MANUAL

OF

IRRIGATION ENGINEERING.

BY

HERBERT M. WILSON, C.E.

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PREFACE.

THE need of a comprehensive treatise on irrigation has been so frequently brought to my attention during the last few years, that I have undertaken to write this book with the hope that it may help those who are engaged in the study or practice of irrigation engineering. It is chiefly the result of original investigation, the descriptions of works being made from personal observation in America, Europe, and India.

Some of the matter contained in Part I is compiled, and in its preparation I am especially indebted for information and suggestions to the valuable work on "Water Supply Engineering," by Mr. J. T. Fanning. There is added, however, much that is new, a portion of which was obtained from the reports of Mr. F. H. Newell, Chief Hydrographer of the U. S. Geological Survey. The purpose has been to include in Part I only so much of hydraulics as is an indispensable preliminary to the remainder of the book, or is original matter. Wherever the subject has been treated by others the reader is referred to their works.

The entire book relates directly to the conditions surrounding Western irrigation practice. The examples given and the suggestions made apply immediately to Western methods, though many useful hints are borrowed from foreign experience. The classification adopted is original, I believe, and follows closely that employed in reports made by me to the Government, which seem to have met with general approval. In this classification the terms "diversion weirs" and "dams" have been used with special signification. Under the term "diversion weirs" are included all obstructions built across running streams and designed to act as overflow weirs, though their functions may be those either of storage dams or diversion weirs or both. Under the term "dams" are included all retaining walls, of whatsoever material, which are intended only to impound water and are not so constructed as to withstand the shock of falling water. These classes necessarily overlap to some extent.

The subject of the application of water to crops is but briefly touched upon. It would in itself require a volume, and is one of more interest to the farmer than to the engineer. Part III, which treats of storage works, contains much new material never before brought together, and this is especially true of the chapters on Earth Dams and Pumping. The theory of high masonry dams is but briefly considered, as this subject has already been exhaustively treated by previous writers, to whose works reference is made. What little has been said concerning it is partly compiled, the chief source being Wegmann's admirable treatise on masonry dams. Great care has been taken throughout the volume to avoid the use of mathematics, since many of the formulas given on the flow of water in open or closed channels, on the discharge from catchment basins, and on strains in masonry dams are exceedingly faulty and misleading. We have much to learn before we can apply mathematics to these subjects with accuracy. I consider it better to follow practical usage and experience than theory where the latter is founded on doubtful premises and is liable to produce inaccurate results if adhered to closely.

The endeavor has been to prepare a work which will be of value to the practical engineer as well as to the student. It was found impossible to include within the covers of one volume the necessary tables on hydraulics and flow of water. It is believed, however, that this book contains much that will be useful to the practical engineer, and that the teacher of irrigation engineering will find the facts assembled in such manner as to be materially helpful.

The effort has been to illustrate all the important works described, as well as types of works, in order that practising engineers may obtain suggestions from the experience of others.

I am indebted to the courtesy of the Director of the U. S. Geological Survey for numerous electrotypes of illustrations, which had been previously published in reports made by me. Several illustrations were also obtained through the courtesy of the Secretary of the American Society of Civil Engineers, being electrotypes of those used in papers read by me before that society.

WASHINGTON, D. C., January, 1893.

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CHAPTER I.

INTRODUCTION.

1. Extent of Irrigation.—The extent to which irrigation can be practised is enormous. The total area irrigated in India is about 25,000,000 acres, in Egypt about 6,000,000 acres, and in Italy about 3,700,000 acres. In Spain there are 500,000 acres, in France 400,000 acres, and in the United States 4,000,000 acres of irrigated land. This means that crops are grown on 39,000,000 acres of land which but for irrigation would be barren and unproductive. In addition to this there are some millions more of acres cultivated by the aid of irrigation in China, Japan, Australia, Algeria, South America, and elsewhere.

2. Control of Irrigation Works.—The development of irrigation has resulted in many legal complications, while a diversity of social and physical conditions has given rise to a variety of methods for its control. Practically all the works in India are now under the direct control of the government, which employs its engineers and legal staff, owns the land and the water, constructs the works, and collects the rentals for the use of water and land. In the Piedmont valley of Italy the land is the property of individuals, and in some cases individuals are owners of the irrigation works. In the case of the Cavour canal, however, the government owns and operates the works, and the water is sold to the cultivators. In the United States all irrigation works are the property of individuals who construct and maintain them and collect the rentals for the use of water. In some cases the same individual owns both land and water; but usually farmers and irrigators have no property interest in the irrigation works. These are owned and operated by independent organizations who collect a revenue from the sale or rental of water.

3. **Value as an Investment.**—As an investment irrigation works are not always successful. There should be a ready market for the products of irrigation, and the value of land and water must not be so great as to materially reduce the profits derived from crops. The value of irrigation as an investment is especially dependent on the humidity of the climate. In the semi-humid region, where during occasional seasons the rainfall is sufficient to mature the crops, there is little or no demand for water furnished for irrigation, and no profit is derived from its sale. In the arid region, where crops cannot be raised without the aid of irrigation, the demand for water is constant. In the northern provinces of India water is in constant demand for irrigation and returns excellent profits. In Bombay and other places where the demand for water is intermittent, because the rainfall is frequently sufficient to mature crops, the construction of irrigation works has usually resulted in financial disaster. Perhaps the most important factor bearing on this subject in our own country is the degree of habitation. Nearly anywhere that a good market can be found and irrigation is essential to the production of crops, fair interest can be obtained on money invested in irrigation works. Many failures, however, have occurred, due chiefly to the lack of population and consequent lack of demand for water. Where all the water furnished is utilized the works almost invariably pay fair returns on the investment.

4. **Incidental Values.**—Not only is the direct money return from an irrigation investment to be considered, but there are several incidental means whereby profit may be derived from such investments. On broad principles of general government and policy the construction of irrigation works is of benefit to the whole country. They furnish homes and agricultural pursuits for many who must otherwise be idle or find less substantial support in other ways. Irrigation adds to the

general wealth of the country by increasing the amount of its agricultural products. It furnishes excellent investment for capital where the projects are well designed. It results in the conversion of barren and desert lands into delightful homes, and aids in the general development of the other resources of the region in which it is practised, as mining, lumbering, grazing, etc. One of the great advantages of irrigation is that it becomes practically an insurance on the production of crops. Its practice may not be necessary in the semi-humid or humid regions, but even there occasional droughts occur and crops are lost. Where an irrigation system exists in such cases, it will probably be called into requisition once or twice in the course of a year, and may save vast sums which would otherwise be lost by the destruction of crops.

5. **Cost and Returns of Irrigation.**—The following table compiled from the reports of the U. S. Census of 1890 gives an excellent idea of the extent and cost of irrigation, and of the value of the land and water after irrigation has been provided:

TABLE I.

EXTENT AND COST OF IRRIGATION.

States and Territories employing Irrigation.	Crop Irrigated. Acres.	Average Size of Irrigated Farm in Acres.	Average First Cost of Water per Acre.	Average Value of Water per Acre as estimated by Irrigators.	Average Annual Cost of Water per Acre.	Average Cost of Preparing Land for Cultivation per Acre.	Average Value of Land Irrigated per Acre.	Average Value of Products from Irrigated Land per Acre.
Total U. S.	3,564,416	67	$8.15	$26.00	$0.99	$12.12	$83 28	$14.89
Arizona	65,821	61	7.07	12.58	1.55	8.60	$48.68	$13.92
California	1,004,233	73	15.84	52.28	1.60	22.27	150.00	19.00
Colorado	890,735	92	7.15	28.46	.79	9.72	67.02	13.12
Idaho	217,005	50	4.74	13.18	.80	9.31	46.50	12.93
Montana	350,582	95	4.63	15.04	.95	8.29	49.50	12.96
Nevada	224,403	192	7.58	24.60	.84	10.57	41.00	12.92
New Mexico	91,745	30	5.58	18.30	1.54	11.71	50.98	12.80
Oregon	177,944	56	4.64	15.48	.94	12.59	57.00	13.90
Utah	263,473	27	10.55	26.84	.91	14.85	84.25	18.03
Washington	48,800	47	4.03	13.15	.75	10.27	50.00	17.09
Wyoming	229,676	119	3.62	8.69	.44	8.23	31.40	8.25

From this table it will be seen that while the average first cost of water, that is, the cost of constructing canals to bring the water to the land, is $8.15 per acre, the average value of water per acre as estimated by the owners after they obtain it is $26. This shows clearly the inherent value which the mere fact of possessing the water gives to it. In other words, the water is so scarce and valuable of itself as to increase by threefold the cost of making it available. The average value of the land before irrigation is from $2.50 to $5 per acre, while the same land after a water supply has been provided is valued at $83.28 per acre, and the products from this land have an average value of $14.89 per acre, which represents an unusually large interest on the money invested.

PART I.

HYDROGRAPHY.

CHAPTER II.

PRECIPITATION.

6. Relation of Rainfall to Irrigation.—In a region where the climate and soil are favorable for the production of agricultural crops the necessity of irrigation depends wholly on the amount of rainfall. The necessity of irrigation cannot be judged, however, from the total amount of precipitation in the year. Where the precipitation is less than 20 inches per annum in the United States, irrigation is generally supposed to be necessary, and our arid region is usually considered as including that portion of the country where the annual precipitation is below 20 inches. This, however, is not a safe gauge in all cases. Thus in Italy, where the annual precipitation averages perhaps 40 inches, irrigation is necessary, because most of this occurs during the winter months or at other times than in the agricultural or cropping season. In India the rainfall is in some places as high as 100 to 300 inches per annum. Yet nearly all of this occurs in one or two seasons of the year, and the actual rainfall during the winter months, when most of the cropping is done, may be as low as 5 to 10 inches. Generally speaking, the cropping season for our West may be taken as occurring between April and August inclusive, and these are among the dryest months in the year.

7. General Rainfall Statistics.—Tables II and III show in a general way the extent of precipitation over the arid region. From them it will be seen that the average annual rainfall over the northern portion of the Pacific Coast would be sufficient in amount for the production of crops, providing it fell during the irrigating season. There is also a small area near San Diego, and one on the headwaters of the Gila and Salt rivers in Arizona, where the annual rainfall is apparently sufficient for the maturing of crops. The amount of precipitation is greatly influenced by altitude. Thus in the same latitude in the region between Reno, Nevada, and San Francisco, California, the average annual precipitation in the bottom of the Sacramento Valley is about 15 inches. To the eastward of this the precipitation increases in amount with the height of the mountains until along their summits it averages from 50 to 60 inches. Still further east it decreases with the diminishing altitude until in Nevada the mean precipitation is from 5 to 10 inches. Everywhere throughout the West precipitation in the high mountains is much greater than in the adjacent low valley lands. As a result of this, while the rainfall is frequently insufficient to mature crops in the low lands and valleys, sufficient precipitation occurs in the mountains to furnish an abundant supply for the perennial discharge of streams or for the filling of storage reservoirs.

8. Rainfall Distribution in Detail.—In the lower Colorado and Gila river valleys in Arizona the average annual precipitation is between 4 and 6 inches. In the Gila and Salt river valleys in the neighborhood of Phœnix it is between 10 and 15 inches, while on the headwaters of these streams it averages 20 inches. In Northern Arizona the annual average precipitation is about 10 inches, most of which occurs in winter. During the summer or irrigating months the precipitation is from 1 to 3 inches in the lower Gila and Colorado river valleys, from 3 to 5 inches in the neighborhood of Phœnix and Florence, and about 5 inches in Northern Arizona.

In the lower Rio Grande and Pecos river valleys in New

Mexico the average annual precipitation is 10 inches. Over the remainder of the agricultural portion of the Territory it averages about 15 inches. In winter the precipitation is comparatively low in the valleys, but comparatively high in the uplands. In the summer or irrigating months it ranges between 4 and 8 inches in the Rio grande and Pecos valleys. In California in the Sacramento valley the annual average precipitation is about 15 inches, and in the San Joaquin valley from 10 to 15 inches. Over the agricultural portions of Southern California it averages about the same. A large proportion of this rainfall occurs during the early spring months, but in the latter portion of the irrigating season the rainfall diminishes very rapidly, averaging from May till October scarcely two inches in the Sacramento valley and less than an inch in the San Joaquin valley and in Southern California. Over the plains of Western Nevada the average annual precipitation is between 5 and 10 inches, most of which occurs at periods other than in the irrigating season. On the plains of Utah the annual average precipitation is from 10 to 15 inches, while the precipitation during the summer months is but an inch or two.

In the upper Missouri and Yellowstone valleys and other principal agricultural portions of Montana the average annual precipitation is from 12 to 20 inches, of which about 5 inches falls during the irrigating season. In the Snake River valley of Idaho the average annual precipitation is about 10 inches, of which about 3 inches falls during the irrigating season. In the Platte and Arkansas valleys of Colorado the average annual precipitation is about 15 inches, of which from 7 to 10 inches fall during the irrigating season. In the eastern portion of Colorado on the plains nearer the Kansas line the precipitation is a little less than this and about the same as in the upper Rio Grande valleys.

9. **Great Rainfalls.**—One of the important considerations in designing irrigation projects, and especially storage reservoirs, is the maximum amount of rainfall which may occur. Great floods are the immediate result either of the sudden melting

of snow in the mountains or of heavy and protracted rainstorms. In most of the river valleys just considered there are periods of extreme or maximum rainfall, the recurrence and effect of which are worthy of note. In the neighborhood of Yuma, Arizona, the average annual rainfall is about 3 inches, yet in the last week of February, 1891, $2\frac{1}{2}$ inches fell in 24 hours. The average annual rainfall in the neighborhood of San Diego, California, is about 12 inches, yet in the storms of February, 1891, 13 inches fell in 23 hours and $23\frac{1}{2}$ inches in 54 hours. In the neighborhood of Bear Valley reservoir east of Redlands, California, during the same storm 17 inches of rain fell in 24 hours. Such storms as these may be very destructive both to crops and works. The average annual discharge of Salt River in Arizona is about 1000 second-feet, and the average flood discharge is perhaps 10,000 second-feet; yet, as the result of a sudden rainstorm of unusual violence which occurred in the spring of 1891, this river increased to a flood discharge of 140,000 second-feet, and in the spring of 1892, as the result of a still greater cloud-burst, its discharge reached the enormous figure of nearly 350,000 second-feet. Over certain portions of the western region these sudden cloud-bursts are of not uncommon occurrence and must be provided for in the construction of works.

10. Suddenness of Great Storms.—Statistics showing the rainfall in 24 hours are often insufficient to give a safe and correct estimate of the suddenness and danger of floods resulting from great storms. The greatest and most sudden storm on record is probably that which occurred on the line of the Lower Ganges canal in the Northwest Provinces of India. 30th On the 13th of September, 1884, 16 inches fell; on October the 1st 22 inches, on the 2d $22\frac{1}{2}$, on the 3d 18 inches, and on the 4th $17\frac{1}{2}$ inches of rain fell. In some cases and at some times the precipitation was as high as 5 inches per hour. In some of the cloud-bursts in our own West it is not unlikely that the precipitation has reached from 3 to 5 inches per hour. Such storms as these do far greater damage than protracted storms of less violence.

11. Precipitation on River Basins.—The following table of rainfall on a few of the principal river basins of the West shows very clearly the variation in the amount of precipitation at different altitudes:

TABLE II.

PRECIPITATION BY RIVER BASINS.

Station.	Altitude. Feet.	Mean Annual Precipitation. Inches.
RIO GRANDE RIVER:		
Summit, Colorado	11300	29.00
Fort Lewis, Colorado	8500	17.19
Fort Garland, "	7937	12.74
Saguache, "	7740	42.60
Santa Fé, New Mexico	7026	14.69
Fort Wingate, New Mexico	6822	14.71
Las Vegas, " "	6418	22.08
Albuquerque, " "	5032	7.19
Socorro, " "	4560	8.01
Deming, " "	4315	8.95
GILA RIVER:		
Fort Bayard, New Mexico	6022	14.72
Prescott, Arizona	5389	17.06
Fort Apache, Arizona	5050	21.04
Fort Grant, "	4914	16.65
Phœnix, "	1068	7.38
Texas Hill, "	353	3.47
Yuma, "	141	2.81
PLATTE RIVER:		
Pike's Peak, Colorado	14134	28.65
Fort Saunders, Wyoming	7180	12.92
Fort Fred Steele, "	6850	11.03
Cheyenne, "	6105	11.32
Colorado Springs, Colorado	6010	14.79
Denver, "	5241	14.32
Fort Morgan, "	4500	8.08
MISSOURI RIVER:		
Virginia, Montana	5480	16.00
Fort Ellis, "	4754	19.60
Helena, "	4266	14.26
Fort Shaw, "	2550	10.22
Poplar, "	1955	10.50

12. Rainfall Statistics by States.—Table III gives the average annual precipitation, and the precipitation during the irrigating season, from April to August inclusive, for various places in each of the Western States:

TABLE III.

PRECIPITATION BY STATES.

Locality.	Altitude. Feet.	Mean Annual Precipitation. Inches.	Mean Precipitation, April to August. Inches.
ARIZONA:			
Fort Apache	5050	21.04	10.27
Holbrook	5047	9.29	3.68
Casa Grande	1398	4.28	1.32
Phœnix	1068	7.38	2.27
Texas Hill	355	3.47	.66
Prescott	5389	17.06	7.94
NEW MEXICO:			
Springer	5766	11.82	8.86
Las Vegas	6418	22.08	12.70
Albuquerque	5026	7.19	4.22
Santa Fé	7026	14.68	8.32
Fort Wingate	6822	14.71	6.97
Socorro	4565	10.31	3.87
Deming	4327	8.95	3.90
CALIFORNIA:			
Yreka	2635	16.34	3.33
Fort Bidwell	4640	20.84	4.54
Redding	556	34.60	5.61
Oroville	188	25.14	3.48
Bowman Dam	5400	71.21	....
Summit	7017	43.56	....
Placerville	2110	45.17	8.26
Sacramento	64	19.80	2.73
San José	94	14.52	2.08
Merced	171	10.30	1.73
Fresno	328	9.02	1.80
Visalia	348	8.84	1.86
San Bernardino	950	17.16	2.37
Banning	2317	14.39	1.80
Los Angeles	330	18.31	1.81
San Diego	93	9.86	2.47
Yuma	276	3.16	1.06
NEVADA:			
Reno	4497	5.17	0.71
Winnemucca	4358	8.61	2.70
Palisade	4840	8.42	2.17
Fort Churchill	4284	5.31	1.70
Carson	4628	11.25	2.05
Pioche	6110	11.19	4.41
RADO:			
Greeley	4750	13.41	9.16
Breckenridge	9524	28.25	....
eadville	10200	11.56	....
ike's Peak	14134	28.65	....
Canyon City	4700	11.52	7.01
Pueblo	4753	9.87	7.10
Fort Lyon	4000	11.07	8.15
Monte Vista	7765	6.91	4.18
Trinidad	6070	21.61	15.06
Denver	5241	14.32	9.00

TABLE III.—*Continued.*

Locality.	Altitude. Feet.	Mean Annual Precipitation. Inches.	Mean Precipitation, April to August. Inches.
UTAH:			
Ogden	4340	13.46	4.12
Salt Lake	4354	16.85	6.26
Nephi	5550	18.19	7.40
St. George	2880	6.74	1.32
IDAHO:			
Eagle Rock	4781	18.67	4.69
Boisé	1198	14.74	4.11
Lewiston	....	18.25	5.55
Fort Hall	....	17.51	6.44
WYOMING:			
Cheyenne	6105	1.32	5.55
Fort McKinney	....	9.60	4.45
MONTANA:			
Fort Benton	2730	13.30	5.45
Miles City	4372	12.00	5.55
Helena	4266	14.26	4.48
Fort Shaw	2550	10.22	4.25

13. **Gauging Rainfall.**—The common rain-gauge or pluviometer generally employed in this country in the measurement of precipitation is illustrated in Fig. 1. It consists of

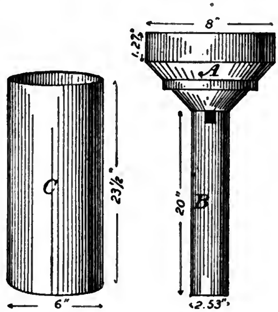

FIG. 1.—RAIN-GAUGE.

three parts, the collector *A*, the receiver *B*, and the overflow attachment *C*. A measuring-rod graduated to inches and tenths is furnished with each gauge and is used in measuring the depth of water. This gauge should be placed in an open space, preferably over grass sod, and, to obtain a free exposure to the rain, should be at least 30 feet from any building or obstruction. It should be enclosed in a close-fitting box and sunk into the ground to such a depth that the upper rim of the gauge shall be about one foot above the surface, and care should be taken to maintain it in a horizontal position. The sectional area of the receiver being only .1 of the area of the collector, the depth of water measured is ten times the true rainfall.

In the measurement of snowfall the funnel and receiver should be removed and only the overflow attachment used as the collecting vessel. It should be set as in the case of rainfall and the snow should be melted after being collected. Where the wind is blowing hard it is advisable to measure the snow in a different manner. After the snow has ceased to fall a spot should be selected where it has an average depth. The overflow attachment is inverted and lowered until the rim has reached the full depth of the newly-fallen snow, when a piece of flat tin or other material is slipped under the rim and the gauge lifted and the snow melted as before.

14. **Works of Reference.**—For fuller information consult:

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CHAPTER III.

EVAPORATION AND ABSORPTION.

15. Evaporation Phenomena.—The rapidity with which water, snow and ice are converted into vapor is dependent upon the relative temperatures of the water and atmosphere and upon the amount of motion in the latter. Evaporation is greatest when the atmosphere is dryest, when the water is warm and a brisk wind is blowing. It is least when the atmosphere is moist, the air quiet and the temperature of the water low. In summer the cool surfaces of deep waters condense moisture from the warm air passing across them and thus gain in moisture when they are supposed to be evaporating. When the reverse conditions exist in the atmosphere and the winds are blowing briskly across the water the resultant wave-motion increases the agitation of the body and permits its vapors to escape freely into the large volumes of unsaturated air which are rapidly presented in succession to attract its vapors. Evaporation is constantly taking place at a rate due to the temperature of the surface and condensation is likewise going on from the vapors existing in the atmosphere, the difference between the two being the rate of evaporation.

From the above it will be seen that evaporation should be greatest in amount in the desert regions of the Southwest and least in the high mountains. Tables IV and V show this to be the case and that in the same latitude evaporation differs greatly in amount according to the altitude.

16. Measurement of Evaporation.—Two or three methods have been devised for measuring evaporation none of which are wholly satisfactory. Elaborate and expensive apparatus has been employed in evaporation measurements made by Mr. Desmond Fitzgerald, chief engineer of the Boston Water Works; by Mr. Charles Greeves of England, and others. A simple apparatus and one which is as successful as most of the more elaborate contrivances is that employed by the U. S. Geological Survey. It consists of a pan, Fig. 2, so

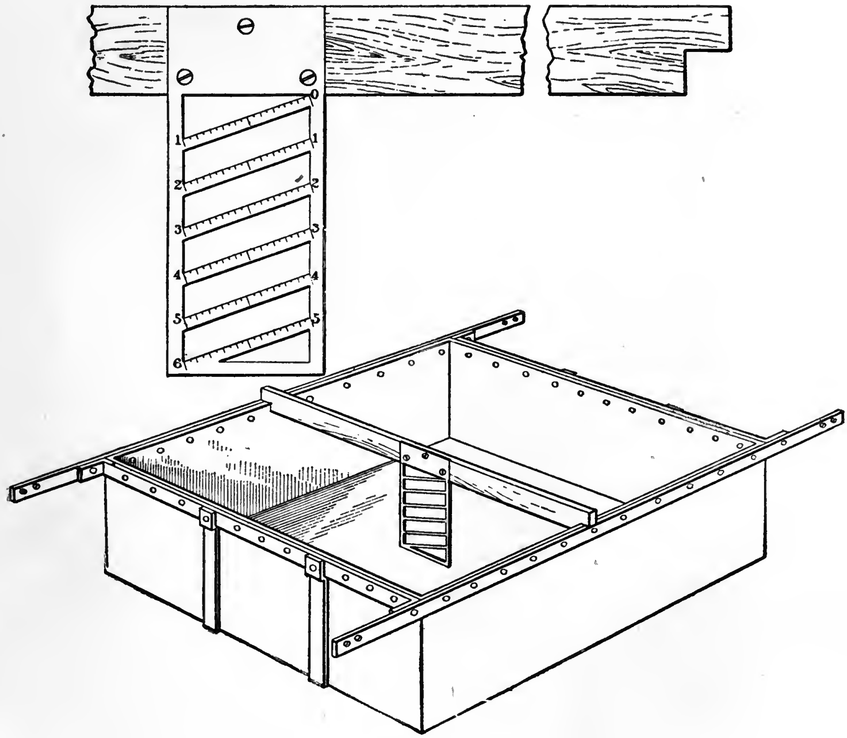

FIG. 2.—EVAPORATING-PAN.

placed that the contained water has as nearly as possible the same temperature and exposure as that of the body of water the evaporation from which is to be measured. This evaporating pan is of galvanized iron 3 feet square and 10 inches deep, and is immersed in water and kept from sinking by means of floats of wood or hollow metal. It should be placed in the canal, lake, or other body the evaporation of which it is intended to measure in such position as to be exposed as nearly as possible to its average wind movements. The pan must be filled to within 3 or 4 inches of the top in order that the waves produced by the wind shall not cause the water to slop over, and it should float with it srim several inches above the surrounding surface, so that waves from this shall not enter the pan. The device for measuring the evaporation consists of a small brass scale hung in the centre of the pan. The graduations are on a series of inclined crossbars so proportioned that the vertical heights are greatly exaggerated, thus permitting a small rise or fall, say of a tenth of an inch, to cause the water surface to advance or retreat on the scale .3 of an inch. By this device, multiplying the vertical scale by three, it is possible to read to .01 of an inch.

In 1888 a series of observations were made with the Piche evaporometer by Mr. T. Russell of the U. S. Signal Service to ascertain the amount of evaporation in the West. While it is probable that results obtained with this instrument are not particularly accurate, comparisons of these results with those obtained by other methods in similar localities show such slight discrepancies that they may be considered of value until superseded by results obtained by other and better methods. Observations were made with this instrument in wind velocities varying from 10 to 30 miles per hour, from which it was discovered that with a velocity of 5 miles an hour the evaporation was 2.2 times that from one in quiet air; 10 miles per hour 3.8 times; 15 miles 4.9; 20 miles 5.7 times; 25 miles 6.1, and 30 miles 6.3 times.

17. Amount of Evaporation.—In Table IV is given the amount of evaporation by months in the year 1888 in various sections of the West as derived from experiments with the Piche apparatus.

As in the case of precipitation, evaporation decreases with the altitude because of the diminished temperature in high mountains. Experiments were made to determine the amount

TABLE IV.

DEPTH OF EVAPORATION, IN INCHES PER MONTH, IN 1887–88.

Stations and Districts.	Jan., 1888.	Feb., 1888.	March, 1888.	April, 1888.	May, 1888.	June, 1888.	July, 1887.	August, 1887.	Sept., 1887.	Oct., 1888.	Nov., 1887.	Dec., 1887.	Year.
NORTHERN SLOPE:													
Fort Assiniboine	0.8	1.2	1.2	3.1	4.1	4.2	6.8	5.5	4.8	3.5	2.5	1.1	39.5
Fort Custer	0.6	1.5	1.3	5.4	6.8	4.9	9.6	8.0	6.1	3.4	2.9	1.5	52.0
Fort Maginnis	1.1	1.4	1.1	3.3	3.2	4.6	6.8	4.6	3.8	2.8	2.0	1.1	35.8
Helena	1.1	3.6	2.1	6.1	4.3	5.5	7.2	7.7	6.4	4.3	3 0	2.1	53.4
Poplar River	0.4	0.8	0.8	2.7	4.9	5.7	6.0	4.8	4.4	2.5	1.7	0.7	35.4
Cheyenne	3.3	5.7	4.0	8.2	5.2	10.4	8.0	7.7	8.6	5.8	6.1	3.5	76.5
North Platte	0.8	1.8	1.8	5.4	3.9	6.9	6.0	4.8	3.7	2.8	2.3	1.1	41.3
MIDDLE SLOPE:													
Colorado Springs	3.0	3.3	4.1	6.7	5.6	4.3	6.7	7.2	6.8	4.6	4.2	2.9	59.4
Denver	2.8	3.7	3.5	7.6	5.8	10.5	8.3	8.5	6.1	4.9	4.2	3.1	69.0
Pike's Peak	2.1	1.3	1.5	2.1	1.8	1.9	3.0	4.0	3.0	2.3	2.8	1.0	26.8
Concordia	1.3	2.8	1.8	4.8	4.3	5.7	7.3	5.2	4.3	4.5	3.4	1.8	47.2
Dodge City	1.4	2.4	2.8	4.1	4.6	7.4	8.3	6.6	5.5	5.2	4.2	2.1	54.6
Fort Elliott	1.3	1.9	3.2	5.1	5.4	8.2	7.6	6.2	5.4	4.7	4.2	2.2	55.4
SOUTHERN SLOPE:													
Fort Sill	1.6	2.0	2.6	3.8	4.0	4.4	4.8	7.5	5.1	4.2	4.1	2 0	46.1
Abilene	1.8	1.7	3.1	4.2	5.0	5.8	9.5	7.5	6.2	4.5	3.4	1.7	54.4
Fort Davis	5.4	5.7	6.7	8.5	11.0	12.0	11.4	9.0	5.9	5.2	5.7	4.9	96.4
Fort Stanton	3.9	3.9	5.2	7.3	9.5	10.9	9.4	11.6	3.9	4.0	3 6	3.8	76.0
SOUTHERN PLATEAU:													
El Paso	4.0	3.9	6 0	8.4	10.7	13.6	9.4	7.7	5.6	5.2	4.6	2.9	82.0
Santa Fé	3.0	3.4	4.2	6.8	8.8	12 9	9 2	9.8	6.6	6.7	5.7	2.7	79.8
Fort Apache	2.6	3.0	3.6	6.8	9.4	9.1	7.1	6.7	5.3	5.2	4.1	2.6	65.5
Fort Grant	5.2	4.8	6.4	9.2	10.2	13 8	12.4	10.5	9.0	7.9	7.2	4.6	101.2
Prescott	1.4	2.8	3.6	5.4	6.2	8.1	6.6	6.5	4.7	4.9	3.6	2.2	56.0
Yuma	4.4	5.2	6.6	9.6	9 6	12.6	11.0	10.2	8.2	8.2	5.5	4.6	95.7
Keeler	3.0	4.6	6.3	8.7	9.3	11.9	12.8	13.9	10.6	8.8	5.9	4.8	100.6
MIDDLE PLATEAU:													
Fort Bidwell	0.8	1.8	1.8	4.6	5 2	4.0	8.8	8.1	5.0	4.6	2.4	1.3	48.9
Winnemucca	0.9	2.8	6.2	9.1	9.3	10.1	11.5	12.0	9.9	6.6	3.7	1.8	83.9
Salt Lake City	1.8	2.7	3.6	7.2	6.9	8.9	9.2	10.7	9.6	6.5	5.0	2.3	74.4
Montrose	1.8	2.7	3.7	6.2	7.0	11.1	10.2	8.3	6.9	5.2	3.4	2.0	68.3
Fort Bridger	1.6	2.5	2.7	4.3	4.3	6.5	7.7	6.8	5.6	4.2	5.2	4.7	56.1
NORTHERN PLATEAU:													
Boisé City	1.6	2.5	3.8	6.1	6.5	6.6	10.0	9.2	7.4	5.2	3.2	1.8	63.9
Spokane Falls	0.7	1.7	2.7	4.4	5.4	4.4	7.7	6.4	3.8	2.5	1.7	1.4	42.8
Walla Walla	1.1	2.9	3.6	6.2	7.7	5.7	9.9	7.9	5.1	3.4	1.8	2.4	57.7
NORTH PACIFIC COAST:													
Fort Canby	1.2	1.1	1.8	2.1	2.8	2.3	1.8	2.9	1.8	1.8	1.5	0.9	21.1
Olympia	1.3	1.2	1.8	2.5	4.1	3.3	3.2	3.1	2.4	1.5	1.3	1.1	26.8
Tatoosh Island	1.2	1.1	1.8	1.4	1.8	1.8	1.4	1.4	1.4	1.6	1.8	1.4	18.1
Roseburg	1.2	1.6	2.7	3.9	4.7	3.5	5.4	4 7	5.0	3.2	1.7	1.6	39.2
MIDDLE PACIFIC COAST:													
Red Bluffs	3.0	4.6	5.4	6.1	7.0	6.9	11.0	10.7	10.1	10.5	5.9	3.6	84.8
Sacramento	1.8	3.1	3.7	4.3	4.2	5.6	5.9	5.6	6.5	7.3	3.9	2.4	54.3
SOUTH PACIFIC COAST:													
Fresno	1.8	2.8	3.0	5.6	6.0	7.0	9.1	10.2	7.6	6.7	3.8	2.2	65.8
Los Angeles	2.3	2.0	2.8	3.4	3.0	3.8	3.2	3.5	3.1	4.1	3.0	3.0	37.2
San Diego	2.9	2.7	2.5	2.7	3.3	2.8	3.2	3.3	2.9	4.3	3.2	3.7	37.5

of evaporation in different portions of the West by the hydrographers of the U. S. Geological Survey. These were made with the evaporating pan, and the results are probably more reliable than those obtained with the Piche instrument. These experiments were unfortunately conducted for a relatively short space of time. From a comparison of a few of these which are complete it will be seen that they agree well with the results given by Mr. Russell's observations.

TABLE V.

DEPTH OF EVAPORATION PER MONTH, IN INCHES.

Year.	Place.	Annual.	Jan.	Feb.	March.	April.	May.	June.	July.	August.	Sept.	Oct.	Nov.	Dec.
1889	Bozeman, Mont.									3.4	4.5	5.3	1.9	
1890	" "							2.6						
1889	Great Falls, "											2.7	1.0	
1889	Springdale, "									6.8	7.1	3.1	2.9	
1889	Hogan, "										6.1			
1889	Fort Douglas, near Salt Lake City, Utah									10.5	5.7	4.9	1.0	
1890	"	34.9				3.7	4.1	5.1	7.6	6.5	4.6	2.1	1.2	
1891	"	36.4				3.2	4.8	5.2	7.6	6.5	5.2	2.5	1.4	
1889	Nephi and Provo							3.9	5.0	4.6	2.9	3.3		
1889	Cherry Creek, Col							8.1	7.9	8.6	6.2	4.2	2.5	
1889	Canyon City, "									7.1		3.6		2.2
1890	" " "				3.8	4.8	5.2	7.3	6.0					
1889	Lamar, Col										7.2			
1889	Embudo, New Mexico	3.0	2.9	3.6	4.9									
1889	Fort Bliss, near El Paso, Texas						10.9	10.7	9.6	11.4	9.2	6.8	4.6	2.9
1890	"	80.7	2.0	2.0	7.0	7.3	10.8	11.7	9.6	7.6			3.7	3.0
1891	"		2.7	2.9	5.5	7.4								
1889	Tempe, Ariz.								13.7	14.1	11.0	6.4	4.4	
1890	" "					5.8	5.5	5.6	6.6		5.8	5.2	4.6	3.2
1891	" "	85.5	3.9	3.6	3.7	4.2								
1890	Florence, "				5.8	8.2	11.5	13.5						
1889	Yuma, "												2.7	1.8
1890	" "		2.0	2.8				7.2	8.5	7.2	7.1	4.3	3.6	2.5
1890	Bloods, Cal.									7.9				
1890	Lake Eleanor, Cal.									7.2				
1890	Tuolumne Mead, Cal.										5.9			
1890	Lake Tenaiya, "										5.7			
1890	Little Yosemite "										6.2			

18. Evaporation from Snow and Ice.—From some experiments conducted at the Boston Water Works the amount of evaporation from snow and ice was found to be greater than is generally believed. From snow it amounted to about .02 of an inch per day, or nearly $2\frac{1}{2}$ inches in an ordinary season. From ice it amounted to .06 inch per day, or about 7 inches in an ordinary season. The evaporation from snow is greater than this in the arid regions of the West, especially on barren mountain-tops such as those in Arizona, Nevada, and Utah, where they are exposed to the wind and the bright sunshine.

19. Evaporation from Earth.—The amount of evaporation from earth in the West is a doubtful quantity. The most important experiments bearing on this were made in England between 1844 and 1875. From these it appears that the amount of evaporation from ordinary soil is about the same as that from water, sometimes exceeding it a little and sometimes being a trifle less, though generally averaging about 3 inches less than the corresponding evaporation from water surfaces. The evaporation from sandy surfaces was found to be only about one-fourth to one-fifth that from water. Thus in the observations of 1873, where the mean evaporation from water was 20.4 inches, that from earth was 19.7 inches and from sand 3.7 inches.

20. Effect of Evaporation on Water Storage.—The value of water storage for irrigation in the West is realized chiefly between May and August inclusive. The only loss due to evaporation which practically affects the amount of storage water is that occurring during these months. Little or no rain falls in the arid region during this period, so that comparatively little of the loss of evaporation is replaced by rain. As an example, take Central California, where the average rainfall during these months amounts to a trifle less than 1 inch. The evaporation during the same period amounts to about 21 inches. The total resultant deficiency chargeable to evaporation is about 20 inches.

21. Percolation and its Amount.—The losses due to percolation in canals and storage reservoirs are very considerable, and added to those due to evaporation they increase the total loss by from 25 to 100 per cent according to the character of the soil. It is difficult to ascertain the losses due to percolation alone. For this reason it is desirable to consider losses from percolation and evaporation together and include them under the joint head of "absorption."

From the experiments previously alluded to which were conducted by Mr. Greaves in England, it was found that while the evaporation from earth during the period of 23 years was 73.4 per cent of the rainfall, the percolation was but 26.6 per cent. From sand this percentage was nearly reversed, the loss by percolation being about 30 inches, while the loss by evaporation was but 7 inches. There was no loss from percolation at all for several consecutive months. As an average year take that of 1872, when the rainfall amounted to 23.8 inches and the evaporation from water 20.4 inches, the losses by percolation amounted to 4 inches in earth and 20.1 inches in sand. From observations and experiments made in Bavaria it appeared that in the warm summer months whereas the depth of percolation on open bare ground was 11 per cent of the rainfall, in forests it amounted to as high as 36 per cent of the rainfall. In our West these quantities will be materially different. The amount of rainfall is relatively small on the ordinary mountain catchment basin. The slopes are steep and generally rocky. As a result of this the percentage of percolation will be low, the amount of runoff being relatively higher. Where there are dense forests, the soil beneath which is covered with a depth of litter, or where the slopes are low, the percentage of percolation will be relatively high.

22. Absorption.—As here considered, absorption is the resultant or total loss due to the combined action of evaporation and percolation. From experiments made in India, where the climate is somewhat similar to our western country, it was found that the loss by evaporation on a canal of about 30 miles in length would be a little over 2.5 second-feet, or about 5 per cent of the probable discharge. As this amount is comparatively small, it appears that the greater portion of the loss is from percolation. Mr. Beresford argues that the losses by percolation are due to capillary attraction and the action of gravity. The latter takes place only through coarse sand or gravel, while the former is a more complicated process acting where the particles are fine and in close contact one with the other. Capillary attraction stops where the absorbing medium is limited, for as soon as water which has been carried by its action through a bank reaches the outer surface, percolation ceases and evaporation comes into play. It is for this reason that banks of sand even when well rammed will retain water. The more extensive the absorbing medium the greater the losses from this cause; but if its extent be limited by a bed of clay placed under either the reservoir or canal in which percolation occurs, then the losses due to this cause are rapidly diminished in quantity. The layer next the wetted perimeter limits the quantity absorbed, and the greater its area the more will it pass through to the still greater area of the next layer; hence percolation varies as the wetted perimeter.

23. Amount of Absorption in Reservoirs and Canals.—The volume of this is very difficult to ascertain and varies greatly with soil and climate. If the bottom of the reservoir is composed of sandy soil, the losses from percolation and evaporation combined will be about double those from the former alone. Whereas, if the bottom of the reservoir be of clayey material, or if the reservoir be old and the percolation limited by the sediment deposited on its bottom, this loss may be considerably less than that of evapoiation.

On a moderate-sized canal in India the total losses due to absorption have been found to amount to about one second-foot per linear mile. In new canals these losses are greatest. If the soil is sandy, the losses on new canals may amount for long lines to from 40 to 60 per cent of the volume entering the head. In shorter canals the percentage of loss will be proportionately decreased, though they will rarely fall below 30 per cent in new canals of moderate length. As the canal increases in age the silt carried in suspension will be deposited on its banks and bottom, thus filling up the interstices and diminishing the loss. In old canals with lengths varying between 30 and 40 miles the loss may be as low as 12 per cent in favorable soil, though in general for canals of average length the loss will be about 20 to 25 per cent of the volume entering the head. On the Ganges Canal in India, the length of which is several hundred miles, the losses in some years have been as high as 70 per cent.

24. Prevention of Percolation.—An excellent method for the reduction of the loss by percolation is that recommended by Mr. J. S. Beresford of India, who advises that pulverized dry clay be thrown into the canals near their headgates. This will be carried long distances and deposited on the sides and bottom of the canal, forming a silt berme. The losses by absorption are greatly increased by giving the canal a bad cross-section. Thus depressions along the line of a new canal are often utilized to cheapen construction by building up a bank on the lower side only, thus allowing the water to spread and consequently increasing the absorption. The least possible wetted perimeter and the least surface exposed to the atmosphere will cause the least loss from this cause.

25. **Seepage Water.**—In many instances where canals and reservoirs are bordered by steep hillsides the amount of water lost may prove to be much less than would be expected. This is due to the fact that large amounts of seepage water may enter the canal or reservoir from the surrounding country and thus replenish to a large extent the losses from absorption.

Before irrigation becomes universal in any locality it is frequently impossible to derive any water from wells. The subsurface water level may be situated at a great depth below the surface. After irrigation has been practised for some time, however, the soil becomes filled with water and the subsurface level rises so that shallow wells often yield persistent supplies. In portions of California, especially in the neighborhood of Fresno where the subsurface water level was originally from 60 to 80 feet below the surface, wells 10 and 15 feet in depth now receive constant supplies, the result of seepage from the canals. Water used in irrigating is in large part returned to the drainage channels and can be again diverted for irrigation. On the Cache la Poudre Creek in Colorado experiments made in 1889 showed that while the original discharge in the canyon was 127.6 second-feet, the volume at a point considerably lower down on the stream had increased to 214.5 second-feet after supplying fifteen canals and without receiving additional naturald rainage; an addition of more than two-thirds of the original volume, available to supply canals lower down. Measurement of the volume of water in the Sweetwater reservoir in Southern California shows that after water ceases to be drawn out of the reservoir in the fall, it begins to fill up while no water is entering it from the streams. This proves that seepage from the hillsides add to the volume in the reservoir faster than water was lost by absorption.

CHAPTER IV.

RUNOFF AND FLOW OF STREAMS.

26. Runoff.—By "runoff" is meant the quantity of water which flows in a given time from the catchment basin of a stream. It includes not only that portion of the rainfall which flows over the surface during storms, but also water which is derived from subsurface sources, as springs, etc. The runoff of a given catchment area may be expressed either as the number of second-feet of water flowing in the stream draining that area, or it may be expressed as the number of inches in depth of a sheet of water spread over the entire catchment. The latter expression indicates directly a percentage of rainfall in inches which runs off. Finally, runoff may be expressed volumetrically as so many cubic feet or acre-feet.

27. Variability of Runoff.—As runoff bears a direct relation to precipitation, it appears that, knowing the amount of rainfall and the area of the catchment basin, the amount of runoff can be directly ascertained. This is not the case, however, as the amount of runoff is affected by many varying climatic and topographic factors. Many formulas, none of which give satisfactory results, have been worked out for obtaining the relation between runoff and precipitation. If the climate be the same over two given catchment basins, the runoff will be affected by the depth of the soil, the amount of vegetation, the steepness of the slopes, and the geologic structure.

The climatic influences bearing most directly on runoff are the total amount of precipitation, its rate of fall, and the temperature of air and earth. Thus, where most of the precipitation occurs in a few violent showers the percentage of runoff is higher than where it is given abundant time to enter the soil. If the temperature is high and the wind strong, much greater loss will occur from evaporation than if the ground is frozen and there is no air movement. Within a given drainage basin the rates of runoff vary on its different portions. Thus in a large basin the rate of runoff for the entire area may be low if the greater portion of the basin is nearly level, but at the head-waters of the streams where the slopes are steep and perhaps rocky the rate of runoff will be higher. The coefficient of run-off increases with the rainfall. Thus in humid regions where the rainfall is greatest the rate of runoff is highest.

28. Formulas for Runoff.—Several formulas for ascertaining the percentage of runoff or the quantity of discharge from a given catchment basin have been obtained both empirically from known measurements and by theoretic processes. Mr. J. T. Fanning found by plotting a curve derived from the flood discharges of some American streams that the resulting equation for flood flow became

$$D = 200(M)^{\frac{5}{6}}, \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (1)$$

in which M is the area of watershed in square miles, and D the volume of discharge of the whole area in second-feet.

In India Colonel Ryves derived the following formula for runoff,

$$D = C\sqrt[3]{M^2}, \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (2)$$

and Colonel Dickens the formula

$$D = C\sqrt[4]{M^3}. \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (3)$$

No such formulas can be strictly applied with the same coefficient to areas of varying size, and all must be used with discretion, as their results are greatly influenced by different conditions from those under which they were obtained. In regions where maximum recorded rainfalls of from 3 to 6 inches in 24 hours have occurred the following coefficients have been determined:

Rainfall 3.5 to 4 inches in flat country, $C = 200$; mixed country, $C = 250$; hilly country, $C = 300$; and for a maximum rainfall of 6 inches, C varies between 300 and 350. For Ryves' formula the coefficient varies between 400 and 500 in flat country, and for hilly areas where the maximum rainfall is high it may reach 650. The shape of the catchment basin is an important factor in the formula of maximum discharge.

29. Examples of Runoff.—On the headwaters of the Arkansas River in Colorado, at altitudes varying between 7000 and 14,000 feet, the depth of runoff varies between 20 and 50 inches. On the Arkansas basin above Canyon City the runoff averages 18 inches. In the Sierras in Western Nevada, on the headwaters of the Truckee and Carson rivers, the runoff ranges between 25 and 45 inches in depth, while the average runoff over larger catchment areas on these streams, above Reno and Genoa, varies between 14 and 25 inches. In nearly every case the depth of runoff is about 60 per cent of the precipitation. In Arizona the slopes are more abrupt and barren; yet, as the rainfall is less regular and very much less in amount, the volumes of runoff are much smaller. On the upper Gila River basin the total depth of runoff in 1890 for 15,000 square miles of catchment basin was 0.45 of an inch, the discharge amounting to 0.35 second-feet per square mile of catchment area. On the upper Salt River basin above Phœnix the depth of runoff in 1890 was 4.2 inches and the discharge of the stream 3.7 second-feet per square mile of catchment area. In Montana, on the headwaters of the Gallatin and Madison rivers, the total annual depth of runoff averages from 10 to 14 inches, the discharge varying between 10 and 14 second-feet per square mile of catchment area. In the winter it is as low as 0.4 second-foot, and in May and June as high as 3 second-feet. On the Rio Grande basin above Del Norte, Colorado, the in 1890 was annual runoff amounts to about 10 inches in depth or to 10.5 second-feet, while on the entire basin of the Rio Grande above El Paso the runoff amounts to but 0.5 second-feet per square mile of catchment area. On the Bear River at Colliston, Utah, the annual depth of runoff is about 6.6 inches, and the discharge 6 second-feet per square mile. On the Provo River above Provo, Utah, the runoff amounts to 10.5 second-feet of discharge per square mile. On the Snake River above Eagle Rock, Idaho, the average annual runoff is 14 inches in depth or 10 second-feet per square mile of catchment area.

30. Flood Discharges of Streams.—It is desirable to know the monthly and daily rates of runoff as well as the mean annual runoff of a catchment basin. This is necessary in order that dams and weirs may be provided with ample wasteways. The greatest floods occur either on barren catchment basins having steep slopes or where heavy snowfalls are followed by warm, melting rains. On the Gila and Salt river basins in Arizona the percentages of runoff are exceptionally high during occasional severe storms. The highest recorded flood on the Salt River above Phœnix occurred in February, 1891, and amounted to about 350,000 second-feet from a catchment basin of 12,260 square miles. This is equivalent to nearly 30 second-feet per square mile of catchment area, while the stream a few days prior to the occurrence of the storm was not discharging over 1000 second-feet, or one-twelfth of a second-foot per square mile.

31. Discharge in Seasons of Minimum Rainfall.—Where the number of storage basins is limited it becomes desirable to save all of the water possible and frequently to impound enough to carry over a period of two or three years of minimum rainfall. In general it has been found that cycles of mean low rainfall occur every two or three years when the amount of precipitation is less than 0.8 of the mean. The least of these three-year low cycles has been found to average as low as 0.7 of the mean annual rainfall.

32. Regimen of Western Rivers.—The Eastern rivers usually drain comparatively level catchment basins, well covered with timber and grass. As a result of this the soil is deep and the rate of runoff is consequently low and the streams are comparatively constant in their discharge, being subject to few and not excessive flood rises. This is because the larger portion of the water reaches these streams from subterranean sources by seepage. In the more arid portions of the West the regimen of the streams is the reverse of this. The catch-

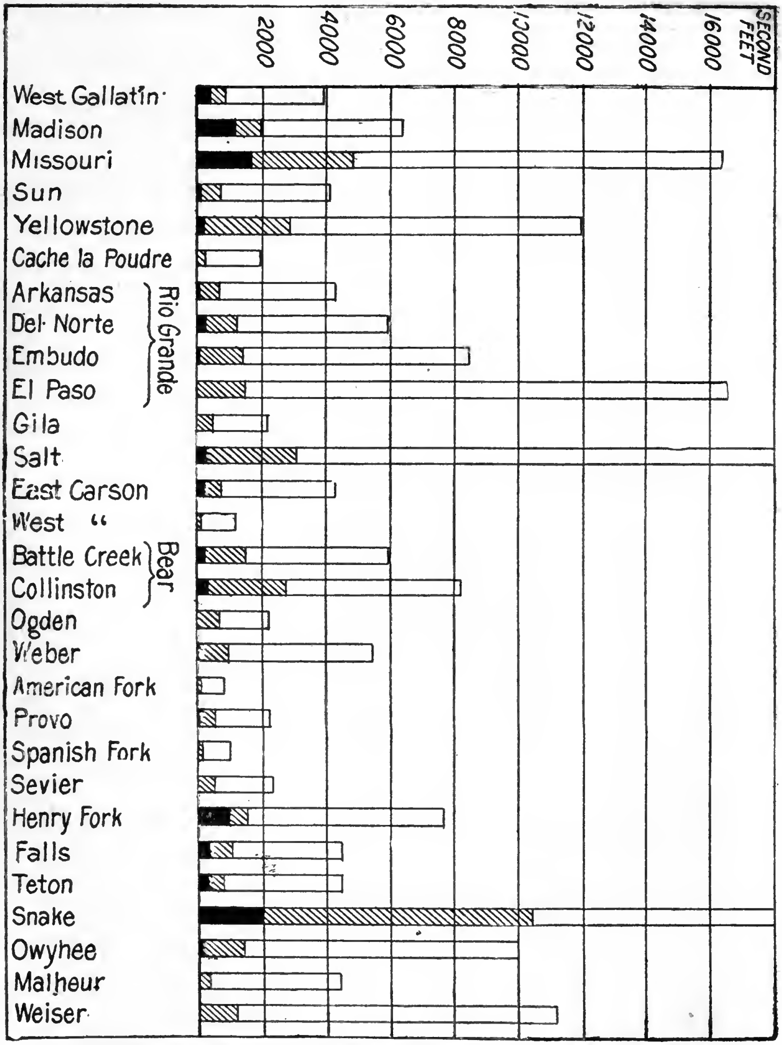

FIG. 3.—MAXIMUM, MINIMUM, AND MEAN DISCHARGE OF SOME WESTERN RIVERS.

ment basins are precipitous and barren. Little water soaks into the soil to supply the streams from springs. After a heavy storm most of the water runs off in a very short period of time, resulting in great floods. Thus streams which at flood height may reach from 10,000 to 15,000 second-feet discharge for a few hours or days may sink within a week or so to paltry rills of a few second-feet discharge or may entirely disappear. (Fig. 3.) With such streams it becomes necessary to so design works that most of the discharge may be saved by storage within a short period of time.

33. **Mean Discharge of Streams.**—When definite data of the annual discharge of a stream is not available it may be obtained approximately by multiplying the depth of runoff in inches into the area in square miles of its catchment basin. As shown in article 29, the proportion of rainfall which runs off varies between 50 and 80 per cent, according as the slopes are flat or steep, wooded or barren. The discharge ranges between 8 and 20 second-feet per square mile of catchment area.

34. **Available Annual Flow of Streams.**—Where irrigation is practised all of the water flowing in the streams is not available for storage, since much of it is already appropriated by irrigators, and this quantity must be deducted from that available for storage. A large portion of the discharge occurs in winter when the streams are covered with ice which renders it practically impossible to divert the water for storage, though it is available for such reservoirs as may be on the main streams. As nearly all of the flow occurring in the irrigating season is appropriated, only the surplus and flood water is available for storage.

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CHAPTER V.

SUBSURFACE WATER SOURCES.

36. **Sources of Earth Waters.**—The water which enters the soil by percolation either from rain or from canals, reservoirs, or lakes finds its way through the soil to some lower level where favorable geologic structure enables it to again reach the surface. This seepage water may move slowly through the particles of subsoil, its motion being rather that due to absorption or capillary attraction than to direct percolation; or it may enter some seam between two formations from which it may find an exit perhaps at some great distance through a spring or artesian well. The flow of water by percolation is limited not only by the degree of porosity of the strata, but by their inclination. Yet comparatively impervious rocks frequently furnish abundant supplies which are the result of capillary attraction.

37. **Sources of Springs and Artesian Wells.**—Wells and springs usually derive their water supplies from shallow formations as gravels, sands, and marls. Their temperature may be variable owing to the changes in the temperature of the surface of the soil, while their flow is effected by precipitation of recent occurrence and by evaporation from the surface of the ground.

Gravitation tends to draw the water toward the centre of the earth, and it percolates in that direction until intercepted by some impervious stratum along which it finds its way. If the water fills a pervious stratum so surrounded by impervious strata that it is prevented from escaping, and the hydrostatic pressure due to the inclination of the beds is sufficient to bring the water to the surface, the conditions are favorable for the production of an artesian well. All that is necessary is to pierce the upper confining stratum by boring, when the water will escape. Generally artesian supplies exist in the newer sandstones and other equally porous rocks. Waters are frequently gathered into such strata from distant catchment basins. Where such a water-bearing stratum approaches the surface in a broad plain it forms an extensive artesian basin.

38. **Artesian Wells.**—Deep wells do not always overflow. The condition of overflow depends on whether the pressure is sufficiently great to force the water above the surface, in which case they are known as artesian wells. Frequently the water will reach within but a few feet of the surface. when an ordinary well or shaft can be excavated and the water pumped to the desired height. In many other cases the pressure is such that the water spouts forth from the well under considerable pressure to great heights. In an artesian area of considerable extent the various wells seriously influence each other. In the San Gabriel and San Bernardino valleys in Southern California it has been found that after a certain number of wells have been sunk, each additional well affects its neighbors by diminishing their discharge. There thus comes a point in the sinking of wells when the number which can be utilized in any given area or basin is limited.

39. **Examples of Artesian Wells.**—Some great wells have been sunk in different parts of the world. The celebrated Grinnell well in Paris has an 8-inch bore and is 1806 feet in depth. A well is now being bored in the neighborhood of Wheeling, West Virginia, which has reached a depth of over 5000 feet. In St. Louis is a well which reaches a depth of 3850 feet; about 3000 feet below the sea-level. In San Bernardino and San Gabriel valleys in Southern California and in the upper San Joaquin valley in the neighborhood of Bakersfield are some very extensive artesian areas, but the greatest artesian basins of the West are found in the neighborhood of Waco, Texas; Denver, Colorado, and of the James river valley and the neighborhood of Huron in the Dakotas.

In 1890 there were 8097 artesian wells on farms in the arid region. Of these 3210 were in California, 2524 in Utah, 596 in

Colorado, and between 460 and 530 each in North Dakota, South Dakota, and Texas, besides a few in each of the remaining States and Territories. Of these wells 48½ per cent were used in irrigating 51,896 acres. Their average depth is 210 feet; average cost, $245; and average discharge, 54.4 gallons per minute.

40. Supplying Capacity of Wells.—The supplying capacity of common wells is frequently increased considerably by irrigation. As water is applied to the soil through a period of years the subsurface level rises and it may be reached at lesser depths than previously. In this way irrigation water may be used over several times; by pumping it from wells it may find its way by seepage back to the streams from which it may be again diverted.

41. Tunneling for Water.—Tunnels are sometimes driven in sloping or sidehill country to tap the subterranean water supplies. These are practically horizontal wells, differing from ordinary wells chiefly in that the water has not to be pumped to bring it to the level of the surface, but finds its way by gravity flow to the lands on which it is to be utilized. Near the Kojah Pass in India is a great tunnel of this kind. This is run near the dry bed of a stream into the gravels for a distance of over a mile. The slope of its bed is 3 in 1000, its cross-section is 1.7×3 feet, and its discharge about 9 second-feet. The Ontario Colony in Southern California derive their water supply from a tunnel 3300 feet in length, run under the bed of San Antonio creek through gravel and rock. Its cross-section is 5 feet 6 inches high, 3 feet 6 inches wide at bottom, and 2 feet wide at top. It is partly timbered and partly lined with concrete, having weep-holes in the upper part of the tunnel. Its discharge is about 6 second-feet. The supply from several sub-tunnels has been such as to average nearly 10 second-feet per linear mile of tunnel.

42. Other Subsurface Water Sources.—Earth waters may be gathered for irrigation by other means than springs, common or artesian wells, or tunnels. In portions of the plains region, especially in Kansas, subsurface supplies have been obtained by running long and deep canals parallel to the dry beds of streams or in the low bottom lands and valleys. These canals, acting like drainage ditches, receive a considerable supply of water and lead it off to the lands. In the dry beds of streams in California submerged dams have been built which reach to some impervious stratum and cut off the subterranean flow, thus bringing the water to the surface. In some experiments made on two subcanals in Kansas the amount of water obtained was 15 second-feet for each mile in length of excavation, which was 6 feet in depth below the subsurface water. It was found that the depth and length were the controlling factors, the breadth of the canal having little effect on the amount of water entering. It was also found that the increase of flow due to the deeper cuts was nearly as the square of the depth.

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CHAPTER VI.

ALKALI, DRAINAGE, AND SEDIMENTATION.

44. **Harmful Effects of Irrigation.**—When irrigation is practised without proper attention to drainage it is liable to result in the following evils: (1) production of alkali or flocculent salts on the surface of the ground; (2) souring or waterlogging of the soil due to supersaturation; (3) fevers and other injurious effects the result of the same cause.

45. **Alkali.**—The white efflorescent salt known as "alkali" is to be found in many portions of the West both as a result of irrigation and occurring naturally over extensive areas. These salts have been analyzed and are found to consist chiefly of chlorides, carbonates, and sulphates of sodium. Sometimes a small amount of potassium salts, sodium phosphate, or manganese sulphate are present. In most cases sodium sulphate predominates, ranging in amount from 5 to 75 per cent. The effect of this alkali on the surface of the ground is to kill all vegetable growth and to render the soil barren and unproductive.

46. **Causes of Alkali.**—Where the natural drainage of the country is defective and the strata underlying the surface are impervious or the soil not deep, irrigation or rainfall causes the subsurface water plane to rise to such a height that finally the soil becomes saturated. Evaporation then takes place from the surface, and as this process continues there is left on the soil the salts contained in the water. Thus the more water that evaporates from the surface the more alkali will be deposited, and increased rainfall or irrigation will increase the amount of alkali. It is thus seen that the direct cause of the production of alkali is the rise of the subsurface water plane

due to defective drainage. Seepage from badly constructed canals is a great producer of alkali. Thus where the velocity of the canal is slow, time is given for water to soak into the soil and permeate it.

47. **Waterlogging.**—Where the rise of water from the subsurface or its addition to the surface from natural causes or irrigation is more rapid than the losses by evaporation or drainage, the water stands in pools and the soil becomes soft and marshy, producing the effect known as "swamping" or "water-logging." Like alkali, waterlogging is directly traceable to defective drainage and the careless use of water. Where the conditions are sufficiently well balanced for drainage to prevent the rise of the subsurface water to within 10 or 15 feet of the surface, continued irrigation produces good results by soaking up the lower strata and giving an abundance of water near the surface for wells and for moistening the deeper rooting plants.

48. **Prevention of Alkali and Waterlogging.**—Several preventatives for the rise of alkali or the excessive soaking of the soil have been recommended, and some have been employed with success. Where it is impossible to entirely remove the alkali, the cultivation of deep-rooting or such plants as shade the soil and reduce the amount of evaporation may permit some use to be made of the land. Irrigating only such lands as have good natural drainage, and exercising care not to interfere with this, is one of the best and surest preventives of the production of alkali and waterlogging. The introduction of artificial drainage produces the same effect, while in a lesser degree the same result may be obtained by the use of deep ditches or furrows which themselves act as drainage channels. When the quantity of alkali is small the evil effects resulting from its presence may be mitigated by the application of chemical antidotes, and lastly relief may be obtained in some cases by watering the surface and drawing off the water without allowing it to soak into the ground. This system of reclaiming the land by surface washing and drawing off the salt-impregnated water is known as "leaching."

One of the most effective methods for the prevention of alkali is the judicious and sparing use of water in irrigation. If the least amount of water necessary for the production of crops is applied to the soil, the soaking of this with water will be a much slower process and may not result in oversaturation, even though the drainage be defective.

49. Chemical Treatment and Leaching.—A cheap antidote for most alkaline salts is lime, while neutral calcareous marl will answer in some cases. When the alkali consists of carbonates and borates the best antidote is gypsum or landplaster sown broadcast over the surface. Leaching may be practised by building temporary embankments around the land and flooding, then rapidly drawing or pumping off the salt-impregnated waters.

50. Drainage.—Generally the drainage of irrigated land will take care of itself if the natural drainage channels are not interfered with or obstructed. Where the surface has a moderate though sufficient slope to allow the water to flow off, or the soil is underlain by deep beds of gravel or porous rocks which will carry off the percolation water, irrigation may be practised for all time, and even an excessive amount of water may be used without seriously affecting the crops. In a few cases the drainage may be improved by digging drainage channels or ditches or laying drainage pipes under the surface. Such methods as these, however, are usually too expensive.

In many portions of the West, and especially in the San Joaquin Valley in California, old sloughs and abandoned natural drainage lines have been utilized as irrigation channels. The effect of this is bad, as the natural drainage lines thus become overloaded, resulting in waterlogging the soil. In this way large areas in Fresno County and its neighborhood have been rendered uncultivatable, whereas with a proper system of irrigating channels, providing the natural drainage channels had been left open, no evil effects would necessarily have resulted.

51. Excessive Use of Water.—This is one of the greatest evils at present noticeable in our Western irrigation methods. Almost invariably too much water is employed in irrigating crops. The result is the waste of water and the oversaturation of the soil. As the value of water rises it will be used with less extravagance. Proper care in the location and construction of the canal banks will aid greatly in reducing the evil effects of irrigation. If the location is bad, the natural drainage channels may be interfered with. If the construction is bad, the loss by seepage from the canal into the soil becomes great.

52. Silt.—Great volumes of silt are transported by Western rivers in times of flood. This is the result of the erosion of the alluvial banks of the streams. The heavier sand and gravel is soon deposited in the upper reaches of the river, and only the finest silt reaches the canals. As the velocity in these is relatively low, much of this sediment is deposited near the canal head, in storage reservoirs or in other slack water, thus choking the canal or diminishing the volume of the reservoir.

53. Amount of Sediment.—The quantity of this sediment which is carried in suspension during floods is frequently greater than is generally appreciated. From investigations made by the U. S. Geological Survey on the Rio Grande in 1889 it was found to range from $\frac{1}{4}$ to $\frac{1}{2}$ of 1 per cent of the volume of flow. It was estimated that in about 150 years the amount of this sediment would seriously impair a reservoir 60 feet in depth. On the American River at Folsom, California, in one year a depth of nearly 10 feet of wet silt was deposited in a reservoir at that point; much of this, however, was heavy matter, as gravels and boulders carried by the swift current of the stream.

54. Prevention of Sedimentation in Reservoirs and Canals.—There are practically but two methods of mitigating the injury due to sedimentation in reservoirs. One is by building higher up on the stream cheap settling reservoirs which may be destroyed in the course of a number of years, or the dams be increased in height as they silt up. The other method is by the construction of under- or scouring-sluices in the bottom of the dam. These have not as yet proved effectual, as their influence is felt at but a short distance back from the opening. Experience has shown that they do not remove silt which has already been deposited, but, providing their area is large compared with the flood volume of the stream, they may effectively prevent the deposition of sediment by permitting the silt-laden waters to flow through the reservoir; the latter only being filled after the flood has subsided and the waters become less turbid.

Canals should be so designed that the angle at which they are diverted from the main stream shall be such as to cause the least back eddy in front of the headgates and the least deposit at that point. Where a canal is taken off at right angles to the line of the stream and scouring-sluices are placed in the weir immediately adjacent to the headgates, the main stream may be so trained as to have a straight sweep past the headgates and thus scour out any deposits occurring at that point. In designing a canal the endeavor should be to so change the grade with the cross-section that a constant velocity shall be maintained throughout the main line and all its minor branches. In this event the silt will be maintained in suspension and will be carried through the minor ditches and not deposited until it reaches the fields. It thus becomes valuable, as it acts as a fertilizer. As the velocity of the current is generally diminished in the upper portion of the canal in its passage from the main stream, the deposit of silt is likely to occur at this point. It may be well to encourage this by increasing the cross-section of the canal and reducing its grade so that its capacity shall remain the same but its velocity be diminished. Then the deposit of silt will occur all at once in the first half-mile or less of the canal, and it may be either dredged out or perhaps scoured out by an escape.

55. Fertilizing Effects of Sediment.—The value of silt-bearing water as a fertilizer is well known. Where it is possible to keep the silt in suspension until the water reaches the fields, such waters are especially valued for purposes of irrigation. In the valley of the Moselle, France, on land absolutely barren and worthless without fertilization, the alluvial matter deposited by irrigation from turbid water renders the soil capable of producing two crops a year. In the valley of the Durance, France, the turbid waters of that stream bring a price for irrigation which is ten and twelve times greater than that paid for the clear cold water of the Sorgues River. It has been estimated that on the line of the Calloway Canal in California land which has been irrigated with the muddy river water gives 18 per cent better results after the fifth year than the same land which has been irrigated with clear artesian water.

56. **Weeds.**—When from any cause it becomes necessary to give a canal a low velocity, the growth of water weeds and the deposition of silt are encouraged. Water-plants grow most freely where the current has a slow velocity and the depth is such that the sunshine reaches the bottom. They thrive in shallow reservoirs, thus diminishing their capacity. Brush, willows, weeds, and rushes may encroach on the channels of canals where the slopes of the banks are low and so diminish the water-way as to greatly reduce their carrying capacity. Providing a high velocity cannot be given the only possible way of remedying this is to draw off the water and destroy the plants.

CHAPTER VII.

QUANTITY OF WATER REQUIRED.

57. Duty of Water.—The duty of water may be defined as the ratio between a given quantity of water and the amount of land which it will irrigate. In order to determine what amount of water is sufficient to supply a given area of land it is first necessary to at least approximately determine its duty for the specific case under consideration. On the duty of water depends the financial success of every irrigation enterprise, for as water becomes scarce its value increases. In order to estimate the cost of irrigation in projecting works, it is essential to know how much water the land will require. In order to ascertain the dimensions of canals and reservoirs for the irrigation of given areas the duty of water must be known.

58. Units of Measure for Water Duty and Flow.—Before considering the numerical expression of water duty, the standard units of measurement should be defined. For bodies of standing water, as in reservoirs, the standard unit is the "cubic foot." In the consideration of large volumes of water, however, the cubic foot is too small a unit to handle conveniently and the "acre-foot" is the unit employed by irrigation engineers. This is the amount of water which will cover one acre of land one foot in depth, that is 43,560 cubic feet. In considering running streams, as rivers or canals, the expression of volume must be coupled with a factor representing the rate of movement. The time unit usually employed by irrigation engineers is the second, and the unit of measurement of flowing water is the cubic foot per second, or the "second-foot" as it is called for brevity. Thus the number of second-feet flowing in a canal are the number of cubic feet which pass a given point in a second of time. A unit still generally employed in the West is the "miner's inch." This differs greatly in different localities and is generally defined by State statute. In California one second-foot of water is equal to about 50 miner's inches, while in Colorado it is equivalent to about 38.4 miner's inches. The period of time during which water is applied to the land for irrigation from the time of the first watering until after the last watering of the season is usually known as the "irrigating period." This is generally divided into several "service periods," by which is meant the time during which water is permitted to flow on the land for any given watering. Thus the irrigation period in the majority of the Western States extends from, say, April 15th to August 15th, about 120 days. The service period, or the duration of one watering, is generally from 12 to 24 hours, according to the soil and crop. The number of waterings making up the irrigation period vary between 2 and 5, according to the soil, climate and crop.

In the following table are given some few convertible units of measure.

TABLE VI.

UNITS OF MEASURE.

1 second-foot	=	450 gallons per minute.
1 cubic foot	=	7.5 gallons.
1 cubic foot weighs 62½ pounds at average temperature.		
1 second-foot	=	2 acre-feet in 24 hours (approx.).
1,000,000 cubic feet	=	23 acre-feet. (approx.).
100 California inches	=	4 acre feet in 24 hours.
100 Colorado inches	=	5⅙ acre-feet in 24 hours.
50 California inches	=	1 second-foot.
38.4 Colorado inches	=	1 second-foot.
1 Colorado inch	=	17,000 gallons in 24 hours (approx.).
1 second-foot	=	59½ acre-feet in 30 days.
2 acre-feet	=	1 second-foot per day or .03⅓ second-feet in 30 days.
100 California inches	=	3.97 acre-feet per 24 hours.
1 acre-foot	=	25.2 California miner's inches in 24 hours.

59. Measurement of Water Duty.—The duty of water may be variously expressed by the number of acres of land which a second-foot of water will irrigate; or by the number of acre-feet of water required to irrigate an acre of land; or in terms of the total volume of water used during the season. It may also be expressed in terms of the expenditure of water per linear mile of canal, though this form can only be satisfactorily employed when the location of the canal line has been previously determined. In considering the duty of water care should be taken to show whether it is reckoned on the quantity of water entering the head of the canal or the quantity applied to the land, since the losses by seepage, evaporation, etc., in the passage of water through the canal are considerable. Thus, if in a long line of canal the duty is estimated at 150 acres per second-foot and the losses by seepage and evaporation are 33⅓ per cent, the duty would be reduced to 100 acres at the point of application.

60. Duty per Second-foot.—The duty of water in various portions of the West is a matter of extreme doubt. As recently as in 1883 it was estimated in Colorado to be from 50 to 55 acres per second-foot. In Montana and portions of Colorado the farmers still state the duty as being one miner's inch to the acre, or 38.4 acres per second-foot. Recent experiments show that the duty is rapidly rising, for as land is irrigated through a series of years it becomes more saturated, and as the subsurface water plane rises the amount of water necessary to the production of crops is diminished. The cultivation of the soil causes it to require less water. The adoption of more careful methods in designing and constructing distributaries and care and experience in handling water increase its duty. The State Engineer of Colorado now accepts 100 acres per second-foot as the duty for that State. In Utah 60 acres per second-foot is accepted as the present duty. In Montana it is about 80 acres per second-foot.

In the following table the duty of water is given for a few foreign countries and for various portions of the West. These duties cannot be taken as fixed. They are apt to be increased with experience, and in the same State or even in the same neighborhood they will differ according as the crops, soil, altitude, and the skill in handling the water vary.

TABLE VII.

DUTY OF WATER.

Locality.	Duty per Second foot in Acres.
Northern India	250–300
Valencia, Spain	200–325
Northern Chili	190
Italy	65–70
Colorado	80–100
Utah	60–80
Montana	80–100
Wyoming	70
Idaho	60
New Mexico	60
Southern Arizona	100
San Joaquin Valley, Cal	100–150
Southern California, surface irrigation	150–300
" " sub-irrigation	300–500

The reason for the high duty given for such an arid region as Southern California is because the water there, being valuable, is handled with great care. Where sub-irrigation is employed the duty has in some cases reached as high as 1000 acres per second-foot. In Wyoming, where care was taken on an experimental farm in handling water, its duty was found to be as high as 94 acres on oats and 230 acres on potatoes.

61. Depth of Water required to Soak Soil.—Experiments conducted in India have shown that a good heavy rain amounting to about $5\frac{1}{2}$ inches soaks into the earth to a depth of from 16 to 18 inches. If this amount of water were applied three times in the season, it would be equivalent to a total depth of $16\frac{1}{2}$ inches to the crop. Experiments made in Colorado showed that good crops could be raised by the application of a depth of $18\frac{1}{2}$ inches of water, while in Wyoming 12 inches applied to potatoes and $24\frac{1}{2}$ inches to oats proved sufficient. In Idaho the depth of water necessary is now assumed to be about 2 feet, while in Montana 15 to 18 inches is believed to be sufficient.

62. Duty per Acre-Foot.—An average depth of 3 inches of water on the surface is sufficient to thoroughly water an average soil. In sandy soil 4 inches is required. This is equivalent to 10,454 cubic feet, or about $\frac{1}{4}$ of an acre-foot per acre. The average crop requires about four waterings in the season. This at the above rate would be equivalent to 42,500 cubic feet, or nearly an acre-foot per acre. From the results shown in article 61 it will be seen that from $1\frac{1}{4}$ to 2 acre-feet in depth applied to the land is sufficient to irrigate it. In estimating the duty of water stored in a reservoir allowance must be made, however, for the loss due to evaporation and absorption in conducting the water to the fields. As this will rarely average below 25 per cent it follows that where a duty of one acre-foot per acre is possible $1\frac{1}{4}$ acre-feet must be stored in the resorvoir, and where 2 acre-feet per acre is the duty $2\frac{1}{2}$ acre-feet must be stored.

63. Linear and Areal Duty.—From experiments made in India it was found that from six to eight second-feet of water should be allowed per linear mile of canal. This quantity, of course, depends on the area on either side of the canal which it will command. On the Soane Canal in India a more convenient unit was employed, it having been discovered that about three fourths of a second-foot was sufficient for a square mile of gross area. As the net area irrigated, however, is rarely more than two thirds of the gross area commanded, perhaps about one half a second-foot is sufficient to irrigate a square mile when the most economic use is made of the water.

64. Percentage of Waste Land.—In every irrigated area it has been discovered that but a small percentage of the total area commanded is irrigated in any one season. Some of the land is occupied by roads, farm-houses, or villages. Some is occupied by pasture lands which receive sufficient moisture by seepage from adjoining irrigated fields; and some by barnyards, while occasionally fields are allowed to lie idle for a season. In this way it has been discovered in India that generally but two-thirds to four-fifths of the total area commanded has been irrigated, though in some localities this percentage is a trifle larger. This is particularly so in the neighborhood of the Soane Canals in India, where about 500 acres out of every 640 are irrigated. From estimates made of the area under cultivation in well-irrigated portions of the West it has been discovered that if water is provided for 500 out of every 640 acres, it will be sufficient to supply all the demands of the cultivators. Keeping this in mind, it will be seen that the actual duty of water when estimated on large areas is at least 20 per cent greater than the theoretic duty per acre.

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CHAPTER VIII.

PRESSURE AND MOTION OF WATER.

66. Physical and Chemical Properties of Water.—Water is composed of an infinite number of minute particles, each of which has weight and can receive and transmit this in the form of pressure in all directions. The particles composing water move upon and among each other with an inappreciable amount of friction. Water is composed of at least two atomic substances, oxygen and hydrogen, combined in the ratio of one of oxygen to two of hydrogen, the whole forming a molecule of water. These molecules are so fine that it has been estimated that there are from 500 million to 5000 million in a linear inch.

67. Weight of Water.—Water reaches its maximum density at about 39.2° Fahrenheit, and the weight of a cubic foot of distilled water at this temperature is 62.425 pounds; and of a U. S. gallon 8.3799 pounds. Below and above this temperature the weight of a given volume of water increases. The weight of a cubic foot of ice is 57.2 pounds. At 32° Fahrenheit a cubic foot of distilled water weighs 62.417 pounds, and its weight increases from this to the maximum density above given, from which it decreases to 62.367 pounds at 60° Fahrenheit, and continues to decrease almost uniformly to a weight of 59.707 pounds at a temperature of 212°, which is the boiling point of water. Ordinary pond, brook, or spring water is heavier than distilled water because of the trifling amounts of salts carried in solution in most fresh waters, while salt water or water laden with sediment is still heavier, according to the amount of mineral or suspended matter in a given volume.

68. Pressure of Water.—Each molecule of water is independently subject to the force of gravity, and therefore has weight. When water is pressed by its own weight or that of any other force, this pressure is transmitted equally in all directions. The pressure at any point of a volume of water is in proportion to the vertical depth of that point below the surface, and is independent of the breadth of the volume of water. If water be contained in a vessel of any form in which an orifice is made the particles of water at that point are relieved of the resistance of the confining surface, and at once slide on each other and flow out of the orifice with a velocity proportional to its depth below the surface, or to what is known as the "head." The pressure due to a column of water in a vertical tube is directly proportional to its height, and if the column be bent or inclined at any angle the pressure will not be dependent on the length of the crooked confining channel, but to the height of the surface vertically below the lowest part of the column.

69. Amount of Pressure of Water.—A cubic foot of water is ordinarily taken as weighing 62.5 pounds, and the pressure per square inch for each vertical foot of depth below the surface of water is about 0.434 pound. By means of the ordinary methods adopted in considering the parallelogram of forces, the pressure of a body of water against an inclined surface at any given point may be determined by representing the depth (or the weight due to the depth at that point) by a line, the length of which bears a certain proportion to the weight, and by resolving this inclined line into its resultant horizontal and vertical components, these latter will then represent the relative horizontal and vertical pressures exerted by the water against that point. To find the total pressure of water on any surface its area in square feet should be multiplied by the vertical depth of its centre of gravity below the water surface in feet, and the total by the weight of one cubic foot of water.

Making $h =$ to the head or depth below the surface, $p =$ the pressure in pounds at that point, and g the depth of the centre of gravity of the mass of water below the surface, or one half of h, and the weight of a cubic foot of water being 62.5 pounds, we have $p = 62.5hg$.

70. Centre of Pressure.—The force which tends to overturn or push a surface about a given point, is not in the centre of gravity of the body of water, but at two thirds of the depth from the surface to that point, and is known as the centre of hydrostatic pressure, while the centre of gravity is at one half the vertical depth of the point. The total pressure upon a curved surface is proportional to the total length of that surface, but the horizontal effect of this pressure is directly proportional to the vertical projection of the surface.

71. Atmospheric Pressure.—The weight of the atmosphere upon the surface of any substance at the level of the sea is about 14.75 pounds per square inch. This quantity is known as an atmosphere, and will sustain a column of water 34.028 feet in height. In other words, the pressure of the atmosphere would raise a column of water to this height. It is on this account that it is possible to raise water by pumping or to cause water to flow through a siphon. The act of pumping or of raising water by a siphon produces a vacuum above the water, and the pressure of the atmosphere forces the water up to fill this vacuum to a height, approximately, of 34 feet. Owing, however, to friction and other causes, water can never be raised to quite this height; while at altitudes above the sea-level where the atmosphere is lighter, its sustaining power is diminished and the height to which it will force water is diminished proportionately.

72. Motion of Water.—The motion of water is due to a destruction of the equilibrium among the particles forming its mass, and it is said to "flow" because the action of gravity generates motion and destroys equilibrium. The motion of a falling body is constantly accelerated by the force of gravity in regular mathematical proportion. At the level of the sea a body falling freely *in vacuo* drops a height of 16.1 feet during the first second of time, its velocity at the end of the first second being 32.2 feet, and it is accelerated by this amount for each succeeding second. It is this quantity which is known as the acceleration of gravity, and which is usually designated by the letter g in hydraulic formulas. The velocity g of a body at the end of a given space of time is equal to the product of time into its acceleration by gravity. Thus, $v = gt$. It has been shown that the height h, through which the body falls or through which its pressure is accelerated, is equal to one half of the gravity, and the heights fallen in any given time are as the squares of the time; hence $t = \sqrt{\frac{2h}{g}}$, and substituting, transposing and eliminating we have $v = \sqrt{2gh}$.

CHAPTER IX.

FLOW AND MEASUREMENT OF WATER IN OPEN CHANNELS.

73. Factors affecting Flow.—If an open channel be given the smallest possible inclination in one direction, the water contained therein will be at once set in motion by the act of gravity, and its particles will fall one over the other in the direction of the inclination until motion or flow in that direction takes place. The effect of the action of gravity to produce motion is dependent on the slope, and this is usually represented by the ratio of the vertical to the horizontal distance; so we have as factors representing the velocity of flow the length of the channel, l, for a vertical fall of any given height, h. The amount of friction offered by the sides of the channel to the flow of water and tending to impede its velocity is one of the important factors, and is dependent chiefly on the nature of the bed and sides of the channel, that is, to the lining or surface of the channel against which the water flows, and on the length of wetted perimeter or the sectional area against which the water presses. Other quantities on which the coefficients of flow in channels depend are the hydraulic mean depth, r, which is equal to the area of the cross-section of the water in square feet, A, divided by the wetted perimeter in linear feet, p. A simple formula representing the mean velocity of flow is

$$v = \sqrt{\frac{2g}{m}} \times \sqrt{ri},$$

in which i is the sine of the inclination, h divided by l in feet; h being the fall of the water surface in the distance l; m is a variable coefficient, which includes most of the minor modifying factors. Tables of the value of m are published in nearly all books on hydraulics, from which it will be found that m varies between .05 for a hydraulic mean radius of .25, to .0298 for a hydraulic mean radius of 1, and diminishes constantly thence to a value of .0074 for a hydraulic mean radius of 10, and .002 to a hydraulic mean radius of 25.

74. **Formulas of Flow in Open Channels.**—There are many formulas for finding the mean velocity of flow in open channels. These have all constant coefficients, and are therefore incorrect outside of a small range of dimensions. Recently, as a result of experiments on the Mississippi by Humphreys and Abbot, and of experiments made in India, Kutter has devised a formula which takes into account the resistance due to the varying quantities n and k, which depend on the nature of the surface of the channel. Bazin made some experiments on small canals, from which he devised a formula which has received popular favor. This formula is arranged with various constant factors, according to the four grades of roughness of the surface of the channel. Modifications of this formula have been devised by D'Arcy which are still more convenient to use. D'Arcy's formula is

$$v = r\sqrt{\frac{1000i}{.08534r + 0.35}},$$

in which i equals the fall of water in any distance, l divided by that distance $= \frac{h}{l} =$ the ~~sign~~ sine of the slope.

75. **Kutter's Formula.**—The formula which is now most approved for determining the velocities of flow in open channels is Kutter's formula,

$$v = \left\{ \frac{\frac{1.811}{n} + 41.6 + \frac{.00281}{i}}{1 + \left(41.6 + \frac{.00281}{i}\right) \times \frac{n}{\sqrt{r}}} \right\} \times \sqrt{ri}.$$

Substituting for the first term of the right-hand factor the letter C, we have Chezy's formula

$$v = C\sqrt{ri}.$$

For small channels of less than 20 feet bed width Bazin's formula gives fair results where the sides and bottom are well built. The coefficients in this formula depend on the nature of the surface of the material and the hydraulic mean depth. The following table, from Flynn's "Flow of Water in Open Channels," gives the value of C for a wide range of earth-channels, and will cover nearly everything occurring in ordinary practice.

TABLE VIII.

VALUE OF C FOR EARTH CHANNELS BY KUTTER'S FORMULA.

Slope.	$n = .0225$					$n = .035$				
	$\sqrt{r}$ in feet.					$\sqrt{r}$ in feet.				
1 in.	0.4	1.0	1.8	2.5	4.0	0.4	1.0	1.8	2.5	4.0
	C	C	C	C	C	C	C	C	C	C
1000	35.7	62.5	80.3	89.2	99.9	19.7	37.6	51.6	59.3	69.2
1250	35.5	62.3	80.3	89.3	100.2	19.6	37.6	51.6	59.4	69.4
1667	35.2	62.1	80.3	89.5	100.6	19.4	37.4	51.6	59.5	69.8
2500	34.6	61.7	80.3	89.8	101.4	19.1	37.1	51.6	59.7	70.4
3333	34.	61.2	80.2	90.1	102.2	18.8	36.9	51.6	59.9	71.0
5000	33.	60.5	80.3	90.7	103.7	18.3	36.4	51.6	60.4	72.2
7500	31.6	59.4	80.3	91.5	106.0	17.6	35.8	51.6	60.9	73.9
10000	30.5	58.5	80.3	92.3	107.9	17.1	35.3	51.6	60.5	75.4
15840	28.5	56.7	80.2	93.9	112.2	16.2	34.3	51.6	62.5	78.6
20000	27.4	55.7	80.2	94.8	115.0	15.6	33.8	51.5	63.1	80.6

.25
30.
35.

This table is arranged with two different values for the factor n which are dependent on different qualities of surface in the channel. The accuracy of Kutter's formula depends chiefly on the selection of the coefficient of roughness n, and experience is required in order to give the right value to this coefficient. In order to provide for the future deterioration of the channel surface by the growth of weeds or its abrasion, it is well to select a high value for n. The following are some of the values of n for different materials as derived from Jackson, Hering, Kutter, and others:

$n = .009$ for well-planed timber;

$n = .01$ for plaster in cement, glazed iron pipes, and glazed stoneware pipes;

$n = .012$ for rough timber;

$n = .013$ to $.017$ for ashlar masonry, tuberculated iron pipes, and brickwork according to the smoothness of the surface and its condition;

$n = .02$ for rubble in cement and coarse rubble of nearly all kinds; also for coarse gravel carefully laid and rammed, or for rough rubble where the interstices have become filled with silt;

$n = .0225$ in good earth canals;

$n = .025$ to $.03$ in canals from those having tolerably uniform cross-section and slopes to those which are in rather bad order, and have some stones and weeds obstructing the channels;

$n = .035$ to $.05$ from canals and rivers with earth beds in bad order and obstructed by stones, etc., to torrents covered with all varieties of detritus.

76. Discharge of Streams and Velocities of Flow.—The quantity of discharge of a canal or river, Q, in second-feet is obtained by multiplying its velocity, v, in feet per second into the cross-sectional area, A, of the channel in square feet. Algebraically expressed, $Q = Av$.

Since the discharge of an open channel depends primarily on a knowledge of its mean velocity, it will be well to consider the relation of this to the velocities in other portions of the channel. In any open channel the film of water in contact with the open air has a velocity which is a trifle slower than that in the centre of the mass owing to the retarding effect of friction against the atmosphere. This velocity is known as the surface velocity. The velocities of the films adjacent to the sides and bottom of the channel are retarded to a still greater extent by the roughness of the same, and in direct proportion to this roughness. It has been found that in a channel of trapezoidal cross-section, with an average depth to width, the film of water having a mean velocity of the entire channel is located in the centre of the channel and at a point about one-third of the depth below the surface.

77. Surface and Mean Velocities.—The surface velocity is that which is most readily obtainable by simple methods. Experiments have been made by DuBuat, Francis, Brunning, and others to determine the ratio of the maximum to the surface velocity. From these it has been found that the approximate mean velocity $v = .915 V$, in which large V is the central surface velocity. In other experiments the ratio has been found to vary between .911 and .915. From careful experiments made in Germany it has been found that the mean velocity bears the ratio to the mean surface velocity, account being taken for the reduction toward the shore, of about .837. It will thus be seen that the ratio of the surface to the mean velocity varies with the section of the channel, and with the roughness of its sides as well as with the depth.

78. Measuring or Gauging Stream Velocities.—One of the simplest methods of gauging the velocity of a stream, but one which does not give the most accurate results, is by means of simple wooden floats or bottles, or some similar contrivance, thrown into the centre of the stream and timed for a given distance. For convenience 100 feet may be measured off on the bank and the time of the float ascertained in passing over this distance. This will give the central or maximum surface velocity. The mean surface velocity may be obtained by throwing a number of floats in different portions of the surface of the stream. This quantity multiplied by .8 will give approximately the mean velocity of the stream.

The velocity of the stream may be ascertained with still more accuracy by getting the mean velocity not of the surface but of the entire body of the stream by means of upright wooden floats so weighted that their bottoms shall float within a few inches of the bed of the channel. A number of these placed in different portions of the cross-section of the stream and timed over a course of a given length should give the mean velocity of the channel. In making careful gaugings of this sort the stream should be divided accurately into sections, and each float be permitted to pass down its section.

79. **Current Meters.**—Current meters are mechanical contrivances so arranged that by lowering them into the stream

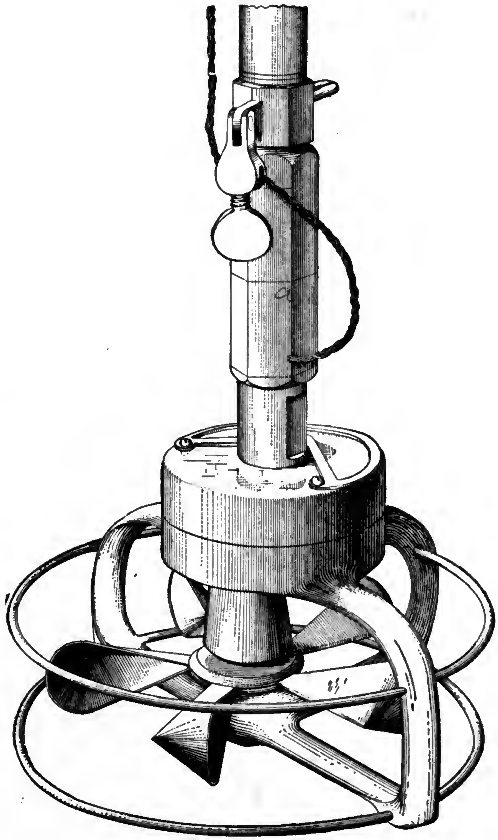

FIG. 4.—COLORADO CURRENT METER.

the velocity of the current can be ascertained. Various forms of current meters have been designed and used, the two general classes being the direct-recording meter, in which the number of revolutions is indicated on a series of small gear-wheels driven directly by a cog-and-vane wheel. And the electric meter, in which the counting is done by a simple make-and-break circuit, the registering contrivance being placed any desired distance from the meter. There are several different makes of current meters of both kinds. Of direct-acting meters, that which has recently found favor in the West is known as the Colorado meter, and is employed by the hydrographers of the U. S. Geological Survey, Fig. 4. The stem *a* is of iron pipe, several lengths of which may be joined together, though it is difficult to handle if over 8 feet in length.

There are several varieties of electric meters, one of which, Price's, has been received with considerable favor. This meter, however, like most of those which are employed in the Eastern States is not satisfactory in Western practice, since the Western

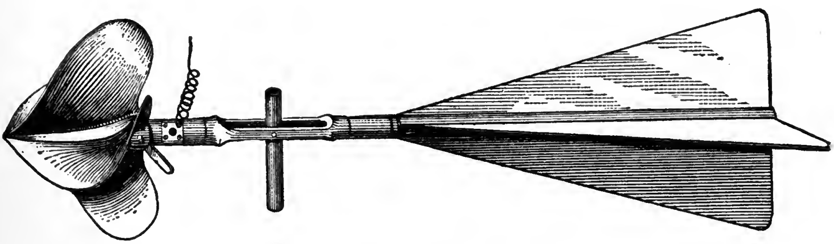

FIG. 5.—HASKELL CURRENT METER.

streams are so charged with sediment, weeds and driftwood in times of flood that the ordinary meter becomes clogged. The meter which has found the most favor in the West and is used by the hydrographers of the U. S. Geological Survey is a modification of the Haskell meter, Fig. 5.

80. Gauging Stations.—The first operation in making a careful gauging of velocity by means of a current meter is the choosing of a good station. This consists in finding some point on the course of the stream where its bed and banks are nearly permanent, the current of moderate velocity, and the cross-sections are uniform for about 100 feet above and below the gauging station. At this point a wire should be stretched across the stream and tagged with marks placed every 5, 10, or 20 feet apart, according to the width of the stream. An inclined gauge rod is firmly set into the stream at some point where it can be easily reached for reading, and gauge heights are recorded through a long period of time in order that variations in the velocity and discharge may be had for different flood heights.

81. Use of the Current Meter.—The current meter may be conveniently used either from a boat attached to a wire cable strung a little above the tagged wire, or from a bridge which does not impede the channel so as to make currents or eddies in the water. In using the direct-acting meter the gauger holds it in his hands by the rod, and inserting it in the water at any desired depth allows it to register for a certain number of seconds. In obtaining the mean velocity of the stream he plunges it slowly up and down from the bottom of the stream to its surface a few times for a given length of time at each section marked on the tagged wire, and in this way gets the mean velocity of each section. The area of this section is of course already ascertained by a cross-section made by measurement or sounding of the stream, and the mean velocity multiplied into the area of each section gives the discharge at that point. Care must be taken to hold the rod vertically, as any inclination of the meter materially affects its record.

In using the Haskell meter it is suspended and inserted in the same manner for moderately shallow streams, but in deep flood streams it is generally suspended by a wire instead of being pushed down by a rod, and a very heavy weight is attached to its bottom to cause it to sink vertically. In the Colorado meter the registering wheels are stopped and started by pulling a wire which throws the wheels in and out of gear. With the Haskell meter the registering is done by electricity,

82. Rating the Meter.—Before the results can be obtained each meter must be rated; that is, the relation between the number of revolutions of the wheel and the velocity of water must be ascertained. This is usually done by drawing the meter through quiet water over a course the length of which is known, and noting the time. From the observations thus made the rating is determined either by formula or by graphic solution. The distance through which the meter is drawn divided by the time gives the rate of motion or velocity of the meter through the water. The number of revolutions of the wheel divided by the time gives the rate of motion of the wheel. The ratio of these two is the coefficient by which the registrations are transformed into velocity of the current. This is not a constant. Taking the number of registrations per second as abscissæ represented by x, and the velocity in feet per second as ordinates represented by y, we get the equation $y = ax + b$, in which a and b are constants for the given instrument.

In determining the rating of the meter graphically, the values of x and y gotten directly from the instrument are plotted as co-ordinates, using the revolutions per second as abscissæ and the speed per second as ordinates. In this way a series of points are obtained through which a connecting line is drawn, giving the average value of the observations. From the position of the line thus plotted the coefficient of velocity can be read off corresponding to one, two, or any number of revolutions per second. When in actual use it is evident that at each rate of speed of the meter there is a different coefficient of velocity. Three or four of these for average variations in velocities may be used in getting the true velocity from the meter record.

83. Rating the Station.—After daily readings of the gauge height of the water have been taken at the station for some time, and the velocity measured by means of the meter at different heights of stream, the results should be plotted on cross-section paper, with the gauge heights as ordinates and the discharges (obtained by multiplying the velocities into the cross-section) as abscissæ. These points generally lie in such a direction that a line drawn through them gives nearly half a parabolic curve and represents the discharge for different heights. Having once plotted this line it becomes possible to determine the discharge of the stream at any time by knowing the height of the water from the gauge-rod.

84. Measuring Weirs.—The method of measuring discharge which is most popular among the irrigators of the West because of its simplicity is by means of weirs. This method is best suited to streams and canals of moderate size, while the results are generally quite accurate. It is exclusively used in Australia, and is extensively employed in Colorado and other portions of the West. Where the contraction is complete its coefficient remains constant, and the Francis formula gives the discharge with errors not exceeding one half of one per cent for depths of water varying between 3 and 24 inches, providing the length of the weir is not less than three or four times the depth of the water flowing over it. The three forms of weir which are most popular are the rectangular weir with vertical sides, and the trapezoidal or *v* weir, both of which have inclined sides with slopes of about one fourth horizontal to one vertical. The discharge of the weir Q is equal to the product of its area A into its mean velocity v; thus $Q = Av$.

85. Rectangular Measuring Weir.—In using the ordinary weir, Fig. 6, this should be placed at right angles to the

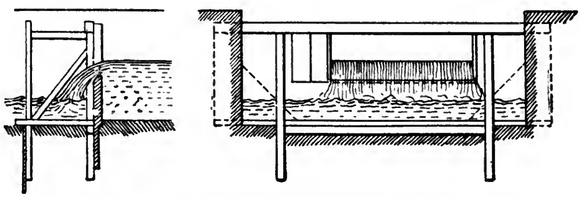

FIG. 6.—RECTANGULAR MEASURING WEIR.

stream, with its up-stream face in a vertical plane. The crest and sides should be chamfered so as to slope downward on the lower side with an angle of not less than 30°, while the crest should be practically horizontal and the ends vertical. The dimensions of the notch should be sufficient to carry the entire stream and yet leave the depth of water on the crest not less than five inches. The sectional area of the jet should not exceed one fifth that of the approaching stream. In order that the proper proportion of the area of the notch to that of the jet shall be maintained, central contractions may be introduced, dividing the weir crest into several orifices.

86. Francis' Formulas.—The form of equation indicated by theory is

$$Q = clh^{\frac{3}{2}},$$

where l is the effective length of the weir in feet, and h the depth in feet of water flowing over it. Because of the downward curve of the water after passing over the weir, this height h must be measured at some distance above the weir in order to be free from its influence. The constant c is a coefficient which, according to the experiments of Mr. J. B. Francis, was determined to be 3.33. Owing to this falling away of the crest and to the contraction at the ends, if l' be the effective length of the weir, one end contraction makes $l' = (l - 0.1h)$, and any number of end contractions make $l' = (l - 0.1nh)$. The reduction of volume by the crest contraction can be compensated for by the coefficient m, and inserting these factors in the ordinary formula of discharge,

$$Q = l \times \tfrac{2}{3} h^{\frac{3}{2}} \sqrt{2g},$$

we have

$$Q = \tfrac{2}{3} m \sqrt{2g}(l - 0.1nh)h^{\frac{3}{2}}.$$

The factors $\frac{2}{3}m$ and $2g$ are constants, and representing them by c we can substitute it in the formula as a coefficient. This is the coefficient which was determined to be equal to 3.33 by Francis' experiments, and substituting this value and transposing we get $m = .622$. Substituting these values into the former equation and eliminating, we get for approximate results,

$$Q = 3.33(l - 0.1nh)h^{\frac{3}{2}},$$

which is Francis' formula.

87. Conditions of using Rectangular Weir.—If the weir be placed so as to meet the following conditions, a more simple formula than that just given can be employed for ordinary use: thus we may say

$$Q = 3.33lh^{\frac{3}{2}}.$$

The conditions are, that the water shall not exceed 24 or be less than 4 inches in depth; that the depth on the crest shall not exceed one third the length of the weir; that there shall be complete contraction and free discharge; and that the water shall approach without perceptible velocity or cross-currents. To obtain these conditions the distance from the side walls to the crest should be at least equal to the depth on the weir, and the distance of the crest above the bottom of the channel should be at least twice the depth of water flowing over it. Air should have free access under the falling water, and the approaching channel should be much larger than the weir opening.

Table IX, on the next page, is adapted from L. G. Carpenter on the Measurement of Water.

88. Trapezoidal Weirs.—As a result of experiments made in Italy in 1886 by Cippoletti, he adopted a trapezoidal weir the sides of which have an inclination of one fourth horizontal to one vertical. The conditions called for in placing a rectangular weir must be nearly fulfilled with a trapezoidal weir, but the distance of the sill of the weir from the bottom of the canal must be at least three times the depth of the weir, and its length must be at least three times the depth of the water flowing over it. In using this form of weir the equation becomes:

$$Q = 3.36\tfrac{2}{3}lh^{\frac{3}{2}}.$$

This weir seems to possess some excellent qualities, the chief difficulty in connection with it being the same as arises in using the rectangular weir, namely, that where silt-laden water is employed this may fill up above the front board of the weir.

In using a triangular weir a convenient formula has been found to be the following:

$$Q = 2.65th^{\frac{5}{2}},$$

TABLE IX.

DISCHARGE OVER RECTANGULAR WEIRS OF VARIOUS LENGTHS, WITH VARIOUS DEPTHS OF WATER AND TWO COMPLETE CONTRACTIONS.

Formula, $Q = 3.33(l - 0.2h)^{\frac{3}{2}}$

Depth from Still Water on Crest, in feet = h.	Discharge Q, in Second-feet.				Cor. to be added to give Q, with no contraction.
	l = 1 ft.	l = 2 ft.	l = 5 ft.	l = 10 ft.	
.10	.1033	.2078	.5240	1.0519	.0021
.15	.1879	.3816	.9627	1.9312	.0058
.20	.2861	.5843	1.4787	2.9690	.0119
.25	.3959	.8126	2.0227	4.1462	.0208
.30	.5149	1.0725	2.7057	5.4441	.0328
.35	.6420	1.3423	3.4032	6.8547	.0483
.40	....	1.6160	4.1489	8.3655	.0674
.45	....	1.9221	4.9410	9.9725	.0905
.50	....	2.2392	5.7748	11.6672	.1178
.55	....	2.5698	6.6489	13.4474	.1496
.60	....	2.9128	7.5607	15.3072	.1859
.65	....	3.2663	8.5064	17.2399	.2271
.70	....	3.6313	9.4882	19.2497	.2733
.75	....	4.0052	10.5002	21.3252	.3248
.80	....	4.3884	11.5434	23.4684	.3816
.85	....	4.7806	12.6135	25.6790	.4440
.90	....	....	13.7177	27.9477	.5123
.95	....	....	14.8451	30.2766	.5864
1.00	....	....	16.0000	32.6667	.6667
1.05	....	....	17.1784	35.1099	.7531
1.10	....	....	18.3825	37.6110	.8460
1.15	....	....	19.5080	40.1615	.9455
1.20	....	....	20.8569	42.7654	1.0516
1.25	....	....	22.1269	45.4184	1.1646
1.30	....	....	23.4189	48.1224	1.2846
1.35	....	....	24.7318	50.8753	1.4117
1.40	....	....	26.0625	53.6710	1.5460
1.45	....	....	27.4122	56.5122	1.6878
1.50	....	....	28.7814	59.3999	1.8371
1.55	....	....	30.1675	62.3290	1.9940
1.60	....	....	31.5727	65.3042	2.1588
1.65	....	....	32.9935	68.3185	2.3315
1.70	....	....	34.4269	71.3710	2.5120
1.75	....	....	35.8827	74.4662	2.7008
1.80	....	....	37.3520	77.6020	2.8980
1.85	....	....	38.8341	80.7716	3.1034
1.90	....	....	40.3321	83.9816	3.3174
1.95	....	....	41.8436	87.2271	3.5399
2.00	....	....	43.3665	90.5061	3.771
2.50	....	....	....	125.16	6.59
3.00	....	....	....	162.79	10.39

in which t is the tangent of half the angle in the notch of the triangle. If the triangle be right-angled, this formula becomes

$$Q = 0.317h^{\frac{5}{2}},$$

which is one of the simplest formulas that can be used, and gives excellent results on small streams.

89. **Weir Gauge Heights.**—In order to determine the depth of water flowing over the weir a post should be set in the stream a short distance above it, and on this a gauge rod suitably marked should be attached. For very exact measurements a hook gauge has been employed, which consists of a hook attached to a sliding rule fastened or hung so that its point shall be below the surface of the water. By turning a tangent screw the hook can be raised until it is exactly level with the surface, thus giving an accurate measurement of the depth of water.

90. **Measurement of Canal Water.**—No method has as yet been devised by which water flowing in open channels can be cheaply and conveniently measured. In order that canal water may be sold by quantity it is necessary that the volume admitted to the canal should be readily ascertained at any time, and that the method of admission should be so regulated that it cannot be tampered with. As no method has yet been devised for easily and cheaply accomplishing this, water is almost universally disposed of by canal owners by some means other than its direct sale by quantity. It is customary in India to charge a land rental which is regulated in accordance with the character of crop, as on this is dependent the amount of water used. In our country water rentals are charged per acre irrigated rather than by the amount of water required in this irrigation. In other words, water is not sold as it should be, like other commodities which have an instrinsic value, by the yard, pound, or gallon, though such would unquestionably be the most satisfactory method of disposing of it, both to the vendor and the user. Various endeavors have been made to devise some cheap and convenient method of measuring water at a cost commensurate with its value, but none of these can as yet be said to have achieved success.

91. Methods of Measurement.—In Italy and in some other portions of southern Europe a "module" or measuring apparatus has been employed with some success for the measurement of canal water. This module consists essentially of inserting in the canal bank a regulating gate on which the height of head can be maintained. The size of the orifice being known, the amount of water passing through it can be at any time ascertained. Modifications of this module are employed to a limited extent in India and to a greater extent in the United States. The unit of measure commonly employed in America and Italy is the "miner's" or statute inch, though the better unit is the second-foot. In India the amount of water flowing in canals and distributaries is measured either by a gauge rod placed in some smooth portion of the channel, as in a masonry lined aqueduct, while floats are timed for a given length in the aqueduct; or by means of a V-shaped measuring weir.

In the West the ordinary module employed for measuring the miner's inch is a box flume closed by a lifting gate, in which case the head above the orifice is changeable and the amount passing through is indeterminate. Sometimes a modification of this module devised by A. D. Foote is used, whereby the head over the orifice can be maintained with some degree of certainty. None of these modules are satisfactory, however, for the measurement of large volumes of water. The measuring weir is in all probability the most satisfactory method yet devised of obtaining an accurate measure of the volume of water passing through a canal. Where water passes through pipes, some of the standard varieties of water meters can be employed; or if these are considered too expensive, some modification of them, such as those employed on the Allesandro tract in California, may prove satisfactory. (Art. 217.)

92. The Statute Inch or Module.—As already stated, the statute inch is a variable quantity, depending on its designation in different States. As an example, the statute inch of Colorado (Art. 58) is defined as follows: An inch-square orifice shall be under a 5-inch pressure, measured from the top of the orifice to the surface of the water, in a box set in the banks of the ditch.

This orifice shall in all cases be 6 inches perpendicular inside measurement, and all slides closing the same shall move horizontally, while from the water in the ditch the box shall have a descent greater than one eighth of an inch to the foot.

93. **Foote's Water Meter.**—This apparatus is extensively used on the canals in Southern Colorado and on some of the canals in Idaho for the measurement and distribution of water by the inch. It acts both as a distributary head to minor

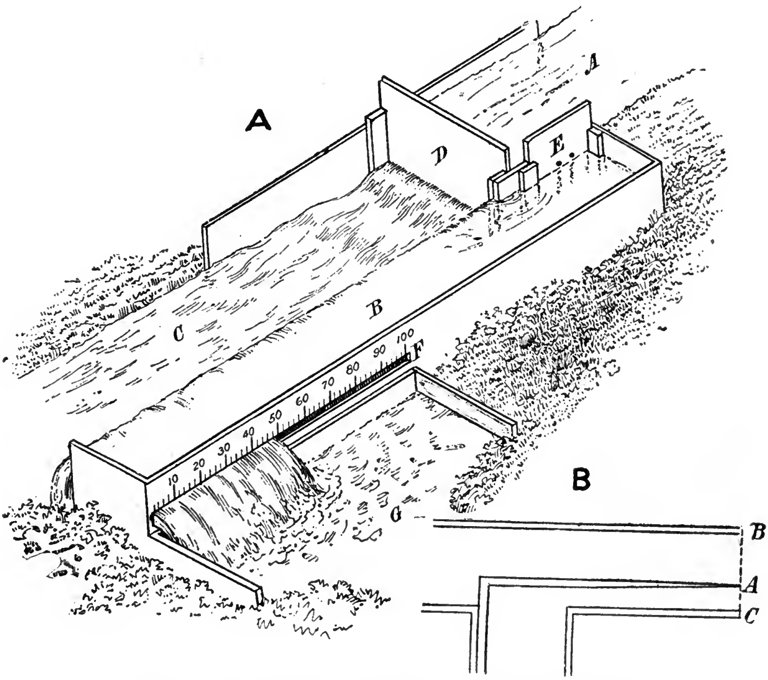

FIG. 7.—FOOTE'S MEASURING WEIR, A. WATER DIVISIOR, B.

channels and as a module or measuring box. It is constructed of wood, its chief merit consisting in that it renders it possible to maintain very nearly the standard head prescribed by statute over the opening. As shown in Fig. 7, *A*, it consists of a flume placed in the main lateral *A*, and of a side flume *B*, in which is constructed the measuring gate, while opposite to it is a long overfall *C*, the height of which is such as to maintain a standard head above the measuring slot. Such a weir is cheaply constructed and easily placed in position, while its cost is but trifling. Its chief fault as at present constructed is the fact that it measures water by the inch instead of by the second-foot, while like all such similar devices it can only be used in moderately small channels, since the difficulty of handling a slot on a large stream would be insurmountable.

94. **Rating Flumes.**—Under the laws of the State of Colorado rating flumes are constructed by the owners of private channels for the measurement of the flow of water, while the State Engineer is directed to compute the amount of water passing through them at various stages. They offer a convenient means of ascertaining the amount of water flowing in laterals and distributaries at various depths. They consist of a simple open flume which is placed in a straight portion of the channel a few hundred yards below its head gate. They are of even width with the channel, on the same grade, and their sides are sufficiently high to carry the amount of water likely to enter. For channels exceeding 6 feet in width an apron and wings of one-inch plank are built for 7 feet above and below the flume. The latter is generally 16 feet in length, consisting of a framing of 6 by 6 scantling, placed 4 feet apart and lined with one-inch or two-inch plank.

After these flumes have been constructed and placed the engineer rates them by means of a current meter, and furnishes the Water Commissioner and owner of the private channel with a table showing the quantities of water which will flow through them at various depths. It is then only necessary to raise the head gate until the desired depth flows through the flume, when the gate may be locked. The great difficulty with this, as with any similar device, is the changeability of head in the main channel above the head-gate and the fluctuation therein causing a change in the volume passing through the flume, necessitating a corresponding change in the position of the gate.

95. Divisors.—Another method of distributing water to consumers is that by means of a dividing box, the object of which is to give each consumer a definite portion of the water flowing in the lateral. The difficulty of dividing the water into two or more equal parts arises from the fact that the water has not a uniform velocity across the entire channel. If therefore equal openings be made across a channel, those near the centre have the greater discharge. As a consequence the use of a divisor gives only approximate results. A simple form of divisor is that shown in Fig. 7, *B*. In this there is a movable partition *A*, which can be slid out into the main channel so as to give the amount of water required in the branch. In order to maintain an equal velocity, the water is brought to a state of approximate rest by a weir board a few inches in height, the crest of which is sharp on the up-stream side.

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PART II.

CANALS AND CANAL WORKS.

CHAPTER X.

CLASSES OF IRRIGATION WORKS.

97. Gravity and Lift Irrigation.—All irrigation works may be divided into these two great classes. Under the head of gravity or natural-flow irrigation are included all those works in which the water may be conducted to the land by the force of gravity or natural flow. They include—

1. Perennial canals;
2. Periodical and intermittent canals;
3. Inundation canals
4. Storage works;
5. Artesian-water supplies.
6. Subsurface or ground-water supplies.

Lift irrigation includes those forms of irrigation in which the water does not reach the land by natural flow, but is transported to it by pumping or other means of lifting. It may be divided into two main classes:

1. Irrigation by watering-pots, hose, or sprinkling-carts;
2. Irrigation by pumping.

The first needs no explanation; the last may be divided into five principal classes:

1. Pumping by animal power;
2. Pumping by water-wheels;
3. Pumping by windmills;
4. Lifting by elevators;
5. Pumping by steam-power.

The sources of supply for all forms of gravity irrigation are defined by the titles of the classes. They are from perennial streams, intermittent streams, artesian wells, submerged dams, tunnels or cuts, or by the storage of perennial, intermittent, or flood waters. The sources of supply for lift irrigation may be from wells, canals, storage works, or flowing streams.

98. **Navigation and Irrigation Canals.**—Canals may be used for irrigation alone or for irrigation and navigation combined. The conditions required to develop an irrigation canal are: first, that it shall be carried at as high a level as possible so as to have sufficient fall to irrigate the land to a considerable distance on both sides of it; second, it should be fed by some source of supply that will render it a running stream, so that the water used in irrigation may be constantly replaced; third, it should have such a slope and velocity as to reduce to a minimum the deposition of sediment and the growth of weeds; fourth, its velocity should be the greatest possible in order that the cross-section may be reduced to a minimum for a given discharge. On the other hand, navigation requires of a canal; first, that the water in it shall be as nearly still as possible, so that navigation may be equally easy in both directions; and, second, it requires no further supply of water than is necessary to replace the loss by evaporation and absorption, and at the points of transfer from higher to lower levels. It is thus seen that the requirements of the two classes are conflicting, and it is not deemed good practice to make irrigation canals for purposes of navigation.

99. **Sources of Supply.**—The climate, geology, and topography are the chief factors in deciding the class of work which belongs to a given region. Where the precipitation is small, occurring during a short period of the year, and resulting in the intermittent or periodical flow of the streams, canals of this class or storage works must be employed. Intermittent and periodical canals are usually very small in dimensions, commanding relatively small areas of land, and are generally employed by individual farmers for the utilization of the waters of some stream which may be safely counted upon for a tem-

porary supply during a few occasional spring storms or the melting of the mountain snows. They can only be used with safety where the precipitation is nearly sufficient for the cultivation of crops and the little water which they supply is of value in helping this out. Storage works receive their supply from intermittent streams carrying moderate volumes of water at flood times, or perhaps from perennial streams, artesian wells, or in fact from any source from which a permanent supply of water may be obtained. Inundation canals are used almost exclusively in India and Egypt, and derive their supply from streams the beds of which are at an altitude relatively high compared with the surrounding country. This is the case of the river Indus, which practically flows on a ridge; and it is simply necessary when the water in this river is high to make a cut through its banks, and thus permit it to flow out into the canals which take it over the surrounding country. These inundation canals are thus supplied by flood waters which flow above the general level of the surrounding country, and rarely require any permanent headwork to control the entrance of the water to the canal.

Artesian wells derive their supplies from artesian water sources, which have their origin usually at some great distance and at an altitude considerably higher than the outlet of the well. Subsurface cuts, tunnels, and wells derive their supply from the seepage water with which the soil in nearly every country is permeated.

100. Perennial Canals.—Perennial canals derive their supply from perennial streams or from storage reservoirs. They may be divided into two classes, according to the location of their headworks. These are:

1. Highline canals, and
2. Low-service or deltaic canals.

Highline canals are generally of moderate size, and are designed to irrigate lands of limited area which lie close under the foot of the higher hills. They are generally given the least possible slope, in order that their grades may remain high and command the greatest amount of land. In such canals it is necessary to locate the headworks high up on the stream, frequently in rocky canyons where the first portions of the line may encounter heavy and expensive rock work. Low-service canals are constructed where the majority of the lands are situated in low-lying and extensive valleys and where the location of the head of the canal depends not so much on its being at a relatively high altitude and commanding a great area as upon the suitability of the site for purposes of diversion. Highline canals are more frequently constructed where the water supply is abundant and it is desirable to obtain the largest amount of land to which to apply it. Low-service canals are constructed where the irrigable lands exceed in area the amount of water available.

Deltaic canals have been constructed chiefly in Egypt and India at the deltas of some of the great rivers, as the Nile, Ganges, Orissa and others. They are essentially low-service canals and are built in regions where the slope is very small. As a consequence their cross-section must be relatively large, that they may carry a given discharge with the least velocity. They are usually navigable canals, and in most cases the water supply is abundant.

101. Dimensions and Cost of some Perennial Canals.—In table X, on page 70, are given the dimensions, including the capacity and area commanded, and the cost in various terms of some of the great perennial canals of the world.

102. Parts of a Canal System.—The machinery of a great perennial canal consists essentially of the following parts, which are treated here in the order given:

1. Source of supply;
2. Irrigable lands;
3. Main canal;
4. Head and regulating works;
5. Control and drainage works;
6. Distributaries and laterals.

The principal units of this system are the main canals and distributaries. Between different canal systems the greatest points of difference are found in the headworks and in the first few

TABLE X.

SOME GREAT PERENNIAL CANALS.

Name of Canal.	Locality.	Area commanded, Acres.	Length, Miles.	Capacity, Second-feet.	Grade.	Bed-width, Feet.	Depth, Feet.	Cost per Acre irrigated.	Cost per Second-foot for Water used.
Bear River Canal	Utah	200,000	150	1,000	1 in 5,280	50	7	$5.00	$125
Idaho Mining & Irrig. Co. Canal	Idaho	350,000	70	2,585	1 in 2,640	40	10	2.16	190
Pecos Canal	N. Mexico	200,000	75	1,100	1 in 6,707	45	6	5.00	690
Turlock Canal	California	176,000	93	1,500	1 in 5,280	70	7.5	14.50	730
King's River & San Joaquin Canal	"	90,000	67	600	1 in 5,280	32	4.5	7.18	277
Calloway Canal	"	80,000	32	700	1 in 6,600	80	3.5	10.00	710
Arizona Canal	Arizona	60,000	41	1,000	1 in 2,640	36	7.52	10.00	700
Highline Canal	Colorado	90,000	70	1,184	1 in 3,000	40	7	13.00	600
Del Norte Canal	"	200,000	50	2,400	1 in 2,112	65	5.5	....	....
Ganges Canal	India	1,820,000	456	6,700	1 in 4,224	170	10	5.25	290
Lower Ganges Canal	"	2,435,000	564	6.500	1 in 10,560	216	8	9.00	....
Sirhind Canal	"	800,000	503	3,500	1 in 4,800	190	6	13.00	121
Agra Canal	"	750,000	137	1,100	1 in 10,560	70	10	12.60	233
Soane Canal	"	1,000,000	367	5,950	1 in 10,560	180	9	8.70	....
Carpenteras Canal	France	16,800	32	212	1 in 4,000	33	2.8	35.65	2,830
Henares Canal	Spain	27,000	28	177	1 in 3,067	8	4.9	46.66	7,500
Cavour Canal	Italy	490,000	53	3.250	1 in 4,000	66	12	30.60	....

miles of diversion line, where numerous difficulties are frequently encountered, calling for variations in the form and construction of drainage works and canal banks.

The headworks consist usually of the diversion weir with its scouring sluices, of the head regulating gates at the canal entrance, and of the head or first escape gates. The control works consist of regulating gates at the head of the branch canals, and of escapes on the line of the main and branch canals. The drainage works consist of inlet or drainage dams, flumes or aqueducts, superpassages, inverted siphons, and drainage cuts. In addition to these works there are usually constructed falls and rapids for neutralizing the slope of the country, and tunnels, cuttings, and embankments. Modules or some form of measuring box or weir are necessary for the measurement of the discharge.

CHAPTER XI.

ALIGNMENT, SLOPE, AND CROSS-SECTION.

103. Location of Headworks.—The headworks of a canal are almost invariably located high up on the supplying stream, in order to command a sufficient area and to tap the stream where the water is clear and contains the least amount of silt. By so locating the headworks it is usually possible (owing to the greater slope of the country) to reach the water-sheds or interfluves with the shortest possible diversion line. The disadvantages of this class of location are serious, since the canal line is sure to be intersected by hillside drainage, the passage of which entails great difficulties; and as the adjacent slopes of the country are heavy, much expensive hillside cutting is required.

104. Diversion Line.—By diversion line is meant that portion of the canal line which is required in order to bring it to the neighborhood of the irrigable lands. It is that waste construction which does not command any irrigable land. The endeavor should always be made in locating the canal to reduce the length of diversion line to a minimum, so that the canal shall command irrigable land and derive revenue at the earliest possible point in its course.

105. Relation between Lands and Water Supply.—In designing an irrigation work the first consideration is the land to be irrigated. The projector must consider the area of this, its nearness to market, the quality of the soil, the climate, and the character and value of the crops which it will produce. In addition, the value and ownership of the land must necessarily be considered. All of these quantities having been satisfactorily determined and the necessity of supplying water for irrigation having been ascertained, the next question is the source of supply and its relative location to the lands. This supply may be found in some adjacent perennial stream, or it may be necessary to transport it across an intervening ridge from a neighboring water-shed, or it may be necessary to conserve in storage reservoirs the intermittent flow of minor streams. The relation of the water supply to the land, the extent of the latter, and the volume and permanency of the former are the most important items to be ascertained in the preliminary investigation of any irrigation project.

106. Survey and Alignment.—Having determined the source of water supply and its relation to the irrigable lands, the third question in order of importance is the alignment of the canal. This should be so made that the canal shall reach the highest part of the irrigable lands with the least length of line and at a minimum expense for construction. The line of the canal should follow the highest line of the irrigable land, preferably skirting the surrounding foothills and passing down the summit of the water-shed dividing the various streams.

In order that the best possible alignment may be obtained, careful preliminary and location surveys are necessary. That all possible locations may be examined, it is desirable, first, to construct a general topographic map on some large scale,—perhaps 800 to 1500 feet to the inch,—and with contour lines showing differences of elevation of from 5 to 10 feet. On such a map as this it is possible to at once lay down with a near degree of approximation the final position of the canal line. It is also frequently possible from inspection of such a map to save many miles of canal by the discovery of some low divide or some place in which a short but deep cut or a tunnel will save a long roundabout location. Having laid down this line on the map, the final location may be made on the ground, with the aid perhaps of a few short trial lines to determine its exact position.

107. Obstacles to Alignment.—Such obstacles as streams, gullies, ravines, unfavorable or low-lying soil or rocky barriers are frequently encountered in canal alignment. The best method of passing these must be carefully studied. It may be cheapest to carry the canal around these obstructions, or it may be better to at once cross them by aqueducts, flumes, or inverted siphons, or to cut or tunnel through the ridges. Careful study should be made of each case and estimates made of the cost not only of first construction, but of ultimate maintenance. In crossing swamps or sandy bottom lands it may be cheaper, because of the losses which the water will sustain from evaporation and absorption, to carry the canal in an artificial channel through such places. If water be abundant it may be less expensive on hillside work to simply build the canal with an embankment on its lower side, permitting the water to flood back on the upper side according to the slope of the country. In such cases the losses by evaporation and absorption will be great in the beginning, but ultimately these flat places may become silted up and a permanent channel made through them. The relative cost of building a sidehill canal wholly in excavation or partly in embankment should be considered. If the hillside is steep and rocky, the advisability of tunnelling, of building a masonry retaining wall on the lower side of the canal, or of carrying it in an aqueduct or flume will have to be considered.

108. Sidehill Canal Work.—It is extremely difficult to carry a large canal along steep sidehill slopes. In order to get a sufficient cross-section to carry the volume required without unduly increasing the velocity demands the exercise of careful judgment. It is possible to get the same cross-sectional area by employing different proportions of depth to bed width. The less the cross-sectional area of a channel, the less its cost and the expense for maintenance. It is therefore first necessary to choose the highest possible velocity which the resistance of the material and the necessity of commanding land will permit, and then to give the canal such a cross-sectional area as will produce the required discharge. The great difference in excavation of two canals of equal capacity but different proportions of bed width to depth is graphically shown in Fig. 8. In one case many times the amount of material will have to be removed than in the other, while the surface exposed to evaporation and absorption is greatly increased. Where the ma-

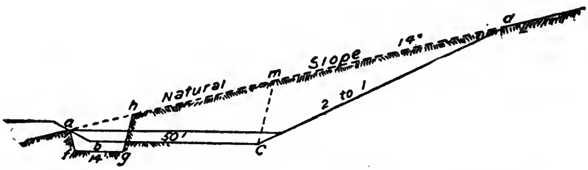


FIG. 8.—CANAL CROSS-SECTIONS FOR VARYING BED-WIDTHS.

terial is suitable and not too liable to cause loss by percolation, it is well to equalize the cut and fill. In this way still less material will have to be moved, for, as shown in the illustration, the depth of excavation is diminished by raising the lower bank.

109. Curvature.—A direct or straight course is the most economical, as it gives the greatest freedom of flow and causes the least erosion of the banks. It also greatly diminishes the cost of construction and the losses by absorption and evaporation consequent on the increased length of a less direct location. It is an error in alignment to adhere too closely to grade lines following the general contour of the country. By the insertion of an occasional fall it is frequently possible to obtain a more desirable location and to diminish the cost of construction by the avoidance of some natural obstacle.

One of the most serious errors in alignment is the careless location of curves, to which detail too little attention is ordinarily paid. The insertion of sharp bends inevitably results in the destruction of the canal banks, or requires that they shall be paved or otherwise protected to prevent their erosion. Instances have been noted where engineers have inserted great curves carefully constructed on some fixed radius of absurd length, as though the canal were a railway line. Curvature diminishes the delivering capacity of the canal, and too sharp a curve endangers the structure itself. In large canals of moderate velocity it will be safe in most cases to take the radius of curvature at from three to five times the depth of the canal. As the cross-section becomes smaller or the velocity is increased, the radius of curvature should be correspondingly increased. To keep up the discharge of a canal either its cross-section or grade should be increased in proportion to the sharpness of the curve.

110. Borings, Trial Pits, and Permanent Marks.—In finally locating an expensive work, borings and trial pits should be made, the former with a light steel rod and the latter by simple excavation in order to discover the character of the material to be encountered. In making the final survey of a canal it is well to place at convenient intervals permanent bench marks of stone or other suitable material. The establishment of these along the side of the canal in some safe place will give convenient datum points to which levels can be referred whenever it may be necessary to make repairs or run branch lines. Mile or quarter-mile posts or permanent stakes should also be set in the canal banks so that future surveys and changes in the line may be referred to these.

111. Example of Canal Alignment—Ganges Canal.—An excellent example of a typical alignment on one of the great Indian canals is that of the Ganges canal, which heads in the Ganges river at Hurdwar, where the stream issues suddenly from between the foothills of the Himalayas on to the broad level plains. In the first 20 miles of its course the canal encounters considerable sub-Himalayan drainage, and the works for the passage of this and for the reduction of slope in the canal by means of falls are important (Pl. II). The slope of the river bed and country averages from 8 to 10 feet per mile.

At the site of the headworks the river is divided into several channels, one of which, about 3 feet in width, follows the Hurdwar shore and rejoins the main stream half a mile below that town. As the discharge of the canal is 6700 second-feet and that of the river never falls below 8000 second-feet, only a portion of the water is required at any time. This is diverted to the Hurdwar channel by means of training works and temporary bowlder dams, and the current has deepened the channel until it now has a uniform slope of $7\frac{1}{2}$ feet per mile to the canal head. The regulator is about half a mile below the first training works, and consists of a weir and scouring sluices across the channel. In the first few miles the canal crosses several minor streams which are admitted by means of inlets. At the sixth mile it is crossed by the Ranipur torrent, which is passed over it in a masonry superpassage 195 feet in breadth (Pl. XVI). In the tenth mile the Puthri torrent, having a catchment basin of about 80 square miles, or twice that of the Ranipur, is carried across the canal by a similar superpassage 296 feet in breadth. The sudden flood discharges in these torrents are of great violence, the Puthri discharging as much as 15,000 second-feet and having a velocity of about 15 feet per second.

In the thirteenth mile the canal encounters the Rutmoo torrent (Article 183), which has a slope of 8 feet per mile and a catchment basin half as large again as that of the Puthri. This torrent is admitted into the canal at its own level, and in the side of the canal opposite to the inlet is an open masonry outlet dam or set of escape sluices. Just below this level crossing is a regulating bridge by which the discharge of the canal can be readily controlled; thus in time of flood, by opening the sluices in the outlet dam and adjusting those in the regulator so as to admit into the canal the volume of water required, the remainder is discharged through the scouring sluices, whence it continues in its course down the torrent.

In the nineteenth mile, near Roorkee, the canal crosses the Solani river and valley on an enormous masonry aqueduct (Article 189). The Solani river in times of highest flood has a discharge of 35,000 second-feet and the fall of its bed is about 5 feet per mile. The total length of the aqueduct is 920 feet. The banks of the canal on the up-stream side are revetted by means of masonry steps for a distance of 10,713 feet, and on the down-stream side for a distance of 2,722 feet.

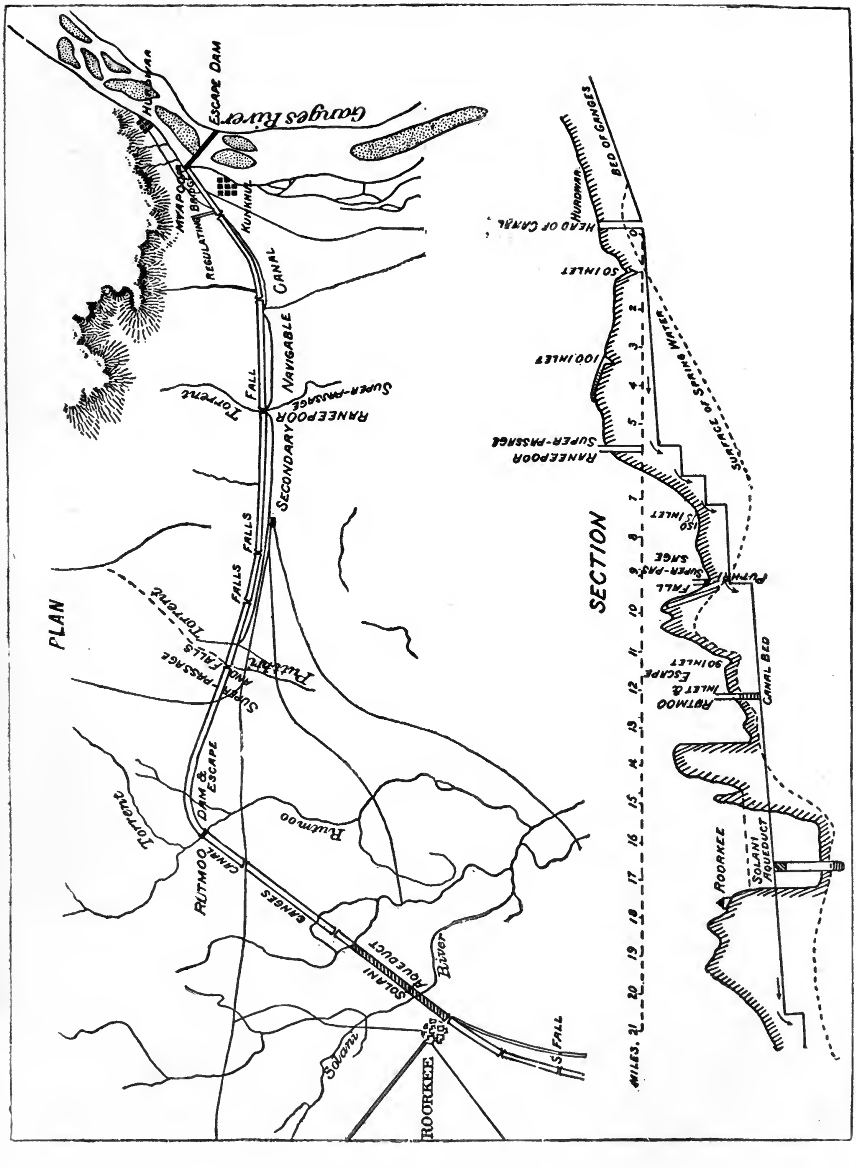


PLATE I.—PLAN AND CROSS-SECTION OF GANGES CANAL, HURDWAR TO ROORKEE, INDIA.

For $1\frac{3}{4}$ miles the bed of the canal is raised on a high embankment previously to its reaching the aqueduct, and for a distance of half a mile below it is on a similar embankment. The greatest height of the canal bed above the country is 24 feet (Pl. XIV). The aqueduct proper consists of fifteen arches of 50 feet span each. In addition to these great works there are in the first 20 miles of the canal five masonry works for damming minor streams and a number of masonry falls.

Beyond Roorkee the main canal follows the high divide between the Ganges and the west Kali Nadi, and continues in general to follow the divide between the Ganges and the Jumna rivers to Gopalpur, a short distance below Aligarh, where the main canal bifurcates, forming the Cawnpur and Etawah branches. The former tails into the Ganges river at Cawnpur and is 170 miles in length. The Etawah branch is also 170 miles long and tails into the Jumna river near Humerpur. The Vanupshahr branch leaves the main line at the fiftieth mile, and flows past the towns of Vanupshahr and Shahjahanpur. It formerly terminated at mile $82\frac{1}{2}$, emptying into the Ganges river; but it is now continued to a point near Kesganj, where it tails into the Lower Ganges canal. The first main distributaries are taken from both sides of the canal a short distance below Roorkee. The nature of the country offers abundant facilities for escapes from the canals, of which five are constructed on the main line, four on the Cawnpur branch, and three on the Etawah branch, besides numerous small escapes to the distributaries.

112. Example of Canal Alignment—Turlock Canal.—A typical American canal alignment is that of the Turlock canal, which is diverted from the Tuolumne river in California at a point where it emerges from the Sierras between high rocky canyon walls. For the first 5 miles the canal is built along steeply sloping hillside, and it crosses numerous drainage channels in its endeavors to surmount the bluffs bordering the river and gain the irrigable lands. The topography is so irregular that the first attempts which were made at diversion were unsuccessful. The present location was discovered only after a careful detailed topographic map had been made of the entire region, and from this the canal line was laid down (Fig. 9).

The headworks of the Turlock canal consist of a masonry dam which is constructed as a common diversion weir for the

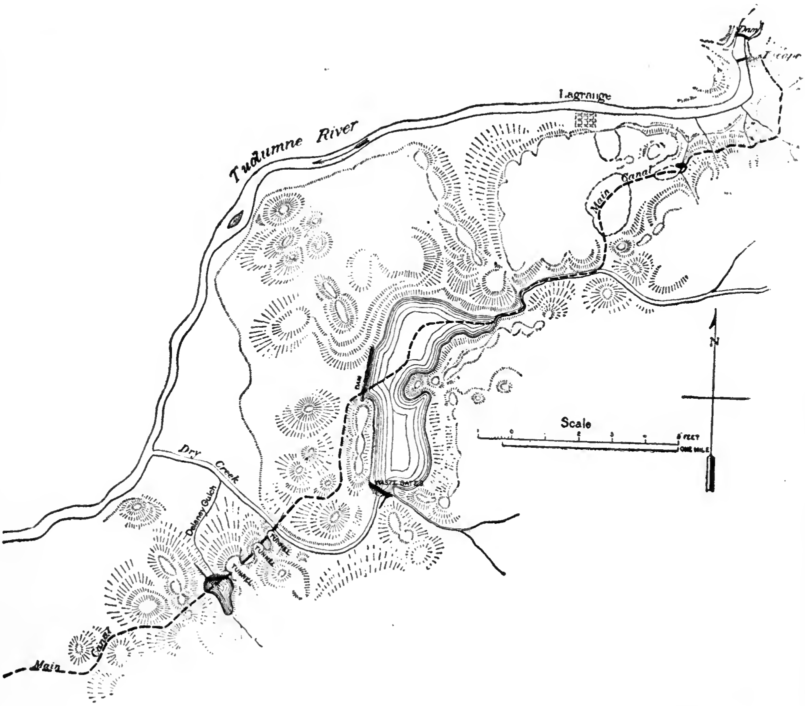


FIG. 9.—TURLOCK CANAL. PLAN OF DIVERSION LINE.

Turlock canal and the canal of the Modesto Irrigation district, which latter heads on the opposite or north bank of the river. This weir (Article 278) is located between high canyon walls, two miles above the town of La Grange, at a point where the abutments and foundation of the weir consist of firm homogeneous dioritic basalt, in which scarcely any excavation is required. The canal is diverted from the south bank of the river at a point about 50 feet above the end of the main weir. Owing to the great floods which occur in this narrow canyon the water may rise as much as 15 feet in an hour and the maximum height which it is estimated to reach above the sill of the canal is 16 feet. The pressure of this height of water on the regulator head would be so great as to materially increase the cost of its construction. Accordingly the canal heads in a tunnel 560 feet in length, blasted through the rock

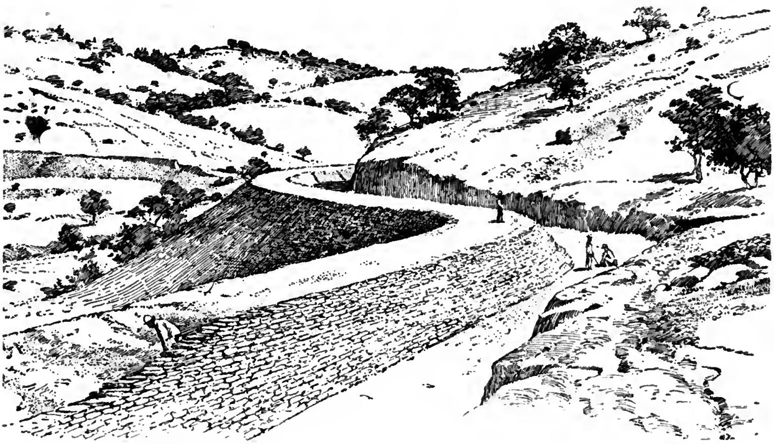


FIG. 10.—TURLOCK CANAL. VIEW OF SIDEHILL WORK.

of the canyon walls, and having no regulating apparatus at its entrance. Where it discharges into the open cut, which is the commencement of the canal, regulating gates and scouring or escape sluices are placed. The entrance tunnel is 12 feet wide at the bottom, 5 feet in height to the spring of the arch, above which it is semicircular with a 6-foot radius. Its slope is 24 feet per mile and it is excavated in a firm dioritic rock which requires no lining. The regulator in the canal head below the exit of the tunnel consists of six gates, each 3 feet wide in the clear and 12 feet in height. These gates are constructed of timber and iron, and slide on angle-iron bearings let into the rock and firmly set in concrete. The escape is set at right angles to the canal line heading immediately above the regulator, between it and the end of the tunnel, and tailing back into the Tuolumne river a short distance below the subsidiary weir. Like the regulator, the escape consists of six gates, each 3 feet wide in the clear, 12 feet high, and constructed of similar material and in like manner. It is estimated that whereas

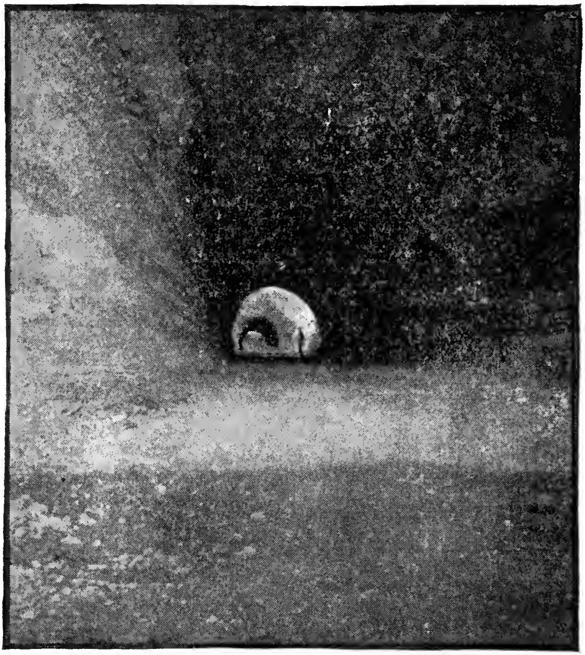


FIG. 11.—TURLOCK CANAL. VIEW IN TUNNEL.

a maximum flood of 16 feet over the sill of the tunnel will give a discharge in front of the regulator and escape of about 4000 second-feet with a velocity of 20 feet per second, the wasting capacity of the escape will be at least 6000 second-feet, thus fully insuring the canal against accident from this source.

Below the regulating gates the main canal proper begins, having a capacity of 1500 second-feet. For the first 6200 feet it is excavated in slate rock on a steep hillside (Fig. 10). It has a bed width of 20 feet, depth of water 10 feet, the upper rock slope being $\frac{1}{2}$ to 1, while the lower bank or downhill slope, where gullies are crossed, is built up with an inner slope of $\frac{1}{2}$ to 1 and is faced with 18 inches of dry-laid retaining-wall inside and outside, the interior of the bank consisting of a well-puddled earth core 12 feet in top width (Fig. 14). Where this portion of the canal is on ordinary sloping ground, not crossing gulches, its dimensions are the same but the inner face only has the 18 inches of riprapping the downhill slope of the bank consisting of dirt and other soil. The top width of the bank in such places is 5 feet and the puddle wall 5 feet in thickness. This portion of the canal line has a grade of 7.92 feet per mile, which gives a velocity of $7\frac{1}{2}$ feet per second.

At the end of this slate-rock work the canal empties into Snake ravine, up which the water of the canal runs for 940 feet. This is effected by constructing an earth dam across the mouth of the ravine just below the entrance of the canal, which raises the surface of the water so as to form a small settling reservoir and produces a flow up the course of the ravine for the distance above mentioned. The earth dam is 20 feet wide on top, 318 feet long on the crest, with slopes of 2 to 1 and a maximum height of 52 feet. This dam was partly constructed of material borrowed from its abutments and the canal excavation and partly by a silting process from material washed out of a hydraulic cut at the upper end of the ravine. This hydraulic cut, which is utilized as the canal bed, is 800 feet in length and 45 feet in maximum height, with slopes of 1 to 1 and a grade of 5 feet per mile. Owing to the abundance of water procurable this cut was more cheaply excavated by the hydraulic process than it could have been by other means. At the far end of the cut the canal enters an old hydraulic washing which is utilized for its channel for a length of 2380 feet, after which it enters a rock cut 860 feet long, with a maximum depth of 45 feet and a similar cross-section to the cut first described.

At the end of this rock cut the canal water is discharged into Dry creek, down which it flows for a distance of 6500 feet on a grade of 12 feet to the mile, and from which it is diverted by means of an earth dam 460 feet long. This dam has a maximum height of 23 feet with side slopes of 3 to 1, and is riprapped to a depth of 3 feet on its upper face. At its south end the dam abuts on sandstone rock in which a waste-way is cut 50 feet wide with its sill 4 feet below the crest of the dam, and which will discharge back into the creek 180 feet below the toe of the dam. Between the waste-way and the end of the dam is a waste-gate which it is intended shall be used in the time of freshets, for Dry creek has a maximum discharge of 4000 second-feet and as the freshets are quick and violent a large wasting capacity is necessary. These waste-gates are ten in number, each 3 feet wide in the clear and 10 feet in depth. They fall automatically outward or down-stream, being hinged at the bottom to a concrete floor laid on the bed-rock, and when raised they are attached by chains to the piers.

For about a mile below Dry creek the canal is excavated in heavy, sandy loam, in which it has a bed width of 30 feet, with slopes 2 to 1, a depth of 10 feet and a grade of 1½ feet per mile. At the end of this excavation the canal crosses Dry creek in a flume 62 feet in height and 450 feet long, after crossing which the canal enters a series of three tunnels, the cross-sections of which are nearly similar to that of the first tunnel, while they are excavated in a tufa and sandstone which will require no timbering. The first tunnel (Fig. 11) is 211 feet in length, the second 400 feet and the third 400 feet in length, while they are separated by short, open cuts excavated in hardpan and clay, which are respectively 250 and 300 feet in length. The last tunnel discharges into Delaney gulch, which is crossed by constructing a high bank or earth dam below the canal, the total length of which is 180 feet, its maximum height being 40 feet and its top width 20 feet. The volume of discharge of this gulch is so trifling that it was unnecessary to provide a waste-way or escape at this point. Immediately after crossing the gulch the canal enters a cut 8 feet in maximum depth, with the same cross-section and grade as the first cut and having a length of 3300 feet. The canal is then widened to a bed width of 35 feet and depth of 10 feet and is given a grade of 1 foot per mile. At the end of a mile and a half Peasley creek is crossed on a trestle and flume 60 feet in height and 360 feet long, the water-way on which is 20 feet wide and 7 feet in depth. This flume is provided with an escape constructed in its bottom and discharging into two small sloping flumes which lead the water down into the bed of Peasley creek (Article 168).

At the end of the flume the main canal is reached and traversed for a distance of 11 miles, in which are two rock cuts, each 3000 feet long and respectively 20 and 30 feet wide on the bottom, depth of water $7\frac{1}{2}$ feet and grade 5 feet per mile. The remainder of this length of the canal varies in cross-section according to the soil, but most of it has a bottom width of 70 feet and depth of water of $7\frac{1}{2}$ feet, slopes 2 to 1 and a grade of 1 foot per mile.

The main canal as outlined above consists for the 18 miles of its length of a purely diversion channel, the object of which is to bring the water to the irrigable lands included within the area of the Turlock district. At the terminus of this diversion line the canal begins at once to do duty by watering the lands, and below this point the main line is divided into four main branches, each of which has a bottom width of 30 feet, depth of water 5 feet, and grade of 2 feet per mile, their aggregate length being 80 miles. In addition to these main branches minor distributaries, having a total length of 180 miles, lead the water to each section of land. The discharge of the branches is so designed as to give a uniform velocity of $2\frac{1}{2}$ feet per second, in order that any matter carried in suspension will be held up until deposited on the agricultural lands instead of in the canals.

113. Slope and Cross-section.—These two quantities are nearly related and are interdependent one upon the other. Having determined the discharge required, the carrying capacity for this quantity can be obtained by increasing the slope and consequent velocity and diminishing the cross-sectional area; or by increasing the cross-sectional area and diminishing the velocity. The determination of the proper relation of cross-section to slope requires considerable judgment. If the material in which the excavation is to be made will permit, it is well to give a high velocity, as the deposition of silt and the growth of weeds are thus reduced to a minimum. A steep slope may result, however, in bringing the canal to the irrigable lands at such an elevation that it will not command the desired area. Again, it may be inadvisable to give too great a cross-section if the construction is in sidehill or in rock, or other material which is expensive to remove. Other things being equal, the correct relation of slope to cross-section is that in which the velocity will neither be too great nor too slow, and yet the amount of material to be removed will be reduced to a minimum. Where the fall will permit, the slope of the bed of the main canal should be less than that of the branches, which should be less than that of the distributaries and laterals, the object being to secure a nearly uniform velocity throughout the system, so that sedimentary matter carried in suspension may not be deposited until the irrigable lands are reached.

114. Limiting Velocity.—In order that the proper slope may be chosen, one which will produce a velocity that shall not cause silt to be deposited on the one hand, or erode the banks on the other, the amount of such velocities for different soils should be known. In a light, sandy soil it has been found that a surface velocity of from 2.3 to 2.4 feet per second, or mean velocities of 1.85 to 1.93 feet per second, give the most satisfactory results. It has been discovered that velocities of from 2 to 3 feet per second are ordinarily sufficiently swift to prevent the growth of weeds or the deposition of silt, and, other things being equal, this velocity is the one which it is most desirable to attain. In ordinary soil and firm sandy loam velocities of from 3 to 3½ feet per second are safe, while in firm gravel, rock, or hardpan the velocity may be increased to from 5 to 7 feet per second. It has been found that brickwork or heavy dry-laid paving or rubble will not stand velocities higher than 15 feet per second, and for greater velocities than this the most substantial form of masonry construction should be employed.

115. Grades for Given Velocities.—The grade required to give these velocities is chiefly dependent on the cross-sectional area of the channel. Much higher grades are required in small than in large canals to produce the same velocity. The velocity which is required being known, the grade can be ascertained from Kutter's or some similar formula. In large canals of 60 feet bed width or upwards, and in sandy or light soil, grades as low as 6 inches in a mile produce as high velocities as the material will stand. In more firm soil this grade may be increased to from 12 to 18 inches to the mile, whereas smaller channels will stand slopes of from 2 to 5 feet per mile, according to the material and dimensions of the channel.

116. Examples of Canal Grades.—On the Ganges canal, the bottom width of which is 170 feet and the depth 7 feet, a slope of 14 inches per mile given in sandy soil produces such a velocity that the current just ceases to cut the banks or to deposit silt, showing that this is the correct slope for that canal and material. In another portion of the same canal slopes of from 15 to 17 inches have been found too great, and much damage has been done to the banks. A velocity of 3 feet per second given to the Soane canals is found too great for the material, as much damage was caused by erosion. Careful observations of the slope on the Ganges canal show that a current apparently perfectly adjusted to light, sandy soil was produced by a surface velocity of about 2.4 feet per second, or a mean velocity of about 1.9 feet per second. In one of the distributaries in sandy soil having some clay in it a mean velocity of 1.93 feet per second caused slight deposits of silt, but did not permit the growth of weeds. On the western Jumna canal silt was deposited in small quantities with a velocity of from 2 to 2.75 feet per second, while in sandy soil the latter velocity was the highest permissible for non-cutting of the banks.

In the light, sandy loam soils of the San Luis valley in Colorado a slope of 6 inches to the mile given on the Citizens' canal has proven very satisfactory. So low a slope as this is possible, because the water is comparatively free of silt and there is little chance of its deposition, while the temperature is so low that there is little likelihood of the growth of weeds affecting the canal bed. In the gravelly clays through which the Turlock canal runs a satisfactory grade has been found to be 1.5 feet per mile, though the grade is changed on portions of this canal according to the character of the soil, until in the cut through loose shale near the canal head a grade of 7.9 feet per mile is given, producing a velocity of $7\frac{1}{2}$ feet per second with satisfaction. On the main line of the canal, the bed width of which is 70 feet and depth of water $7\frac{1}{2}$ feet and the soil a light alluvial loam, the grade adopted is one foot per mile. Perhaps the highest grade on any canal is that on a short portion of the Del Norte canal in Colorado, where the fall is 35 feet per mile through a rock cut. On several miles of this canal the grade is 8 feet per mile, but after it reaches the earth soil in the valley it is reduced to 1.2112.

117. Cross-sections.—The most economical channel is one with vertical sides and a depth equal to half the bottom width, but this form is only applicable to the firmest rock. The best trapezoidal form is one in which the width of the water surface is double the bottom width and equal to the sum of the side slopes. Such a cross-section as this, however, would call for an unusually compact material. In the interest of economy the side-slopes above water-level should be as steep as the nature of the soil will permit. As before shown, the cross-sectional area depends on the velocity and slope and their relation to the quantity of water to be discharged. The exact form of this cross-section is dependent on the topography and the material through which the canal passes. The greater the depth the greater will be the velocity and consequent discharge for the same form of cross-section.

Very large canals, such as some of those in India, have been given a proportion of depth to width similar to that of the great rivers. This proportion has been found to be most nearly attained when the bed width is made from 13 to 16 times the depth. In sidehill excavation the greater the proportion of depth to width the less will be the cost of construction (Art. 108.), and in all rock and heavy material it is desirable if possible to make the bottom width not greater than from 2 to 3 times the depth. Such a proportion as this, however, is rarely practicable. In a large canal, one for instance having a capacity of 2000 second-feet, with a velocity of 2 feet per second, the cross-sectional area should be 1000 square feet. If the proportion of 2 to 1 were maintained, this would call for a bed width of about 45 feet to a depth of 22½ feet. Such a depth as this unless in very hard material, is readily seen to be absurd, as the cost of construction would be greatly increased over that of a canal having a lesser depth. In this case a fair proportion would be 125 feet bed width to about 8 feet depth. A rule which has been proposed and which will prove fairly good on moderate sized canals, is to make the bottom width in feet equal to the depth in feet plus one, squared. This, however, will not apply to large canals and is not altogether true for any size of canal.

118. Form of Cross-section.—The cross-section of a canal may be so designed that the water may be wholly in excavation, wholly in embankment, or partly in excavation and partly in embankment (Fig. 12). The conditions which govern the choice of one of these three forms are dependent primarily on the alignment and grade of the canal, and secondarily on the character of the soil. For sanitary reasons it is sometimes desirable to keep a canal wholly in cutting, for if the material of which the banks are constructed is porous the water may filter through and stand about in stagnant pools on the surface of the ground. If the material is impervious to the passage of water and will form good firm banks, it may be well to keep the canal in embankment where possible, though this may necessitate the expense of borrowing material. In order to lessen the cost of construction, it is desirable, where the surface will permit, to keep a canal half in cut and half in fill, thus reducing to a minimum the amount of material to be moved. Ordinarily the surface of the ground is irregular and undulating, and in order that the grade may be maintained the canal will of necessity be sometimes wholly in cut and at others wholly in fill, and at others at all intermediate stages between these. Where the canal is wholly in embankment there is always considerable loss from leakage, and consequent

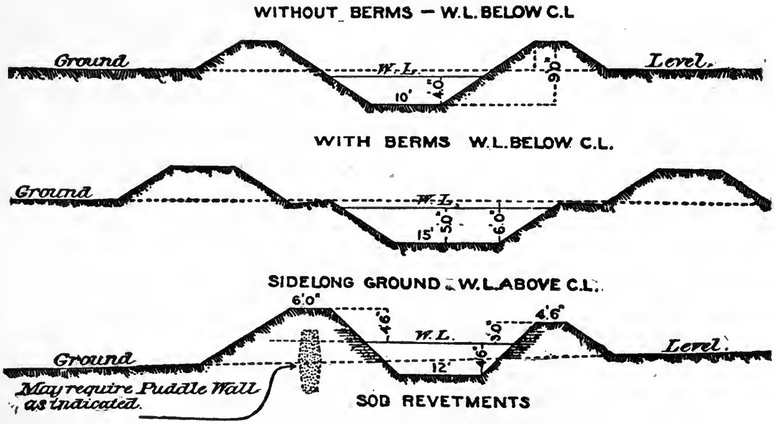


FIG. 12.—VARIOUS CANAL CROSS-SECTIONS.

danger of breaches. Where the canal is wholly in cut, care must be taken to discover the character of the soil in which the excavation is to be made, as rock may be encountered at a few inches below the surface, thus increasing the cost of excavation, or a sandy substratum may be discovered which would cause excessive seepage.

Most main canals follow the slope of the country on grade contours running around sidehill or mountain slopes. In such cases it is necessary to build an embankment on one side only, when the cutting will be entirely on the upper side. If there is a gentle slope on the upper side, and consequently an embankment on that side, it is desirable to run drainage channels at intervals from this embankment to keep the water from making its way through it to the canal. These drainage channels may be taken through the embankment into the canal, or may be led away to some natural watercourse.

In designing the cross-section of a canal it may be desirable to give a berm, and this may be above or below the water-level (Fig. 12). Ordinarily the berm is left at a level with the ground surface, though it may be constructed in excavation or embankment,—an unusual practice, however. The chief object of the berm is to provide against the destruction of the slopes in the lower part of the banks by giving a terrace or bench on which the upper bank may slide, providing it fails to maintain the slope originally given; it also serves in some cases as a tow-path or foot-path. The width of berm varies between 2 and 6 feet, and it is common to change the slopes at the point of junction between cut and embankment, making the slope of the latter a little flatter than that of the former.

119. Side Slopes and Top Width of Banks.—In large canals it is always desirable to have a roadbed on at least one bank, and the width of this will determine the top width of the bank. The inner surfaces of the canal are usually made smooth and even, while the top is likewise made smooth, with a slight inclination to the outward to throw drainage away from the canal. The inner slopes of the banks vary in soil from 1 on 1 to 1 on 4, according to the character of the material. In firm clayey gravel or hardpan slopes of 1 on 1 are sufficiently substantial for nearly any depth of cutting or embankment. On the Turlock canal in California is a cut 80 feet in depth with side slopes of 1 on 1, while on the Bear river canal in Utah are similar slopes in disintegrated shale in coarse gravel. In ordinary firm soil mixed with gravel or coarse loamy gravel slopes of 1 on $1\frac{1}{2}$ are sufficient. In firm soil and slightly clayey loam slopes of 1 on 2 may be required; on lighter soils these slopes may be increased until the lightest sand is reached, when slopes of 1 on 4 may be necessary.

The top width of the canal bank is generally from 4 to 10 feet, according to the material, depth, and whether or not the water is in embankment. If there is to be no roadway on the top of the embankment, and the surface of the water does not rise more than a foot or so above the foot of the embankment, a top width of 4 feet is sufficient. Where the depth of water on the embankment is greater, this width should be 6 or 8 feet, and if the soil is light it should be at least 10 feet. It is sometimes necessary to build a puddle wall in the embankment, or to make a puddle facing on its inner slope where it is particularly pervious to water. The same effect is obtained by sodding or causing grass to grow on the bank. It may be well to puddle the entire bank during construction by laying and rolling it in layers. The carrying capacity of a canal should be so calculated that the surface of the water when in cut shall not reach within one foot of the top of the ground surface. In fill the depth of water carried should be such that the surface shall not rise higher than within $1\frac{1}{2}$ feet of the top of the bank, while if the fill is great it is often unsafe to let the water rise within 2 feet of the top of the bank.

120. Cross-section with Subgrade.—In the light soils of the San Luis valley in Colorado and in Kern valley in California it has been found advantageous to use a different form of cross-section than that above described. Experience in the regions above cited has shown that the subgrade produces a form approaching that of the ellipse. This cross-section tends to keep the current in the centre of the channel, and to keep up its flow with the least exposure to friction and seepage when the volume of water in the canal is low. The subgrade (Fig. 13) is given by practically designing the canal as

FIG. 13.—CROSS-SECTION OF CALLOWAY CANAL SHOWING SUBGRADE.

if it were to have a trapezoidal cross-section with berm, and then evening off the slope by removing the berm and continuing the slope from the bottom of the canal toward the centre to a depth or subgrade of from 1 to 2 feet below the original bed of the canal. In such construction as this it has sometimes been found desirable to give the bank practically no top width, simply rounding it off from the inner to the outer surface, where the waste is carelessly scattered, allowing the soil to assume its natural slope.

121. Shrinkage of Earthwork.—It is well known that when soil which has been removed from an excavation is formed into embankment it settles or shrinks in volume. That is to say, the excavated and embankment soil occupies a less space than it did in the ground; while, on the contrary, rock or loose stone occupies a greater space, depending on the dimensions of the fragments. The percentage of this shrinkage differs according to different soils. The following list gives an idea of the amount of this shrinkage for different soils:

Sand, about 10 per cent; in other words, after excavation sand will ultimately occupy 10 per cent less space than it did in its natural bed.

Sand and gravel shrink 8 per cent.

Earth, loam, and sandy loam shrink 10 to 12 per cent.

Gravelly clay shrinks 8 to 10 per cent.

Puddled clay and puddled soil shrink 20 to 25 per cent.

Rock expands or increases in volume from 25 per cent in the case of small or medium fragments and road-metalling to 60 or 70 per cent in large fragments carelessly thrown.

122. Cross-section in Rock.—In firm rock it is desirable to make the proportion of depth to width about as 1 to 2,

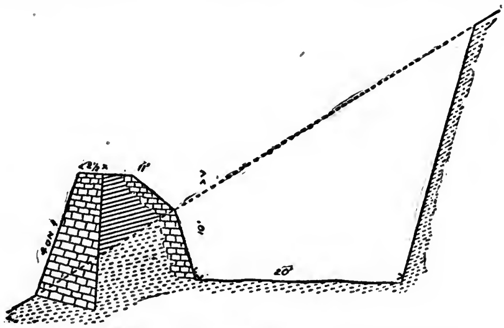


FIG. 14.—ROCK CROSS-SECTION. TURLOCK CANAL.

with side slopes of about 4 on 1. In less firm rock lighter slopes and a less proportional depth are desirable. In friable shale, as on the Turlock canal in California, a different cross-section is desirable (Fig. 14). In this instance a retaining-wall of hand-placed stones, with an outer slope of 4 on 1 and a top width of 2½ feet, is built on the lower side. Inside this is a puddled earth bank, riprapped on the water surface with 10 inches in thickness of loose stone. The upper or excavated slope is about 2 on 1, the depth 10 feet, and the bed width 20 feet. On the Bear River canal in Utah, the cross-section

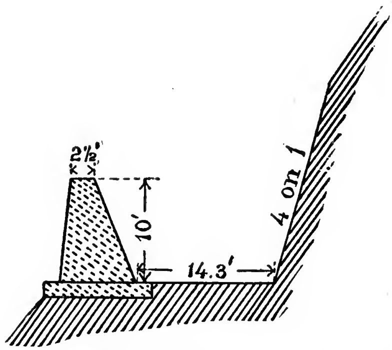


FIG. 15.—ROCK CROSS-SECTION, BEAR RIVER CANAL.

shown in Fig. 15 was given in order to avoid too much excavation in extremely rocky sidehill.

CHAPTER XII.

HEADWORKS AND DIVERSION WEIRS.

123. Location of Headworks.—The headworks of a canal are generally placed where the stream emerges from the hills. At such a point the slope of the country and of the stream is steep, making it possible to conduct a canal thence to the irrigable lands with the shortest diversion line. Moreover, the width of the channel of the stream is generally contracted, and it flows through firm soil or rock, thus permitting a reduction in the length of the weir and in the cost and character of its construction.

When the volume of flood water occurring in the stream is great it is sometimes necessary to locate the headworks at a point where the width between banks is greatest, in order that the depth of water flowing over the weir may be reduced to a minimum and danger of its destruction reduced accordingly. While such a location may be the most permanent, it is also most costly for construction. The site of the headworks should be such that the most permanent weir can be constructed at the least cost, and yet they should be so located that the diverting canal can be conducted thence to the irrigable lands at a minimum cost. The location of the headworks high up on the stream is usually antagonistic to the last object, since it generally results in the canal having to encounter heavy rock work and difficult construction until it gets away from the river banks.

124. Character of Headworks.— The headworks of a canal consist—

1. Of the diversion weir, in which is usually built:
2. A set of scouring sluices;
3. Of a regulator at the head of the canal for its control;
4. Of an escape for the relief of the canal below that point.

Sometimes to these are added river training or regulating works for the protection of the banks of the stream above and below the obstruction formed by the headworks. Too careful attention cannot be given to an examination of the stream at the point of diversion. Soundings and borings should be made to ascertain the depth of water and character of the foundation. The velocity of the stream and its flood heights should be studied, as should the material of which the banks are composed. Where possible, a straight reach in the river should be chosen for the location of the headworks in order that the stream shall have a direct sweep past them, thus reducing to a minimum the deposition of silt in front of the regulating gates. If possible, a point should also be chosen where the velocity in the river will not exceed that in the canal, so that the deposition of silt shall be further reduced.

There has been too great a tendency in American construction to build works of a temporary and transient character. The headworks of a canal are the most vital portions of its mechanism; they are to a canal system what a throttle-valve is to a locomotive. Through them the permanency of the supply in the canal is maintained, and any injury to them means paralysis to the entire system. They should therefore be most substantially and carefully designed throughout. The employment of wood is altogether too common in the United States. It is very well to make use of wood as a temporary makeshift until money and time can be found for substituting more substantial material. It may be generally laid down as a principle, however, that only iron and masonry should enter into the construction of the headworks. It is impossible to form wood, with the addition of little or no iron or masonry, into permanent and substantial headworks. The best and most abundant examples of substantial headworks must still be sought in Europe and India.

In some cases it has been found unnecessary to construct diversion weirs as a part of the headworks of a canal. This has been the case especially where the discharge of the stream was great relative to the discharge of the canal, and only when a portion of the water in the stream was required. Thus, on the Central Irrigation District canal in California no diversion weir is required. The canal heads in a simple cut, its bed being a few feet below the lowest water-level in the Sacramento river. At the head of the Ganges and Jumna canals in India there are no permanent diversion works, the water being turned into the canal head by means of temporary structures of bowlders, or by means of training the water of the river so that it shall flow directly against the canal head.

125. Diversion Weirs.—In this book the word *weir* as distinguished from *dam* is generally employed to mean a structure intended either for the impounding or diversion of water and over which flood waters may safely flow. Thus weirs are usually built at the heads of canals for the diversion of the waters of the streams into their heads, while the surplus water is permitted to flow over the weir and to pass on down the stream. In some cases, however, dams over which it would be unsafe to permit flood waters to pass are used for the purpose of diversion, and a wasteway is constructed at one end of the dam for the passage of surplus waters.

A weir across a stream is analogous to a bar and should be located and treated as such. If it is placed at the widest part of the stream the cost of construction may be increased. In the great rivers of India where diversion is made in the level and sandy plains below the hills and where permanent foundations cannot be obtained, weirs have generally been placed in the broadest reaches of the streams. This is the case at Okhla at the head of the Agra canal, and at Narora at the head of the Lower Ganges canal. In our own country diversion for canals has generally taken place in the foothills, and accordingly the narrower portions of the streams have been chosen for this purpose.

126. Classes of Weirs.—Weirs may be divided into two classes according to the mode of building their foundations. Thus they may rest directly on some permanent material; or they may rest on some unstable material, as quicksand, gravel, or clay, in which case an artificial foundation of piles, caissons, or wells or blocks must be constructed. Where, in western practice, a firm foundation has not been found piling has usually been employed. In India and Egypt wells or blocks are employed for foundations in unstable material.

These consist of rectangular boxes or cylinders of brick, which rest on a sharp cutting edge, and from the interior of which the earth is excavated as the well sinks. After it has reached a suitable depth it is filled in with concrete, the whole depending for its stability chiefly on the friction of its sides against the surrounding material.

The most convenient classification of diversion weirs is according to the construction of their superstructures. These may be—

1. Temporary brush or bowlder barriers;
2. Rectangular walls of sheet and anchor piles filled with rock or sand;
3. Open weirs;
4. Wooden crib and rock weirs;
5. Masonry weirs.

127. Brush and Bowlder Weirs.—The simplest and crudest form of weir is the brush and gravel barrier, which was originally used by the Mexicans and is still employed in the West on minor streams. These weirs are formed by driving stakes across the channel and attaching to them fascines or bundles of willows from three to six inches in diameter at the butts, which are laid with the brush end up-stream, and are weighted with bowlders and gravel. More willow or cottonwood branches are laid on the top of these and again weighted with bowlders, this operation being continued until the structure is built to a height of three or four feet. Such structures are of the crudest character and can be built without any engineering knowledge or supervision.

128. Rectangular Pile Weirs.—These have been employed in wide sandy rivers like the Platte, in Colorado. They consist of a double row of piling driven into the river bed, the two rows being about 6 feet apart, and the piles about 3 feet apart between centres. Between these is driven sheet piling to prevent the seepage or travel of water through the barrier, and the upper portion of the structure is planked so as to form a rectangular wall the interior of which is filled in with gravel, sand, etc. Such walls are usually low, rarely exceeding 8 feet in height, and after the upper side is backed with the silt deposited from the stream they form substantial barriers which may last for many years. Such structures cannot be employed where the flood height is great, as they would soon be undermined unless substantial aprons were constructed.

129. Open and Closed Weirs.—Diversion weirs may again be classified as open or closed. A closed weir is one in which the barrier which it forms is solid across nearly the entire width of the channel, the flood waters passing over its crest. Such weirs have usually a short open portion in front of the regulator known as the "scouring sluice," the object of which is to maintain a swift current past the regulator entrance, and thus prevent the deposit of silt at that point. An open weir is one in which scouring sluices or openings are provided throughout its entire length.

The advantage of the closed weir is that it is self-acting, and if well designed and constructed requires little expense for repairs or maintenance. It is a substantial structure, well able to withstand the shocks of floating timber and drift; but it interferes with the normal regimen of the river, causing deposit of silt and perhaps changing the channel of the stream. Open or scouring sluice weirs interfere little with the normal action of the stream, and the scour produced by opening the gates prevents the deposit of silt, while their first cost is generally less than that of closed weirs.

The closed weir consists of an apron properly founded and carried across the entire width of the river flush with the level of its bed, and protected from erosive action by curtain-walls up and down stream. On a portion of this is constructed the superstructure, which may consist of a solid wall or in part of upright piers, the interstices between which are closed by some temporary arrangement. This portion of the weir is called the scouring sluice. The apron of the weir should have a thickness equal to one half and a breadth equal to three times the height of the weir above the stream bed. During floods the water backed against the weir acts as a water cushion to protect the apron, and as the flood rises the height of the fall over the weir crest diminishes, so that with a flood of 16 feet over an ordinary weir its effect as an obstruction wholly disappears.

An open weir consists of a series of piers of wood, iron or masonry, set at regular intervals across the stream bed and resting on a masonry or wooden floor. This floor is carried across the channel flush with the river bed, and is protected from erosive action by curtain-walls up and down stream. The piers are grooved for the reception of flashboards or gates, so that by raising or lowering these the afflux height of the river can be controlled. The distance between the piers varies between 3 and 10 feet, according to the style of gate used. If the river is subject to sudden floods these gates may be so constructed as to drop automatically when the water rises to a sufficient height to top them. It is sometimes necessary to construct open weirs in such manner that they shall offer the least obstruction to the waterway of the stream. This is necessary in weirs like the Barage du Nil below Cairo, Egypt, or in some of the weirs on the Seine, in France, in order that in time of flood the height of water may not be appreciably increased above the fixed diversion height. Should the height be increased in such cases the water would back up, flooding and destroying valuable property in the cities above. Under such circumstances open weirs are sometimes so constructed that they can be entirely removed, piers and all, leaving absolutely no obstruction to the channel of the stream, and in fact increasing its discharging capacity, owing to the smoothness which they give to its bed and banks.

130. Open Frame or Flashboard Weirs.—A form of cheap open weir which has been commonly constructed in the West is the open wooden frame and flashboard weir. This type of structure is used only on such rivers as have unstable beds and banks, where any obstruction to the ordinary regimen of the stream would cause a change in its channel. It consists wholly or in part of a foundation of piling driven into the river bed, upon which is built an open framework closed by horizontal planks let into slots in the piers. These weirs are constructed of wood, and are temporary in character, their chief recommendation being the cheapness with which they can be built in rivers the beds of which are composed of a considerable depth of silt or light soil.

Two varieties of this weir are in common use. One (Fig. 16), which has been employed at the heads of the Del

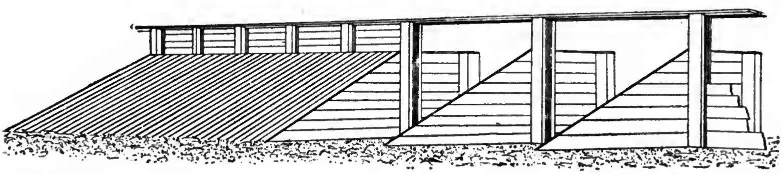


FIG. 16.—OPEN WEIR. MONTE VISTA CANAL.

Norte, Monte Vista and other canals in the San Luis valley of Colorado, is partly open and partly closed. An earth bank or dam is built for a portion of the way across the stream and of such height that it will not be topped by floods. The remainder of the weir consists of a framework of rough-hewn logs founded on piles, the abutments of which are protected by wooden planking built against the earthen dam. The openings between the frames or piers are about 6 feet apart, and the crest of the weir rarely exceeds 5 feet in height above the normal water surface. Between the piers horizontal planks or flashboards can be inserted one at a time, thus

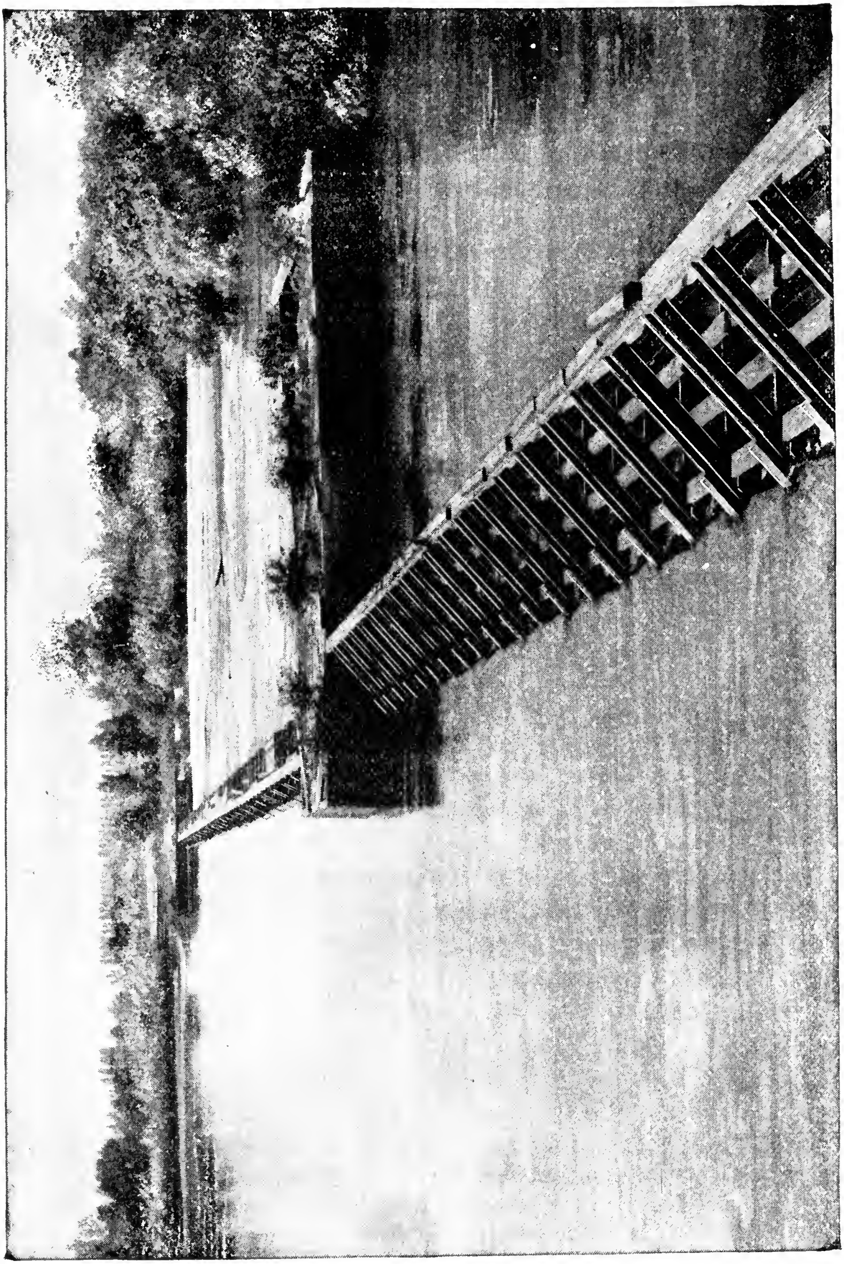


PLATE II.—KERN RIVER DIVERSION WEIR. HEAD OF CALLOWAY CANAL.

closing the waterway to any desired extent up to the level of the weir crest.

A more common and finished type of frame or flashboard weir is that employed on the Kern river in California, at the heads of the canals in that neighborhood. An example of these is the weir at the head of the Calloway canal (Fig. 17), which consists of 100 bays, each separated by a simple open triangular framework of wood founded on piles, the width of

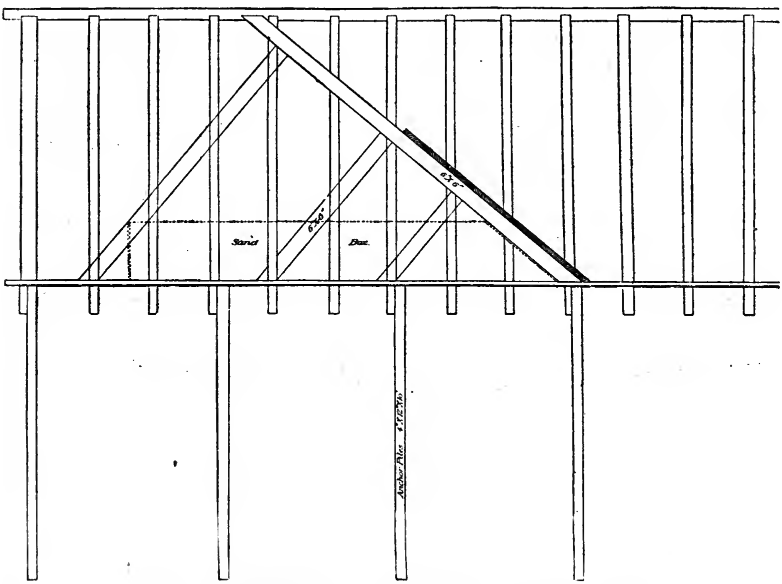
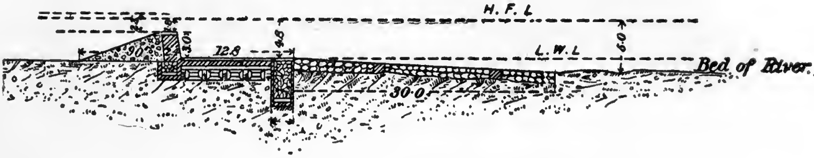


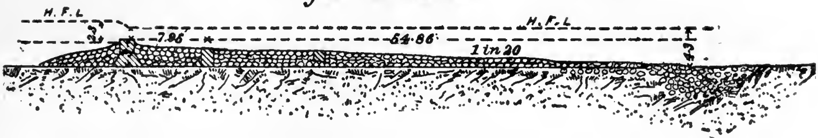
Fig. 17.—Cross-section of Open Weir, Calloway Canal.

each opening or bay being 4 feet. In constructing this weir the area to be built upon was inclosed in sheet piling and covered with a floor placed $2\frac{1}{2}$ feet below the bed of the stream. Above this floor is a second floor, about 2 feet in height, the walls forming compartments which are filled with sand, thus making a sand box apron, on which the waters fall. This apron is carried up and down stream for a distance of about 10 feet in each direction. The weir proper is formed of frames or trusses of 6 by 6 inch timber, placed transversely

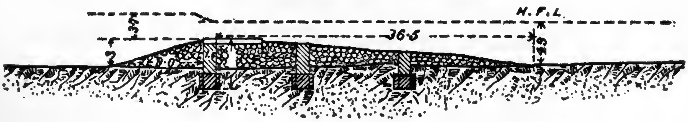
NARORA WEIR—LOWER GANGES CANAL.
Length 1260 metres



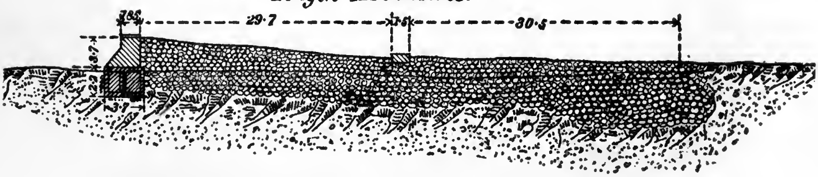
OKHLA WEIR—AGRA CANAL.
Length 743 metres



DEHREE WEIR—SOANE CANAL.
Length 3825 metres



BEZWARA WEIR — KISTNA CANAL.
Length 1150 metres.



GODIVERY WEIR.
Length 6274 metres.

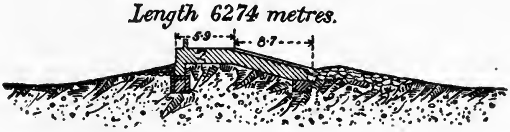


PLATE III.—CROSS-SECTIONS OF INDIAN WEIRS.

4 feet apart. These frames consist of 2 pieces, the up-stream piece being 7 feet 2 inches long and set at an angle of 38 degrees, while the other supports it at right angles and is 5 feet 4 inches long. The lower ends of these rafters thrust against two pieces of 6 by 2 inch timber running the whole length of the weir and nailed to the flooring. These frames are supported directly on anchor piles, one at each end joiced into the framing. These trusses are kept in vertical position by means of a footboard running transversely the entire width of the stream. On the up-stream face of the trusses planks or flashboards which slide between grooves formed by nailing face-boards on the trusses are laid on to the required height. This weir is 10 feet in height above the wooden floor, which is flush with the river bed.

131. Open Masonry Weirs, Indian Type.—A substantial form of open masonry weir is that generally constructed on Indian rivers, where the banks and bed are of sand, gravel, or other unstable material. These weirs generally rest on shallow foundations of masonry, in such manner that they practically float on the sandy beds of the streams. The foundation of such a weir is generally of one or more rows of wells sunk to a depth of from 6 to 10 feet in the bed of the river, the wells and the spaces between the rows of wells being filled in with concrete, thus forming a masonry wall across the channel. This form of construction is illustrated in Pl. III, which exhibits several different types of such works. The weir at the head of the Soane canals, which is typical of this class of structure, consists of three parallel lines of masonry running across the entire width of the stream, and varying from 2½ to 5 feet in thickness. The main wall, which is the upper of the three and the axis of the weir, is 5 feet wide and 8 feet high, and all three lines of walls are founded on wells sunk from 6 to 8 feet in the sandy bed of the river. Between these walls is a simple dry stone packing raised to a level with their crests, thus forming an even upper surface. The up-stream slope is 1 on 3, and the down-stream slope 1 on 12, the total length of this lower slope being 104 feet, while the total height of the weir including its foundation is 19.3 feet.

The Soane weir has a total length across stream of 12,480 feet, of which 1494 feet consists of open weir disposed in three sets of scouring sluices (Fig. 18), one in the centre and two

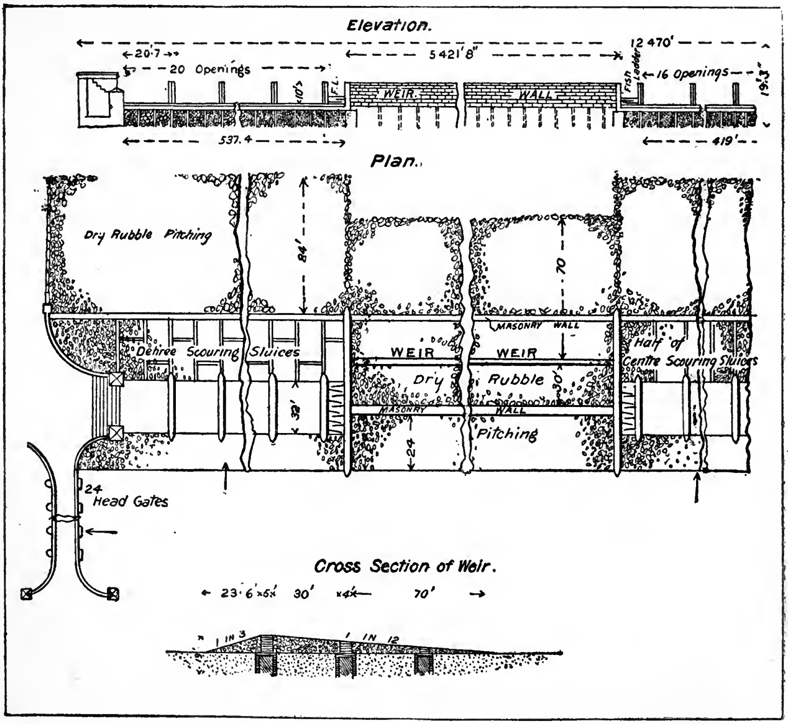


FIG. 18.—HALF-ELEVATION AND PLAN, AND SECTION OF SOANE WEIR, INDIA.

adjacent to either bank and in front of the regulating gates at the head of the canals. These scouring sluices consist of three parts,—the foundation, the floorway or apron, and the superstructure. The floor is deep and well constructed of substantial masonry, and is continued for a short distance above the weir and for a considerable distance below it. It is 90 feet wide parallel to the river channel, and is founded on wells, the ashlar pavement of the floor being 15 inches thick in the bottom of the scouring sluices between the piers, and 9 inches thick over the remainder of the apron. Up-stream from the sluice floor for a distance of 25 feet is a line of wells sunk to a depth of 10 feet as a curtain-wall to the apron. Twenty-five feet down-stream from the flooring of the sluices is a similar line of wells formed into a wall, and the spaces between these two curtain-walls and the main ashlar flooring of the sluice-way is packed with dry-laid bowlders and rubble covered with a pavement of masonry 9 inches in thickness. Down-stream from the lower curtain-wall a paving of large bowlders stretches for 50 feet further, the whole of this sluice floor parallel to the river channel being 200 feet in length. This is a typical floor to an Indian open weir or sluiceway, on top of which, in line with the centre of the crest of the weir, are built up masonry piers at regular intervals of from 6 to 12 feet apart, grooved for the reception of planks or flashboards, or closed with lifting or automatic drop-gates.

A peculiar form of open weir is that constructed at the head of the Sidhnai canal in India. At the point where the weir is built the bed of the river gives a good clay foundation for a short distance from either bank, while in the centre of the channel the bed is of sand for a considerable depth. Sheet piling 10 feet long was driven into the sandy bed of the river to prevent excessive percolation. On these piles (Fig. 19) rests a series of piers which support masonry arches, the piers being 16 feet between centres and filled between with clay. Above this masonry arch is built a continuous wall across the entire width of the streem from 4 to 6 feet wide on top and from $3\frac{1}{2}$ to $8\frac{1}{2}$ feet in height. Over this wall, parallel to the channel of the river, is built a masonry flooring, the upper slope of which is 1 on 3, while its lower slope varies between 1 on 5 and 1 on 10, according as it is near the centre or ends of the weir. The total width of this floor parallel to the channel of the stream is 12 feet above the axis of the weir and 40 feet below it, the lower toe terminating in a series of wells. On top of this flooring are erected a series of piers 23 feet apart between centres, and projecting $2\frac{1}{2}$ feet up-stream from the central wall and 9 feet down-stream, their total length parallel to the channel being $15\frac{1}{2}$ feet and their width on top 6 feet. The crests of these pillars are $6\frac{1}{2}$ feet in height above the crest of the floor, while the total height of the weir above the summit of the pile foundation is about 21 feet. It will thus be seen that this weir offers a clear waterway across the entire channel, obstructed only by the piers, which are $6\frac{1}{2}$ feet above the stream-bed. The openings between these piers are closed by means

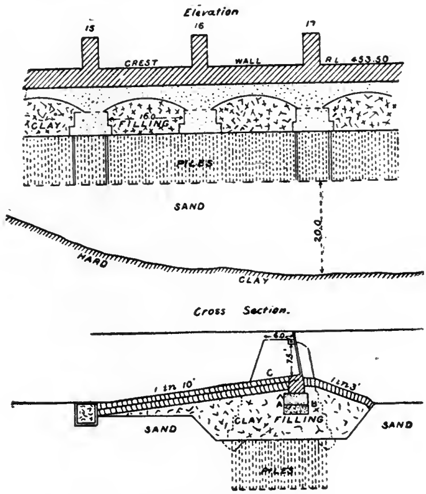


FIG. 19.—ELEVATION AND CROSS-SECTION OF SIDHNAI WEIR, INDIA.

of needles, which consist of a heavy beam laid along the crest wall from pier to pier, against which rest wooden sticks or needles inclined at a slight angle. These needles are each $7\frac{1}{2}$ feet long by 5 inches wide and $3\frac{1}{2}$ inches in thickness, and are laid along the upper face close together so as to form a close paling or barrier when in place.

The weirs on the river Seine in France differ materially from the open Indian weirs. They consist of a series of iron frames of trapezoidal cross-section, somewhat similar in shape to the frames of the open wooden flashboard weirs of California. On these frames rest a temporary footway, and on their upper side is placed a rolling curtain shutter or gate which can be dropped so as to obstruct the passage of water across the entire channelway of the stream, or can be raised to such a height as to permit the water to flow under them. In

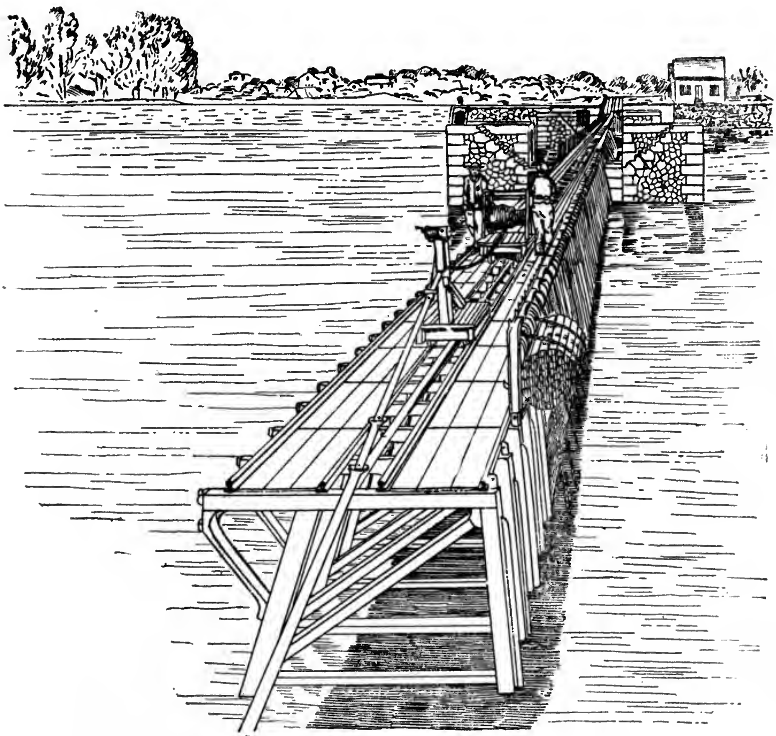


FIG. 20.—VIEW OF OPEN WEIR ON RIVER SEINE, FRANCE.

times of flood the curtains can be completely raised and removed on a temporary track to the river banks, the floor and track can then be taken up, leaving nothing but the slight iron frames, which scarcely impede the discharge of the river and permit abundant passageway of the floods over, around, and through them (Fig. 20).

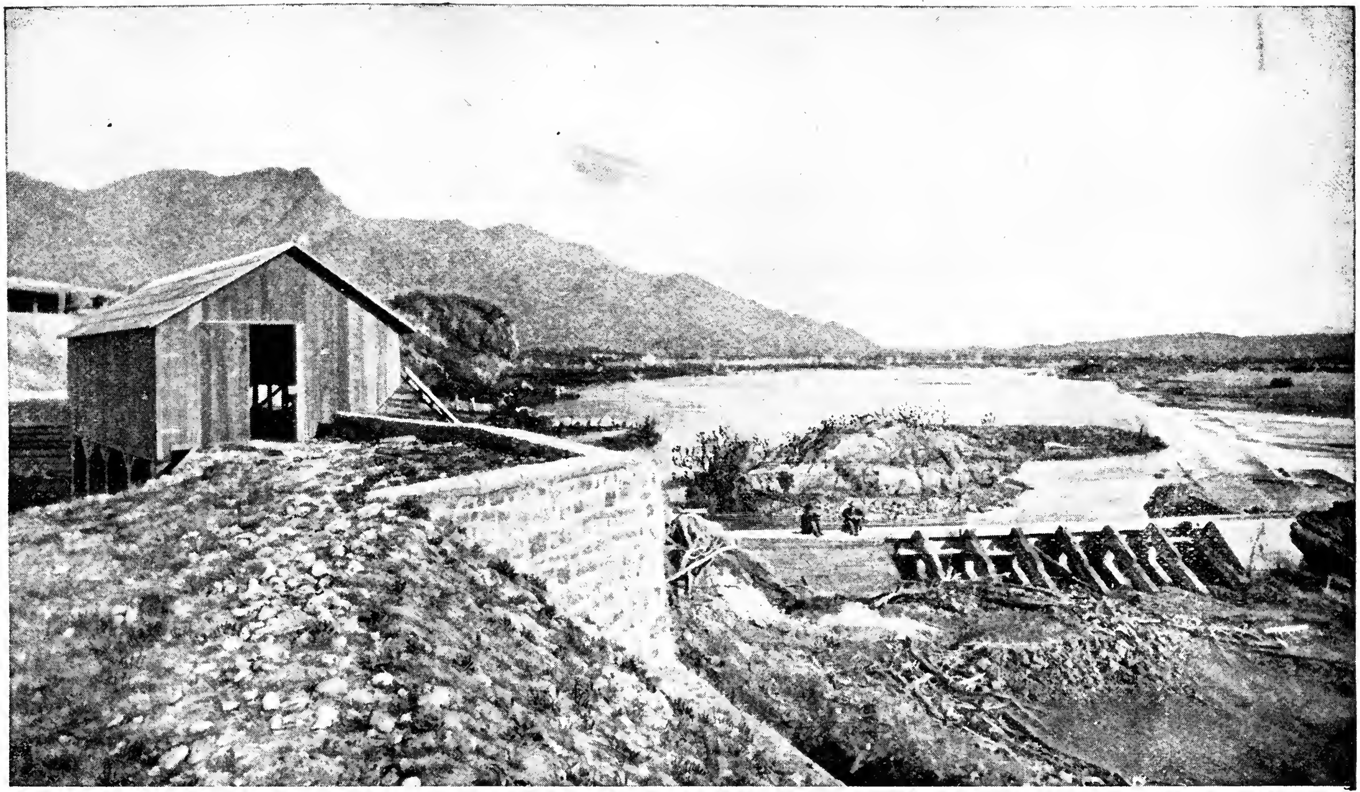


Plate IV.—View of Weir and Scouring Sluices, Head of Arizona Canal.

132. Wooden Crib and Rock Weirs.—This type of weir is generally built where the bed and banks of the river are of heavy gravel and bowlders or of solid rock, and it may be employed for diversions of greater height than is possible with open weirs. Crib weirs consist essentially of a framework of heavy logs, drift-bolted or wired together, and filled with broken stone and rocks to weight and keep them in place. Such works may be founded by sinking a number of cribs one on top of the other to a considerable depth in the gravel bed of the stream, or they may be anchored by bolting them to solid rock. They may consist of separate cribs built side by side across the stream and fastened firmly together as in the case of the weir at the head of the Arizona canal, or they may be made as one continuous weir, as in the case of the structures at the heads of the Kraft Irrigation District canal in California, and the Bear river canal in Utah. After its completion the weir is planked over on its exposed faces and forms one continuous wall across the channel of the stream.

The weir at the head of the Arizona canal (Pl. IV) consists of crib boxes of hewn logs about 9 by 9 feet, the logs being fastened with drift-bolts, and the whole wired together and filled with rocks. This weir was constructed by laying mudsills in a trench excavated in the bed of the stream, and on these was built up the cribwork. In the central and deepest portion of the river channel the weir was sunk to a depth of 33 feet in the gravel bed of the stream, while its crest is everywhere 10 feet above mean low-water. The base of this weir in the deepest part of the channel is from 36 to 48 feet in width parallel to the course of the stream, and the mudsills, which are 8 by 12 by 48 feet, were wired together with 1-inch cable to act as a hinge between the sections. Each section was floored and cribbed and built up as a box, only the alternate sections being closed at first. the others being left open for the passage of water. These openings were planked on the bottoms and sides. The alternate sections were closed by dropping timbers into place. Instead of bringing up the face batter, as is ordinarily done, the weir was built in four sections transversely to its axis (Fig. 21). The first section consisted of two rows of cribs, the upper faces of which were given a slight

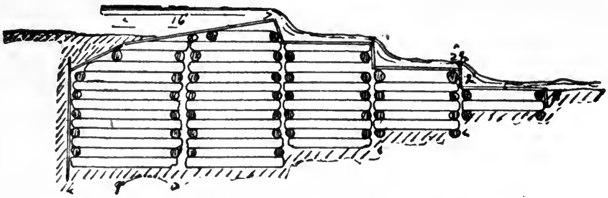


FIG. 21.—CROSS-SECTION OF ARIZONA WEIR.

batter, and on them silt has since deposited and helps to weight the structure. Immediately below the crest and with its upper surface $2\frac{1}{2}$ feet lower is another row of cribs which drop off $2\frac{1}{2}$ feet to the third row of cribs, below which at a distance of $2\frac{1}{2}$ feet still lower are a couple of depths of swinging cribs wired to the projecting part of the dam. The whole of this upper surface is planked over and forms a series of steps upon which the water falls, its force being thus broken.

The crib weir at the head of the Bear river canal in Utah

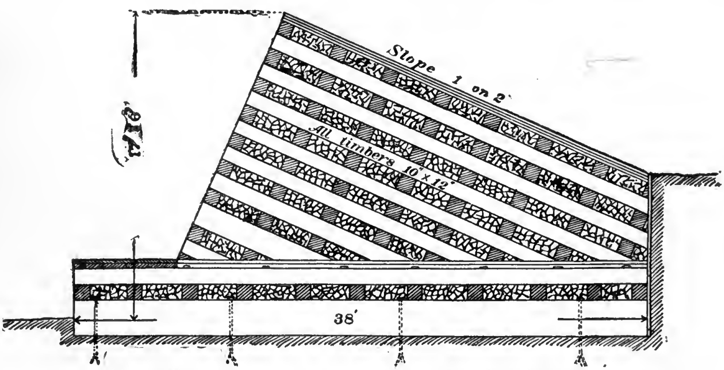


FIG. 22.—CROSS-SECTION OF BEAR RIVER WEIR.

is 370 feet in length on its crest, which is $17\frac{1}{2}$ feet in maximum height above the river bed, while the greatest width at its base parallel to the course of the stream is 38 feet (Fig. 22). The up-stream face has a slope of 1 on 2 while, that of the down-stream face is 1 on $\frac{1}{2}$, the water falling on a wooden apron anchored by bolts to the bed-rock of the river. This weir consists of heavy 10 by 12 timbers, drift-bolted to the rock and firmly spiked together. The interstices between these timbers are filled with broken stone, and it is backed by silt deposited from the river.

Sometimes crib weirs are founded on piles, as in the case of the weir across Stony creek, at the head of the Kraft Irrigation District canal. This is composed of timber cribs sheathed with 3 inches of plank on the up-stream face and 7 inches on the lower face, and it rests on two rows of piles driven across the entire width of the stream, 6 feet apart between centres. One of these rows of piles is driven to a depth of 12 feet under the toe of the apron, while 8 feet below this is a row of sheet piling and 22 feet above the upper row of piles is another row of sheet piling, both of these being of 4-inch double piling 8 feet in length or driven to bed-rock.

The crib weir across the Connecticut river at Holyoke, Mass., is about 1017 feet in length, its ends abutting against heavy masonry wings at either extremity. Between these the crib weir is composed of 12 by 12 timbers, built in such a way as to present on the upper face a surface of planking inclined at an angle of 21 degrees to the horizon. These timbers are separated by transverse timbers at distances of 6 feet apart, and the whole is drift-bolted to the solid rock of the channel. The cribwork is filled with loose stone to a height of about 10 feet, and the upper surface of the weir is planked over. On the upper toe of the weir rests a bed of concrete to prevent seepage, and over this is a filling of gravel to a height of about 10 feet (Fig. 23). The down-stream face of this structure consists of an apron or rollerway of similar crib timbers, a little more substantially built. Originally the down-stream face was nearly vertical, but the water soon so undermined the structure that it was found necessary to add this rollerway to prevent its destruction. This addition has the same slope on the down-stream face as has the up-stream face for a distance of about 50 feet below the

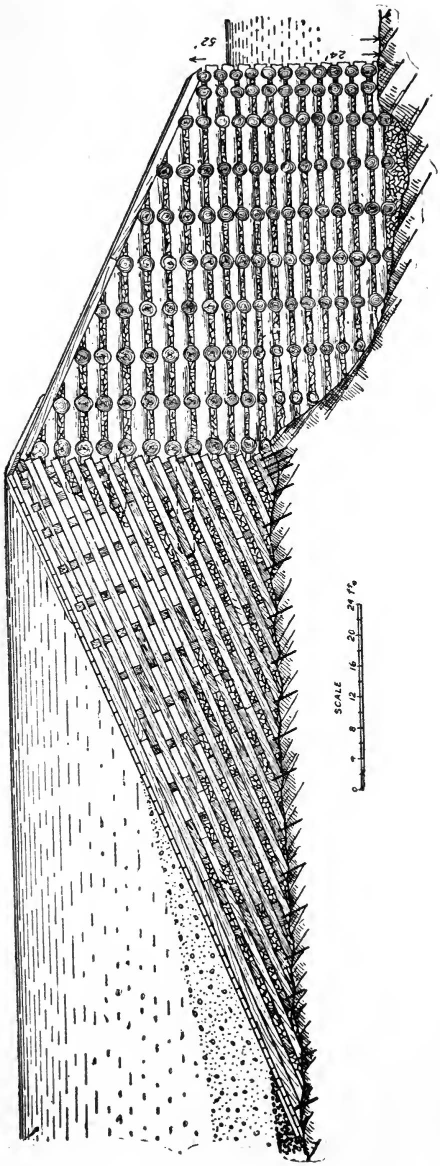


FIG. 23.—CROSS-SECTION OF HOLYOKE WEIR.

crest of the weir, at which point it falls away vertically, its end being nearly level with the surface of the river, though its vertical height at this point is about 25 feet. As the water rolling over this drops immediately into a water cushion of considerable depth, no injury is done the structure from its impact.

133. **Construction of Crib Weirs.**—A crib weir should never be left hollow, as was the upper part of the Holyoke weir, but should be completely filled in with gravel or rock. Many engineers advise against rock filling, as this permits the passage of air to the wood, and thus promotes its decay. The action of air in causing decay is still more marked if the weir is left hollow. Gravel well puddled around the woodwork becomes air-tight, and protects every timber which it encases. This material is therefore the most desirable filling. No timbers should butt on top of the course next beneath, as this gives each timber 6-inch bearing at the most, and if the lower timbers become decayed the strength of the bearing is speedily reduced. The shape of such a weir should always be such as to prevent the water which falls over it from excavating beneath its toe, especially if the foundation is of gravel or soft rock. In such cases a roller apron should be built, backed still lower down by a horizontal apron which will take up the scouring force of the water. Even on a firm rock foundation a clear overfall should not be given unless a deep water cushion can be furnished or the bed of the river can be laid dry for examination and the repair of the weir.

134. **Composite Gravel and Rock Weir.**—There are several varieties of mixed weirs other than those described which have given satisfaction in the West. One of these is built across the Lower Fox river at Little Kukuna. The foundation of this weir (Fig. 24) is of gravel and loose material, and the structure is held in place by two parallel rows of piling driven across the entire width of the stream. One of these rows runs through the centre of the weir, its summit being on a level with the crest; the other is 10 feet further down-stream, and forms the edge of the lower portion

of the apron. These piles were driven 14 feet into the gravel and bowlder bed, and the two rows were braced together by 10 by 10 timbers and the intervening space filled with broken stone. On the upper side of the upper row 4-inch planking was spiked to within 2 feet of the river bed, below which sheet piling was driven against this piling 4 feet into the gravel bed

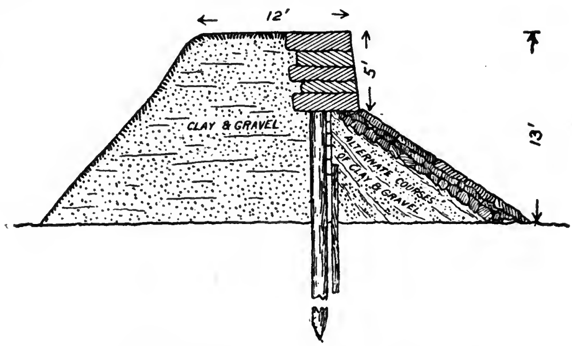


FIG. 24.—CROSS-SECTON OF LITTLE KUKUNA WEIR.

to prevent seepage. On the upper side of this barrier, against the planking and sheet piling, alternate layers of clay and gravel were laid, at a slope of 1 on $1\frac{1}{2}$, and on top of this was placed a thickness of $1\frac{1}{2}$ feet of loose stone, the whole being faced with large flat stones 4 inches thick. The top surface of the down-stream face between the two rows of piling has an inclination of about 1 on $3\frac{1}{2}$, and is faced with 4 inches in thickness of planking, below which the loose rock is given a slope of 1 on $1\frac{1}{2}$.

135. Scouring Effect of Falling Water.—In the construction of weirs various subterfuges have been employed to deliver the falling water so quietly that it shall not erode the stream-bed below. The erosive force of falling water is such that it is capable of wearing away even the hardest rock. The principal forms which have resulted from the endeavor to reduce this action are: 1, aprons, 2, sloping roller-ways, 3, ogee curves to the lower side of the weir, and 4, water cushions. Each of these forms has its advocates, and each is especially adapted to certain conditions, dependent chiefly upon the height of overfall and the character of the material of which the stream-bed is composed. Under similar conditions aprons are employed in all countries. Ogee shapes appear to have originated in India, and are very popular there. They have been adopted to a limited extent in this country.

136. Weir Aprons.—Where the foundation of the weir is of some unstable material, as earth, sand, or gravel, an apron is built below its down-stream toe. These aprons are made of wood, of dry-laid masonry, or of masonry in cement. They form a substantial artificial flooring to the stream-bed on which the force of the falling water is taken up, thus protecting it from erosion and preventing undercutting of the weir. Where an apron is employed, the weir depends on its efficient construction and careful maintenance for its security. Such works are built of masonry in the most substantial manner in India, where a rough general rule is to give the masonry apron a thickness equal to one half and a length parallel to the stream channel equal to from three to four times the vertical height of the obstructive part of the weir. Beyond this a loose stone apron is generally added, with a length equal to one and one half times, and a depth equal to two thirds of the height of the weir. Another rule adopted in India is to give the apron a width equal to from six to eight times the square root of the maximum depth of water above the weir crest, and a thickness equal to one fifth to one fourth of the overfall height of the weir plus the depth of water on the crest.

According to the American standards both of these rules seem to give unnecessarily substantial results. With us wooden aprons are generally employed which rarely exceed from 2 to 6 feet in thickness for the greatest height of overfall. Aprons, however, cannot be used with security with weirs in which the drop is considerable. No limit, other than that of expense, can be set to the height for which aprons are serviceable, for a point is ultimately reached where an ogee-shaped or rollerway weir or a water-cushion will be less expensive and more serviceable.

137. Rollerway and Ogee-shaped Weirs.—Ogee-shaped weirs probably originated as a development of roller aprons. The first ogee weirs of any magnitude were those built on the falls in the eastern Jumna canal in India. The original sloping apron or rollerway is still largely employed, the chief objection to it being the amount of material required in its construction and its consequent cost. Such structures are the weirs of the Soane and Agra canals, illustrated in Pl. III. In these the lower slope of the weir is made extremely flat, so that the friction of the water rolling over it shall retard its

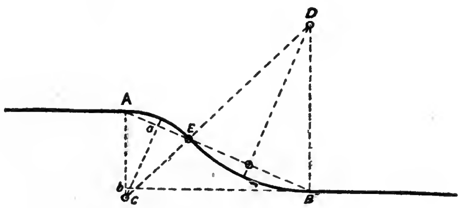


Fig. 25.—Diagram of Ogee Curve.

velocity and diminish its erosive action. In our own country a similar long sloping rollerway is that on the Holyoke weir (Fig. 23).

The ogee shape is an improvement on the rollerway. It reduces to a minimum the amount of material required, while producing nearly the same effect. The object of the ogee shape is to cause the water to slide instead of to fall over the weir, and the exact moment when water ceases to slide and commences to fall is shown by its losing its bluish color and commencing to become whitish. The ogee curve is best understood from the accompanying diagram (Fig. 25). Bisect *AE*, and from the point of bisection at *A* draw a perpendicular cutting the perpendicular let fall from *A* at *C*. Join *CE* and prolong this line until it cuts the perpendicular projected on B at D. From the points C and D as centres, draw the curves of the ogee

$$bB = \frac{5Ab}{2},$$

$$AE = \frac{AB}{3}.$$

A good example of ogee-shaped weir is shown in plate V.

138. Water-cushions.—The principle involved in the water-cushion is that which nature has laid down for herself on all natural falls, namely, that of having a deep enough cistern below the fall to take up the shock of the falling water and reduce its velocity to the normal. It has been noticed below cataracts and falls, for instance, that they erode a cistern the depth of which bears a certain relation to the height of the fall. The method of constructing a water-cushion is not to excavate such a cistern below the weir, but to create a corresponding depth by building a subsidiary weir below the upper weir. This subsidiary weir backs the water up against the lower toe of the main weir to the required depth, at the same time practically reducing the height of the fall by the height of the subsidiary weir.

It is difficult to find any set rule for determining the depth of water-cushion for a given height of fall. From observations of several natural waterfalls it has been discovered that the height of fall is to the depth of the water-cushion as from 5 or 7 to 1. In an experimental fall constructed on the Bari Doab canal in India it was found that, with a height of fall to a depth of water-cushion as 3 to 4 the water had no injurious effect on the bottom of the well. On canals where the height of fall is not great it has been discovered that the depth of the water-cushion may be approximately determined from the formula $D = c\sqrt{h^3}\sqrt{d}$, in which D represents the depth of the water-cushion below the crest of the retaining wall; c is a coefficient the value of which is dependent on the material which is used for the floor of the cushion and varies between .75 for compact stone and 1.25 for moderately hard brick; h is the height of the fall, and d is the maximum depth of water which passes over the crest of the weir. The breath of the floor or the bottom of the cistern of the water-cushion parallel to the stream channel is dependent on the section of the weir and will not exceed $8d$ and should not be less than $6d$. A rule laid down for determining the dimensions of water-cushions and their cisterns on the smaller canals in India is that the depth of the cistern at the foot of the weir shall equal one third of the height of the fall plus the depth of water. Thus on a fall 4 feet deep on a canal carrying 5 feet of water the cistern depth will equal $\frac{1}{3}(4 + 5) = 3$ feet. The minimum cistern length is equal to three times the depth from the drop-wall to the reverse slope of the cistern, which latter will be 1 in 5. The width of the cistern must be twice the mean depth of the water in the channel.

On the Ganges canal it was found that the ogee form of weir was not entirely satisfactory. The shock of the falling water proved so great as to materially injure the structure, and all of these ogee falls have since been remodelled in such a manner as to form water-cushions. Thus on falls 15 feet high the ogee has been cut so as to give first a vertical fall of 5 feet to a short level bench 10 feet in length, then a vertical drop of 10 feet ending in a shallow water-cushion the floor of which is of masonry 4 feet in thickness. It may be generally asserted that experience in India has proved unfavorable to the ogee form. In this country a few ogee weirs have been designed and constructed with a partial ogee curve to the lower face, the water dropping into a water-cushion. The most notable of these is the great weir at the head of the Turlock and Modesta canals in California (Article 276). A water-cushion 15 feet in depth is obtained below this weir by the construction of a subsidiary weir 20 feet in height, placed at a distance of 200 feet below the main weir. The height of overfall from the main weir is 98 feet, thus giving a ratio of depth of water-cushion to height of overfall of about 1 in 6. In the case of this weir, however, its down-stream face is not made vertical, but is made somewhat after the design which would be obtained by using one of the gravity formulas and adding to this sufficient material to produce the ogee curve.

The Indian method, which has proved very satisfactory in practice, is well illustrated in the Vir weir (Article 146) and the Betwa weir (Article 275). In each of these the water is permitted a clear vertical overfall to the water-cushion, the weight necessary to give the weir stability being obtained by increasing its cross-section on the up stream side. In both of these cases subsidiary weirs are constructed at some distance below the main weir in the rock bed of the river, which back up the water to the required height on the toe of the main weir. A subsidiary weir of a form somewhat similar to that below the Vir weir is illustrated in Fig. 84. This weir is employed below the main escape weir of the Periar dam in India to form a water-cushion on which the floods fall.

139. Masonry Weirs.—If it is intended that the weir shall be permanent, only masonry and iron should be used in its construction. It is frequently necessary, however, to build weirs of less durable material, the object being to economize on the first cost. Masonry weirs may be built of concrete throughout; of uncoursed rubble in cement; of ashlar; of brick; and of various combinations of these, including loose packed, uncemented rubble retained in place by masonry walls (Articles 256 to 260).

The principal classification of masonry weirs is dependent on the foundation. Where practicable such structures should only be founded on firm rock, but occasionally the depth of this below the surface is so great as to render it necessary to found the weir on gravel or sand. Masonry weirs may be classified according to the superstructure as follows: first, simple weirs with a clear overfall to the stream-bed; second, simple weirs with clear overfall to an artificial apron; third, weirs with rollerway on lower face; fourth, weirs with heavy cross section and ogee shape; fifth, weirs with clear fall to water-cushion.

140. Masonry Weirs founded on Piles.—In the construction of masonry weirs in gravel or earth, several methods have been employed for obtaining a permanent foundation. In America it is usual to found the weir on wooden piles driven deep into the river-bed. Occasionally hollow iron piles have been sunk by dredging from their interiors and filling them with concrete. In a few instances cribs and caissons have been sunk for foundations. In India the usual foundation in unstable material is the "well" (Article 143).

The weir of the Norwich Water Power Company across the Shetucket river in Connecticut is a good example of a weir founded on piles. The bed of the river at the site of the weir is composed of gravel containing small bowlders and is 30 feet or more in depth. This weir (Fig. 26) is 15 feet wide at the base and 7½ feet wide on top, its maximum height being about 20 feet. It is constructed of rubble masonry with a cut-stone coping-wall. The lower slope is covered with one foot of concrete faced with planking secured to it with long iron bolts. The up-stream face has a batter of 12 on 5 and is backed by an earth filling having a slope of about 1 on 1½, which reaches to the crest of the weir. As this structure is founded on gravel, there was great danger that the flood waters, which pass over it to a depth of 14 feet, might undermine it, accordingly a heavy timber apron was built, projecting down-stream for 22 feet, while the last 8 feet of the apron has an upward pitch designed to throw the water out and form a shallow water-cushion of about a foot in depth. This apron is composed of two thicknesses of timber the intervening space being filled with sand and loose stone. The entire structure is founded on anchor piling and is protected by sheet piling from 10 to 12 feet in depth.

141. Masonry Weir founded on Piles and Cribs.—On the Chicopee river in Connecticut, is a weir built at a place where the stream-bed is partly of rock and partly of deep gravel. Its cross-section is the same both where it rests on rock and on gravel, and is similar to that of the weir just described. Where the river-bed is composed of gravel the weir rests directly on a depth of 3 feet of cribwork, composed of squared timbers laid horizontally and transversely about 2 feet apart, the interstices being filled with broken stone. Below this portion of the weir and connected with its timber foundation is an apron 10 feet in length which rests on anchor piles, its lower extremity being protected by a row of sheet piling, while two rows of sheet piling extend along the edges of the timber foundation below either toe of the weir. This apron is of the same general character as the timber foundation, its total

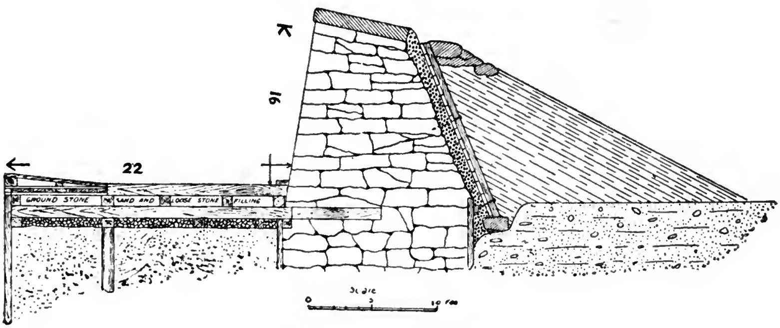


FIG. 26.—CROSS-SECTION OF NORWICH WATER POWER COMPANY'S WEIR.

thickness being 5 feet. The crest of the weir is from 15 to 16 feet above the river-bed, and it is composed of rubble masonry surmounted by an inclined coping of ashlar between 6 and 7 feet in width. The upper face of the weir has a batter of 7 on 1 and the down-stream slope a batter of 3 on 1.

142. Masonry Weir founded on Cribs.—One of the most interesting and largest masonry weirs founded on unstable soil is that on the middle branch of the Croton river, in New York. This work was constructed essentially for water-storage purposes but acts also as a weir since the flood waters of the stream pass over it. The construction of this weir is peculiarly composite, a large portion of it resting on firm rock,

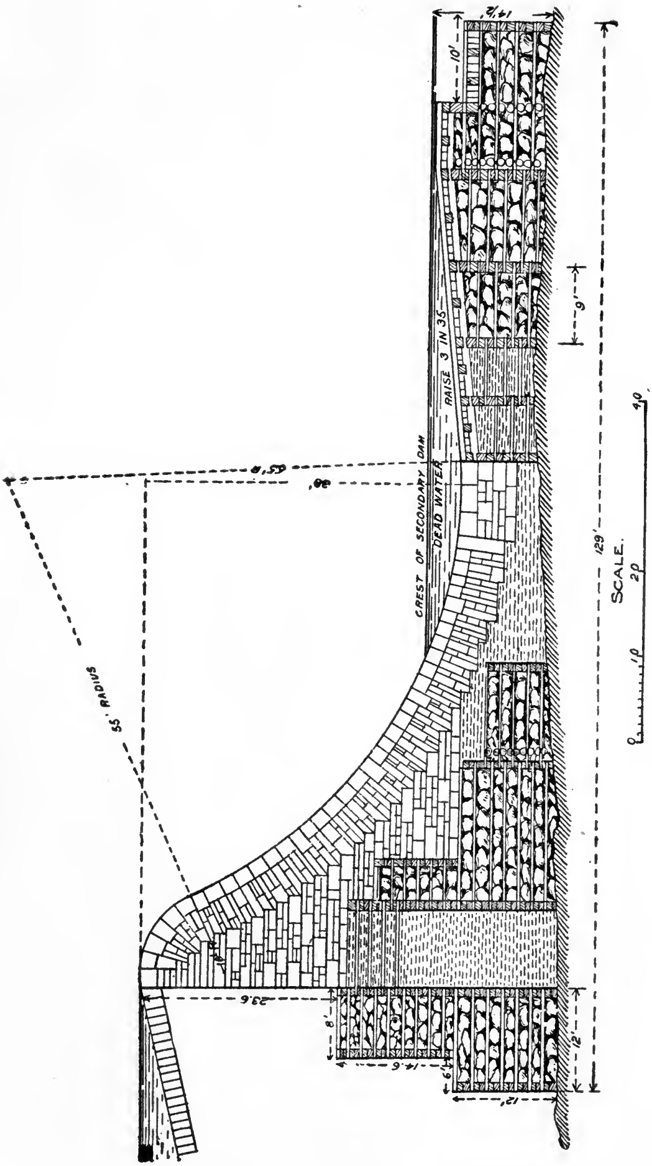


PLATE V.—CROSS-SECTION OF CROTON DAM.

while the remainder is fouuded on a stratum of alluvial soil containing bowlders. The piers (Plate V) are of timber crib-work, the walls of which are connected by ties and the whole filled with stone. These cribs are planked on top, and upon them are built two smaller wooden piers, similar in all respects to the first and likewise planked over. The space between was then filled with concrete and the top of the piers connected by ties of timber. An additional pier similar to those just described was built below the first and filled with concrete. Upon this foundation the masonry weir was constructed. It consists of stone set in hydraulic cement, the main body being laid in horizontal layers. The facing is of finely cut granite ashlar well bonded together and inclining at right angles to the curved face of the weir.

This structure is 50 feet in maximum height and 76 feet in maximum width at the base. Its up-stream slope is vertical for 23½ feet, below which it is broken into two vertical benches by the piers just mentioned. It is backed behind by an earth embankment having a very low and flat slope. The down-stream face has an ogee curve similar to that which would be assumed by the water flowing over it. The crest of this face is convex with a radius of 10 feet, below which is a reverse or concave curve with a radius of 55 feet. Below the lower end of this weir is built a raised apron 55 feet in total length and connected with the main weir. The rise of this apron is 1 in 11½, and the amount of this rise is 2½ feet, giving a water-cushion of this depth in the lower part of the apron. The latter consists of five sets of cribs, the two nearest the weir being filled with concrete and the remainder with broken stone. They are of 12 by 12 timbers and are covered with planking. At a distance of 300 feet from the extremity of this apron is built a secondary weir of crib timber filled with broken stone. The object of this secondary weir is to divide the head of water, thus causing it to fall in two steps, the first 38 feet in height to the lowest part of the apron, and the second 15 feet in height over the secondary weir to the stream bed. This secondary weir answers the additional purposes of creating a shallow water-cushion at the foot of the main weir, and of protecting the timber of the apron from deterioration by keep-it under water. Near the left shore of this weir is a wasteway by means of which the water can be drawn off from this water-cushion.

143. Masonry Weirs founded on Wells.—This class of weir is as yet peculiar to India, where it is built on sand or gravel stream-beds. In Pl. III are illustrated several examples of these structures, while that built across the Soane river is described in Article 131. They consist essentially of one or more walls of masonry running across the entire width of the stream and founded on wells, while the space between these is filled in with loose packed stone. The slopes of these weirs are generally long and low, varying between vertical and 1 on 3 to 5 on the upper face, but on the lower face ranging from 1 to 10 to 20. In the case of the weir across the Ganges river at the head of the lower Ganges canal, the main obstruction to the stream channel is a masonry wall founded on wells. On the lower or down-stream face, however, instead of the usual long slope there is a vertical drop of $9\frac{1}{2}$ feet. The top width of the wall is 7 feet, and the water falling over this drops to an apron nearly 150 feet long which is composed of masonry resting on four rows of shallow wells for a distance of about 40 feet, below which a loose stone apron kept in place by rows of wells extends for the remaining 110 feet.

144. Weirs founded on Rock. San Diego Weir.—One of the first masonry diversion weirs built in the west is that on the San Diego river in California at the head of the San Diego flume. This weir (Fig. 27) is built in two tangents, the exterior angle of which points up stream. At a distance of 108 feet from the south end is the outlet sluice, beyond which the weir is reinforced on its lower side by a great mass of loose stone, the object of which is to break the force of the falling water. At a distance of 32 feet beyond the outlet sluice is an open wasteway 20 feet wide, the crest of which is 4 feet lower than that of the remainder of the weir. Fourteen feet beyond this wasteway is another which is 165 feet in length, its crest being at the same height as that of the first described. In the bottom of the weir are two undersluices, one near the centre and the other under the outlet sluice, and respectively 18 and 14 feet below the crest of the weir. In cross-section this weir is 35 feet in height, 5 feet wide on top and 16 feet

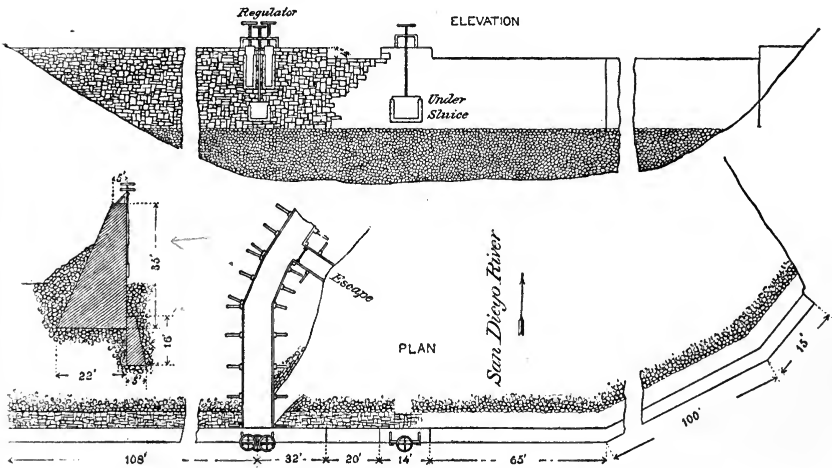


FIG. 27.—PLAN, ELEVATION, AND CROSS-SECTION OF SAN DIEGO WEIR.

wide at the bottom. It was sunk to a depth of from 15 to 25 feet in the gravel bed of the river, its crest being about 10 feet above the stream-bed.

145. Henares Weir.—This weir is at the head of the Henares canal in Spain. As shown in cross-section (Fig. 28) it is 23 feet in maximum height, its upper slope having a batter for the lower two thirds of about 6 on 1, and for the upper third of 12 on 1. Its top width is 3.14 feet, its thickness at the base is 45.8 feet, and its face has an easy flat ogee-shaped curve. This weir is 390 feet in length on the crest, being curved in plan and running obliquely across the river at a tangent to the axis of the canal. Its body is composed of concrete, while the crest and lower slope are faced with cut stone blocks alternating in headers and stretchers. Great care was taken in the construction of this work to prevent leakage.

This was obviated by cutting a channel in the rock along the central axis of the weir for its entire length, and in this is fitted a line of stone, half bedded in the rock and half in the concrete

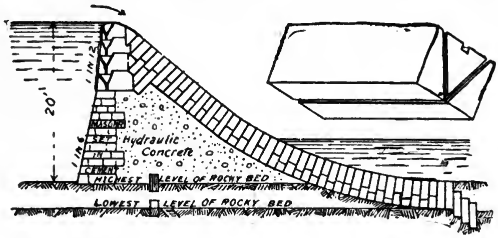


FIG. 28.—CROSS-SECTION OF HENARES WEIR, SPAIN.

of the weir. These stones were built into the rock and the joints were then run with pure cement. In the sides of each of the four upper courses of stones near the crest of the weir were cut V-shaped grooves, and expanding horizontal grooves were cut in the upper and lower faces of each stone, forming a continuous channel which was filled with pure cement so as to form a tight joint between each stone. As the bed of the river was uneven, it was found necessary to carry down the lower portion of the weir as an apron by means of a series of blocks of stone formed in steps, the last of which is firmly embedded 3 feet in the rock.

146. Appleton Weir.—The upper weir at Appleton, Wisconsin (Fig. 29), across the lower Fox river is built throughout

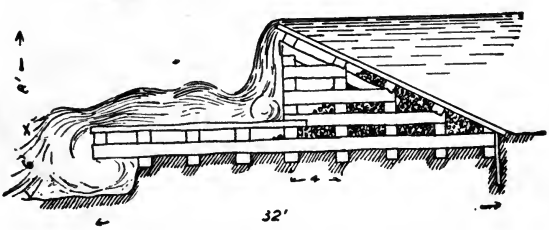


FIG. 29.—CROSS-SECTION OF APPLETON WEIR.

of dressed rubble limestone with the exception of the coping, which is of ashlar. This weir is 11 feet in maximum height, the total height of overfall being about 10 feet. It is 15 feet in width at its base and 4 feet wide on the crest, the down-stream slope being about 4 on 1, while the up-stream slope is nearly 1 on 1. Each coping-stone is fastened by a band of 1-inch round iron to a hook set into the masonry at the toe of the weir, and the adjoining coping stones are fastened to each other by $1\frac{1}{4}$-inch iron strap dowelled into the stones. In this way a most substantial bondage is obtained throughout the work.

147. Vir Weir.—The Vir weir at the head of the Nira canal in India is built of uncoursed rubble masonry and is protected by a water-cushion. It is 2340 feet in length, $43\frac{1}{2}$ feet in height, and 9 feet wide on top, and is constructed of uncoursed rubble masonry. The down-stream slope is 8 on 1 for 20 feet from the crest and 6 on 1 for the remainder of the weir, while the up-stream face has a uniform batter of 20 on 1 and at no place is the mean thickness of the weir less than half its height. This weir is founded on solid rock and in order to form a water-cushion a subsidiary weir is provided 2800 feet below the main weir. This subsidiary weir is located in a narrow portion of the river channel, its total length being 615 feet, its height $24\frac{1}{2}$ feet, while its crest is 20 feet lower than that of the main weir, thus forming a permanent water-cushion 20 feet deep. The maximum flood which is estimated to pass over this weir is 158,000 second-feet, producing a depth of 32 feet in the water-cushion and a height of overfall of but 8 feet.

148. Other Masonry Weirs.—A masonry diversion weir which is different from any of those described is that across the Pequannock river near Newark, New Jersey. This weir (Fig. 30), which also serves for purposes of storage, is built of rubble masonry, coursed and dressed on its faces and having an ashlar capstone. That portion of the structure which acts as a dam, since flood waters do not pass over it, is 38 feet in maximum height, 5 feet 10 inches wide on top, and 21 feet wide at the base. The remainder of the structure, which is built as an overfall weir, is set nearly at right angles to the main dam and is curved with a radius of 640 feet. This overfall weir is 22 feet in height, its crest being 7 feet below that of the main dam. It is 15 feet in width at the base and 5 feet wide on top, its lower slope on the up-stream face being vertical for 7 feet, above which it has an inclination of 3 on 1. The down-stream face has an inclination of 8 on 1 for 8 feet below the crest, below which it changes to about 5 on 1 for 8 feet more, and then to 3 on 1. The result of this is to give a clear overfall to the bed-rock below, which is protected

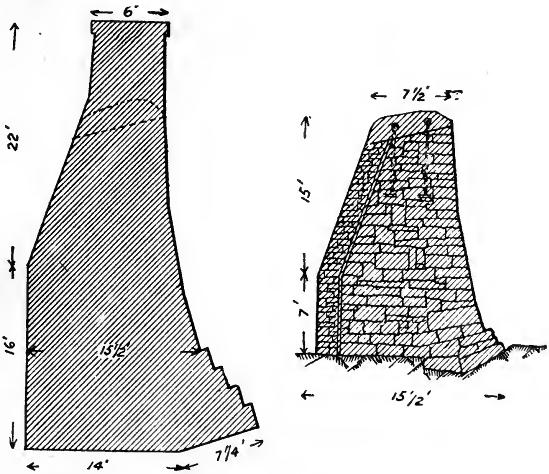


FIG. 30.—CROSS-SECTIONS OF NEWARK DAM AND WEIR.

by a trifling depth of water in the river channel, which acts as a water-cushion of moderate depth. The coping stone of the overfall weir is irregular in shape and is made continuous by means of dowels between the several stones, and is secured to the structure by anchors let into the masonry which hold down the dowels every 12 feet.

The weir across the Merrimac river at Lawrence, Massachusetts, appears to have an unnecessarily heavy cross-section (Fig. 31). It is 33 feet in maximum height, its extreme breadth at the base being 35 feet. The down-stream face has a batter of 12 on 1, and the structure is surmounted by a coping stone which is level for 3 feet and then slopes up-stream with a batter of 1 on 6 for 12 feet, beyond which the weir is stepped off with a batter of 1 on 1 to within about

10 feet of its base, which latter portion is vertical. It is composed of rubble masonry founded on firm rock, the front of the dam resting against the edge of a trench excavated in the rock. The face and coping of the weir are of dressed ashlar,

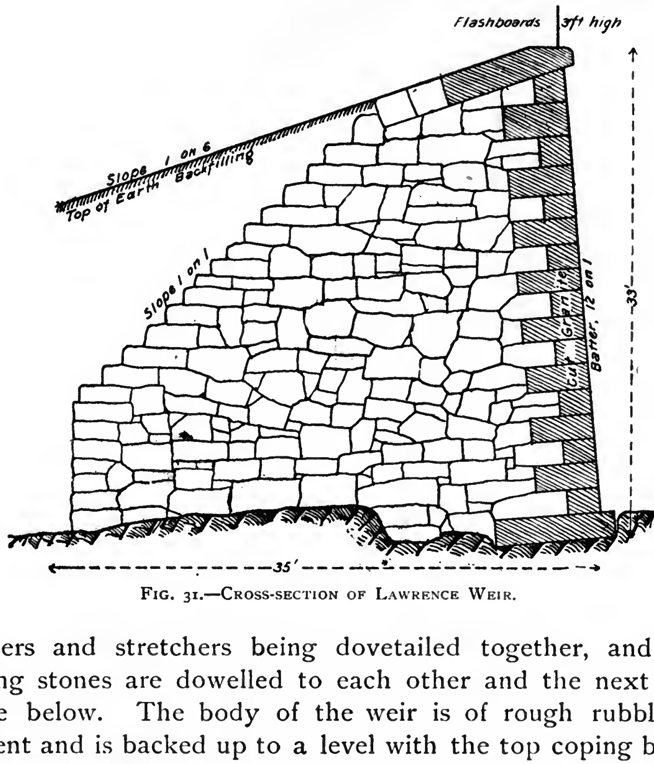


FIG. 31.—CROSS-SECTION OF LAWRENCE WEIR.

headers and stretchers being dovetailed together, and the coping stones are dowelled to each other and the next face stone below. The body of the weir is of rough rubble in cement and is backed up to a level with the top coping by an earth filling having a slope of 1 on 6. The level of the water may be raised by means of flashboards 16 feet in length to a height of 3 feet.

149. Diversion Dams.—There are several structures of considerable magnitude which, from the functions they perform, should be classified as diversion weirs rather than storage dams. Prominent among these are the Betwa dam in India and the Folsom and Turlock dams in the United States. The two latter were built solely for purposes of diversion, while the former serves to store as well as to divert water. These works, however, are of such magnitude that the principles involved in their design and construction are essentially those employed in designing masonry dams for water storage, and for this reason they will be described among masonry dams. Another variety of diversion dam employed also for storing water, is represented by those at the head of the Idaho Mining and Irrigation Company's canal and the Pecos canal in New Mexico. These structures are both of composite character, being built of a combination of earth and loose rock. As it would be unsafe to permit flood waters to pass over their crests, all surplus water is passed around them through wasteways. It will thus be seen that they are essentially storage dams, and their uses for purposes of diversion will be described among that class of works (Arts. 237 and 238).

CHAPTER XIII.

SCOURING SLUICES, REGULATORS, AND ESCAPES.

150. **Scouring Sluices.**—Scouring or undersluices are placed in the bottom of nearly every well constructed weir or dam, at the end immediately adjacent to the regulator head. The object of these is to remove, by the erosive action of the water, any sediment which may be deposited in front of the regulator. If the flow in the stream is sufficiently great to permit it, these scouring sluices are kept constantly open and thus perform their functions by keeping the water in motion past the regulating head and thus preventing the silt from settling. If sufficient water cannot be spared to leave the undersluices constantly open, they are opened during flood and high waters, and by creating a swift current are effectual in removing silt which has been deposited at other times.

The scouring effect of sluices constructed in the body of the weir is produced by two classes of contrivances; namely, by open scouring sluices and by undersluices. The open scouring sluice is practically identical with the open weir, as the latter consists of scouring sluices carried across the entire width of the channel. Where the weir forms a solid barrier to the channel and is only open for a short portion of its length adjacent to the canal head, the latter is spoken of as a scouring sluice. The waterway of a scouring sluice is open for the entire height of the weir from its crest to the bed of the stream.

Undersluices are more generally constructed where the weir is of considerable height and the amount of silt carried in suspension is relatively small. In these the opening does not extend as high as the crest of the weir, nor does the sill of the sluiceway necessarily reach to the level of the stream-bed. It is chiefly essential that its sill shall be as low as the sill of the regulator head. Undersluices are more commonly employed in the higher structures, such as weirs and dams which close storage reservoirs (Articles 288 and 289).

Scouring sluices are practically open portions of the weir and consist of a foundation, floorway, and superstructure. The

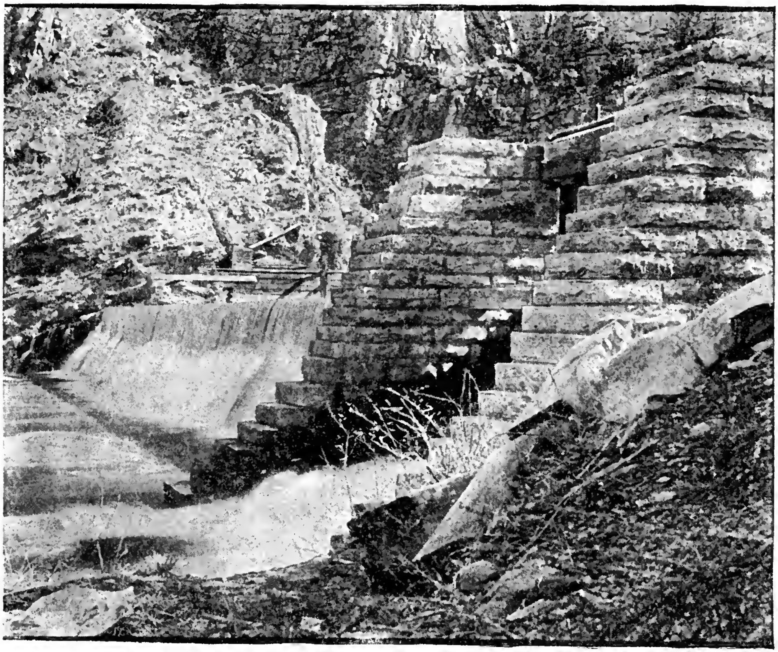


FIG. 32.—VIEW OF HIGHLINE CANAL WEIR.

floor must be deep and well constructed and carried for a short distance up-stream from the weir axis and for a considerable distance below it. On it are built up piers grooved for the reception of planks or gates, so that the sluiceway may be closed or opened at will. Scouring sluices have been built in very few American weirs, the most substantial structure of this kind being in the weir at the head of the Highline canal in Colorado. In Fig. 32 is shown a view of this wier with water passing over it, and in the foreground is the scouring sluice, which consists of two masonry piers built into the end of the weir and forming its abutment. The opening between these is the entire height of the structure and can be closed by four sets of iron gates which slide vertically between iron columns. These gates are each 4 feet wide between centres and 7 feet in height, and can be raised by means of screws turned by hand wheels from above, and their sills are set 2 feet below the level of the canal head gates. This structure, however, is not a true scouring sluice in that it is not at the end of the weir adjacent to the canal head. It is expected to clear out silt which has deposited above the weir, though it is not entirely successful in producing this effect.

151. Examples of Scouring Sluices.—At the head of the Monte Vista canal in Colorado true scouring sluices have been constructed, though these are of wood. This wier (Fig. 16) is built across the gravel bed of the Rio Grande, and is founded on piles sunk to a depth of 10 feet. The wier is 8 feet in height above the stream-bed and consists of an earth bank 16 feet in length at the end furthest away from the canal head, and of a crib weir 74 feet in length, terminating at the end adjacent to the regulator head in an open way of five scouring sluices. These are founded on piles, and the stream-bed beneath is floored with planking to form an apron to protect it against erosion. The openings are separated by upright posts of wood reaching to the crest of the weir, and can be closed by flashboards dropped between grooves.

An excellent example of masonry scouring sluice is furnished by that in the weir at the head of the Agra canal in India. In the end of the weir adjacent to the canal head are a set of 16 openings having a clear sluiceway of 138 feet. These openings are each 6 feet in width between the upright piers separating them and are 10 feet in height, surmounted by a masonry superstructure or bridge the height of which is 19 feet above the stream-bed. The object of this bridge is to give a platform from which to operate the sluice gates, which are of wood, well braced and fastened with iron, and slide vertically between masonry piers each 2½ feet in thickness. They are raised by means of a winch which is operated from above, travels on a hand car on rails so that it can be placed at will above any gate. The floor, which is flush with the stream-bed and on a level with the sill of the regulator head, is 12 feet in width parallel to the stream channel and extends 8 feet up-stream and 41 feet down-stream from the line of the piers. When these gates are opened all the heavy silt-laden waters are carried through the sluices, and when closed and then suddenly opened the scour produced by the rush of water is effective in removing the silt from in front of the canal head.

152. **Automatic Sluice Gates.**—Various devices have been employed whereby the gates closing scouring sluices may be opened rapidly and under the greatest pressure of water which may be brought against them by sudden flood rises. But few such structures have been built in this country. In the Dry creek diversion dam on the line of the Turlock canal is a wasteway closed by automatic or quick-dropping gates. These are ten in number, each 3 feet wide in the clear, 10 feet in height, and are constructed of wood. They are hinged at the bottom and fall outward or down-stream. When raised they are held in position by chains attached to their upper edges and fastened to the piers separating them. When they are to be dropped these are detached and the gates fall outward, striking on a shallow water-cushion built in the floor of the sluice-way beneath them. After the flood has subsided the gates can again be raised by chains and geared windlasses.

In nearly all the scouring sluices so far described the mode of operating the gates is from a superstructure above the level of the highest flood. This form of construction is expensive and interferes with the free flow of water by stopping and perhaps choking the sluices with floating brushwood and logs. To remedy this defect and obtain the largest percentage of free space between the piers for the passage of flood waters, some of the more modern Indian works have been given much larger openings between piers, and the gates are so operated that no superstructure is necessary above the level of the weir crest. As a result the floods may pass with little obstruction over as well as through the weirs. Such structures as these are of necessity strongly constructed and are made capable of quick operation. Two excellent examples of this class of structure are furnished by the shutters in the Mahanuddy weir at the head of the Orissa canals and those of the Dehree weir at the head of the Soane canals in India.

153. Mahanuddy Sluice Shutters.—These shutters are designed somewhat after the plan adopted on some of the older weirs across the river Seine in France. The sluiceway consists of ten bays each 50 feet wide and separated by masonry piers. Each bay is closed by a double row of timber shutters fastened by wrought-iron bolts and hinges to a heavy beam of timber embedded in the masonry floor of the sluice (Fig.

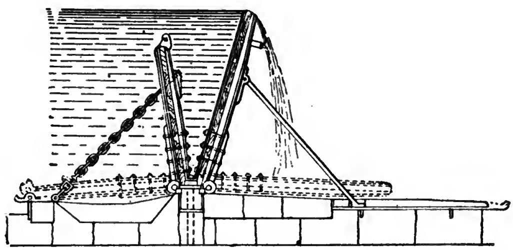


FIG. 33.—CROSS-SECTION OF MAHANUDDY AUTOMATIC SHUTTERS, INDIA.

33). These shutters are arranged in pairs so that there are seven up-stream and seven down-stream shutters in each bay. The lower shutters are 9 feet in height above the floor, and the upper shutters $7\frac{1}{2}$ feet in height. Each bay is separated from the next by a stone pier 5 feet thick, to which the gearing for working them is attached. During the floods the upper row of shutters, which fall forward up-stream, are held to the floor of the weir in an almost horizontal position by means of iron clutches. The rear or lower row of shutters which fall down-stream are kept in a horizontal position by the rush of water over them. In order that the down-stream row of shutters may be retained in position and act as dams when raised, they are provided with strong wrought-iron struts attached to their lower sides. In order to lift the lower set of shutters when the water is resting on top of them the up-stream set of are first raised, this operation being aided by the upward pressure of water from beneath, and they are retained in a vertical position by means of chains guyed to the piers above them. Relieved of the water pressure by this upper set of shutters, it then becomes possible to raise the lower set by means of a hand windlass, after which the upper set are lowered again into their original position and the weir is ready to withstand the next flood, as the lower set can then be instantly dropped by merely removing the bolts which support them.

154. **Soane Automatic Sluice Gates.**—The shutters of the Mahanuddy weir have never been successfully operated against a greater head than 6½ feet, and the jar produced by opening the upper gates and by the fall of the lower gates has always been very violent. In order to diminish this jarring action and to obtain a more easy and successful operation in the shutters of the Soane weir, a new design was devised, and it furnishes what is probably the best example of self-acting sluice gate which has yet been constructed.

The crest of the Soane weir is 9½ feet above the river-bed, and the gates by which the sluice ways are closed are each 20 feet in length and 9½ feet high. They are separated by masonry piers 6½ feet thick by 32 feet in length. The floor of these sluices is very substantial and is 90 feet in length parallel to the river channel. As the velocity of the current through them may be as high as 17½ feet per second, it was found necessary in order to withstand its erosive action to found the flooring on wells or blocks upon which an ashlar pavement 15 inches in thickness has been built up. The gates are constructed of wood well braced and set in pairs in each opening (Pl. VI.). A low masonry wall 12 inches high has been built up on the downstream side of the flooring in each alternate bay, thus giving a water-cushion of that depth on which the lower gate falls, relieving the piers of a portion of the shock. The upper gate falls up-stream, being hinged to the floor at its bottom and held upright by a series of six struts. These are hollow iron cylinders with small ventholes, and in them pistons work in such manner that when the gate is raised by the pressure of water beneath it the impact against the struts is relieved by the pistons plunging into the cylinders, from which the water is slowly forced through the vent holes. The lower gates fall down-stream and are supported by four iron rods hinged to their upper faces below the centre of gravity, and when in position are held upright by chains attached to the piers above. If both gates are open and it is desired to close the lower one so as to cause it to dam up the water, it is first relieved by pushing aside the catch which attaches the upper gate to the floor when this is raised a little by means of a hand lever, after which the force of the water brings it up slowly for a short distance and then with a jar against its hydraulic struts or rams. The pressure is now relieved from the lower gates, which can be raised by hand levers and chained in an upright position to the piers. The upper gate is again lowered, now falling chiefly by its own weight through the water, and is fastened down by clutches. The lower gate, which now acts as the dam, is prepared to be released at a moment's notice.

155. **Relation of Weirs to Regulators.**—A diversion weir retards the flow of the stream and raises the level of the water to a sufficient height to enable it to enter the canal head. The regulator is the controlling valve which admits this water to the canal if required, or prevents its entrance and causes it to pass on down the stream over or through the weir. The weir is the boiler which generates the power; the regulator is the throttle-valve which controls its entrance to the machinery. The regulator should be so located with relation to the weir that the water held up by the latter will pass at once and with the least loss of head through the former and into the canal. This is effected most successfully by placing the canal head immediately adjacent to the weir and building it in unison with and as part of the structure. The weir should not be so aligned as to cross the river diagonally at an angle inclined either to or from the regulator head. In the former case it tends to force the water against the regulator, creating

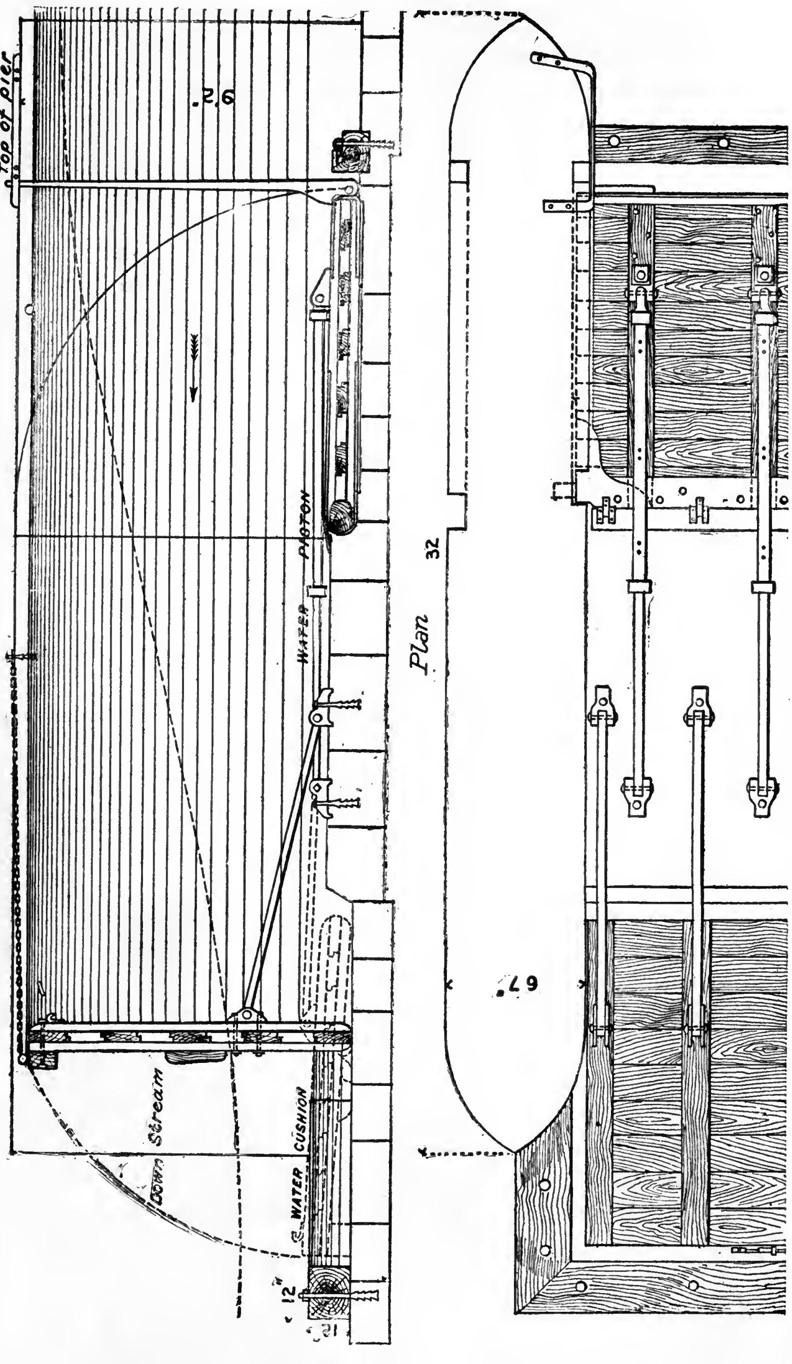


Plate VI.—Automatic Sluice Gate. Soane Canal, India.

an unnecessary scour at that point and producing an undue pressure or strain upon the head. It should not incline away from the regulator, as the reverse effect would be produced and it would cease to perform its function of directing the water into the canal. The best alignment for the weir with relation to the regulator is to have it cross the stream at right angles to the line of the weir. This gives a clear even scour past the regulating gates and keeps them clear of silt, at the same time furnishing the required amount of water.

The regulator should not be located at a distance from the end of the weir; otherwise a dead water is created between the weir and the regulator in which deposits of silt occur, blocking the entrance to the canal and diminishing the volume available for its supply. An excellent example of such faulty location is that illustrated in Fig. 34, showing the head of the Ganges canal, where the front of the regulator is not at right angles to the weir and is at a short distance from it, resulting in the formation of a sand-bar at its entrance. A better arrangement would be a regulator built as indicated by the dotted outlines, with its face at right angles to the line of the weir. Another example of the improper relation of weir to regulator is that at the head of the Del Norte canal in Colorado, where the two structures make an angle with each other, besides being separated a short distance. The result is the formation of a sand-bar in front of the regulator and a corresponding diminution in the supply to the canal. In Egypt at the head of the Ibrahmia, Bahr Yusef, and other canals heading in a common basin of some magnitude, a considerable deposit of Nile mud has occurred owing to the slack water created in front of the canal heads.

A good example of the proper location of the regulator and weir with relation to each other are furnished by the head of the Agra canal in India, where these works are in juxtaposition to and at right angles with each other, resulting in a clear waterway in front of the head where the main channel of the stream is maintained. The proper location of regulator to weir is well illustrated by the head of the Monte Vista canal in Colorado, where these two structures head at right angles and adjacent to each other; the result being complete freedom from deposit of sediment in front of the regulating gates and a clear channel past their entrance. Fig. 35 shows

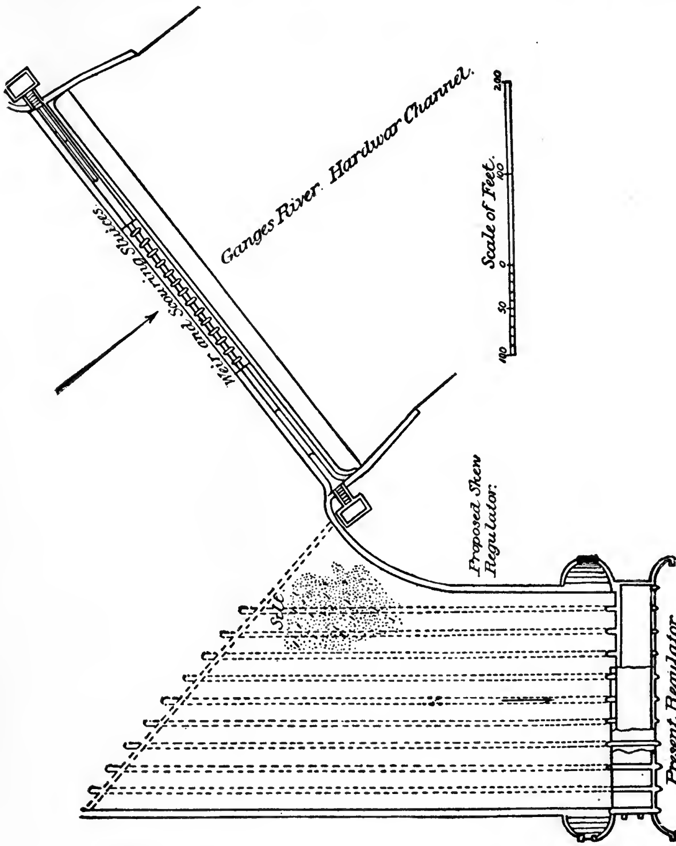


FIG. 34.—PLAN OF HEADWORKS. GANGES CANAL, INDIA.

the plan of the old and new Arizona headworks. The first weir, as shown by the full lines, was built at an angle to the channel of the stream, and the regulator head was built at an angle both to the stream channel and to the weir, the result being to force a great pressure of water against the regulating gates, resulting ultimately in their destruction, while the deposition of sediment between the gates and weir was greatly encouraged. The present headworks, indicated by the cross-lined weirs and the dotted canal, lines offer an excellent

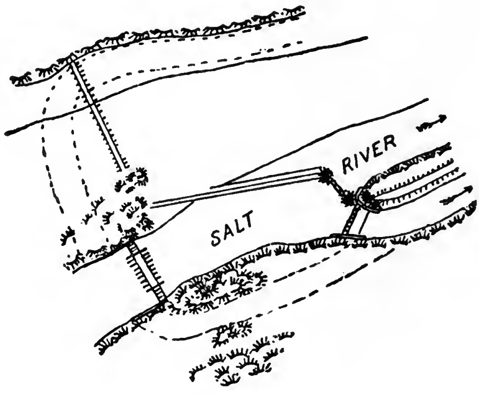


FIG. 35.—ARIZONA CANAL. PLAN OF HEADWORKS.

example of good location, the weir being at right angles to the river channel and the regulator at right angles to the weir; resulting in perfect freedom from silt deposit and permanency in the regimen of the stream.

The proper relative location of these two works has been obtained by different methods in other cases. Thus at the headworks of the Idaho canal (Fig. 66) the stream channel is bordered by a basalt ledge about 12 feet in height. The weir is constructed between the walls of this ledge and at right angles to the stream channel. The regulating head is placed on top of the ledge with a scouring sluice in the weir immediately in front of and below it. The result is that no silt is deposited in front of the regulator head, though this is not quite at right angles to the weir. Any sediment which may tend to settlement at this point falls below the top of the basalt ledge and is carried off by a scouring sluice.

156. Classification of Regulators.—The type of regulator employed depends upon the character of the foundation and the permanency which is deemed desirable. Regulators may be classified according to the design of the gate and the method by which it is operated. With nearly any type of foundation varying degrees of permanency may be given the superstructure and various methods may be employed for operating the gates. Accordingly regulators are classified here as follows: first, wooden gates in timber framing; second, wooden gates in masonry and iron framing; third, iron gates in masonry and iron framing. They are further classified according to the method of operating the gates as follows: first, flashboard gates; second, gates raised by hand lever; third, gates raised by chain and windlass; fourth, gates raised by screw gearing.

Simple flashboard or needle gates can only be used where the pressure upon them is low. When under great pressure the opening should generally be closed by a simple sliding gate which may be raised by hand lever or windlass. Where under considerable pressure, a double series of gates one above the other, each separately raised by a lever or windlass, may be employed, and these should be operated by a screw and hand gear from above.

157. General Form of Regulator.—The regulator should be so constructed that the amount of water admitted to the canal can be easily controlled at any stage of the stream. This can only be done by having gates of such dimensions that they can be quickly opened or closed as desired. Accordingly, when the canal is large and its width great the regulator should be divided into several openings, each closed by a separate and independent gate. The width of these openings should be rarely less than 2 feet nor more than 6 feet. The channel of the regulator way should consist of a flooring of timber or masonry to protect the bottom against the erosive action of the water, and of side walls or wings of similar material to protect the banks. The various openings will be separated by piers of wood, iron, or masonry, and the amount of obstruction which they offer to the channel should be a minimum, in order that the width of the regulator head shall be as small as possible for the desired amount of opening. For convenience in operation it is customary to surmount the regulator by arches of masonry or a flooring of wood, so as to give an overhead bridge from which the gates may be handled. Lastly, the height of the regulating gates and the height of the bridge surmounting them must exceed the height of the weir crest by the amount of the greatest afflux height which the floods may attain, in order that these shall not top the regulator and destroy the canal. The regulator must be firmly and substantially constructed to withstand the pressure of great floods, and a drift fender should be built immediately in front of or at a little distance in advance of the gates. Wooden regulator heads are usually constructed much as are open flumes, and consist of a fluming or boxing of timber lined with planks on the bottom and sides and with cross bracing above. In this are set the piers and gates.

158. Arrangement of Canal Head.—As already shown, the regulator gates should be as close as possible to the end of the weir in order to prevent the deposit of silt at this point. Owing to the character of the banks and to avoid excessive cost in first construction, it is sometimes found necessary to set the regulator back in the canal a short distance. In such cases an escape should be introduced in front of and adjacent to it to relieve it of pressure and aid in its effective operation.

At the head of the Cavour canal, Italy, the regulator is set back in the head cut, and immediately in front of it is placed an escape discharging into the river. At the head of the Turlock canal in California the flood heights are so great that the water may rise above the weir crest to a height of 16 feet. In order to relieve the gates of this pressure the canal heads directly in a tunnel which is 560 feet in length and 12 feet wide at the bottom and is cut through the solid rock. It discharges into an open rock cut across which is placed the regulator, while immediately above it and at right angles to it are a series of escape gates discharging back into the river. The wasting capacity of this escape is made greater than the possible discharge of the tunnel under the greatest head of water, so that the regulator gates are relieved of most of the pressure.

At the head of the Pecos canal in New Mexico, the regulator gates are set back in a deep rock cut some distance from the entrance. This cut is 850 feet in length, and at its lower end between the abutment of the weir and adjacent to the regulating gates is an escape-way discharging into the river. By this means a clear scour can be maintained past the gates and the deposit of silt prevented, while at the same time the pressure is reduced. At the head of the Central Irrigation District canal there is no weir, as the discharge of the Sacramento river is always more than sufficient to fill the canal, the bed of which is from 1 to 2 feet below low-water level. The regulator at the head of the canal consists of two parts, a main set of masonry headgates set back in the cut one third of a mile from the river banks, and a secondary set of regulating gates and a waste gate placed three miles further back in the cut. There is no pressure to be withstood by the first set of masonry gates, since the water is held up by the second set in such manner as to equalize the pressure on both sides of the first set of gates and thus permit them to be raised by a simple contrivance.

159. Wooden Flashboard Regulators.—Simple flashboard regulators are constructed as are flashboard weirs. A satisfactory regulator of this kind is that at the head of the Calloway canal in California, which is almost identical in construction with the weir (Fig. 17) and therefore scarcely requires description. It consists of a wooden fluming having a rectangular cross-section built into the canal head and resting on piles and protected by sheet piling. Above and below this regulator head are built a wooden flooring and wings to prevent erosion. Flashboards are laid in the regulator head and can be removed or replaced one at a time, according to the amount of water to be admitted.

160. Wooden Regulator Gate lifted by Lever.—This form of regulator consists of a rectangular fluming similar to that just described, which generally extends from 8 to 10 feet up-stream from the gates and 15 or 20 feet below them. Sometimes, instead of the flooring being horizontal and having sheet piling at its termini to prevent seepage, its ends are carried down at an angle of from 30 to 45 degrees for a depth of several feet into the river-bed. As shown in Fig. 36, these simple lifting gates consist ordinarily of boards laid together horizontally and framed or braced with wood or iron so as to make a firm shutter or gate. Above this extends an upright post or handle with holes in it, into which the point of a hand lever is inserted and the gate can be thus raised. It slides vertically between upright timbers and is held in position when raised by the insertion of an iron plug into the lever holes. This type of gate is used on the Cavour canal in Italy and on the Arizona, Merced, and many other canals in this country.

161. Wooden Gate lifted by Windlass.—One of the most notable examples of this type of gate is that at the head of the Ganges canal in India, the regulator of which is of masonry, the gates being separated by masonry piers. The head on the gates is such that it is necessary to have three tiers of gates one above the other, the most advanced or up-stream gate having its sill on a level with the canal-bed and the two higher gates having their sills each 6 feet higher, while they

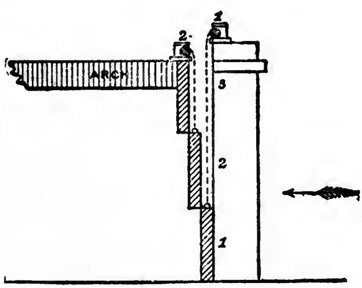


FIG. 36.—REGULATOR GATES, GANGES CANAL.

retrograde toward the face of the bridge by the width of a gate. On the bridge above are two simple horizontal wooden windlasses, and the gates are raised by turning these.

162. Gate lifted by Travelling Winch.—This is the most common form of gate employed in India where the width of canal head is great and the number of openings correspondingly large. As before stated, the regulator heads there are invariably built of masonry, each opening being separated by masonry pillars. As shown in Fig. 37, the gate is constructed

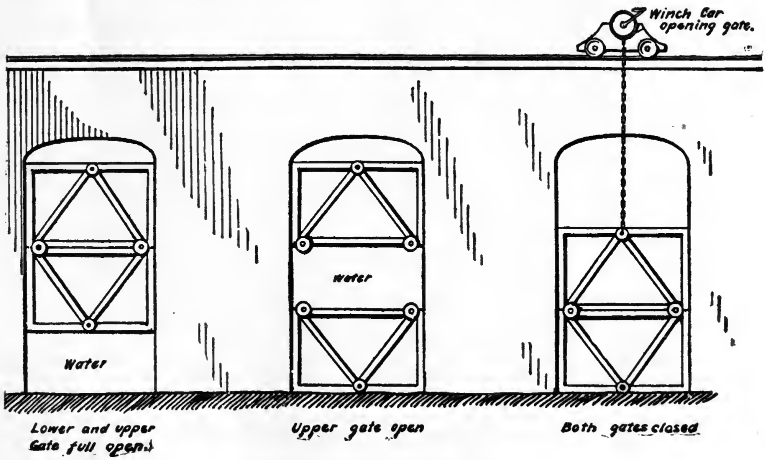


FIG. 37.—REGULATOR GATES, SOANE CANAL.

of wood, cross-braced, and to its top are attached chains which run over the windlass of the travelling winch. Above these gates is a bridge, and on the parapet immediately over the gates is a simple railroad track on which a handcar is run. On this is placed a simple hand winch, and by turning this each gate can be successively raised or lowered and the winch pushed along to the next gate.

163. Gate raised by Gearing or Screw.—This type of gate is common both in this country and abroad. They are generally employed where there is pressure to be overcome and are slow in their operation. As a consequence a few simple lifting gates are generally inserted in a few of the openings, to be used when the pressure is light, and a few geared gates are employed to be operated under pressure. Such a gate is that at the head of the Arizona canal (Fig. 38), which is constructed of wood framed with iron. Above it projects a heavy steel screw, 1½ inches in diameter, and this passes through a female

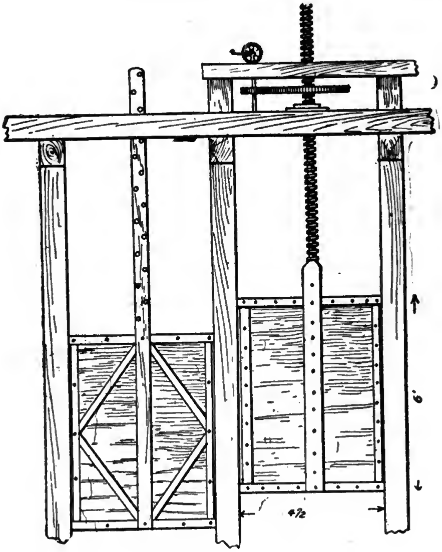


FIG. 38.—REGULATOR GATES, ARIZONA CANAL.

screw of malleable iron on which the wear is taken up. As the pressure which this gate has to withstand is great, the

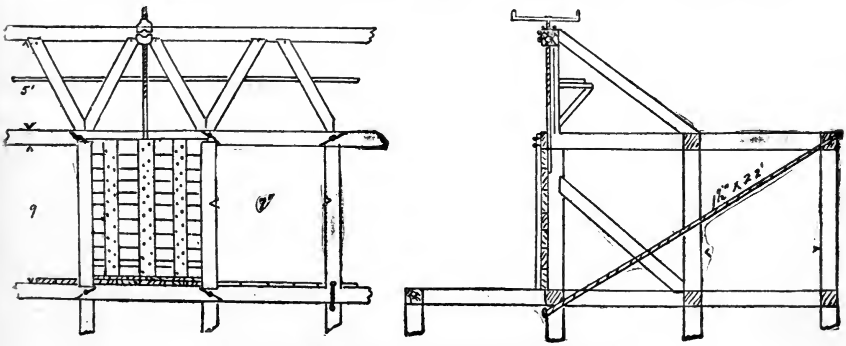


FIG. 39.—REGULATOR GATES, DEL NORTE CANAL.

simple screw is not sufficient, and the female screw forms the inner surface or axis of a geared or cogged wheel, and this is

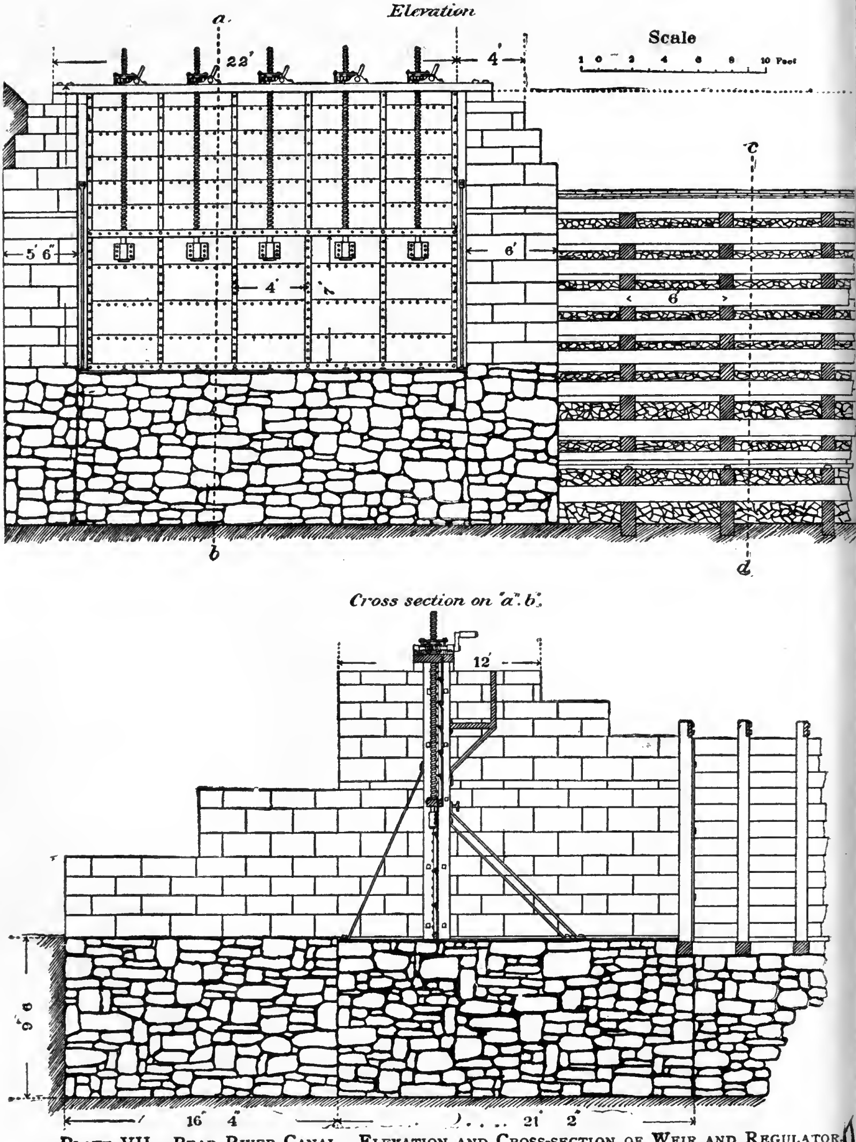


PLATE VII. BEAR RIVER CANAL. ELEVATION AND CROSS-SECTION OF WEIR AND REGULATOR.

turned by a smaller cog operated by a hand wheel; thus the gate, while moving very slowly, can be raised with the application of but a trifling amount of power, owing to the multiplicity of gearing employed.

A simpler gate of the same general type is that at the head of the Del Norte canal in Colorado. As shown in Fig. 39, the lifting screw is attached to the gate and turns in a female screw attached to the overhead bridge.

A more substantial gate is that at the head of the Bear River canal in Utah, which is set between firm masonry abutments and slides in an iron frame. This gate is of iron and to it is attached an upright screw which works in a female screw the outer circumference of which is cogged, and is turned by an

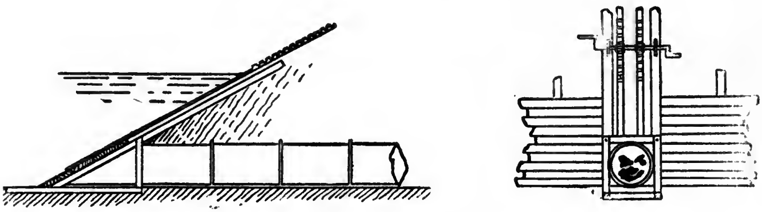


FIG. 40.—SLIDING REGULATOR GATE, IDAHO CANAL.

endless wheel operated by a hand lever (Pl. VII). An ingenious method of operating gates is that employed on the Idaho Mining Company's canal at the head of the escapes and smaller regulators. To the upper part of the gate are attached two uprights (Fig. 40) on which are plain iron cogged racks. On these work cogged pinions turned by hand levers, which cause the gates to move up and down.

164. Rolling Regulator Gate.—This form of gate (Fig. 41) is employed at the head of the Idaho Mining Company's canal, and is similar to that employed on the open weirs on the river Seine in France (Fig. 20). The regulator consists of eight openings, each 8 feet wide and 19 feet high, and is constructed of substantial masonry, surmounted by a bridge the height of which is 21 feet above the canal bed. The gates which close the openings are separated by masonry piers 3 feet in thickness,

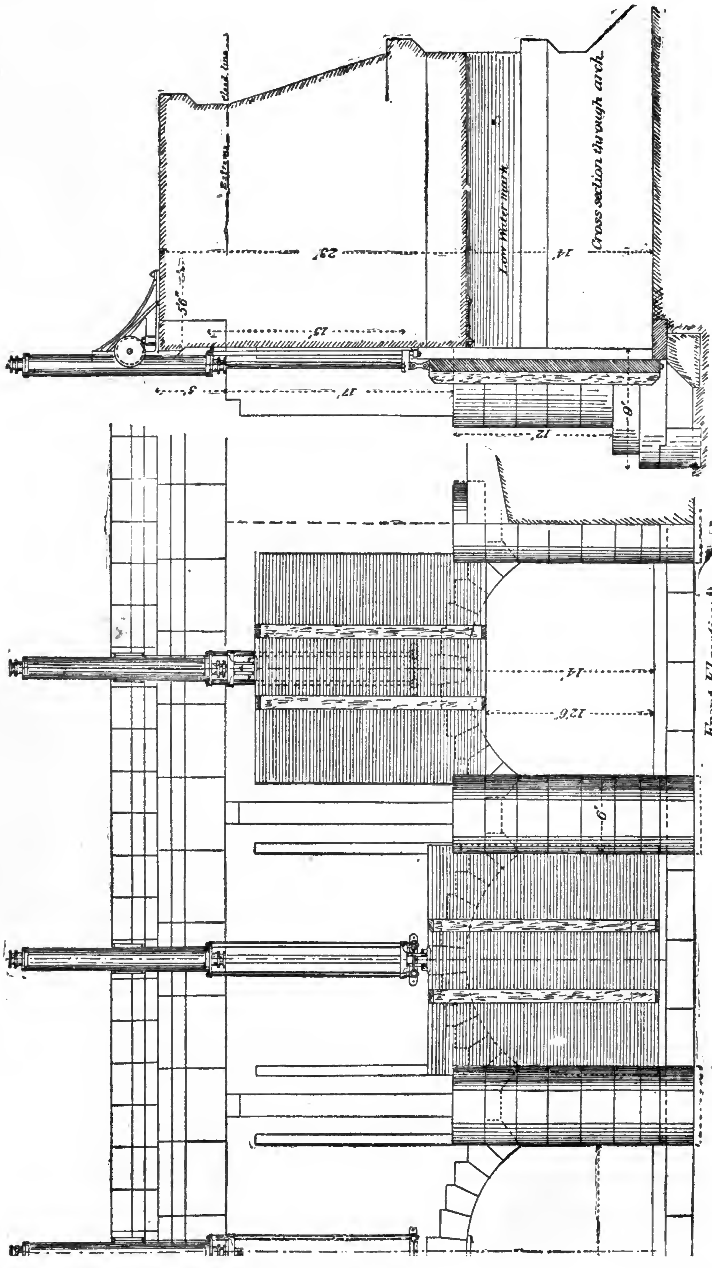


PLATE VIII.—CROSS-SECTION AND ELEVATION OF REGULATOR GATES, FOLSOM CANAL.

and consist of roller curtains made of steel plates and angle iron to a height of 10 feet from the bottom, above which the curtain is constructed of pine slats, each 6 inches wide. There are 20 steel slats and 8 of wood, and the bottom of the curtain

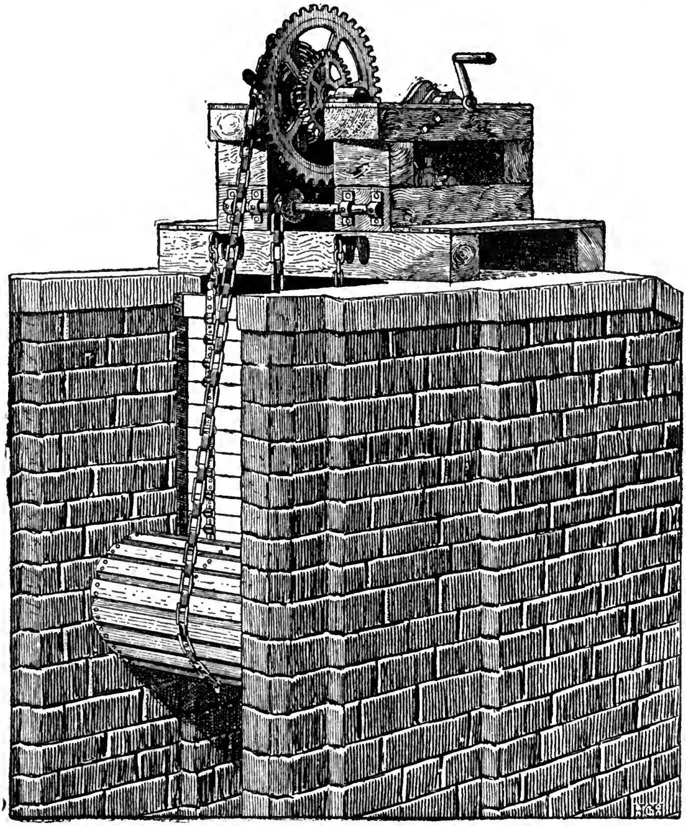


FIG. 41.—ROLLING REGULATOR GATE, IDAHO CANAL.

is fastened to a cast-iron roller, on which it is wound up from above, in the form of a spiral, by means of a chain operated from the overhead bridge by a winch.

165. Hydraulic Lifting Gate.—At the head of the Folsom canal in California the regulating gates (Pl. XXVII) are operated by hydraulic power from an accumulator fed by water power from a fall in the canal. This regulator is constructed in the most substantial manner of granite masonry, and has a total width of 66 feet between the abutments. The gates (Pl. VIII) are three in number, each 16 feet in width and 14 feet in height to the crest of a semi-circular arch, and are separated by masonry piers 6 feet in thickness. They are of wood, well braced, and slide vertically in grooves let into the masonry piers separating them. One hydraulic jack is attached to each gate, and its cylinder is fastened to the masonry above. In this works a steel plunger having a 14-foot stroke and directly connected at its lower end with the gate.

166. Escapes.—In order to establish a complete control over the water in a canal channel, provisions should be made for disposing of any excess which may arise from sudden rains or floods or from water not required for irrigation. This is effected by means of *escapes*, or, as they are more commonly called in this country, *wasteways*. These are short cuts from the canal to some natural drainage way into which the excess of water can be discharged. Escapes perform the additional service of flushing the canal and thus preventing or scouring out silt deposits.

If the heads of distributaries be opend they relieve the main canal, and the former are in turn relieved by opening the escapes; hence the distributary heads act as the safety-valves and the escapes as the waste-pipes of a canal system. Escapes should be provided at intervals along the entire canal line, the lengths of the intervals depending on the topography of the surrounding country, the danger from floods or inlet drainage, and the dimensions of the canal. On large canal systems in India it is customary to place them at intervals from 20 to 40 miles. In our own country they are placed more frequently, usually 10 to 20 miles apart. Where the regulator head is placed back from the river a short distance, as in the case of the Cavour, Pecos, and Turlock canals, an escape should be provided immediately above the regulator head for the discharge of surplus water and in order that the channel may be kept free from silt. The first or main escape on the canal line should always be constructed at a distance not greater than half a mile from the regulator, in order that in case of accident to the canal the water may immediately be drawn off. This main escape has the additional advantage of acting as a flushing gate for the prevention and removal of silt deposits. Where used for the latter purpose it is customary to decrease the slope of the canal between its head and the escape, in order that the matter carried in suspension may be deposited at that point.

167. Location and Characteristics of Escapes.—Escapes should be located above weak points, as embankments, flumes, etc., in order that the canal may be quickly emptied in case of accident. Their position should be so chosen that the escape channels through which they discharge shall be of the shortest possible length. These must have sufficient discharge to carry off the whole body of water which may reach them from both directions, so that if necessary the canal below the escape may be laid bare for repairs while it is still in operation above.

The greatest danger from injury to canals is during local rains, when the irrigator ceases to use the water, thus leaving the canal supply full, while its discharge is augmented by the flood waters. Hence it is essential where a drainage inlet enters the canal that an escape be placed opposite it for the discharge of surplus water. During floods the escape acts in relieving the canal of surplus water as though the head regulator of the canal had been brought so much nearer the point of application. In order that the escape way may act most effectively the slope of its bed should be increased by at least 12 inches immediately below its head; in addition to which the slope of the remainder of the bed should be a little greater than that of the canal, and it should tail into the drainage channel with a drop of a few feet. It is common in this country to build escapes in the sides of flumes, thus taking advantage of the wooden construction as an escape head and avoiding the expense of constructing an escape cut, as the water is discharged immediately into the drainage channel beneath the flume. While this practice is economical and may serve well where cheap construction is necessary, it is far from the best method unless great care is taken. The water falling from the flume may damage its foundations while the escape does not add to the security of the structure in which it is placed, as it does not shut off the water above it.

168. Design of Escape Heads.—Escape heads and the regulators placed in the canal adjacent to and below them are built on similar designs to the main regulating gates at the head of the canal. A maximum limit is given to the dimensions of each gate, and as many are inserted as are necessary to pass the entire discharge of the canal without obstructing its velocity. These gates may be of wood or iron, and may be framed between timber, iron, or masonry piers and abutments. They are operated as are the head regulating gates; but as the pressure on them is never great, some simple form of lifting apparatus, as flashboards or sliding gates raised by hand lever, windlass, or simple screw, is sufficiently effective.

On the Calloway canal in California wooden flashboard escape gates are used which are similar to the Calloway falls and regulating gates (Fig. 17). The escapes on the Idaho canal consist of cylindrical pipes let through the banks, the entrance to each being closed by a sliding gate raised by rack and pinion (Fig. 40). On the Highline canal in Colorado the first main escape is in the bench flume 600 feet below the head regulator, and consists of a set of four wooden gates, each 3 by 4 feet, set into the side of the flume and raised by simple rack and pinion. In the flume below and adjacent to this escape head are a set of flashboard checks for regulating the discharge of the canal, or, if necessary, of closing it and forcing all the water through the escape. In addition to this there are several other escapes along the line of the canal, a few at drainage inlets, and one in each of the important flumes on the line. For complete control of the water on the Bear river canal there are two head escapes, one 1200 feet and the other 1800 feet below the head regulating gates, and discharging back over the canyon sides into the river. Each of these escapes has 12 feet of clear opening closed by three wooden gates sliding between iron posts and raised by screw gearing. Below and adjacent to the lower escape is a set of regulating gates in the canal.

On the line of the Turlock canal abundant escape way has been provided, as the canal flows in natural drainage channels for a portion of its course. One of these, Dry creek, has a large catchment basin, and the diverting dam which turns the water back into the canal is provided with an escape weir 51 feet in length, besides an escape way 30 feet in length. An interesting escape on the line of this canal, however, is that at the bottom of the flume crossing Peasley creek. This flume is 20 feet wide and 7 feet deep and is carried on a trestle 60 feet in height above the stream bed. In the bottom of the flume is built an escape which is of sufficient capacity to discharge the full volume of water flowing in the flume. It is built by laying an iron beam across the flume bed, and this revolves on an axis turned by means of a hand wheel, thus converting a portion of the floor into a revolving gate by opening the bottom of the flume for its entire width. Beneath this gate is a receiving box which discharges up and down stream into two inclined wooden flumes which lead the water into the creek.

169. Sand Gates.—Sand gates are practically escape gates, though they are so designed and arranged in some canals as to be of service only in scouring or removing silt deposits. The main or head escape on a canal system acts as a sand gate, and is generally built as much for the purpose of flushing and scouring sediment as for the control of water in the canal. The gate in the Highline flume acts effectively as a sand gate, because a board check from 1 to 2 feet in height is placed across the flume below the escape head. This causes the deposit of silt immediately above it, whence it can be removed by the scour through the escape.

Careful provision has been made for the removal of silt on the Folsom canal. Immediately in front of and above the regulating head is a set of four sand gates placed 6 feet below the grade of the canal and discharging directly back into the river. These are practically undersluice gates, and are each 5 by 6 feet in the clear and set in substantial masonry. Sediment which is dropped into the subgrade in the canal opposite these gates is scoured out through them. In addition to these sand gates, seven others are placed in the first 1700 feet of the canal. These are all similar in construction, 5 feet wide by 10 feet high, framed in substantial masonry, and consist of iron gates sliding vertically and raised by means of a hand wheel and endless screw working on ratchets set on the back of the gate. Across the bed of the canal opposite and below each of these sand gates is a subchannel and catch-basin 1 foot in depth, the object of which is to collect silt which is afterwards scoured out through the gates.

CHAPTER XIV.

FALLS AND DRAINAGE WORKS.

170. Excessive Slope.—As the natural fall of the country through which a canal runs is usually greater than the slope of the canal, the tendency of the water in the latter is to erode its bed. In a small section of the line the erosive action of the water on the bed is noticeable providing the velocity of the stream be great. When this erosive action is extended to long reaches of the channel it produces what is known as retrogression of levels, which is the direct result of too great a slope and consequent too high velocity. If the canal is straight little harm is done by this, other than to cause the level of the water to sink below the ground surface and prevent its diversion. Where it is necessary to divert the water or where there are curves which the increased erosive action of the water would injure, it becomes necessary to compensate for the difference between the slope of the country and the canal-bed, so as to reduce the velocity. This is done by concentrating the difference of slope in a few points where vertical falls or rapids are introduced. The location of these is usually fixed by the place where the canal comes too high above the surface of the ground, while their distance apart is so arranged that they shall not have an excessive height or fall. If a canal can be so located and aligned that it will skirt the slopes of the country on a grade contour, it becomes possible to give it the most desirable slope throughout its length without the introduction of falls; but where it runs down the slope of the country, compensation must be made for the difference between the excessive ground slope over that of the canal.

171. Falls and Rapids.—There are two general methods of compensating for slope: one is by the introduction of vertical drops or falls, and the other by the use of inclined rapids or chutes. Falls and rapids are of various kinds and may be generally classified according as they are of wood or masonry. In design the fall may be of three general types: 1, it may have a clear vertical drop to a wooden or masonry apron; 2, the lower face of the fall may be given an ogee-shaped curve (Article 137) with the object of diminishing the velocity and consequent erosive action of the water; 3, the water may plunge into a water-cushion (Article 138). To prevent the scour above the fall induced by the increased velocity of approach; 1, a flashboard weir may be erected at the crest; 2, the channel may be contracted, or 3, gratings may be introduced. To prevent the erosive action in the lower level at the foot of the fall a water-cushion may be employed, or the channel may be increased in width, terminating in wings which shall deflect the eddies back against the fall.

172. Retarding Velocity by Flashboards on Fall Crest.—The effect of a fall is to increase the velocity and to diminish the depth of water for some distance above it. This increase of velocity produces a dangerous scour on the bed and banks of the canal, which in a properly constructed fall is guarded against by means of flashboards or by narrowing the width of the channel. The height to which it is necessary to raise the crest of the fall is found by the following formula devised by Colonel Dyas of the Indian Engineers:

$$h = \left(\frac{900a^2r}{l^2f}\right)^{\frac{1}{3}} - 125.8122\frac{r}{f}, \quad . \quad . \quad . \quad . \quad (1)$$

in which h = height in feet of the water surface above the crest of the fall;

a = the sectional area of the open channel in square feet;

r = the hydraulic mean depth of the same in feet;

l = the length of the crest of the fall in feet;

f = the length of slope to a fall of one in the same.

This formula has been somewhat simplified and modified by Mr. P. J. Flynn in order to make it agree with Kutter's formula. Mr. Flynn finds the discharge over the fall complete to be

$$Q = ml\left(h + \frac{c^2rs}{2g}\right)^{\frac{3}{2}}, \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (2)$$

in which $Q =$ the discharge in second-feet;
$c =$ the coefficient of discharge of open channel;
$m =$ coefficient of discharge over a weir, and varies between 2.5 and 3.5;
$s =$ the sign of slope; and finally he gives the following:

$$h = \left(\frac{a^2c^2rs}{m^2l^2}\right)^{\frac{1}{3}} - \frac{c^2rs}{2g}. \quad . \quad . \quad . \quad . \quad . \quad . \quad (3)$$

If from this value of h we deduct the depth of water in the channel, we have the height to which the weir must be raised above the bed of the canal in order that the water shall not increase in velocity in approaching the crest of the fall.

173. Retarding Velocity by contracting Channel.—If, instead of raising the crest of the fall, it is desired to narrow the channel above the fall in order to diminish the velocity of approach and the consequent erosive action, the amount of narrowing may be calculated by the common weir formula (No. 2) above given, and substituting for Q its value $ac(rs)\frac{1}{2}$, and transposing we finally get

$$l = \frac{2agc}{m} \times \frac{(2grs)^{\frac{1}{2}}}{(2gh + c^2rs)^{\frac{3}{2}}}, \quad . \quad . \quad . \quad . \quad . \quad . \quad (4)$$

in which l is the length of the weir crest or the width of the channel immediately above the fall, in feet.

174. Gratings to retard Velocity of Approach.—Gratings have not been employed on American canals for the purpose of retarding the velocity of approach to the crest of falls, but are used with excellent results on some canals in India. They consist of a number of inclined wooden bars placed just above the crest of the fall, and the method of spacing them is such that the velocity of no one part of the stream shall be either increased or retarded by the proximity of the fall. The wooden bars which rest on one or more overhead cross-beams, are laid at a slope of about 1 on 3, and are made of such length that the full supply level in the canal is half a foot below their ends. In canals with $6\frac{1}{2}$ feet depth of water the following dimensions have been used for the bars: lower end $\frac{1}{2}$ inch broad by $\frac{3}{4}$ of an inch deep; upper end $\frac{1}{4}$ inch broad by $\frac{3}{4}$ of an inch deep. They are supported on 12×12 inch beams, and are placed such distance apart that 18 go into one 10-foot bay.

According to the experience had in India vertical falls terminating in a water-cushion and having gratings above them are the best form that has yet been devised, the erosive action being diminished to a minimum.

175. Simple Vertical Fall of Wood.—On the line of the Calloway canal in California simple flashboard checks similar to the regulating heads are used for the falls. By increasing or diminishing the number of flashboards inserted in these

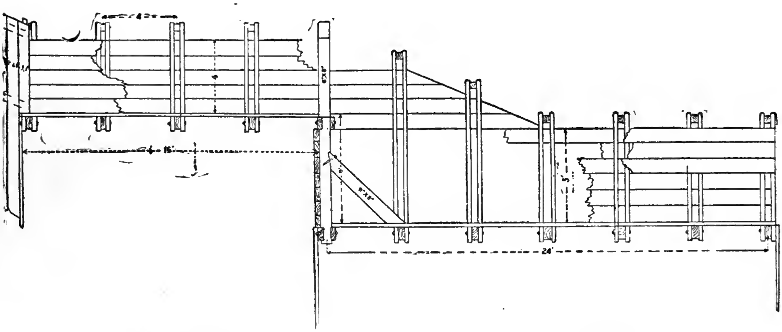


FIG. 42.—LONGITUDINAL SECTION OF FALL, ARIZONA CANAL.

checks the height of fall can be increased or diminished as desired. These checks are inclined at a slight angle to the vertical, and the water drops to a wooden apron or flooring resting on mudsills and protected by sheet piling at its ends, while the bank is protected by wings. On the line of the Arizona canal (Plate IX) a somewhat similar fall is used though the check is vertical. There are a number of these falls, averaging about 5 feet in height each and varying from

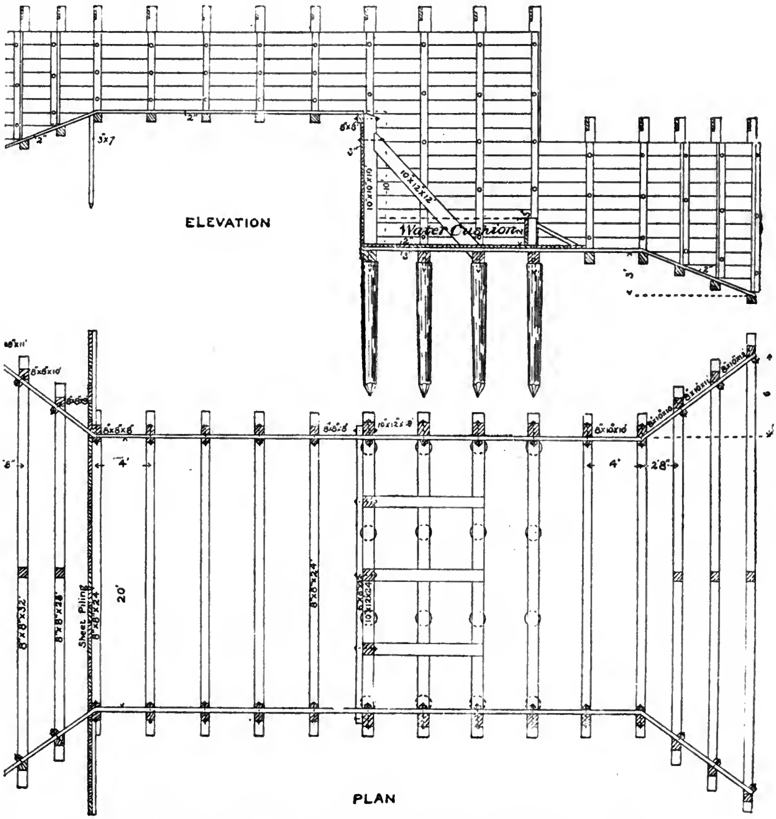


FIG. 43.—PLAN AND CROSS-SECTION OF FALL, BEAR RIVER CANAL.

18 to 21 feet in length on the crest. They consist (Fig. 42) of wooden fluming, the flooring of which is 12 feet in length above the fall, which rests on sheet piling, while the floor below the fall is continued for a length of about 16 feet.

A somewhat similar fall is that employed on the Fresno canal, only in this the flooring of the apron below the fall is

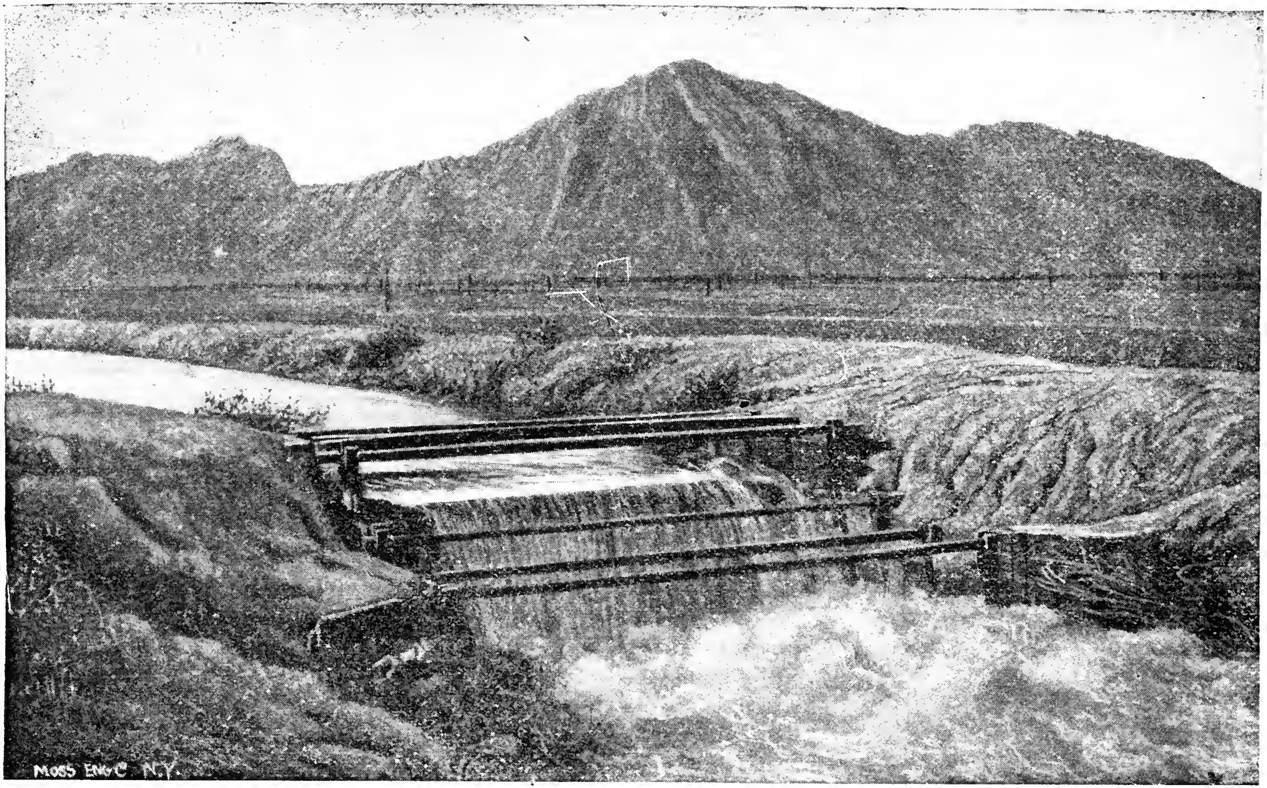


 Plate IX.—View of Fall on Arizona Canal.

depressed $1\frac{1}{2}$ feet below the bed of the canal and an earth filling is placed above this, thus giving a sand box on which the water falls. Above the crest of the fall instead of the horizontal flooring is an inclined apron 12 feet in length and sloping downwards at an angle of 45 degrees.

176. Wooden Fall with Water-cushion.—On the Bear river canal are a large number of falls, ranging from 4 to 12 feet in height (Fig. 43). In these the flooring has been made especially heavy, and above and below the apron it slopes down into the bed of the canal to prevent percolation. On the line of the Turlock canal in California are falls varying from 4 to 11 feet in height. Immediately above these the canal is contracted from its ordinary bed-width of 70 feet to a clear width of 40 feet at the fall crest in order to reduce the velocity and prevent the scour above it. These falls (Fig. 44) are con-

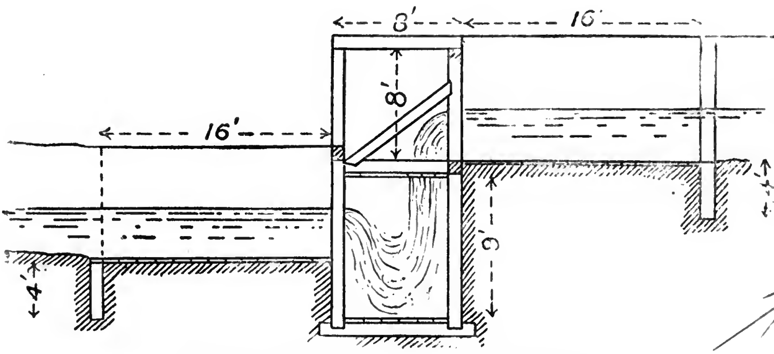


FIG. 44—CROSS-SECTION OF FALL, TURLOCK CANAL.

structed of wood much as are those just described, while below the fall is a depressed water-cushion of such dimensions that for a 5-foot fall the water-cushion is 4 feet in depth, while the 11-foot fall has a water-cushion of 6 feet in depth. Below the water-cushion a wooden apron is carried out for 16 feet, while a similar apron 16 feet in length extends above the fall crest. The falls are divided into four bays of 10 feet each by means of vertical rows of planking in order to direct the current and prevent back eddies.

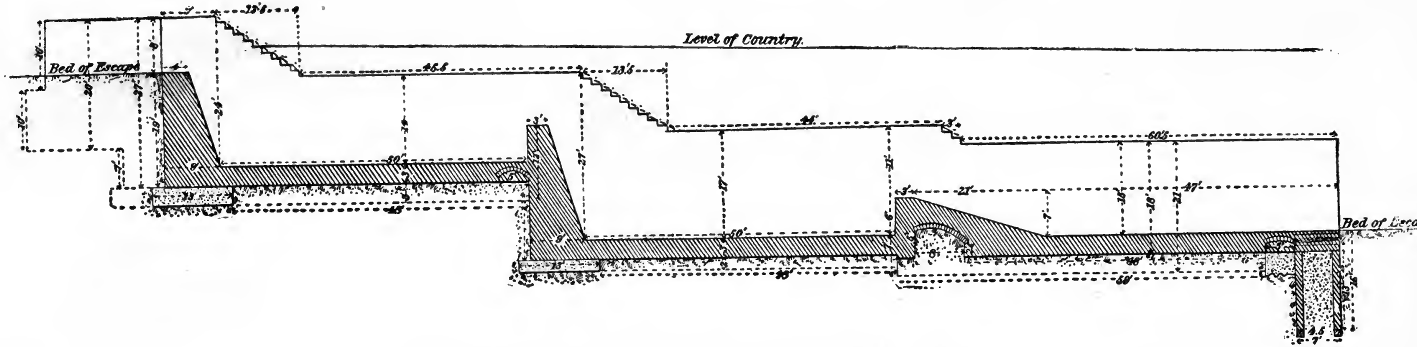


Plate X.—Cross-section of Kushuk Fall, Agra Canal, India.

177. Masonry Falls.—In all the falls employed in India masonry work alone is used. These falls have sometimes simple vertical drops, at others they terminate in water-cushions. It is invariably csutomary, however, in the case of wide canals to divide the falls into bays of 10 feet each, or thereabouts, by means of vertical partitions of masonry in order to prevent scour and back eddy and keep the water moving in a direct course. By this means each may be separately closed and repaired if necessary. An interesting series of two falls terminating in water-cushions on the Agra canal is shown in cross-section in Plate X.

178. Wooden Rapids or Chutes.—A notable wooden rapid is the "Big Drop" on the Grand River canal in Colorado. The canal above the rapid is 30 feet wide and 4 feet deep and is narrowed down at the head of an inclined flume

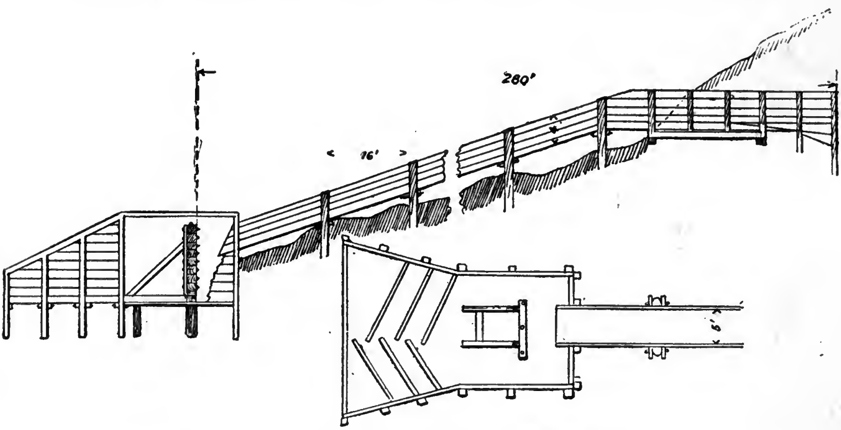


FIG. 45.—PLAN AND ELEVATION OF BIG DROP, GRAND RIVER CANAL.

which forms the rapid to a cross-section of 5 by 4 feet. The flume descends with a total fall of 35 feet in a length of 125 feet (Fig. 45), the water being discharged against a solid bulkhead of timbers which throws it back into a wooden penstock. From this it escapes over a riffled floor 16 feet in length, beyond which is an additional flooring 16 feet in length, whence it emerges in the open canal.

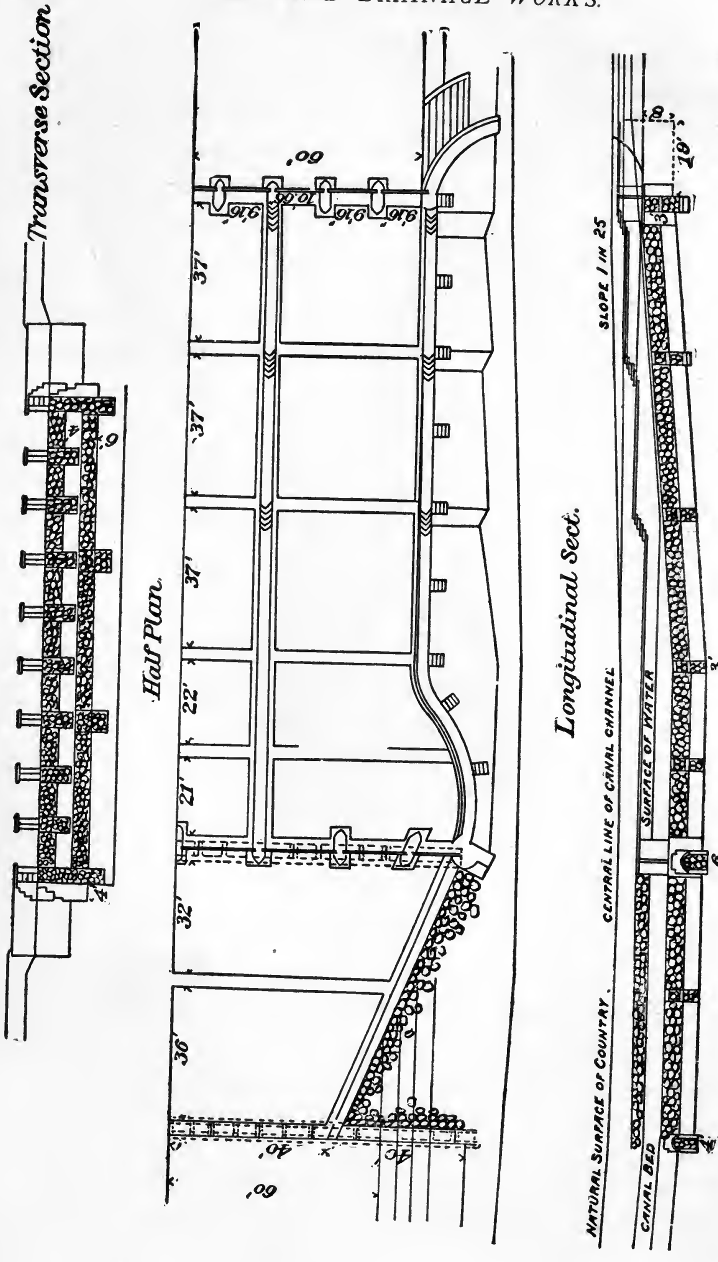


PLATE XI.—PLAN OF RAPIDS, BARI DOAB CANAL, INDIA.

Wooden rapids similar to those just described are employed on the line of the Phyllis branch canal in Idaho. These are practically inclined wooden flumes with slopes of from 1 to 5 in 100 and ranging in height from 12 to 50 feet.

179. **Masonry Rapids.**—On the Bari Doab canal in India rapids paved with loose bowlders have been used with great success. The floors of these rapids (Pl. XI) are confined between low masonry walls so as to prevent the movement of the loose bowlders, and the banks are protected by masonry wings. Bowlders form a better material for the flooring of a rapid than does brickwork, which could not safely be used with velocities exceeding 10 feet per second. The bowlder floors are grouted in mortar and will safely withstand a velocity of 15 feet per second. The tail walls of these rapids are peculiarly carved in order to turn back the current and protect the canal banks from the direct action of the water.

180. **Drainage Works.**—Where the diversion line of a canal is carried around the sides of hills or sloping ground, great difficulties are sometimes encountered in passing side drainage. The higher the canal heads up on a stream the more liable is it to encounter cross drainage. On low slopes much may be done by diverting the watercourses by cuts emptying into natural drainage lines. When this cannot be done it may be passed in one of the following ways:

1. By simple inlet dam;
2. Level crossing;
3. Flume or aqueduct;
4. Superpassage;
5. Culvert or inverted siphon.

181. **Drainage Cuts.**—An instructive example of diversion by means of a drainage cut is the case of the Chuhi torrent on the Bari Doab canal in India. This torrent had two outlets, one running into the Beas and the other into the Ravi river just above the canal crossing. The latter was embanked close to the bifurcation by a bowlder dam, and by this means the water was forced down the Beas and the expense of crossing the canal saved. On the Betwa canal in India is another

interesting diversion cut. The first six miles of this line are protected by a drainage channel 15 feet wide at the bottom and 6 feet deep, which runs parallel to the canal and catches the minor drainage from small streams, which it discharges into the Betwa river above the point of diversion of the canal.

182. Inlet Dams.—Where the drainage encountered is intermittent and its volume is small relatively to that of the canal, much expensive construction may be saved by admitting the water directly into the canal and permitting it to be discharged through the first escape on its line. If the canal crosses a depression in the hillside, a heavy bank will of necessity be built on its lower side to keep the level of its crest at the desired height. The result will be to back the water up the drainage depression, thus causing wastage where water is scarce, as the area of surface exposed to evaporation and seepage is increased. In such a case an inlet dam should be built at the mouth of the depression to confine the canal channel within reasonable limits.

Inlet dams may be of wood, masonry, or loose stone. If the depth of the canal is small and the consequent height of overflow from the crest of the dam to the canal bed small, a wooden fluming or flooring may be laid in the bed of the canal and a barrier or dam of piles and sheet piling be built across the upper side. In the course of a short time the sediment carried by the stream will fill in behind the dam to a level with its crest and the water will simply fall over it onto the wooden apron. The inlet dam may be made as a loose rock retaining-wall when the bed and banks of the canal below and opposite should be riprapped with stone to protect them from erosion. In case the drainage torrent is of some magnitude more substantial works than this may be required, and it may be necessary to build a masonry inlet dam and perhaps to build a portion of the canal channel of masonry, revetting the opposite bank with loose stone.

183. Level Crossings.—When the discharge of the drainage channel is large and it is encountered at the same level as the canal, it may be passed over, under, or through the latter.

In the latter case the water is admitted by an inlet dam on one side and discharged through an escape in the opposite bank. The discharge capacity of the escape must be ample to pass the greatest flood volume likely to enter, and a set of regulating gates must be placed in the canal immediately below the escape in order that only the proper amount of water may be permitted to pass down the canal. The inlet dam must be constructed as described in Article 182, while the escape and regulators should be built of the usual pattern.

On the line of the Turlock canal in California are several level crossings of peculiar design, built where the canal skirts

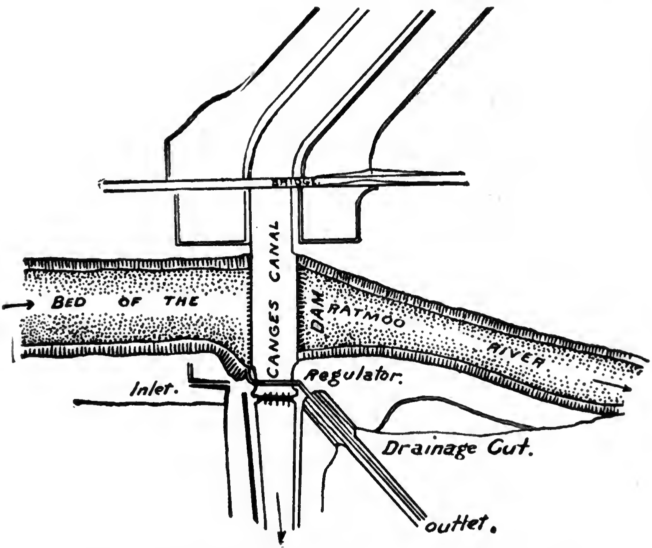


FIG. 46.—PLAN OF RUTMOO CROSSING, GANGES CANAL, INDIA.

steep sidehill slopes, causing the embankment on the lower side to become practically a high earthen dam. The top of the bank is made a little higher, firmer, and wider than elsewhere along the canal line, and in the case of two of these drainage crossings no inlet dam has been constructed. As a result the water is retained on the upper side of the canal as in a large reservoir. With a new canal this has no great disadvantage, as such construction saves considerable expense in the beginning, while in the course of a few years, and by the time the canal water becomes valuable, this reservoir will have silted up and the canal can then be confined between proper limits. These earthen drainage dams are of considerable height, one 23 feet and the other 40 feet high, and in them are constructed escapes, or wasteways for the discharge of surplus waters.

The most interesting level crossing built is that of the Rutmoo torrent on the Ganges canal in India. This consists of a simple inlet at the torrent entrance, of a masonry outlet dam, of an escape regulator in the opposite canal bank, and of a regulating bridge across the canal channel just below the inlet (Fig. 46). The escape dam consists of 47 sluiceways, each 10 feet wide, with their sills flush with the canal bed and flanked on either side with overfalls of the same width with their sills 6 feet higher, while on the extreme flanks are platforms 10 feet above the canal-bed. The closing and opening of these sluiceways is accomplished by means of small flashboards fitting into grooves.

184. Flumes and Aqueducts.—These structures are practically the same, the term flume being more commonly employed in this country to mean a wooden structure for carrying the waters of a canal either around steep rocky hillsides or across drainage lines. The word aqueduct may be more properly applied to those flumes which are of some magnitude and are built of permanent material, as iron or masonry. Where the drainage encountered is at a lower level than the bed of the canal, it may most conveniently be passed under the latter, which crosses over it in a flume. Care must be taken to study the discharge of the stream crossed in order that the waterway under the flume may be made amply great to pass the largest flood which may occur. The foundations of the flume must be substantial, and the area of water-way must not be greatly impeded; otherwise the velocity in the drainage channel will be so great as to cause scour of its bed and perhaps the destruction of the work. Care must be exercised in connecting the ends of the flume with the canal banks on either side so that leakage may not occur at these points.

As the flume or aqueduct is built across a depression, expense in construction is usually saved by limiting the length of the structure as much as possible. This is done by making its approaches on either side of earth embankments, thus causing the canal at either end of the flume to flow on top of an embankment which must be carefully constructed and of ample width in order that it may not settle greatly or be washed away. This embankment must be faced with abutments and wing walls at its junction with the flume in order to protect it against erosion. That the dimensions of the flume may be as small as possible, its cross-section is generally diminished and it is given a slightly greater slope than the canal at either end to enable it to carry the required volume.

185. Sidehill Flumes.—The simplest form of wooden flume is what is generally known as a bench flume, built on steep sidehill to save the cost of canal excavation. Such flumes are common in the West, notable examples of which are the bench flume on the Highline canal in Colorado (Pl. XII) and the great San Diego flume in California. The former was built to avoid expense in construction, its length being a little over half a mile. It is 25 feet wide and 7 deep, its grade being $5\frac{1}{4}$ feet per mile, and its discharge 1184 second-feet. The San Diego flume, on the other hand, was built chiefly to give the canal the most permanent form of water-way and one least liable to the losses of evaporation and absorption. In this case fluming is employed for the entire length of the canal, which is 36 miles.

Such structures should never be built on embankments; they should rest everywhere on excavated material or trestles to avoid the danger of subsidence and consequent destruction. This excavated bench should be several feet wider than the flume, in order to give a place on which loose rock from the sidehills may lodge without injury to the structure, and the flume itself should rest on a permanent foundation of mudsills or posts.

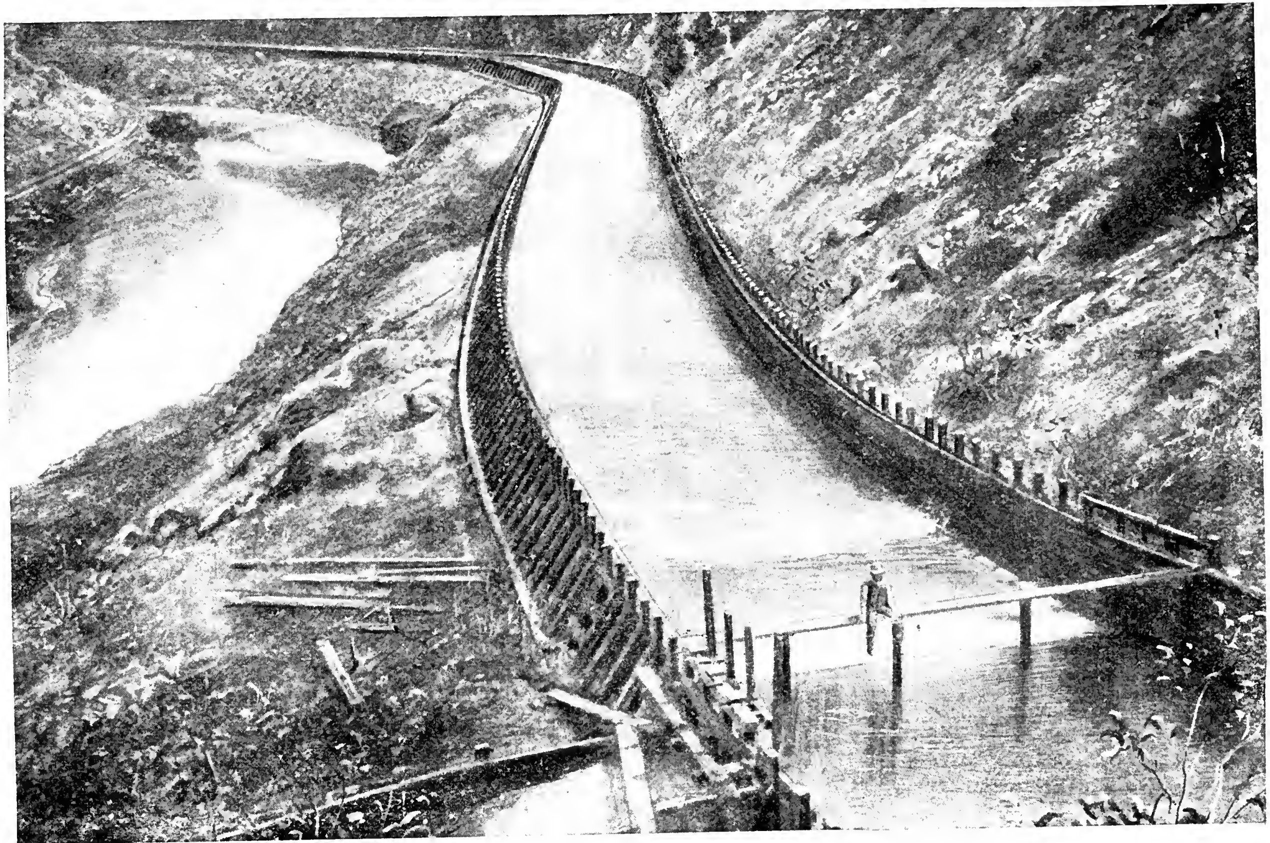


Plate XII.—Highline Canal, Colorado. View of Bench Flume.

186. Construction of Flumes.—The boxing of flumes is generally of three types:

1. The floor may be built directly on stringers and the planking be laid at right angles with the current of the stream.

2. The floor beams may be laid on stringers braced at intervals calculated to bear the water pressure; the standard and floor beams being boxed in and bolted to the outside braces, the whole forming the foundation for putting on the inside sheeting or boxing.

3. The floor beams and stringers may be formed in cross beams yoked to receive the boxing.

The lumber forming the boxing of the flume should be from 1 to 2 inches in thickness, according to the dimensions of the flume, and all joints should be calked with oakum. An

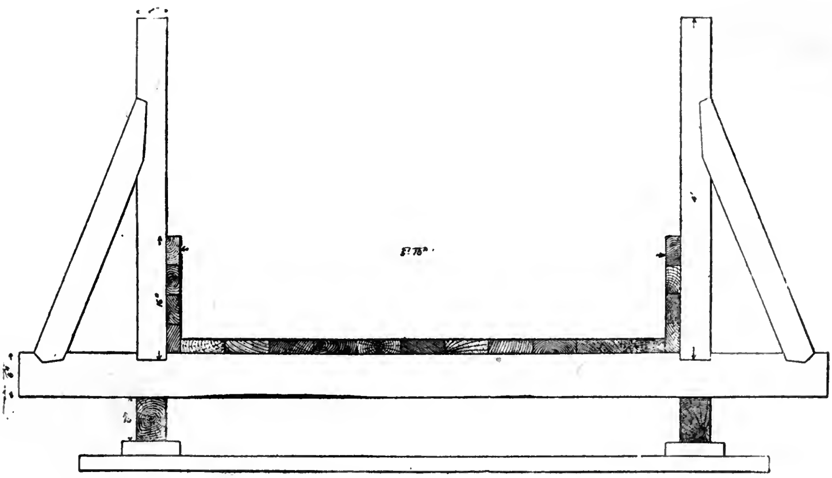


FIG. 47.—CROSS-SECTION OF SAN DIEGO FLUME.

excellent example of bench flume is that of the San Diego Flume Company (Fig. 47), which is 6 feet wide in the clear and 4 feet high; the bottom and sides are planked with 2-inch redwood, and the boxing rests on transverse sills of 2-inch planking laid 4 feet apart, and upon these are 4 by 6 longitudinal stringers, above which is constructed the framework of

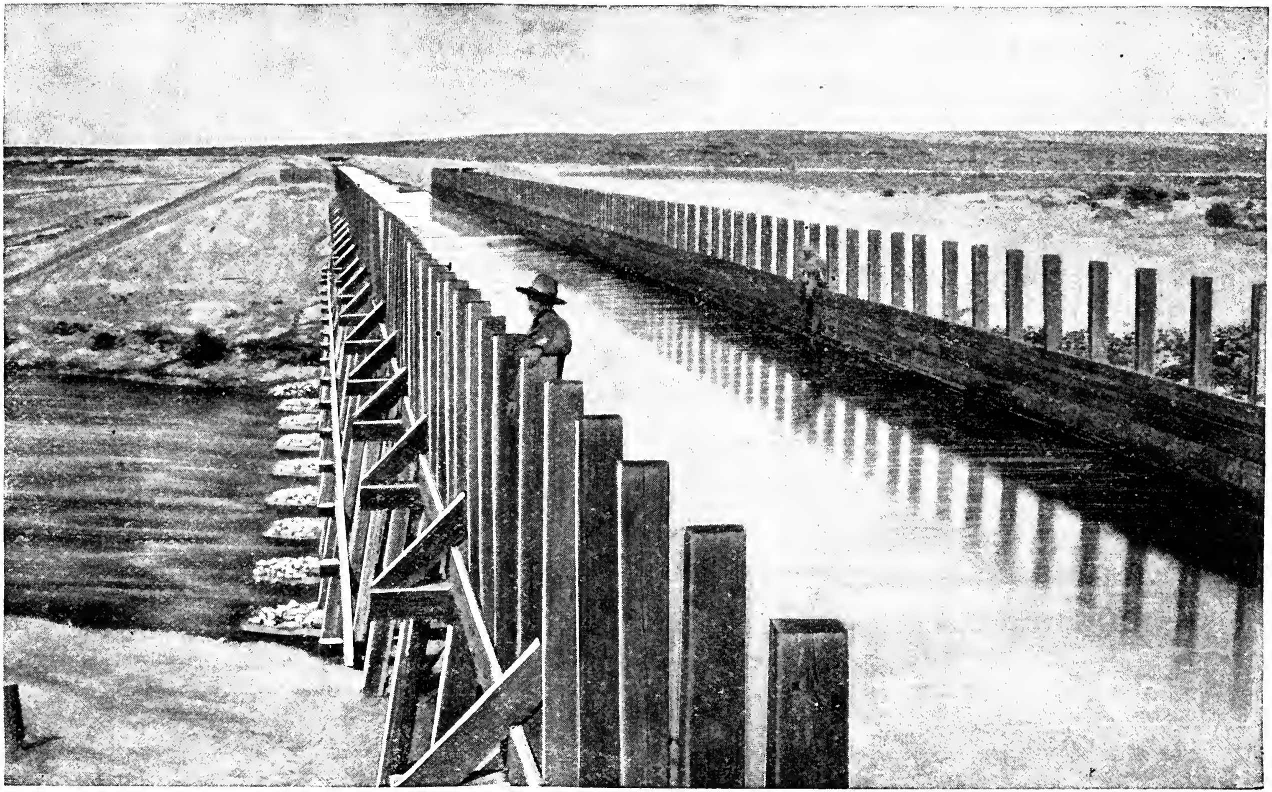


PLATE XIII.—VIEW OF PECOS FLUME.

the flume, consisting of 4 by 4 scantling placed at intervals of 4 feet and braced by diagonal uprights 2 by 4 inches and 3 feet in length.

187. Flume Trestles.—Where the flume crosses a depression it rests on trestles. These are constructed as are the ordinary trestles on railway lines, and are built of various designs. Where the trestle rests on dry ground it may be founded on mudsills or on short posts let into the soil, but where it crosses drainage channels it must be substantially founded on cribs or piling. The superstructure of a flume crossing a drainage line is similar to that of bench flumes. A large and imposing flume is that across the Pecos river in New Mexico (Pl. XIII). The approaches to this flume consist of a terre plein or raised embankment 105 feet wide at the base, 24 feet in maximum height, and 80 feet wide on the top, while the top width of the canal is 70 feet, thus giving 5 feet in width of embankment for the canal channel. The flume terminates at either end in substantial wooden wings extending for 12 feet into the earth embankments and well braced and supported by sheet and anchor piling. This flume is 40 feet in height above the river, 25 feet wide, 8 feet deep, and 475 feet long, and rests on a substantial trestlework, the spans of which are 16 feet in length.

188. Iron Aqueducts.—But few of these have been constructed, though it is probable that they will continue to grow in favor and will be largely substituted for wood. The chief difficulty encountered in constructing long aqueducts of iron has been the expansion and contraction of the metal, though in fact this has proven to be an imaginary rather than a real danger. In practice it has been found that the metal of the structure has approximately the same temperature as that of the water, and as this is somewhat uniform but little change takes place in the dimensions of the aqueduct. On the Bear River canal in Utah are two aqueducts, one of which consists of a wooden flume resting on iron trestles founded on masonry columns. The other is a simple iron aqueduct resting on iron trestles. The floor of this is 37 feet above the bed of the stream, and its length is 130 feet (Fig. 48), disposed in three bents the centre span of which is 60 feet long, the other two being respectively 25 and 45 feet long. This aqueduct is essentially a plate-girder bridge resting on iron columns and founded on iron

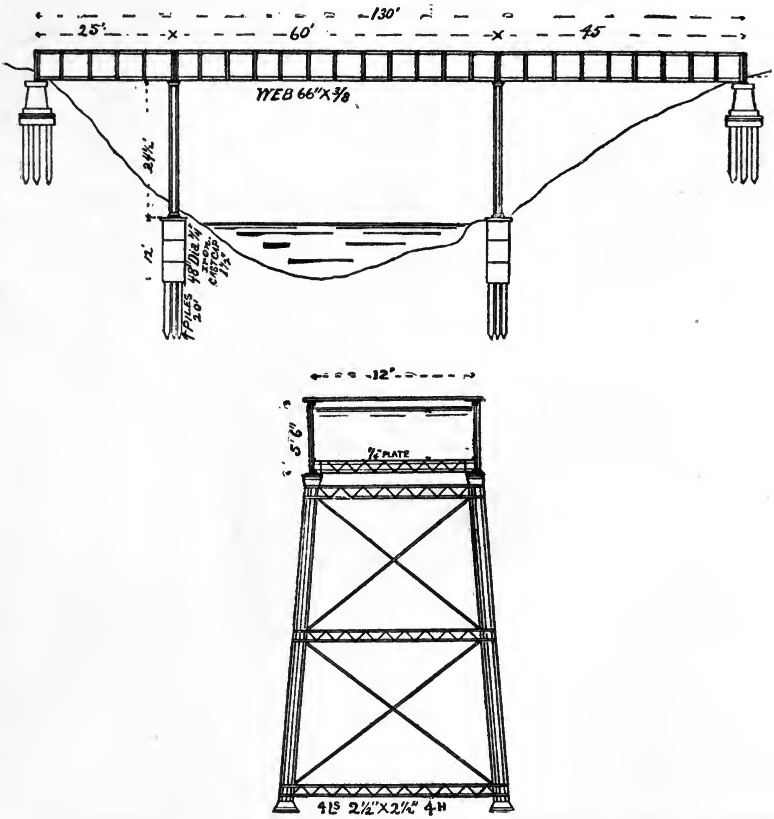


FIG. 48.—BEAR RIVER CANAL. ELEVATION AND CROSS-SECTION OF IRON FLUME ON CORINNE BRANCH.

cylinders filled with concrete and resting on piles. The plate girders forming the sides of the aqueduct are $5\frac{1}{2}$ feet in depth, the available depth of water being 4 feet. The sides of the girder are braced by vertical angle-iron riveted to it every 5 feet apart, while the top is cross-braced by similar angle-iron.

These angle-irons vary between 3 and 4 inches in width, while the web of the sides of the aqueduct consists of ⅜-inch iron.

On the Henares canal in Spain is an iron aqueduct over the Majanar torrent. This aqueduct is 70 feet long with a clear span of 62 feet. Its water-way is 10.17 feet wide, its capacity being 177 second-feet. The sides are composed of box girders 6.2 feet deep (Fig. 49), and each girder is calculated

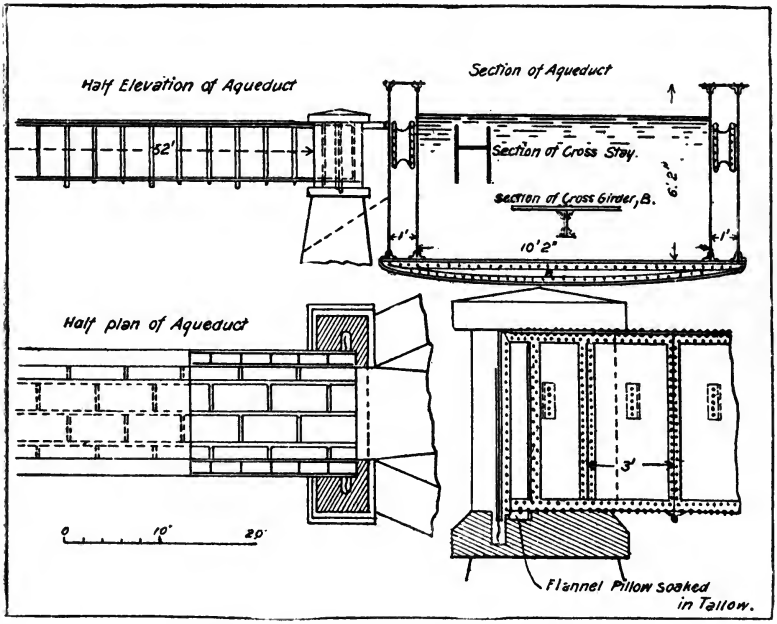


FIG. 49.—AQUEDUCT, HENARES CANAL, SPAIN.

to bear 200 tons or the entire structure to carry 400 tons. To prevent leakage the ends of the aqueduct rest on stone templates, and 4 inches from each end is a pillow composed of long strips of felt carpet 9 inches wide and soaked in tallow, which is let into the stone below the aqueduct. This presses on it with its full weight, thus making a water-tight joint. In addition to this lead flushing is riveted to the aqueduct and let into a recess of the stone abutments. This recess is 12 inches deep and 4 inches wide, and around it is poured, hot, a

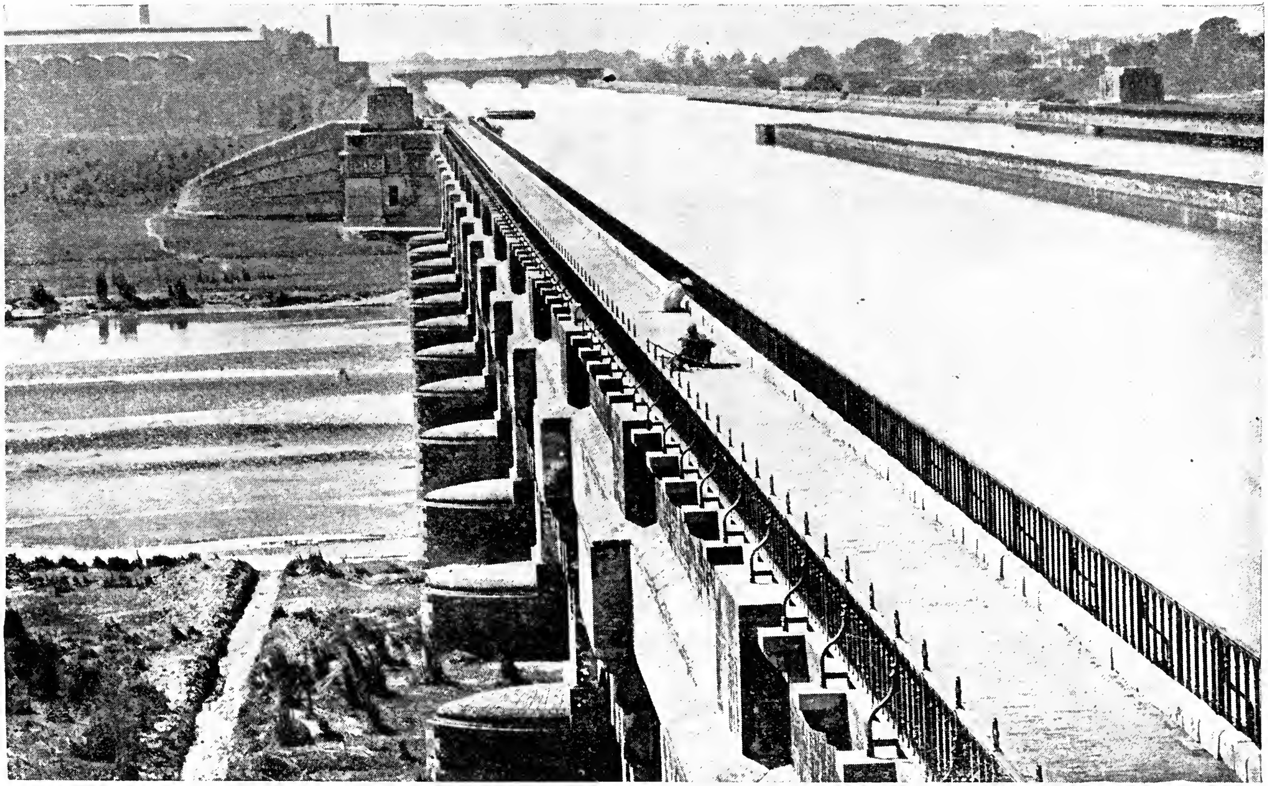


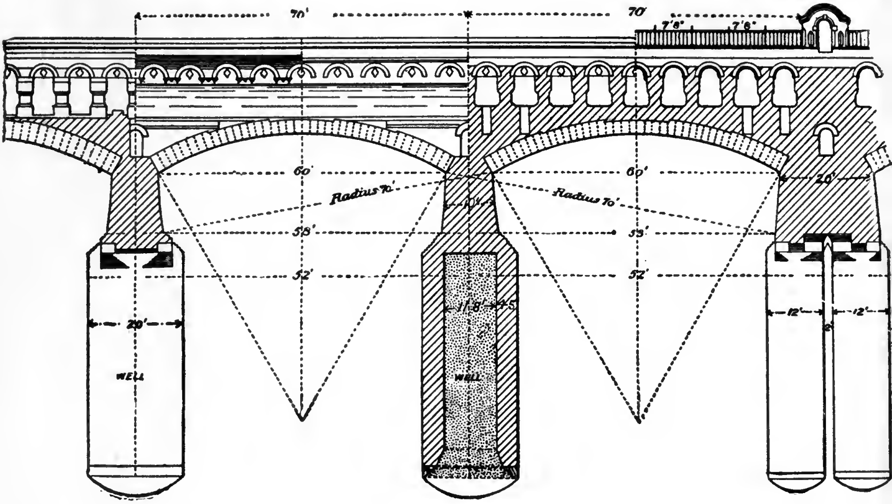
PLATE XIV.—VIEW OF SOLANI AQUEDUCT, GANGES CANAL, INDIA.

mixture of tar, pitch, and sand, which allows slight play during its expansion and contraction and yet is water-tight.

189. Masonry Aqueducts.—In general design masonry aqueducts are planned and constructed much as are those of wood or iron. One of the greatest structures of this kind is the Solani aqueduct on the Ganges canal in India (Pl. XIV). This consists of an earth embankment approach or terre plein $2\frac{3}{4}$ miles in length across the Solani valley, its greatest height being 24 feet. This embankment is 350 feet wide at the base and 290 feet wide on top, and on this the canal banks are formed, the width of the banks being 30 feet on top and the bed-width of the canal 150 feet. The aqueduct is 920 feet in length with a clear water space between piers of 750 feet, disposed in fifteen spans of 50 feet each. The breadth of each arch parallel to the channel of the river is 192 feet and its thickness 5 feet. The greatest height of the aqueduct above the river valley is 38 feet, and the walls of the water-way are 8 feet thick and 12 feet deep. This structure is founded on masonry piers resting on wells sunk 20 feet in the river bed.

Perhaps the most magnificent aqueduct ever built is that carrying the Lower Ganges canal across the Kali Nadi torrent in India (Plate XV). The present structure was built to replace another of similar design which was destroyed by a flood which the water-way under the aqueduct was too small to pass. This was calculated to discharge 30,000 second-feet, whereas the flood which destroyed it amounted to 135,000 second-feet in volume. The present aqueduct consists of fifteen masonry spans each 50 feet in width and supported on masonry wells sunk to a maximum depth of 50 feet. Under the aqueduct is built up a concrete floor 5 feet in thickness to prevent erosion and destruction of the foundation.

190. Superpassages.—Where the canal is at a lower level than the drainage channel, a superpassage is employed to carry the latter over the canal. A superpassage is practically an aqueduct, though there are some elements entering into its design which are different from those affecting aqueducts. The volumes of streams which are to be carried in superpas-



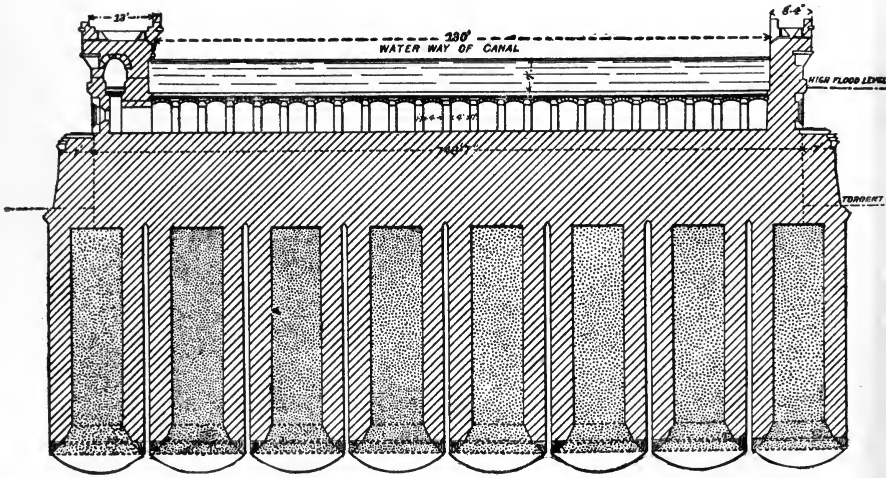


PLATE XV.—ELEVATION AND CROSS-SECTION OF NADRAI AQUEDUCT, LOWER GANGES CANAL, INDIA.

sages are variable; at times they may be dry, while at others their flood discharges may be enormous. No provision has to to be made for passing flood waters under the structure, since the discharge of the canal beneath it is fixed. On the other hand, the water-way of the superpassage must be made amply large to carry the greatest flood which may occur in the stream, and much care must be taken in joining the superpassage to the stream bed above and below to prevent injury by the violent action of the flood waters.

No instances can be cited where superpassages have been constructed in the United States. In nearly every case where these would have been required the canal has been taken under the stream-bed in an inverted siphon. In India, however, superpassages have frequently been used on the canals, where they have been employed in preference to inverted siphons chiefly because of the requirements of navigation. It would probably be a dangerous experiment to attempt to construct a superpassage of wood, because it would be so constantly subjected to alternate drying and wetting, according as there was or was not water flowing in the stream, that it would soon decay. A small iron superpassage has been constructed across the Agra canal in India which is 99 feet long, 30 wide, 10 feet deep, and is constructed of boiler-iron strongly cross-braced. It is well built and is supported on masonry piers. Its slope is steep, thus giving a high velocity. The connection between its ends and the abutments is made by means of heavy sheet lead to accommodate the changes due to expansion of the iron. This precaution is more necessary in a superpassage than in an aqueduct, as it is more subject to changes of temperature when empty.

On the Ganges canal in India are two of the largest and most interesting superpassages ever constructed. One carries the Puthri torrent and the other the Ranipur torrent over the canal. The discharge of the former amounts in times of flood to as much as 15,000 second-feet. The Ranipur superpassage (Pl. XVI) is built of masonry founded on wells, and its flooring, which is given a steep slope in order that the velocity shall

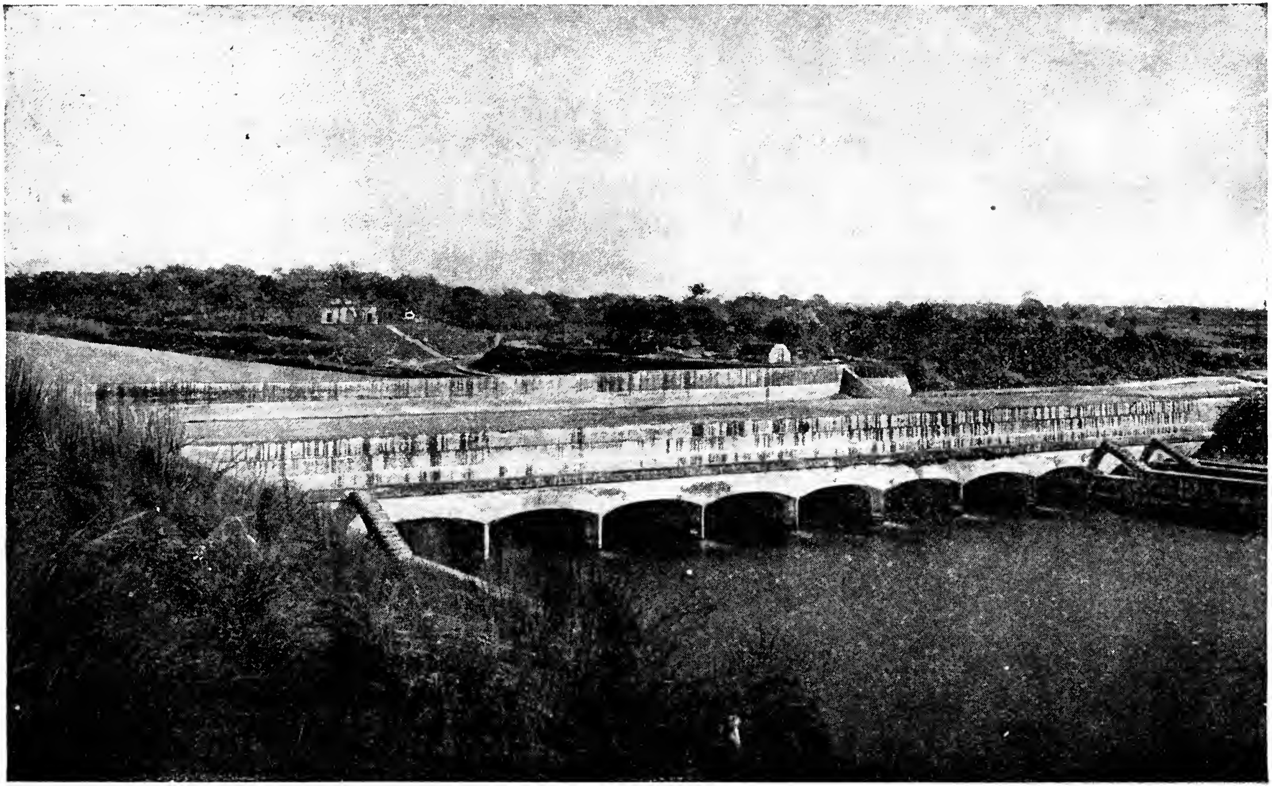


PLATE XVI.—VIEW OF RANIPUR SUPERPASSAGE, GANGES CANAL, INDIA.

prevent its filling up with sediment, is 3 feet in thickness above the crown of the arches and is bordered by parapets 7 feet wide and 4 feet high. The flooring and parapets continue inland from the body of the work a distance of 100 feet on each side, the latter expanding outward so as to form wings to keep the water within bounds. The superpassage is 300 feet long and provides a water-way 195 feet wide and 6 feet deep.

191. Inverted Siphons.—Where the canal is not used for purposes of navigation and encounters drainage at a relatively low level, the most convenient and usual form of crossing is by means of inverted siphons. The ordinary method of using these is to carry the water of the canal in the siphons under the stream, though sometimes the stream is carried in the siphon and the canal is taken over this in a half aqueduct. The dimensions of the siphon are to be computed by means of one of the many formulas for the flow of water through pipes, though the formula for flow through channels may also be used in some cases. Many examples of these are to be found in works on hydraulics, and therefore they will be but briefly referred to here.

To find the velocity of flow in a pipe, given its diameter, length, fall, and value of n, or the coefficient of roughness, we can use the formula $v = c\sqrt{rs}$. To determine the discharge we can use the formula $Q = av$, or the velocity into the cross-section. The various other dimensions of the pipe, such as the velocity and grade given to find its diameter, are obtained in like manner from these formulas by looking up their equivalent values in published tables.

192. Inverted Siphon of Wood.—An excellent example of a small work of this kind is the wooden culvert or inverted siphon used on the Del Norte canal in Colorado (Fig. 50). This consists of two parallel wooden boxes, each 4 feet 6 inches wide by 3 feet high, supported on piling and framed and braced with 6 by 8 scantling. The bottom and sides are floored with 2-inch plank, while the top, which has to bear the weight of the superincumbent earth and water, is covered with 6-inch planking laid crosswise.

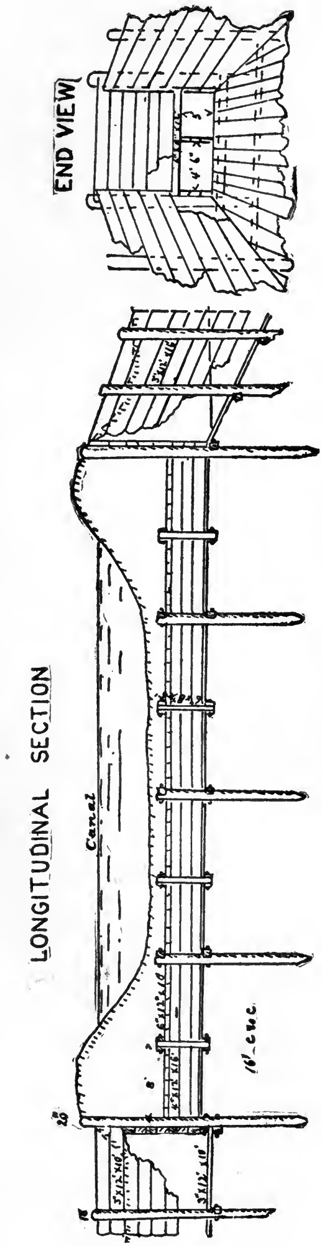


FIG. 50.—SECTIONS OF WOODEN SIPHON, DEL NORTE CANAL.

A most interesting wooden siphon is that which carries the Central Irrigation District canal under Stony creek in Colusa county, California. In addition to acting as a conduit for the waters of the canal it is so arranged as to act as an escape and regulating gate to the canal, while its crest acts as an inlet from the creek. The length of the siphon is 650 feet, and it terminates at either end in an inlet and outlet masonry well protected by substantial walls and approaches, as shown in Pl. XVII. This siphon consists of seven parallel lines of semicircular wooden tubing fastened under a horizontal platform of wood the top of which is level with the stream-bed. Above and below the platform in the creek-bed are wooden aprons, while light training works keep the current of the stream in its channel. At the inlet to the culvert are a set of simple flash-board regulating gates which act as an escape to the canal. The outlet culvert-well is planned as a simple inlet to the canal. As shown in the illustration, the semicircular wooden culvert rests on a bed of concrete $1\frac{1}{2}$ feet in thickness. The tubes of the culvert are each 5 feet 5 inches in diameter and consist of $2\frac{1}{2}$-inch staves laid longitudinally and bound together by semicircular iron hoops which terminate in bolts above the platform floor.

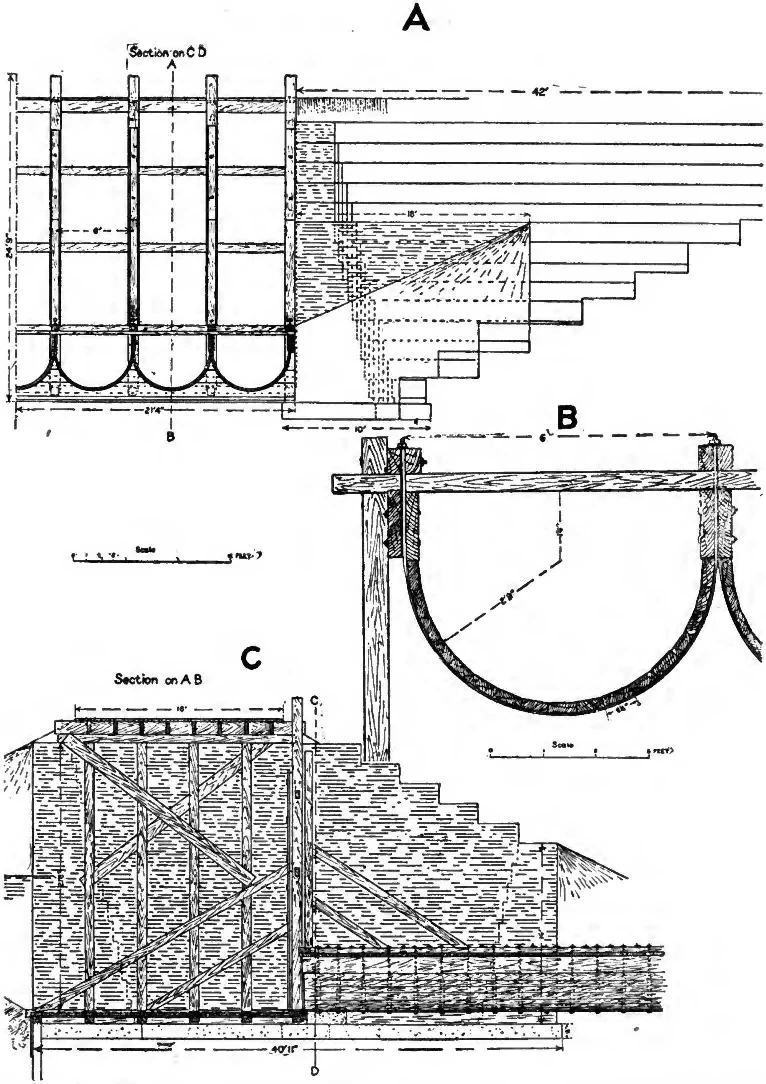


PLATE XVII.—CENTRAL IRRIGATION DISTRICT CANAL. ELEVATION AND CROSS-SECTION OF STONY CREEK CULVERT.

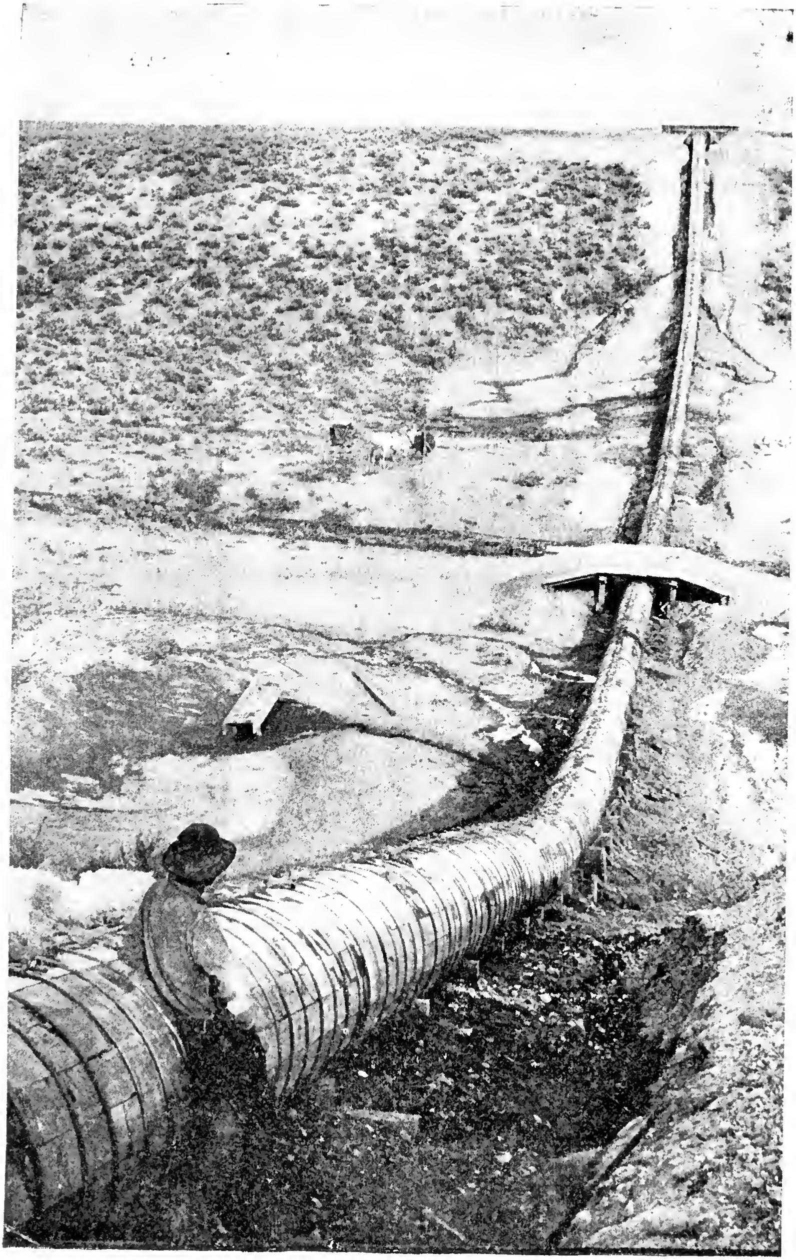


Plate XVIII.—Idaho Irrigation Company's Canal. View of Wooden Siphon on Phyllis Branch.

Instead of this form of built-up wooden inverted siphon, ordinary wrought-iron, cement, or wooden pipes are frequently employed, especially where the head is great. These wooden

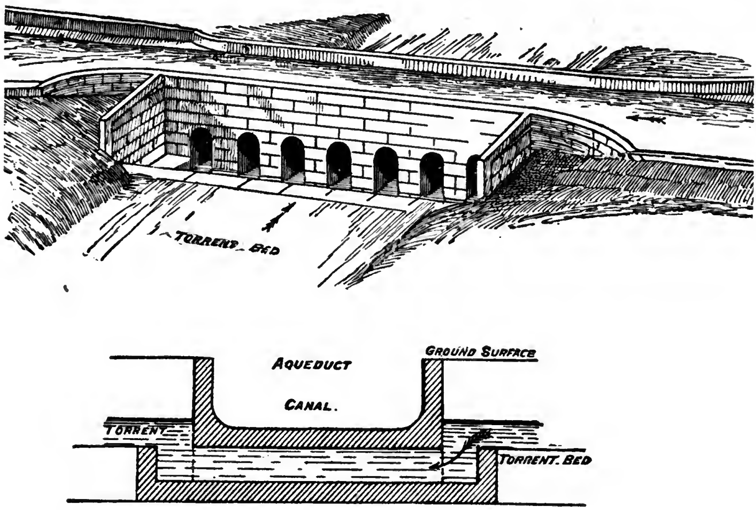


FIG. 51.—SOANE CANAL. CROSS-SECTION OF KAO NULLA SIPHON-AQUEDUCT.

pipes may be of the ordinary wrought-iron hydraulic mining type or of the same type as the Colorado wooden pipe described in article 204 (Pl. XVIII).

193. Inverted Siphons of Masonry.—An interesting structure of this kind which is practically a siphon aqueduct, since the waters of the stream are carried under those of the canal, is that carrying the Kao torrent under the Soane canal in India (Fig. 51). This work is built of the most substantial masonry, the area of the superstructure being contracted and given a slightly increased grade to carry the waters of the canal, while the waters of the torrent flow over a masonry floor which is depressed a few feet.

The most magnificent masonry siphon ever built is that carrying the waters of the Cavour canal under the Sesia river in Italy. Its total length is 878 feet and it consists of five oval orifices (Fig. 52) each 7.8 feet in height by 16.2 feet in width, the amount of depression of the water surface in the canal being $7\frac{1}{2}$ feet. The siphon consists of a substantial concrete floor or foundation $11\frac{1}{2}$ feet in thickness under the river bed, its roof forming the floor of the river channel and being about 3

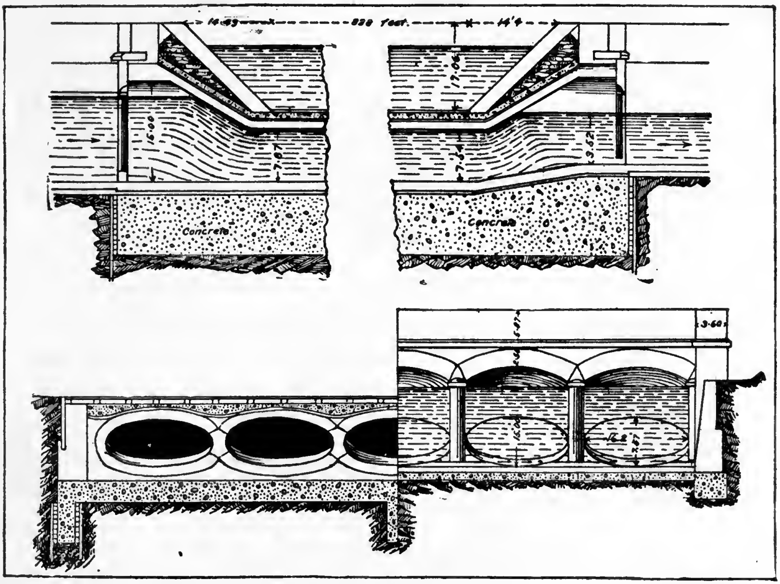


FIG. 52.—SECTIONS OF SESIA SIPHON, CAVOUR CANAL, ITALY.

feet in thickness. Another large siphon is that on the Sirhind canal in India crossing the Hurron torrent. The total length of this is 212 feet, and it consists of two openings each 4 feet high by 15 feet wide. The water drops from the canal almost vertically into a well the floor of which is on a level with the floor of the siphon, while at its exit it is raised again to the level of the outlet canal up an incline built in steps.

CHAPTER XV.

DISTRIBUTARIES.

194. Object and Types.—Distributaries are to a main canal system what service pipes are to the mains in city water service. The minor ditches or laterals which are owned by the irrigators and from which water is directly applied to the crops should never be diverted from the main canal nor from its upper branches. It is desirable to have as few openings in the bank of the main canal as possible, so as to reduce to a minimum the liability of accident. The water is drawn at proper intervals from the main line into moderate-sized branches which are so arranged as to command the greatest area of land and to supply the laterals and small ditches of the irrigators in the most direct manner. Wherever water has not a high intrinsic value it is conducted to the lands in open distributaries and laterals excavated in the earth. Where, however, its value is relatively high and it is scarce it is desirable to reduce the losses from percolation and evaporation to a minimum. In such cases the distributaries consist of wooden flumes or of paved or masonry-lined earth channels, while in extreme cases, such as are frequently encountered in Southern California, water is conducted underground to the point of application in pipes, and is applied to the crops from these instead of being flowed over the surface. By such methods of handling the highest possible duty is obtained and the most effective use made of the water at command.

195. Location of Distributaries.—Distribution from a canal is most economically effected when it runs along the summit of a ridge so that it can supply water to its branches and to private channels on either side. In the case of main canals this location can be made only in occasional instances; but the distributaries taken from these mains should be made to conform to the dividing lines between watercourses. The capacity of the distributaries which then traverse the separate drainage divides are proportioned to the duties they have to perform, the natural bounding streams limiting the area they have to irrigate.

In designing a distributary system too little care and attention are ordinarily paid to its proper location and survey; yet it is in the distribution and handling of water that the greatest losses occur, and accordingly it is there that the greatest care should be taken in its transportation. Careful surveys should be made of the area to be traversed by the distributaries, as described in Chapter XI for the location of main canals, and the greatest care should be taken to balance cuts and fills and to so locate the distributaries that the least loss of water shall occur from percolation.

In Fig. 53 is shown an ideal distributary system. The con-

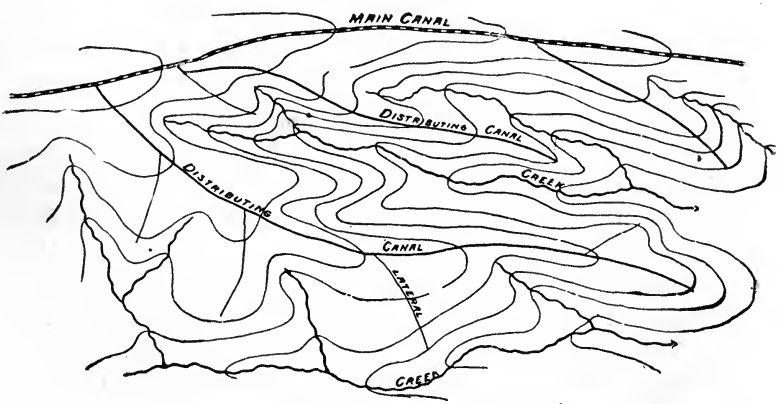


FIG. 53.—DIAGRAM ILLUSTRATING DISTRIBUTARY SYSTEM.

tour lines and drainage courses show the general slope and lay of the country, and the main canal and its tributaries should be run down the divides between these drainage lines as indicated. Such an arrangement enables the least mileage of channels to command the greatest area of country by furnishing water to both sides of its line. At the same time perfect drainage is obtained by the water flowing in both directions into the natural watercourses.

196. Design of Distributaries.—For the more complete and efficient distribution of water the engineer treats distributaries as of as much importance as the main branches. Attention is devoted to the character of the soil traversed, to the alignment, to the safe and permanent crossing of natural drainage lines, and especially to so maintaining the surface of the canal with relation to the ground as to command the largest irrigable area. In all well-designed distributary systems the capacity of the channels is exactly proportioned to the duty to be performed, the cross-sectional area being diminished as the quantity of water is decreased by its diversion to private watercourses.

The distributary should be taken off from the main canal as near the surface of the latter as possible. That is, the bed of the distributary should not be on a level with the bed of the canal, but should be placed with reference to the full supply of the main canal, in order to get the clearest water, and in order that the bed of the distributary may be kept at a high level and admit of surface irrigation throughout its length. In level country great care should be taken in designing distributaries that the natural drainage lines into which they tail shall be sufficiently large to accommodate any flood volume it may be necessary to pour into them; otherwise the stream courses might become clogged and flood the surrounding country. In order to avoid the construction of costly embankments and to insure the surface of the water being above that of the country, the slope of the distributary should be made as nearly parallel as possible to that of the land it traverses. To effect this alignment falls must be frequently introduced; and to dispose of storm-waters escapes into natural drainage lines should be provided at least every 10 miles in the course of the distributary.

196. Efficiency of a Canal.—According to Mr. J. S. Beresford, an Indian engineer, we may look upon a great canal system as a machine composed of four parts and calculate its efficiency in the same way as that of a steam-engine. These parts are:

1. The main canal;
2. The distributaries;
3. The private irrigating channels;
4. The cultivators who apply the water to the soil.

Each cubic foot of water entering the canal head is expended in five ways:

1. In waste by absorption and evaporation in passing from the canal head to the distributary head.

2. In waste from the same causes between the distributary head and the head of the private channel.

3. In waste from the same causes in passing from the private channel to the field to be watered.

4. In waste by the cultivators in handling the water, both by causing losses from evaporation or from percolation where an unnecessary amount is applied.

5. In useful irrigation of the land.

The object is plainly to increase the last item by the reduction of all the rest. Calling D^t the theoretic duty of a foot of water entering the canal head, we have the actual duty of the canal

$$D = C^{me} \times D^t, \quad \ldots \ldots \quad (1)$$

where C^{me} represents the mean efficiency of the main canal. Now if the efficiency of water entering a distributary head for use in watering a field from an outlet is called E, the duty of water used in this field will be

$$D = E \times D^t \quad \ldots \ldots \quad (2)$$

and

$$E = E^d \times E^w \times E^c, \quad \ldots \ldots \quad (3)$$

where E^d is the efficiency of the distributary, E^w is the efficiency of the private watercourse between its head and the field, and E^c is the efficiency of the cultivator who waters the field.

The efficiency of any distributary is the fraction whose denominator is the quantity entering the distributary head, and the numerator this same quantity minus the loss down to the point in question. If W represents the waste down to any outlet, Q the discharge at the head of the distributary, and E^o the efficiency at the point under consideration, then

$$E^o = \frac{Q - W}{Q} = 1 - \frac{W}{Q}. \quad \ldots \ldots \quad (4)$$

The waste W, down to any point may approximately be expressed as the product of the loss of the first mile into some function of the length, or

$$W = AP \times L^x; \quad \ldots \ldots \ldots \quad (5)$$

or substituting in the above equation, we get

$$E^o = 1 - \frac{AP \times L^x}{Q}, \quad \ldots \ldots \quad (6)$$

where AP is the ascertained loss by absorption and percolation in the first mile and L^x is some function of the length, which will be found by experiment to be about $\frac{5}{6}$ or $\frac{6}{7}$ of L in most cases, or near the head of the distributary L^1.

Taking l as the length of the private watercourse, q as its discharge, and l^x as the same function of its length as in the case of L^x, we have the efficiency of the private channel

$$E^w = 1 - \frac{ap \times l}{q}. \quad \ldots \ldots \ldots \quad (7)$$

The efficiency of the cultivator E^c varies between .5 and .9 where unity represents his efficiency at the theoretical limit. Now for an outlet at the head of the distributary and with the irrigating field close to this outlet. $L = 0$ and $l = 0$.

Therefore the second terms of the equations (6) and (7) vanish and E^o and $E^w = 0$.

An application of these rules as laid down by Mr. Beresford is given in the following cases: Say the discharge $Q = 50$ cubic feet; that the outlet is at the 10th mile, whence $L = 10$; the losses from percolation, etc., being 1.25 in the first mile and $x = \frac{5}{6}$. The discharge of the watercourse $q = 1$ cubic foot, $l = 6$ furlongs, and $ap = .03$ of a cubic foot per furlong. Then

$$E^o = 1 - \frac{1.25 \times 10^{\frac{5}{6}}}{50} = .829;$$

$$E^w = 1 - \frac{0.3 \times 6^1}{1} = .820;$$

$$\text{say} \quad E^c = .75;$$

$$\text{and} \quad E = .829 \times .82 \times .75 = .51;$$

or leaving out the cultivator, this is equal to .68. That is, of each cubic foot entering the distributary head only .68 of a cubic foot is available at the 10th mile and 6 furlongs. Whatever the actual amount of loss in either distributary or private channel, it varies directly with L and l; it also varies directly with AP and ap, and great waste is due to the cultivator if he is careless. It will thus be seen from the above that every effort should be made to reduce the value of AP and to induce the cultivator to use the greatest possible care in handling the water.

198. Private Watercourses.—As a result of Mr. Beresford's experiments it is evident that the widest field for improvement is in the private watercourses. As generally constructed these are much longer than is necessary, and are usually so constructed as to avoid low lands, whereas flumes or proper alignment would remedy this. They often run long distances through sandy soil, which absorbs the water, and frequently parallel each other, thus adding to the losses by absorption by unnecessarily increasing the wetted perimeter.

Where sandy soil is encountered or depressions are to be crossed the channels should be puddled or flumes employed.

199. Dimensions of Distributaries.—Experiments made in India show that the greater the amount of water discharged by a distributary the smaller will be the proportion of cost of maintenance. Thus a channel 12 feet wide discharges more than double the volume discharged by two channels each 6 feet wide, while the cost of patrolling and repairing the banks would be half that of both the smaller ones. Experience has proved that irrigation can be most profitably carried on from channels 18 feet wide at the bottom and carrying about 4 feet in depth of water. Thus on the eastern Jumna canals during the years 1858 to 1860, inclusive, the expenditure of water on all the distributaries of 12 feet bed-width and upwards was 0.123 of the revenue, while on all those below 12 feet it was 0.223 or nearly double that of the first. From the same examinations the relative value per cubic foot per annum on channels of respectively 12, 6, and 3 feet in bed-width was as 10 : 7 : 4. The increased action of absorption in small channels with diminished volumes and velocities accounts for the difference. The depth of water should accordingly seldom be less than 4 feet and the surface of the water should be kept at from 1 to 3 feet above that of the surrounding country; not only to afford gravity irrigation, but because the loss by absorption is thereby diminished.

The principle which is so commonly employed in the West on minor private channels of diverting the water by raising it to the surface of the country by means of earth check-dams, or by introducing plank stops in grooves, is to be condemned. It converts freely flowing streams into stagnant pools, encourages the growth of weeds and the deposit of silt, and produces an unhealthy condition of the neighborhood. It is moreover extremely wasteful of water, since much of the latter is dissipated because of loss of head and because of absorption and evaporation. Where these stop planks or checks are used in private channels with a view to diverting the water to the irrigable fields, little or no damage is done, since the planks remain in but a short time, during which no damage is likely to occur.

200. Distributary Channels in Earth.—The cross-section of the main or larger distributaries should be relatively the same as for main canals (Articles 117 to 120.) In designing the canal banks their top width should be sufficient to admit of easy inspection. On moderate-sized distributaries 3 feet may be taken as the minimum width. Should the cut not be so deep that a berm is necessary, it is always well to let the latter slope away from the canal and be drained off through the bank. The top of the bank likewise should not be level but should drain away from the canal. For smaller distributaries or minor private channels a small trapezoidal cross-section both for the bank and the canal will usually be sufficient, and as far as possible the larger portion of this cross-section should be in embankment, thus keeping the water above the level of the surrounding country. In such small channels it is not necessary to construct berms, to give subgrades or other complex cross-sections.

201. Wooden Distributary Heads.—Distributary heads on Western canals are arranged much as are the heads of main canals and escapes. They consist essentially of two parts, a regulator or check below the head on the main canal, in order to divert the water into the distributary, and a regulating gate in the latter to admit the proper amount of water. These heads usually consist of a wooden fluming, which is practically an apron to the bed of the distributary and planking to protect the banks. Inthis fluming are inserted the gates, which consist either of flash boards, as in Kern county, California, or of simple wooden lifting gates, as in most other portions of the West.

In Fig. 54 is shown a distributary head on the line of the Calloway canal in California. Immediately below the regulator is shown a minor headgate leading to a private channel, while a sort of well is formed in the distributary flume just below this minor headgate to retard the velocity of the current. On the line of the Idaho canal the distributary heads are

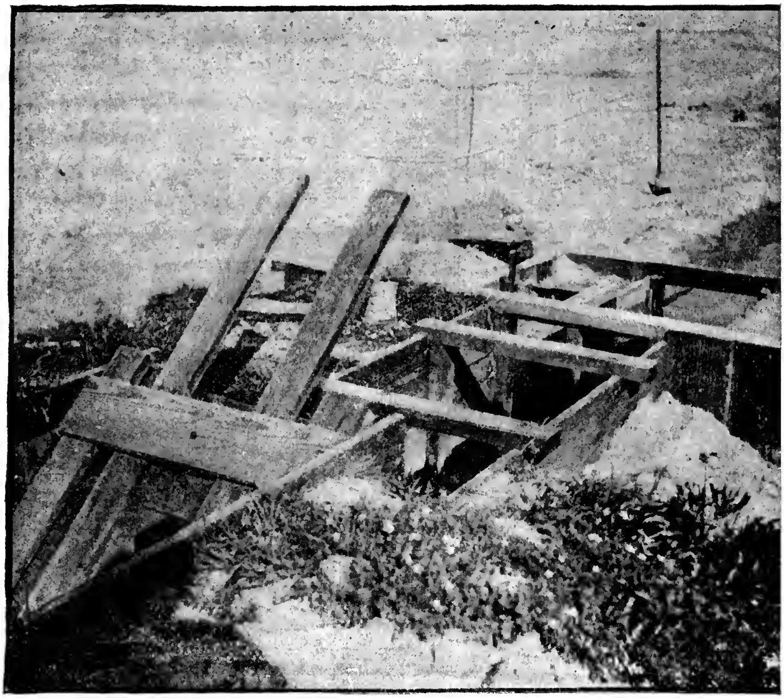


FIG. 54.—VIEW OF DISTRIBUTARY HEAD, CALLOWAY CANAL.

designed much as are the main heads on the same canal (Fig. 40).

On the Del Norte canal in Colorado a few of the distribu-

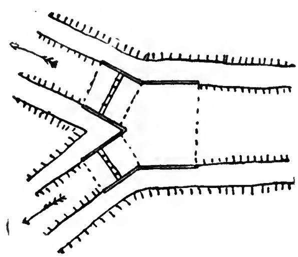


FIG. 55.—PLAN OF BIFURCATION, DEL NORTE CANAL.

taries are diverted by practically bifurcating the main branch, the latter thus terminating in two distributaries, in the heads of which are placed regulating gates (Fig. 55).

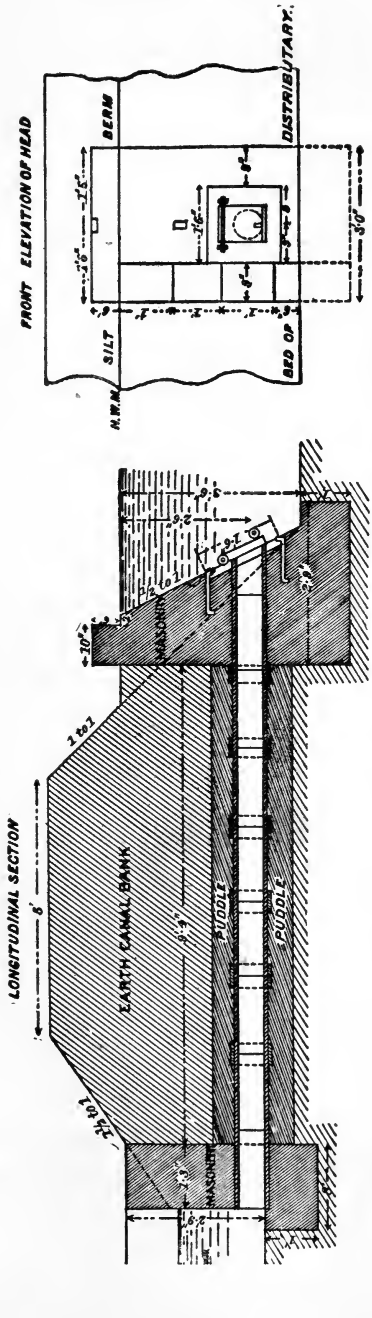


PLATE XIX.—STANDARD MASONRY OUTLET FOR DISTRIBUTARIES, PUNJAB, INDIA.

202. Masonry Distributary Heads.—In Europe and India masonry is employed almost exclusively in the construction of distributary heads. These are generally so built that the water passing from them can be measured and the volume turned into the private channels thus ascertained at any time. In Pl. XIX is shown the type of distributary head used on the canals of the Punjab. On the Mutha canals in Bombay a V-shaped weir is placed in the head of each private channel or lateral for the purpose of water measurement, while a water-cushion is built in the lower portion of the distributary head in order to diminish the shock of the falling water. The rules for the dimensions of water-wells or cushions are about the same as those given for main canals (Article 138). Distributaries are passed over or under each other or the country drainage in flumes or siphons as are main canals (Chapter XIV).

203. Iron and Steel Distributary Pipes.—Where water is conveyed in pipes instead of open channels, these are generally of iron, steel, wood, or occasionally of cement. The iron or steel pipes are constructed of sheet metal, the varieties being spiral riveted pipe, converse lock-joined kalamined lap-welded pipe, and straight double-riveted pipe. The dimensions of these distributary pipes range from 6 to 30 inches in diameter, and the thickness of the metal is trifling, varying between No. 8 and No. 10 plate. With straight riveted pipe the distance apart of rivets in the rows ranges from 1.33 to 1.40 inches, and the distance between any two rows is about $\frac{3}{4}$ of an inch. This wrought-iron or steel pipe is invariably coated with hot asphaltum by inserting the pipes in a tank of refined asphaltum fluxed with crude oil heated nearly to burning point. This class of pipe will bear pressures of from 100 to 200 pounds per square inch. In laying it air-valves are attached at all high places, and an air standpipe generally at the highest point, besides which blow-offs are placed at proper intervals.

204. Wooden Distributary Pipes.—There are several types of patented wood pipe. That which is now finding most favor is the invention of Mr. C. P. Allen of Denver and is known as the Colorado wooden pipe (Fig. 56). It is made of varying sizes from 20 to 36 inches in diameter, the walls of the pipe being formed of longitudinal staves braced together with iron or steel bands. These staves are shaped on the broad sides to cylindrical circles and the edges to true radial lines, so that when put together they form a perfectly cylindrical pipe. To join the ends of the staves, a thin metallic tongue is inserted which is a trifle longer than the width of the stave and cuts into the adjoining ones. The confining

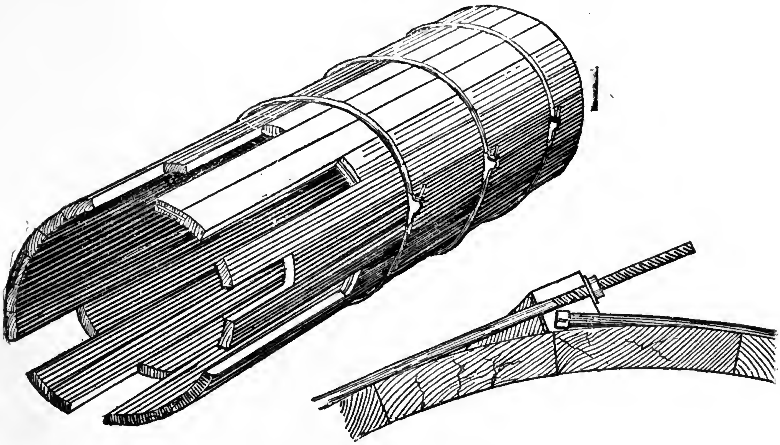


FIG. 56.—COLORADO WOODEN PIPE.

bands are of round or flat iron or steel of from $\frac{3}{8}$ to $\frac{3}{4}$ inches in diameter and are shipped from the factory as rods, provided at one end with a square head and at the other with a thread and nut. They are bent on the ground on a bending-table and coated with mineral paint or asphalt varnish, and are cut about 6 inches longer than the outside circumference of the pipe, on which they are slipped loose. These confining bands are placed at varying distances apart, according to the pressure which the pipe has to bear.

205. Rotation in Water Distribution.—The water in distributaries can be most economically handled if a system of rotation be employed in admitting it to the heads of the private channels. It is more convenient and economical to move water in as large volumes as possible. This may be done by regulating the amount admitted to the private channels and the periods of time in which they shall receive it. Thus the outlets to these channels may be closed in the first length of the canal for four days, in the second for three days, and so on; and then this order may be reversed, the period of rotation being such as to change the length of closure along the various portions of the canal. It is better to impose these systems of rotation on long portions of the distributary at once, as the effect in forcing the water down to the tail of the distributary is then more noticeable. Thus if a distributary be 20 miles in length and all the outlets in the first 5 miles be closed, those in the second 5 miles opened, those in the third 5 miles closed, and those in the fourth 5 miles opened at the same time, the effect will be to produce a stronger head and to carry the desired amount into all the channels in the last portion of the canal; then for a period of a few days this order may be reversed and without difficulty the maximum duty obtained from the water in the distributary. To make this system effective rules should be made compelling irrigators to accept water when their irrigation heads are open, and refusing it to them when their turn has gone by.

CHAPTER XVI.

APPLICATION OF WATER, AND PIPE IRRIGATION.

206. Methods of Applying Water.—The cultivator applies water to the crops by various methods, depending chiefly on the nature of the crop and the slope of the surface of the ground. These methods are:

1. By absorption from water sprinkled over the surface.

2. By filtration of a sheet of water downward through the surface of the soil.

3. By lateral percolation from an adjacent source of supply.

4. By absorption from a subsurface supply.

The first method includes irrigation by nature in the form of rain, or by sprinkling with a watering-pot or hose. This method is of such simple character as to require no further consideration here.

The second method of irrigation is called flooding, and is accomplished in three ways, depending on the character of the crop and on the slope of the soil:

1. Flooding of meadows by simply conducting a ditch along the upper slope of the land and allowing the water to flow from this completely over the meadow.

2. Flooding by checks, by dividing gently sloping surfaces into level benches by means of check levees and permitting the water to stand in these as in still ponds.

3. Flooding by the checkerboard system, by dividing nearly level ground into squares by surrounding levees and allowing the water to stand in these.

The third method of application is generally called the furrow method and is accomplished in four ways:

1. By running small ditches close to fruit-trees and vines, and allowing the percolation from these to moisten their roots.

2. By letting a large number of small streams flow from flumes through ditches between fruit-trees and vines, and allowing the water to percolate from these to their roots.

3. By flowing the water in small streams through the furrows between such crops as potatoes and corn, and thus gradually moistening them.

4. By drilling grain in rows or shallow furrows and running the water through these. This is practically a combination of flooding and sidewise soakage.

The fourth method of irrigation is conducted by laying pipes underground and having outlets in these under each fruit tree; or by so placing these outlets that the water escaping therefrom shall moisten the roots of vines and trees near by.

207. Sidehill Flooding of Meadows.—This method is the most wasteful of water, but it is that most commonly practised

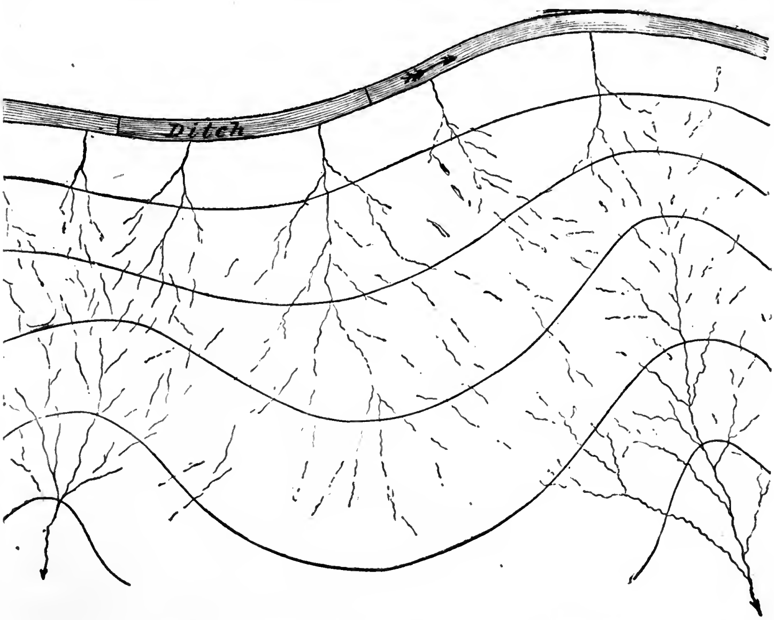


FIG. 57.—DIAGRAM ILLUSTRATING FLOODING OF MEADOWS.

in the cultivation of grass and cereals. Wild meadow lands and hayfields are flooded by simply turning the water on them when the slope of the ground is sufficient and allowing it to sink into the soil. To accomplish this the water is made to enter the field at its highest point in a ditch conducted around an upper contour of the field. Breaks are made at intervals in the side of the ditch, and the water being allowed to flow through these, finds its way in a thin sheet over the field (Fig. 57). This method is very expensive of water and can be employed on but few soils, since clayey soils bake or parch, forming a thin crust which kills the growth of plants. Instead of making breaks in the side of the ditches checks are sometimes formed by little dams of earth or wood.

208. Flooding by Checks.—This method consists in running check levees around the slope of the land on contour lines. These are low ridges of earth about 1 foot in height, turned up with a plough or scraper and placed at such distances apart that the crest of each shall be on a level with the base of the check above it (Fig. 58). If properly built these checks

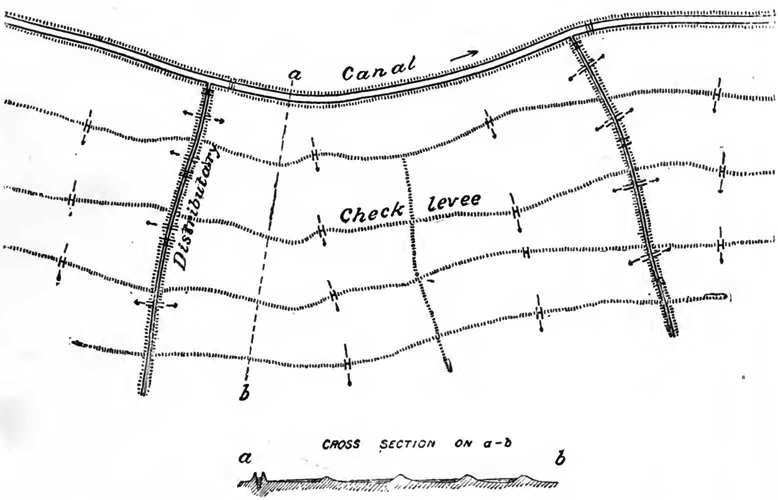


FIG. 58.—IRRIGATION BY SYSTEM OF CHECK-LEVEES.

will last for many years, and the field may be ploughed and re-ploughed without injury to them or their in any way affecting the handling of the crops. In comparatively level country like that in Kern county, California, the distributary ditches are placed as much as a quarter of a mile apart, their banks forming two of the bounding ridges or levees, the third or lower boundary being a contour levee connecting the ditch banks. The less the height of this levee the better, because the quantity of water spread over the land will be of more uniform depth and will interfere less with ploughing and harvesting; the greater the width of the levee base the better. From 6 to 12 inches is the best height and from 15 to 20 feet the best width of base. In such country as that described the checks range from 10 to 50 acres each in area and require from 12 to 20 miles of levee per square mile of check, while a mile of levee contains about 3000 cubic yards of earth. The water is run through the ditches (Fig. 58) and admitted by gates into each separate check. When the latter is full the water is drawn off to the next lower level, or if the soil is porous it is allowed to stand until it has been absorbed.

209. Flooding by Checkerboard System of Squares.—This method is practised extensively on the level plains of Southern Arizona and in India. The fields are divided into squares of from 20 to 60 feet on each side (Fig. 59), and these are separated by ridges or levees of from 10 to 12 inches in height from which openings are made leading from one square to the other. In some cases the fields are divided into much larger squares, often of an acre in extent, depending on the slope of the ground. Again, especially in India, very small squares are employed, and the height of the dividing ridges is made as low as 6 inches, so that these do not interfere materially with the harvesting and ploughing of the fields. The chief objection to this method is the obstruction created by the check levees. When these can be placed far enough apart they interfere but little with the operations of the cultivator: otherwise he must use spade and hoe instead of plough.

Water is admitted to one square at a time and is either permitted to soak into the soil or is drawn off to be used in the next square below, much as in the check method. The chief crops irrigated by this method are hay, grain, and vegetables. Where flooding is practised by checks or squares, anywhere from 4 to 12 inches in depth of water is let on at a single watering. The number of these waterings may range between two and five in a season, according to the crop, soil, and climate. Rice and sugar cane are irrigated in India and

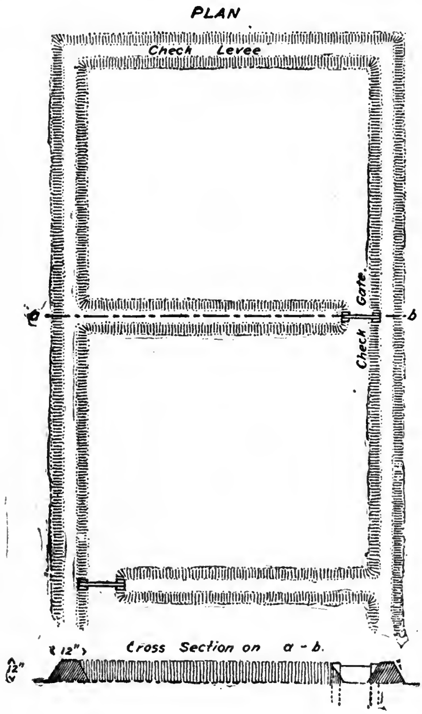


FIG. 59.—FLOODING BY SYSTEM OF SQUARES.

South America by squares. These crops require a very large amount of water, and as a consequence the height of the levees is rarely less than a foot and is often greater. These are filled with water and it is allowed to stand on them for long periods of time, the soil being seldom permitted to dry.

210. Flooding by Terraces.—This method is employed chiefly in India and China, and has recently been adopted on a small scale in the neighborhood of Newcastle, California. It consists of laying out steeply sloping sidehill ground in terraces, the lower sides of which are surrounded by high levees. These are practically exaggerated forms of checks, and as employed in California are maintained and operated on the same general principle, though they receive a large proportion of their water supply from the drainage of the hillsides above. As employed in India or China, these terraces also receive their water supply chiefly from the drainage above, and hold it as in a small tank or reservoir of a few feet in depth. As the water soaks into the soil of the terrace, rice or similar crops are sown, and the amount of moisture retained in the earth by such a volume of water entering it is sufficient, with the addition of what may be received from occasional rains, to irrigate the crops.

211. Furrow Irrigation of Vegetables and Grain.—This method is practised by laying the field off in shallow ditches run around its upper slope. From these ordinary plough or V-shaped furrows radiate down the slope of the field, and between

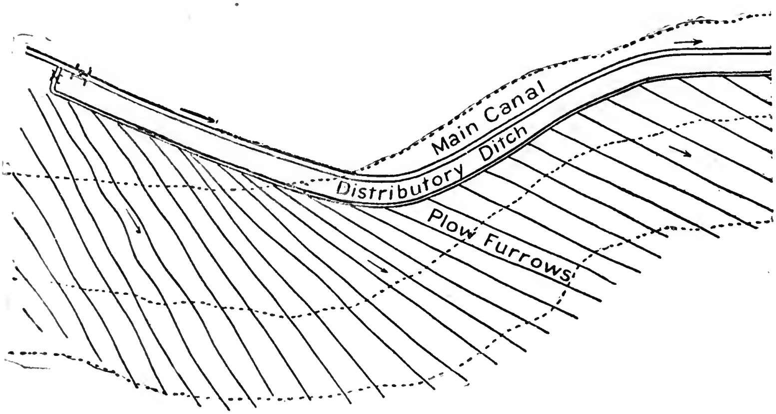


FIG. 60.—FURROW IRRIGATION OF GRAIN.

these the vegetables, potatoes, or grain are planted. Where the country slopes more irregularly or steeply the furrows are run at various angles down the slope in such manner that their grade shall not be too steep. The water is then turned into a few of these furrows at a time by blocking the ditch above with a clod of dirt or a board (Fig. 60), and the water penetrates by sidewise soakage to the crops. Corn is irrigated by the furrow method by ploughing a ditch along the upper slope of the field as above described, and by drilling the grain down the slope of the field radially from this ditch and permitting the water to enter a few of the drill rows at a time. Grain fields are sometimes prepared for this method of combined flooding and furrow irrigation by rolling the field after the grain is planted with a heavy roller on the surface of which are angular projections of from $\frac{1}{2}$ to 1 foot apart and a few inches in height. These make grooves in the surface of the soil in a direction parallel to the slope, and the water is admitted to these and permitted to flow through them as in the case of ploughed furrows or drill rows.

212. Combined Flooding and Furrow Irrigation of Orchards.—Where orchards are directly flooded the tendency of the water is to bring the roots to the surface and thus enfeeble them. To prevent this furrows are run from the upper

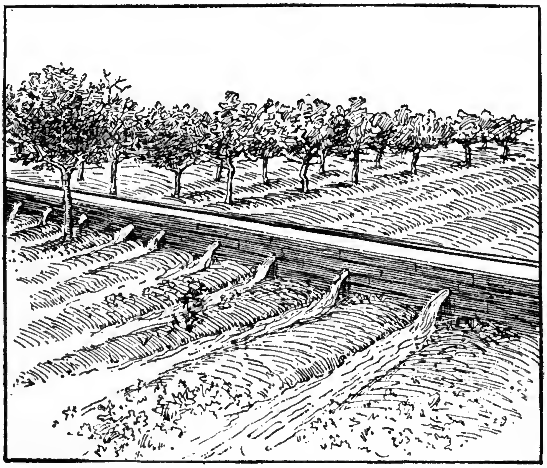


FIG. 61.—FURROW IRRIGATION OF ORCHARDS.

ditches, generally in a double row, one on either side of and at a short distance from the trees or vines (Fig. 61). By this means the water percolates into the soil and reaches the roots of the tree by sidewise soakage at some depth beneath the surface, thus moistening and encouraging their growth. Another method of flooding orchards is to protect the trees by earth ridges thrown up so as to prevent the water from reaching within 3 to 4 feet of them. In this method the entire field is flooded with the exception only of the areas immediately adjacent to the trees. This practice is wasteful of water, as much more is employed than is required. Olive and orange trees are watered from three to four times in a season, vines once or twice and often not at all after the first few seasons.

213. Irrigating Orchards by Small Furrows. — This method is practised as yet chiefly in the neighborhood of San Bernardino valley, California. The principle underlying this method is that the ground shall be put in the condition which it would be in after several days of long soaking rain, rather than in the condition which it would be in after a small cloud-burst, which is the condition resulting from most other methods of surface irrigation. This is done not by running large streams of water through the furrows for a short period of time, but by running small streams through them for a long time. It is accomplished (Fig. 61) by running a number of ploughed furrows between the rows of trees, the nearest furrow not being closer than 3 feet from the trees, and the distance between furrows from 2 to 3 feet. The volume of each of the streams running through these does not exceed one four-hundredth of a second-foot, and the water is run through them for two and three days at a time. Where the soil is not too loose or sandy this method seems to give the best results for fruits and vines and may be used with some success on grain and corn.

In order that the method shall be successful, the laterals from which the furrows are filled and which come from the main distributary must have a uniform depth and slope to a degree which cannot be secured in open earth. This is accomplished by running wooden laterals or flumes along the surface of the ground down its slope. These simple flumes are but a few inches in cross-sectional area, generally the width of a plank at base and on the sides. They are given a sufficient grade to produce a good velocity and where the natural slope is too great falls are introduced. The water escapes from these flumes into the furrows through auger-holes bored in their sides opposite each furrow and on a level with the bottom of the flume (Fig. 61). The flow through these holes is regulated by wooden buttons or plugs which are inserted in them. For small orchards, these flumes generally have a capacity of about $\frac{1}{2}$ a second-foot. Fruit trees thrive well on from three to five waterings and vines on from two to three waterings when supplied by this method.

214. Subsurface Irrigation.—Irrigation from beneath the surface, or sub-irrigation, is the most perfect method of supplying water to plants. The idea is to replace soakage from above by means of flooding or furrows, by absorption from below, which, to be perfect, should not wet the surface. This is effected by laying pipes underground, and these derive their supply from distributaries which are usually sheet-iron or steel pipes. While the cost of preparing land for this method of irrigation is relatively great, it is more than repaid by the saving in water charges, since the duty of water is great, reaching from 500 to 1000 acres per second-foot. This method has been most extensively employed among the valuable fruit lands of Southern California, and where these lands are divided into and sold in orchard lots of from 10 to 20 acres in area, the distributing pipes are carried to the highest point in each one of these lots, and from this the sub-irrigation pipes are conducted through the orchards.

215. Sub-irrigation Pipes.—These are made of sheet-iron or steel or of some porous or glazed material, the former being usually a combination of cement, lime, sand, and gravel, with a small admixture of potash and linseed oil, and are known as *asbestine* pipes. Glazed earthenware pipes are becoming more popular than any other form. Asphalt-concrete pipes have been successfully employed for sub-irrigation and have the advantage over simple concrete pipes of being impervious to water. These are united by heating so as to form a continuous pipe. These distributing pipes are usually made in various dimensions, according to the circumstances under which they are to be used and the area which each is to control. In some cases they are as small as 2 inches in diameter, and from this they range to 6 inches where the principal distributaries are reached.

216. Method of Laying Pipes.—Sub-irrigation pipes are laid in open trenches at a depth of 1 to $1\frac{1}{2}$ feet below the surface, parallel to the rows of trees or vines in the orchard, and the trench is then filled in with earth. A method has been attempted of laying the pipes by means of machinery, though as yet this has not met with success. Irrigation is effected from these pipes sometimes by cutting a hole on the upper side and inserting therein a wooden plug opposite each tree or vine. Each plug is surrounded by a larger standpipe set loosely on top of the distributary pipe, open at the bottom and reaching to the surface of the ground for the purpose of keeping the dirt away from the outlet and rendering it accessible at all times for inspection.

The process of irrigation consists in simply turning the water off or on from the main pipe, when it finds its way through the outlets, fills the standpipe, and slowly percolates to the surface of the ground. One of the great objections to the use of pipes for sub-irrigation is the necessity for having these small holes or openings from which water can escape, and the resultant danger to the pipe of roots growing into the openings and clogging or destroying them. If muddy water is let into the pipe there is danger of clogging unless sufficient pressure can be used to flush them. One of the most satisfactory methods of letting the water escape consists in cutting a section several inches in length out of the continuous pipe where the plug-hole should be inserted, and by replacing it by a U-shaped shoe placed below the cut in the pipe. A tile a little longer than the gap covers it and water escapes between the two surfaces. By this method of irrigation plants do not receive the fertilizing elements brought to them by the sediment carried in surface waters. On the other hand, the pipes have the advantage of acting as drains to carry off surplus water and thus prevent the rise of alkali and other evils attending supersaturation, especially as the water, when properly handled, does not reach the surface and evaporate there.

217. Measuring Sub-irrigation Waters.—In the Alessandro district in California a water-measuring apparatus is employed which consists of a 4-inch iron standpipe resting on the 6-inch vitrified service-pipe (Fig. 62). At the top of the stand-

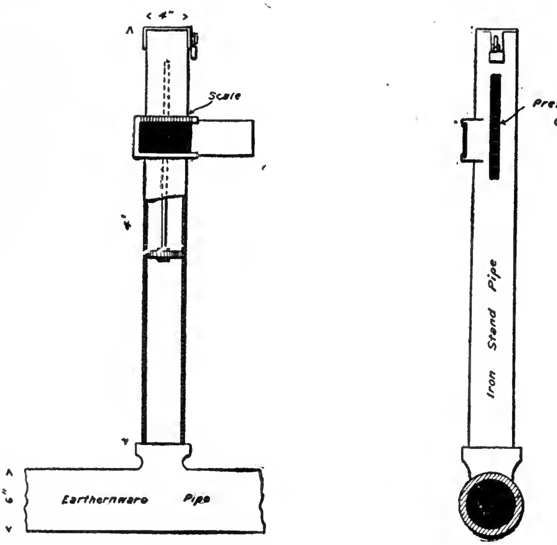


FIG. 62.—ALESSANDRO HYDRANT.

pipe a scale is so arranged that the amount of water flowing through can be measured by simply reading it. A valve inside the standpipe, which can be locked by a simple device, is operated by a screw attachment and admits the proper amount of water. On the outer surface of the standpipe is a pressure-gauge which shows the head of water on the measuring-slot. The unit of measure used on these pipes is the miner's inch. This device has met with some favor, but is open to the same objection as all similar water meters, namely, that it is expensive and troublesome, requiring much attention for its proper management.

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PART III.

STORAGE RESERVOIRS.

CHAPTER XVII.

LOCATION AND CAPACITY OF RESERVOIRS.

219. Classes of Storage Works.—A storage work is any variety of natural or artificial impounding reservoir or tank for the saving of superfluous or flood waters. Storage works are employed to insure a constant supply of water during each and every season regardless of the amount of rainfall. They may be classified according to the character and location of the storage basin, or the design and construction of the retaining wall or dam which closes it. Under the former classification are:

1. Natural lake basins;
2. Reservoir sites on natural drainage lines, as a valley or canyon through which a stream flows;
3. Reservoir sites in depressions on bench lands;
4. Reservoir sites which are in part or wholly constructed by artificial methods.

Under the second classification are:

1. Earth dams or embankments;
2. Combined earth and loose-rock dams;
3. Hydraulic-mining type of dam, or dams constructed of loose rock or loose rock and timber;
4. Combined loose-rock and masonry dams;
5. Masonry dams.

220. Relation of Reservoir Site to Land and Water Supply.—There are several modifying considerations affecting the value of the reservoir site. Among the more important of these are:

1. The relation of the site to the irrigable lands;

2. The relation of the site to its catchment basin or source of supply;

3. The topography of the site;

4. The geology of the site.

The cost of water storage depends chiefly on the last two, while the value of the site for storing water and the possibility of filling the reservoir depends on the first two.

In considering the relation of the reservoir site to the irrigable lands, the former should be situated at a sufficient altitude above the latter to allow of the delivery of water to them by natural flow. The area of these lands should be sufficient to require the entire amount of water stored, that the maximum return may be derived from water rates, and the reservoir should be as near as possible to the irrigable lands in order that the loss in transportation shall be a minimum. It not infrequently happens, however, that the reservoir is of necessity located at some distance from the irrigable lands, thus requiring either a long supply canal or that the water be turned back into the natural drainage channel, down which it will flow till diverted in the neighborhood of the irrigable lands. This is very wasteful of water, since the losses by absorption, percolation, and evaporation are great, especially if the bed of a natural channel is used as a portion of the supply line.

As related to the source of supply, the reservoir site may be on a perennial stream the discharge of which is more than sufficient to fill it, in which case the supply is assured. It may be on a stream the available perennial discharge of which is sufficient to fill it in times of flood. It may be on an intermittent stream subject to occasional flood discharges of sufficient volume to fill the reservoir so as to enable it to tide over a couple of seasons of moderate supply. Or the reservoir site may be situated above and away from any natural drainage line, in which case it will receive its supply either by a canal diverted from some perennial stream or from artesian wells or springs.

221. Character of Reservoir Site.—If situated in a natural lake basin, a short drainage cut or a comparatively cheap dam or both may give a large available storage capacity. Such sites are usually the best and cheapest, costing for construction as low as 20 cents per acre-foot stored, and in unfavorable cases rarely exceeding $3 per acre-foot. The most abundant reservoir sites are those on natural drainage lines, though these are usually the most expensive of construction owing to the precautions which it is necessary to take in building the dam to provide for the discharge of flood water. Almost equally abundant are those reservoir sites found in alkaline basins or depressions on bench or plain lands, especially on the plains sloping to the eastward of the main Rocky mountains and in the foothills of the Sierras in California. The utilization of such basins as reservoir sites is comparatively inexpensive; they can be converted into reservoirs by the construction of a deep drainage cut or of a comparatively cheap earth embankment. Scarcely any provision is necessary for the passage of floods. The heaviest item of expense in connection with these sites is the supply canal for filling them from some adjacent source.

Artificial reservoirs are occasionally constructed where water is valuable, by the erection of an earth embankment above the general surface of the country or by the excavation of a reservoir basin by artificial means. Such constructions are usually insignificant in dimensions, as the expense of building large reservoirs of this kind would ordinarily be prohibitive. Shallow reservoirs should not be constructed. The loss from evaporation and percolation is proportionately great, and the growth of weeds is encouraged where the depth is less than seven feet, by the sunlight penetrating to the bottom.

222. Topography and Survey of Reservoir Sites.—Knowing the position of the irrigable lands, a careful preliminary survey should be made of the entire neighborhood to discover all possible reservoir sites, and the outlines of the catchment basins of each of these should be mapped, while stream gauging should be conducted and examinations and inquiries made to ascertain the minimum discharge of the streams and their flood heights, as well as the amount of evaporation and percolation (Chapters III and IV). Having determined in a general way upon the location of the reservoir site, a detailed survey of it should be made. This can ordinarily be best done by means of a plane table. The highest possible point to which the dam may reach may be taken as a basis and a top contour run out closing around the entire site. In addition to this a main traverse should be run through the central or lowest line of the site from the dam to the extreme end where it will connect with the top contour. Cross-section lines may be run from this with the plane table, and the topography of the site sketched in 5-foot contours and plotted to some large scale, preferably 500 to 1000 feet to the inch. Where the country is open and unobstructed by timber the site may be triangulated from one side, as a check on the cross-section lines, and where the slopes are even these may be best determined by means of gradienter lines run up and down them from a base contour. Such a map will enable the engineer to determine the capacity of the reservoir for various depths of water.

The dam site should be surveyed in greater detail, several possible sites being cross-sectioned and mapped in 1-foot contours and at a scale of perhaps 100 feet to the inch. This work should be done with transit and chain, whereas in the reservoir survey the stadia may be satisfactorily employed on most of the cross-section lines. With such a knowledge of the topography of a catchment basin and of the reservoir and dam sites as the resulting map will give, the engineer may readily compute the cost of construction of dams for various heights as well as the contents of the reservoir for these heights, and thus determine what height of dam will be most economic of construction, for there is always some height which will render the cost of storage a minimum.

223. Geology of Reservoir Sites.—Having ascertained the desirability of the reservoir site topographically and hydrographically, a few test borings or trial pits should be sunk at various points on the reservoir basin, and especially at the dam site, to ascertain the character of the soil and the dip of the strata underlying the proposed reservoir. The geological conformation may be such as to contribute to the efficiency of the reservoir, or it may prove so unfavorable as to be irremediable by engineering skill. A reservoir site which is situated in a synclinal valley as shown in *A*, Fig. 63, is the most favorable.

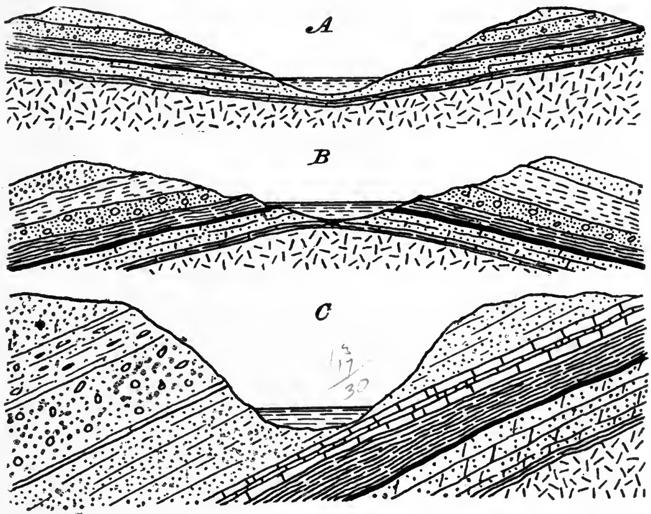

FIG. 63.—DIAGRAMS ILLUSTRATING GEOLOGY OF RESERVOIR SITE.

In this the strata incline from the hills towards the lower lines of the valley, and any water which may fall on to these hills will find its way by percolation through the strata into the reservoir, thus adding to its volume. An anticlinal valley is the least favorable for a reservoir site (Fig. 63, *B*). In such a valley as this the strata dip away from the reservoir site and would permit of the escape of much of the impounded water, percolation through the strata leading it off to adjoining valleys. A class of geological formation intermediate between these two is that represented in *C*, Fig. 63, in which the valley has been eroded in the side of strata which dip in one direction. Here the upper strata lead water from the adjoining

TABLE XI.

COST AND DIMENSIONS OF SOME STORAGE RESERVOIRS.

Name of Reservoir.	Locality.	Material of Dam.	Capacity, Acre-feet.	Maximum Height of Dam, Feet.	Length on Top, Feet.	Cost per Acre-foot stored.
Sweetwater	California	Masonry	18,000	94	380	40.90
Bear Valley	"	"	40,550	64	300	5.30
Hemet Valley	"	"	138,000	150	250	9.98
Periar	India	"	160,000	155	1,230	4.65
Bhatgur	"	"	126,500	127	4,067	3.20
Betwa	"	"	36,800	61.5	4,300	8.90
Alicante	Spain	"	3,300	134.5	190	.....
Beetaloo	South Australia	"	18,400	110	580	31.84
Villar	Spain	"	15,500	170	546	25.20
Gran Cheurfas	Algiers	"	14,800	98.4	508.4	.....
Cuyamaca	California	Earth	11,500	40	635	9.00
Long Valley	"	"	32,910	96	950	2.21
Merced	"	"	15,000	54	4,000	26.60
Ashti	India	"	32,600	58	12,907	4.80
Ekruk	"	"	76,100	72	7,200	4.00
Castlewood	Colorado	Loose rock and masonry.	5,300	63.6	586	38.00
Walnut Grove	Arizona	Loose rock.	7,000	110	420	16.10
Bowman	California	Loose rock and timber	......	100	425	11.18

hills into the reservoir, while the strata on the lower side tend to carry it off from the reservoir by percolation. In such a case it is probable that the reservoir would neither gain nor lose.

If the surface proposed reservoir site is composed of a deep bed of coarse gravel or sand or even limestone, crevices in the latter or between the interstices of the former will tend greatly to diminish the capacity of the reservoir by seepage from it. Again, the geologic formation may be most unfavorable, yet if the surface of the reservoir site be covered with a deep deposit of alluvial sediment or of clay or dirty gravel or other equally impervious material, little danger may be apprehended from loss by seepage.

224. Cost and Dimensions of some Great Storage Reservoirs.—In Table XI are given the capacities, material, dimensions of dam, and cost per acre-foot stored of some of the great storage reservoirs which are used for purposes of irrigation.

CHAPTER XVIII.

EARTH AND LOOSE-ROCK DAMS.

225. Earth Dams or Embankments.—The choice of the material of which the dam shall be constructed, whether it shall be of earth, masonry, or loose rock, is dependent largely upon the character of the foundation and the cost of transportation. Earth dams when well constructed are fully as substantial as those of masonry, and in many cases they are far more so. In countries subject to earthquakes, or where the rock foundation is not thoroughly homogeneous, an earth dam is decidedly preferable to one of masonry. They are usually cheaper, and where transportation is expensive they are very much cheaper. Providing a substantial and abundant wasteway of a sufficient capacity to carry the greatest possible flood be provided, an earth dam is generally to be preferred in mild, damp climates. In warm, dry climates they are liable to dry and crack. For reservoirs over 100 feet in depth masonry dams are to be preferred, as earth dams are nearly as expensive when transportation is cheap, and are more liable to be badly built.

As before stated, the choice between the two depends largely on the foundation. A substantial masonry dam cannot be founded on loose gravel or soil; an earth dam should rarely be founded on rock, owing to the difficulty of making a tight joint between it and the earth. There are three general types of earth dams:

1. Earth dams having a central core or wall of puddled earth;
2. Earth dams having a central core of masonry or wood;
3. Earth dams built up in layers of homogeneous material, without central core or puddle facing.

226. Dimensions of Earth Dams.—An earth dam may be supposed to fail in two ways, either by yielding to the horizontal pressure of the water overturning it, or by sliding on its base. The simplest form of calculation clearly demonstrates what is fully acknowledged by all engineers, namely, that the dam will not be destroyed by overturning or revolving about its lower toe; hence the only theory as to its destruction is that it may slide on its base. The conditions of stability will be satisfactory when the horizontal component of the water pressure against the bank equals the weight of the latter plus the vertical pressure exercised by the water to hold it down, and multiplied by the coefficient of friction. Such a case is rarely or never apt to occur. In point of fact such structures usually fail, not by overturning or sliding on their bases, but by the disintegration of their particles due to the percolation of water.

When subjected to the contact of water earth loses a certain amount of its stability, and therefore it is customary to give the inner slope of an embankment a greater inclination than the outer slope. These slopes depend on the character of the material. When the outer slope will stand with an inclination of 1 on $2\frac{1}{2}$ the inner slope should be 1 on 3.

The interior and exterior slopes of earth dams may be considered as planes forming together an angle of not less than 90 degrees, and the figure should be so formed in order to increase its stability, that lines of pressure passing from the interior faces at right angles may fall within its base. As one cubic foot of rammed earth weighs about 100 pounds and a cubic foot of water $62\frac{1}{2}$ pounds, we find the base of a prism resisting the lateral thrust of the water does not require to be more than two thirds of the depth of the column it supports. Hence all quantities above that are due to the natural slopes, the stability of the dam, and the prevention of percolation.

In large works it is frequently a matter of close calculation to determine which will be the more economical,—dams exclusively of earth or those whose inner slopes are supported by retaining walls of masonry. The outer slope of the dam may vary between 1 on $1\frac{1}{2}$ and 1 on $3\frac{1}{2}$, according to the character of the material. Light sand requires the flattest slope. A firm mixture of gravel and clay will stand a slope of about 1 on $1\frac{1}{2}$. The inner slope of the dam should be about $\frac{1}{2}$ on 1 greater than the outer slope. It is not unusual, as in the case of the Ashti dam (Pl. XX), to make the inner slope near the top a little steeper than the lower portion of the slope, the object being that a steep slope from 1 on 1 to $1\frac{1}{2}$ reflects the waves, while a flatter slope breaks them up.

The top width of the dam depends somewhat on circumstances. A top width of 6 feet is perhaps the minimum which should be employed, and for a high dam this is usually too small. A good rule as to the minimum top width of earthen dams 50 feet in height and over is to make their breadth 10 feet. For dams under 50 feet the top width should be 8 to 6 feet. As the dam settles in course of time, its top should be built up by adding material to the required height. The dam should always be several feet higher than the highest flood mark in order to prevent waves from topping it. Thus the height of the dam above the crest of the discharge weir should be

$$H = D + X + C;$$

in which D equals the depth of water in the reservoir above the weir crest at maximum flood;

X equals the height of the top of the stone pitching above the surface of the maximum flood;

C is a constant equal to 2 or 3 feet according to circumstances, and is equal to the vertical height of the top of the dam above the top of the pitching.

227. Foundations.—The foundation of an earth dam should be examined with great care. The best material on which to found it is sandy or gravelly clay, fine sand or loam. Such a structure should never be built on shale or slate or on firm rock. Great care should be taken in searching for springs or quicksands in the foundation. Sometimes a quicksand may be discovered at some little depth beneath a hardpan or other suitable foundation. In such a case it is sometimes possible to seal over the quicksand under the embankment, and found the latter on the upper stratum. Such an expedient is not entirely free from risk, and great care should be taken in joining the toe of the embankment to the foundation material, if necessary spreading earth and clay over the surface of the valley for some distance on either side of the dam.

The first thing to be done in preparing the site of the dam is to clean off all soil, removing it to a depth equal to that penetrated by the roots of the grasses, bushes, and trees. If firm and impervious, the soil may be scored by longitudinal trenches, which will give the proper adhesion between the foundation and the embankment, and prevent the slipping of the latter. If a puddle wall or masonry core is to be built into the dam, the foundation for this should be sunk to a sufficient depth to secure its permanence. If a homogeneous dam is to be built and the foundation material exposed is not imprevious, a trench should be dug, and this filled with some puddle material, as clayey gravel or gravelly loam, moistened and rolled or packed in layers.

228. Foundations of Masonry Core and Puddle Wall.—No rule can be laid down for the depth to which the foundations for the centre wall, if one is built, should be carried. If a rock foundation is encountered the problem is simplified, as the wall may be founded on this after removing the loose and disintegrated surface; if the test pits or borings reveal only the existence of coarse or permeable strata, the masonry core must be carried well down. In some cases it has to be carried to great depths, though when this is the case a foundation consisting of a puddle wall would appear to be preferable. The finer the material, the better it is adapted for a foundation for the centre wall. Fine gravel and sand and clay, or even quicksand when at a sufficient depth to prevent its being forced up, form good foundations.

Where a puddle wall is employed instead of a masonry core or heart-wall, the same general precautions with regard to its foundation are necessary as for the foundation of the latter. Every care should be taken to insure it a firm seat on some impervious stratum.

229. Springs in Foundations.—It is a very common occurrence to encounter springs in the excavations for the foundations of dams either of masonry or of earth. These springs are a great menace to the integrity of the structure, and it is due to their presence that some of the most disastrous failures of dams have occurred. Some engineers recommend that springs be taken up and carried away in proper drains securely puddled. This, however, is a very difficult operation and one rarely possible of accomplishment. When a single large spring is discovered this mode of treatment may be easily resorted to by following it back in a cutting until it can be taken up in a pipe. But ordinarily the foundation is underlain by a number of small springs, since water appears in such cases to rise from all over the surface of the stripped foundation. The best method of dealing with such cases is to strip the foundation of the inner embankment down to good firm earth, and then commence placing that part of the embankment which is next to the centre wall and advance it outward toward the toe of the slope with a view of smothering down the springs. Large springs frequently give trouble in closing the gap in the foundation of the centre wall. They may sometimes be carried up with the wall until a point is reached above which they do not rise, or they may be handled by reducing the width of the gap left in the wall until it becomes too narrow for the passage of the water. There are several methods of treatment, but the rule is to get the wall built up so that the water does not wash out the mortar and run through it.

230. Masonry Cores, Puddle Walls, and Homogeneous Embankments.—There is still a wide difference among engineers as to the best type of earth dam. Occasionally in England and in a few cases in our own country earth dams have been built up homogeneously, the front or water face being covered with a deep layer of some puddle material, as clayey loam. This practice, however, is falling into disuse, and engineers now rarely trust to a puddle face alone for protection against leakage.

A wooden or plank core should never be employed. The material is sure to rot and decay, while the smooth surface of the boards offers a most excellent line along which leakage water will travel until it finds an outlet. Again, it is impossible to make a wooden wall sufficiently substantial and heavy to withstand leakage and the tendency to rupture which may result from the settling of the bank.

The masonry core is in great favor with many engineers, both in Europe, India, and America. A central core of puddled earth is subject to rupture from the settlement of the embankment. Both are practically impervious to leakage. In building them they must be carried sufficiently deep to reach some impervious stratum, and far enough into the side walls of the valley to prevent the passage around their ends of seepage water which would travel along their impervious faces. The construction of a dam composed for a portion of its length of earth and for the remainder of loose rock or masonry is dangerous, and the writer is opposed to such combinations. Moreover, masonry, either as a retaining wall, core, or culvert, is rigid, while the other material is flexible, and any settlement in the latter leads to rupture in the former. Furthermore, masonry offers a smooth surface for the travel of seepage water.

The earth dam with masonry core is probably the most popular at present, but engineers to a limited extent in India and to a large extent in our western irrigation region are coming to favor the earth dam built up in homogeneous layers, each carefully rolled or tramped over in such a manner that the whole dam is a puddle wall. This character of construction has all the advantages of imperviousness to leakage if the work is well done, while it is free from the disadvantages possessed by dams with central cores, namely, a smooth surface along which water may travel, and liability to rupture in the wall. To be sure, the liability to rupture is very trifling, and is a matter of sentiment and theory rather than fact, as probably no case is on record of such an accident occurring in a well-built dam. Still, a homogeneous earth dam (Art. 235) is one of the simplest and cheapest to construct, and may be so built up as to be practically indestructible. With such a form of dam a puddle trench is usually excavated in the centre of the foundation and filled with puddle material to prevent leakage under and around the dam, and the material as laid down may be so selected as to get the finest and least pervious constituents in the front portion of the dam, leaving the heavier and coarser material to the rear to give stability. Such a form of construction practically converts the dam into one having a puddle face of great thickness.

231. Masonry Cores.—The primary object of a masonry centre wall is to afford a water-tight cut-off to any water of percolation which may reach it through the bank. Where the masonry wall is employed, it is the dam proper, for it is this which retains the water in the reservoir, the earth embankment surrounding it on either side being only of service in keeping the centre wall from being thrown down. One of the great advantages of the masonry core is that it affords an excellent opportunity for making the connections with the outlet tower and the culverts for the discharge sluices. These masonry culverts running through the centre of an earth dam constitute one of the weakest points in its construction, and offer the greatest opportunity for the passage of seepage water. They can be so bonded with the masonry core as to form a part of it, and preclude the possibility of the water following along the culverts.

The masonry core should be carried to a height equal to that of the sill of the escape-way, while in very high dams it is well to raise it to the extreme flood height. It should be as thin as possible in order to reduce its cost, yet as some movement may take place in the embankment owing to settlement, it should be sufficiently heavy to be self-supporting. A safe and usual rule is to give it a top width of 4 to 5 feet, and to increase its thickness toward the bottom at the rate of about 1 foot in 10. Sometimes this thickness is increased beyond the amount here stated. This centre wall should be composed of the best hydraulic masonry, preferably of concrete composed of sharp broken stone mixed with clean sand and Portland cement. Concrete, however, is not essential: rubble in cement is equally good, and ordinarily quite as convenient and satisfactory. When such material is used, however, stones of moderate size should be employed which shall not run through the wall from side to side, and for purposes of economy the rubble should be uncoursed, though very compactly and carefully laid.

An excellent example of a masonry core or centre wall for an earth dam is that in the New Croton dam at Cornell's (Fig. 81). This masonry core is 18 feet thick at the base, where it is founded on rock, and retains the same dimensions for a height of 89 feet, above which it tapers to 6 feet in thickness at the top, which is 20 feet below the top of the embankment.

232. Puddle Walls and Faces.—The puddle wall is not considered as satisfactory nor as efficient as the masonry wall, though it is much cheaper of construction in some portions of the West, where transportation is expensive. The proper material for a puddle is not always obtainable, while water for moistening it is frequently difficult to obtain in the arid region. It is difficult to prepare, and requires careful manipulation in placing it. A good puddle should, when placed, resemble in character and composition an unburnt brick. Where too much responsibility is rested in the imperviousness and security of the puddle wall it is frequently a menace to the structure, as it is rarely built with sufficient care. A puddle wall should have a thickness of 8 or 10 feet at the level of the water line, and should increase in thickness downward to the surface of the ground at the rate of about 1 foot in 10. Where a puddle wall is employed, the material of which it is constructed is usually clay, or gravel and clay moistened and puddled in layers of about 6 inches in thickness, and permitted to dry slowly. On either side of it selected material is usually placed, the remainder of the dam downward consisting of the poorer and most available material.

As before stated, a puddle face is rarely employed. Where it has been used it consists generally of a covering on the whole inner face of a layer of puddle 8 or 10 feet in thickness at the base and 2 or 3 feet in thickness near the summit, and on the whole is placed a layer of common soil on which the riprap is laid. In a few instances the puddle face has been mixed with small stones or furnace cinders as an obstruction to the passage of moles, gopher, or other vermin.

233. **Puddle Trench.**—This is employed only where the dam is built up in homogeneous layers without a central wall. It consists of a trench excavated longitudinally the entire length of the dam down to some impervious stratum, or if none can be found, for a very considerable depth. This trench is then filled either with puddle material built up the same as is a puddle wall or with a wall of masonry built up as a core wall, and the material filling this trench is carried up several feet above the surface of the ground. The trench should be carried up the slopes of the surrounding hills till it terminates at a level with the top of the embankment, and its bottom should be level in all directions, all changes of level being made by means of vertical steps. The same rule applies to the foundation of a puddle wall or masonry core.

One of the most excellent examples of a puddle trench is that illustrated in Plate XX, and employed in the Ashti dam in India. This trench was carried down to a hard bed of trap-rock, and in some places to consolidated clay. In this a puddle was laid in layers 4 inches thick which were reduced to 3 inches by watering and rolling. This puddle trench is rectangular in cross-section, 10 feet in width throughout, and generally 16 feet in height to the summit of the material filling it. The crest of the material filling the puddle trench was raised to a height of 1 foot above the surface of the ground so as to form a water-tight junction with the earthwork of the dam. Across the bed of the river along the centre line of the dam the trench was made but 5 feet in width, and was carried down to bed-rock and extended 100 feet into the banks of the river on either side, and was filled with a wall of concrete.

234. **Construction of Embankment.**—As ordinarily built the earth embankment changes outward from the central core, as before described, to a body of selected material on each side of it, the remainder of the dam being constructed of the most available common material. The result is a dam composed of 5 layers, each of different density and weight and

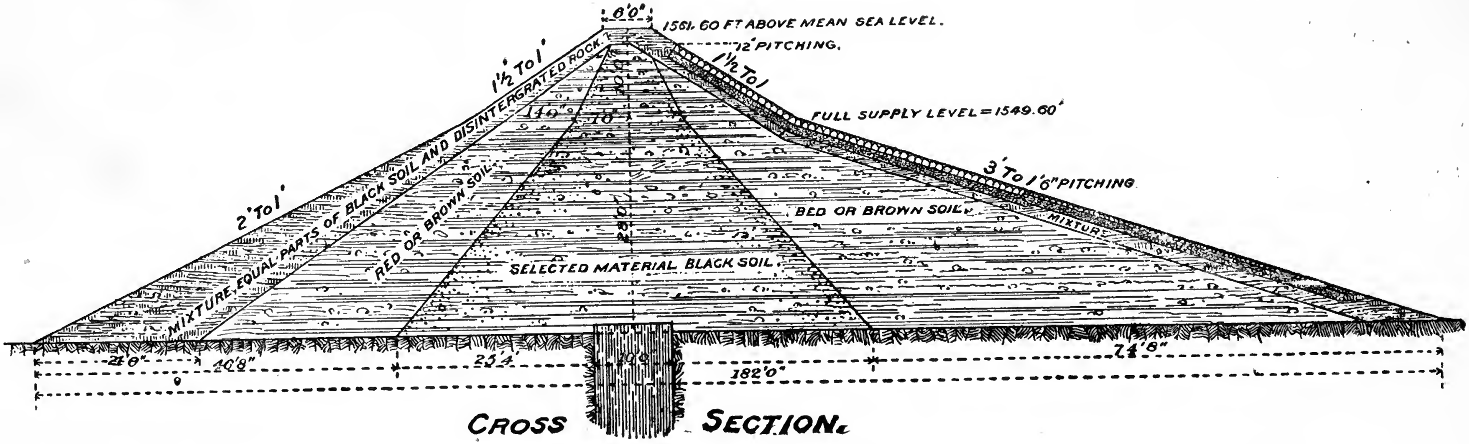

PLATE XX.—CROSS-SECTION OF ASHTI DAM, INDIA.

each likely to settle in different amount. This material is carried up generally in layers of a foot or so in thickness, and the result is a structure not homogeneous in character and with a series of horizontal surfaces with cleavage and vertical lines on which settlement and shrinkage may occur. The material, when laid in the embankment, should be disposed in layers which are thicker at their outer edges than at the centre.

When well built the centre third of the dam is composed of the best selected material, while on either side of it is laid common soil, which is usually not so impervious to water as that in the centre. On the lower side of the dam is laid any heavy material available. The main object in constructing an earth dam which has some impervious central core is to make this central wall and a small portion of the bank in the rear and a large section in front impermeable to the percolation of water; then the remainder of the bank to the rear is put in merely with the object of giving stability to the water-tight portion.

235. Homogeneous Earth Embankment.—This type of dam is considered by the writer and many other engineers as the most safe and efficient as well as economic. It is generally preferable in the arid region because of the saving in transportation of cement, rock, or selected materials for a puddle wall. Such a dam should be of the same density throughout, and composed of material practically impervious to water. It should form in itself and with the natural material on which it rests a perfectly homogeneous mass. Practically it is difficult to obtain such a structure, though the engineer should come as near as possible to the ideal. A puddle or masonry core is considered by some Western engineers as an element of weakness in the structure. They say that in a homogeneous earth dam the up-stream face is that point at which the water pressure ceases either by the water ceasing to penetrate the body of the dam or by its having free egress from the down-stream side. The puddle or masonry wall will stop the small amount of water coming through a new dam, and this will accumulate in the earth against the core, and will finally permeate the whole body of the dam above the wall, thus causing the water pressure which should be exerted against the up-stream face to be exerted against the core. The whole duty of the dam is then performed by the masonry core and the material below it.

If enough impervious material cannot be had to build the whole structure up homogeneously in layers, the up-stream third or half should be built of the best material available, the poorest and heaviest being put in the lower side. These two classes of material should be well worked into one another so as to give a perfect bonding. This practically converts the principal third of the dam into a puddle face, only the whole structure is built up at the same time in irregular layers of 1 or 2 feet in thickness, and well tramped over or puddled. By not building it in uniform layers a better bond is given to the structure. With such a form of construction any water which may soak through the upper third will find free egress from the dam on its lower side. The result will be to keep water out of the dam if possible, but when it enters to pass it through quickly. In building a dam up in irregular layers in this way these layers should be so disposed that the outer edges or extremities of each layer shall be higher than the centre of the layer by from 2 to 4 feet. As built in the West with teams and scrapers, no runways should be provided, the teams being driven over the whole surface, thus adding to the density and compactness of the structure. As each layer is built up it is well to drag or harrow it, and then pass a heavy roller over it. The same result can be produced by rolling it with a heavy roller having annular projections or rings on its surface.

236. Embankment Material.—The ideal material of which to construct an earth dam is such a mixture of gravel, sand, and clay that all the coarser interstices between the particles of the former shall be filled by the sand, and that all the minute openings between the particles of this material shall be filled by the still finer particles of clay. This would give such a composition that water would pass through it with the greatest amount of resistance, and the bank would be practically impervious. In practice, with proper care to mix the materials so as to thoroughly incorporate them one with the other, the following proportions should be used:

Coarse gravel	1.00	cubic yard
Fine gravel	0.35	"
Sand	0.15	"
Clay	0.20	"

Giving a total of about 1.70 cubic yards, which when well mixed, compacted, and rolled can be reduced to about $1\frac{1}{4}$ cubic yards in bulk. These proportions will rarely be obtained, but the effort should be to approach as nearly to them as possible in order to produce the best combination of materials. Weight is a valuable property in an earth embankment, and such a combination as above given possesses the greatest amount of weight obtainable with earth. The sand and gravel lack cohesiveness but have stability, while clay though cohesive is liable to slip if unsupported. The combination above given possesses the qualities of weight, cohesiveness, stability, and imperviousness, while the angle of repose or the slope which can be given is about midway between that possible with fine sand and that to be obtained with shingle or a mixture of sand and clay. If judgment be used in choosing materials, dirty gravel, or that possessing a large amount of soil and sandy matter, may often be found which will give nearly the proportions above specified.

237. Interior Slope and Paving.—The interior slope of an earth dam is rarely made uniform, while the exterior slope though usually uniform is sometimes broken by a level bench (Fig. 81), the object of which is to prevent serious effect from the sliding of the embankment. This bench is usually made from 4 to 6 feet in width. On the interior slope one or more similar benches are sometimes introduced, though rarely more than one. In the case, however, of the great dam being built for the Citizens' Water Company in Denver the slope is to be broken by a number of benches. In addition to this break in the slope, it is not uncommon to give a lighter slope below the bench and a steeper inclination for the last 5 to 7 feet at the top of the inner slope (Pl. XX). This steepness at the top is to prevent waves at flood height from slopping over the crest of the embankment, the sharp angle breaking the waves up and reflecting them back. The bottom of the inner slope is sometimes made steeper if the material will stand it, as it is not exposed to the air by the drawing off of the water as is the upper portion of the embankment.

This interior slope is invariably paved with cobble-stones or dry rubble tightly driven home and carefully placed (Pl. XX). The object of this pitching is to protect the embankment against the erosive action of the waves, and its thickness depends on the height and violence of these. The maximum height of the waves depends on the fetch or distance from the shore where their formation commences, and may be determined by Stephenson's formula,

$$X = 1.5 \sqrt{F} + (2.5 - \sqrt[4]{F}),$$

where X equals the height of wave in feet and F equals the fetch in nautical miles. Rankine states that where an embankment of loose stone is exposed to the action of the waves it should be faced with blocks set by hand, the least dimension of any block in the facing being not less than two thirds the greatest wave height. The best way in which to lay the stones is to place them with broad ends downwards, rough squared stones being preferable, in order that they shall fit fairly close one to the other. The interstices should be packed with small stone chippings and finished off with earth (Fig. 81).

The entire height of the inner slope need not be protected by a stone pitching. That portion of the slope which is below the level of the outlet sluices requires no pitching at all, as it will not be subjected to wave action. The lower portion of the exposed slope need be pitched with a lesser thickness than the upper portion, as the fetch will be less, and consequently the wave height less and its erosive action proportionately diminished. At the upper portion of the slope the pitching should be carried quite to the top of the embankment, and for safety might be carried across the top, in order that any spray falling on the top of the embankment should do the least possible amount of damage. It is customary to give the top of the embankment a slight inclination toward the reservoir, so that it will drain into it and not outward over the unprotected lower slope. For better protection of this exterior slope it should be planted with grass, or, better still, sods of considerable size should be placed upon it a few feet apart, in order that the roots of these may spread and entirely protect it from the erosive action of rain and spray.

238. Earth Embankment with Masonry Retaining Wall.—It is sometimes necessary to economize reservoir space, in which case one side of the embankment may be

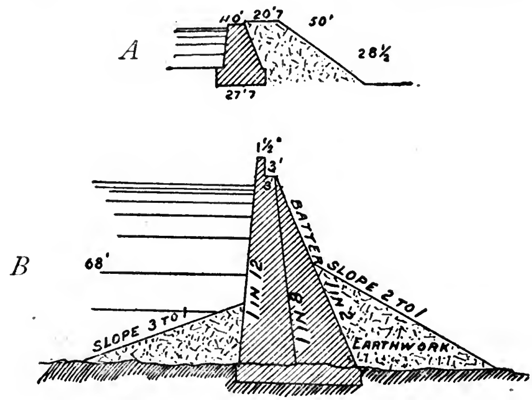

FIG. 64.—CROSS-SECTIONS OF KABRA DAM (*A*) AND EKRUK DAM (*B*), INDIA.

faced with masonry, though this combination is rarely successful or advisable. It has all the disadvantages of both earth and masonry dams without any additional advantages. The Kabra embankment in India (Fig. 64, *A*) is an example of this class of structure. It consists of a masonry wall on the front face of an earth embankment and having a steep batter of about 12 on 1, while the outer portion of the embankment and the lower slope have the natural slope of the earth, which is merely used to give stability to the masonry facing wall, the latter being the dam proper.

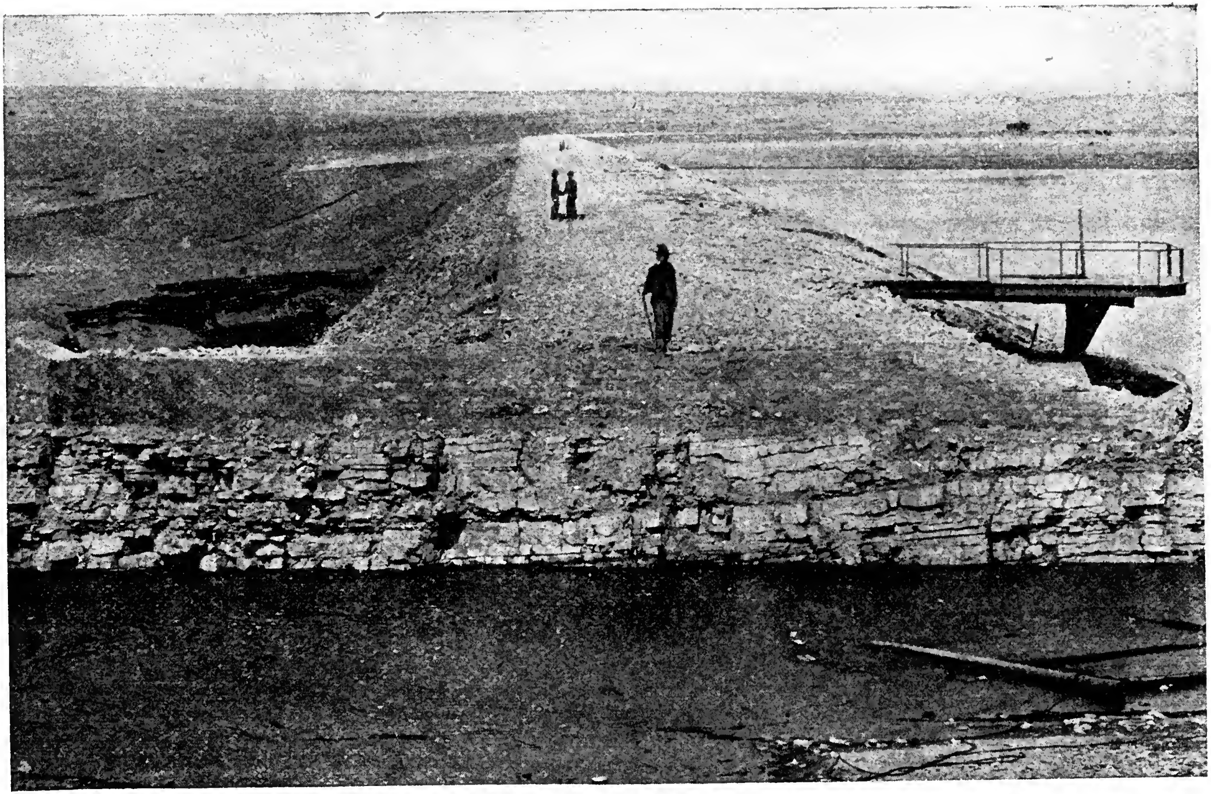

PLATE XXI.—VIEW OF PECOS DAM.

The masonry may be put in as in the case of the Ekruk tank in India (Fig. 64, *B*). This consists of a masonry core of such dimensions as to practically form the entire dam, the earth being merely added to the bottom of the slopes to give stability. In this case the masonry dam has an inner slope of 12 on 1, an outer slope of 2 on 1, and a total height of 68 feet. Against it, on its upper side, is an earth embankment with a slope of 1 on 3, reaching to about 25 feet in height, and on the outer slope another earth embankment with a slope of 1 on 2, reaching to about 35 feet in height. Above this the masonry is unsupported. Still another method of using masonry with earth is where the inner slope of the dam is of earth, its water face being riprapped as before described and a puddle wall placed through its centre to prevent percolation. On the outer slope, in place of the usual mass of material intended to add stability, is built up a rubble retaining wall, the stones being set in mortar, the object of the wall being merely to retain the embankment, and not to prevent percolation; also to avoid covering land below the dam which may be of value.

239. Earth and Loose-rock Dams.—Pecos Dam.—The dam at the head of the Pecos Irrigation Company's canal, in New Mexico (Pl. XXI), furnishes an excellent example of this combined construction. This dam is shaped in plan like the letter L, the re-entrant angle of which points up-stream. The

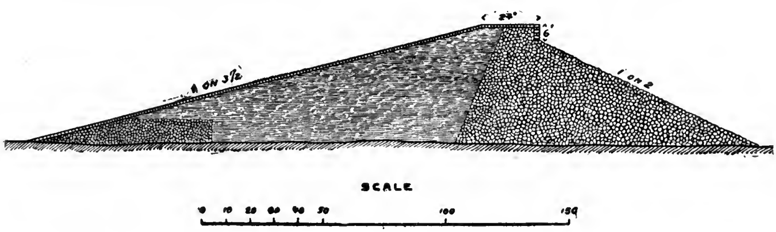

FIG. 65.—CROSS-SECTION OF PECOS DAM.

long arm which composes the main dam is 1070 feet in length and varies from 5 to 50 feet in height; the short arm consists of a simple earth embankment 530 feet in length, with an average height of 52 feet. Adjacent to the end of the dam farthest from the headgate is a wasteway 250 feet wide, excavated in limestone rock, its bed being 5 feet below the crest of the dam. This wasteway is 300 feet long and has a grade of 1 in 3. At the lower end of the rock cut on the left bank of the river is an additional wasteway just below the end of the dam. This wasteway has a total length of 206 feet, its sill being about 2 feet lower than the one first mentioned. The main dam (Fig. 65) is composed of a prism of loose rock 12 feet wide on top, 100 feet wide at bottom, with a lower or outer slope of 1 on $1\frac{1}{2}$ and an inner slope of 1 on $\frac{1}{4}$. Its maximum height is 50 feet, and the up-stream face is backed with an earth embankment the width of which is 10 feet at top and 200 feet at the bottom; its up-stream slope being 1 on $3\frac{1}{2}$ and paved with 18 inches of stone riprapping. The lower portion of this slope near the outlet sluice is replaced by 10 feet in depth of loose rock for a total width through the dam of 75 feet, to prevent undercutting by currents. At the top of the outer slope is a low masonry wall 5 feet in height and 2 feet in width, built as a retaining wall to give the requisite top width to the embankment. In the bottom of the dam near the end adjacent to the canal head is an undersluice the sill of which is 37 feet below the dam crest. This sluice is 4 by 8 feet in the clear and has a grade of 1.2 in 100, its discharge capacity with a full reservoir being 2000 second-feet. The lining of the culvert composing the undersluice is of rubble masonry 8 feet in thickness.

240. Loose Rock and Earth Dam.—Idaho Dam.—An excellent example of this class of structure is that being built at the head of the Idaho Mining and Irrigation Company's canal (Fig. 66). The site of this dam is at a point where solid basalt outcrops across the channel of the Boise river, and the dam is to be founded on this. Just above the dam is a basalt ledge 12 feet in height which borders the river bank, and on this will be constructed the wasteway, with a width of 450 feet. This wasteway is to be excavated in gravel and carried to a depth of 8 feet below the crest of the dam. It will be 720 feet in length, and will discharge back into the river 100 feet below the dam. In it will be built a waste weir of rubble masonry across the entire width of the channel and founded on the basalt underlying the gravel. The object of this weir is to make the crest of the wasteway at the required height with relation to that of the main dam. Its pro-

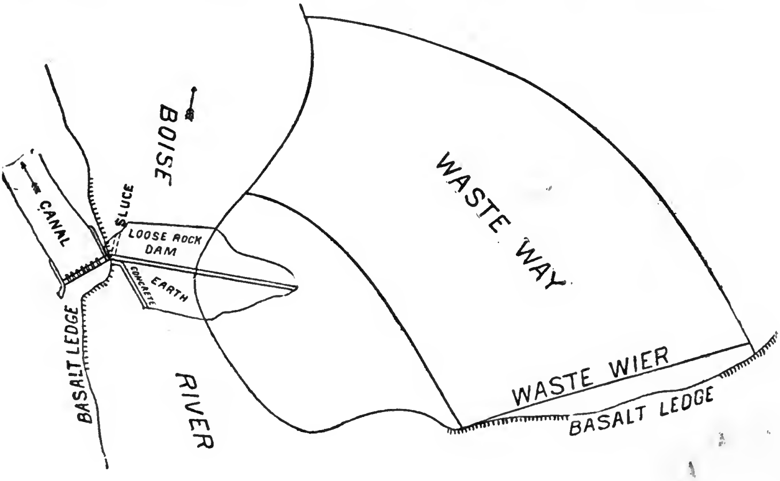

FIG. 66.—PLAN OF IDAHO DAM.

posed cross-section is peculiar, its base being 19 feet in width and its maximum height 8 feet. Its upper slope will have a batter of 6 on 1, while its lower slope will have an ogee-shaped curve.

The main dam (Fig. 67) is to be constructed of loose rock with an earth facing. It will be 220 feet in length on its crest and

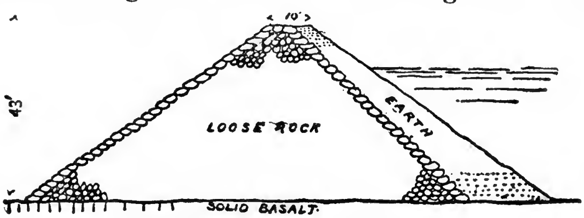

FIG. 67.—CROSS-SECTION OF IDAHO DAM.

43 feet in maximum height, its top width being 10 feet, of which 3 feet on the inner slope will consist of the top of the earth backing, which at the bottom of the dam is to be 20 feet in width. The lower or rock face of the dam will have a slope of $1\frac{1}{2}$ on 1, the up-stream or earth slope of the rockwork being the same.

241. Loose-rock Dams.—When properly constructed and well founded there is no apparent reason why a loose-rock dam should not be nearly as substantial as one of masonry. Such dams should be founded only on solid rock, hardpan, or on beds of very stiff clay or other unwashable material. This dam is the outcome of Western engineering practice, and was first introduced for the purpose of storing water for placer mining: hence it is generally known as the hydraulic-mining type of dam. It consists of a mass of loose rock placed together with some degree of care, the smaller stones being used to fill the interstices between the larger ones so that the settlement shall be the least possible. Such slopes are given the mass as it can safely stand, and it is rendered impervious to water by a heavy sheathing of tarred planking or an earth embankment on its upper face. Water should not be permitted to flow over the crest or back of such a structure, as it is liable to cause settlement which may result in its rupture.

It is claimed by their advocates that rock-filled dams are cheaper than those of masonry or earth. The latter is unquestionably cheaper than a rock-filled dam in nearly every case, while if transportation is not expensive a masonry dam is frequently cheaper than a rock-filled dam owing to the difference in cross-section and the correspondingly small amount of material required in the former, though the cost per cubic yard is relatively high. One of the great advantages of the rock-filled dam is that it may be constructed with very little difficulty in flowing water; another advantage is that a leak is not the menace it is in an earth or masonry dam, since the whole structure is expected to leak. A masonry dam is, during its existence, in a state of unstable equilibrium, while a rock-filled dam from the process of its construction tends to improve with time, and if properly built it may be benefited by causes which threaten other dams. Such a dam as this should not be used where water is valuable unless great care is taken in providing against leakage, and this can only be well done when an earth filling or facing is used. In preparing the foundation for a loose-rock dam the only precaution necessary is that it shall be founded on impervious and unwashable material. If there be a surface covering of loose soil or gravel it should either be removed by carts, or if the current in the stream is sufficient it may be washed away as the dam is built up.

A loose-rock dam should be built up in layers as is done with an earth dam, and in such manner that the centre of each layer shall be lower than the outer extremities. The best cross-section for such a dam is an upper slope of $\frac{1}{3}$ to $\frac{1}{2}$ on 1 and a lower slope of 1 on 1; anything less than this cannot be considered secure.

242. Walnut Grove Dam.—This is an excellent example of the rock-filled dam. It was destroyed in February, 1890, by a great flood, though its destruction was not a result of faulty design, but of carelessness in one or two details of its construction,—notably in the failure to provide an ample waste-way and in the careless manner in which the stones were dumped in the centre of the structure. This dam (Fig. 68) rested on the firm rock of the stream bed throughout its length, with the exception of a small portion of the upper wall, which is believed to have rested on from 5 to 12 feet in depth of loose earth and gravel. This was one of the weak points of its construction.

The dam was 420 feet long on top, 138 feet wide at the bottom, 15 feet in width on top, and 110 feet in greatest height, and contained nearly 50,000 cubic feet of material. It consisted of a front and back wall, each 14 feet thick at the base and 4 feet on top, with a loose-rock filling between; the whole made water-tight by a wooden sheathing. The upper slope of the dam was $2\frac{1}{4}$ on 1 and the lower slope $1\frac{1}{2}$ on 1. This latter, however, was increased for the lower half of the dam to about 1 on 1 by the addition of a pile of loose rock after the completion of the structure. The wooden sheathing consisted of logs from 8 to 10 inches in diameter and from 6 to 12 feet in length, built into the wall on its upper face and projecting therefrom about 1 foot. The upper and lower faces consisted of rough blocks of granite dry-laid in such manner as to form two loose-rock retaining walls, between which the body of the loose stone was dumped. Vertical stringers about 8 by 10 inches were bolted to the projecting ends of the logs built into the upper face, and these stringers were placed about 4

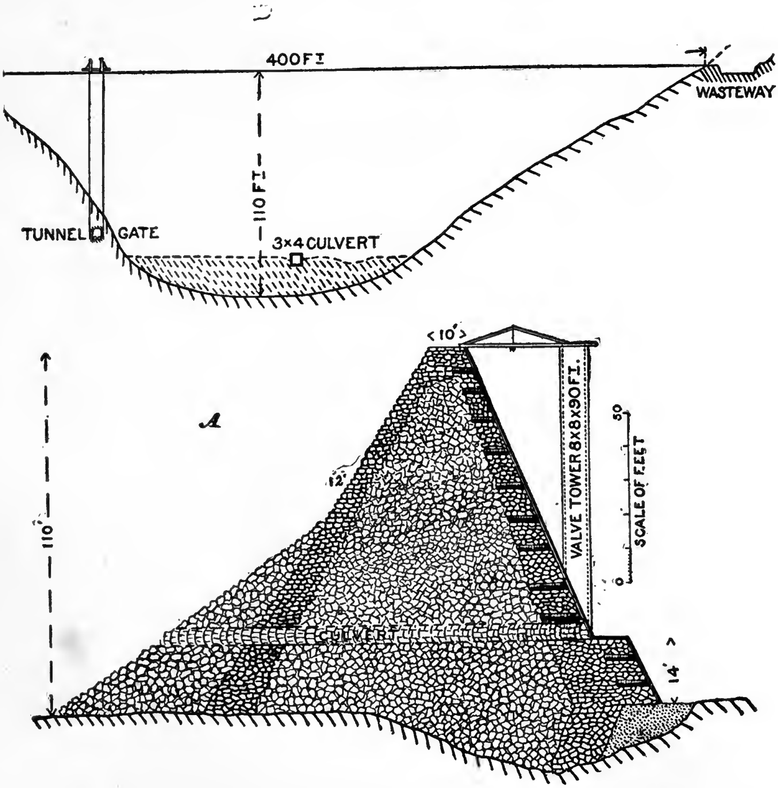

FIG. 68.—ELEVATION AND CROSS-SECTION OF WALNUT GROVE DAM.

feet apart. Upon the face of the dam and over these stringers two thicknesses of 3 by 8 inch planking were spiked, and tarred paper was laid between the two. The outer face of this sheathing was finally calked, and the whole covered with paraffine paint.

243. Crib Dams.—The general form of construction and several examples of crib weirs were given in Articles 132, 133.

Structures of similar design have occasionally been built of sufficient height to form storage reservoirs. The employment

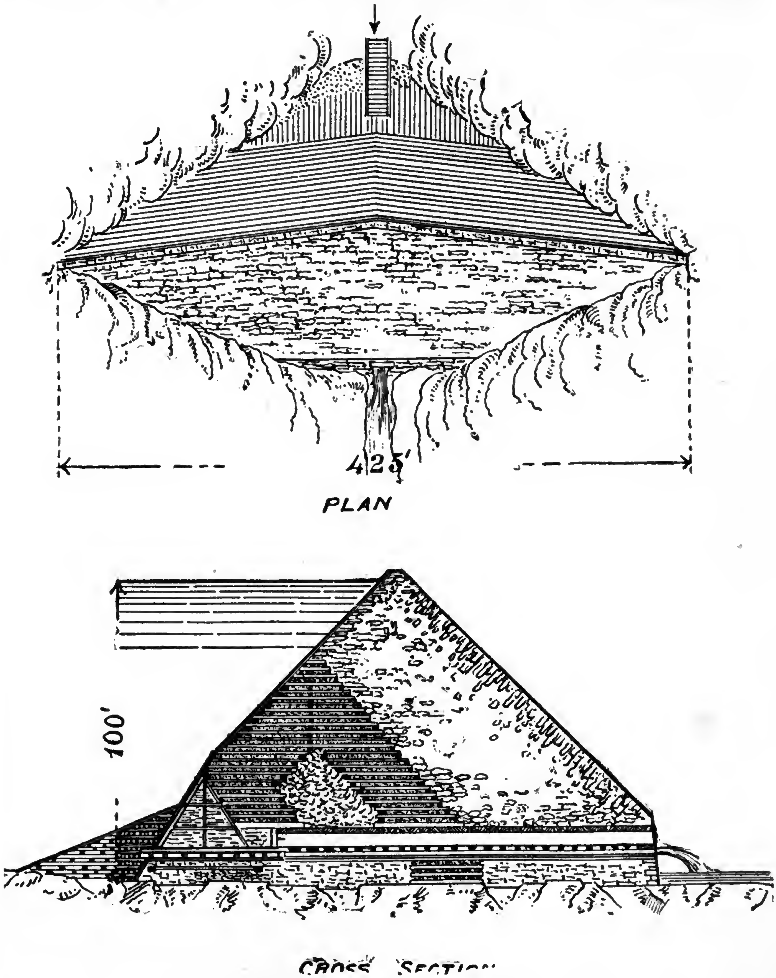

FIG. 69.—PLAN AND CROSS-SECTION OF BOWMAN DAM.

of cribwork in a storage dam is not recommended, as such work is essentially temporary in character. As a result of the alternate wetting and drying which it receives it is very liable to rot, and the life of such a dam is manifestly shorter than that of an earth, loose-rock, or masonry dam.

Several types of crib and combined crib and loose-rock dams have been constructed in the Sierras of California for the storage of water for hydraulic mining. One of the most notable of these was the crib and loose-rock dam built to close the English reservoir in Sierra county, California. This consisted of the usual form of timber crib made of tamarack logs and filled with stones. The height of this dam was 79 feet, and its width at the base 100 feet, the inner slope being a trifle steeper than $1\frac{1}{2}$ on 1 and the outer slope $1\frac{1}{2}$ on 1. The water face was covered with a heavy planking of pine, thus forming a water-tight lining to the dam. The lower slope of this crib-work was backed up by a loose-rock filling, hand-placed on the surface so as to have an even slope; the width of this filling being 55 feet at the base and 8 feet on top, its outer slope being 1 on 1. The discharge sluice of this dam consisted of a timber culvert built through it at its base.

Another typical dam of the same type is the Bowman dam, used for water storage by the North Bloomfield Mining Company, in California. This dam (Fig. 69) has a total height of 100 feet and uniform slopes on both faces of 1 on 1. Its lower third on the up-stream side consists of a cribwork of logs filled with rock, the cross-section of which is 1 on 1, while the remainder of the dam consists of loose rock hand-placed and carefully laid. The upper slope of the dam is sheathed with planking and the lower slope is faced with rubble masonry laid in cement. Through the bottom of the dam is an outlet culvert constructed of masonry and cement.

244. Loose-rock Dam with Masonry Retaining Walls.—Probably the only existing example of this type of construction is that closing the Castlewood reservoir in Colorado. This dam (Fig. 70) is founded on a bed of clay and bowlders from 7 to 30 feet in depth, and is composed of an outer shell or wall of large blocks of coarse rubble masonry, the thickness of which on the up-stream face is about 6 feet on top and 12 feet at the bottom. On the down-stream face the wall is from 5 to 7 feet in thickness, this face being laid in steps the height of which vary from $1\frac{1}{2}$ to $2\frac{1}{2}$ feet according to the dimensions of the stone blocks forming them. The main body or centre of the dam consists of dry-laid rubble enclosed between these two walls. The maximum height of the dam is 63½ feet, it is 586 feet in length on the crest, and 100 feet of this length is lowered 4 feet in order to form a wasteway over which flood waters may discharge. The upper 4 feet of this dam is vertical on both sides, and its top is 8 feet in width and constructed of rubble masonry in cement. The outer slope of the remainder of the dam is 1 on 1, while the inner slope is 10 on 1. It

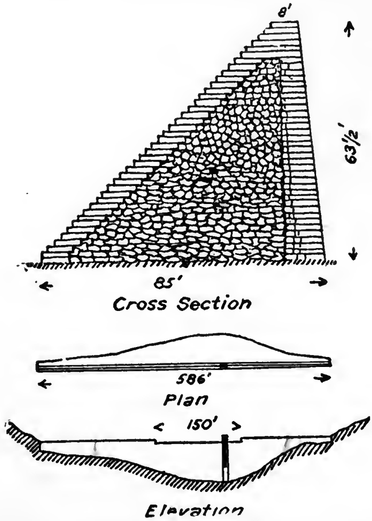

FIG. 70.—ELEVATION, PLAN, AND CROSS-SECTION OF CASTLEWOOD DAM.

is possible that in the future such a type of dam as this may become popular. It possesses all the good qualities of the loose-rock dam and need be no more expensive, since its slopes may be made a little steeper. It is doubtful if so steep a slope as 10 on 1 for the upper face is safe: probably 5 on 1 would be better, while 1 on 1 for the rear face is ample to give stability. In such a structure as this great care should be taken to firmly found it on solid rock or on a deep bed of hard and impervious clay, while the loose-rock centre should be carefully laid to prevent any inclination to slide or thrust outward against the confining walls.

CHAPTER XIX.

MASONRY DAMS.

245. Theory of Masonry Dams.—Masonry dams are employed both for diversion and storage works, and may be so constructed as either to permit flood water to pass over their crests or have it passed around one end. If the dam is to be used for storage purposes only, and a sufficient wasteway can be provided, it may be designed according to one of the theoretical formulas or from one of the type profiles given hereafter. Dams constructed by these formulas contain the minimum amount of material necessary to enable them to perform their functions of holding up the storage water, and are not sufficiently substantial to withstand the shock produced by water falling over their crests. Where a masonry dam is used as a diversion weir or as an overflow weir, it is impossible to design it on any of the theoretical profiles. The chief calculation then requisite in its design is, that the pressure of the masonry on the foundation shall not pass the limit which the material can withstand, and also that its cross-section shall be more ample and substantial than that which would be required by one of the theoretical profiles.

The first and most vital rule in building a masonry dam is that it shall rest on solid and practically homogeneous rock. A masonry dam is practically an absolutely rigid structure, and settlement in any portion of its foundation will result in cracks and ultimate rupture in its mass. There are two ways in which a masonry dam may resist the thrust of water: first, by the inertia or weight of its mass, and, second, as an arch. Its safety depends upon compliance with the conditions—

1. That the horizontal thrust of the water must be held in equilibrium by the resistance of the masonry to sliding forward or overturning; and,

2. That the pressure sustained by the masonry or its foundation must never exceed a certain safe limit.

The thrust of the water may be resisted by being transmitted to the abutments, the dam acting as an arch. But three dams have as yet been built which depend in any degree for their stability on arch action, and the laws governing this action in a dam are as yet so uncertain that they cannot be depended upon with any degree of security. Some attempt at solving the rules on which a dam is dependent for its stability as an arch are given in Articles 255 and 256. According to J. B. Krantz, a dam which is curved in plan, with a radius of 65 feet or less will transfer the pressure of the water to the sides of the valley whatever the height of the structure. This, however, does not lessen the effect of the weight of the masonry, so that whether the structure be curved in plan or not, its weight must be supported in the same way, and the height must be such that this weight will not exceed the limit of pressure permissible on the base. In France, and in the case of the Fife dam near Poona, India, and elsewhere, reservoir walls have been reinforced by means of masonry counterforts. If the wall is strong enough by itself the counterforts are a useless expense, and if the wall is not sufficiently strong they will not prevent it from yielding. The masonry intended for the counterforts would always be better used if spread over the mass of the dam.

246. Stability of Gravity Dams.—The author will make no attempt here to enter into a tedious mathematical discussion of the theory of the stability of masonry dams. This question is one which has been investigated with great thoroughness within the past 15 years, and nothing which could be stated in this place will add to the value of the theories now held. For the benefit of students who desire to enter into the mathematics of this subject a list of authors is appended at the end of this chapter. Sufficient of the principles of the subject may be obtained from the works of Baker, Fanning, Wegmann, McMasters, Church, and Merriman, who are the more modern American writers on the subject.

The conditions on which the stability of gravity dams are calculated are:

1. The hydrostatic principles involved in the pressure of a volume of liquid on an immersed surface; the fact that this pressure is perpendicular to the surface; and that for rectangular surfaces it may be considered as a single force applied below the water surface at a distance equal to $\frac{2}{3}$ of its depth.

2. That a gravity dam may fail: 1, by sliding on a horizontal joint; 2, by overturning; or 3, by crushing of the masonry or foundation.

The stability of the dam against its liability to destruction, as enumerated in condition 2, page 249, must be determined—

1. When the reservoir is full; and,
2. When the reservoir is empty.

These two conditions give the extreme positions of the lines of pressure in a dam. The first causes the maximum pressure in any horizontal plane to be at the down-stream face of the wall, and the second produces them at the up-stream face. When the reservoir is empty the wall supports only its own weight, but if the wall has a uniform thickness the pressure per square inch will be about 85 pounds if the height of the structure is 85 feet. If the faces be inclined so as to reduce the mean thickness, the pressure on the base diminishes and the height can be accordingly increased. From this it is clearly seen that it is absolutely necessary to widen the base of the dam by inclining its faces if the wall is to have any great height; otherwise it would rupture from the pressure of the material composing its own mass. When the reservoir is full, however, the water contained in it bears upon the up-stream face with a pressure that increases with the square of the depth. In deep reservoirs this pressure is great, and exerts its effect in a resultant which is nearly horizontal in direction and carries the maximum load to the down-stream toe of the wall. For stability this resultant must pierce the base in front of this lower edge. From these considerations arises the necessity of giving the down-stream face a greater batter than the up-stream face.

The tendency of the water pressure to produce overturning or sliding and the weight of the material are greater for each successive layer of the mass of the dam from the top downwards. As a result of this the width of the dam at the top might theoretically be *nil*, and should be increased downwards in such a proportion as to render the dam capable of resisting tendencies to crushing, sliding, and overturning. From theoretical examinations of the effects of these forces it has been found, keeping constantly in view the necessity of making the batter of the down-stream face the greater, that the dam should have a triangular profile, somewhat similar to that represented in Fig. 71.

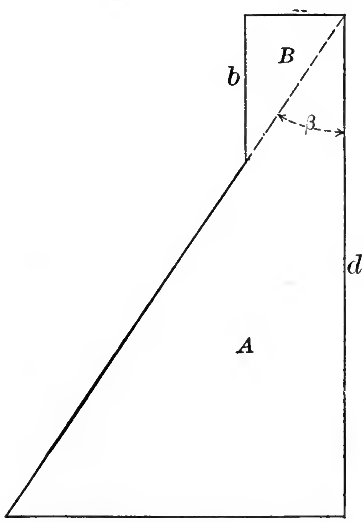

FIG. 71.—THEORETICAL TRIANGULAR CROSS-SECTION OF DAM.

247. Stability against Sliding.—The tendency of the water pressure to slide any portion of the dam forward on a given horizontal plane is resisted by the friction due to the weight of the mass above it. The dam is necessarily founded on firm rock the disintegrated and weaker portions of which must be removed, and as a result the base is usually sufficiently rough to offer considerable resistance to sliding. If this is not the case steps must be cut for a few feet in depth in the foundation rock, or this must be irregularly cut in such manner as to leave trenches in which projections of the dam will fit. The dam, if properly constructed, is safe against any liability to slide providing its profile is such that it will resist overturning; therefore the usual computations entered into to determine whether it will resist sliding are practically unnecessary. If it be constructed of rough rubble masonry without regular beds, and so built as to form a monolithic mass, sliding is impossible. It is well known that the force required to make two pieces of smooth stone slide upon each other when dry or joined by fresh mortar is equal to about .75 of the normal pressure. Hence sliding would only be possible when the horizontal was equal to $\frac{3}{4}$ of the sum of the vertical pressures. In none of the formulas or profile types ordinarily employed is the ratio of the thrust to the pressure beyond .7, while it more ordinarily ranges between .3 and .5

248. Coefficient of Friction in Masonry.—In the following table are given the coefficients of friction in dry masonry of various kinds:

TABLE XII.

COEFFICIENTS OF FRICTION IN MASONRY.

	Coefficient.
Point-dressed granite on like granite	.70
Point-dressed granite on brick	.63
Point-dressed granite on smooth concrete	.62
Fine-cut granite on like granite	.60
Fine-cut granite on béton block	.60
Dressed granite on granite with fresh mortar	.50
Béton blocks on béton blocks	.65
Common brick on common brick	.65
Common brick on common brick with wet mortar	.50
Common brick on dressed limestone	.60
Dressed hard limestone on limestone	.65
Dressed soft limestone on like limestone	.75

According to J. T. Fanning, let

$S =$ the symbol of friction of stability;
$x =$ the horizontal water pressure resultant;
$c =$ the coefficient of friction of the given section;
$w =$ the weight of masonry above that section;
$e =$ the vertical downward water pressure resultant;
$z =$ the maximum upward water pressure resultant;
$c' =$ the ratio of effective upward water pressure to the maximum.

Then, when S and x are equal to each other, the wall is on the point of motion and S must be increased. This has to be done by adding more weight to the wall. This weight should be increased until it is able to resist a thrust of at least $1.5x$, when

$$S = (w + e - c'z) \times c = 1.5x.$$

The wall has a small margin of fractional stability when $x = 2.25$ tons. Ordinarily the weight or pressure of the wall far exceeds this figure, and is usually from 5 to 12 tons per square foot. For equilibrium, let

$$x < cw + ml,$$

in which m is the cohesion of the masonry per square unit and l the length of the joint at the section above x. The value of m is so considerable that ml may be considered as a margin of safety, when we have $x = cw$. To find what value of c will prevent sliding, we have $c = \dfrac{x}{w}$.

A masonry wall must be founded upon solid rock which is either naturally uneven or must be made so, and it must be made of rubble masonry or concrete not laid in courses. As there can therefore be no smooth planes to slide one upon the other, the coefficient of friction in the mass must be many times the superincumbent weight; and we may conclude, therefore, that there is no possible danger of failure from sliding.

249. Stability against Crushing.—According to the method given by Debauve, when the reservoir is full and the resultant of the pressure of the water and the weight of the masonry intersects the base at one third of its width from the down-stream toe, the maximum pressure is at this toe, and is double what the pressure per square inch would be if the weight were uniformly distributed over the whole base. When the reservoir is empty the conditions are reversed, the maximum pressure being at the up-stream toe and equal to double the average pressure on the base.

From this proposition Mr. James B. Francis differs. He believes that the pressures near the base of the wall are practically zero, and that these pressures are transferred to the central part of the mass, where the resistance to crushing is greatest. In other words, that the masonry is not perfectly rigid, and that it becomes accordingly unnecessary to take account of crushing pressures in a dam less than 200 feet in height. In this opinion other authorities agree with Francis to a limited extent, though all prefer to calculate the limit of pressure in the usual manner, namely, to measure the pressures near the face of the wall, as that gives a safer factor, though it may be unnecessarily high. As parts of the dam are built at different times in the year and under different conditions, the structure cannot be truly homogeneous. The absence of fractures at the thin portion near the toe of the dam indicates the absence of excessive strains at that point; it is therefore more probable that the real point of distribution of pressure lies somewhere between the extremes enumerated by Debauve and Francis. Up to the limit of 200 feet in height there is no doubt that the crushing strength of well-laid masonry need not be considered.

The following, from Wegmann, is a brief synopsis of a simple formula for finding the distribution of pressure at any point in a dam:

Let $W =$ the total pressure on the base;
$u =$ the distance of W from the nearest edge;
$p =$ the maximum pressure on the foundation;
$q =$ the minimum pressure on the foundation;
$l =$ the length of the joint or base under consideration.

Then $p = \frac{2W}{l}\left(2 - \frac{3u}{l}\right)$. When $u = \frac{l}{3}$, or in other words the pressure is within the middle third of the base, $p = \frac{2W}{l}$. If the pressure is without the middle third there will be tension in the mass. As it is unsafe to depend on the tension in masonry, it would be best to neglect this in calculating the pressure on the foundation, and this will become $p = \frac{2W}{3u}$. Another simple formula for determining the pressure on the base, and one which leads to practically similar results, is the following, given by Ira O. Baker:

$$p = \frac{W}{l} + \frac{6Wu}{l^2}.$$

250. Limiting Pressures.—The limiting pressures which it may be safe to permit in masonry differ considerably according to various authorities. From actual tests these pressures differ according to the dimensions of the masonry blocks, and it is probable that much greater pressures can be sustained per unit of area in the interior of large masses than in the smaller experimental blocks or near the surface of the mass. The following pressures are ordinarily accepted: Brick, 120 pounds; sandstone, 130 pounds; limestone, 152 pounds; granite, 155 pounds per square inch. It is not advisable to allow either a direct or resultant pressure exceeding 140 pounds per square inch within 1 foot of the face of rubble masonry or exceeding 200 pounds per square inch in the heart of the work. On some of the great structures already built limits of pressure as low as 85 pounds have been adhered to, while pressures exceeding 200 pounds per square inch have been permitted in the Almanza and the Gros Bois dams in Europe.

Among the great dams which have been constructed the pressures vary between 5.8 tons per square foot in the Verdon dam in France and 14.6 tons per square foot in the Gros Bois dam, while the proposed Quaker Bridge dam, in New York, was designed for a maximum pressure of 16.6 tons per square foot. It is probable, however, that a safe average limit is that already given of from 140 to 200 pounds per square inch.

251. Stability against Overturning.—To insure ample safety against all the causes of failure in a dam in addition to the other conditions already fixed, the lines of pressure must lie within the centre third of the profile, whether the reservoir be full or empty. This last condition precludes the possibility of tension, and insures a factor of safety of at least two against overturning. In Fig. 72 suppose the lines of reaction R and

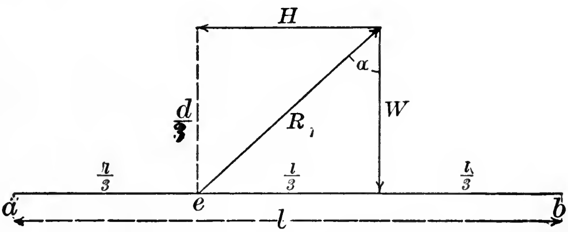

FIG. 72.—DIAGRAM ILLUSTRATING WEGMANN'S FORMULA.

W to intersect the joint l at the limit of its centre third. Taking the moments of the three forces, H, R, and W, which are in equilibrium at about the point e, we find $\frac{Hd}{3} = \frac{Wl}{3}$, in which $d =$ the depth of water at the joint above the plane of l. If the moments are taken about the front edge a, the lever arm of W will be double, while that of H remains unchanged; the factor of safety against overturning is therefore two. It is equally evident that if the line of reaction of W or R should intersect l within its centre third, the factor of stability would be greater than two.

The following formulas are taken from the treatise of Edward Wegmann, Jr., on Masonry Dams, because the author considers them simple and accurate. For their deduction and discussion the student should refer to this work. The mass of the cross-section of the dam should be rectangular and will contain an excess of material as regards resistance to the hydrostatic pressure of the water; P' will pass through the centre of the rectangle, and P will gradually approach the front face eventually reaching some joint $x = a$ where $u = \frac{a}{3}$. The depth of this joint below the top of the dam is $d = a\sqrt{r}$, where

$P =$ the line of pressure, reservoir full;
$P' =$ the line of pressure, reservoir empty;
$x =$ the unknown length of the joint;
$u =$ the distance of P from the front edge of the joint x;
$a =$ the top width of the dam;
$d =$ the depth of water at the joint x;
$r =$ the specific gravity of the masonry.

For the next course below the joint x, where the dam begins to assume a trapezoidal cross-section, we have

$$x^2 + \left(\frac{4w}{h} + l\right)x = \frac{6}{h}\left(wm + M\right) + l^2, \quad . \quad . \quad . \quad (2)$$

in which $w =$ equals the total weight of masonry resting on the joint l.

$l =$ the known length of the joint above x;
$h =$ the depth of a course of masonry, assumed as 10 feet;
$m =$ the distance of P' from the back edge of the joint l; = up stream
$M = \frac{d^3}{6r} =$ the moment of H on the joint x;
$H = \frac{d^2}{2r} =$ the horizontal thrust of the water.

Equation (2) may be used for a series of joints down to a depth where the back surface of the dam begins to slope or until a joint is found where $n = \frac{x}{3}$; n being the distance of P' from the back edge of the joint x. For the next course both faces will have to be sloped, and $u = n - \frac{x}{3}$, when we obtain

$$x^2 + x\left(\frac{2w}{h} + l\right) = \frac{6M}{h}. \quad . \quad . \quad . \quad . \quad . \quad (3)$$

In applying equation (3) for finding the value of x, the maximum pressure must be obtained both with reservoir full and empty. This may be done by the formula

$$x^2 = \frac{6M}{p}, \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (4)$$

in which p = the limiting pressure per square foot at the front face of the dam. This equation may be employed until the limiting pressure is reached at the back face, when the following formula must be used:

$$x^2\left(p + q - h\right) - 2x\left(w + \frac{lh}{2}\right) = 6M, \quad . \quad . \quad . \quad (5)$$

in which q is equal to the limiting pressure per square foot at the back face of the dam, and is generally assumed to be greater than p.

These equations give the successive lengths of the joints, but do not give their position. This may be found by determining the value of y = the batter of the back face; the formula being

$$y = \frac{2w(x - 3m) - hl^2}{6w + h(2l + x)}, \quad . \quad . \quad . \quad . \quad . \quad (6)$$

and for equation (5),

$$y = \frac{w(4x - 6m) + lh(x - l) + x^2(h - q)}{6w + h(2l + x)}.$$

The theoretical profile resulting from calculating the dam by the above formulas will have polygonal faces. It only becomes necessary then to make the value of h sufficiently small to determine a profile with a smooth surface which will fulfil all of the conditions.

252. Molesworth's Formula and Profile Type.—Mr. Guilford L. Molesworth has worked out the following formula, the application of which gives the profile shown in Fig. 73:

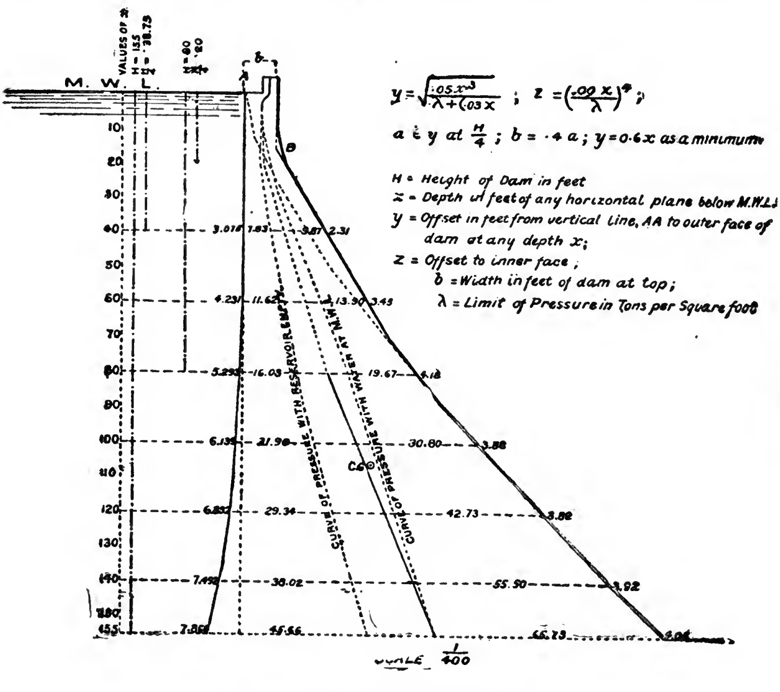

FIG. 73.—MOLESWORTH'S PROFILE TYPE.

$$y = \sqrt{\frac{.05x^3}{\lambda + (.03x)}}. \qquad z = \left(\frac{.09x}{\lambda}\right)^4.$$

$y =$ the distance measured along any joint in the masonry from the down-stream face to a vertical line drawn from the top front edge of the dam to the base;

$z =$ the corresponding distance on the same joint to the up-stream face;

x = the distance from the top of the dam to the joint above mentioned;

$y = .6x$ as a minimum;

λ = the limit of pressure of the masonry in tons per square foot;

H = the minimum height of dam;

$a = y$ at $\frac{H}{4}$ from the top;

b = top width $= \frac{a}{2}$.

253. Height and Top Width of Dam.—As far as the forces already considered are concerned, the top width of the dam might be zero and the water might rise to its crest. In practice a certain definite top width must be given in order to enable the dam to withstand the shock of waves and ice, and the top of the dam must be continued above the maximum flood-water line for a sufficient height to prevent its being topped by waves. Ordinarily the top width of the dam should be sufficient to enable it to act as a roadway and afford communication between the two slopes of the valley. It should never be less than 5 or 6 feet, and for the highest dams need never exceed 15 feet, varying between these according to the height of the wall.

Having calculated the height of the dam for maximum flood heights of water, this should be continued upward a sufficient amount to insure it against being topped by the waves. The height of waves depends on complex causes, chiefly on the depth of the reservoir and the fetch, a formula for computing which was given in Article 237. The maximum amount to which it will be necessary to increase the computed height of the dam need rarely or never exceed 10 feet, its minimum being as low as one foot in an extremely shallow and small reservoir. On top of the crown of the dam there should always be a parapet as an additional precaution against its being topped by waves, and this parapet may be from 3 to 5 feet in height.

254. Profile of Dam.—In Fig. 74 is given a comparison of the profiles obtained by several of the more common formulas, while that which is shown in full lines is the practical profile type No. 3, adopted by Wegmann. This profile (Fig. 75) can be changed to another having any desired top width equal to one tenth the height by simply changing the scale of the

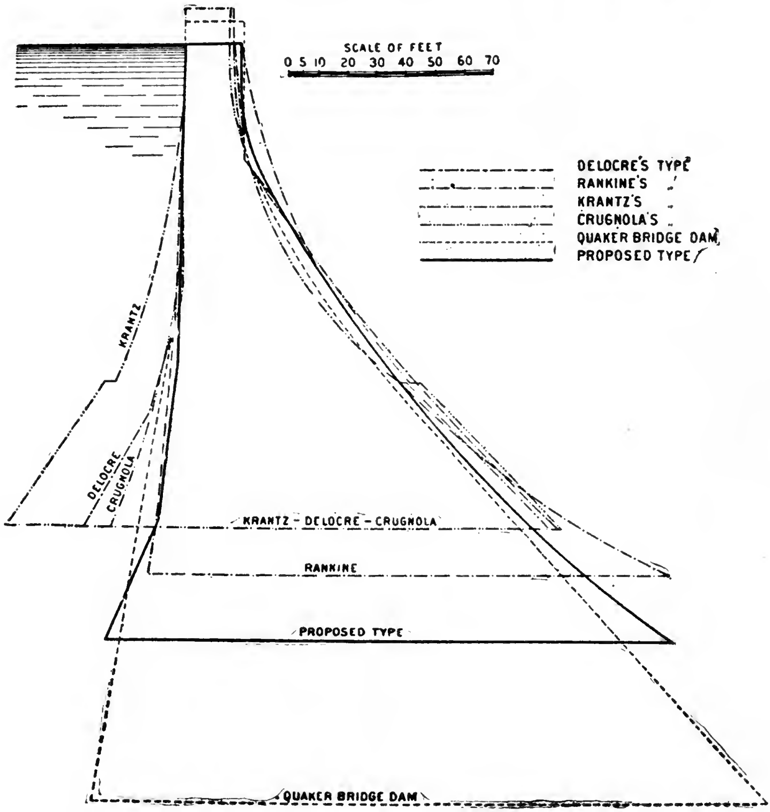

FIG. 74.—COMPARISON OF PROFILE TYPES.

drawing. In the following table are given the dimensions and pressures for this profile type. The specific gravity of the masonry employed in making these computations is assumed at $2\frac{1}{3}$.

255. Curved Masonry Dams.—A dam of the kind already considered is of the pure gravity type and relies for its stability solely on the weight of the masonry and its friction. A

TABLE XIII.

WEGMANN'S PRACTICAL PROFILE NO. 3.

Depth of Water below top of dam, in feet.	Horizontal Thrust of Water in cubic feet of Masonry.	Moment of Water in cubic feet of Masonry.	Joint referred to a vertical axis. Left in feet.	Joint referred to a vertical axis. Right in feet.	Joint referred to a vertical axis. Total in feet.	Total area in square feet.	Distance from front face to line of pressure, Reservoir full, in feet.	Distance from back face to line of pressure, Reservoir empty, in feet.	Maxima Pressures. Reservoir full. In feet of Masonry.	Maxima Pressures. Reservoir full. In tons of 2000 lbs.	Maxima Pressures. Reservoir empty. In feet of Masonry.	Maxima Pressures. Reservoir empty. In tons of 2000 lbs.	Coefficient of Friction necessary for Equilibrium.	Factor of Safety against Overturning.
0.000	0	0.0	18.74	0.00	18.74	0.00	9.37	9.37	0.00	0.00	0.00	0.00	0.00	0.0
16.585	59	325.9	18.74	0.00	18.74	310.80	8.32	9.37	22.16	1.62	16.59	1.21	0.19	8.9
20.000	86	571.4	18.86	0.00	18.86	374.98	7.93	9.41	29.37	2.14	20.01	1.46	0.23	6.2
30.000	193	1928.5	20.56	0.00	20.56	570.33	7.67	9.51	48.87	3.57	33.97	2.48	0.34	3.3
40.000	343	4571.4	24.52	0.00	24.52	793.65	8.78	9.98	59.93	4.38	50.43	3.68	0.43	2.5
50.000	535	8928.6	29.95	0.00	29.95	1065.69	10.65	10.92	66.41	4.84	64.49	4.70	0.50	2.2
60.000	771	15428.6	35.71	0.43	36.14	1395.80	12.44	12.64	74.72	5.45	73.44	5.35	0.55	2.1
70.000	1049	24500.0	41.81	0.87	42.68	1789.57	14.40	14.59	82.84	6.04	81.72	5.96	0.59	2.0
80.000	1370	36571.4	48.29	1.30	49.59	2250.59	16.62	16.72	90.28	6.58	89.73	6.54	0.61	2.0
90.000	1734	52071.4	55.15	1.73	56.88	2782.60	19.16	19.01	96.81	7.06	97.58	7.12	0.62	2.0
100.000	2141	71428.6	62.41	2.17	64.58	3389.58	22.06	21.45	102.38	7.46	105.35	7.68	0.63	2.0
110.000	2591	95071.4	70.11	2.60	72.71	4075.87	25.37	24.02	106.87	7.79	113.12	8.25	0.63	2.1
120.000	3084	123428.6	78.28	3.65	81.93	4848.70	29.16	27.32	110.34	8.04	118.31	8.63	0.63	2.1
130.000	3619	156928.6	86.94	4.71	91.65	5716.17	33.45	30.75	112.90	8.23	123.92	9.04	0.63	2.2
140.000	4197	196000.0	96.13	5.76	101.89	6683.36	38.28	34.28	114.51	8.35	129.96	9.48	0.63	2.3
150.000	4818	241071.4	105.90	6.82	112.72	7755.93	43.69	37.95	115.21	8.40	136.23	9.94	0.62	2.4
160.000	5482	292571.4	116.32	7.87	124.19	8939.87	49.72	41.75	115.02	8.39	142.74	10.48	0.61	2.5
170.000	6188	350928.6	127.44	12.16	139.60	10258.14	56.51	48.88	115.45	8.42	139.54	10.18	0.60	2.6
180.000	6938	416571.4	139.34	16.45	155.79	11734.32	64.24	56.05	114.93	8.38	138.69	10.11	0.59	2.8
190.000	7730	489928.6	152.14	20.73	172.87	13376.80	72.98	63.27	113.52	8.28	139.59	10 18	0.58	3.0
200.000	8565	571428.6	165.96	25.02	190.98	15195.10	82.75	70.62	111.40	8.13	141.72	10.33	0.56	3.2

The specific gravity of the masonry is $2\frac{1}{3}$.

dam of the pure arched type relies solely on the arched form for stability, in which case the pressure of the water is transmitted laterally to the abutments. If our knowledge of the laws governing masonry arches were more complete, the arched or curved dam would probably be the best type, since

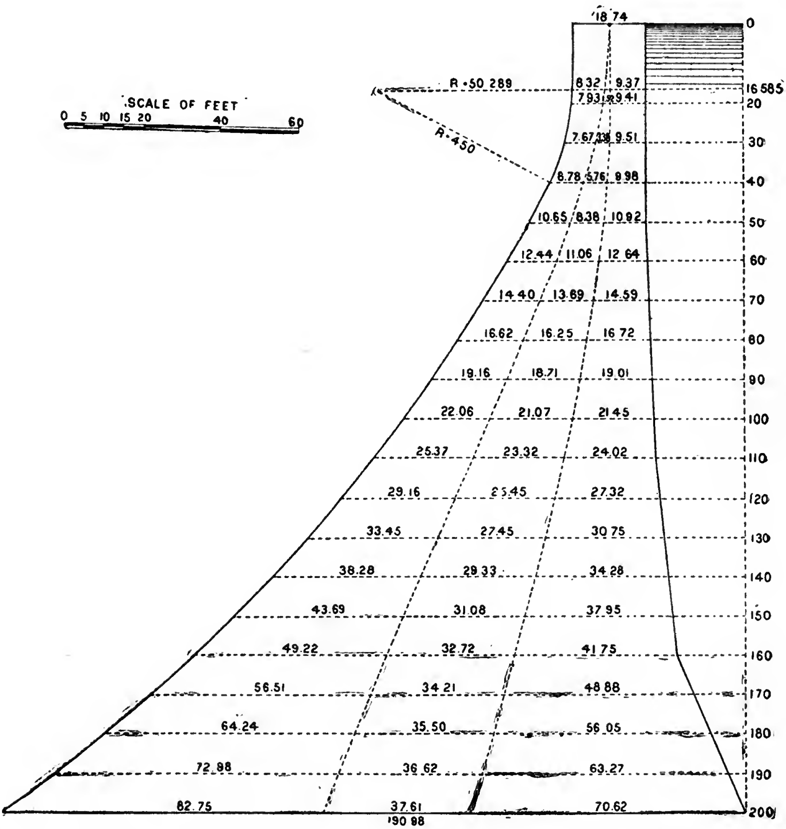

FIG. 75.—PRACTICAL PROFILE FROM WEGMANN.

it will contain the least amount of material. As it is, we know something of the laws governing such true masonry arches as those supporting bridges. In these the two extremities of the arch are raised at their springing on some firm abutment and the whole is keyed together at the centre; but in a masonry dam of arched form not only is the arch supposed to transmit the pressures laterally to the side of the abutments, but as the dam rests on the bottom of the valley it is sustained again at that point, so that it cannot act as a true arch, —nearly perfect arch action only occurring at the top, where the pressure is a minimum, while near the bottom, where the pressure is greatest, probably very little of this is transmitted to the abutments. For this reason it is not yet considered safe to build a dam depending purely on the arched form, and such few dams as have been constructed on this principle have been given somewhat of the gravity cross-section, increasing downward in width, so that they presumably resist the pressure both by gravity and arch action. The three best existing types of such works are the Zola dam in France and the Bear Valley and Sweetwater dams in California (Arts. 275 and 281).

That a masonry dam constructed across a narrow valley can resist the water pressure by transmitting it to its abutments is proved by the dams above cited. The question then arises, can the profile be reduced from what would be required if the plan were straight? As stated at the beginning of this chapter, Krantz asserts that a dam curved in plan and convexed up-stream with a radius 65 feet or less will transfer the pressure of the water to its abutments. Dams, however, of even greater radius than this do transfer the pressure to the abutments. The radius of the Zola dam is 158 feet and its length on top is 205 feet. The length of the Bear Valley dam, which depends almost wholly on its arched form for its stability, is 230 feet, the radius at the top being 335 feet and at the bottom 226 feet. The Sweetwater dam is 380 feet in length on top, its radius at the same point being 222 feet. M. Delocre says that a curved dam will act as an arch if its thickness does not exceed one third of the radius of its up-stream or convex side. M. Pelletreau fixes the limiting value of the thickness at one half of this radius. When a dam acts as an arch it only transmits the water pressure to the sides of the valley; its own weight must still be borne by the foundation.

256. Design of Curved Dams.—Mr. Wegmann gives the following formula for calculating the thrust in curved dams of circular plan:

$$t = pr,$$

in which $t =$ the uniform thrust in the circular rings of any plane of the masonry;

$p =$ the pressure per unit of length of this section of the ring;

$r =$ the radius of the rings of the outer surface.

Arch action can only take place by the elastic yield of the masonry; but little is known of the elasticity of brick, stone, etc., and nothing of the elasticity of masonry; hence it is impossible to determine the amount of the arch action.

It may be shown theoretically that in the case of a narrow valley a profile of less area may be employed for a dam which is curved in plan than one in which the plan is straight. An excellent theoretical discussion of this subject has been published by Messrs. Hubert Vischer and Luther Wagoner. The result of the investigations of these gentlemen goes to show that arch action, as usually understood, adds little to the strength of a curved dam. Notwithstanding this, the curved form may to a marked degree afford additional resistance, and this in a manner less dependent on the radius of the curve than the arched theory implies. The general conclusion reached by these gentlemen is, further, that the rate of efficiency of a curved dam over the straight decreases with the increased length of the dam; that very narrow cross-sections are not justifiable; and they ascribe the high duty of the Bear Valley dam to a favorable combination of conditions which could not have held good if the span had been considerably longer or the workmanship less excellent.

Engineers are now generally agreed upon the advantages of the curved plan. Its chief disadvantage is the increased length of the dam over a straight plan, and the consequent increase in the amount and cost of material to within certain limits of top length and radius. Though the cross-section of a curved dam should unquestionably be somewhat reduced, it would be unsafe to reduce it as much as has been done in the case of the Bear Valley and Zola dams, though these have withstood securely the pressures brought against them. It might with safety be reduced to the dimensions of the Sweetwater dam, thus saving largely in the amount of material employed. All of the more conservative writers, as Wegmann, Rankine, and Krantz, recommend that the design of the profile be made sufficiently strong to enable the wall to resist water pressure simply by its weight, and to curve the plan as an additional safeguard whenever the topography makes it advisable. American engineers, and especially those of the West, however, are prone to be more liberal; and the tendency is toward a slight reduction in the cross-section where a curved plan is practicable. An additional advantage of the arched form of dam is that the pressure of the water on the back of the arch is perpendicular to the up-stream face, and is decomposed into two components, one perpendicular to the span of the arch and the other parallel to it. The first is resisted by the gravity and arch stability, and the second thrusts the up-stream face into compression, which has a tendency to close all vertical cracks and to consolidate the masonry transversely.

An excellent manner in which to increase the efficiency of the arch action in a curved dam is that employed in the Sweetwater and Buchanan reservoir dams, the latter of which has recently been designed for construction in California. This consists in reducing the radius of curvature from the centre towards the abutments. The good effect of this is to widen the base or spring of the arch at the abutments, thus giving a broader bearing for the arch on the hillsides. In the Sweetwater dam the effect of this is seen in projections or rectangular offsets made on the down-stream face of the dam (Pl. XXIV), the centre of the dam sloping evenly, while the surface is broken by steps where it abuts against the hillside. In the Buchanan dam, the length of which is 780 feet on top, the maximum radius at the centre is 1146 feet, and this is diminished gradually to 736 feet at the abutments. These changes in the radii are made gradually, and are not shown in the surface of the dam in projections, as the entire outer surface is smoothed off evenly.

257. **Foundations.**—Masonry dams must be founded on solid rock, and great care and judgment are required in determining just when the excavation for the foundation has proceeded sufficiently far. If the looser and partially decomposed surface rock is not entirely removed there is danger of leakage under the dam, and consequent liability of its destruction. If the excavation is carried too far into the underlying rock much money may be wasted. Frequent cases might be cited where it has been found necessary to make unusually deep excavations in order that a sufficiently firm foundation might be reached. In the case of the Turlock dam the average depth of excavation in the large bowlders and underlying porphyry was from 5 to 10 feet to the homogeneous material. In one or two cases, however, seams full of huge bowlders weighing several hundred tons apiece were encountered, which necessitated excavation to a depth of 25 to 35 feet in order that they might be worked out and homogeneous rock reached. A masonry dam is an absolutely rigid structure, and the least unequal settlement in any portion of it tends to produce a crack. A clay or hardpan foundation is almost sure to yield under the weight of a masonry dam, and be the loose material ever so little in amount, if it offers opportunity for subsidence it will result in the rupture of the dam. The safe load on the lower courses of a masonry dam depends on the character of the material of which it is composed, and may reach from 10 to 15 tons per square foot, and nothing but the most substantial rock will bear such a weight as this.

258. **Material of which Constructed.—Ashlar Masonry.**—Reservoir dams may be built of cut masonry, of rubble or concrete with dressed-stone facing, or of random rubble. The first would be the best for the purpose on account of its strength, but while only twice as strong as rubble, it costs three or four times as much. As the form of the upper part

of the dam depends on the positions of the lines of pressure and not on the strain in the masonry, the great strength of cut-stone work would only avail in the lower portion of the dam. Great care would have to be employed in the use of cut masonry in order that it should not be laid in horizontal beds, which might permit of shearing or sliding, and in order that it should break joints with a proper degree of irregularity. Neither the vertical nor the horizontal joints in a dam should be continuous; therefore if made of cut or ashlar masonry or of square stone the joints should be carefully broken.

Rubble or concrete with cut-stone facing is not a desirable material of which to construct a dam, because of the difference in settling of the two kinds of masonry, which might result in the formation of cracks and seams. Where the facing becomes detached in this manner from the remainder of the body of the wall the strength of the structure is reduced to that of the uncoursed or concrete centre. The most prominent examples of the use of cut-stone facing with rubble or concrete interior are to be found in the Vir, Bhatgur, and Betwa dams of India, which are briefly described in Articles 270 and 277, and the new Croton dam in New York (Art. 271). In each of these the cut stone is laid as headers and stretchers, and the former are well bonded into the mass of the dam. The use of this form of construction is condemned by many Indian engineers, and is not approved in this country.

259. Concrete.—Some engineers consider concrete too pervious a material to be placed in a dam. It has, however, been successfully employed in four of the greatest dams yet constructed, namely, the San Mateo dam in California, 170 feet in height; the Periar dam in India, 155 feet high; and in the Geelong and Betaloo dams in Australia, respectively 60 and 110 feet in height (Articles 272–274). The Periar and Betaloo dams are two of the best examples of the homogeneous use of concrete. The great disadvantage in using this material, aside from engineering considerations, is the added cost of cement where the latter is expensive. The great advantage of the use of concrete and that which determined its employment in the Periar dam is the saving effected in labor; for concrete can be mixed and handled entirely by machinery worked by water-power furnished by the reservoir while under construction. In the Beetaloo dam for 46 feet above the foundation the concrete was made of one part Portland cement, two parts washed sand, and four parts broken stone of 2-inch gauge. In building the structure great care was taken to have the surface of the set concrete picked, washed, and brushed before a fresh layer was deposited, and the new concrete was kept shaded from the sun while setting. This dam was built up as a monolithic mass, the concrete being laid between boards or framing bolted in the body of the dam. After removal these boards left their imprint on the sides of the structure, which marking still remains.

In choosing concrete as the material to be employed in the construction of the Periar dam in India the engineer held that concrete is nothing more than uncoursed rubble reduced to its simplest form. As regards resistance to crushing or percolation, he holds that the value of the two materials is identical, unless it be considered as a point in favor of concrete that it must be solid, while rubble may, if the supervision be defective, contain void spaces not filled with mortar; he holds that the selection between the two depends entirely on their relative cost. The proportion of materials employed in this dam were: for every 100 cubic feet of concrete, 60 cubic feet of solid stone plus 10 per cent for wastage, 25 cubic feet of native hydraulic lime, and 30 cubic feet of sand.

The San Mateo dam in California was not built up as a monolithic mass of concrete as were those just described, but is composed of great concrete blocks of uniformly irregular dimensions. These blocks (Pl. XXIII) weigh 9 tons each, and were built up in the body of the dam in such manner as to key in with each other both in horizontal and vertical plan, so as to produce a nearly homogeneous mass and create the greatest amount of friction between blocks. The material was mixed at the site of the dam, and run out in a tramway and built in place inside of a wooden boxing which was afterwards removed. The blocks were left surrounded by the boxing for one week, during which time they set sufficiently for the wood to be removed and to permit of other blocks being built against them. The concrete consists of 2-inch-gauge metal mixed in the proportion of 6 of broken stone to 2 of sand and 1 of Portland cement.

In mixing concrete one of the best proportions to use, measured by volume, is 1 part of cement, 2 of clean sharp sand, and 3 to 4 of broken stone. This concrete should be laid immediately after mixing, and should be thoroughly rammed and compacted until the water flushes to the surface. It should be allowed to stay for 12 hours or more before any further work is laid upon it.

260. Rubble Masonry.—Rough random rubble masonry is considered the best material that can be used for building a dam. It possesses strength, can be readily adapted to any form of profile, and is relatively cheap. In building a dam the main object is to form as nearly homogeneous a monolithic mass as possible. Horizontal and vertical courses must therefore be avoided, and the stones interlocked in all directions. The sizes of these stones may differ greatly. The mass of the wall may be composed of stones of such a size as may be carried between two men, as is the case in India, where machinery is rarely employed; or it may consist of cyclopean rubble measuring from one to several cubic yards in volume, each block perhaps weighing several tons. To prevent leakage, all spaces between the stones must be completely and compactly filled with impervious mortar or cement. To prevent sliding, the blocks must be irregularly bedded, and as each course is laid a large proportion of the stones must be permitted to project above the general surface. The spaces between the larger stones may be filled with concrete or small rubble. Grouting must never be permitted, and the best stones are generally reserved for the facing, in which they are laid as headers in such manner as to give an even contour to the outer surface.

261. Cement.—The center of a large work may be of some cheaper variety of cement, as Rosendale or other natural or American cement. Portland cement should be used in the facing stones and in pointing. All cement used should be hydraulic and of some well-known brand, whether natural or Portland. The cement should be carefully enclosed in a tight shed with a close floor set above the ground to protect it against dampness, and should be subjected to strict inspection and tests. All mortar used should be prepared from the best quality of cement of the kind above described, and of clean sharp river sand well washed and free from dirt. They should be mixed dry in the proper proportions, and then a moderate amount of water should be added and the whole thoroughly worked together. Portland cement and mortar should generally be mixed in the proportion of about 1 of cement to 2 of sand in laying the puddle work; while for laying the rubble work and concrete 1 of cement to 3 of sand may be used. In laying masonry great care should be taken that water shall not interfere, and in no case should it be laid in water. No masonry should be built in the winter time during freezing weather, unless exceptional precautions be taken to cover it and protect it from frost.

262. Details of Construction.—Rubble stone masonry should always be made of sound clean stone, of suitable size, quality and shape for the work. All awkward projections should be hammered off so that the stones shall become rectangular in form. Their beds should present such even surfaces that when the stones are lowered on the surface prepared to receive them there can be no doubt that the mortar will fill all spaces. The stones should be well rammed into the bed of mortar if they are light, and this should be at least one inch in thickness. Where large stones are employed a moderate quantity of spawls may be used in the preparation of suitable surfaces for receiving them. Especial care must be taken to have beds and joints full of water, as no grouting or filling of joints should be allowed after the stones are placed. The work must be thoroughly bonded, and if mortar joints are not full and flush they should be taken out to a depth of several inches and properly repointed. In such work various sizes of stones should be employed, and regular coursing should be avoided in order to obtain both vertical and horizontal bonding. The sizes of the stones may vary with the character of the quarry, but where the thickness of the masonry is great a considerable proportion of large stones should be used. Where exceptionally large stones are employed the joints may be filled with concrete instead of mortar. In such cases only so much water should be employed as can be brought to the surface by ramming.

In carrying out the construction of rubble-masonry work it should not be built in horizontal courses; at the same time it must be built in beds, and these should be irregularly stepped, and various parts of the structure worked upon and allowed to set at different times. The surface of these horizontal steps or courses should bristle with projecting stones, so as to secure a perfect bond in every direction. This is done by working up the mortar or concrete between the stones to about half their height, and wherever the work is stopped over night or for a period of time these projections insure bond with the next layer to be worked. No stones should be deposited or dressed upon the wall, but on platforms or planking, so that no dirt shall be brought in contact with the material. The same precaution must be taken in handling concrete and mortar.

The rubble facing stones should be of large size, not less than 2 feet deep, with frequent headers. Where especial jar is brought on the masonry work, as in overfall weirs, facing stones should be of range rubble, of the soundest and most durable quality, and should be cut so true that joints not exceeding $\frac{1}{2}$ inch shall be necessary for 3 inches from the surface, the remainder of the joint not exceeding 2 inches in thickness at any point. In such work it is well to alternate about two stretchers for one header, and to make the former not less than 3 feet in length, while the header should not have less than 12 inches lap under ordinary circumstances.

The concrete used in work of this character should be made of rough broken stone metal, and of clean river gravel not exceeding from 2 to 2½ inches gauge. This material should be washed free of dirt before being used, and be mixed in boxes or mortar mixers with mortar of a proper quality. The proportions used in mixing differ greatly, and are described in technical books treating on this subject.

263. Submerged Dams.—In a few instances submerged dams have been constructed for the purpose of stopping the underground or underflow water in the beds of streams. This

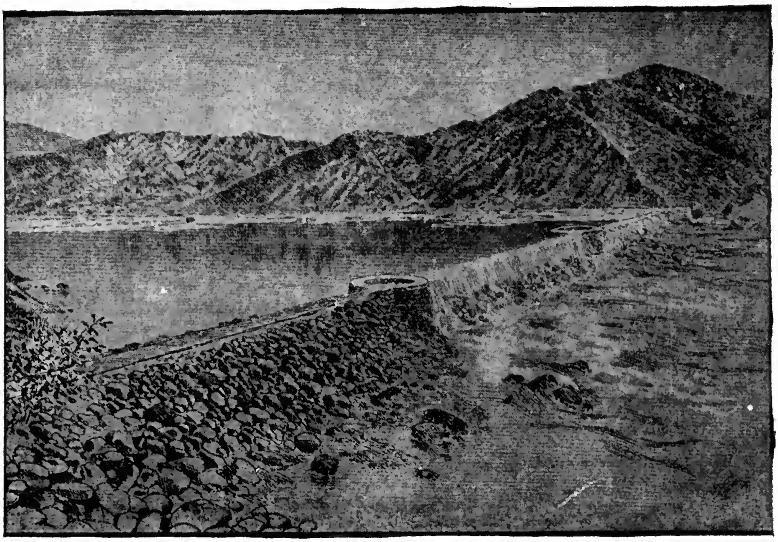

FIG. 76.—VIEW OF SAN FERNANDO SUBMERGED DAM.

has been resorted to particularly in a few streams in the mountains of Colorado and California, where the surface flow is large, but as the streams reach the plains the water sinks and disappears. Its downward course then is stopped by some impervious bed of clay or rock, and there is created practically a slow-moving river under a bed of deep gravel. This can be brought to the surface by sinking a dam entirely across the stream bed to the impervious substratum, when the water will be raised, forming an underground reservoir; or a series of cribs may be built on the impervious stratum under the gravels, and these will catch the water and lead it off, whence it may be removed by an open cut or by pumping (Art. 295).

The former method is employed on the San Fernando Land and Water Company's property on Pacoima creek in California. At the site of the dam the canyon walls are about 800 feet apart and the bed-rock about 75 feet below the gravel surface of the stream. Through this a trench was excavated, and in this a masonry wall was built up, its bed width being about 3 feet and its top width 2 feet, its greatest depth being 53 feet and rising to a height of from 2 to 3 feet above the stream bed (Fig. 76). On the line of this wall are two large wells, and on its upper face pipes are laid in open sections, so that the seepage water caught by the dam might enter these and be led through them into the wells, from which it is drawn off for purposes of irrigation.

264. Construction in Flowing Streams.—In building any variety of dam across a flowing stream the expense of construction is considerably increased by the necessity of handling the flowing water and keeping it away from the work of construction. Several methods are pursued, depending largely upon the discharge of the stream. If this is small, one of the simplest methods is to build an under or scouring sluice in the dam and construct this portion of the work first, so that the water may be permitted to flow off through it while the remainder of the work is being built. If the stream is subject to violent floods or its discharge is too large to be conveniently handled in this manner, wasteways at varying heights may be left in the crest of the dam over which the floods may fall. It is frequently necessary to build a temporary dam above the main structure with a view to retaining the water until the latter is completed; or a temporary channel may be built for the stream around the dam, and through this the water may be carried off. In the great Tansa and Bhatgur dams in India, where the floods discharged are very large, a portion of the masonry adjacent to either abutment was maintained at a lower height than the rest in order that the floods might flow over it as over a wasteway.

In commencing the construction of a dam where flowing water has to be controlled, if the discharge is not too great the stream may be diverted temporarily while the main portion of the dam is being built; or if undersluices are to be provided for the discharge of the water, these should be built first, the stream being passed to one side during their construction, after which it may be turned back through them, and the remainder of the structure carried up. If no undersluices are to be constructed, pumping may be resorted to if a temporary channel cannot be provided, though this method is not advisable and should rarely be resorted to. In founding a dam in quicksand two or three methods may be employed. Pneumatic caissons may be sunk, and the foundation built in these as would be done for a bridge pier; or if the sand is comparatively dry and semi-fluid, it may be frozen by the Poetsch process, and the excavation for the foundation can then be made within the frozen walls.

265. Specifications and Contracts.—There are many trivial details of construction which must be considered by the engineer in designing earth, crib, and masonry dams. It is customary to have such structures built by contract, and for this purpose careful specifications are drawn up by the engineer, detailing the character of material and construction. For those who are unfamiliar with such forms of specifications, such books on the subject of specifications and contracts as those of Gould and Haupt can be purchased; or specifications which have been used by other engineers can be obtained through them.

The usual form of specification opens with a general description of the work and its location, a statement of the methods and appliances to be used in construction, a description of the protective work, highways, bridges, and diverting works, as well as pumping plant and other temporary work to be employed during construction. For earth dams the specifications then go into a description of the soil to be used, and where it is to be obtained; the depth of excavation and its character, and the method of retaining it; a description of the refilling of excavations and the building of embankments; and the question of sodding and paving or revetting the embankments.

If the dam is to be of timber or loose rock, a description of the timberwork and cribwork is given, and the character of the rock excavation and explosives to be employed is entered into. If of masonry, the matter of excavation for foundation, measurement and disposal of the material removed, and method of stepping the foundation are first considered. Then the hydraulic masonry is described, the cement and its tests, the proportions used in mixing mortar and concrete, the character of the brickwork and of the stone masonry, whether of dry rubble, rubble masonry, range-rubble facing, or cut-stone. In addition to these there is usually some iron work connected with the superstructure and gate-houses.

266. Examples of Masonry Dams.—In Table XI on page 222 were given the general dimensions of several of the largest masonry dams which have been built. An account of the construction of masonry dams would be incomplete without a few examples of the larger and more typical of the modern dams, and accordingly brief descriptions and illustrations of some of these are given here. These are divided for convenience into two general classes: 1, those which act as retaining walls for the water and over which the latter is not expected to flow; and 2, those which act both as retaining walls and overflow weirs. The older and less typical forms of dams, such as those built in Spain in earlier days, and a few of those built in France and elsewhere, do not require description here, as no such works are likely to be designed in the future. For those who are interested in their study, descriptions and cross-sections of these can be found either in Wegmann's "Design and Construction of Masonry Dams," Krantz's "Reservoir Walls," or in the 12th and 13th Annual Reports of the U. S. Geological Survey.

267. Furens Dam, France.—This is one of the largest and first of the great dams built according to modern formulas (Fig. 77). It is 170.6 feet in maximum height above bed-rock, the maximum depth of water being 164 feet; its thickness at top 9.9 feet, and at the base 161 feet. The maximum pressure on the masonry is 6.82 tons per square foot, while its

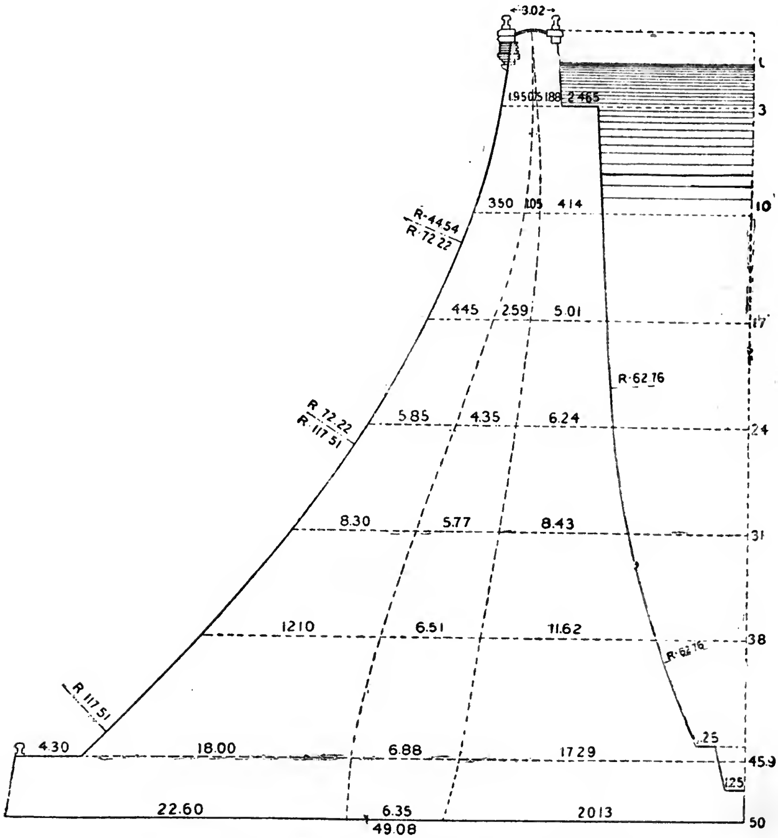

FIG. 77.—CROSS-SECTION OF FURENS DAM, FRANCE.

total length is 328 feet on top. In plan it is curved with a radius of 828.4 feet, and it is built entirely of rubble masonry, the facings being of the same material. The top of the dam is finished off as a roadway 9.8 feet wide, and this is protected by two parapets, one on either side, each 1.6 feet in height.

268. Gran Cheurfas Dam, Algiers.—This dam (Fig. 78) was built in 1882, and has a total height above its foundation of 98.4 feet. Its width at top is 13.1 feet, at the base 72.2 feet, and its top length is 508.4 feet. It is built practically in two parts, the first consisting of a trapezoidal-shaped foundation

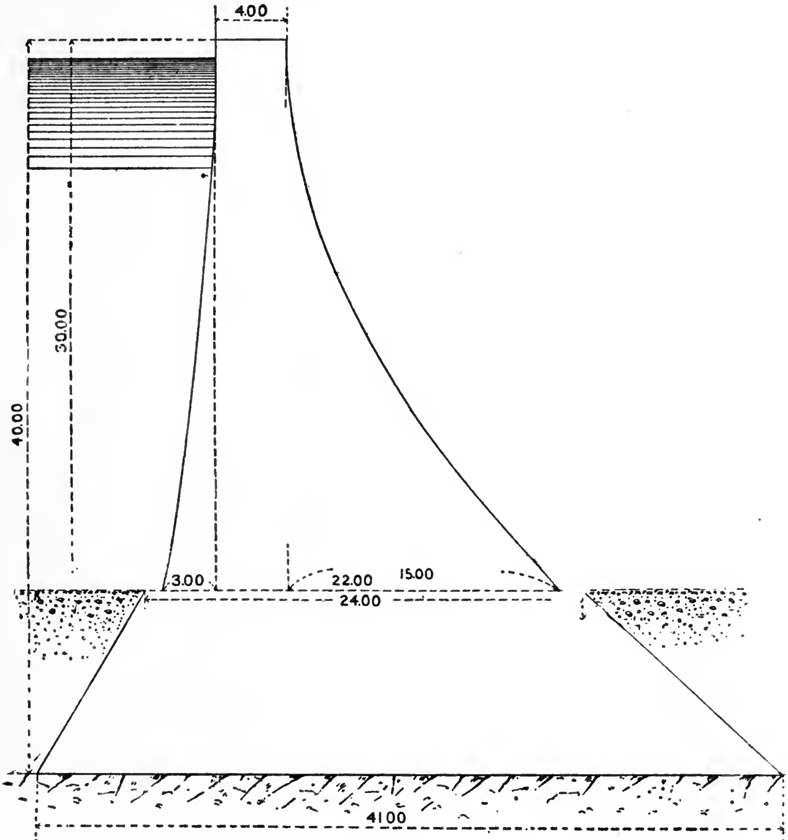

FIG. 78.—CROSS-SECTION OF GRAN CHEURFAS DAM, ALGIERS.

mass of rubble, on which is built the dam, the upper and lower surfaces of which are parabolic. The depth of water which this dam will hold is 132.2 feet, and the maximum pressure on the masonry within it is 6.14 tons per square foot. In plan it is straight.

269. Tansa Dam, India.—This great dam is built throughout of uncoursed rubble masonry. It is designed to have a total height of 133 feet, though it has as yet been completed only to a height of 118 feet (Fig. 79). At this height its maximum top width is 15.2 feet, while its maximum width at

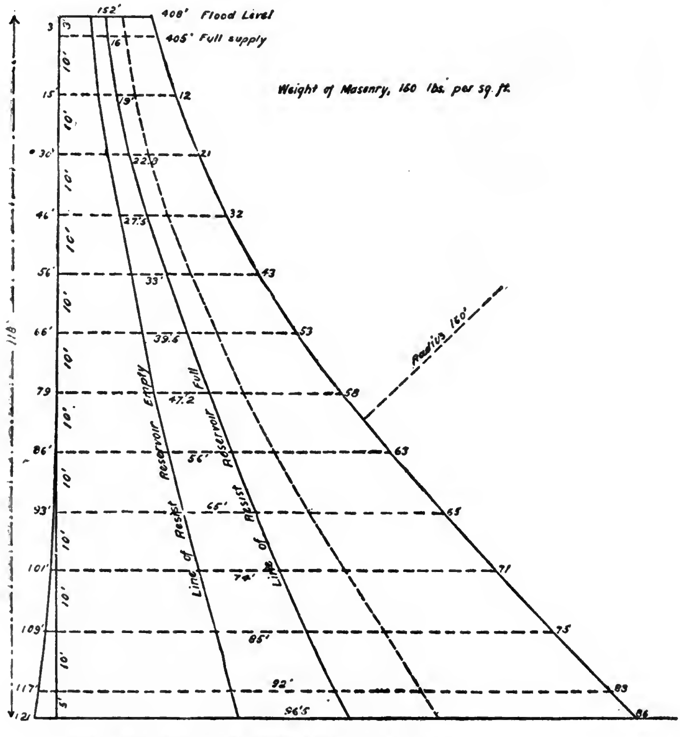

FIG. 79.—CROSS-SECTION OF TANSA DAM, INDIA.

base is 96.5 feet. Its total length on top is 9350 feet, while in plan it is built in two tangents, the apex pointing up-stream. Near the south end is built a wasteway 1800 feet in length, its crest being 3 feet below that of the dam. This wasteway is built in a portion of the dam where its height is but a few feet, and it discharges back directly into the river channel below the toe of the structure. Near the base of the dam

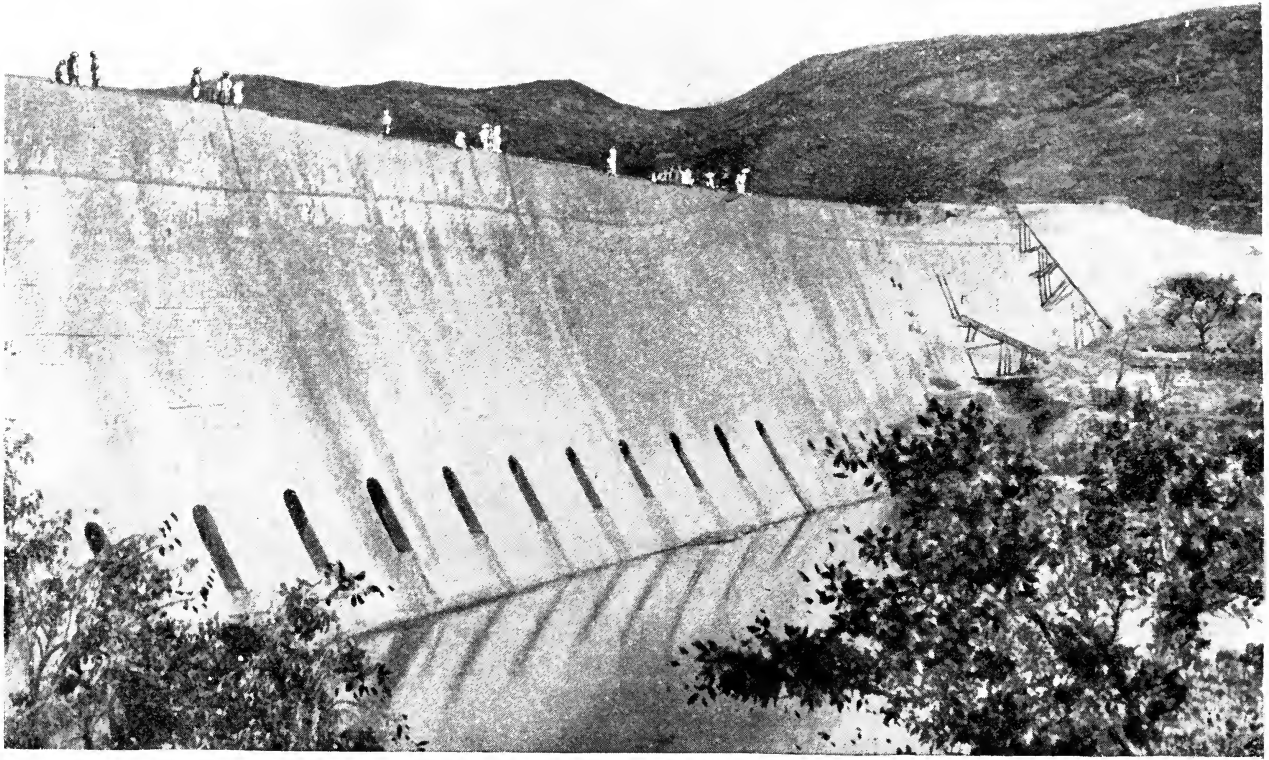

PLATE XXII.—VIEW OF BHATGUR DAM, INDIA.

is a large outlet tunnel, which discharges into the conduit which carries the water to Bombay for the supply of that city.

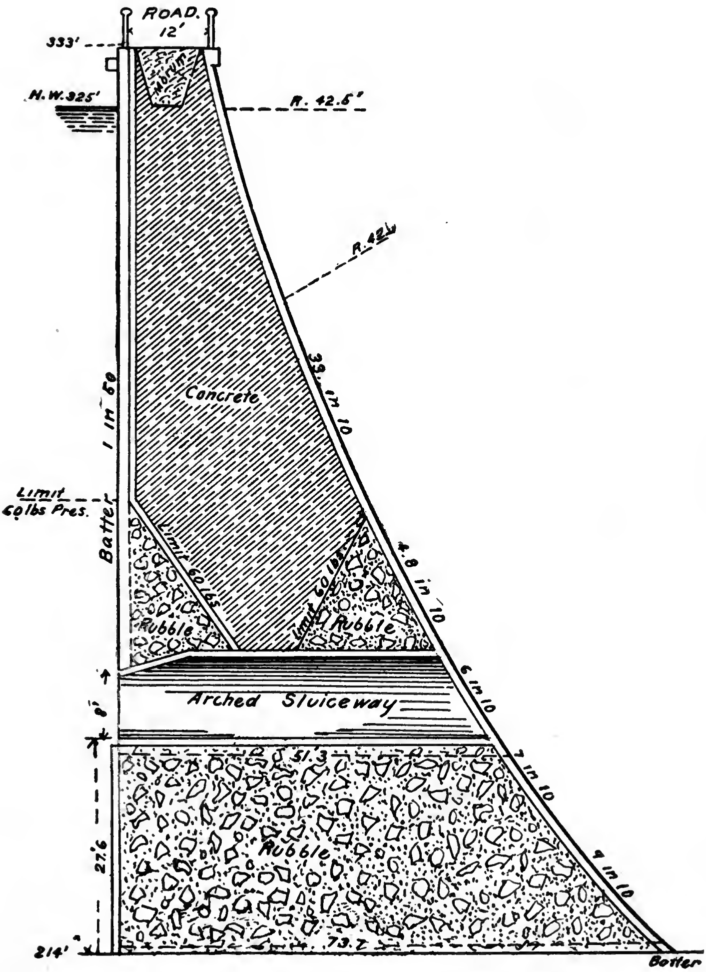

FIG. 80.—CROSS-SECTION OF BHATGUR DAM, INDIA.

270. Bhatgur Dam, India.—This dam (Pl. XXII) is 4067 feet in length, and is constructed throughout of the best uncoursed rubble masonry in cement. On the faces the dressed rubble is laid up in courses. It is 127 feet in height, 74 feet in width at the base, and 12 feet wide on top (Fig. 80). When full the pressure on the lower toe is 5.8 tons per square foot, and when empty the pressure at the upper toe is 6.7 tons per square foot. In plan the dam curves irregularly across the valley, following an outcrop of rock. Portions of either end of the dam, where it is not high, are left 8 feet lower than the remainder so as to act as wasteways. The total length of these wasteways is 810 feet, and they are arched over in such manner as to leave a roadway across their tops. Below the dam and jutting from it are masonry walls which lead the waste water off in such manner that it flows clear of the foot of the dam and passes off through separate channels to the main stream below. For the purpose of scouring silt which may be deposited in the reservoir, fifteen undersluices are constructed near the centre of the dam, at its deepest part. These are placed 17 feet apart and are 4 by 8 feet in dimensions, their sills being 60 feet below high-water mark. Above these are two other undersluices for discharging the water to be used in irrigation when the reservoir is full. One of these is 20 feet and the other 50 feet above the main row of undersluices.

271. New Croton Dam, New York.—This monster dam will be of composite construction. For about 530 feet from the left bank it will be of earth. The next 630 feet of its length will consist of a high masonry dam designed on a theoretic profile. Thence to the left bank the structure will consist of a masonry overfall weir of heavy cross-section and 1020 feet in length on the crest. The capacity of the reservoir will be 92,000 acre-feet.

The earth dam will be 245 feet in extreme height above its foundation and 120 feet above the ground surface (Fig. 81). Its top width will be 30 feet and will be 20 feet above high-water. Through its centre will be built upon a rock foundation a masonry core-wall 18 feet wide at the base and sloping on both faces to a top width of 6 feet at a level with high-water. The upper or water face will have a slope of 1 on 2, and will be paved with from 1½ to 2 feet of cobbles laid on 1 to 1½ feet of broken stone. The lower slope will be 1 on

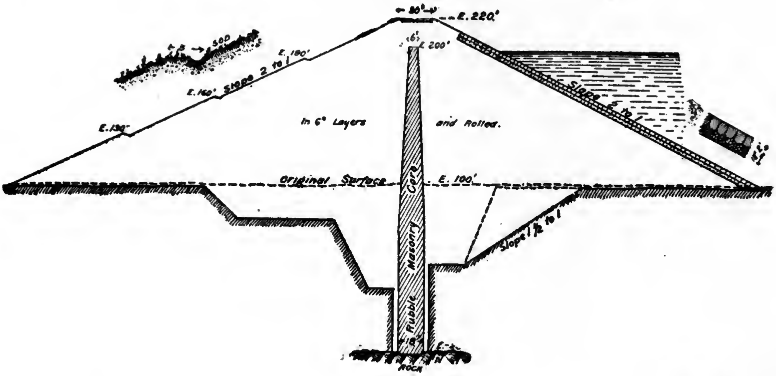

FIG. 81.—CROSS-SECTION OF EARTH EMBANKMENT. NEW CROTON DAM, CORNELL'S.

2, and will be broken by three benches each 5 feet wide and paved to make a gutter to catch drainage. This slope will be carefully sodded.

The main dam will be connected with the earth dam by heavy masonry wing walls and the masonry core wall. It will have an extreme height of 248 feet above its foundation and will be 163 feet in height above the river bed. The high-water level or crest of the overfall weir will be 14 feet below the crest of the dam. Its extreme width at base will be 185 feet and at its top 18 feet, surmounted by a 4-foot coping. This structure will be built throughout of the best rubble-stone masonry, faced above the ground surface with coursed stones set in Portland cement.

In plan the earth and masonry section will be straight to the masonry overfall weir, which will curve up-stream nearly at right angles to the main structure. The water falling over this weir will spill into an artificial channel excavated in the hillside and emptying into the main channel below the toe of the dam. The extreme height of the weir will be 150 feet and its extreme width at base 195 feet. It will have a very slight

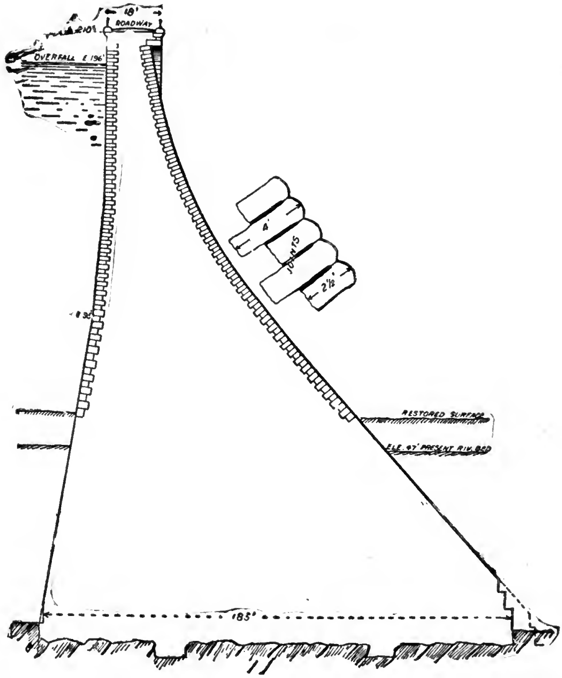

FIG. 82.—CROSS-SECTION OF MASONRY DAM. NEW CROTON DAM. CORNELLS.

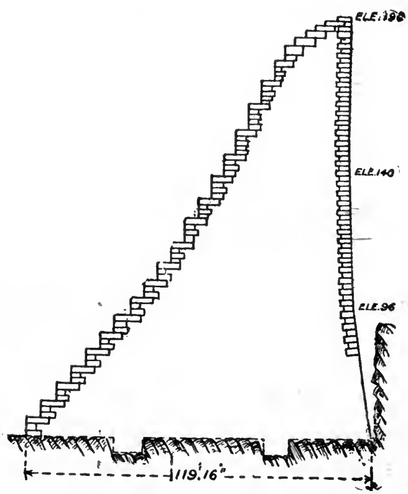

FIG. 83.—CROSS-SECTION OF OVERFALL WEIR. NEW CROTON DAM, CORNELL'S.

batter on the up-stream side, while its lower side will have a slightly ogee-shaped curve and will be broken by 25 steps varying from 2 to 10 feet in height. This weir will be constructed, like the dam, of an uncoursed rubble masonry interior and coursed faces.

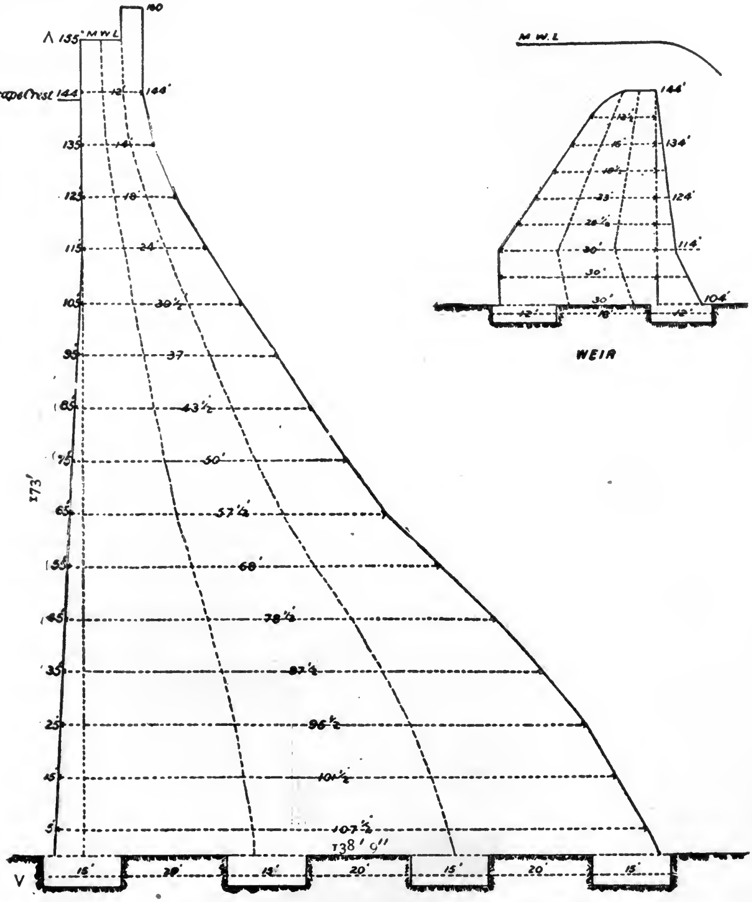

FIG. 84.—CROSS-SECTIONS OF PERIAR DAM AND WASTE WEIR, INDIA.

272. Periar Dam, India.—This dam, which is constructed throughout of concrete, is 1230 feet long on top. It has a maximum height (Fig. 84) of 173 feet (the numbers on the illustration being incorrect as they were taken from a preliminary

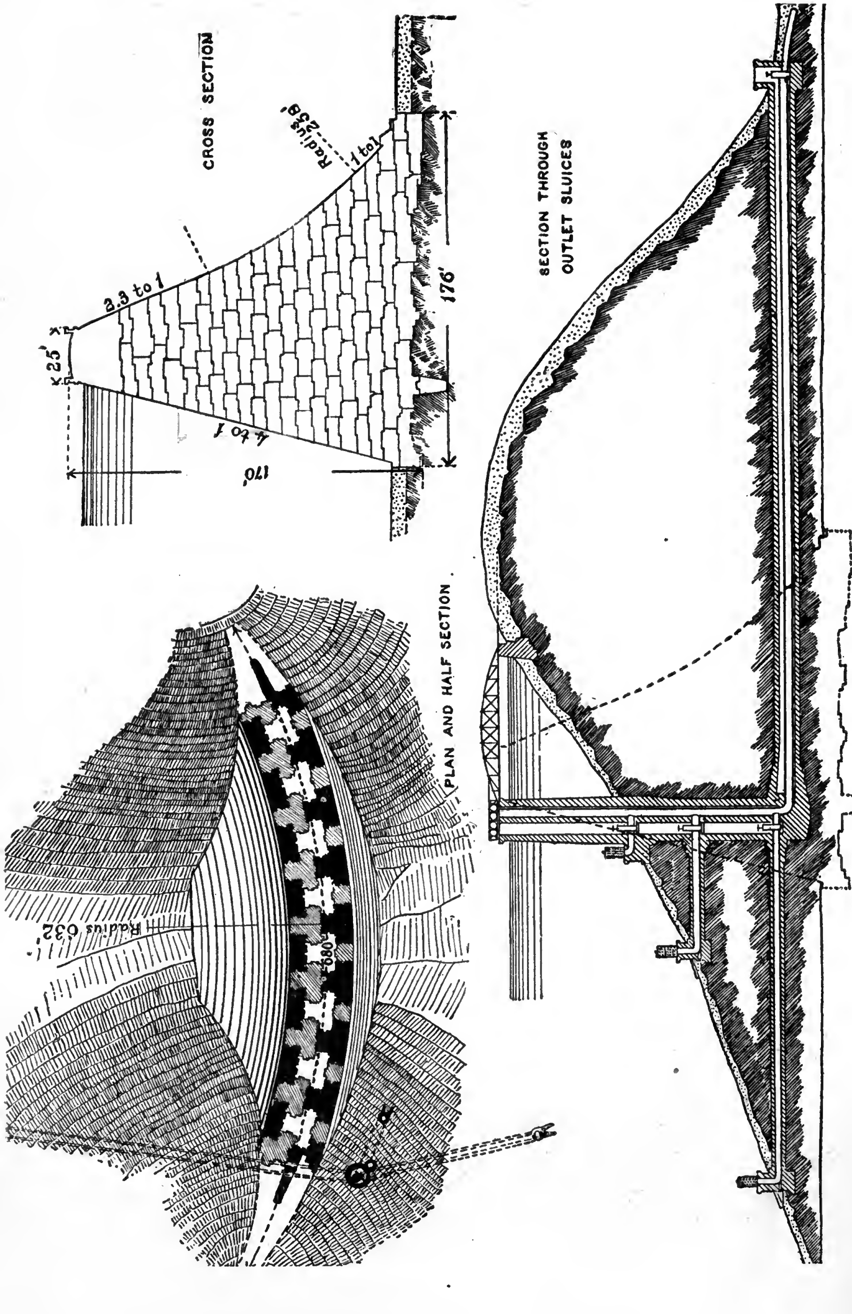

PLATE XXIII.—SAN MATEO DAM. PLAN, CROSS-SECTION AND OUTLET SLUICES.

design for the dam). Its crest is surmounted by a parapet 5 feet in height, the maximum depth of water which the dam will hold being 160 feet, and its width at base 138 feet 9 inches, its top width being 12 feet. At either end are two wasteways built in solid rock, forming the abutments of the dam and separated from it, their aggregate length being 920 feet. The maximum capacity of the reservoir will be 306,000 acre-feet, its available capacity being 157,000 acre-feet.

273. Beetaloo Dam, South Australia.—This structure (Fig. 85) is 110 feet in maximum height, 110 feet wide at the base, and 14 feet wide on top. Its length on top is 580 feet, and it is curved in plan, the convex side facing up-stream. It is

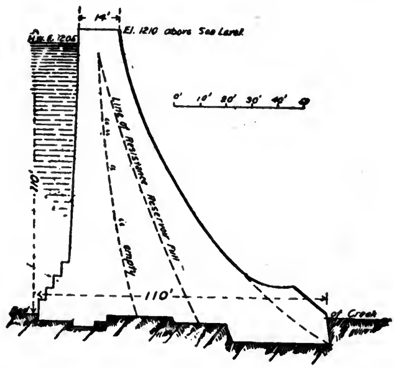

FIG. 85.—CROSS-SECTION OF BEETALOO DAM, AUSTRALIA.

constructed throughout of concrete, and in one end of the dam is built a set of three wasteways, their total length being 200 feet with their crests 5 feet below that of the main structure. These wasteways are separated by masonry walls, which lead the flood waters back into the river below and clear of the structure.

274. San Mateo Dam, California.—This structure is built throughout of concrete, not as a monolithic mass, as is the case with the Beetaloo and Periar dams, but as described in Article 259, it was built up in blocks set in place, the weight of each being about 9 tons. In cross-section this structure is heavier than theory alone would require. As shown in Pl. XXIII, its maximum height is 170 feet, its crest being 5 feet above high-water mark, at which level is a wasteway built a

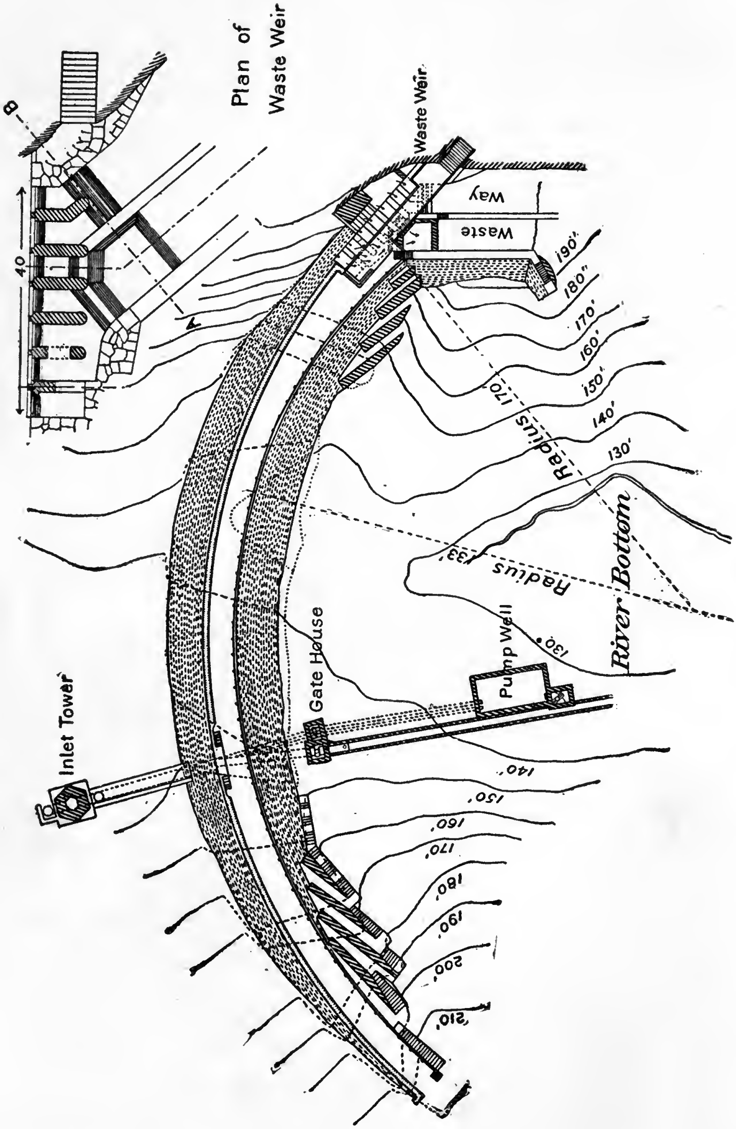

PLATE XXIV.—PLAN OF SWEETWATER DAM.

short distance above the north end of the dam and separated from it by a low ridge. The top width of the dam is 25 feet and its width at the bottom is 176 feet. Its upper slope has a uniform batter of 4 on 1, while the lower slope, beginning with a batter of $2\frac{1}{3}$ on 1 at the top, curves to within a few feet of the bottom, where the batter becomes 1 on 1. In plan this structure is curved up-stream.

275. **Sweetwater Dam, California.**—This dam (Pl. XXVI) is slighter in cross-section than theory would require, and depends to a certain extent on its curved plan for its stability. As shown in Plates XXIV and XXV, it is 90 feet in maximum height, 380 feet long, 12 feet wide on top and 46 feet wide at the base. The radius of its curvature is 222 feet, and as the length of the radius is small and the curvature great, this adds considerably to its stability. The structure is built throughout of large uncoursed rubble masonry, the greatest care having been used in every detail of construction. At its southern end are a set of seven escape-ways 40 feet in aggregate width, so arranged that the water issuing through them drops first into a series of water cushions, and is then led off by a directing wall so as to clear the dam. Near its base is a discharge sluice, operated from a water tower in the reservoir.

276. **Vyrnwy Dam, Wales.**—This structure is peculiar in cross-section (Fig. 86), being unusually heavy, and much greater than theory would demand. The reason for this is that the crest of the whole dam acts as a waste weir, which is surmounted by arches on which rests a roadway, and beneath these arches the waste waters are permitted to flow. Its lower face is given an ogee-shaped curve so as to reduce to a minimum the shock of the falling water, and there is a depth of 45 feet of back-water on its toe, which forms a sort of water cushion. Its maximum height is 136 feet, while the greatest depth of water is 129 feet. Its width at base is 117.7 feet, and the upper curved portion rests on a massive pedestal nearly rectangular in cross-section and 43 feet in height. This dam is straight in plan, its total length on top being 1350 feet, and it is built

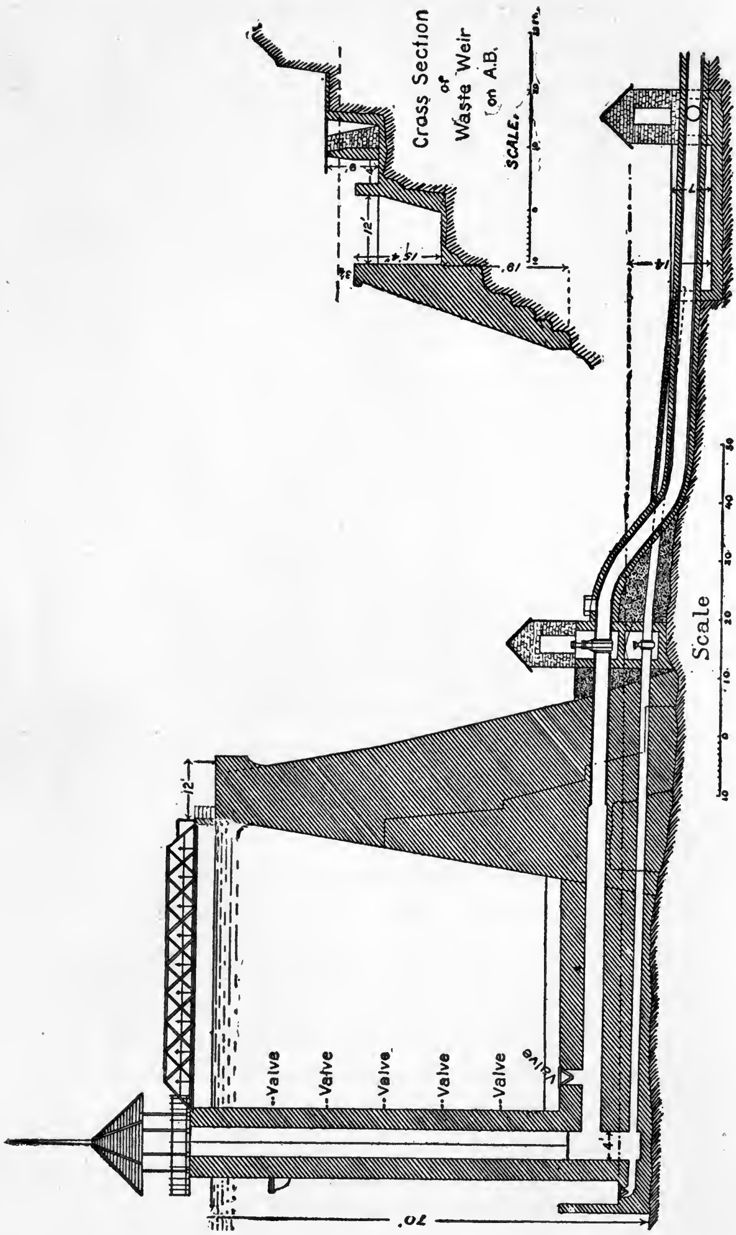

PLATE XXV.—CROSS-SECTION OF SWEETWATER DAM.

throughout of large cyclopean rubble, the stones weighing from 2 to 8 tons apiece.

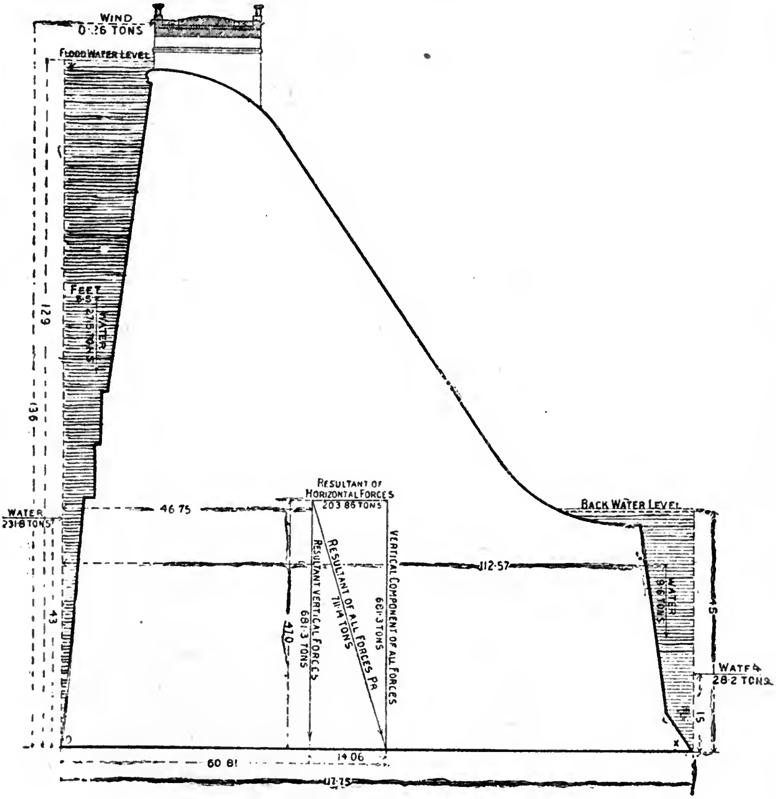

FIG. 86.—CROSS SECTION OF VYRNWY DAM, WALES.

277. Betwa Dam, India.—This structure, which has an unusually heavy cross-section (Fig. 87), performs the functions of a weir, the flood waters passing over the entire crest to an extreme depth of $6\frac{1}{2}$ feet. In plan it is built in three tangents, following the line of an outcrop of rock. Its total length is 3296 feet, its top width being 15.2 feet, and its maximum height about 64 feet. The down-stream face of this weir is supported by a buttress or block of masonry 15 feet in width

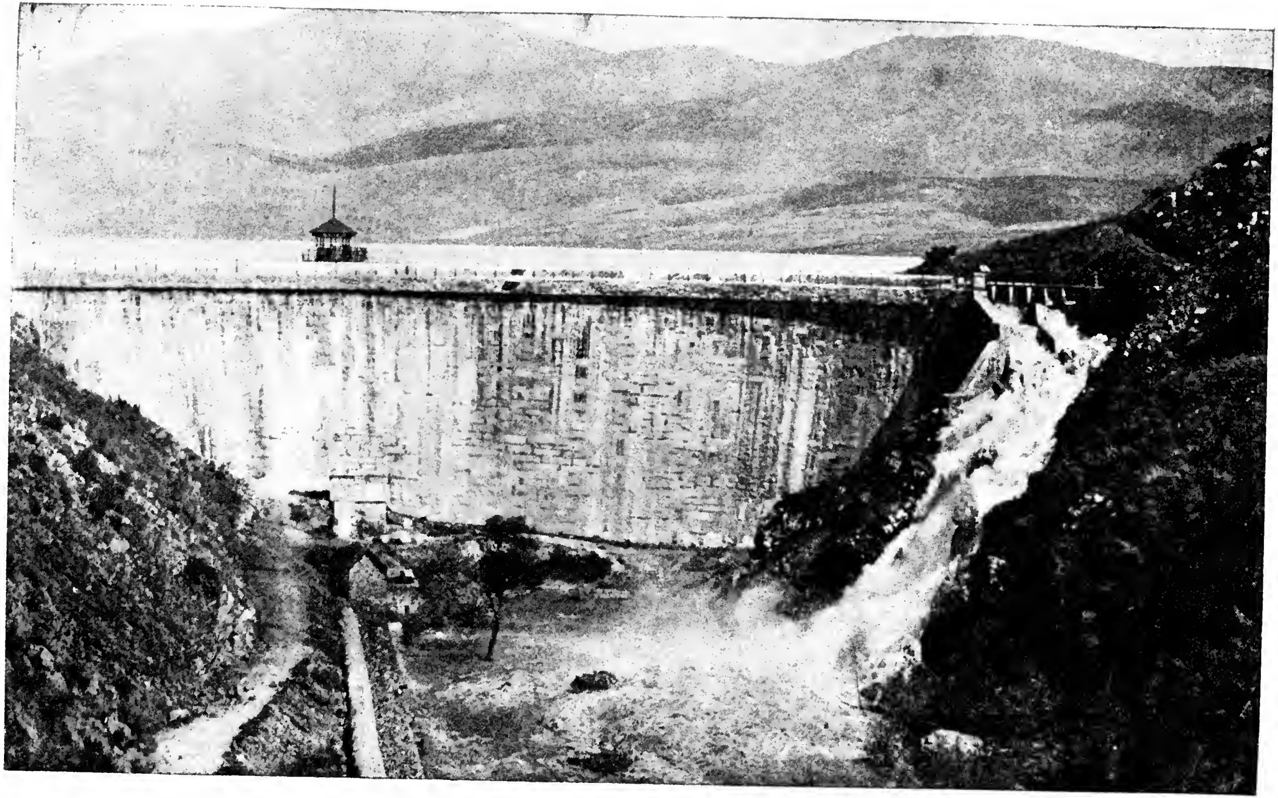

PLATE XXVI.—VIEW OF SWEETWATER DAM.

and 20 feet in height, while above it the back-water in the river rises to an additional height of about 10 feet, so that the flood waters will fall on a water cushion of this depth and then on the solid buttress. This structure is built throughout of uncoursed rubble masonry, its faces, however, being coursed with dimension stone and the coping being of ashlar. In the river some distance below its highest portion is built a subsidiary

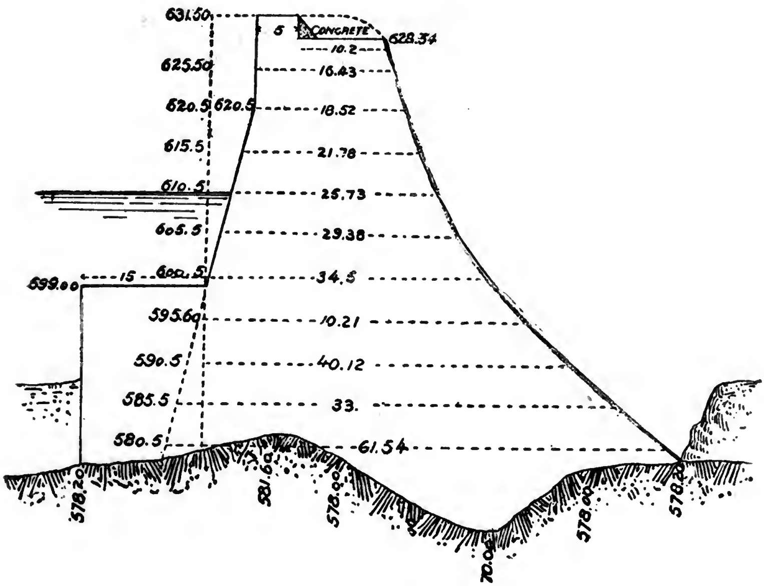

FIG. 87.—CROSS-SECTION OF BETWA DAM, INDIA.

or smaller weir, which backs the water up against the toe of the main weir in such manner as to form the water-cushion on which the floods may fall. The extreme height of this subsidiary weir is 18 feet, and the height of overfall from the main weir to the surface of the water cushion is $21\frac{1}{2}$ feet, though in time of greatest flood this will be reduced to 8 feet. The top width of the subsidiary weir is 12 feet, and its walls are nearly vertical on the down-stream side, with a slope of 10 to 1 on the up-stream side.

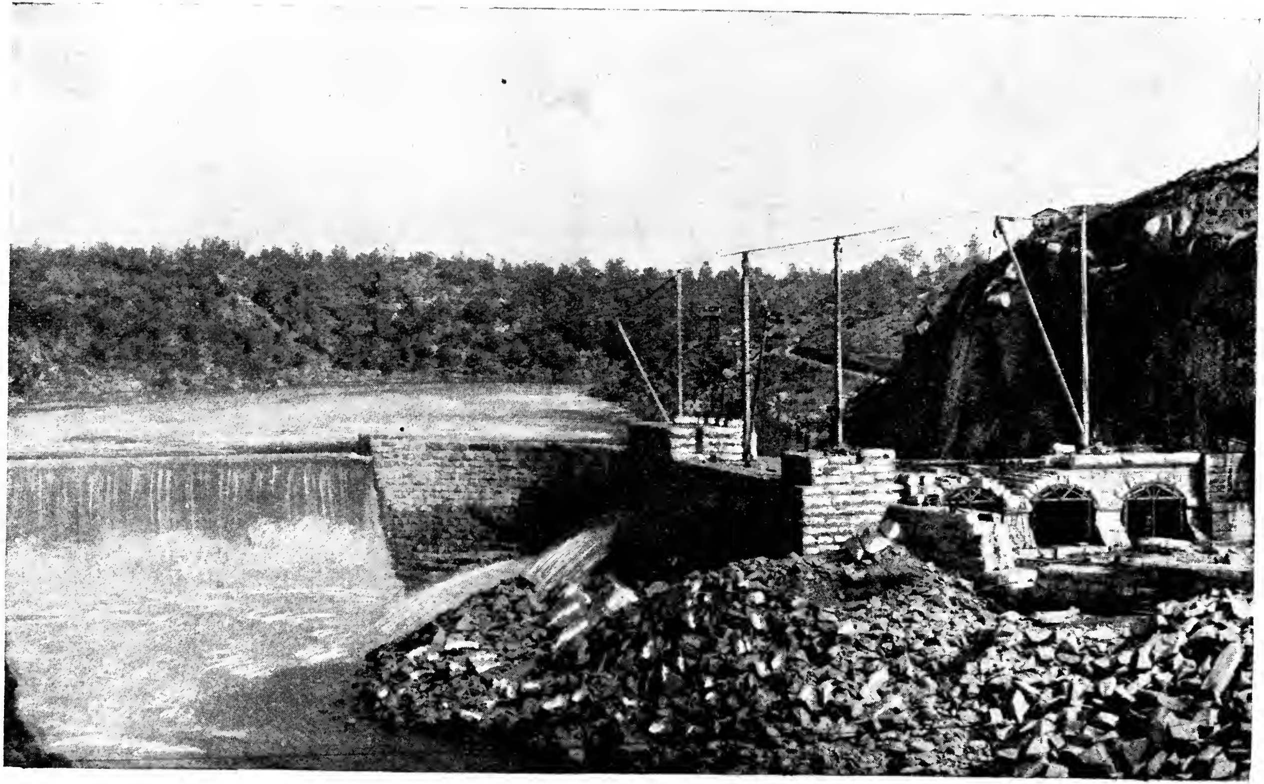

Plate XXVII.—Folsom Canal, View of Weir and Regulator.

278. Turlock Dam, California.—This structure (Fig. 88) is a little heavier in cross-section than theory alone would demand, as it is expected that the flood waters of the Tuolumne river will pass over its entire crest to a possible maximum depth of 16 feet. About 200 feet below the main dam is built a subsidiary weir 20 feet in height and 120 feet in length, its top width being 12 feet. This weir will back the water up against the toe of the main weir to a depth of 15

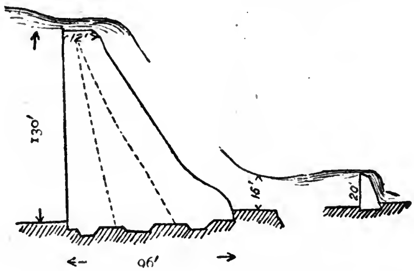

FIG. 88.—CROSS-SECTION OF TURLOCK DAM.

feet, thus giving a water cushion on which the floods may fall. The main weir is straight in plan, 310 feet in length on top, 96 feet in width at the base, 20 in width on top, and 130 feet in maximum height, and is built throughout of uncoursed rubble masonry. There is no escape-way, while there are a couple of undersluices which served to pass water during construction.

279. Folsom Dam, California.—This structure (Pl. XXVII), like that just described, acts only as a diversion weir. It is 69½ feet in maximum height on the up-stream side, and 98 feet in height on the down-stream side. Its cross-section is unusually heavy, as flood waters to a depth of over 30 feet are expected to flow over its crest (Pl. XXVIII). Its top width is 24 feet and its extreme width at base 87 feet, the toe terminating in a heavy buttress of masonry. Its total length on the crest is about 520 feet, a large portion of which consists of a retaining wall leading to the canal entrance. One hundred and eighty feet in length in the centre of the main dam is lowered a depth of 6 feet to form a wasteway over which the

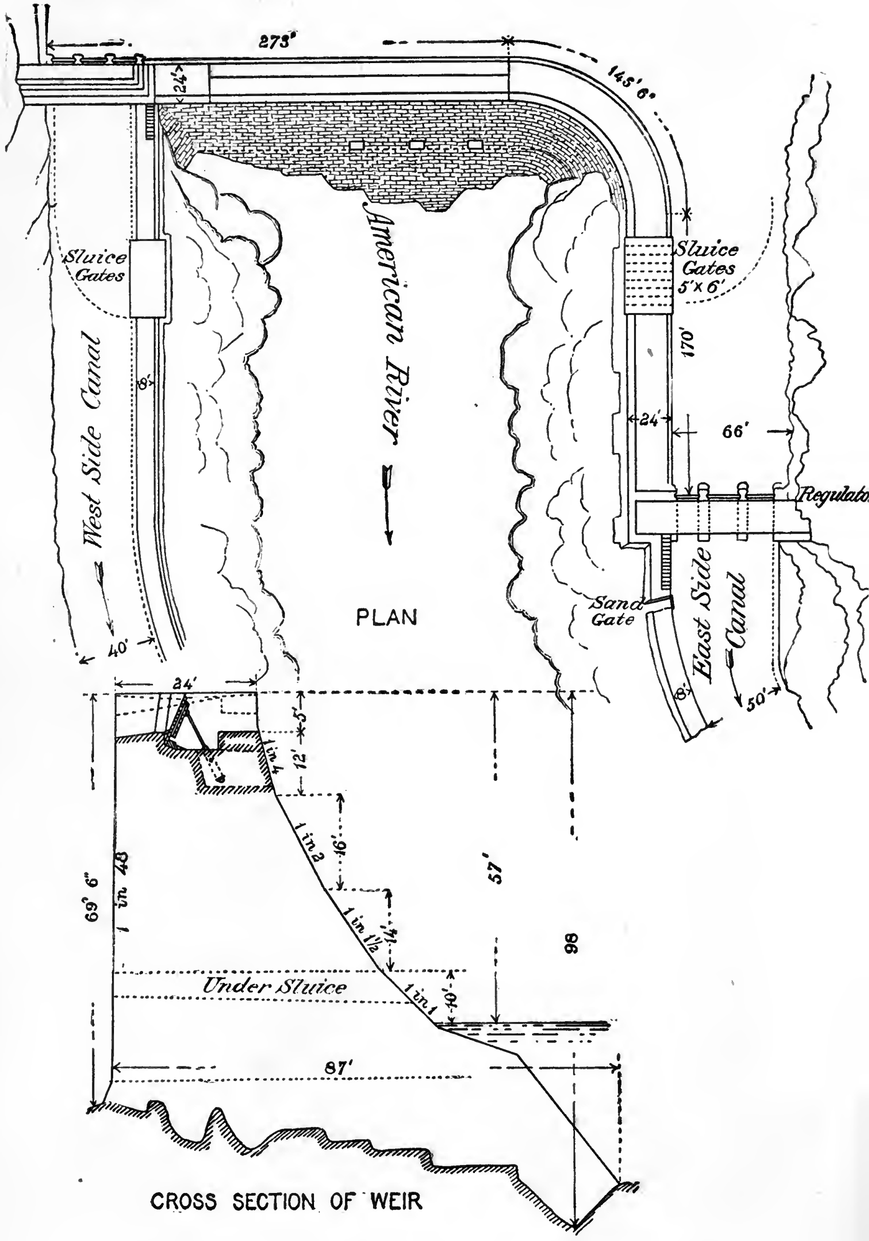

PLATE XXVIII.—FOLSOM CANAL, PLAN AND CROSS-SECTION OF WEIR.

floods may pass, and this wasteway is closed by a single long shutter, consisting of a Pratt truss backed with wood, which can be raised and lowered by means of hydraulic presses, operated from a power-house near by. The dam is constructed throughout of uncoursed rubble masonry.

280. Colorado River Dam, Texas.—This dam is built across the Colorado river for the supply of water and water-power to the city of Austin, Texas. Its interior is of rubble masonry, faced on both sides and on top with large cut blocks of coursed granite. It is 1275 feet long on top, 1125 feet of which are constructed as an overfall wasteway, and 66 feet in maximum height, its upper face being vertical. The lower face has an easy ogee-shaped curve (Fig. 89), calculated to pass the

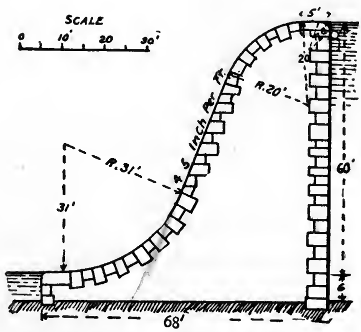

FIG. 89.—CROSS-SECTION OF COLORADO RIVER DAM.

waters with such ease that the erosive action at the base will be reduced to a minimum. The structure is practically a great overfall weir, the maximum flood to be passed being estimated at 250,000 second-feet from a catchment basin of 50,000 square miles.

The cross-section is somewhat heavier than theory would demand if the dam were built to act as a retaining wall only. The lower portion of the down-stream face is curved with a radius of 31 feet tangent at the bottom to low-water surface, so as to deliver the floods away from the toe and against the back-water in the river. The upper end of the curve is tangent to the main slope, which has a batter of 3 in 8, and ends on top in a curve of 20 feet radius. This top curve is tangent to the horizontal crest line, which is 5 feet wide. The total top width is 16 feet, and the maximum width at base 68 feet.

281. Bear Valley and Zola Dams.—The most notable curved dams are the Bear Valley dam in California, and the

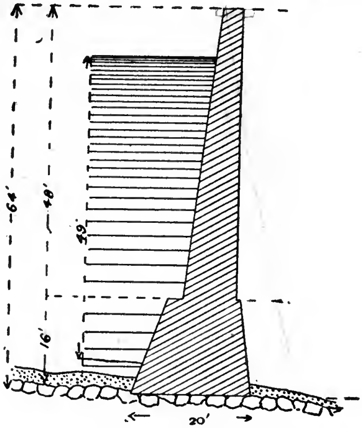

FIG. 90.—CROSS-SECTION OF BEAR VALLEY DAM.

Zola dam in France, the cross-sections of which are unusually light, as they depend chiefly on their curved plan for their

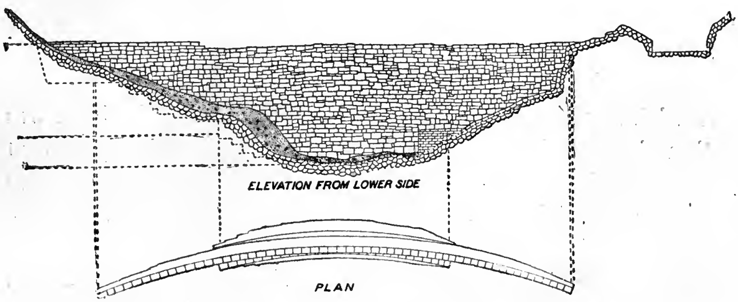

FIG. 91.—PLAN AND ELEVATION OF BEAR VALLEY DAM.

stability. The former (Fig. 90) is but 3.2 feet in width on top, and at a depth of 48 feet below its crest its width is but 8.4 feet. At this point an offset of 2 feet is made on each side, and its width thence increases to 20 feet at its base, which is at a point 64 feet below its crest. This structure is 450 feet in

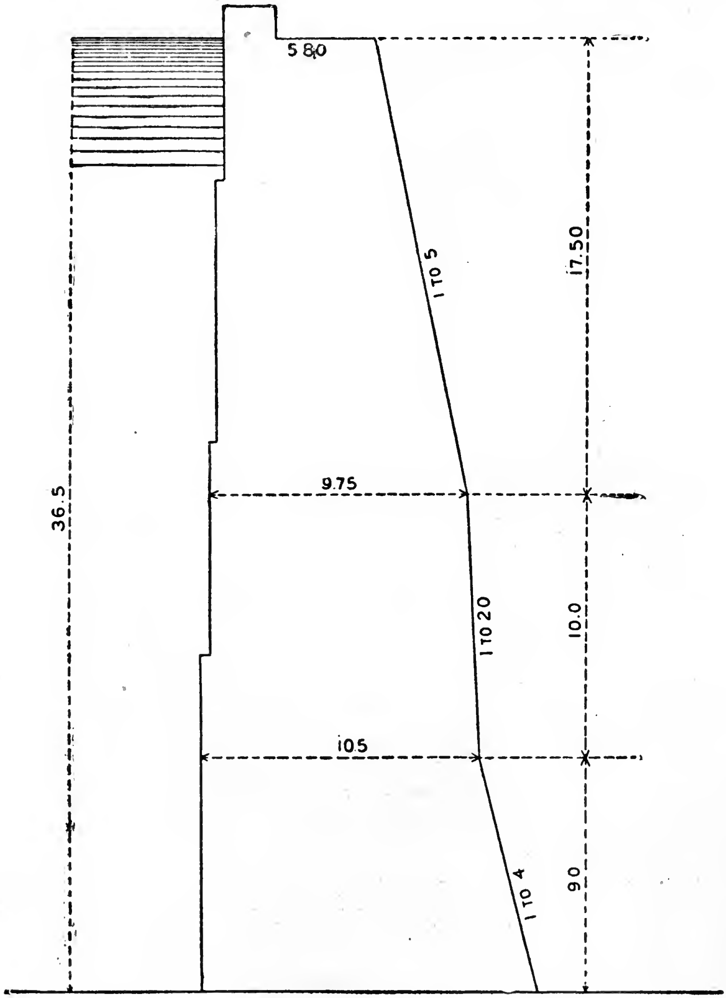

FIG. 92.—CROSS-SECTION OF ZOLA DAM, FRANCE.

length on top, and in plan it is curved with a 300-foot radius (Fig. 91). It is built throughout of the best uncoursed rubble granite masonry, and depends almost wholly on its curved plan and the excellence of its construction for its stability, since the lines of pressure with the reservoir full fall from 13 to 15 feet outside of its base.

The Żola dam (Fig. 92) is 123 feet in maximum height, 19 feet in width on top, and 41.8 feet in width at the base. Its length on top is 205 feet, and it is curved with a radius of 158 feet. Like the Bear Valley dam, it depends chiefly on its curvature and the excellence of its construction for its stability. The material of which it is built is uncoursed rubble masonry.

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CHAPTER XX.

WASTEWAYS AND OUTLET SLUICES.

283. Wasteways.—Wasteways, escapes, or spillways as they are sometimes called, are an essential adjunct of every dam. They are to a reservoir what a safety-valve is to a steam-engine; the means of disposing of surplus waters due to floods and preventing these from topping the dam and possibly causing its destruction. Water should not be permitted to flow over the crest of a masonry dam unless it has been built in an unusually substantial manner calculated to withstand the shock of this overfall. It should never be permitted to flow over the face of a loose-rock or earth dam. The outer slope of an earth dam is its weakest part, and if water is permitted to top it it will speedily cut it away and cause a breach.

Too many of the great floods which have occurred in recent years bear testimony to the necessity of constructing substantial and ample wasteways. Moreover, an ample wasteway being provided, the greatest care should be exercised to maintain it always open and ready for use, independent of all undersluices and other discharge outlets which may be closed by valves or other mechanical means. To the lack of one or both of these precautions was due the destruction of the South Fork dam in Pennsylvania in 1889; of the Walnut Grove dam in Arizona in the spring of 1890, and many other similar catastrophes. Had the wasteway of the South Fork dam been ample, as it originally was, the water would not have flowed over the crest of the dam and have caused its destruction. But the wasteway was barred by fish-screens, and these not only obstructed the passage of the water but caught floating timber and logs brought down by the flood, which so diminished the area of the spillway as to cause the waters to top the dam. In the case of the Walnut Grove dam the area of the wasteway was unquestionably insufficient, resulting consequently in the passage of much of the flood water over the dam crest and resulting in the destruction of the work.

284. Character and Design of Wasteways.—In designing a wasteway for a reservoir data relating to the greatest floods likely to occur must be sought for in its catchment basin, and the dimensions of the wasteway must be proportioned for the extraordinary floods. The methods of determining the great floods and the necessity for looking for signs of these in the valleys has already been discussed in Chapter IV. Should other reservoirs exist above that under consideration provision should be made for the discharge of their contents lest their embankments give way; this can only be done by considering their volume and calculating the velocity and consequent quantity which will reach the dam at any one time.

Having fixed on the area of the wasteway from a knowledge of the maximum flood to be discharged, the chief consideration to be borne in mind is the relation of its depth to its length. A long wasteway may permit the loss of too great a volume of water if exposed to the action of the wind, whereas a short one renders it necessary to give the dam an increased height in order that it may have the required capacity. The depth of the wasteway will be largely regulated by the probable wave-height, and this will depend on the depth and fetch of the reservoir (Article 237). The difference in height between the crest of the dam and the wasteway will generally vary between 3 and 10 feet as limits. Care should always be taken in designing a wasteway to rapidly increase the slope of its bed immediately below the crest of the waste weir, so that there shall be no piling or banking up of water to retard the discharge. A quick drop beyond the crest considerably enhances the discharging capacity.

285. Discharge of Waste Weirs.—For the calculation of discharge the wasteway can be considered as a measuring weir subject to the weir formulas. If the crest of the wasteway has a sharp square edge or falls away with considerable suddenness on the lower side, Francis' formula (Art. 86) may be applied with approximate results, and we have

$$Q = 3.33(l - .1nh)h^{\frac{3}{2}}. \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (1)$$

The mean velocity of flow over the crest is

$$v = \tfrac{2}{3}\sqrt{2gh},$$

and multiplying the depth of water on the weir h into its length l we get the volume of discharge.

When the overfall from the crest is not sudden

$$Q = 5.35mlh^{\frac{3}{2}}, \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (2)$$

in which m is a coefficient of contraction with the value of about .62. Where the overfall weir has a wide crest the following formula, suggested by Mr. Francis, is the most accurate for depths between 6 and 18 inches, viz.,

$$Q = 3.012lh^{1.53}. \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (3)$$

Another formula and one commonly used in India for determining the discharge of wasteways is

$$Q = l \times \tfrac{2}{3}c \times 8.02\sqrt{d^3},$$

in which c is a coefficient which varies with the form of the weir and rarely exceeds .65, though with a majority of weirs it is about equal to .62. In which case

$$Q = 3.33l\sqrt{d^3},$$

where d is the maximum depth in feet of water to be permitted to pass over the weir. Ordinarily there is no velocity of approach to a reservoir wasteway, though should the water reach the latter by a cut it may be necessary to take the velocity of approach into account.

286. Classes of Wasteways.—Wasteways may be divided into three general classes, depending upon the character of the dam and the topography of the site. First, the entire structure, if of masonry, may be utilized as a wasteway. This can only be done by making the cross-section of the dam unusually heavy and providing it against the shock of falling water as in the case of the Folsom, Turlock, Betwa, Colorado River, and Vyrnwy dams (Articles 276 to 280). Second, if the dam is of masonry it may be given the theoretical cross-section and the wasteway made in one end of it, if the dam at this point is sufficiently low not to subject it to great shock from the falling water. This is the case with the Bhatgur, Tansa, and New Croton dams (Articles 269 to 271).

It is never advisable to build a wasteway in earth or loose-rock dams, as it is difficult to make a safe bond between the masonry wasteway and the earth dam, and unless extraordinary circumstances demand it such an arrangement should be avoided. In some cases, however, this has been done, great care being taken in connecting the two classes of work and the wasteway being carefully lined with masonry and provided with masonry wing walls for the protection of the earth embankment.

The third general class of wasteways is where these are built in the hillsides at some distance from the dam. If on the slopes adjacent to one end of the dam, the discharge water must be so directed by retaining walls that it will flow back into the stream channel clear of the toe of the dam. Such wasteways may be excavated in the solid rock, or if in earth they should be paved or lined with masonry. The safest disposition for the wasteway is at some favorable point in the rim of the reservoir entirely free and away from the dam. This may be through some low saddle, which if too low may be filled in with a waste weir of masonry, or if too high may be excavated to the proper elevation. Such an isolated channel is frequently found beyond some spur immediately adjacent to one end of the dam and discharging back through a separate channel. This is the case in the Oak Ridge reservoir dam in New Jersey, the Ashti and Periar dams in India, and the Pecos and Idaho dams in the West.

287. Shapes of Waste Weirs.—The forms of waste weirs for dams vary considerably with the circumstances under which they are constructed. Their general design is very similar to that of weirs used for purposes of diversion and thoroughly discussed in Chapter XII. It is therefore unnecessary here to enter into any general discussion of the thickness and dimensions of waste weirs or their shapes. They may be given the ogee shape (Article 137) in order that the water falling over them shall produce the least vibration in the structure; or water-cushions may be employed to deaden the effect of the falling water (Article 138).

288. Examples of Wasteways.—Brief descriptions and illustrations of wasteways were given in Articles 269 to 273. The wasteway of the Sweetwater dam is peculiar. It is built as a continuation of the main dam and, as shown in Plates XXIV and XXV, the water from the reservoir enters the several separate passageways over a waste weir and drops into a shallow water-cushion. Thence it flows through a channel partly excavated in the side of the ravine and partly constructed by means of an artificial wall which carries the water clear of the toe of the dam. The wasteways to the Periar dam are two in number, one at either end of the structure; both are separated from the main dam by means of low saddles of rock. That on the right bank is cut down for a length of 420 feet till its crest is 11 feet below that of the main dam. On the left bank the solid rock is 50 feet below the crest of the dam, and the saddle is closed with a waste weir of masonry (Fig. 84) built up to the same level as that of the wasteway on the other bank. At a distance of 60 feet from this waste weir is built a low subsidiary weir 10 feet in height with its crest 30 feet below the upper wall, thus forming a water-cushion on which the floods fall. This escape weir is so designed that the lines of pressure fall within the middle third when a depth of 12 feet of water is passing over the crest, and so that the water shall fall clear of the weir to the water-cushion below.

A similar waste weir to that just described and one somewhat similarly situated is that at the Idaho Mining and Irrigation Company's dam described in Article 240. The wasteway of the Ashti tank in India consists of a channel having a clear width of 800 feet excavated through a saddle in the high ridge bounding the reservoir on its western side. The bed of this channel at its entrance forms the weir crest and is level for a length of about 600 feet and then falls away with a slope of 1 in 100 to a side drainage channel. The dam is 12 feet in height above the crest of the wasteway and the greatest flood anticipated would raise the water in this wasteway to 7 feet above its crest or to within 5 feet of the top of the dam—just sufficient to prevent waves from topping it.

289. Automatic Shutters and Gates.—The use of flashboards or any similar permanent obstruction in a wasteway in order to increase the storage capacity of the reservoir is greatly to be condemned. Such obstructions must be removed at the time of great floods or else these will top the dam. The result of their use is that the area of the wasteway is diminished below the point of safety, while the integrity of the structure depends upon the careful attention of the watchmen, who should remove the flashboards. Automatic shutters, however, have been used with considerable success in a few instances. These, however, should only be employed where water is of the greatest value and the saving of every drop is essential.

One of the most desirable forms of these is that shown in Fig. 93. It consists of a row of upright iron shutters, each 18 feet long and 22 inches high. These are supported by struts or tension rods hinged to the crest of the weir on the up-stream side and to the upper side of the shutter at about two thirds of the distance from its crest, or, in other words, below its centre of gravity. As soon as the water level approaches the top of the shutter it causes its lower end to slide inward and the whole falls flat against the top of the weir, offering no obstruction to the passage of the water.

An ingenious form of automatic weir gate (Pl. XXIX) was devised and patented by Mr. E. K. Reinold for use on the Bhatgur reservoir in India. This gate is of value where water is precious, and can be utilized with considerable safety to retain water to the full storage capacity of the reservoir. The gate falls automatically as soon as the water reaches its crest, and continues to fall as the flood rises until the full discharge capacity of the wasteway is brought into action. The gate then closes as the flood subsides, enabling the reservoir to retain the maximum amount of water.

The gate slides vertically on two contact surfaces one of which is the face of the wasteway against which it presses while the other surface is attached to the face of the gate. These surfaces slide parallel to each other and are the surfaces of inclined planes. The gate rests on wheels running on rails, and the axes of the wheels are parallel to the line of the rails and at a slight angle to the contact planes (Pl. XXIX),

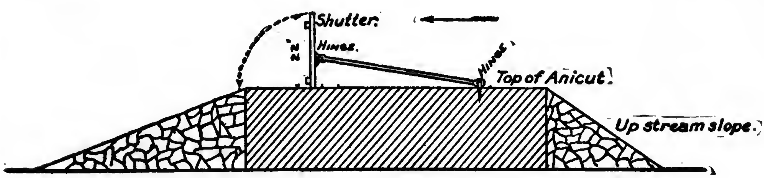

FIG. 93.—CROSS-SECTION OF SHUTTER ON SOANE WEIR, INDIA.

so that the latter do not touch until the gate is fully raised or closed, thus permitting by leakage a large amount of flood water to run out of them until the last moment. The gates are operated by means of counterpoises balanced in water cisterns, the weight of these counterpoises exceeding the weight of the gate by a little more than the amount of friction, and they act by displacing their volume in the water cisterns in which they plunge, thus lessening their weight by that volume of water. As the water flows over the top of the gate it simultaneously enters the cast-iron cisterns in which the counterweights hang. When the water ceases to enter the cisterns owing to its level having fallen below that of the inlets,

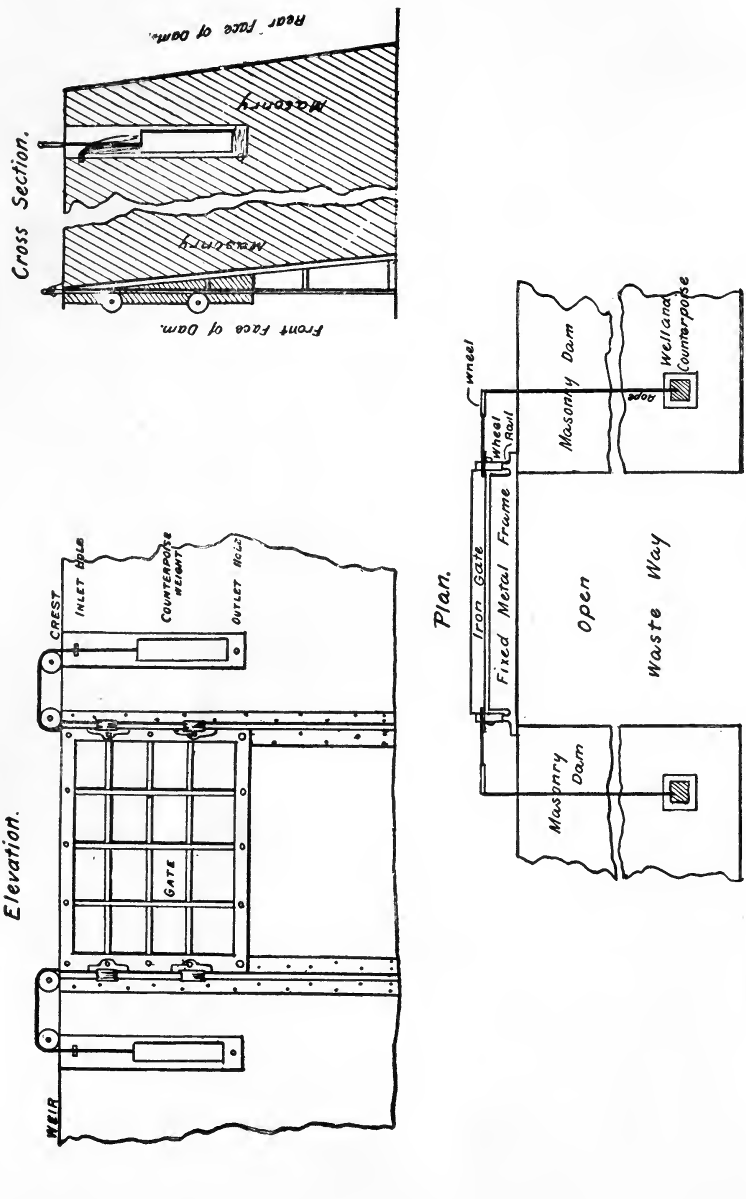

PLATE XXIX.—PLAN, ELEVATION, AND CROSS-SECTION OF REINOLD'S AUTOMATIC WASTE GATE, INDIA.

it runs out from holes in the bottom and the weights then become heavier than the gate and raise it.

290. Undersluices.—Undersluices perform the same function for storage dams as do scouring sluices in diversion weirs. Their object is to remove or to prevent the deposition of sediment in the reservoir. Undersluices have little effect in preventing the deposition of silt unless the area of their opening is great compared to the area of the flood, while they are useless for the removal of silt already deposited. This is shown by the manner in which such reservoirs as Lake Fife and the Vir reservoir in Bombay, India, and the Folsom reservoir in California have silted up in spite of them. If the dam is high and the discharge through the undersluices will keep the flood level below the full supply level, they may be efficient in preventing the deposit of silt by carrying it off in suspension. If the dam is low and the area of the undersluices will not enable them to keep the flood-level below full-supply level, they will have but little effect. This has been partly proved at the Betwa and Bhatgur reservoirs in India, where experience shows that their scouring or preventive effect is felt but a few feet to either side of the sluice, and silt will deposit close to the entrance. In other words, undersluices do little more than keep an open channel above them.

291. Examples of Undersluices.—The most successful attempt to utilize undersluices for the clearance of silt is at the Bhatgur reservoir in India. There are fifteen undersluices in the centre of the dam near its bottom, their sills being 60 feet below high-water mark (Pl. XXII). Each of these undersluices is 4 by 8 feet in interior dimensions, and they are lined throughout with the best ashlar masonry. Under a full head they will discharge 20,000 second-feet, and the velocity through them is 36 feet per second. Each undersluice is closed by a heavy iron gate which slides vertically and weighs about 2 tons. They are operated by steel screws worked from above by a female capstan screw turned by hand levers. Stout wooden gratings protect the gates from injury by floating objects. The undersluices are placed about 30 feet apart, and this space was filled with sediment shortly after the completion of the dam.

In the bottom of the Folsom dam in California there is a set of three undersluices, the object of which is to remove silt deposited in the reservoir (Pl. XXVIII). These undersluices are built in the centre of the weir near its bottom and are under a head of 60 feet, the area of each one being 4 by 4 feet. While these undersluices have not impaired the integrity of the structure, they have been of little service in preventing the deposit of silt, as their area compared with that of the floods is comparatively small. Where undersluices have been employed to carry away silt-laden waters from in front of a canal head they have proved more effective. In the bottom of the Idaho Mining Company's dam an undersluice is projected the sill of which will be 13 feet below the headgates of the canal and 24 below the crest of the dam. It will be 4 feet wide by 8 feet high inside, closed by a gate operated by a screw from the top of the dam. A similar under or scouring sluice is built in the bottom of the Pecos dam adjacent to the entrance to the canal head.

292. Outlet Sluices.—As the object of a storage dam is to impound water that it may be drawn off when wanted, one or more outlet sluices must be constructed at the level at which water can be drawn off. These outlet sluices either terminate in pipe lines which carry the water to the point of distribution or discharge directly into the canal head or back into the stream channel, to be again diverted lower down. The greater the depth at which these sluices are placed, the greater the available capacity of the reservoir. They may either be built in the body of the dam or through the confining hillsides independently of the dam. The latter is by far the better and safer method, and wherever practicable should be employed, as anything which breaks the homogeneity of the dam is a menace to its integrity. With an earth dam this is especially true, and its greatest source of weakness is the masonry discharge conduit passing through it. Simple pipes should never be laid through an earth embankment, as under the pressure of the water in the reservoir this is certain ultimately to find its way along the line between the pipe and the earth embankment or through a loose joint in the pipe, and the water which enters the embankment in this manner will rapidly increase in quantity until the structure is destroyed.

It is essential that the outlet sluices, valves, pipes, etc., should always be accessible for inspection and repair in order that the constant use of the reservoir may not be interrupted. When they must be placed in the embankment a masonry conduit should be built through it, and for convenience of inspection an iron pipe should be placed in this. The conduit should be of such dimensions that a man can pass through it, and the pipe should be so placed within it as to be easily seen and repaired. In order to prevent the travel of seepage water along the outside of the conduit, rings of masonry should be placed at short intervals along its length, and these should project not less than from 1 to 2 feet from its surface. The chief objection to laying a conduit through a dam is its liability to fracture through settlement.

Better and safer than this is to lay the discharge pipes in a trench dug under the foundation of the dam in the surface rock or soil. Such a trench should be substantially lined and roofed with concrete, and will offer little inducement for travel of seepage water. The best method of all, however, for the placing of outlet pipes is to build them through the surface rock or soil of the country, excavating a tunnel for this purpose and laying the pipes in it, the whole being away from and independent of the dam. This insures them against any damage from settlement in the structure.

Sometimes the entrance to the outlet culvert is not placed at the lowest level of the reservoir, but at about two thirds the way up the embankment from the bottom, or at such height that the pressure will enable a siphon to draw water off from the lowest depths of the reservoir. This siphon pipe is carried down to the bottom of the reservoir and passes up through the culvert in which is placed the main pipe connected with the valve chamber and supplied directly from orifices above the level of the conduit (Fig. 94). Where a reservoir embankment is very low—say 25 feet or under—it may be discharged by simply carrying a siphon pipe over the top of the embankment with no outlet pipe or conduit through the embankment.

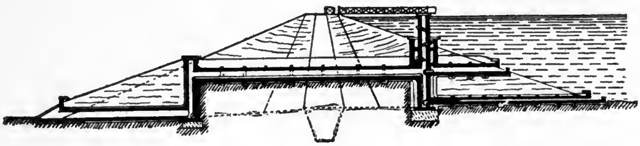

FIG. 94.—CROSS-SECTION OF EARTH DAM

293. Gate Towers and Valve Chambers.—The valves for controlling the admission of water to the outlet sluice are either operated from a valve chamber let into the body of the dam or from a gate tower situated in the reservoir at a point vertically over the inlet to the discharge conduit. In order that these valves shall not be worked under too great pressure, water is usually admitted to the tower or well from orifices placed at several depths, and in this well the conduit heads. At its exit at the lower side of the dam is generally placed a second valve chamber or gatehouse for the control of water which is admitted to the distributing pipes or canal. The orifices admitting water to the well tower are closed on the outside by plugs or close-fitting valves which can be operated from the top of the tower or valve chamber; while the valve admitting the water from the bottom of the well to the outlet sluices is operated either from the tower or from the bottom of the well pit by screws and hand gearing. In this manner the attendant in charge has full control of the whole outlet works, and all pipes and valves are under perfect control so that the supply can at any time be arrested for the repair of pipes. In case a gate tower is constructed independently of and away from the body of the dam, great care must be taken to make it sufficiently substantial to withstand the thrust of ice, or it should be buttressed against the side of the dam.

The outlet sluicepipe which passes through the embankment may be connected on the inside of the reservoir by a flexible joint with another pipe of the same diameter, to the end of which is attached a float. This pipe can thus be moved vertically, and admits of the water being drawn off from the surface where the pressure on the valve is the least. Where the expense will permit, the better method is that of admitting the water to a valve well through orifices situated at varying heights. One of the great difficulties encountered is to insure a constant discharge from the reservoir with a constantly varying head in it or in the gate well. The usual method of insuring a constant discharge is by opening the valve gates controlling the admission of water to the outlet sluice to a greater or less extent according to the amount of water required, though automatic systems of maintaining a constant discharge irrespective of the head have been used with more or less success in a few cases. The inlets to the valve chamber are of two general classes. That illustrated in Fig. 95 is of the kind em-

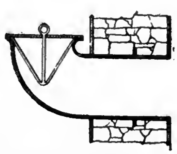

FIG. 95.—VALVE-PLUG, SWEETWATER DAM.

ployed on the Sweetwater dam in California, and consists of a simple cast-iron plug let into the top of the pipe, the end of which is bent upward. This plug is held in position by the pressure of the water and is removed by a chain operated from above by a windlass. In Plate XXV is shown the method of placing the valves at varying heights and the arrangement of air valve and gatehouse at the lower end of the dam.

Another method of admitting water to the valve chamber is by means of rectangular openings in the side of the chamber on the inner surface of which stop valves are bolted. These are usually of cast-iron, the seat and bearing of the valve being faced with bronze composition. Above this projects a screw stem which is operated from above by means of a female capstan screw. Where the area of such valves exceeds 4 or 5 square feet or the pressure is more than 20 to 25 pounds, some geared motion is usually necessary to enable a single man to operate it. The intake valve permitting the water to pass from the valve chamber to the outlet sluice is usually a sliding valve, working on bronze bearings and operated from above by a screw and hand gearing. It is not unusual to employ more than one such valve, according to the amount of water to be admitted and the consequent number of outlet pipes required. The foundations for gate towers must be of the most substantial character, especially where they are attached to loose rock or earth dams,—in which case the foundation must be carried down to a sufficient depth to insure stability.

294. Examples of Gate Towers and Outlet Sluices.—Owing to the low inclination of the inner surface of earth embankments or loose-rock dams, it is necessary to construct the gate tower controlling the outlet sluice at some little distance in the reservoir so that it shall come above the entrance to the sluice. This method of construction is occasionally employed on masonry dams, and an excellent example of such a work is that illustrated in Plates XXIV and XXV, showing the gate tower to the Sweetwater reservoir. In Fig. 96 are shown in plan and cross-section the arrangement of the valve chamber and intakes of the proposed Bear Valley dam in California. As will be noticed, the valve chamber or tower is built of masonry as a projection on the inner surface of the dam, thus becoming practically a gate tower attached to the centre of the dam. The intake valves in this case are similar to those employed in the Sweetwater dam, and discharge directly into a valve well.

A much better practice, however, is that followed on the Vyrnwy dam in Wales and the San Mateo dam in California. In the case of the former there are two discharge sluices operated from valve-houses built in the body of the dam for discharging compensation water back into the stream. The main valve chamber, however, for the supply of water to the aqueduct is situated at a point on the shore of the reservoir about three fourths of a mile distant from the dam; entirely independent of it, and out in the lake at such a distance as to control water at nearly the maximum depth. The valves and other mechanisms employed in this tower are all operated by hydraulic power furnished from a water-wheel supplied by a small mountain reservoir. In the case of the San Mateo dam (Pl. XXIII), and the proposed Citizens' Water Company dam in Colorado, the valve tower is situated at a point quite independ-

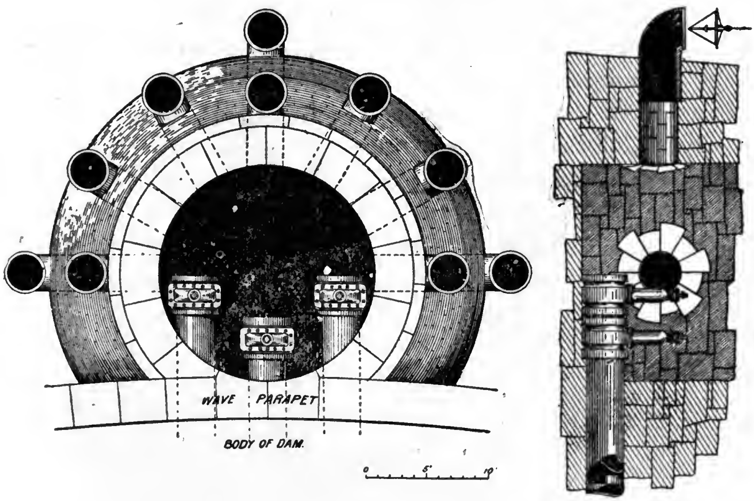

FIG. 96.—VALVE CHAMBER AND VALVES.

ent of the dam, and the outlet conduit passes through the country rock at a sufficient distance from the abutments of the structure to be entirely free from the pressure of its possible subsidence. As shown in the illustration, water is admitted at three different elevations through inlet pipes which discharge directly into a main iron standpipe passing vertically through a shaft which is the entire height of the dam. The entrance of this water to the standpipe is controlled by plunger valves operated by hand wheels and approached by a stairway passing through the tower. At the outer end of the discharge pipe is another gate-well where the main supply is regulated.

CHAPTER XXI.

PUMPING, TOOLS, AND MAINTENANCE.

295. Underground Cribwork or Tunnels.—Submerged cribs have been satisfactorily employed by the American Water Company on Cherry creek in Colorado, and by the Citizens' Water Company on the South Fork of the Platte river in Colorado. The former enterprise consists of a submerged open crib dam sunk in the gravel bed of Cherry creek,

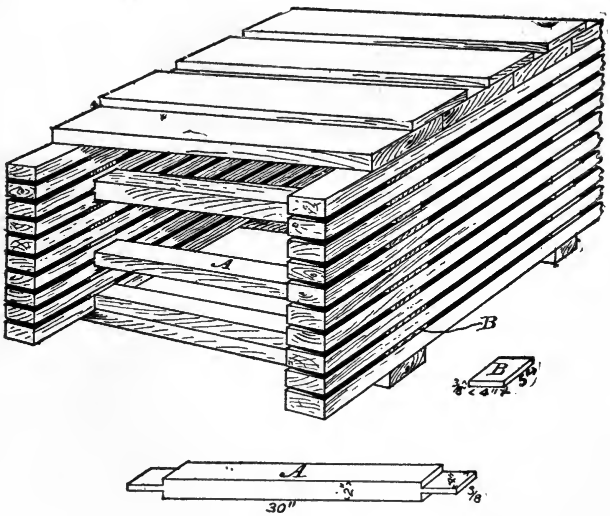

FIG. 97.—GATHERING-CRIBS, CITIZENS' WATER CO., DENVER.

and resting on solid rock which is 73 feet below the surface of the stream. This cribwork is 70 feet in height, and its crest is 3 feet below the bed of the stream. This is not a dam, as it does not extend across the entire channel of the stream, but it stops the movement of that portion of the subsurface water which enters the cribwork. This is open on the upper side but closed on the down-stream side, and consists of timbers 14 inches in dimension at the bottom of the dam, which is decreased to 8 inches at the top. These timbers are placed 4 feet apart across stream, and are planked on both faces with interstices of 3 inches on the upper face. The water caught in this cribwork is pumped to the surface.

The Citizens' Water Company develope the underground waters of the Platte river by means of a series of gathering galleries, consisting of perforated pipe and open cribwork laid at a depth of from 14 to 22 feet below the surface of the gravel bed of the stream. The cribs (Fig. 97) are 30 inches square, and about a mile of these have been built running up the bed of the stream, besides about a mile of perforated pipe 30 inches in diameter. The average daily yield obtained by these galleries is nearly 10 acre-feet of water, which is led off through the pipes by natural flow.

296. Tunnelling Underground.—For the development of underground waters, tunnelling, which is a little different from the cribwork just described, has been resorted to in a few instances. For the development of the water supply of Ontario Colony (Art. 42), and at the mouth of the Santa Anna river in California, tunnels have been built under the stream bed, the cross-section of these being trifling, and the tops roofed by open lagging, while the sides and bottom are formed into an impervious channel by a framing of woodwork or a cement lining. The seepage water which enters these tunnels is led off through open cuts, and is let into the irrigating ditches.

297. Pumping or Lift Irrigation.—The methods of irrigation heretofore considered are those in which the water reaches the irrigable land by means of gravity or natural flow. Frequently, however, there are large volumes of water which are situated at such low levels that gravity will not carry them to the field to be irrigated, and this water must be raised or lifted by means of pumps or other lifting devices. Lift irrigation may be employed to utilize the water from wells or from natural streams flowing at a lower level than the field worked, or it may be employed to raise water from the canals to higher levels than those reached by them.

When the gravity sources of supply have been entirely utilized, large areas of land may still be brought under cultivation by the employment of pumps. As irrigation is practised the subsurface soil becomes saturated, the ground-water level is raised, and much of the water which is delivered by gravity systems may by pumped up and re-employed for irrigation, thus greatly adding to the duty of the ultimate sources of water supply. The value of pumping for this purpose has been recognized in the older European and Asiatic countries for ages. A very large proportion of the irrigation in Europe, China, Japan, India, and Egypt is by means of lifting. Among the various methods more commonly practised in Asia for lifting water from wells are the *Mot* of India, which consists of a rope passing over a pulley down into the well and to the bottom of which a bucket or other receptacle is attached. This is raised by two bullocks walking away with the rope and raising the bucket to the top of the well, where it is emptied into the distributing ditches. One of the more common methods of pumping is by means of the *Persian Wheel*, which consists of a vertical wheel on the outer rim of which are attached buckets which dip into a well, and as they reach the upper circumference of the wheel spill their water into a trough which leads it to the fields. This wheel is made to revolve by means of bullock walking in a circle and drawing a sweep attached to rough, cogged gearing. By this means two bullocks are estimated as capable of lifting 2000 cubic feet of water per day. Still another method of lifting water is by means of the *Paecottah*, which is simply the old-fashioned well-sweep of this country. By its use from 400 to 2000 cubic feet of water can be raised a day, while with the Mot, two bullocks working 10 hours a day will raise about $3\frac{3}{4}$ acre-feet of water in a season of 90 days.

In this country the value of pumping as a means of irrigation is not yet fully appreciated: a few windmills and water wheels are utilized for this purpose, and some small amount of pumping is done by steam-power, though the value of the water supply to be derived from the latter mode of lifting is destined to increase greatly in the near future.

298. Windmills and Elevators.—Windmills have been extensively used in the San Joaquin valley in California and in a few places in the Colorado plains and elsewhere in the West for raising water for purposes of irrigation. As yet they have been employed chiefly for pumping for domestic uses, but as water becomes more valuable windmills are finding greater favor. Most of these machines are patented, and the makers furnish all the information desired relative to their cost, capacity, and duty. A modern ten-foot wheel will average about one eighth horse-power developed for a stiff breeze and will cost about six cents per horse-power per hour. Larger wheels are much cheaper. A fifteen-foot wheel will irrigate about seven acres at a cost of $8 per acre per annum.

A link-belt water elevator manufactured in Chicago has been successfully employed in the West for raising water for irrigation. It is operated by horse-power, and consists of a link belt erected at a slight inclination from the vertical and revolving over two wheels, one pivoted a little below the level of the water surface and the other at the summit of the height to which the water is to be lifted. On this belt are a number of iron vanes attached at intervals of about 8 inches apart, and these pass up through a closed wooden boxing, so that each vane acts as a lift and raises the water above it, as does the old chain pump used in shallow wells. This water is emptied out through a lip to a flume, from which it runs to the irrigated lands. With a belt speed of 300 feet the smallest of these elevators will raise about 20 cubic feet per minute to a height of 10 feet; the largest will elevate nearly 5 second-feet of water to the same height.

299. Water-wheels.—Lifting water by means of undershot water-wheels has been practised ever since the early placer operations in California, while in the older countries this method of lifting water is extremely ancient. The Noria of Italy is simply an undershot water-wheel of this description. As used in a few occasional instances in the West, these wheels are very similar in appearance to an old steamboat paddle-wheel, varying from 15 to 20 feet in diameter, the width of the wheel or the length of the paddles being from 6 to 10 feet. Such a wheel (Fig. 98) will rest either on cribwork abutting on the shores and in the river, or if the change in the flood height of the river is considerable it may rest on some variety of

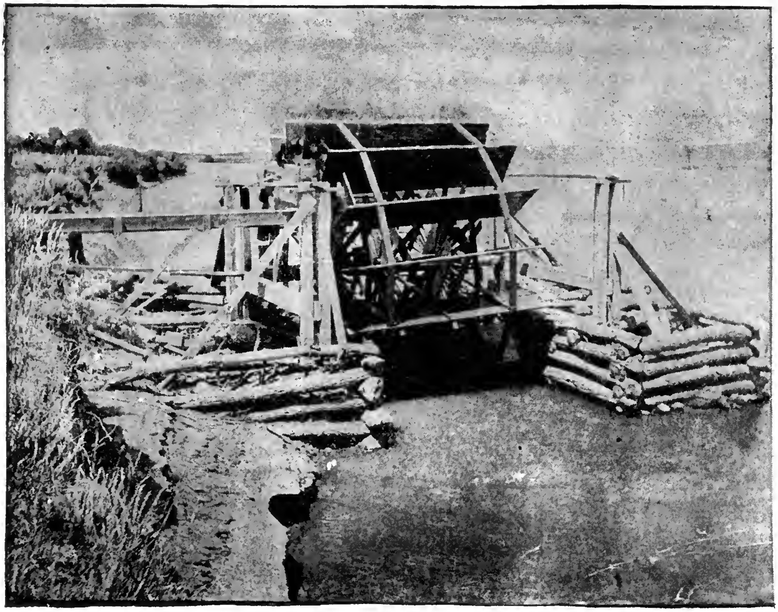

FIG. 98.—VIEW OF WATER-WHEEL.

anchored float which will permit it to rise and fall with the stream. On the outer circumference of these wheels are placed a series of buckets, one attached to each blade or paddle. These buckets may be constructed of tin, as old tin cans, or sometimes are constructed of wood, and as the wheel revolves they are filled as they are successively immersed.

When they reach such a point in their revolution that the water begins to spill out of them, it is caught in a trough suitably placed, and from this runs into a flume which leads it to the fields.

Some of the water-wheels of this variety which have proved most successful on the Green river in Colorado are from 20 to 30 feet in diameter, the wooden axle being 5 inches in diameter, while the paddles dip about 2 feet into the water of the stream. The buckets, which are of wood, have an air-hole in the bottom closed by a suitable leather valve which permits of the bucket being rapidly filled by forcing out the air. These buckets are of wood, about 6 feet in length and 4 inches square, the capacity of each being about 5 gallons. There are sixteen paddles, to each of which is attached a bucket, thus enabling one revolution of the wheel to lift 80 gallons. The wheels make about two revolutions a minute, but as a large percentage of the water raised is spilled in emptying into the flume, each wheel has been found to handle about 4000 cubic feet a day.

300. **Steam Pumps.**—The value of steam pumps for purposes of irrigation is not fully appreciated. There are many places where water can be pumped at comparatively small cost, and yet where the land it will serve must otherwise remain uncultivated but for water obtained by this means. Steam pumping for irrigation has been practised to a limited extent in Colorado, in Arizona, and in California, and many varieties of pumps have been employed for this purpose. It is not the intention in a work of this sort to describe the mechanical details of pumps, the value of each type, or the theories and formulas on which its operation and coal consumption depend. These can all be found fully discussed in the many books and pamphlets which have been written, more particularly on the subjects of "Mine Pumping" and "Pumping for Waterworks," or they can be obtained from the trade catalogues of pump makers. The chief point of difference between pumping for irrigation and pumping for mines and waterworks is in the height to which the water has to be forced. For purposes of irrigation it has rarely to be lifted to heights exceeding 25 or

30 feet, the water having to be raised generally from a well or river merely to a sufficient height to enable it to flow to the fields by the action of gravity.

In a few notable instances it has been necessary to force the water to greater heights. In one case near Tucson, Arizona, the depth of the well is about 70 feet, and the water has to be raised this height to bring it to the surface of the ground. Perhaps the most remarkable instance of pumping for irrigation is in Italy, above Saluggia, on the Cavour canal. In this case the river Dora Baltea runs between rather high banks, and it was found impossible to bring water to the highest levels by means of natural flow. Accordingly the water that is taken from the river by one of the canals is pumped to the high level, whence it flows through a gravity system to the fields. There are in all four canal levels along the hillside. Between the two lower is placed an extensive pumping plant operated by turbines, which receive their water from the upper of these two and tail into the lower canal, whence the water is distributed to low-lying fields. The lower of the two upper canals supplies water by means of an immense wrought-iron pipe 3 feet in diameter, with a head of 66 feet, to the pumps below, and these force it through another pipe of the same dimensions a total height of 140 feet to the high-level or distributive canal. The head of 66 feet on the pumps practically counterbalances that height in the 140-foot force-pipe.

The varieties of pumps more commonly employed in the West are: 1. Centrifugal pumps, which for their operation require small steam-engines; 2. Vacuum pumps, pulsometers, and a variety of patented pump made in Greeley, Colorado, known as the Huffer and Nye pumps; and, 3. Pumping engines.

301. Centrifugal Pumps.—The ordinary centrifugal pumps employed for irrigation have capacities varying between 500 and 1500 gallons per minute, the height raised ranging from 20 to 80 feet. The average pump handles about a thousand gallons a minute or 2 second-feet, with heights of from 25 to 40 feet. Such a pump will irrigate from 5 to 10 acres per day, and in the course of an irrigation season will handle about 100 acres. It is easily operated by one man, and the cost of maintenance for a season of three months amounts to $2.50 per acre,—a relatively low water rate. A plant of this kind erected, including engine, boiler, and pumps, costs about $1500 —equivalent to a first cost of about $15 per acre.

302. Huffer and Nye Pumps.—These pumps have been used in large numbers in Colorado, Wyoming, and other portions of the West. Their capacity is small, averaging about 400 gallons a minute, or about one second-foot of water. They are capable of lifting water to heights of 15 to 20 feet, and of forcing it to low heights not exceeding 40 feet. They will irrigate from 3 to 5 acres per day, and if carefully handled from 50 to 100 acres in a season. The cost of operating these pumps, or the water rate, ranges between $3 and $5 per acre, while the first cost of the plant erected is about $1500, or from $15 to $30 per acre.

303. Pumping Engines.—The writer is strongly in favor of the use of steam pumping engines in preference to centrifugal pumps or any of the peculiar patented varieties. The regular steam pumping engines, such as those made by Worthington; Knowes; Smith, Vaile & Co., and numerous others, cost little or no more than the varieties of pumps just mentioned. Their maintenance cost is no higher, especially if compound or condensing engines are employed, while for large pumping plants they are much cheaper. Their operation requires more skilled labor than do the other pumps just mentioned, but they are far less liable to get out of order, and the injuries can be more readily repaired. A high-pressure pumping engine which the writer saw in operation in Arizona was capable of irrigating 100 acres. This pump cost $1000 erected, and its running expense was about $5 per half second-foot of water raised. Its original cost was about $10 per acre irrigated, while the annual charge for running expenses amounted to about $5 per acre. A much better and more modern plant, operated near Tucson, Arizona, by Mr. A. Hartt, consists of two compound pumping engines, capable of irrigating 600 acres per day of 12 hours at a cost of $3 per day. The first cost of this plant laid down was $4200, while the well, which is 70 feet in depth through quicksand, cost $5000. Allowing the well to have been of average cost, the whole plant would have cost a little over $5000—equivalent to a charge of $8.50 per acre. The daily working expenses are about $3 for raising $5\frac{1}{2}$ second-feet a height of 70 feet—equivalent to an annual charge of about 70 cents per acre.

The following is considered by the writer as a first-class pumping plant for the irrigation of about 1000 acres, where the water is to be pumped directly from a river or from an inexpensive well. This plant should consist of a duplicate set of the best of duplex compound pumping engines capable of raising each about 1200 gallons per minute, with a suction height not greater than 15 feet, and a force height of 20 to 40 feet additional. In developing the irrigable lands from such a plant as this a boiler capable of serving both pumps should be purchased at first, but only one pump need be purchased until sufficient of the land is developed to necessitate the purchase of the other pump. Then only one pump will be required for the performance of the requisite service during much of the time, the other being a duplicate or relay pump in case of accident. When, however, the entire property is to be irrigated, both pumps will be called upon to do their highest duty. Such a pumping plant can be erected in nearly any portion of the West for about $5000, or at a charge of $10 per acre. The cost of maintenance and operation for this plant should not exceed 75 cents an acre, which is much lower than the ordinary water rates for gravity systems.

304. Irrigation Tools.—There is little to say of the tools required in the construction and management of irrigation works. The only tools here discussed will be such unusual mechanisms as special-shaped ploughs and scrapers. The toolmakers now manufacture hoes, spades and shovels, ploughs and scrapers, of special designs for the making and control of ditches and furrows. Special ditching ploughs of unusual depth and reach are made as right and left ploughs, or sometimes to throw dirt in both directions, having a V-shaped shear, thus making a V-ditch at one operation. Ploughs of this kind are also arranged in gangs on sulkies.

Corrugated ribbed rollers are employed where the surface of the country is even and level, and for such crops as grain and alfalfa. These consist essentially of a roller of the ordinary form, on the outer surface of which are iron rings or projections of from 2 to 3 inches in height and of about the same width, placed from 4 to 8 inches apart. These projections are sometimes V-shaped. In running this roller over the surface of a well-harrowed field it leaves small furrows, down which the

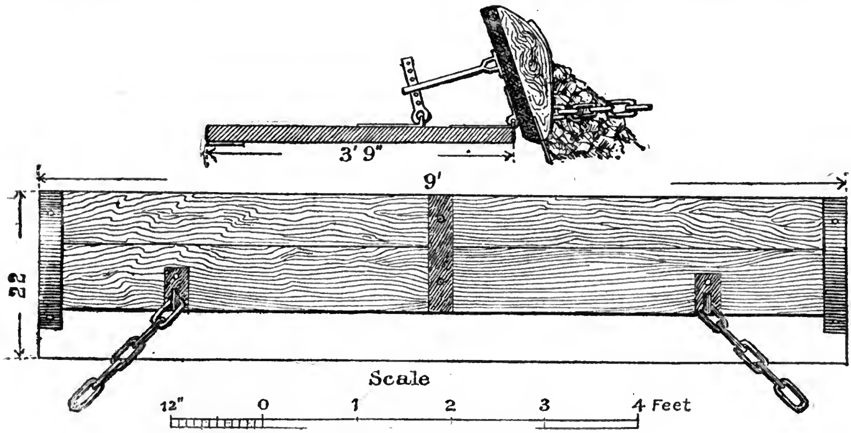

FIG. 99.—BUCK SCRAPER.

water runs, thus irrigating the crop much as if it were flooded.

305. **Scrapers.**—The most useful implement for the ditch and canal maker is the scraper, of which there are many forms and with most of which engineers are familiar. Two forms of scrapers which have peculiar advantages in ditch-making over the ordinary road scraper are the Fresno and Buck scrapers. The latter is especially useful in sandy soil with a low lift and short haul, and cheaper work has been done with it than with any other implement. A common form of Buck scraper consists of a working or frond board with an effective length of about 9 feet and a height of 22 inches. This board rests horizontally on edge on the ground, and consists of two planks each

2 inches in thickness, below which is fastened an iron cutting edge which reaches 7 inches lower (Fig. 99). At either end of the scraper is a cam-shaped roller 4 inches in height, on which the scraper is turned over. This board is fastened at the back to a tailboard 3 feet 9 inches in length, on which the driver stands, and is drawn forward by from two to four horses, the scraper being dumped by the driver merely stepping off the tailboard, the forward pull upsetting it. This implement handles a load of from 1 to $1\frac{1}{2}$ cubic yards, while its average daily capacity is about 130 cubic yards. For two horses a scraper of this form is rarely made over 6 feet in length, and the angle of the faceboard to the ground is about 28 degrees, and is regulated by the attachment to the tailboard. The Fresno scraper is most satisfactory in handling tough earth too heavy to be handled by a Buck scraper, and which would even give trouble to a road scraper. This implement is usually drawn by four horses and handles about 100 cubic yards a day, each load averaging a third of a cubic yard.

306. Excavating Machines.—One of the most popular ditching machines now employed in the West is the New Era ditcher and excavator, which consists of a series of gang-ploughs suspended on wheels. An endless belt or elevator is attached to the truck above these ploughs in such manner that it catches the dirt turned up by them and deposits it on the banks of the canal (Fig. 100). This machine requires from eight to twelve horses and three men to operate it, its maximum lift being about 10 feet, while each plough makes a furrow 12 inches wide and 6 inches deep. These machines have attained an average capacity of 100 cubic yards per linear mile and handle about 1000 cubic yards in a day's run. They are of use not only in excavating and building canals, but also in building low earth embankments for storage reservoirs.

The most elaborate apparatus yet employed in canal construction is the great canal excavator built by the San Francisco Bridge Company. This machine consists of a bridge truss supported on wheels running on rails on either bank of the canal. This deck truss has on it a track on which the engine-house and machinery travel back and forth across the canal, and the excavator consists of a dredging arm carrying an endless chain of buckets. The material brought up by these is deposited on one of two endless belt-carriers running on booms which dump it on either spillbank. The engineer can cause the excavator to move across the canal on the truss bridge, or can raise or lower the excavating arm carrying the buckets, causing these to move forward and perform their work. There are twenty-

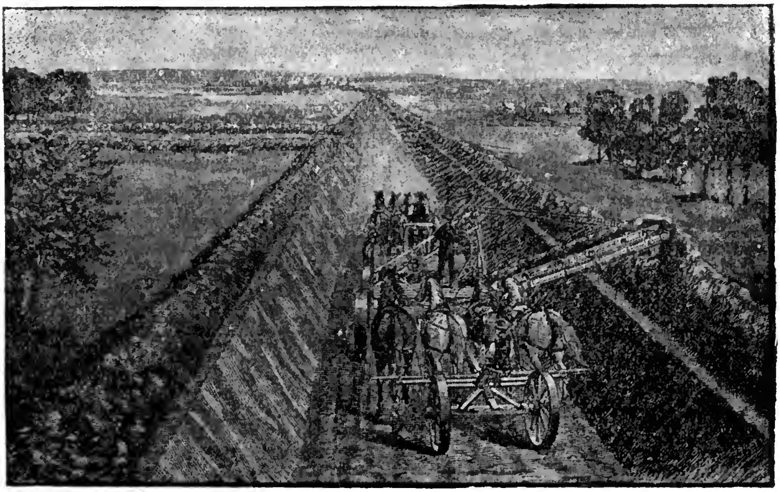

FIG. 100.—NEW ERA EXCAVATOR.

six of these buckets, each having a capacity of $\frac{1}{2}$ cubic yard, and the apparatus will excavate 3000 cubic yards a day in hard-pan. This machine has been found cheapest and most effective in material so hard that a pick will hardly penetrate it, and especially in excavating under water where scrapers cannot be used. In earth it has excavated from 4000 to 5000 cubic yards a day, at an average cost of 7 cents per cubic yard.

Dredges of various forms are employed on the larger canals to remove silt which may be deposited in them, and to repair and straighten banks which have been cut down or eroded by the action of the water. Such dredges are usually employed on scows or flatboats, and are operated by small steam engines, being similar in design and in construction to the ordinary dredges employed in river and harbor work, and in like operations.

307. Maintenance and Supervision of Canal Works.—Careful attention should be paid to the proper maintenance and the making of all needful repairs on the lines of canals, reservoirs, and other irrigation works. The expenditure of an exceeding small amount of time or money in repairing an injury to canal banks or other works may, if done in time, prevent great destruction of life and property consequent on an injury to the canal system. In order that these repairs may be intelligently made, and that damage to the canal property may be discovered in time, a suitable system of supervision must be inaugurated upon the completion of construction. Such a system should include an engineer, a superintendent, and patrolmen.

308. Sources of Impairment of Irrigation Works.—These are:

1. Erosion of the canal banks by water.
2. Filling of the canal channel or reservoir from deposition of sediment.
3. Erosion of the outer banks due to storm and flood waters.
4. Damage from cattle, horses, and trespassers destroying the banks, channel, and dams by walking over them.
5. Injury or destruction to the headworks, regulators, escapes, or wasteways by floods.
6. Incendiarism.
7. Decay in timbers forming structures.
8. Destruction of earth banks due to burrowing by gophers.
9. Injury from growth of weeds or water plants choking the channel, and thus diminishing its discharge.

The first and second causes of impairment may be diminished by the use of intelligent engineering skill in the alignment and construction of the canals, and by the vigilance of patrols in discovering indications of erosion and rectifying them. If the amount of sediment deposited is large, it will have to be removed by dredges or scrapers, and such changes will have to be made in the headworks or slope of the canal or by the insertion of flushing escapes as to rectify them. Little injury should be caused the outer banks of the canal by storm waters if the canal is properly aligned and ample provisions made for the passage of drainage channels. Injury due to rain falling on the banks may be reduced to a minimum by the encouragement of the growth of grass and trees.

Damage to the canal from the fifth and seventh causes may be provided against in the construction by building the structure of some permanent material as masonry or iron, and during operation by proper supervision and repairs of the weakened part. Much damage may result from the burrowing of gophers and moles. This can only be prevented by careful supervision, the discovery of the holes, and the destruction of the pests. The discharge of a canal may be considerably reduced by the growth of aquatic plants and willow along the banks. This is to be prevented only by pulling up or mowing the brush or by destroying it by fire when the canal is empty.

309. **Inspection.**—In order that the supervision and inspection of works may be properly performed, the canal line should be divided into a number of sections, each of which should be patrolled by a ditch rider, while the whole should be in charge of a superintendent. Where the line is long, telephone communication should be had from each section to the main office of the engineer and superintendent. In addition to this piles of lumber or other building material should be placed at each bridge, escape, or other work on a canal, and by this means any damage inflicted to the property by whatever cause may be immediately repaired by the patrol, or he may telephone to headquarters for further assistance and proper advice. The length of a division of the patrol should be regulated by the number of irrigation outlets and the character of the works, and they should be of such length that every portion can be visited daily.

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