	18.6
Channels with bridge piers	23	1.19	2.37	1.53	35.4

TABLE 5 Comparison of Channel Hydraulics for Different Stream Types on the Basis of Channel Efficiency Ratio

Type of Stream	Sample Size	Channel Efficiency Ratio at a Cross Section (%)			Average Decrease in Channel Efficiency in Relation to Canals (%)
		Minimum	Maximum	Mean	
Canals	26	41.4	70.4	55.9	0
Stream channels	18	40.7	67.0	52.8	5.5
Channels with bridge piers	23	28.2	65.1	46.6	16.6

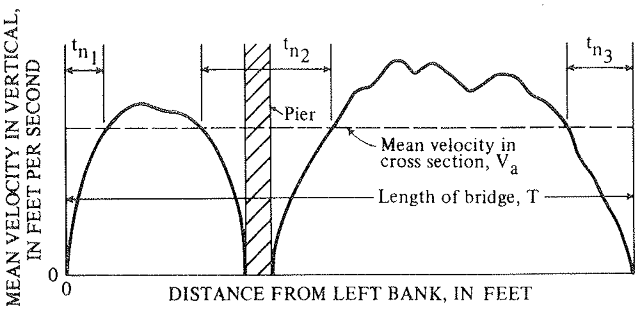


FIGURE 4 Definition sketch of velocity distribution across a channel with bridge piers.

difficult to obtain. Immediately during and after a bridge is constructed, the channel geometry changes in the vicinity of the bridge site. Moreover, most hydraulic data collected at the bridge correspond to postbridge construction conditions.

Assuming no general scour or lateral erosion at the bridge site, the effective flow area is reduced by the flow constriction (vena contracta) between piers and the turbulence associated with overbank flow returning to the opening. The amount of constriction caused by the bridge at the crossing is a function of the pier size, pier shape, number of piers or bents in the waterway, and amount of debris lodged against the pier. If the bridge superstructure is inundated, the wetted perimeter and amount of flow constriction is increased further. The influence of piers on the magnitude and velocity distribution in the bridge opening is shown in Figure 5 for 10 sites. Below each velocity distribution plot, a plan of the bridge showing the location, relative size, and the shape of the piers or bents is shown. In general, bridges with large piers (sites 7a, 7b, 9, and 10a in Figure 5) affect the velocity distribution across the channel much more than bridges with narrow piers (sites 1, 3, and 10b in Figure 5). The size, shape, and placement of the bridge piers also affect the velocity distribution (sites 2, 4, and 5 in Figure 5).

The shape and size of a wall-type pier, described by Brice and Blodgett (3), influences the amount of flow disturbance near a pier, especially at skewed crossings. If the pier is not aligned with the flow, the wall of the pier acts as a barrier and reduces the effective channel flow area. The effect of a wall-type pier on Butte Creek at Gridley-Colusa road near Gridley, California (site 2), is shown in Figure 5. At this site, flow velocities for about 25 ft of the channel width were reduced to zero or negative values, giving a channel efficiency ratio of 28.2 percent (Table 3). This site illustrates the adverse effect of wall-type piers that may occur if they are not situated parallel to the flow.

The variation in velocity distribution is related to the magnitude of velocity at a site. For example, the pile location, size, and spacing for Butte Creek overflow (site 1, Figure 5) and Angel Slough (site 4, Figure 5) are similar, but the amount of velocity distribution disturbance at these sites is very different. Flows are confined for both channels and the effect of vegetation is minimal. The variation in velocity profiles between bents for these bridges is attributed to the low mean velocity (2.1 ft/s) on Butte Creek overflow and the high velocity (5.8 ft/s) on Angel Slough.

The pattern of velocity distribution across an opening is generally the same for different flows, provided no changes in the channel alignment occur. The stability of velocity patterns in a bridge opening is shown for the Sacramento River at Hamilton City (site 7a and 7b, Figure 5). In this case, the shape of the velocity profile for discharges of 10,100 and 155,000 ft³/s are similar in each opening; only the magnitude of velocity is different for the two measurements.

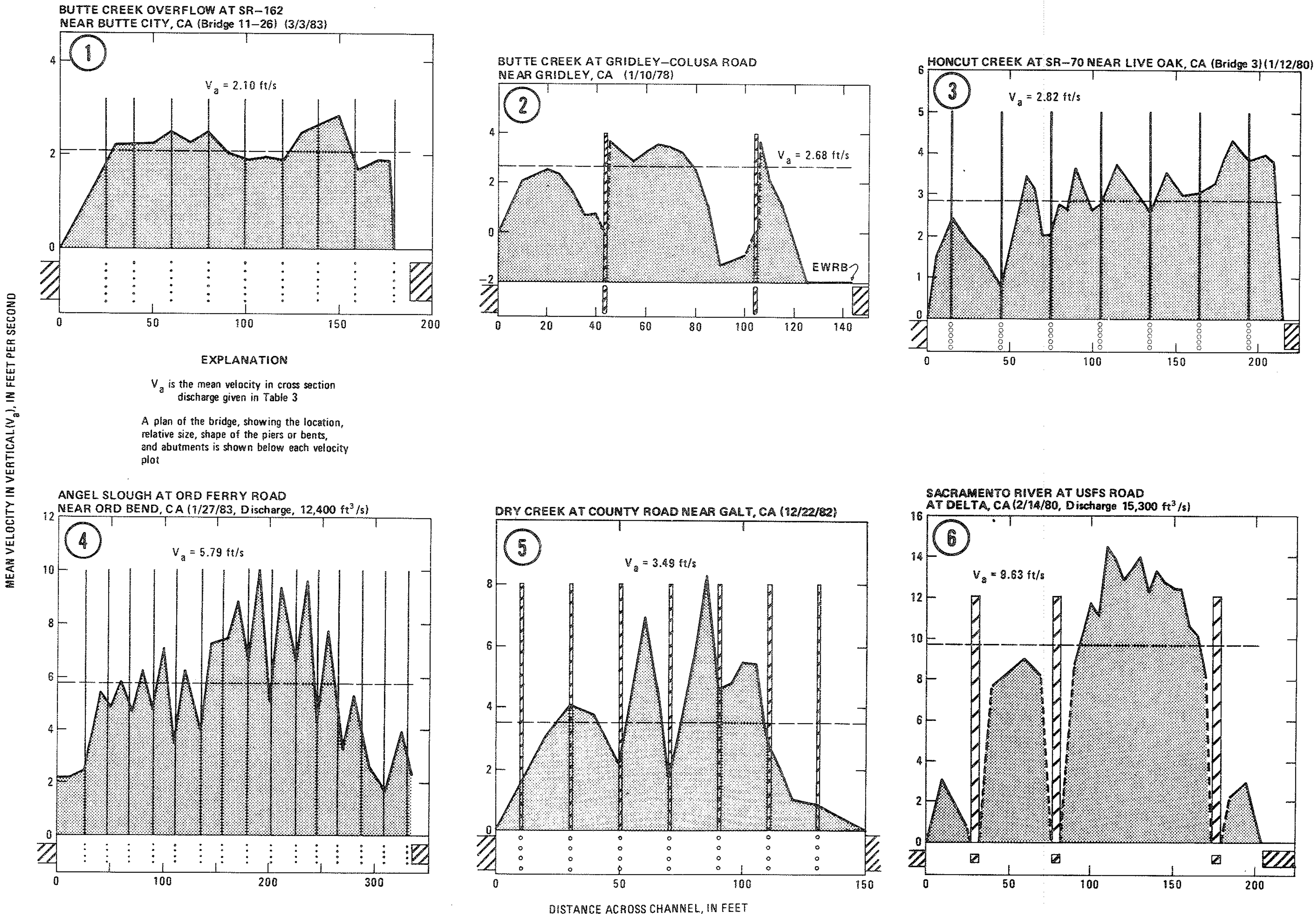
The change in channel hydraulics that occurs when a new bridge with small streamlined piers is built is indicated by the different velocity distributions for the Sacramento River at Colusa (site 10a and 10b, Figure 5). For the old bridge, the ratio of pier area to gross area (A_p/A_g) was 10.5 (Table 3); the velocity ratio (V_m/V_a) was 1.36; and the channel efficiency (C_e) was 40 percent. The corresponding values for the new, more efficient bridge were 2.0, 1.28, and 65.1 percent.

A review of the data in table 3 and the plots in Figure 5 indicate that some characteristics of flow velocity in relation to bridge piers for the 10 sites may be summarized as follows:

1. The pattern of flow velocity across the opening is consistent, regardless of the discharge, provided the channel alignment is stable (sites 7a and 7b).
2. Differences in mean velocities of over 12 ft/s in the vertical between the middle of the span and pier face (site 7b) have been observed.
3. The alignment or geometry of a channel in the vicinity of the bridge significantly affects the velocity pattern if wall-type piers are not placed parallel to the flow (site 2).
4. The amount of velocity profile disturbance in a cross section is related to the number, width, and shape of the piers (sites 4, 10a, and 10b).
5. The amount of velocity distribution disturbance is apparently related to the mean velocity at the cross section (sites 1 and 4).

Maximum Velocity

The mean ratio of maximum velocity (V_m) to mean velocity (V_a) for natural channels that are unaffected by piers (Table 2) averages about 1.3 and is



BUTTE CREEK OVERFLOW AT SR–162 NEAR BUTTE CITY, CA (Bridge 11–26) (3/3/83)

BUTTE CREEK AT GRIDLEY–COLUSA ROAD NEAR GRIDLEY, CA (1/10/78)

HONCUT CREEK AT SR–70 NEAR LIVE OAK, CA (Bridge 3) (1/12/80)

ANGEL SLOUGH AT ORD FERRY ROAD NEAR ORD BEND, CA (1/27/83, Discharge, 12,400 ft³/s)

DRY CREEK AT COUNTY ROAD NEAR GALT, CA (12/22/82)

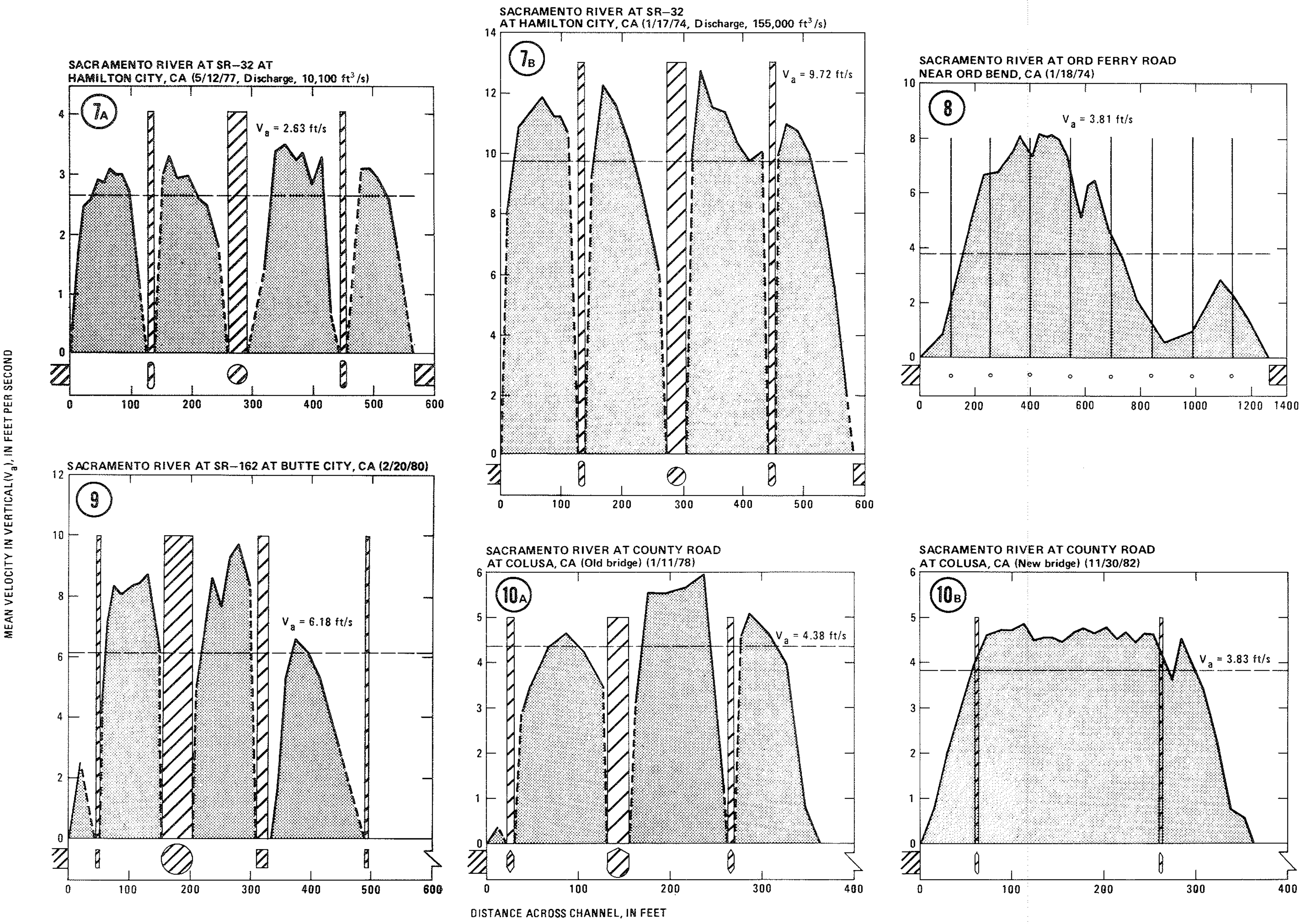
SACRAMENTO RIVER AT USFS ROAD AT DELTA, CA (2/14/80, Discharge 15,300 ft³/s)

EXPLANATION

V_a is the mean velocity in cross section discharge given in Table 3

A plan of the bridge, showing the location, relative size, shape of the piers or bents, and abutments is shown below each velocity plot

FIGURE 5 Typical velocity distribution across stream channels with bridge piers.

 SACRAMENTO RIVER AT SR–32 AT HAMILTON CITY, CA (5/12/77, Discharge, 10,100 ft³/s)

SACRAMENTO RIVER AT SR–32 AT HAMILTON CITY, CA (1/17/74, Discharge, 155,000 ft³/s)

SACRAMENTO RIVER AT ORD FERRY ROAD NEAR ORD BEND, CA (1/18/74)

SACRAMENTO RIVER AT SR–162 AT BUTTE CITY, CA (2/20/80)

SACRAMENTO RIVER AT COUNTY ROAD AT COLUSA, CA (Old bridge) (1/11/78)

SACRAMENTO RIVER AT COUNTY ROAD AT COLUSA, CA (New bridge) (11/30/82)

FIGURE 5 (continued)

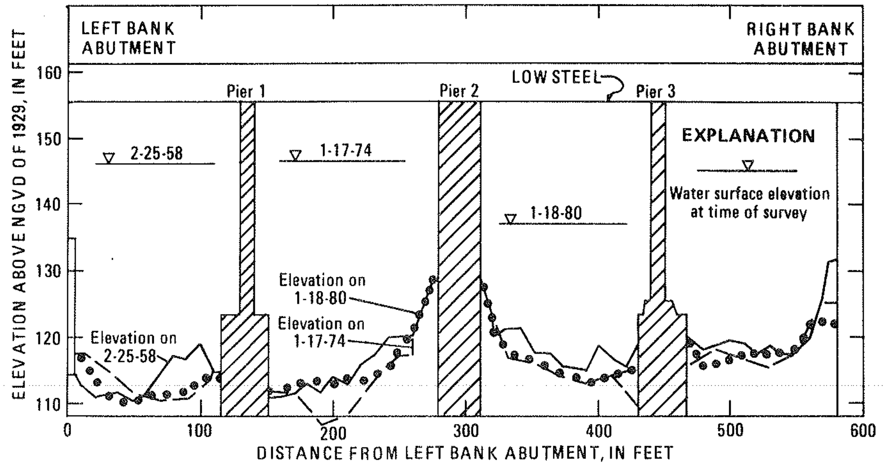


FIGURE 6 Changes in scour between 1958 and 1980 at downstream side of SR-32 bridge, Sacramento River at Hamilton City, California, adapted from Blodgett (4).

nearly constant regardless of discharge or channel size. For channels with piers (Table 3), the mean ratio of V_m/V_a increases to 1.5. The ratio of maximum to mean velocities (V_m/V_a) for the channels with piers is about 14 percent greater than channels without piers. The ratio is affected at low flow by local effects such as piers, boundary roughness, grass, and brush. This is indicated by the increase in the ratio V_m/V_a for decreasing discharges of Angel Slough at Ord Ferry Road near Ord Bend, California (Table 3). These estimates of the relation between maximum and mean velocities are limited to flows in the main channel (less than bankfull).

EFFECT OF PIERS ON CHANNEL GEOMETRY

Depth of Scour

The lateral constriction of flow caused by a bridge usually results in a general lowering of the streambed. This lowering of the streambed is defined as general scour. The depth of general scour is related to the amount of lateral constriction of flow caused by the piers and abutment. Scour is computed as the difference in elevation between the normal and present channel bed at the bridge. In most cases, it was necessary to estimate the normal bed elevation on the basis of the overall channel-bed profile that may extend several hundred feet upstream and downstream from the constriction.

Following bridge construction, the depth and lateral extent of general scour in channels that have sand-and-gravel beds and stable alignment do not change greatly once a state of channel equilibrium is obtained, as illustrated in Figure 6.

Turbulence around piers and abutments causes additional scour defined as local scour (3). The depths and extent of local scour are difficult to measure because soundings near a pier are difficult to obtain.

Both general and local scour may occur at a site. The combined effect of these types of scour may produce unequal depths of scour across the channel. The effects of the piers on depths of scour indicated by the mean ratio d_m/d_a for bridge sites is 1.63 (Table 3). In comparison, the corresponding ratio is 1.57 for natural (unobstructed) streams (Table 2) and 1.23 for canals (Table 1).

The location of maximum scour depths at a bridge site is difficult to determine. In some instances, scour associated with turbulence of flow at the pier may cause depths of local scour greater than general scour. However, the channel-bed profiles in Figure 6 suggest that the location of greatest scour may be midchannel between piers. This observation also is supported by field surveys at other bridge sites. The condition of greater scour depths at midchannel is attributed to higher flow velocities in midchannel, away from the piers (Figure 5). A summary of maximum observed scour depths below the unobstructed channel bed is given in Table 6. The amount of scour may vary during the period of flooding, independent of discharge. Therefore, discharges given in the table may not be related to the depth of scour observed.

Longitudinal Extent of General Scour

The effect of piers on the channel bed may extend several hundred feet upstream and downstream from the bridge. The extent of general scour is related to the magnitude of channel constriction by the bridge, bed-material, size, discharge, and size and shape of the piers. Surveys of channel-bed profiles for the Sacramento River at SR-32 near Hamilton City, California (Figure 7), show that the influence of the piers and the draw rest extends more than 400 ft upstream and downstream from the bridge. The

TABLE 6 Depth of Scour Below Normal Channel-Bed Elevation at Selected Bridge Sites

Location	Date of Survey	Discharge (ft^3/s)	Mean Depth of Flow (ft)	Scour (ft)
California				
Sacramento River at SR-32 at Hamilton City[a]	1-17-74	154,800	27.7	10.7
Sacramento River at SR-162 at Butte City	1-17-70	79,100	26.5	12.1[b]
Sacramento River at County Road at Colusa	1-11-78	40,200	28.2	4.6[b]
Eel River at US-101 near Ferndale[a]	2-?-63[c]	275,000	—	21[c]
Oklahoma				
Canadian River at US-75 near Calvin[a]	10-?-70	94,100	—	13[c]
Canadian River at US-270 near Calvin[a]	10-?-70	130,000	—	20[c]
Mean				13.6

[a] Data adapted from Brice, Blodgett, et al (3).
[b] Unpublished data.
[c] Depths of scour estimated from divers' reports and observation of pier footings at the time of excavation for repair work.

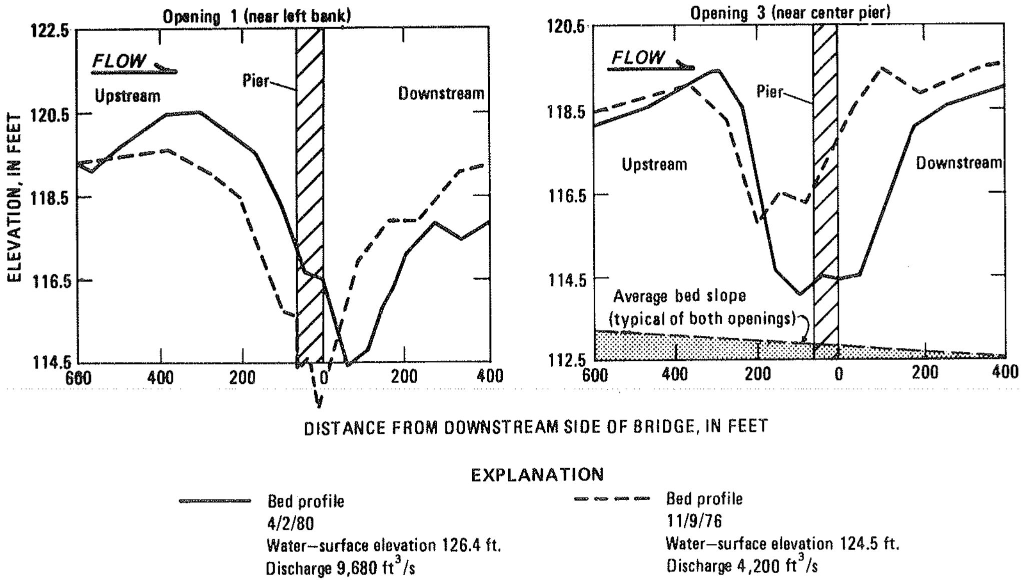


FIGURE 7 Channel-bed profiles through two openings of SR-32 bridge, Sacramento River at Hamilton City, California. (A draw rest at the center pier extends upstream 200 ft.)

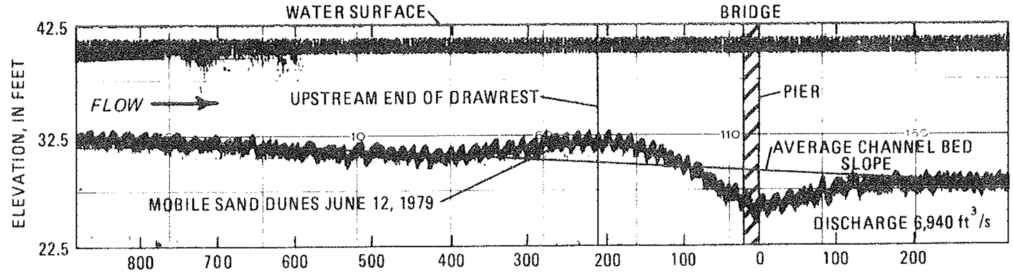


FIGURE 8 Channel-bed profile through right opening of old county bridge showing mobile sand dunes, Sacramento River at Colusa, California.

amount of channel constriction associated with the channel-bed scour, expressed as the ratio of pier area to gross area (A_p/A_g) of the unobstructed channel, is about 8 percent (Table 3). The deepest point of scour may be near either the upstream or downstream side of the pier. The apparent negative slope in the channel-bed profile is caused by levees on both banks upstream from the bridge preventing deposition of material and overbank flow downstream that allows deposition.

The upstream-downstream extent of bed scour in a sand-bed channel is indicated in Figure 8 for a county bridge across the Sacramento River at Colusa, California. Scour extends about 400 ft upstream and 200 ft downstream from the bridge. Maximum scour depths occurred at the downstream side. The amount of flow constriction caused by the piers for the original bridge, expressed as a ratio of pier area to gross area (A_p/A_g) associated with the depth of scour, is about 8 percent (Table 3). Depending on flow conditions, mobile sand dunes cause ripples on the channel bed (Figure 8) that fluctuate in height. The presence of sand dunes on a channel bed tends to increase the boundary roughness. If no dunes are present, which means the channel bed is smooth, the roughness (Manning's coefficient n) would be about 0.015 to 0.020. If dunes are present, and depending on the depth of flow, the roughness coefficient may increase to 0.040 or higher. Estimates of the channel capacity at a bridge site should assume a rough, less efficient channel boundary if changes in the bed form are anticipated. A bridge designed with an inadequate capacity for handling discharge will increase the potential for scour and lateral erosion, in addition to backwater.

Long-Term Trends to Equilibrium Channel Size

Construction of piers in the waterway may cause scour, lateral erosion, and changes in the channel geometry that occur for many years after the bridge is built (3). The following site history for the Sacramento River at SR-160 at Butte City, California, illustrates the effect of large piers on the channel geometry. A swing-span bridge with four large piers (4, 45, 4, and 4 ft wide) was constructed in 1946. The piers required to support the swing span and adjacent approach spans in the main channel occupied about 9 percent of the original channel (measured in 1946) at a bankfull discharge. The channel bed was composed of gravel and cobbles. The primary factor causing scour and lateral erosion at the site was the presence of the large piers, timber draw rest, and pier fenders that cause a lateral constriction of the waterway and induce turbulence of flow.

Beginning in 1943, streamflow data collected at the site were used to determine channel conditions before construction of the bridge. Following bridge construction, the lowest point in the channel bed

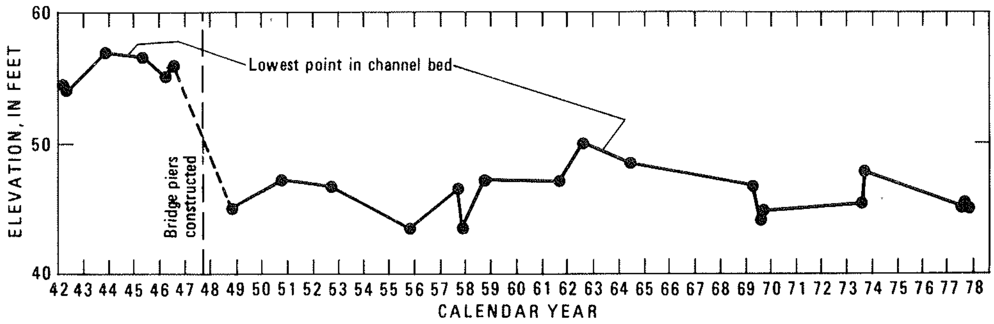


FIGURE 9 Variation in elevation of lowest point in channel bed between 1943 and 1978, Sacramento River at SR-162, Butte City, California.

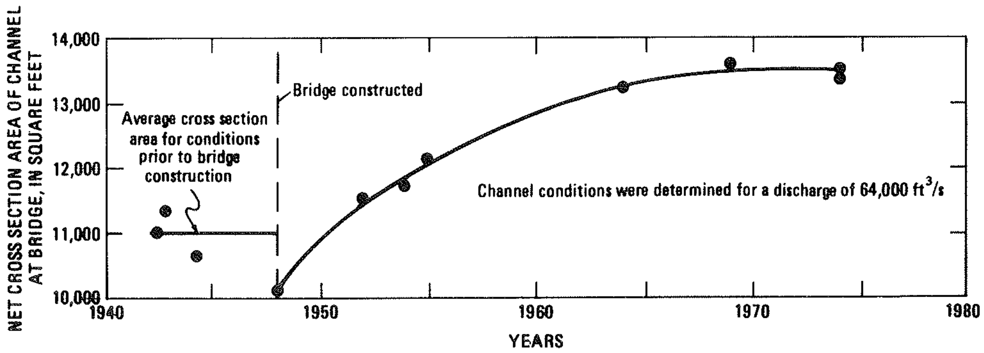


FIGURE 10 Variation in net channel area before and after bridge construction for period 1943 to 1974, Sacramento River at SR-162, Butte City, California, adapted from Brice, Blodgett, et al. (3).

scoured about 10 ft as shown in Figure 9. Floodflows in 1947 and 1948 were smaller than the mean annual flood, so scour at the site was probably continuous between 1946 and 1949. The variation in elevation of the lowest point in the channel bed between 1949 and 1978 indicated no overall trend of increasing scour, but the annual changes in scour and fill are probably dependent on floodflow conditions. For example, the 2 ft of scour observed between 1969 and 1970 is attributed to large floods in January 1969 and January 1970.

Although the low point in the channel bed remained at a relatively stable elevation between 1949 and 1978 and did not show progressive scour, the channel geometry was undergoing a significant long-term change. As shown in Figure 10, the net cross-sectional area at the bridge began to increase shortly after the bridge was built. The channel ultimately enlarged 23 percent to compensate for the flow inefficiency and reduction in area caused by the piers. This increase in flow area apparently reached an equilibrium about 1974, or 28 years after bridge construction.

LATERAL MIGRATION OF LOWEST POINT IN CHANNEL BED

Lateral migration of the lowest point in the channel bed may result in undermining pier footings that were originally placed in shallow water near the margins of the channel. The rate and magnitude of bed shift are substantial for some streams (Table 7).

In the design of foundations and placement of piers, it should be assumed that the lowest point in the channel bed may move laterally to any location within the active channel banks. The potential of lateral migration of the stream is greatest for gravel and sand-bed channels or channels with banks of small-grained material such as sand or silt-loam material.

TABLE 7 Lateral Movement of Lowest Point in Channel Bed Between Bridge Abutments

Location	Year	Distance to Lowest Point in Channel Bed from Left Abutment (ft)	Net Change in Distance Since Initial Date (ft)
Deer Creek near Vina, California	1951	84	0
	1962	127	+43
	1974	297	+213
	1976	308	+224
Snake River near Heise, Indiana	1971	123	0
	1977	247	+124
Canadian River near Sanchez, New Mexico	1928	186	0
	1943	255	+69
	1965	196	+10
	1976	73	-113

Note: Data adapted from Brice, Blodgett, et al. (3).

HYDRAULIC CONSIDERATIONS IN PIER DESIGN

In designing bridges across waterways of the type discussed in this report, several hydraulic factors need to be considered to minimize the impact of the structure on the stream. The effect of bridge piers on streamflow increases if the bridge is constructed so that the abutments constrict the main channel or approach embankments force overbank flow into the opening at the bridge, causing cross-channel flow.

To improve the hydraulic efficiency of bridge piers and reduce the potential for scour, lateral erosion, and bridge damage at a site, the following hydraulic factors, which are based on data for 10 bridge sites, should be considered in bridge design:

1. As a general guideline, bridge maintenance problems associated with general scour and lateral erosion have been observed at sites where bridge piers occupy more than about 5 percent of the original waterway.

2. Using canals as the most efficient open channel, typical streams are 5 percent less efficient, and streams with bridge piers are 17 percent less efficient. To minimize future bridge maintenance related to scour and lateral erosion, an effort should be made to prevent encroachment by the abutments, minimize the number and size of piers or pile bents in the waterway, and select pier shapes that will provide minimal obstruction to the stream.

3. The effect of piers or pile bents on the velocity distribution across a stream is apparently related to the mean velocity. For a low velocity stream (2 ft/s or less), the effect of piers is very low; at high velocities (6 ft/s or more), the effect of piers on the velocity profile is more pronounced.

4. At sites where flows are not normal to the opening, such as at bends or where the channel alignment is unstable, rounded or cylindrical piers have a lessor effect on the pattern of velocity distribution than wall-type piers. At these sites, the use of wall-type piers or multiple-pile bents (which act as wall-type piers) should be discouraged.

5. The average ratio of the maximum velocity to mean for the cross section, for channels unaffected by piers, is 1.3. For channels with piers, the ratio increases to 1.5.

6. For natural streams the average maximum depth to mean depth ratio (d_m/d_a) is 1.57; for streams with bridge piers the ratio is 1.63.

7. The lowest point in the channel bed attributed to scour may occur in midchannel between the piers, where the velocity of flow is the highest, rather than adjacent to a pier.

8. Near bridge piers, scour depths have been documented that are up to 20 ft below the normal lowest point in the channel bed.

9. General scour of the channel bed has been observed for distances more than 400 ft upstream and downstream from a bridge.

10. During the first 2 or 3 years after construction of a bridge that constricts a channel, scour may lower the channel bed several feet.

11. Scour resulting from flow being constricted by bridge piers may increase the channel area to compensate for the decreased flow efficiency.

12. Abutments and piers that create large amounts of constriction may induce changes that require maintenance to prevent lateral erosion and undermining of piers. The period of instability may continue for decades.

13. When selecting a pier footing elevation, the designer should assume that the location of the stream and the lowest point in the channel bed may move laterally to any location within the active channel banks.

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Publication of this paper sponsored by Committee on Hydrology, Hydraulics and Water Quality.