Linda, California, during the flood of February 1986. The North Fork of the Feather River flows from top center, and the Yuba River flows from the middle right. (Courtesy of the State of California Department of Water Resources)

9

Flood Control

9-1 Flooding and Historical Control

People have had to cope with floods throughout time. As long as people were nomadic, floods were only an inconvenience, requiring that camp be moved to higher ground during the wet season. In some parts of the world, this practice is still prevalent. In the Sudd there are large swamps created by the Nile River in Sudan and herds of cattle are moved to higher ground during the months when the river flow is high and vast areas are flooded. As the water recedes in the spring and summer, more grass is exposed, and the herds are moved back into the area to graze. The culture produced by this annual flooding has persisted for centuries.

In more populated areas, floods produce consequences far more serious than inconveniences. The attraction of the *flood plain*—the flat land along rivers or streams that is periodically flooded — is natural. Year after year, rivers have brought both water and sediment downstream. During flood flows, both the rate of flow and the concentration of sediment carried by the flow are higher. As the river rises, overtops its banks, and flows out of the *main channel* and over the flood plain, part of the transported sediment is deposited. The net result is an accumulation of fertile soil, which was early recognized for its agricultural importance. Thus early settlers farmed the flood plains, frequently building homes and buildings there. Because flood plains are fertile, easy to farm, and easy to build on, people have continually endeavored to occupy them permanently. In early times, people struggled with flood problems, often attempting to build local levees to keep flood waters from reaching their homes.

Because of the convenience of the river as a navigation route, communities developed along the river banks, providing a center from which agricultural products could be shipped and at which other raw materials or products could be landed for sale and distribution. The economic investment in flood plains has therefore grown throughout the world, and consequently annual damages produced by floods have almost continually increased.

As population increases, flood-control measures, such as the construction of levees and flood-control reservoirs, becomes a role of government. In the United States, the U.S. Army Corps of Engineers has general responsibility and authority for the development of flood-control projects (16). Even with the provision of some form of flood protection, the damages from flooding increase annually. This phenomenon is due both to the increasing investment in the flood plains and to the tendency of more people to move onto the flood plain after protection from flooding has been provided. Table 9-1 shows annual flood damages and deaths in the United States and indicates that the annual damage, though erratic, is increasing.

Hoyt (13) developed an annual flood index by summing flooding depths that occurred at index points on several key U.S. rivers each year. Thus a large flood index generally indicates a large amount of flooding. Hoyt showed that the amount of flooding as well as actual damages increased between 1903 and 1940 and concluded that of the increase in annual flooding damages during that

Table 9-1 Annual Deaths and Flood Damages in the United States (U.S. National Weather Service)

Year	Lives Lost	Property Damage (thousands of 1985 dollars)	Year	Lives Lost	Property Damage (thousands of 1985 dollars)
1903	178	800,000	1945	91	1,200,000
1904	0	100,000	1946	28	430,000
1905	2	170,000	1947	55	1,400,000
1906	1	6,000	1948	82	1,100,000
1907	7	200,000	1949	48	460,000
1908	11	130,000	1950	93	900,000
1909	5	700,000	1951	51	4,200,000
1910	0	300,000	1952	54	1,200,000
1911	0	130,000	1953	40	520,000
1912	2	1,000,000	1954	55	460,000
1913	527	2,200,000	1955	302	4,000,000
1914	180	260,000	1956	42	280,000
1915	49	200,000	1957	82	1,400,000
1916	118	270,000	1958	47	900,000
1917	80	250,000	1959	25	580,000
1918	0	65,000	1960	32	390,000
1919	2	21,000	1961	52	580,000
1920	42	150,000	1962	19	300,000
1921	143	220,000	1963	39	590,000
1922	215	440,000	1964	100	2,300,000
1923	42	260,000	1965	119	3,000,000
1924	27	290,000	1966	31	420,000
1925	36	70,000	1967	34	1,700,000
1926	16	180,000	1968	31	1,300,000
1927	423	2,800,000	1969	297	3,100,000
1928	15	360,000	1970	135	800,000
1929	89	490,000	1971	74	900,000
1930	14	150,000	1972	554	10,600,000
1931	0	29,000	1973	148	4,100,000
1932	11	60,000	1974	121	1,400,000
1933	33	400,000	1975	107	3,000,000
1934	88	100,000	1976	193	5,800,000
1935	236	1,300,000	1977	210	2,300,000
1936	142	2,500,000	1978	143	1,200,000
1937	142	3,500,000	1979	121	5,900,000
1938	180	700,000	1980	82	2,000,000
1939	83	140,000	1981	84	1,400,000
1940	60	380,000	1982	155	3,700,000
1941	47	330,000	1983	204	3,800,000
1942	68	600,000	1984	126	4,800,000
1943	107	1,400,000	1985	304	3,000,000
1944	33	700,000			

period, 45% resulted from an increase in property values, 25% from an increase in the annual magnitude of flooding, and 30% from increased use of the flood plains. Thus although the investment in flood control has increased rapidly during the past decades, annual damages have also increased.

Annual flood damages for the United States were tabulated by the U.S. Water Resources Council until 1980 (20), but since then, they have been tabulated by the U.S. National Weather Service. The U.S. Army Corps of Engineers tabulates annual flood damages prevented by Corps projects.

Flood-control work began in the United States in 1718, soon after the site for the city of New Orleans was selected. Settlers were ordered to build levees to protect themselves and their neighbors from future floods. Until 1917, individuals and local government provided all funds for flood control in the United States, although the federal government did provide some funds for construction of levees along the Mississippi River in the interest of navigation. In 1917, an act of Congress authorized the construction of flood-control levees with federal funds.

Record floods occur as a result of either rainfall or rainfall and simultaneous snowmelt. In general, rainfall produces the highest peak flows, and the combination of rainfall and snowmelt produces the largest flood volumes. The largest flood in the United States, for which official records exist, occurred on the Mississippi River on May 1, 1927, and peaked at 2,270,400 ft^3/s (64,290 m^3/s) at Vicksburg, Mississippi (22). At that point, the Mississippi River has a drainage area of 1,142,100 mi^2 (2,958,000 km^2). Thus that record flood, which had an estimated recurrence interval of more than 100 yr, represented a peak flow per unit area of 1.99 $ft^3/s/mi^2$ (0.022 $m^3/s/km^2$). By contrast, Kawaikoi Creek at Waimea, Hawaii, with a drainage area of only 4.2 mi^2 (10.9 km^2) experienced a flood of 11,264 ft^3/s (319 m^3/s) on January 13, 1967, a peak flow per unit area of 2682 $ft^3/s/mi^2$ (29.3 $m^3/s/km^2$). The Kawaikoi Creek flood was also judged to be approximately a 100-yr event. Thus difference in flood characteristics is pronounced between large and small drainage areas. The primary difference is that a single rainstorm cannot uniformly cover a large drainage basin, whereas a small drainage basin may be totally subject to the peak intensity of the storm.

By contrast, the flood volumes produced by frontal storms that cover a large drainage area are likewise very large, and the large flow rates sometimes continue for many days. Floods produced by intense storms that cover small drainage areas peak quickly and recede quickly giving rise to large peak flow rates but relatively small runoff volumes.

The Mississippi River and Kawaikoi Creek floods resulted from severe rain storms. Snowmelt floods can also be extreme events. For example, the Judith River in Montana, a drainage area of 33 mi^2 (85.5 km^2), experiences regular snowmelt floods, as do almost all streams in northern climates. On June 6, 1927, a flood of 1116 ft^3/s (31.6 m^3/s) occurred, producing a peak discharge per unit area of 33.8 $ft^3/s/mi^2$ (0.37 $m^3/s/km^2$).

Table 9-2, pages 498–499, lists many record floods along with some of the drainage basin characteristics of these floods. Figure 9-1 shows a plot of peak

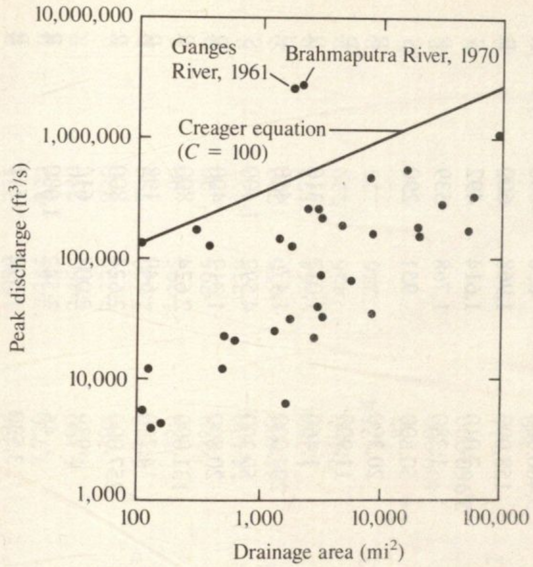


Figure 9-1 Peak discharge as a function of drainage area

flow rate for each of these floods as a function of drainage area and demonstrates that peak flow rates do not correlate with drainage area. A detailed study of several large historical floods from many rivers of the world was done by Creager (6). In his study, Creager developed envelope curves for the maximum floods of record in terms of the equation

$$Q_p = 46CA^{(0.894A^{-0.048})} \tag{9-1}$$

where Q_p = peak discharge in ft^3/s
A = drainage area in mi^2
C = an empirical coefficient dependent on drainage area characteristics with a maximum value of approximately 100 for many areas

Figure 9-1 shows that Creager's equation would predict maximum flow rates above most of those in Table 9-2 but that two floods in Bangladesh substantially exceeded the envelope value. Although the floods used by Creager and those plotted in Fig. 9-1 are recorded maximum floods, most authorities now believe the record maximum flood is a poor indicator of the probable maximum flood.

Figure 9-2, page 500, compares probable maximum floods estimated for the Ohio-Tennessee region with peak flow rates calculated using Creager's equation (Eq. 9-1) and provides further evidence that although some recorded floods seem very large, they should always be considered carefully before they are assumed to be representative of the largest possible flood.

CLEVELAND

Table 9-2 Floods of Record from Various Drainage Basins (22)

Country	River	Date	Q Peak (ft^3/s)	Q Peak (m^3/s)	Drainage Area (mi^2)	Drainage Area (km^2)	Mean Basin Elevation (ft)	Mean Basin Elevation (m)	Type
Bangladesh	Brahmaputra	07/28/70	2,705,100	76,600	1,903	4,930	109	33.3	R*
Bangladesh	Ganges	09/01/61	2,585,030	73,200	1,842	4,770	123	37.5	R
Bulgaria	Iskar	06/29/57	33,200	940	3,232	8,370	2,315	706	R
Canada	Slocan	06/07/61	25,300	719	1,266	3,280	5,051	1,540	S†
Canada	Chilliwack	06/20/72	4,100	116	131	339	4,100	1,250	S
Canada	Chilco	06/20/50	16,920	479	3,232	8,370	5,346	1,630	S
Canada	Rocky	12/27/53	5,690	161	110	285	268	81.6	R
Canada	Churchill	06/27/57	240,850	6,820	28,840	74,700	1,702	519	S
Congo	Congo	11/17/61	2,683,920	76,000	1,343,600	3,480,000	656	200	R
Congo	Sangha	11/06/60	165,270	4,680	61,000	158,000	1,968	600	R
Czechoslovakia	Morava	12/03/41	22,110	626	2,706	7,010	1,614	492	S
Czechoslovakia	Becva	07/27/39	22,310	660	494	1,280	1,768	539	R
Finland	Kemijoki	05/01/73	170,220	4,820	19,610	50,800	951	290	S
France	Rhone	05/31/56	158,920	4,500	7,840	20,300	—	—	R
France	Durance	10/25/82	180,100	5,100	4,590	11,900	—	—	R
France	Seine	02/24/58	2,170	61.6	564	1,460	1,017	310	R
Gabon	Ogooue	11/18/61	480,280	13,600	7,880	204,000	1,476	450	R
Germany	Rhein	12/31/82	160,680	4,550	19,420	50,300	4,592	1,400	S
Ghana	Pra	07/17/57	35,900	1,020	8,030	20,800	1,312	400	R
Hungary	Danube	06/16/65	325,600	9,220	50,580	131,000	2,624	800	R
Hungary	Zagyva	06/12/65	6,390	181	1,625	4,210	649	198	R
India	Krishna	10/07/03	1,059,440	30,000	99,230	257,000	2,624	800	R
Italy	Magra	10/15/60	122,900	3,480	362	939	2,007	612	
Italy	Alli	12/19/30	639	18.1	18	46	3,542	1,080	R
Japan	Mogami	08/29/67	137,730	3,900	1,363	3,530	1,692	516	R

Japan	Toyo	08/05/69	167,900	4,770	279	724	1,099	335	R
Maylasia	Perak	01/06/67	222,480	6,300	3,000	7,770	2,745	837	R
Morocco	Ouergha	12/18/63	248,260	7,030	2,390	6,190	2,460	750	R
Norway	Engera	05/07/34	4,130	117	152	394	2,722	830	S
Poland	Dunajec	06/30/58	123,600	3,300	1,676	4,340	2,620	799	R
Sweden	Baljanea	04/20/70	1,300	37	92	239	295	90	S
Sweden	Baljanea	11/23/63	1,510	43	92	239	295	90	R
USSR	Dnieper	05/01/08	64,270	1,820	5,444	14,100	738	225	S
USSR	Desna	04/20/31	281,600	8,000	31,429	81,400	656	200	S
United Kingdom	Don	05/24/32	12,250	347	486	1,260	564	172	R
United Kingdom	Trent	03/19/47	39,200	1,110	2,892	7,490	456	139	R
United Kingdom	Cannons Brook	07/01/58	500	14.2	8.3	21.4	246	75	R
United States	Ute Creek	05/15/41	629	17.8	32	83	—	—	S
United States	Arkansas	05/27/43	536,780	15,200	157,900	409,000	—	—	R
United States	Mississippi	05/01/27	2,277,800	64,500	1,142,900	2,960,000	—	—	R
United States	Kawaikoi	01/13/67	11,300	320	4.2	11	—	—	R
United States	Quaboag	08/19/55	12,740	362	150	390	—	—	
United States	Eel	12/23/64	752,200	21,300	3,130	8,100	—	—	R

* R = flood created by rainfall.
† S = flood created by rain and snowmelt.

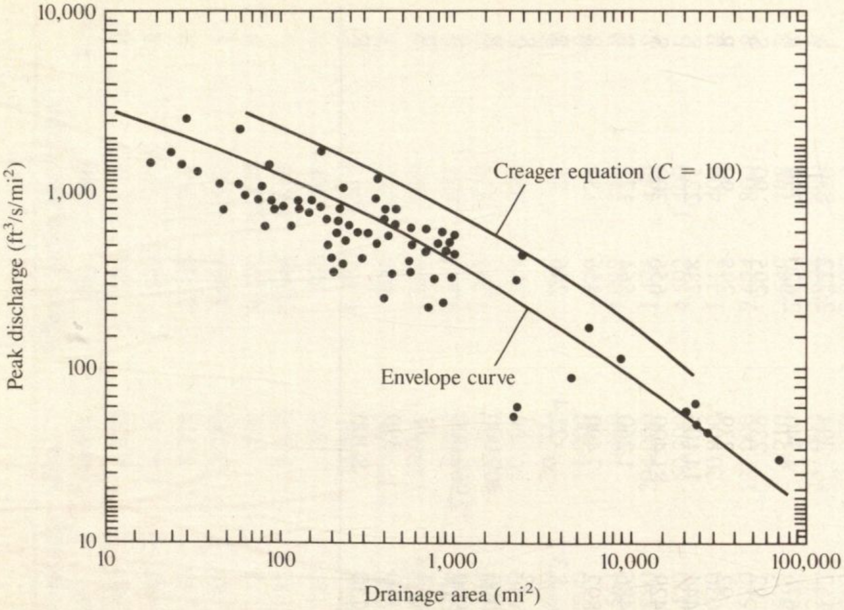


Figure 9-2 Peak discharge for computed probable maximum floods for rivers and streams in the region of Ohio, Tennessee, southern Pennsylvania, West Virginia, Kentucky, Alabama and Virginia (19)

9-2 Control of Flooding

As we mentioned, flood damages are created when river flows are large enough to cause flooding of those areas that are less often covered by water than the *main channel*. Many municipal and rural developments are located within the *flood plain*. As Fig. 9-3 shows, a river channel can be divided into zones. That part of the channel referred to as the *main channel* has been formed by frequently occurring flow rates. The bankfull capacity of the river is approximately the mean annual flood — that flood whose peak flow is exceeded approximately one out of two years. The Gumbel distribution (extreme value type

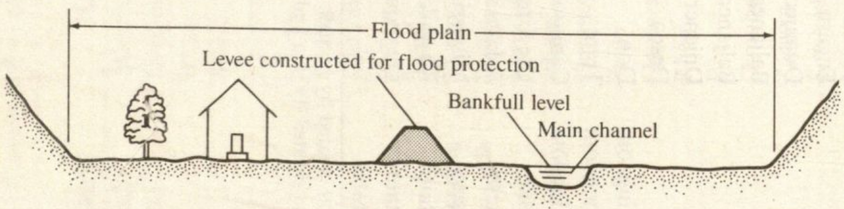


Figure 9-3 Illustration of definitions of flood plain, main channel and levee

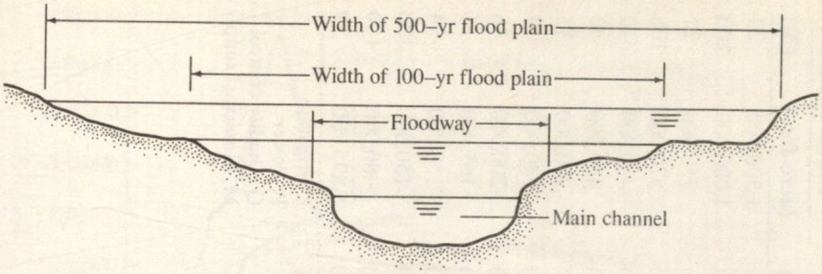


Figure 9-4 River channel and flood plain limits

I) a statistical distribution frequently used for maximum floods gives a recurrence interval of 2.33 yr for the mean annual flood (10). When a flood with a peak flow larger than the mean annual flood occurs, part of the overbank area is flooded. Thus the part of the river valley that is flooded (outside the main channel) is the flood plain, and its extent depends on the magnitude of flood being considered. Thus a 500-yr flood involves greater depths and a larger peak flow rate than a 100-yr flood and covers a greater width of the valley. Figure 9-4 shows a 500-yr and a 100-yr flood plain. The *floodway* is defined by the Federal Agency Management Agency as that part of the stream channel that could contain the 100-yr peak flow with not more than a 1.0-ft increase in depth above that which would occur if the entire cross section carried the 100-yr peak flow (7).

The general goal of flood-control engineering is to prevent or limit flood damages by controlling or managing the flood. In general, a flood can be controlled in only two ways: The peak flow rate of the flood must be reduced, or the capacity of the flood channel must be made large enough to prevent or limit overbank flooding.

The peak flow of a flood can be reduced by:

Storage of at least part of the flood water in an upstream reservoir or reservoirs

Land management upstream to increase infiltration, interception, and detention losses and, thus, both delay and reduce the rate and volume of runoff

Increasing channel storage through diversion to a bypass channel

Diverting flood flows into another river basin

The Sacramento River in California provides an excellent example of the effect of flood-control reservoirs in reducing flooding in the lower portions of the river. Table 9-3 shows the statistics on flood-control storage for several reservoirs in California, of which four are on the Sacramento River or its tributaries. These four reservoirs have succeeded in significantly reducing the peak flood flows occurring along the inhabited stretches of the river. During the January 1974 flood, the operation of Shasta Reservoir alone reduced the peak flood at Sacramento (160 mi downstream) from 330,000 to 128,000 ft^3/s. Figure 9-5

Table 9-3 Some Dams in the State of California with Flood-Control Capacity (4)*

(1) Reservoir	(2) Stream	(3) Completion Date	(4) Operating Agency	(5) Purposes	Storage Capacity: Total (acre-feet)	Storage Capacity: Flood-control (acre-feet)	Storage Capacity: Flood-control (%)
Shasta	Sacramento R.	1943	USBR	F,N,I,P,Q	4,500,000	1,300,000	29
Pine Flat	Kings R.	1954	C of E	F,I,P	1,000,000	1,000,000	100
Oroville	Feather R.	1968	DWR	F,I,M,P	3,500,000	750,000	21
Isabella	Kern R.	1953	C of E	F,I	570,000	570,000	100
Folsom	American R.	1956	USBR	F,I,P	1,000,000	400,000	40
New Exchequer	Merced R.	1966	MID	F,I,P	1,000,000	400,000	40
Friant	San Joaquin R.	1941	USBR	F,I	520,000	390,000	75
New Don Pedro	Tuolumne R.	1971	M & TID	F,I,M,P	2,030,000	340,000	17
Camanche	Mokelumne R.	1963	EBMUD	F,M,P	432,000	200,000	46
New Hogan	Calaveras R.	1963	C of E	F,I	325,000	165,000	51
Black Butte	Stony Cr.	1963	C of E	F,I	160,000	150,000	94
Terminus	Kaweah R.	1962	C of E	F,I	150,000	150,000	100
Success	Tule R.	1961	C of E	F,I	80,000	80,000	100
Total					15,267,000	5,895,000	39

* Symbols: USBR, Bureau of Reclamation; C of E, Corps of Engineers; DWR, California Department of Water Resources; MID, Merced Irrigation District; M & TID, Modesto and Turlock Irrigation Districts; EB MUD, East Bay Municipal Utility District; F, Flood control; N, Navigation; I, Irrigation; P, Power; Q, Quality control; M, Municipal supply

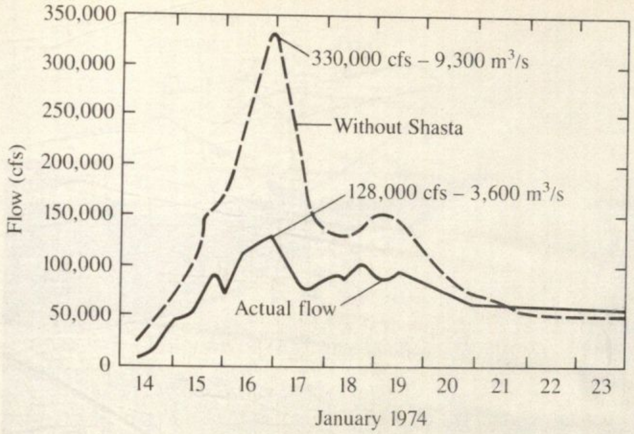


Figure 9-5 Flood hydrograph for the Sacramento River at Sacramento for the 1974 flood showing what the hydrograph would have been without Shasta Dam (4)

shows the actual flood hydrograph at Sacramento and the one that would have occurred without Shasta Dam. Figure 9-6 shows the Sacramento Weir, which diverts flood waters from the Sacramento River to the Yolo bypass — one of two flood bypass channels that can be used to bypass a portion of large floods to the west of Sacramento, California, and thus decrease the flooding depth at Sacramento.

Upstream improvement of land-use management within the United States has been the province of the Soil Conservation Service (SCS) of the U.S. Department of Agriculture since 1935, when the U.S. Congress enacted a policy of preventing soil erosion for the preservation of natural resources, control of floods, and related purposes (16). The SCS has continuously pursued the design and construction of small reservoirs and the implementation of sound land-management practices for preventing floods and conserving soil resources. By 1985, the SCS had constructed more than 23,000 reservoirs throughout the United States, providing some 7,031,700 acre-ft of storage.

Several famous flood-control projects exist within the United States. The Miami Conservancy District was created in Ohio to control flooding by the Miami River after a disastrous flood in March 1913 caused 360 deaths and more than $100 million (1913 dollars) in property damage (2). Arthur E. Morgan was chief engineer of the Conservancy District, and under his direction, many significant advances were made both in the engineering design of flood-control measures and in the hydraulic and hydrologic methodology required to analyze rainfall and floods for design (12, 15, 21).

To develop protection against flooding, Morgan proposed constructing a system of five reservoirs together with substantial channel improvements. The

Figure 9-6 Flood waters are passing through 48 gates that have been opened to relieve flooding in Sacramento by allowing water to flow from the Sacramento River in the foreground to the Yolo Bypass. (Courtesy of the California Department of Water Resources) (8)

reservoirs were formed by dams and acted to detain floods. During the 1913 flood, a total of 1,415,000 acre-ft had fallen as precipitation on the river basin. The proposed detention reservoirs provided a storage capacity of 840,000 acre-ft, and the peak outflow from the basins during a storm as severe as that of 1913 was designed to be not more than the bankfull capacity of the river downstream. Figure 9-7 illustrates the typical form of an uncontrolled outlet from a detention reservoir as used by Morgan.

The Tennessee Valley Authority (TVA) was created by the U.S. Congress in 1933 to construct dams and reservoirs to promote navigation, control floods, and generate electricity. The drainage basin of the Tennessee Valley covers 40,900 sq mi. Before TVA, the flow at the mouth of the Tennessee River had ranged from a low of 4500 ft^3/s (127.8 m^3/s) to a high of 471,000 ft^3/s (13,400 m^3/s) during the flood of 1897. Thirty-six major dams were constructed by the TVA on the Tennessee River, and 14 were constructed on adjoining rivers. Control of the system provides 12,000,000 acre-ft of reserved storage for flood regulation on January 1 and 9,000,000 acre-ft on April 1 of each year.

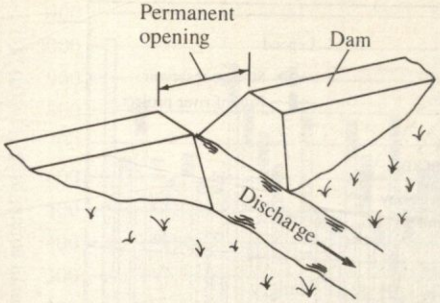


Figure 9-7 Uncontrolled outlet from a detention basin or reservoir

This combination of available flood storage volume has provided substantial flood regulation and has increased the river low flow to 37,000 ft^3/s (1050 m^3/s). From its inception to 1983, the TVA system has prevented flood damages estimated at more than \$2 billion (3).

On the Columbia River in the Pacific Northwest, a system of large dams has been developed by the United States and Canada (9). Figure 9-8 shows the location of the major single- and multiple-purpose projects that have been developed since construction of Bonneville and Grand Coulee dams was begun in 1933. A total of 210 projects (not all are shown in Fig. 9-8) provides an active storage of 62,000,000 acre-ft. The system provides benefits for navigation, hydroelectric generation, flood control, irrigation, and recreation. Figure 9-9 shows the reduction in peak flow of the Columbia River at the Dalles that has been achieved since 1948. Annual benefits due to flood reduction have been estimated at \$507 million (9).

9-3 Flood Plain Management

The construction of reservoirs to store floodwaters or the improvement of river channels to lower flood levels has tended to increase development of the protected flood plains. As a result, when the design flood for the control measures is exceeded, damages are frequently more than would have occurred for the same flood before construction of the control measure. Laws have therefore been enacted to regulate the use of flood plains. This approach was given great impetus in the United States by the Flood Disaster Protection Act of 1973, the function of which was to keep people away from floods rather than floods away from people.

In the practice of flood-plain management, the extent of the flood plain for a particular design flood (usually the 100-yr or 500-yr floods in the United States) is defined. So defined, the flood plain is that area that would be inundated by the design flood if it should occur. The flood plain is frequently divided

Figure 9-8 Reservoirs in the Columbia River Basin (9)

into the *floodway*, where velocities are large enough to cause damage during flooding, and the peripheral area, which is primarily an area of overbank flood storage that is inundated during flooding but experience only small velocities. The *floodway* within communities that have adopted flood-plain regulations is usually subject to strong building restrictions; lesser restrictions apply to the peripheral area. In general, adopting a flood-plain management program is required before federally subsidized flood insurance can be made available to property owners.

9-4 Delineation of the Flood Plain

Hydraulic analysis of flood-control structures and channels in the flood plain involves the use of principles and tools we presented in Chapter 4. The

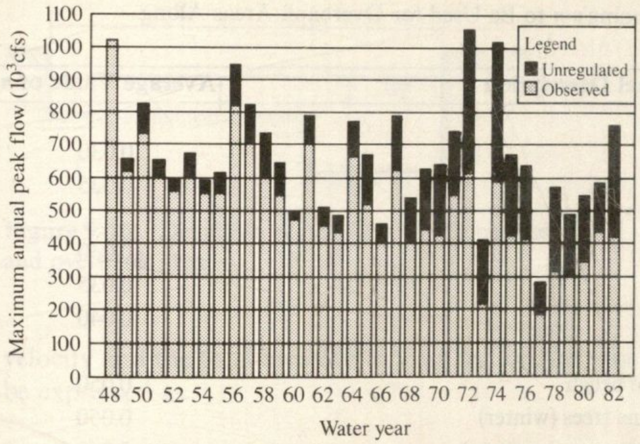


Figure 9-9 Comparison of historial observed flows on the Columbia at the Dalles with what the flows would have been without the reservoirs in the Columbia Basin (9)

flood plain defined in the United States is usually that for the 100-yr flood. The calculation of the 100-yr peak flow or the 100-yr flood hydrograph is performed using methods we presented in Chapter 2. If the length of the channel over which the flood plain is to be defined is relatively short, the methods of gradually varied flow (Chapter 4) can be used. These methods assume the flow is steady at the peak flow rate associated with the design flood. Cross sections must be surveyed along the river at small enough intervals to adequately define the valley geometry. Then values of the resistance coefficient must be estimated for both the main channel and the overbank areas. Table 4-1 and Figs. 4-2, 4-3, and 4-4 (pages 169–170) provide some typical values of Manning's n to be used in the computational process. Table 9-4 on the next page provides values of Manning's n that are more typical for overbank areas during flood flows. The overbank areas, because of typical growth of trees, grass, weeds, and other obstructions tend to exhibit greater resistance to flow than the main channel, with the result that velocities in the main channel (floodway) are always larger than those in the overbank (peripheral) area.

To delineate the extent of overbank flooding, it is necessary to compute the water surface profile in the channel, which involves the use of the energy equation (Eq. 4-28, page 199, or, the more compact form, Eq. 4-33, page 203). For flood flow in channels with overbank flow, it is convenient to use the kinetic-energy correction factor, α. Its use here differs from that in Chapter 5 in that we consider only the difference in mean velocities in the various parts of the channel, we do not consider the vertical variation of velocity. The coefficient is usually significantly larger than unity because the average velocity

Table 9-4 Values of Manning's n to Be Used for Overbank Areas Along Streams or Rivers

Channel Description		Average Value of n
Grassland		
Short grass		0.030
Tall grass		0.035
Cultivated ground		
Bare ground		0.030
Mature row crops		0.035
Mature field crops		0.040
Brushy areas		
Dense weeds and sparse brush		0.050
Brush-covered with some trees	(winter)	0.050
Brush-covered with some trees	(summer)	0.060
Dense brush	(winter)	0.070
Dense brush	(summer)	0.100
Forested		
Densely covered with willows	(summer)	0.150
Cleared land with stumps; no new growth		0.040
Cleared land with stumps; dense new growth		0.060
Dense stands of large trees; flood stage below branches		0.100
Dense stands of large trees; flood stage reaching branches		0.120

in the overbank area is generally much smaller than the average velocity in the main channel. Thus Eq. (4-28) with α included is written between two sections a distance Δx apart as

$$y_a + \frac{\alpha_a V_a^2}{2g} + S_0\,\Delta x = y_b + \frac{\alpha_b V_b^2}{2g} + S_f\,\Delta x \tag{9-2}$$

or, in the form of Eq. (4-33),

$$E_a - E_b = (S_f - S_0)\,\Delta x \tag{9-3}$$

In Eq. (9-2), y_a and y_b are depths at sections a and b, respectively, and can be in either the main channel or the overbank channel. To evaluate α, the definition given by Eq. (5-2), page 242, is applied in finite increment form:

$$\alpha = \frac{1}{V^3 A} \sum_{i=1}^{N} V_i^3 A_i \tag{9-4}$$

where the subscript i indicates the subarea of which V_i and A_i are the average velocity and cross-sectional area, respectively, and N indicates the number of subareas that the section is divided into. V and A are, respectively, the average

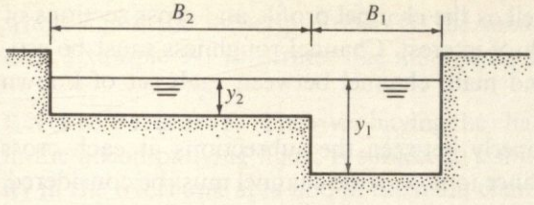


Figure 9-10 Definition sketch for a simple channel and overbank area

velocity and the total area of the cross section. Because $V = Q/A$, Eq. (9-4) can be expressed as

$$\alpha = \frac{A^2}{Q^3} \sum_{i=1}^{N} \left(\frac{Q_i^3}{A_i^2} \right) \tag{9-5}$$

where Q is the total discharge, and Q_i is the discharge in subsection i. For the case of a compound channel such as shown in Fig. 9-10, the free surface must be level across each section if the flow is to be one dimensional as assumed. Thus the friction slope between any two sections of a reach of channel will be the same for the main channel and the overbank channel.

The total discharge in the channel is the sum of the discharges in each part, or

$$Q = \sum_{i=1}^{N} V_i A_i \tag{9-6}$$

In calculating coordinates of the free surface profile in an open channel, Eq. (9-2) must be solved. In the solution, the energy slope S_f can be computed using either the Darcy-Weisbach relationship

$$S_f = \left(\frac{fQ^2}{8gRA^2} \right) \tag{9-7}$$

or the Manning relationship

$$S_f = \left(\frac{n^2Q^2}{2.22A^2R^{4/3}} \right) \tag{9-8}$$

The Manning equation is used more frequently because the bottom roughness, shape, height, and texture vary greatly along a natural channel, and the greater accuracy afforded by using the resistance coefficient f is usually not warranted. The solution of Eq. (9-2) can be performed using standard numerical methods for the solution of ordinary differential equations such as the Runge-Kutta method (14). The flow rate and depth in each subsection must be known at one cross section of the channel as well as the channel profile and cross sections of the stream throughout the length of interest. Channel roughness must be estimated for both the overbank and main channel between each set of known cross sections.

To proportion the flow properly between the subsections at each cross section, the relationship for resistance to flow in the channel must be considered. Equation (9-6) can be rewritten as

$$Q = Q_1 + Q_2 \tag{9-9}$$

where the subscripts 1 and 2 refer to subareas of the cross section. Moreover, the slope of the energy line in the main channel must equal that for the overbank area:

$$S_f = S_{f_1} = S_{f_2} \tag{9-10}$$

The conveyance for an open channel, K, is defined as $Q/\sqrt{S_f}$; thus

$$K = \frac{Q}{\sqrt{S_f}} = \frac{1.5}{n} AR^{2/3} \tag{9-11}$$

which can be conveniently used in the following development.

Using Eqs. (9-9) and (9-11), we can write

$$Q = K_1\sqrt{S_{f_1}} + K_2\sqrt{S_{f_2}} \tag{9-12}$$

or

$$Q = (K_1 + K_2)\sqrt{S_f} \tag{9-13}$$

Equation (9-13) can be rearranged and generalized as

$$S_f = \frac{Q^2}{\left(\sum_{i=1}^{N} K_i\right)^2} \tag{9-14}$$

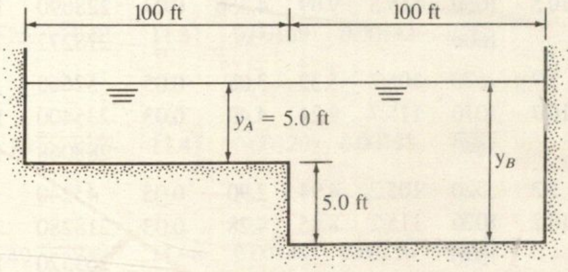
where again the subscript i refers to a subarea of the cross section, and N is the total number of subareas.

It is helpful to evaluate the energy correction coefficient in terms of the hydraulic conveyance and the areas of the subareas of the compound section. Using Eqs. (9-10), and (9-12), Eq. (9-4) can be rewritten as

$$\alpha = \frac{\left(\sum_{i=1}^{N} A_i\right)^2}{\left(\sum_{i=1}^{N} K_i\right)^3} \sum_{i=1}^{N} \left(\frac{K_i^{\,3}}{A_i^{\,2}}\right) \tag{9-15}$$

where again the subscript i refers to the subarea i of the compound cross section. Example 9-1 illustrates one method by which Eq. (9-2) can be solved.

EXAMPLE 9-1 The river having the channel whose cross section is shown in the accompanying figure is subject to a flood of 10,000 ft^3/s. Find the velocity in the overbank area and in the main channel at stations 0 + 00 and 6 + 00 if the depth shown occurs at station 0 + 00. Manning's n is 0.05 in the overbank areas and 0.03 in the main channel. The channel slope is 0.00167.



SOLUTION To solve the problem, we use the total-head equation and the standard-step approach described in Chapter 4. The total-head equation is written between sections 0 + 00 and 6 + 00 as

$$y_0 + \alpha_0 \frac{V_0^2}{2g} + S_0\,\Delta x = y_6 + \alpha_6 \frac{V_6^2}{2g} + S_f\,\Delta x \tag{9-16}$$

or

$$E_0 + S_0\,\Delta x = E_6 + S_f\,\Delta x \tag{9-17}$$

Since the bottom slope is 0.00167, the difference in elevation between station 0 + 00 and 6 + 00 is

$$S_0\,\Delta x = S_0 \cdot (x_6 - x_0) = 600(0.00167) = 1.0 \text{ ft} \tag{9-18}$$

The terms necessary to solve Eq. (9-17) are first computed for section 0 + 00 in the top lines of the accompanying table, pages 512–513. As given and noted on columns 2 and 3, the depth at section 0 + 00 is 5 ft and 10 ft in the subareas of the channel designated A and B, respectively. Thus under column 4, the area of the cross section is computed as 500 ft^2 and 1000 ft^2 for subareas A and B, respectively. The total area is then 1500 ft^2. The wetted perimeters of the subareas are $100 + 5 = 105$ ft for A and $100 + 5 + 10 = 115$ for B. Both are recorded in column 5. Because we will use Eq. (9-11) to compute conveyance, the hydraulic radius of each subarea has been computed as A/P, and the values 4.76 ft and 8.70 ft were obtained for A and B, respectively, and are listed in column 6. In column 7, the hydraulic radius of column 6 has been raised to the $\frac{2}{3}$ power. Column 8 shows the given values of Manning's n for each subarea.

(1) Section	(2) Sub Area	(3) y (ft)	(4) A (ft^2)	(5) P (ft)	(6) R (ft)	(7) $R^{2/3}$	(8) n	(9) K	(10) K^3/A^2
0 + 00	A	5	500	105	4.76	2.83	0.05	42450	3.060×10^8
	B	10	1000	115	8.70	4.23	0.03	211500	94.609×10^8
			1500					253950	97.669×10^8
6 + 00	A	5.5	550	105.5	5.21	3.005	0.05	49582	4.029×10^8
	B	10.5	1050	115.5	9.09	4.356	0.03	228690	108.483×10^8
			1600					278272	112.512×10^8
6 + 00	A	5.7	570	105.7	5.39	3.08	0.05	52668	4.497×10^8
	B	10.7	1070	115.7	9.24	4.40	0.03	235400	113.934×10^8
			1640					288068	118.431×10^8
6 + 00	A	5.2	520	105.2	4.94	2.90	0.05	45240	3.424×10^8
	B	10.2	1020	115.2	8.85	4.28	0.03	218280	99.964×10^8
			1540					263520	103.388×10^8
6 + 00	A	5.1	510	105.1	4.85	2.87	0.05	43911	3.255×10^8
	B	10.1	1010	115.1	8.77	4.25	0.03	214625	96.917×10^8
			1520					258536	100.172×10^8

$Q = 10{,}000$ cfs and $S_0 = 0.00167$

The hydraulic conveyance K has been calculated for each subarea using Eq. (9-11) and has been recorded in column 9. Column 10 shows the ratios of the cube of the conveyance and the square of the area used in Eq. (9-15) to calculate the kinetic energy coefficient α. The sum of column 10 is multiplied by $(1500)^2$ from column 4 and divided by $(253{,}950)^3$ from column 9, as required by Eq. (9-15). The mean velocity for the total cross section V has been computed as Q/A by dividing the total discharge (10,000 cfs) by 1500 (the total area for the cross section) as totaled in column 4. The velocity head has been computed as $\alpha V^2/2g$ in column 13. The value of E_0, the specific energy at 0 + 00, has been computed as 10.93 in column 14 by adding y_B from column 2 to the velocity head from column 13. The slope of the energy gradient S_f listed in column 16 has been computed by dividing the total discharge (10,000 cfs) by the 253,950 from column 9 and squaring the result, as required by Eq. (9-14).

First iteration:

The next step is to calculate the depth of flow at section 6 + 00. To do this, it is necessary to iterate on a depth at section 6 + 00. An initial depth is assumed, as is shown in column 3. The remainder of this iteration determines whether the assumption is correct. As shown, a depth $y_6 = 10.5$ was assumed, which gives $y_A = 5.5$, as noted in column 3. Columns 4 through 15 are then calculated as was done for section 0 + 00. In column 17, the average energy

(11) α	(12) V (ft/s)	(13) $\alpha \dfrac{V^2}{2g}$ (ft)	(14) E (ft)	(15) S_f	(16) Avg S_f	(17) $S_f \Delta x$	(18) $(E_0 + S_0 \Delta x) - (E_6 + S_f \Delta x)$
1.342	6.67	0.93	10.93	0.00155			
1.337	6.25	0.81	11.31	0.00129	0.00142	0.85	−0.23
1.332	6.10	0.77	11.47	0.00120	0.00138	0.83	−0.37
1.340	6.49	0.88	11.08	0.00144	0.00150	0.90	−0.05
1.339	6.58	0.90	11.00	0.00150	0.00152	0.91	+0.01

gradient S_f has then been calculated as $\frac{1}{2}$ the sum of 0.0016 (the value for 0 + 00 in column 16) and 0.00129 (the value calculated for section 6 + 00 and shown in column 16). The total head loss between 0 + 00 and 6 + 00 has been estimated as 600 $S_f = 0.87$ and is shown in column 17. The value of the left-hand side of Eq. (9-16) minus the value of the right-hand side of Eq. (9-16) is shown in column 18. If the value in column 18 is zero, Eq. (9-16) has been satisfied, and the assumed depth is correct.

In order to obtain −0.23 ft in column 18 the value of $E_6 + S_f \Delta x(11.31 + 0.87)$ is subtracted from $E_0 + S_0 \Delta x(10.93 + 1.00)$. The result is not zero, indicating the assumed depth at 6 + 00 is incorrect. Therefore, a second iteration, with a new assumed depth, must be made.

Second iteration:

For the second iteration a depth $y_B = 10.7$ has been assumed and listed in column 3. Values for all 18 columns have been calculated as before, and again the computed values of $E_6 + S_f \Delta x(11.47 + 0.83)$ are subtracted from $E_0 + S_0 \Delta x(10.93 + 1.00)$, yielding a difference greater than that for the first iteration. The assumed depth is obviously still not correct; in fact, our correction after the first iteration was in the wrong direction. Our assumed depth should be slightly less than that for the first iteration. A third iteration must be performed.

Third iteration:

For the third iteration, a depth $y_B = 10.2$ has been assumed and is listed in column 3. Values for all columns have been calculated. The value of $(E_0 + S_0\,\Delta x) - (E_6 + S_f\,\Delta x)$ is $11.93 - 11.98 = -0.05$, a better agreement than in the previous two iterations. However, since the assumed depth is still not correct, a fourth iteration is required.

Fourth iteration:

A depth of $y_B = 10.1$ was assumed and values for all columns calculated. The agreement between $E_0 + S_0\,\Delta x$ and $E_6 + S_f\,\Delta x$ is now satisfactory. Thus the correct depth for section 6 + 00 is $y_B = 10.1$ ft. The velocity in the overbank areas and the main channel can now be computed in Eq. (9-7), since S_f must be the same for both subareas of the channel. Thus

$$V = \frac{1.5}{n} R^{2/3} S_f^{1/2} \tag{9-19}$$

For section 6 + 00,

$$V_A = \frac{1.5}{0.05}(2.87)(0.00150)^{1/2} = 3.33 \text{ ft/s}$$

$$V_B = \frac{1.5}{0.03}(4.25)(0.00150)^{1/2} = 8.23 \text{ ft/s}$$

(check Q) $\quad Q = 3.33(510) + 8.23(1020) = 10090$

For section 0 + 00,

$$V_A = \frac{1.5}{0.05}(2.83)(0.0055)^{1/2} = 3.34 \text{ ft/s}$$

$$V_B = \frac{1.5}{0.03}(4.23)(0.00155)^{1/2} = 8.32 \text{ ft/s}$$

(check Q) $\quad Q = 3.34(500) + 8.32(1000) = 9990$

Thus the velocities in the overbank area are less than half the velocity in the main channel. In each case, we checked to see if the indicated discharge equaled the 10,000 ft^3/s. The slight error that occurred for both 0 + 00 and 6 + 00 is due to the lack of precision in calculating the various column values used in calculating S_f. If desirable, further iterations could be performed using depth values with a precision to two decimal places. The calculated total Q will be in closer agreement with the known flow rate. ■

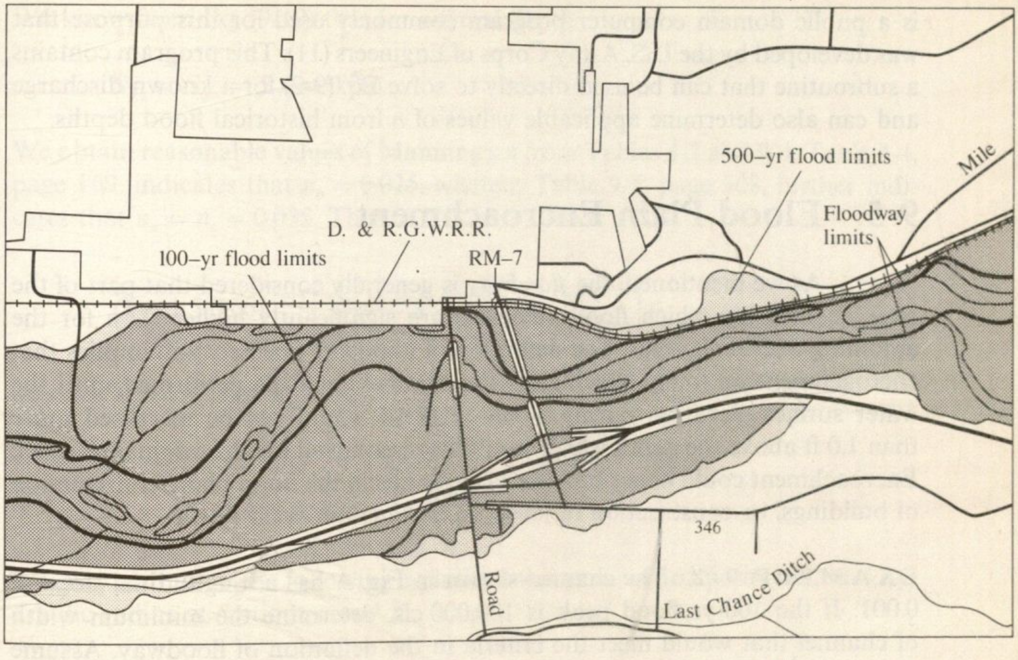


Figure 9-11 Plan of flood plain for Rifle, Colorado on the Colorado River showing floodway, 100-year, and 500-year flood limits (5)

Example 9-1 illustrates the method by which the depth of flooding and the flow velocities are calculated for a given flood discharge. Once the depth has been calculated at each section, the location of the limits of flooding can be established on a contour map of the area. Figure 9-11 illustrates the plan view of a flood plain as determined in an actual flood study.

In the process of flood routing through a river channel, the values of absolute roughness of the channel boundaries are not always apparent. Where trees, brush, fences, and small buildings exist, there is no accurate method by which resistance values can be determined from field observations. In these cases, it may be possible to obtain water surface elevations observed during a historical flood, or elevations may be estimated from debris lines formed during the flood and observed later. The peak discharge which occurred during the flood must be known and can frequently be obtained from gauged records on the river. Using this known discharge, Eq. (9-2), page 508, is solved for a set of assumed resistance coefficients. The process is somewhat difficult because the value of the resistance coefficient (Manning's n or the Darcy-Weisbach f) must be adjusted after each calculation of a free surface profile. If the calculated water surface is below the one observed during the actual flood, the resistance coefficient must be increased for the next iteration. *HEC-2, Water Surface Profiles* is a public domain computer program commonly used for this purpose that was developed by the U.S. Army Corps of Engineers (11). This program contains a subroutine that can be used directly to solve Eq. (9-2) for a known discharge and can also determine applicable values of n from historical flood depths.

9-5 Flood Plain Encroachment

As we mentioned, the *floodway* is generally considered that part of the river channel for which flood velocities are significantly higher than for the adjoining overbank areas. The definition of floodway (see Sec. 9-2) implies that encroachment on (or narrowing of) the channel could be performed until the water surface elevation during a 100-yr flood would not be increased more than 1.0 ft above the depth which would have occurred for the original channel. Encroachment could occur because of earth fills in the flood plain, construction of buildings, or construction of levees to confine the flood flows.

EXAMPLE 9-2 The channel shown in Fig. A has a longitudinal slope of 0.001. If the 100-yr flood peak is 100,000 cfs, determine the minimum width of channel that would meet the criteria in the definition of floodway. Assume the overbank area is covered with long grass and the main channel is a straight gravel-bed river. Further assume the encroachment occurs over a long enough distance so that flow occurs at normal depth throughout the part of the channel that is of interest.

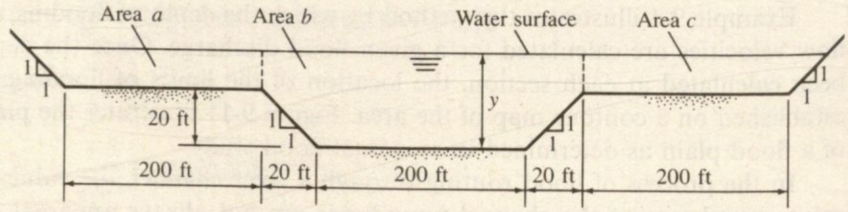


Figure A

SOLUTION Since normal depth is to be assumed, the problem involves determining normal depth both before and after encroachment. First, we will determine normal depth y for conditions before encroachment. The channel is first divided into the three separate subareas a, b, and c, as shown in Fig. A. Using Eq. (9-6), we write

$$Q = V_aA_a + V_bA_b + V_cA_c$$

$$= \frac{1.5}{n_a} A_aR_a^{2/3}S_{f_a}^{1/2} + \frac{1.5}{n_b} A_bR_b^{2/3}S_{f_b}^{1/2} + \frac{1.5}{n_c} A_cR_c^{2/3}S_{f_c}^{1/2}$$

We know from Eq. (9-10) that for normal depth

$$S_{f_a} = S_{f_b} = S_{f_c} = 0.001$$

We obtain reasonable values of Manning's n from Tables 4-1 and 9-4. Table 4-1, page 169, indicates that $n_b = 0.025$, whereas Table 9-4, page 508, further indicates that $n_a = n_c = 0.035$. Thus

$$Q = 1.5(0.001)^{1/2}\left[\frac{A_a R_a^{2/3}}{0.035} + \frac{A_b R_b^{2/3}}{0.025} + \frac{A_c R_c^{2/3}}{0.035}\right]$$

or since $A_a = A_c$ and $R_a = R_c$,

$$Q = 1.355(2A_a R_a^{2/3} + 1.4A_b R_b^{2/3})$$

or $$Q = 1.90(1.429A_a R_a^{2/3} + A_b R_b^{2/3})$$

We may also write the following equations for area, wetted perimeter, and hydraulic radius at sections a and b:

$$A_a = \left(200 + \frac{y-20}{2}\right)(y-20) \qquad A_b = 220(20) + 240(y-20)$$

$$= \left(190 + \frac{y}{2}\right)(y-20) \qquad = 4400 + 240(y-20)$$

$$P_a = 200 + y(2)^{1/2} \qquad P_b = 200 + 2(800)^{1/2}$$

$$= 200 + 1.414y \qquad = 200 + 56.6 = 256.6$$

Thus

$$R_a = \frac{(190 + y/2)(y-20)}{200 + 1.414y} \qquad R_b = \frac{4400 + 240(y-20)}{256.6}$$

To solve these equations for Q, R_a, and R_b, we will assume values of y and then calculate the implied value of Q. This process of iteration continues until the calculated value of Q for the assumed value of y agrees with the given total flow rate. Table A on the next page gives the assumed values of y and the calculated flow rates. In the table,. values of R_a and R_b are calculated from the assumed values of y. Q is then calculated from the resulting values of R_a and R_b. Each line is a separate iteration. In the first line, the calculated discharge is too small, so the assumed value of y is increased for the second iteration. Each of the other lines summarizes a new iteration, continuing until the calculated value of Q equals 99,846 cfs which differs from the given value by only 154 cfs which

Table A

y (ft)	A_a (ft^2)	$R_a^{2/3}$	A_b (ft^2)	$R_b^{2/3}$	Q (ft^3/s)
20	0	0	4400	6.65	55,594
22	402	1.45	4880	7.12	67,600
24	808	2.28	5360	7.58	82,255
26	1218	2.98	5840	8.03	98,967
26.2	1259.2	3.04	5888	8.07	100,840
26.1	1238.6	3.01	5864	8.05	99,846

is a reasonable accuracy for this problem. Note that more precision in the assumed values of the depth will lead to closer agreement between the known and the calculated values of Q. Thus the normal depth before encroachment is 26.1 ft to the nearest 0.1 ft.

Now, it is necessary to determine the amount of encroachment (as shown in Fig. B) that will cause the depth to increase from 26.1 to 27.1 ft. We will use the same set of equations that we used in the previous step, except in this case y and Q are both known. We will assume the encroachment has vertical side walls, as Fig. B shows. The floodway width is indicated as B. Again denoting the left overbank area as a and the main channel as b, we can write

$$A_a = \frac{(B-240)}{2}(y-20) \qquad A_b = 6128$$

$$= (B-240)(3.55)$$

$$P_a = \frac{B-240}{2} + (y-20) \qquad P_b = 256.6$$

$$= \frac{B}{2} - 112.9$$

$$R_a = \frac{3.1(B-240)}{(B/2-112.9)} \qquad R_b = 23.79$$

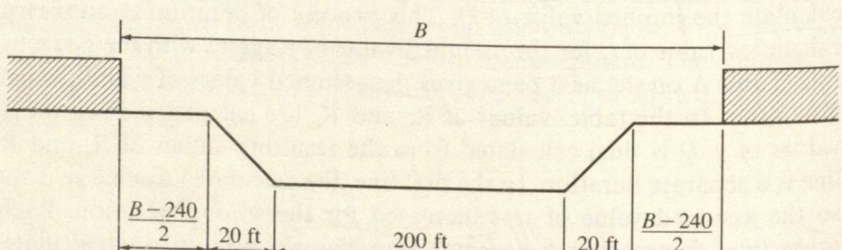


Figure B

Table B

B (ft)	A_b (ft^2)	$R_b^{2/3}$	$A_bR_b^{2/3}$	Q (cfs)
240	0	0	50,491	95,932
260	71	2.58	50,491	96,430
400	568	3.49	50,491	101,315
350	391	3.41	50,491	99,553
360	426	3.43	50,491	99,899
365	444	3.44	50,491	100,079

Again, a trial-and-error solution is required, this time assuming values of B. Table B shows the steps of the calculation. The foregoing equations for R_a and R_b are used with each assumed value of B. The equation developed for Q in the first step is again used to calculate the value of Q once the R_a and R_b values have been determined. Each line of the table is again a separate iteration. Thus the width of the floodway would be slightly less than 365 ft. In our solution, we have assumed encroachment takes place equally on each side of the channel. In the second trial-and-error solution, the value of $A_bR_b^{2/3}$ is constant, since we know that the depth must be $y_b = 27.1$ ft. ■

9-6 Risk Within the Flood Plain

The concept of delineating the flood plain makes it possible to determine to a limited degree, the risk of damage due to flooding. That risk, coupled with the damages to be expected during a specific level of flooding, can be used to estimate the benefits of providing flood protection. A simple example will illustrate the analysis of risk. If the probability of a flood elevation exceeding a given level is p in any year, and if the damage that would be suffered as a result of a single flood of that level is D, the *probable annual damage* due to a flood of that magnitude would be

$$D_A = pD \tag{9-20}$$

This concept is overly simple, since floods having a long duration may create more damage than floods of shorter duration having the same peak stage. The concept of probable annual damages can be used to size flood-control measures for design. Suppose we consider a flood-control levee and consider what recurrence interval of flood should be used in the design of the levee. If the levee is designed high enough so that it will not be overtopped or damaged during a 100-yr flood, it will generally be free from significant damage by flooding due to lesser events, and homes or other facilities protected by the levee will not be damaged by smaller floods. However, to make the levee high enough to avoid all damage for all time, the probable maximum flood would need to be considered the design event. Building a levee to protect against that flood will be too expensive for consideration in most flood-control projects. The optimum design event to use is determined by an incremental economic analysis of the cost of flood protection and the resulting benefits. Example 9-3 illustrates the procedure.

EXAMPLE 9-3 It is desirable to construct a flood-control levee along a river channel in New Mexico. The expected useful life of the levee is assumed to be 100 yrs. Table A gives the annual cost (column 2) of designing and constructing the levee high enough to prevent flooding up to the flood stage. Column 1 of Table A lists the flood stage, or elevation of the water surface in the river for the associated peak flow. The expected magnitude of damage that would be caused by a single flood event of that magnitude without the levee is given in column 4. The average return period for a flood with the given stage is shown in column 5. Using the given data, determine the optimum height to which the levee should be constructed.

Table A

(1) Flood Stage (ft)	(2) Annual Project Costs ($)	(3) River Flow Rate (cfs)	(4) Expected Damage Cost if Flow Rate Occurs ($)	(5) Average Return Interval (yr)
2010	0	8,000	0	1.055
2012	6,300	12,200	25,000	2.257
2014	26,000	25,600	162,000	5.879
2016	66,000	65,000	885,000	14.472
2017	108,000	90,000	2,036,000	27.701
2018	155,000	138,400	3,187,000	56.818
2019	204,000	205,000	3,681,500	116.279
2020	256,000	293,500	4,176,000	250.000
2021	312,000	425,000	4,303,000	666.667
2022	246,046	603,700	4,430,000	3333.333

SOLUTION Table B shows the calculations leading to the probable damages that could be expected if the levee were built high enough to protect against a flood having the stage given in column 1.

Columns 1 and 2 of Table B are the same as columns 1 and 3 in Table A. Thus in line 2 of Table B, the peak flow rate of 12,200 cfs would create a peak flood water surface elevation of 2012 and has an average return interval of 2.257 yr. Column 3 of Table B, shows that a flood of 12,200 cfs would occur, on the average, 100/2.257 = 44.31 times in 100 yr. The number in column 4 of Table B shows the probable average number of times in 100 yr that a flood would occur within the given range in stage as shown in column 1. Thus the probable number of times that a flood would occur between stages 2010 ft and 2012 ft is 94.81 − 44.31 = 50.5 (line 2). If the values in column 3 were divided by 100, the

Table B

	(1) Flood Stage (ft)	(2) River Flow Rate (cfs)	(3) Average Number of Times Flow is Exceeded in 100 yr	(4) Average Number of Times in 100-yr that Flow is in Range	(5) Average Damages in Range ($)	(6) Probable Damage Cost for 100 yr ($)
(1)	2010	8,000	94.81			
				50.5	12,500	631,250
(2)	2012	12,200	44.31			
				27.3	93,500	2,552,550
(3)	2014	25,600	17.01			
				10.1	523,500	5,287,350
(4)	2016	65,000	6.91			
				3.3	1,460,500	4,819,650
(5)	2017	90,000	3.61			
				1.85	2,611,500	4,831,275
(6)	2018	138,400	1.76			
				0.90	3,434,250	3,090,825
(7)	2019	205,000	0.86			
				0.46	3,928,750	1,807,225
(8)	2020	293,500	0.40			
				0.25	4,239,500	1,059,875
(9)	2021	425,000	0.15			
				0.12	4,366,500	523,980
(10)	2022	603,700	0.03			

result would be the average number of times per year that a flood could occur in that range. That concept is sometimes difficult to understand so a 100-yr period has been used here. The 100-yr period is arbitrary and does not imply that the expected useful life of the levee is 100 yr.

Column 5 of Table B lists the average magnitude of damage that would be expected for each occurrence of a flood with a stage between the given range. Table A shows that $0 and $25,000 in damages could be expected for floods having a stage of 2010 ft and 2012 ft, respectively if there were no levee. Thus an average damage per flood with a stage between 2010 ft and 2012 ft would be ($0 + $25,000)/2 = $12,500 as shown in line 2 of column 5. Column 6 shows the probable damages that should be expected in a 100-yr period from floods having stages between the given range. Thus the probable damages that should be expected during a 100-yr period is 50.5 × $12,500 = $631,250, as shown in line 2 of column 6.

The remaining lines of the table are completed similarly. The larger floods produce significantly greater damages for each flood. However, the probability of larger floods occurring is much smaller so that the probable damage to be expected during a 100-yr period becomes small for the large floods. Table B

Table C

(1) Level of Protection (ft)	(2) Annual Benefits ($)	(3) Incremental Benefits ($)	(4) Annual Project Costs ($)	(5) Incremental Costs ($)	(6) Benefits Minus Costs ($)
2010	0		0		
2012	6,312	6,312	6,300	6,300	12
2014	31,838	25,526	26,000	19,700	5,826
2016	84,711	52,873	66,000	40,000	18,873
2017	132,908	48,197	108,000	42,000	6,197
2018	181,221	48,313	155,000	47,000	1,313
2019	212,129	30,908	204,000	49,000	−18.992
2020	230,201	18,072	256,000	52,000	−33,919
2021	240,588	10,387	312,000	56,000	−45,613
2022	246,046	5,458	373,000	61,000	−54,542

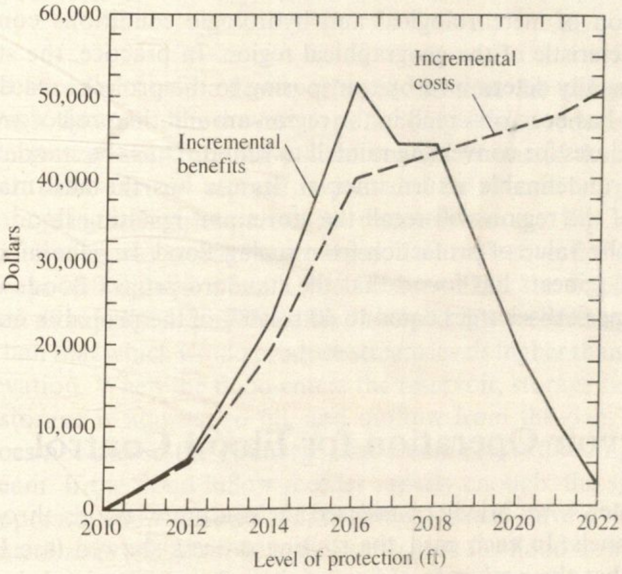
shows that for this case, the probable maximum damages of \$4,831,275 per 100 yr occurs for floods having stages between 2017 ft and 2018 ft. Note that \$4,831,275 per 100 yr is equivalent to a probable annual damage of \$48,313 per yr. Annual damages, rather than damages for 100 yr, are often used in economic analyses of flood protection systems.

The damage values calculated in Table B must be analyzed to determine the optimum height to which a levee should be constructed for flood-control benefits. According to the tabulated values, a levee constructed high enough so that it would not be overtopped (and strong enough to prevent structural failure) by floods having stages up to 2012 ft would prevent damages amounting to \$631,250 in 100 yr. Thus the probable benefits to be realized by building the levee to that height would be \$631,250 (line 2, column 6 of Table B) in 100 yr or \$6312/yr. Column 2 of Table C gives the probable annual benefits that would be realized by building a levee to protect against floods having stages ranging up to the given elevation. These cumulative benefits are calculated by adding successive values from column 6 of Table B and dividing by 100 yr. Thus the annual benefits from constructing a levee to elevation 2014 ft (the levee would be slightly higher to allow for some settlement and freeboard) is (\$631,250 + \$2,552,550)/100 = \$31,838/yr.

Column 4 of Table C (the same as column 2 of Table A) lists the cost of building levees of the given height. We furnish these values here without further explanation. However, they would be estimated by the engineer and would include the cost of right of way; engineering design; stripping the foundation; excavating, hauling, placing, and compacting the embankment; and riprap protection on the river side of the levee. The costs are given as annual costs, which would be determined by calculating the total engineering and construction cost of the levee, relating that to the equivalent annual investment cost, and then adding annual maintenance costs. The incremental costs shown in column 5 of

Table C are determined by subtracting adjoining values in column 4. The incremental costs increase nonlinearly with levee height because the bottom of the levee becomes wider as it becomes higher, necessitating much more embankment material for a 1-ft increase in height of a tall levee than for an equal increase of a short levee.

The following figure shows the incremental annual costs of construction and incremental annual benefits arising from constructing the levee as a function of the design flood for the levee. The figure shows that benefits increase faster than costs up to a levee height for elevation 2016 and increase slower than costs for higher design elevations.



Column 6 of Table C shows that building a successively higher levee achieves increasing benefits that are more than the incremental costs up to protection from floods having stages to elevation 2018 ft. If the levee were increased in height to elevation 2019 ft, the increase in benefits realized would be less than the increase in cost. Thus it is not economically advantageous to build the levee to protect against floods having stages greater than 2018 ft. However, the benefit-cost ratio is greater than 1 up to protection from floods having stages of 2019 ft. Thus if one wished to build an economically feasible project with the maximum benefit, the levee would be constructed to protect from flood stages up to 2019 ft. However, from an optimum economic investment goal, the levee would be built to protect against stages only up to elevation 2018 ft. ■

A similar analysis could be made for any flood control structure or channel improvement. However, such a procedure would have the inherent difficulty that each portion of the overall flood-control plan might end up being designed and constructed for a different design flood. Although there is nothing technically incorrect about such a procedure, the overall analysis required is prohibitive. Moreover, the damages estimated, based on current development conditions, will surely change in the future and render the currently estimated damages incorrect. This effect must be allowed for by increasing the current estimate expected flood damages.

Federal programs have adopted standard design recurrence intervals for the design of most flood protection facilities. The U.S. Army Corps of Engineers has responsibility for flood control on all navigable streams. Their practice has been to study flood-control projects for areas having a high degree of hazard using the "standard project flood," a flood that would be created by the most severe combination of meteorological and hydrologic conditions considered reasonably characteristic of the geographical region. In practice, the standard project flood is usually determined by transposing to the project area the most severe storm that has been observed in the region around the project and using hydrologic procedures for converting rainfall to runoff. Thus the standard project flood has an undefinable return interval. Its use has the advantage that many residents of the region will recall the storm and resulting flood and are able to relate to the value of protection from such a flood. In general, the U.S. Army Corps of Engineers has found that the standard project floods used for design have had peak discharges equal to 40 to 60% of the probable maximum flood (1).

9-7 Reservoir Operation for Flood Control

In Chapter 4, we briefly considered the routing of floods through reservoirs and channels. In each case, the routing process showed (see Example 4-11, page 220) that the peak rate of flow of the inflow hydrograph is reduced as a result of part of the flood volume being stored and released at a later time when the inflow flood has somewhat receded. This is shown in Fig. 9-5, page 503. For this reason dams and the reservoirs they create are frequently the most significant features of a flood-control plan. Both large and small reservoirs are used extensively to reduce the depth and extent of downstream flooding. In an urban setting, the reservoirs are used extensively to reduce the volume of runoff occurring quickly after the storm. The volume of runoff temporarily detained in the reservoir is released as rapidly as possible after the storm while maintaining downstream flows at less than that which would cause downstream flooding.

The reservoirs in urban settings are usually small detention ponds and are frequently built as a part of parks or recreational areas. In regional settings, large reservoirs, are common and are used to store flood waters from large drainage areas. The reservoirs created by large dams usually store water which can be released to meet multiple needs such as municipal and industrial, navigational, and the instream needs of aquatic life. Energy is usually generated by

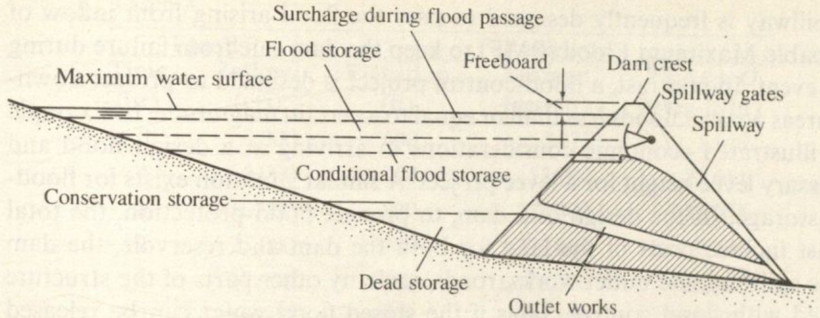


Figure 9-12 Allocation of flood-storage space in a multipurpose reservoir

hydroelectric plants as the flow is released. Because the water is usually stored during high flow periods, downstream flooding is also reduced. Table 9-3 lists several large reservoirs that have flood control as one of their benefits.

When flood-control storage is provided in a multi purpose project, the flood storage is the highest part of the reservoir storage. Figure 9-12 illustrates schematically the way storage is arranged. To be effective, flood-control storage must always be available when a flood occurs. Thus the flood-control storage is emptied as rapidly as possible after the flood has receded using release rates smaller than that which would produce stream levels higher than the downstream flood elevation. When the flood enters the reservoir, storage begins. The flood-control storage is allowed to fill, and outflow from the dam is controlled so that it does not exceed the discharge that would create flooding along the river downstream. If the flood inflow recedes rapidly enough, the spillway may not need to operate, and the outlet works (which usually have a large capacity for flood-control reservoirs) will be sufficient to pass the flood flow. However, after the passage of one flood, the flood-storage space must be emptied as rapidly as permissible so that the reserved flood-control storage will be available to store all or part of the next flood.

Depending on the climate in which the reservoir is located, the storage zone shown in Fig. 9-12 may be allocated differently. In a region like California, the rainfall season is almost totally confined to September through May. Flows due to snowmelt in the mountains may be large until the end of May. However, there is little probability that a significant flood can occur during the summer months. Thus during the summer months, the flood-storage zone is often allocated to conservation storage after the flood season. In regions such as the middle and eastern United States, where summer floods can be expected because of intense rainfall, the flood-storage zone in Fig. 9-12 will be reserved at all times of the year. Allocation of this storage is done by a "rule curve," which we discuss later.

A distinction should be made between the spillway design flood and the design flood for flood control. For a major dam, as we pointed out in Chapter

2, the spillway is frequently designed to pass the flood arising from inflow of the Probable Maximum Flood (PMF) to keep the dam safe from failure during such an event. In contrast, a flood-control project is designed to protect downstream areas against floods less than or equal to a certain magnitude. In Example 9-3, we illustrated economic considerations in arriving at a design flood and the necessary levee height for a levee project. A similar situation exists for flood-control storage. In the design of a dam, to provide flood protection, the total cost must include costs of the land for both the dam and reservoir, the dam itself, the spillway, the outlet works, roads, and any other parts of the structure associated with flood control. Thus if the stored flood water can be released through turbines which turn generators, then at least part of their cost could also be charged to flood control. Revenue generated by the sale of electricity will often greatly offset the cost of the project. Communication facilities and equipment to provide the hydrologic forecasts for control of the project are also part of the real costs.

The benefits from flood control are computed using the reduction in damages as a result of decreased downstream flood levels and reduced flood occurrence. The reductions in flood levels are computed by determining the depth of flood in the river channel that would occur downstream before construction of the dam for a flood of given return interval. Then the same flood is routed through the reservoir, and the flood stage arising from the peak routed outflow is computed. The reduction in damage as a result of the decreased level is then counted as a project benefit.

EXAMPLE 9-4 Table A gives the hydrograph for a 50-yr flood entering a reservoir. Route the flood though the reservoir, whose characteristics are also given in the table. Assume the routed outflow downstream must be limited to 8000 cfs if possible. What is the maximum elevation reached by the reservoir during the flood? If the channel characteristics at a particular location downstream are shown in the accompanying figure, calculate the reduction in flood level achieved by the reservoir. The top of the dam is at elevation 1135, the uncontrolled spillway crest is at elevation 1088, and the bottom of the flood-control pool is at elevation 1049.5. Figure 9-12, which illustrates the storage zones within the reservoir, should be used to identify the pertinent zones, and their respective elevations should be noted.

SOLUTION At what level should the reservoir water surface be assumed to be at the beginning of the flood? Although it is possible that the reservoir will be lower, it is always assumed to be not higher than at the bottom of the flood-control pool when the inflow flood begins. Thus we will assume the reservoir is at elevation 1049.5 at time zero when the flood begins. Similarly, although it is not a factor in this problem, the reservoir level would be assumed to be at the top of the spillway crest at the beginning of a probable maximum flood. Equation (4-51), page 219, is used to perform the routing, and Table B, page 528, (which is similar to the table for Ex. 4-11, page 221) is formed to perform

Table A

	Flood		Reservoir	
Time (hr)	Inflow (cfs)	Elevation (ft)	Storage (acre-ft)	$2S/\Delta t + O$ (acre-ft/hr)
0	2,000	600	975	1,148
4	5,000	700	53,900	27,611
8	10,000	800	272,800	137,061
12	25,000	900	723,000	362,161
16	33,000	1000	1,471,200	736,261
20	45,000	1049.5	1,969,500	985,411
24	50,000	1050	1,975,000	988,161
28	102,000	1052	1,997,000	999,161
32	72,000	1054	2,019,200	1,010,261
36	62,000	1056	2,041,500	1,021,411
40	60,000	1058	2,064,000	1,032,661
44	50,000	1060	2,086,600	1,043,961
48	45,000	1062	2,109,400	1,055,361
52	45,000	1064	2,132,300	1,066,811
56	41,000	1066	2,155,400	1,078,361
60	20,000	1068	2,178,700	1,090,011
64	15,000	1070	2,202,000	1,101,661
68	11,000	1072	2,225,500	1,113,461
72	9,000	1074	2,249,300	1,125,311
76	8,000	1076	2,273,100	1,137,211
80	6,000	1078	2,297,100	1,149,211
84	4,000	1080	2,321,300	1,161,311

the necessary calculations. We will use a 4-hr routing period in performing the calculations. The choice of this routing period is dictated by the base of the hydrograph and must be chosen small enough to define the hydrograph well. The last column of Table A is first calculated assuming the outflow O is 8000 cfs and $\Delta t = 4$ hr. Note that 8000 cfs is converted to 661 acre-ft/hr for use in the table.

The calculation steps for Table B are the same as those described in Example 4-11 with the exception of the outflow. For this example, the outflow was fixed at 8000 cfs (661 acre-ft/hr) after the first two time steps. For the initial step between 0 and 4 hr, the outflow was taken to be 2000 cfs (165 acre-ft/hr), or the same as the inflow. This assumption was made because the dam operator would not normally release more than the inflow up to the time when the inflow equaled the maximum permissible release of 8000 cfs. Similarly, between time 4 and 8 hr, the outflow was taken as equal to the inflow, or 5000 cfs (413 acre-ft/hr). To obtain the last column in Table B, the stage was interpolated from values of $(2S_1/\Delta t + O_1)$, as given in Table A.

Table B

Time (hr)	$I_1 + I_2$ (acre-ft/hr)	S_i (acre-ft)	O_i (acre-ft/hr)	$\frac{2S_i}{\Delta t} - O_i$ (acre-ft/hr)	$\frac{2S_i}{\Delta t} + O_i$ (acre-ft/hr)	Reservoir Elevation (ft)
0		1,969,500				1049.5
4	578		166	984,584	985,162	1049.4
8	1,240		413	984,336	985,576	1049.6
12	2,892		661	984,254	987,146	1049.8
16	4,793		661	985,824	990,617	1050.4
20	6,446		661	989,295	995,741	1051.4
24	7,851		661	994,419	1,002,270	1052.6
28	8,843		661	1,000,948	1,009,791	1053.9
32	14,380		661	1,008,469	1,022,849	1056.3
36	11,074		661	1,021,527	1,032,601	1058.0
40	10,083		661	1,031,279	1,041,362	1059.5
44	9,091		661	1,040,040	1,049,131	1060.9
48	7,851		661	1,047,807	1,055,660	1062.0
52	7,438		661	1,054,338	1,061,776	1063.1
56	7,107		661	1,060,454	1,067,561	1064.1
60	5,041		661	1,066,239	1,071,280	1064.8
64	2,892		661	1,069,958	1,072,850	1065.0
68	2,149		661	1,071,528	1,073,677	1065.2
72	1,653		661	1,072,355	1,074,008	1065.2
76	1,405		661	1,072,686	1,074,091	1065.3
80	1,157		661	1,072,769	1,073,926	1065.2
84	826		661	1,072,604	1,073,430	1065.1

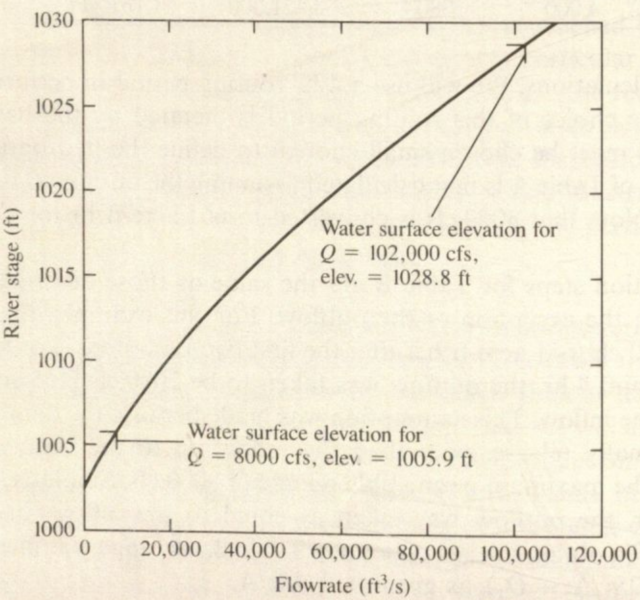


Table B shows that the maximum elevation reached by the reservoir was 1065.3 ft, or nearly 23 ft below the spillway crest. Thus it is possible to pass the given flood without exceeding an outflow of 8000 cfs from the reservoir. The 50-yr flood is thus reduced from a peak inflow of 102,000 cfs to a downstream peak of 8000 cfs as a result of flood-control storage provided by the reservoir. The figure shows that the water surface elevation that would have occurred for the flood of 102,000 cfs would have been 1028.8 ft, whereas for the reduced discharge of 8000 cfs, it is reduced to 1005.9 ft. ■

When reservoirs are constructed for multiple purposes, of which flood control is only one, the space reserved for flood control can sometimes be variable depending on the season and known hydrologic conditions (conditional flood-control storage shown in Fig. 9-12). For example, many major reservoirs constructed by the federal or state governments in California have been constructed to provide conservation storage, flood control, and hydroelectric generation. Examples of these projects include Shasta Dam, built by the U.S. Bureau of Reclamation (18); New Melones Dam, built by the U.S. Army Corp of Engineers (17); and Oroville Dam, built by the California Department of Water Resources (5). All three reservoirs have drainage areas for which the upper reaches (above approximately 6000 feet) often contain significant volumes of water in the form of snow when the runoff season begins. Snow surveys made on established snow courses in the Sierra Nevada mountains provide an estimate of the total volume of water contained in the existing snowpack. Results of these snow surveys are normally published beginning in January, with new data being measured each month. The amount of water that will run off because of snowmelt is estimated, and the result is used to estimate the amount of runoff that could be stored without violating flood-control space. Figure 9-13 shows the amounts of flood storage, conservation storage, and surcharge provided within New Melones Reservoir and the way in which the reservoir is seasonally programmed for control. The graph shown constitutes a rule curve for reservoir operation. As

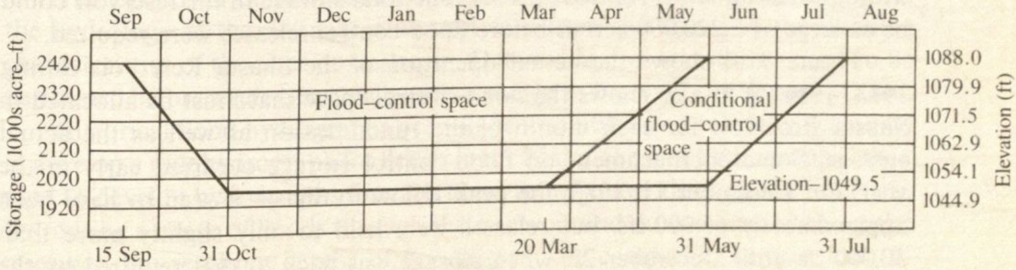


Figure 9-13 Rule curve for operation of flood-control storage space in the reservoir behind New Melones Dam on the Stanilaus River in California (17)

shown in Fig. 9-13, the bottom of the flood-control storage is at elevation 1049.5 ft, and 1,970,000 acre-ft of water can be stored in the reservoir below that elevation. The spillway crest is at elevation 1088.0 ft, and storage above that level is temporary flood surcharge, since the spillway is uncontrolled.

As the rule curve of Fig. 9-13 shows, no flood-control space needs to be retained between July 31 and September 15 of any year. Because a significant amount of rainfall is extremely rare in California between these dates. Beginning on September 15, the reservoir must be drawn down so that the reservoir water surface is down to elevation 1049.5 ft by October 31. Normally, this drawdown is accomplished through required releases of water for irrigation and municipal and industrial uses. If for some reason, those releases did not require all the water stored above elevation 1049.5, water would have to be released and wasted to evacuate the required flood-control space. For a multireservoir system, such as exists for tributaries to the Sacramento River in California's Central Valley, coordinated operation makes efficient overall operation possible, and unnecessary release of water is infrequent. When releases need to be made in one reservoir to empty required flood-control space, that water can usually be used to meet downstream demands that might otherwise have been met by releasing water from another reservoir in the system.

From October 31 to March 20, the entire flood-control space is reserved for flood control. Whenever water is stored in the flood-control space it must be released as rapidly as possible without exceeding 8000 cfs downstream. After March 20, the amount of flood-control space to be maintained is subject to the amount of runoff to be expected before July 31. Thus if on March 20, snow surveys indicated that the total runoff from snowmelt would be approximately 450,000 acre-ft, the lower curve must be followed, and the entire 450,000 acre-ft of flood-control space must be reserved until May 31, after which runoff can be stored until the reservoir is full on July 31.

If, however, the snow surveys indicate a potential snowmelt runoff of only 200,000 acre-ft, water may be stored in the reservoir as long as 200,000 acre-ft is reserved for the snowmelt runoff. Thus on March 20, the dam operator would begin to store water and could store up to 250,000 acre-ft (450,000 – 200,000) without making flood-control releases. The total storage in the reservoir could be as large as 2,220,000 acre-ft before flood-control releases were required.

Figure 9-14 shows the actual operation of the Shasta Reservoir during 1983–1984 (5). It also shows the flood-control space that must be allocated in Shasta Reservoir for each month of the runoff season as well as the actual storage. Some encroachment on flood-control storage occurred early in the year. On December 11, 1983, the peak inflow to Shasta is seen to have been approximately 65,000 cfs, but releases were held to only slightly more than 30,000 cfs until December 22, when storage was back to that required by the rule curve. On December 23, a second large inflow occurred, but it did not exceed 35,000 cfs. Between February 1 and February 10, the snowmelt forecasts indicated that flood-control storage could be reduced, and subsequently releases

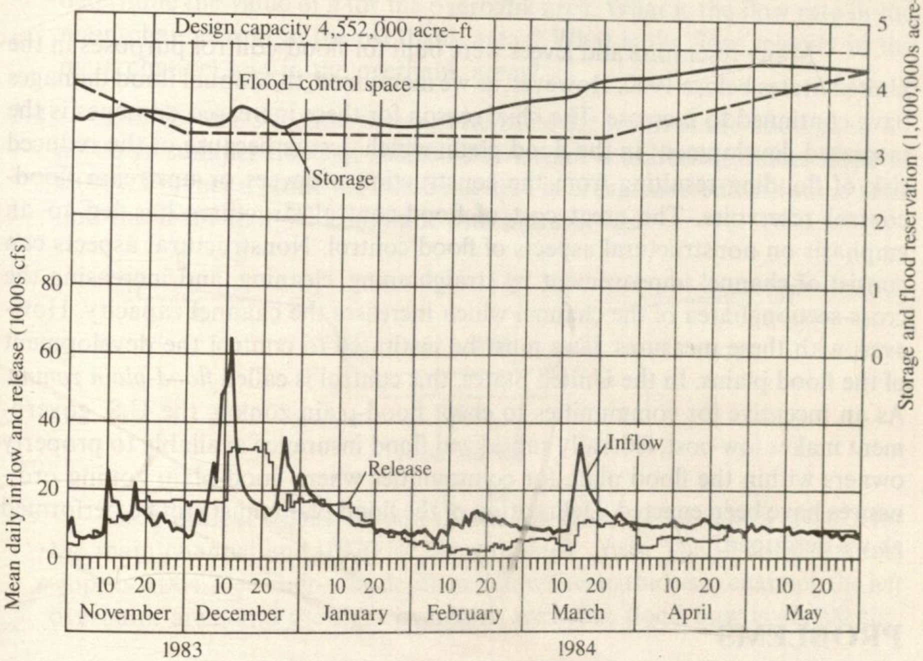


Figure 9-14 Operation of Shasta Dam flood-control space during the 1983-1984 flood on the Sacramento River (5)

were kept to less than inflow so that storage in the reservoir increased to approximately 4,300,000 acre-ft by April 23, 1984. Figure 9-14 is typical of the operation of multipurpose reservoirs subject to snowmelt inflow.

The operation of reservoirs for flood control, thus depends strongly on the hydrology of the drainage area upstream from the dam and the flooding characteristics of the river downstream. Detailed hydrologic studies need to be performed before a rule curve such as that shown in Fig. 9-13 can be constructed.

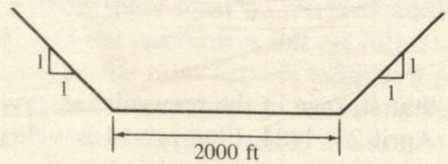
Historical flood hydrographs need to be assembled and carefully examined to develop a unit hydrograph for floods in the basin. The unit hydrograph is then used to develop hydrographs for each flood to be routed through the reservoir. By routing a range of floods through the reservoir and downstream without the reservoir, the resulting depths of flooding can be compared, as was done in Example 9-4. Damages prevented by the dam can then be evaluated. Damages prevented are considered benefits due to the provision of flood control and can be used to assess the economic feasibility of the flood-control aspects of the project.

9-8 Nonstructural Aspects of Flood Control

Many reservoirs and levees were built for flood-control purposes in the United States before 1960. However, as we mentioned, the annual flood damages have continued to increase. The chief reason for these increased damages is the increased development in the flood plain, which occurs because of the reduced risk of flooding resulting from the construction of levees or upstream flood-control reservoirs. The great cost of flood-control structures has led to an emphasis on nonstructural aspects of flood control. Nonstructural aspects can consist of channel improvement by straightening, cleaning, and increasing the cross-sectional area of the channel which increases the channel capacity. However, with these measures, laws must be instituted to control the development of the flood plains. In the United States, this control is called *flood-plain zoning*. As an incentive for communities to enact flood-plain zoning, the U.S. government makes low-cost, federally subsidized flood insurance available to property owners within the flood plain for communities where flood-plain zoning ordinances have been enacted. Delineation of the flood plain must still be performed as we described.

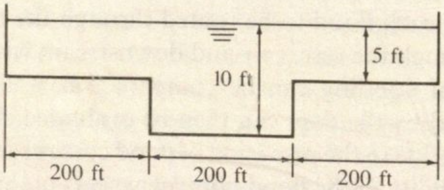
PROBLEMS

9-1 A river channel has the cross section shown in the figure. The channel slope is a uniform 0.02%. If Manning's n for the channel is 0.025, determine the flow depth for a probable maximum flood. The river is in the Ohio-Tennessee region of the United States and has a drainage area of 500 mi^2.



PROBLEM 9-1

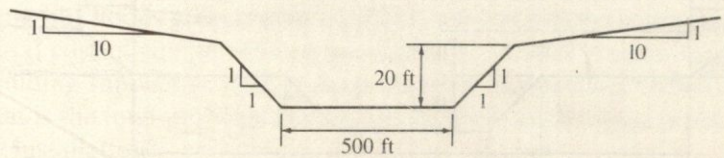
9-2 A river with the cross section shown, experienced a 50,750 cfs flood and produced the depth shown. Determine Manning's n for the main channel and overbank area assuming they are equal. The channel has a uniform slope of 0.45%.



PROBLEM 9-2

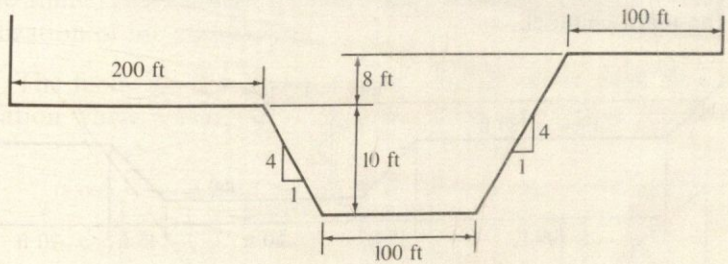
9-3 If, in Prob. 9-2, Manning's n is known to be 0.020 in the main channel, determine the value of n for the overbank area. What is the flow rate in the main channel and in the overbank areas? What is the flow velocity in the main channel and in the overbank areas?

9-4 For the cross section shown, determine the width of the flood plain for a 100-yr summer flood of 100,000 cfs. Assume a uniform channel slope of 0.09%. Further assume the overbank area is brush-covered with some trees and the main channel is straight with a gravel bed.



PROBLEM 9-4

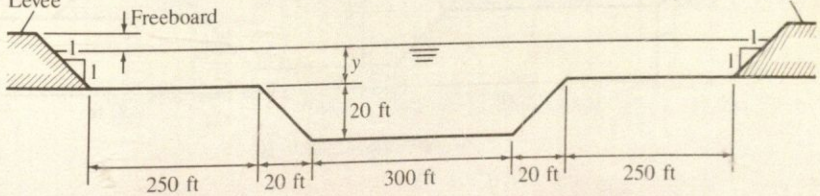
9-5 The compound cross section shown has Manning's n values of 0.025 for the main channel and 0.070 for the overbank areas. The uniform channel slope is 0.004. Determine the depth and flow rate in the main channel, the left overbank area, and the right overbank area. The flood flow is 45,400 cfs.



PROBLEM 9-5

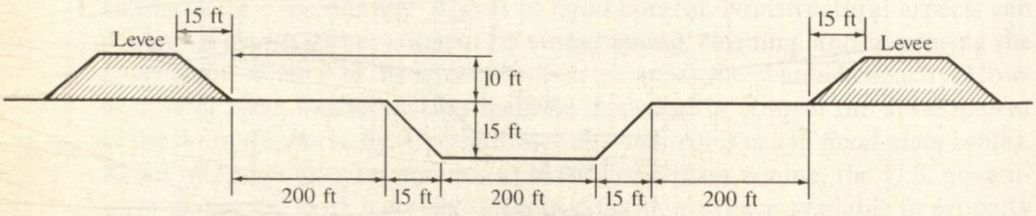
9-6 What would be the depth of flow for the cross section shown for Prob. 9-5 if the flow rate were 85,000 cfs?

9-7 Levees are to be built on both sides of the river whose cross section is shown in the figure. Protection is required from the 50-yr flood, which has a magnitude of 172,000 cfs. The overbank areas are covered with tall



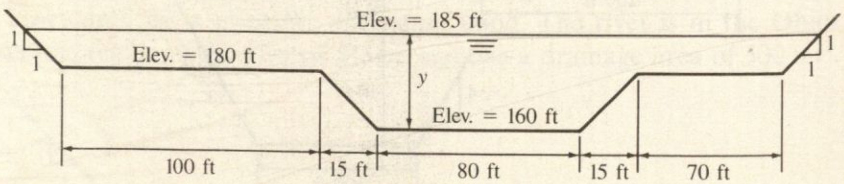
grass, and the main channel is covered with gravel beds and some boulders. How high must the levee be if it is to have 2 ft of freeboard? What velocity of flow will occur near the levee? The uniform slope of the channel is 0.15%.

9-8 The channel cross section shown has been constructed for flood protection. At what flow rate would the levees be overtopped. Assume the channel has a uniform slope of 0.008 and that Manning's n is 0.060 and 0.040 for the overbank and the main channel, respectively.



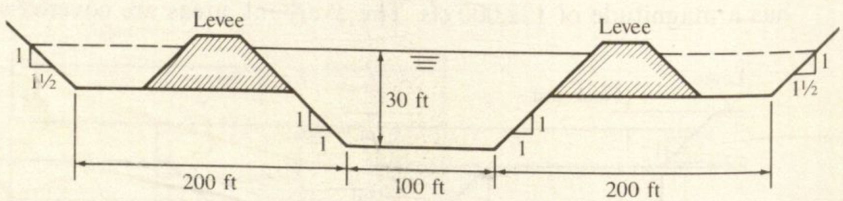
PROBLEM 9-8

9-9 The 100-yr peak flow for the river whose cross section is shown in the figure is 56,000 cfs. If in normal conditions, the river can carry the 100-yr flow with a water surface elevation of 185 ft, what is the width of the floodway? Assume Manning's n is 0.06 for the overbank areas and 0.035 for the main channel.



PROBLEM 9-9

9-10 Levees are constructed that contract a river as shown in the figure. A 50-yr flood of 50,000 cfs creates a depth of 30 ft as shown. Determine the flood water surface profile for a distance of 2000 ft upstream if the channel slope is 0.0015, assume $n = 0.030$.



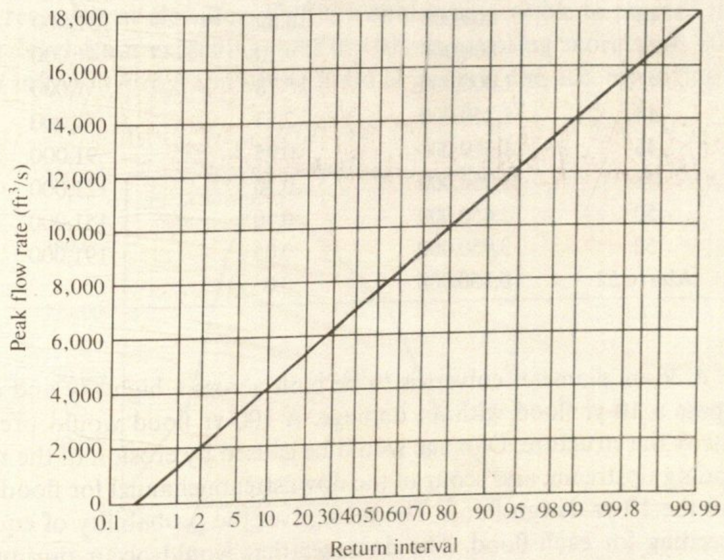
PROBLEM 9-10

9-11 For the cross section used in Prob. 9-2, the Manning's n values are known to be 0.050 for the overbank areas and 0.030 for the main channel. For a flow of 65,000 cfs, the depth in the main channel at a specific section is 14 ft. Calculate the depth of flow in the main channel that will exist at a similar section 4000 ft downstream. Assume the slope between the two sections is 0.005.

9-12 A local agency desires to construct a stream gauge on a nearby stream. The life expectancy of the recorder and other equipment is 10 yr. Floods equal to or smaller than 10,000 cfs will not damage the structure or equipment, but floods greater than 10,000 cfs will destroy the entire installation. Initial cost of equipment and installation is \$12,000. Annual maintenance including supplies is \$600. If the probability of exceeding 10,000 cfs is 1%, what is the total probable annual cost exclusive of the initial investment for the installation?

9-13 A pipeline is to be constructed across a major river by burying the pipeline beneath the riverbed. Burial depth will be sufficient to prevent the pipe from being damaged during a 25-yr flood. The original cost of constructing the crossing is \$75,000. If a 50-yr flood will destroy the crossing and require totally new construction, and damages are linearly proportional to the return period between the 25- and 50-yr events, determine the probable annual cost during the desired 25-yr life of the structure. (Neglect capitalization of the initial cost.)

9-14 The figure gives a graph of return period versus peak flow rate at a location where a bridge is to be constructed. The bridge will be designed



so that it sustains minimal damage during a 10-yr flood but is destroyed by a 50-yr flood. Assuming damage is linearly proportional to the difference between discharge and design discharge between the 10-yr and 50-yr floods and initial construction of the bridge costs \$200,000, calculate the average annual damage over the 20-yr life of the bridge. (Neglect capitalization of the initial cost.)

9-15 The table lists flood stages and the flooding damages that have been found to occur during floods of the indicated stage. Column 3 presents the percentage chance that a flood can equal or exceed that stage. Column 4 presents the annualized cost of constructing a levee high enough to prevent all flooding by a flood of the indicated state. Is it economically feasible to construct a levy? If it is feasible, to what height should it be constructed for

a. The maximum benefit
b. The most economical solution

(1) Flood Stage with No Protection (ft)	(2) Damage for One Flood at Stage (\$)	(3) Chance That Flood Could Be Exceeded (%)	(4) Annual Cost of Constructing a Levy to Prevent Flooding (\$)
35	0	49.50	0
36	100,000	41.50	4,000
37	200,000	33.50	9,000
38	300,000	25.50	14,000
39	450,000	18.50	22,000
40	600,000	13.50	31,000
41	800,000	9.10	41,000
42	1,000,000	5.78	51,000
44	1,250,000	2.53	71,000
46	1,550,000	0.95	91,000
48	1,950,000	0.30	121,000
50	2,450,000	0.10	151,000
52	3,950,000	0.05	191,000
Above 52	6,150,000	0.0	—

9-16 A 36-in. diameter culvert is to be built across a highway and designed to pass a 10-yr flood without damage. A 100-yr flood would presumably destroy the structure. Damage would be caused by erosion of the roadway, flooding upstream, and scour of the downstream channel for floods greater than the 10-yr design flood. The table gives the probability of equaling or exceeding for each flood. The damages that would occur during a flood

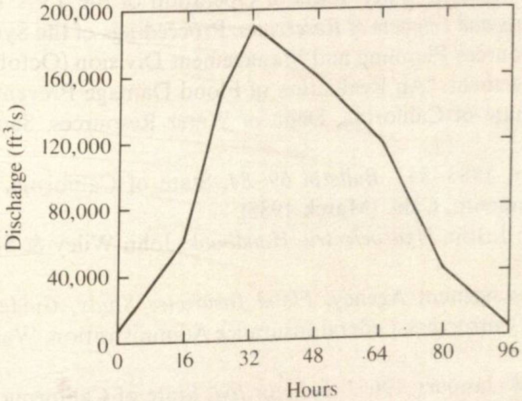
Discharge (ft^3/s)	Probability of Equaling or Exceeding (%)
30	10.0
35	8.0
40	6.0
45	5.0
50	4.0
55	3.5
60	2.5
65	2.0
70	1.5
75	1.2
100	1.0
150	0

can be computed as

$$\$D = 75 + 7(Q - Q_D)$$

where $\$D$ is the damage caused by one flood, Q is the flood discharge, and Q_D is the design discharge in ft^3/s for the culvert. What is the probable annual flood damage that should be planned for in estimating maintenance for this culvert?

9-17 For the New Melones Reservoir, the rule curve for operation is given in Fig. 9-13, page 529. If a flood were to occur beginning October 31 with the reservoir at elevation 1049.5, would it be possible to prevent the reservoir level from exceeding 1088.0 without releasing more than 8000 ft^3/s? The inflow hydrograph of the flood is given in the accompanying figure.



9-18 Use Fig. 9-13, page 529, to determine the maximum volume of inflow that could be reserved in New Melones Reservoir without exceeding an outflow of 8000 ft^3/s.

9-19 A dam is to be built solely for flood control. The annualized cost of the dam is shown in the table. The table also shows the return period for the flood volume that could be passed through the dam without exceeding flood stages downstream for each height of dam considered and the damages that would occur downstream if the dam were not there. Determine the indicated height of dam that should be constructed if the most economical solution is desired.

Height of Dam (ft)	Probability of Equaling or Exceeding Flood Volume That Could Be Passed Without Exceeding Downstream Flood Stage (%)	Annual Cost of Dam ($)	Damages Produced by the Flood ($)
100	1.0	50,000	6,000,000
150	0.5	70,000	15,000,000
200	0.2	140,000	76,000,000
250	0.1	400,000	160,000,000
300	0.05	1,500,000	400,000,000

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