A. H. Blanchard
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MANUAL

OF

IRRIGATION ENGINEERING.

BY

HERBERT M. WILSON, C.E.

SECOND EDITION, REVISED AND ENLARGED.

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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.

PREFACE TO SECOND EDITION.

THE gratifying reception accorded the first thousand of this book has suggested the desirability of thoroughly revising it in view of the necessity of printing a second edition. The original edition was the first attempt made in this country at a general treatise of the engineering phase of irrigation. Since its publication the advances in the engineering practice of irrigation in this country have been so rapid as to materially change our knowledge of some of the principles underlying the proper design and construction of irrigation works.

While the present revision has been so thorough as to affect every portion of the book, it has really consisted rather in additions to the subjects already treated and the introduction of new subjects than in material changes of the original text. The statements of underlying principles and the general subject of hydrography contained in Part I have been more changed in minor detail than has the latter part of the book relative to construction. Contrary to the original intention, as stated in the preface to the first edition, it is now deemed desirable to extend Chapter VI, on Flow and Measurement of Water in Open Channels, by adding such theoretical considerations and tables as will render it more useful to the practising engineer. The same is true of Chapter XIV, in which the subjects of the conveyance of water through pipes and the underlying principles of flow in pipes, and tables for computation of pipe dimensions, have been amplified, though the original intention to exclude theoretical and introduce only such practical mathematical formulas as are indispensable has been adhered to. Chapter VII, on Subsurface Water Sources, has been almost completely rewritten, owing to the addition of much new matter relative to artesian wells and the water supplies therefrom, and the use of sewage in irrigation. Chapter IX has been enlarged in several directions, chiefly by the addition of detailed instructions relative to methods of survey, and Chapters V and XIV on the Duty and the Application of water have been materially changed, owing to the development of later practices in these regards.

Additional descriptions and illustrations of canals and of dams have been inserted as examples of the most recent works, and I am indebted to the courtesy of Mr. Wm. Hammond Hall for information relative to the Santa Anna canal; to Mr. A. Fteley for information relative to dams in the Croton watershed; and to Mr. A. J. Wiley for information relative to the Bruneau river dam. The most radical change and most extensive addition to this edition is in the last chapter, that on pumping, which, in view of the importance which pumping for irrigation has assumed in recent years, has been entirely rewritten in order to keep it abreast of the times. In the preparation of this chapter I am indebted chiefly to the works of Messrs. H. T. Bovey, W. M. Barr, P. R. Bjorling, and A. R. Wolff for suggestions and data.

In addition to those already mentioned, I am especially indebted to the admirable work of the late Mr. P. J. Flynn, on "Irrigation Canals and other Irrigation Works," for practical formulas and tables relative to the flow of water in open and closed channels. Permission to use these was extended to me by Mr. Flynn at the time of the preparation of his book, in exchange for similar favors from myself. I have also received valuable suggestions and hints from Mr. R. B. Buckley's "Irrigation Works in India," which has recently been published. The various lists of "Works of Reference" which are published at the ends of the different chapters contain the titles of nearly all those works to which engineers are referred for further details on the subjects treated. From nearly all of these some suggestion or hint has been obtained, and it is hoped that this general reference to them will suffice to cover any credit due their authors for suggestions which they may have conveyed to me. H. M. W.

WASHINGTON, D. C., December, 1896.

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CHAPTER I.

INTRODUCTION.

1. Meaning of Irrigation.—The word irrigation implies a condition far more imposing than is intended. In dry weather you take a watering-pot and sprinkle such plants and flowers as you consider most valuable, or perhaps with a hose or water-barrel moisten more or less of your garden truck. This is irrigation pure and simple. The only difference between this form and that more generally implied by the word irrigation as used in the arid West is that in the latter region the application of water to crops becomes a business by itself, and the farmer and the engineer unite in the employment of methods whereby water may be applied in the easiest, least expensive, and most certain manner. This is by aid of the action of gravity, and irrigation by natural flow is the result. Ditches are constructed which lead the water from the source of supply, be it well, reservoir, or stream; and they are so aligned and graded that the water shall flow through these and from them into minor channels, and from these again be led by ploughed or drilled furrows through the fields.

The mistake is too commonly made of regarding the work of irrigation as a hardship and the necessity for it as a misfortune. In point of fact, the necessity for irrigation and the ability to irrigate make a fortunate combination of circumstances. They imply a warm, dry climate, as that of the arid regions; and this means that the crops are not liable to destruction by sudden, violent storms or by the lack of sufficient sunshine or by the failure of water-supply, as sometimes results from dependence upon rainfall alone. All of this fortunate combination is not found in the semi-humid region, where the rainfall is considerable and generally sufficient for the maturing of crops. As a result there is not the ever-present sunshine and immunity from damaging storms, yet here irrigation may fulfil one of its most important functions—that of helping Nature through the drought periods, or, in other words, that of an insurance on the crops.

2. **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 40,000,000 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.

The works which provide water for the irrigation of the 40,000,000 acres above specified represent an investment of about $450,000,000, and the area thus rendered culturable yields annually products valued at about $500,000,000. This represents an interest on the original investment which seems absurd, but in fact it means only that the yield of irrigated crops averages about $12.50 per acre controlled.

3. **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.

4. **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 but an intermittent demand for water for irrigation, and consequent irregular return 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, irrigation works are designated as " protective " (against famine), and the revenue derived from them is insufficient to pay interest and working charges. 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.

5. 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 the year, and may save vast sums which would otherwise be lost by the destruction of crops.

6. Cost and Returns of Irrigation.—The table on p. 5, 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. 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 and 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.

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 per Annum.
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

PART I.

HYDROGRAPHY.

CHAPTER II.

PRECIPITATION, RUNOFF, AND STREAM-FLOW.

7. **Relation of Rainfall to Irrigation.**—In a region where the climate and soil are favorable to the production of crops, the necessity for irrigation depends on the available amount of precipitation, a quantity which cannot be judged, however, from the total annual precipitation. Where the precipitation is less than 20 inches per annum, irrigation is generally assumed to be necessary, and our arid region is usually considered as including that portion of the country in which the annual precipitation is below 20 inches, or as including most of the country west of the 97th meridian of longitude. This, however, is not a safe gauge in all cases. In Italy, where the annual precipitation averages about 40 inches, irrigation is necessary, because most of this occurs during the winter months or at other times than the agricultural or cropping season. In India the rainfall is in places as high as 100 to 300 inches per annum, and 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 or 10 inches. Generally speaking, the cropping season in the arid West may be taken as occurring between April and August, inclusive, and this is the driest season in the year.

For convenience in referring to the lands of the United States, in irrigation parlance those of the extreme West are usually called "arid;" those between the Mississippi valley and the Rocky Mountains, where the rainfall is occasionally sufficient to mature crops, are spoken of as "semi-humid;" and the lands to the east of this are usually referred to as "humid," being those on which rainfall is always sufficient for the production of crops. This distinction is to a certain extent arbitrary, and is based largely upon the amount of mean annual precipitation. The true distinction between arid and humid regions is dependent upon the amount of precipitation during the crop-growing season. In general terms, the humid portion of the United States may be considered that in which the precipitation during the cropping season is between 10 and 15 inches, dependent upon the character of the soil and other modifying circumstances.

8. 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.

9. **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.

10. **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, 2½ 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 23½ 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 1890, this river increased to a flood discharge of 140,000 second-feet, and in the spring of 1891, as the result of a still greater cloud-burst, its discharge reached the enormous figure of nearly 300,000 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.

11. **Suddenness of Great Storms.**—Statistics showing the rainfall in 24 hours are often insufficient to give a safe

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

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 Prov-

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
COLORADO:			
Greeley	4750	13.41	9.16
Breckenridge	9524	28.25	....
Leadville	10200	11.56	....
Pike'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

inces of India. On the 13th of September, 1884, 16 inches fell; on October the 1st 22 inches, on the 2d 22½, on the 3d 18 inches, and on the 4th 17½ 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.

12. Precipitation on River Basins.—Table II, giving the 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.

13. 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.

14. 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

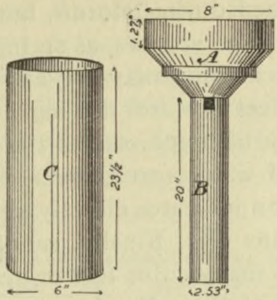

FIG. 1.—RAIN-GAUGE.

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.

15. **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.

16. 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 headwaters of the streams where the slopes are steep and perhaps rocky the rate of runoff will be higher. The coefficient of runoff increases with the rainfall. Thus in humid regions where the rainfall is greatest the rate of runoff is highest.

17. Relations of Rainfall to Runoff.—The following diagram (Fig. 2) was prepared by Mr. F. H. Newell, and gives graphically perhaps the best means available of obtaining the average runoff due to the average precipitation. The relation between these two quantities, as already stated, changes with various conditions, and is chiefly influenced by the topography.

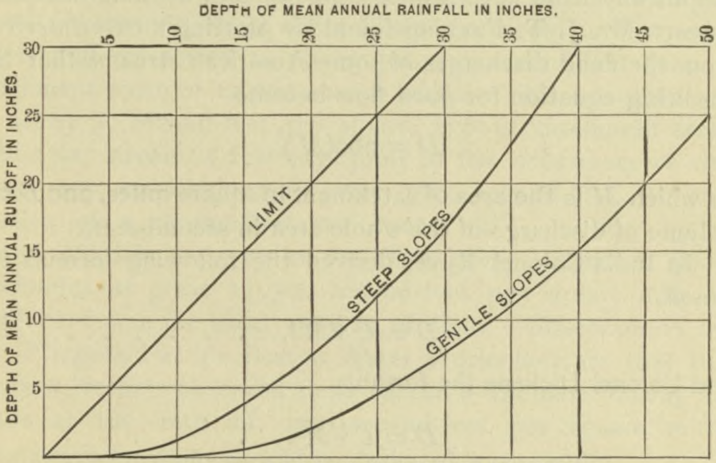

FIG. 2. RELATION OF RUN-OFF TO RAINFALL.

The heights above the base represent the depths in inches of the mean annual runoff, and the distance from left to right the depths of rainfall upon the surface of the drainage basin. The diagonal line represents the limit when all of the rain, falling as upon a smooth, steeply sloping roof, runs off; the horizontal base represents the limit where none of the water flows away. Between these, the lower of the two curved lines represents the conditions prevailing in a catchment basin of broad valleys and gentle slopes, from which the amount of runoff is relatively small; and the upper curve an average condition in mountainous regions, from which the amount of runoff is relatively large. For example, with a rainfall of 35 inches on a mountainous catchment basin the runoff is about 23 inches, while for a rainfall of 35 inches on a slightly inclined or undulating catchment basin the runoff amounts to about 11 inches.

18. **Formulas for Maximum Runoff.**—The maximum discharge from the catchment basin tributary to a reservoir is a factor of great importance in designing its dam or waste-way. Several formulas for ascertaining the maximum 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

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

in which M is the area of catchment 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 values of C have been determined for Dickens's formula:

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.

19. **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 300,000 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. Sudden great storms (Article 11) may in the West cause maximum flood discharges as great as 300 second-feet per square mile of catchment area for short periods of time. Observations by Mr. Fitzgerald at the Boston Water Works indicate that the greatest freshets observed there caused a discharge during 24 hours at the rate of 150 second-feet per square mile. Wasteways for dams should be designed accordingly.

20. Amounts of Stream Discharge and Runoff.—The table on p. 18, derived from observations published by Mr. F. H. Newell and extending over several years, shows the discharge and amounts of runoff from the catchment basins of the more important streams of the arid region.

A study of this table will show that the average daily amount of water which may be furnished by a catchment basin for storage, varies between 0.1 and 3.5 acre-feet per square mile of catchment area. These figures correspond rather closely with those obtained at the Boston Water Works and at San Francisco for approximately corresponding amounts of rainfall.

21. Discharge in Seasons of Minimum Rainfall.—Where the number of storage basins is limited it becomes desirable to

TABLE IV.

STREAM DISCHARGE AND RUNOFF.

River Basin.	State.	Observing Station.	Altitude.	Drainage Area.	Discharge. Maximum.	Discharge. Minimum.	Discharge. Mean Annual.	Discharge. Total Annual.	Runoff. Depth. Max.	Runoff. Depth. Min.	Runoff. Depth. Mean Annual.	Runoff. Per Square Mile per Annum.
			feet.	sq. mi.	sec.-ft.	sec.-ft.	sec.-ft.	acre-ft.	inches.	inches.	inches.	sec.-ft.
W. Gallatin.	Mont.	Spanish Creek.	5380	850	7,000	280	975	710,000	5.48	0.39	15.70	1.15
Madison.	"	Red Bluff.	5020	2,085	6,420	910	1,930	1,430,000	2.67	0.55	12.60	0.93
Missouri.	"	Craig.	3028	17,615	28,650	1740	5,100	3,800,000	1.34	0.14	4.00	0.27
Yellowstone.	"	Horr.	5120	2,700	15,500	285	2,880	2,180,000	4.17	0.12	15.12	1.11
Arkansas.	Colo.	Canyon City.	5330	3,060	4,750	180	814	570,000	1.20	0.08	3.80	0.27
Rio Grande.	"	Del Norte.	7865	1,400	5,900	200	1,040	755,000	3.57	0.21	10.12	0.74
Rio Grande.	N. M.	Embudo.	5806	7,000	8,555	130	1,280	920,000	0.98	0.02	2.70	0.18
Rio Grande.	Texas.	El Paso.	3697	30,000	16,620	0	1,490	1,090,000	0.45	. . . .	0.70	0.05
Salt	Ariz.	Phœnix.	. . .	12,260	143,290	320	3,170	2,297,000	. . . .	. . . .	3.50	0.26
East Carson.	Nev.	Rodenbah.	. . . .	415	5,540	290	760	550,000	7.38	1.00	25.00	1.84
Bear.	Utah.	Colliston.	. . . .	6,000	8,220	340	2,210	1,600,000	1.52	0.06	5.00	0.37
Weber.	"	Devil Gate.	. . . .	1,600	7,280	100	980	690,000	3.26	0.13	8.07	0.59
Provo.	"	Provo.	4456	640	2,340	144	519	376,000	3.47	0.26	11.01	0.81
Sevier.	"	Leamington.	4674	5,595	2,330	35	485	350,000	0.35	0.01	1.15	0.09
Snake.	Idaho.	Idaho Falls.	4714	10,100	54,300	2250	9,380	6,870,000	4.56	0.20	12.72	0.94
Weiser.	"	Weiser.	2125	1,670	11,220	80	1,211	877,000	. . . .	. . . .	9.85	0.72
Owyhee.	Ore.	Rigsby.	. . . .	9,875	18,000	170	2,114	1,536,000	1.53	0.02	2.90	0.21
Sacramento.	Cal.	Collinsville.	0	26,187	160,000	5050	37,630	26,000,000	. . . .	. . . .	16.32	1.20
Cosumnes.	"	Live Oak.	150	580	. . . .	. . . .	1,234	914,000	. . . .	. . . .	29.52	2.18
Tuolumne.	"	Modesta.	90	1,500	22,900	130	2,685	1,960,000	. . . .	. . . .	22.56	1.64
San Joaquin.	"	Hamptonville.	20	1,637	59,800	260	3,074	2,220,000	. . . .	. . . .	25.44	1.84
Kern.	"	Foothills.	. . . .	2,345	4,070	145	1,110	650,000	. . . .	. . . .	5.22	0.38

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.

22. **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 catchment 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. 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.

23. **Mean Discharge of Streams.**—When definite data of the annual discharge of a stream are not available they 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 17, the proportion of rainfall which runs off varies between 30 and 80 per cent, according as the slopes are flat or steep, wooded or barren. The mean discharge ranges between 0.5 and 2.0 second-feet per square mile of catchment area.

24. **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 III.

EVAPORATION, ABSORPTION, AND SEEPAGE.

26. **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 V and VI show this to be the case and that in the same latitude evaporation differs greatly in amount according to the altitude.

27. **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. 3, so

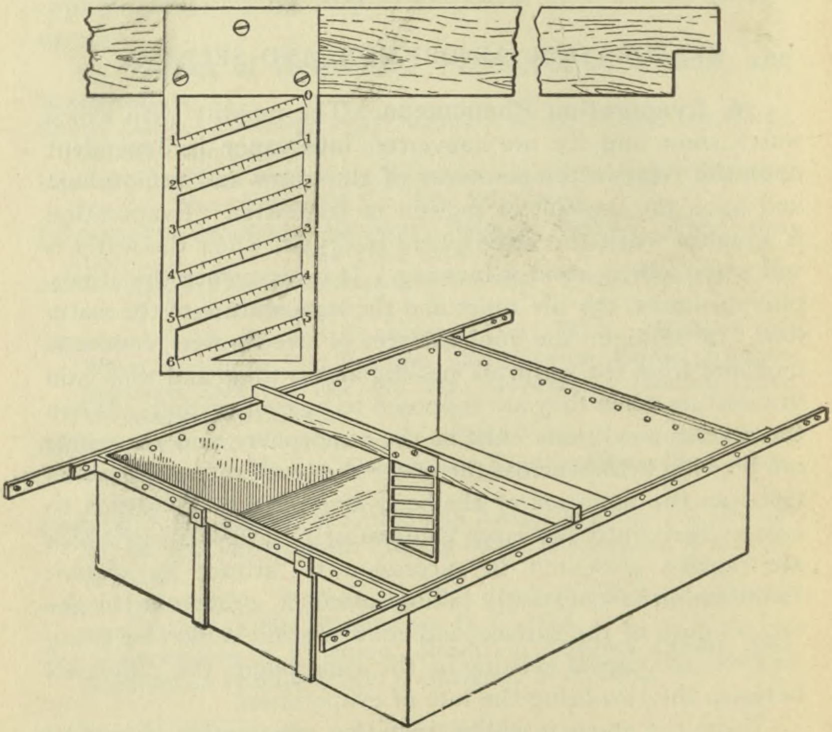

FIG. 3.—EVAPORATING-PAN.

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 its rim 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.

28. Amount of Evaporation.—In Table V 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 V.

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.

TABLE VI.

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									10.5	5.7	4.9	1.0	
1890	Salt Lake City,					3.7	4.1	5.1	7.6	6.5	4.6	2.1	1.2	
1891	Utah					3.2	4.8	5.2	7.6	6.5	5.2	2.5	1.4	
1892		40.0	1.0	1.5	2.1	2.3	4.1	5.3	6.5	7.3	5.2	2.1	1.6	1.1
1889	Nephi and Provo							3.9	5.0	4.6	2.9	3.3		
1889	Cherry Creek, Colo							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. Colo										7.2			
1889	Embudo, New Mexico		2.9	3.6	4.9									
1889							10.9	10.7	9.6	11.4	9.2	6.8	4.6	2.9
1890	Fort Bliss, near El		2.0	2.0	7.0	7.3	10.8	11.7	9.6	7.6			3.7	3.0
1891	Paso, Texas.		2.7	2.9	5.5	7.4								
1892		91.6	2.4	3.2	6.0	7.5	10.0	13.0	12.5	11.9	9.2	6.8	4.2	2.9
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			
1809	Little Yosemite "										6.2			

29. **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.

30. **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 17.9 inches and from sand 3.7 inches.

31. 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.

32. 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.

33. **Absorption.**—As here considered, absorption is the resultant or total loss due to the combined action of evaporation and 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; hence the loss by absorption is greater when a canal is in cutting than when in embankment. If the extent of the absorbing medium be limited by a bed of clay placed under either the reservoir or canal in which perculation 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.

34. 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 latter 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 but little more than that of evaporation alone.

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. Experiments made on Indian canals where the climate and soil are similar to our own, show that the loss by evaporation alone on medium-sized canals is about 5 per cent of the probable discharge, showing that the greater portion of the loss by absorption is from percolation.

35. 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.

36. **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 as 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.

37. **Amount of Seepage Water.**—The State Engineer of Colorado has conducted measurements to determine the amount of seepage water returned to the South Platte and Cache la Poudre rivers during the years 1890 to 1893, inclusive. These measurements show a constant increase in the amount of seepage water returned to these streams and available for diversion by various canals below the points of measurement.

On the South Platte River, in a distance of 397 miles, the entire gain from seepage water was, in 1893, 573 second-feet, or a gain of 430.5 per cent over that in the river at the upper measuring station. In other words, several times the amount which was diverted from the river was returned to it through seepage from the surrounding country and the water which was distributed to the land from the ditches. On the same portion of the river the percentage of increase in 1891 was 300, or three times the flow at the first measuring station. In 1891 the average increase from seepage on the South Platte was 3.24 second-feet per mile. In 1893 it was 4.0 second-feet per mile, but was as great in one mile as 13.7 second-feet, varying between these limits.

On the Cache la Poudre, experiments made in 1889 show that while the original discharge at the canyon was 127.6, the volume at a point considerably lower down the stream had increased to 214.7 second-feet, after supplying fifteen canals and without receiving additional natural drainage. Experiments on the same river during succeeding years showed similar results. The average amount of seepage water returned to the Cache la Poudre during the several years of observation was 2.4 second-feet per mile.

Investigations of a similar nature conducted by the Utah Agricultural Experiment Station and by others all point in the same direction. The amounts of returned water by seepage indicated in the above experiments must not be taken as a criterion of what may be expected in other regions. The circumstances surrounding these cases are believed to be especially favorable for the return of seepage water. It is believed that from this as almost a maximum, the amount of seepage water returned may diminish to practically nothing, dependent upon the soil, quality of underlying strata, their slope and inclination, and the area of drainage basin above and tributary to them.

Observations made at storage reservoirs for New York and Boston and some other Eastern cities, show clearly that the amount of seepage water returned from the surrounding country to reservoirs which have been drawn down for service varies between 10 and 30 per cent of their capacities. This is supposed to be largely due to the fact that the water plane of the surrounding country is filled up from the reservoir as well as from seepage from the adjacent country. Measurements of volume in the Sweetwater reservoir in Southern California show that after water ceases to be drawn from the reservoir it begins to fill up while no water is entering from streams, thus indicating that similar additions from seepage may be anticipated for Western reservoirs. As a result, the actual available capacity of a storage reservoir will probably be found to be greater than its measured capacity in spite of the losses which it sustains from evaporation and percolation. It will perhaps be more correct in designing reservoirs to assume that these gains and losses balance.

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CHAPTER IV.

ALKALI, DRAINAGE AND SEDIMENTATION.

39. **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 water-logging of the soil due to supersaturation; (3) fevers and other injurious effects, the result of the same cause.

40. **Alkali.**—The white efflorescent salt known as "alkali" is to be found in many portions of the West, both as the result of irrigation and occurring naturally over extensive areas. This salt has been analyzed and found to consist chiefly of chloride (common salt), carbonate (sal soda), and sulphate (Glauber's salt) of sodium. The relative proportions of these vary greatly, but the latter is nearly always present and predominates, ranging from 5 to 75 per cent. There is generally also present a small amount of accessory salts, as manganese sulphate and the salts of potassium. Of the latter, the nitrates and phosphates are of value, as they are the ingredients usually supplied in fertilizers. Their presence therefore indicates the occurrence of sufficient plant food in the soil to render fertilization unnecessary.

Perhaps the most harmful of the alkaline salts is sodium carbonate, commonly called "black alkali." This is more commonly found in the warmer climates and in moist close soil, rich in humus, such as is found in the San Joaquin valley of California, and it is mainly found in low ground, where the alkali occurs in spots. It is greatest in amount near the centre of the spots, while potash, on the contrary, increases in amount near the margin. The effect of alkali is to kill all vegetable matter and to render the soil barren and unproductive.

41. **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 and its evaporation from the surface. 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. Prof. E. W. Hilgard's experiments show that the main mass of alkaline salts exist in the soil within a short distance of the surface, and that the amount of these salts is limited. He therefore asserts that the bulk of alkali salts is accumulated within easy reach of underdrains, and if once removed not enough to do harm will again come from below.

42. **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 "waterlogging." 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.

43. **Prevention of Alkali and Waterlogging.**—Several preventives for the rise of alkali or the excessive soaking of the soil have been recommended, and some have been em-

ployed with success. Since evaporation is the cause of rise of alkali, the chief preventive is by reducing this to the lowest point. This may be done by mulching the soil. It is also possible in some cases to cultivate deep-rooting or such plants as shade the soil and reduce the amount of evaporation, or such as are least harmfully affected by its presence, thus mitigating the evil and permitting 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.

44. **Chemical Treatment.**—A cheap antidote for many alkaline salts is common lime, while neutral calcareous marl will answer in some cases. When the alkali consists of carbonates and borates, the best antidote is gypsum or plaster of paris. Notable experiments have been made by Prof. E. W. Hilgard, which prove the value of gypsum in neutralizing the "black alkali," or carbonate of soda. In the case of this alkali—one of the worst—mulching, deep tillage, suitable plant-growth, or any other corrective except gypsum is practically unavailing. Little benefit is to be expected from

gypsum in the case of "white" or neutral alkali, which does little harm, however, under proper tillage; but a soil heavily tainted with black alkali can be rendered profusely productive by the use, once for all, of a ton of gypsum per acre. This is more effective when applied at the rate of about 500 pounds per acre per annum in connection with some seeding at the same time, for the slightest growth aids in shading the ground and preventing an injurious release of salts by evaporation. Gypsum, however, cannot be used on alkali without water; its action must be continued for several months and through two or three seasons; it takes, moreover several weeks before immunity is secured, and therefore the dressing of gypsum should be applied in ample time before the seeding; and thereafter the soil must be well cultivated and ploughed in, and promptly followed by irrigation.

Where there is not a good natural drainage, underdrains must be provided in reclaiming alkaline soil by chemical treatment. Gypsum acts practically by converting the harmful carbonate of sodium into the less harmful sulphate of sodium in the presence of water and with the aid of thorough mixing by ploughing, and these salts are washed through the soil and are carried off into natural drainage channels, or, if the locality treated be a sink, concentrated in its bottom. A cheap form of underdrain which has been used with success consists of simple boards placed together like the letter A, at a depth of about 3 feet beneath the surface. These drains should be so laid out as to unite and discharge either into the sink hole or drainage channel. Such drains can be constructed for about $30 per acre. Mere surface treatment without drainage, in soils strongly impregnated with black alkali, would change the latter to white alkali, but would still leave too much of this in the soil for the growth of useful vegetation. It has been found that ploughing in a great deal of straw at the close of the first season and sowing the same with wheat and barley aids in hastening the reclamation of the land, since the straw keeps the surface loose and enables the grain to germinate. After three years of treatment by this process 45 bushels of oats and a similar crop of barley was produced on one of the worst alkali spots in Tulare County, California. It should be distinctly understood that wherever black alkali exists the use of stable manure to effect the loosening of the soil is harmful by setting free corrosive ammonia vapors.

45. **Mulching and Leaching.**—An excellent preventive against evaporation from the soil surface and the consequent production of alkali is by "mulching." The best mulch is a well and deep tilled surface soil, which is kept so constantly stirred that a crust is never allowed to form. As a result evaporation is reduced to a minimum, and the alkali remains distributed throughout the whole of the tilled layer instead of at the surface as a hard crust, where the bulk of the damage is done. Ploughing in large quantities of straw produces also an effective mulch. The depth or thickness of this protective tilled layer is of the utmost importance, for thereby the strong surface alkali is diluted with the largest possible mass of subsoil. After a proper tilling to a depth of, say, 10 to 12 inches, it requires a long time for the salts to come to the surface again in sufficient amount to injure the crop. As the chief desideratum in mulching alkaline land consists in maintaining a deep and loose tilth during the time when evaporation is active; this implies the growing on such lands of hoed rather than grain crops—preferably deep-rooted crops.

Leaching is not infrequently employed, more especially in Europe, to mitigate the harmful effects of alkali. This is practised by building temporary embankments around the land and then flooding it, after which the salt-impregnated waters are rapidly drawn or pumped off.

46. **Growth of Suitable Plants.**—One of the most effective plants which can be grown on slightly alkaline soil is alfalfa, which when once established brings to bear the action of deep roots and dense shade, and thus by repression of surface evaporation tends to restore the soil to its natural condition. Where mulching is practised it is desirable to grow hoed crops, such as beans, beats, potatoes, corn, onions, and canaigre, choosing preferably the deeper-rooted of these.

Experiments recently conducted by Mr. M. E. Jaffa indicate that Australian salt bush is likely to prove one of the most desirable forage plants for growth on alkali soils. It is readily eaten by stock, is rich in digestible nutrients, and has been successfully grown on alkaline land which will produce no other crop. This plant is wonderful for its productiveness and its drought-resisting power. It is prostrate in its growth, covering the ground with a green cushion 8 to 10 inches thick, and thus effectually shading it. It is perennial, and when cut soon reproduces itself from the same root. Its yield per acre is very large, being about the same as that of alfalfa.

47. **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 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 uncultivable, whereas with a proper system of irrigating channels, providing the natural drainage channels had been left open, no evil effects would necessarily have resulted.

In most regions where the slopes are very slight, as in river bottoms and where irrigation has been practised for many years, like Lombardy, Italy, and in the lower San Joaquin Valley, California, drainage becomes quite as important as irrigation, and it is extremely necessary to provide some means of conducting seepage water to the streams. In some places where the streams flow in channels above the surrounding land it is even necessary to build dikes to confine the seepage water and to pump this back to the streams.

48. 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 waste of water and 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.

49. Silt.—Great volumes of silt are transported by Western rivers in times of flood—a result chiefly of the erosion of the alluvial banks of the stream and its tributaries. The heavier sand and gravel is usually deposited in the upper reaches of he stream, and the great bulk of the silt reaching the canals is of the finest quality. As the velocity in the canals is relatively slow, much of the matter carried in suspension is deposited near their heads, or in storage reservoirs or other slack water, thus diminishing the discharge of the canal or the volume of the reservoir.

Silt consists of both organic and mineral matter; and while the former especially is often a source of advantage to the crops, it is generally a great cause of trouble in the irrigation channels. It is desirable to pass forward to the fields as much of the fertilizing matter carried in suspension as possible, and at the same time to prevent the deposition of sediment in the canal. This result is brought about by various devices in the construction of the canal headworks whereby the heavier sands and gravels are deposited just in front of diversion weirs or carried through undersluices in these, or are deposited in the upper reaches of the canals whence they can be readily flushed by sand gates and escapes, or, if no other means of removal be practicable, by dredging.

50. Character of Silt.—Silt varies greatly in its nature, depending chiefly upon the velocity of the stream and upon the soil and topography of its catchment basin. In the upper reaches of a stream where the slopes are great, the fall of the stream bed rapid and the velocity correspondingly high, the sediment moved by the water consists chiefly of bowlders, shingle, and large gravel. Much of this is not actually carried in suspension, but is merely rolled along the stream bed. Lower down on the same stream, where the velocity is 4 to 7 feet a second, the sediment consists chiefly of coarse sand or mud, the former of which is usually near the bed and is rather rolled upon it than carried in suspension. Still lower down on such a stream, or in canals the velocities of which are but 2 or 3 feet a second, only the finest silt, rich in organic matter, is carried in suspension. It is this which is desirable to retain in suspension by maintaining such velocities as will prevent its deposition until it reaches the fields, where it settles as a soft mud of high fertilizing properties.

51. Amount of Silt.—The amount of sediment which is carried in suspension during floods is greater than is usually appreciated. From investigations made by the United States 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 also estimated that in about one hundred and fifty years the amount of this sediment would seriously impair a reservoir 60 feet in depth. On the American River at Folsom, California, in a single year a depth of nearly 10 feet of wet silt was deposited in a reservoir situated at that point. Much of this, however, was heavy matter, bowlders and gravel rolled along the stream bottom by the swift current. Mr. R. B. Buckley states that the proportion of silt to water by weight, in several rivers, is as follows:

Mississippi	1 to 572
Rhône, France (velocity 8 feet per second)	1 to 45
Po, Italy	1 to 300
Rhine. Germany	1 to 100
Indus, India (velocity from $3\frac{1}{2}$ to 5 feet per second)	1 to 237

On the Soane Canal in India, which is diverted from a river having a maximum velocity of 7.5 feet per second, as much as from 3 to 5 feet of sand and coarser material is deposited in the first quarter mile of the canal, gradually diminishing in depth until but a few inches are deposited at 5 or 6 miles from its head.

As already stated, the greater proportion of the silt is heavy matter floating near or rolled along the bottom; and as shown by experiments conducted on the Rhine with a flood velocity of 8 feet per second, this silt near the bottom was as much as 88 per cent greater in amount than that at the surface. It was observed also that there is more silt in a rising flood than in a falling one, and the maximum amount of silt is carried when the flood has reached about two thirds of its height. It is because of this reason that under-sluices in storage dams (Arts. 186 and 347) are of value, as the water may be permitted to waste through these until the maximum flood height has been reached, after which they may be closed and the remaining flood waters, which are less heavily charged with silt, be stored.

52. Prevention of Sedimentation in Reservoirs and Canals.—In view of the important bearing which sediment carried in suspension has upon irrigation water, it is necessary to consider the quality and amount of sediment in the source of water supply of any irrigation work under consideration. Where a canal is taken from a stream of high velocity, which usually carries but little fertilizing material and much heavier matter, it is necessary to so design the headworks that most of this shall be excluded, and to cause the deposition of the remainder in a short distance in the upper reaches of the canal. In the case of streams of lower velocity, the problem to be solved is usually how to so align the canals and distributaries and choose their slopes and velocities, that the heavier particles which are suspended near the bed of the stream shall be deposited close to the headworks of the canal, while the lighter silt containing the fertilizing properties shall not only be carried through the canal and its branches, but shall remain in suspension until deposited in the fields.

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 may 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 head-gates 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 head-gates, the main stream may be so trained as to have a straight sweep past the head-gates, 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 all occur in the first half-mile or less of the canal, and it may be either dredged out or perhaps scoured out by an escape.

53. **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.

54. **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 waterway 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. On the Pavia Canal, Italy, the growth of aquatic plants is so rank as to require, in addition to two annual clearings, chiefly for silt, the constant use of floating cutting-machines.

55. **Malarial Effects of Irrigation.**—In numerous localities, both abroad and in the West, irrigation has been denounced as a serious menace to the health of the community

because of the creation of swamps and their malarial effects. From the most careful researches which have been made of this subject, both by a committee which reported to the Indian Government and as a result of the investigations of Dr. H. O. Orme, of the California State Board of Health, it appears that these evil effects have been largely exaggerated, and in nearly every case may be avoided, either by more sparing use of water, by proper drainage, or by abandoning irrigation in limited localities which it is impossible to properly drain. In Southern California, between Los Angeles and San Diego, where the natural drainage is of the best, the soil as a rule sandy or gravelly and open to a great depth, the water used in irrigation sinks into the ground or drains off, and the use of almost any amount of water does not produce malarial fevers. On the other hand, in such regions as the low-lying, comparatively level lands of the lower part of the Sacramento and San Joaquin valleys, where the soil is heavy, the slopes slight, and the underdrainage poor, it is undoubtedly true that irrigation has developed malarial disorders, by raising the subsurface water-plane, thus causing the water to stand about in swamps or stagnant pools. This appears to be especially true where it raises the water-levels in wells to a newer and higher stratum of earth. In such cases it appears that malarial troubles may be avoided by continuing the wells to other and deeper strata and using this water for drinking purposes rather than that gotten from the shallower well.

Malarial effects are not attributable directly to the results of irrigation where economically and properly practised, but frequently owing to the careless construction of canals works or the private channels of individuals having intercepted the natural drainage of the country, and thus led to the formation of swampy tracts. It appears certain that when care is taken to irrigate only that land which has an open soil and such slopes and natural drainage as to prevent waterlogging, no unhealthy effects will result from irrigation; also, that when malarial influences are developed by irrigation, their effect is almost strictly local.

It is desirable in order to mitigate the possible evil effects of irrigation to keep the canal as much as possible within soil so that its surface level may be low, and thus only raise the subsurface water-plane to the least height practicable; that earth wanted to complete embankments be never taken from excavations or borrow-pits except where such localities admit readily of drainage; that the canal and its branches be aligned as far as possible along the watershed of the country so as not to interfere with drainage. If wholesome water and not open ditch water be provided for domestic uses the prejudicial effects of irrigation are largely averted. In such climates as will encourage its growth it appears that the *Eucalyptus globulus* has proven beneficial in mitigating the malarial effects of irrigation waters, chiefly because of its great absorbing and transpiring power due to its rapid growth.

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CHAPTER V.

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 area of crop which it will mature. In order to determine what amount of water is sufficient to irrigate 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"—which is the amount of water which will cover one acre of land one foot in depth, that is, 43,560 cubic feet—is preferable, especially as it bears a direct relation to the unit used in defining areas cultivated. Hence the capacity of a reservoir in acre-feet expresses a direct ratio to the number of acres which it will irrigate, or its duty per acre-foot. 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.

Each method of expressing duty is readily convertible into the other, providing the irrigating period be known. The following simple formulas are given by Mr. R. B. Buckley for use in making such conversions:

D = duty of water in second-feet;
B = irrigating period per second-foot;
V = cubic feet of water required to mature one acre of crop;
S = total depth in inches of volume used if evenly distributed over area irrigated;
Q = discharge in second-feet required to irrigate a given area (A), with a given duty (D), and irrigating period (B).

$$V = \frac{B}{D} \times 86{,}400. \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (4)$$

$$S = \frac{B}{D} \times 23.8. \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (5)$$

$$Q = \frac{A}{D} = \frac{AS}{23.8B}. \quad . \quad . \quad . \quad . \quad . \quad . \quad (6)$$

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

TABLE VII.

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	=	5 1/6 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. The duty of water varies primarily with the crop; thus rice requires more than wheat. It varies still more largely with the soil, since sandy requires far more than clayey soil. It varies with the temperature and precipitation and with the condition of the ditches, as flat, shallow channels give smaller duties than steeper and deeper ones. Above all, it varies with the skill of the irrigator.

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 $33\frac{1}{3}$ per cent, the duty would be reduced to 100 acres at the point of application. Careful measurements made on various canals in India show the loss of water between their heads and the heads of the distributaries to vary from 20 to 40 per cent. Where duty on discharge at canal head was 53 acres, that on its discharge utilized was 72 acres; while the duty of the distributaries on their discharge at outlet was 104 acres per second-foot.

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, varying on the supply at the head from 70 to 190 acres. In Utah, 70 to 300 acres per second-foot is the 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 VIII.

DUTY OF WATER.

(Based on Supply entering Canal Head.)

Locality.	Duty per Second foot in Acres.
Northern India	60–150
Italy	65– 70
Colorado	80–120
Utah	60–120
Montana	80–100
Wyoming	70– 90
Idaho	60– 80
New Mexico	60– 80
Southern Arizona	100–150
San Joaquin Valley, Cal	100–150
Southern California, surface irrigation	150–300
" " sub-irrigation	300–500

The above figures are for duty on supply entering canal head. The duty on the supply utilized will average 20 to 40 per cent greater. 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. — Recent experiments conducted under the direction of Prof. L. G. Carpenter throw valuable light on the depth of water required for irrigation. It may be generally stated that the experience of all countries indicates that it is impossible to make an irrigation with a depth of less than 3 inches of water on sod ground and 4 to 6 inches on cultivated crops, though under certain adverse conditions of soil and crop as much as 10 inches of water may be required. 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.

62. **Quantity per Service and Irrigating Period.**—In Colorado alfalfa and clover are irrigated twice in a season, once in May and once in June, to a depth of 6 inches for each period; wheat and oats are irrigated twice, once in June and once in July, to a depth of 9 and 6 inches respectively. Meadow or native hay requires considerably more water; there are usually two service periods, each of which lasts several days, the water being allowed to run in a small quantity during that time. The first is usually in May, and is about two inches in depth for a week; the second in July or August, of about the same amount: in all from 24 to 30 inches in depth of water are applied. Since the application of water is generally followed by a temporary checking of the growth of the plant, the method preferred in the arid region seems to be to give thorough rather than many irrigations; in other words, to have two ample rather than four to six small services. In general it may be stated that two or three service periods varying in depth from 3 to 6 inches are employed in Colorado, and that the irrigation period extends from May to September—123 days. In Utah the practice seems to be to employ a much larger number of service periods,—from three to five on grain crops of 2 to 3 inches in depth each,—the water running 12 to 15 hours per service period, and the irrigation period extending from June to August, inclusive. On vegetables as many as six to ten service periods are employed, each lasting from 3 to 6 hours during June to August, inclusive. The irrigating period in the majority of Western States averages from April 15 to August 15, or about 120 days; while the service period varies from 3 to 15 hours in length according to soil and crop, and there are from two to eight such service periods in an irrigating period. In India there are from three to five service periods, making up an irrigating period of from 100 to 130 days' duration.

63. **Duty per Acre-foot.**—Assuming an average depth of 4 inches of water as sufficient to thoroughly soak the soil, this

is equivalent to $\frac{1}{3}$ of an acre-foot per acre. An average crop requires from two to four waterings per season. Assuming three as the mean, then at the above rate one acre-foot will be required per season to irrigate an acre. Practice, however, clearly indicates that this theoretic amount is too low. Experiments conducted in Wyoming indicate that 12 inches in depth for potatoes to 24 inches in depth for oats are sufficient to mature crops. In Idaho the depth of water generally used is about 24 inches, while in Montana from 15 to 18 inches is believed to be sufficient. These indicate volumes ranging from $1\frac{1}{4}$ acre-feet in Montana to 2 acre-feet in Wyoming and Idaho, as duties estimated on the amount of water entering the canal heads. Measurements on several canals in Colorado show that from 18 to 24 inches in depth of water are required, or from $1\frac{1}{2}$ to 2 acre-feet per acre. Experiments conducted by Mr. Samuel Fortier in Utah indicate that a depth of 24 inches is required for tomatoes, while potatoes yield abundantly with a depth of 17 inches, onions with a depth of 36 inches, strawberries with 27 inches, and orchards with 12 inches. In India the average duty of water on the volume entering the distributary head has been found to vary from 2 to 3 acre-feet per acre.

In estimating the duty of water stored in a reservoir, allowance must be made for the losses due to evaporation and absorption in conducting the water to the fields. As this averages 25 per cent, it follows that where a duty of 1 acre-foot per acre is possible $1\frac{1}{4}$ acre-feet must be stored in the reservoir, and where 2 acre-feet per acre is the duty $2\frac{1}{2}$ acre-feet must be stored. In estimating the total duty of the reservoir consideration must be given to the area of waste land (Art. 65), which will increase the storage duty by about 20 per cent.

64. **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 con-

venient 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.

65. 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 barn-yards, 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, entering the canal head, when estimated on large areas is at least 20 per cent greater than the theoretic duty per acre.

66. Tatils or 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 use water in as large heads and volumes as possible; in fact, the volumes of water flowing in many small distributaries are so small, that if irrigation were being practised all along its line at the same time sufficient would not reach the lower end of the distributary to moisten its bed, much less flow over the fields. It is therefore essential in such cases to divide the distributary into a number of sections, in each of which the water is permitted to flow with full head for a given period, and the irrigators are compelled to use the water at the time when it is available in their section, be it night or day.

This system of irrigating by rotation, or by "tatils," as it is called in India, is of great advantage not only in checking the loss of water in the channels and giving sufficient head to each irrigator to flow his land, but also in teaching economical irrigation to the cultivators and insuring a fair division of water among them. Thus an irrigator who is entitled to but $\frac{1}{2}$ second-foot of water during an irrigating period of 100 days would find that volume too small to be economically and practically handled; but if he were permitted to use 4 second-feet during 12 days, divided into three tatils or service periods of four days each, he would be able to make a satisfactory and economical use of such a head.

These tatils are imposed by regulating the amount admitted to the private channels and the period of time in which it shall enter them. The outlets of these channels may be closed in the first section of the canal for, say, 4 days; in the second for 3 days; and this order may be reversed, the period of rotation being such as to vary and equalize the period of closure among the several sections. It is better to impose these systems of rotation on long portions of the distributaries at the same time, since the effect of forcing water down to the tail of the distributaries is then more noticeable. Thus, if the canal be twenty miles in length and all the outlets in the first five miles be closed, those in the second five miles opened, those in the third five miles closed, and those in the fourth five miles opened all at the same time, the effect will be to produce a stronger head and carry the desired amount to all the channels in the last portion of the canal; then for a few days this order may be reversed, and the maximum duty obtained in the remaining portions of the distributaries without difficulty. To make such a system effective, rules must be enforced compelling irrigators to accept water when their irrigating heads are open and refusing it to them when their turn has gone by.

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CHAPTER VI.

FLOW AND MEASUREMENT OF WATER IN OPEN CHANNELS.

68. 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 to 5000 in a linear inch.

69. 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 decreases. 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 soluble or suspended matter in a given volume.

70. 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 above the lowest part of the column.

71. 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 $p = 62.5h$.

72. 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.

73. 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.

74. 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 v of a body at the end of a given space of time t 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 $v = \sqrt{2gh}$.

75. **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

$$v = \sqrt{\frac{2g}{m}} \times \sqrt{ri}, \quad . \quad . \quad . \quad . \quad . \quad (7)$$

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. The value of 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 for a hydraulic mean radius of 25.

76. **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 been received with 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

$$v = r\sqrt{\frac{1000i}{.08534r + 0.35}}, \quad . \quad . \quad . \quad . \quad . \quad (8)$$

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

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

$$v = \left\{ \frac{\frac{1.811}{n} + 41.6 + \frac{.00281}{i}}{1 + \left(41.6 + \frac{.00281}{i}\right) \times \frac{n}{\sqrt{r}}} \right\} \times \sqrt{ri}. \quad . \quad (9)$$

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

$$v = C\sqrt{ri} = C\sqrt{r} \times \sqrt{i}. \quad . \quad . \quad . \quad . \qquad (10)$$

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 IX.

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

Slope i.	$n = .0225$					$n = .035$				
	$\sqrt{r}$ 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

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.

As an indication of the extent to which the value of n affects the velocity and discharge of channels, let us take an example in which $n = 0.0225$. A bed width of 10 feet, depth of 2 feet, and side slopes of 1 to 1, with a grade of 8 feet per mile, gives a velocity of 3.32 feet per second and a discharge of 79.07 second-feet. For the same channel with a value of $n = .035$ the velocity is 2.05 feet per second and the discharge 49.2 second-feet; thus showing that with the better channel the discharge is 60 per cent greater than with the inferior channel.

78. Tables for Use with Kutter's Formula.—Tables X to XIV inclusive are in large part derived by condensation from Flynn's tables, and greatly facilitate the various computations of flow in open channels, in connection with Chazy's adaptation of Kutter's formulas $v = C\sqrt{ri}$ and $Q = Av$.

Table X gives the sine of the inclination $i = \frac{h}{l}$ in feet, and also $\sqrt{i}$ for given grades and slopes from one fourth of a foot per mile to 30 feet per mile. Tables XI, XII, and XIII give, respectively, the area A in square feet, the hydraulic mean depth r in feet, and the $\sqrt{r}$ for rectangular channels, and also for trapezoidal channels having side slopes of 1 on 1 and 1 on $1\frac{1}{2}$, corresponding to depths of from 1 to 10 feet and bed widths of from 3 to 100 feet. Table XIV gives the coefficients of roughness C for different values of n and for values of r from 0.1 to 10, and of i from .0001 to .01.

TABLE X.

GRADES, SLOPES, AND VALUES OF i AND $\sqrt{i}$ FOR USE IN KUTTER'S FORMULA $v = C\sqrt{ri}$.

Grade in Feet per Mile.	Slope 1 in	i	$\sqrt{i}$	Grade in Feet per Mile.	Slope 1 in	i	$\sqrt{i}$
.25	21,120	.000047	.0069	10	528	.001894	.0435
.50	10,560	.000094	.0097	11	444	.002083	.0456
.75	7,040	.000142	.0119	12	440	.002273	.0477
1	5,280	.000189	.0137	13	406	.002462	.0496
1.25	4,224	.000236	.0154	14	377	.002651	.0515
1.5	3,520	.000284	.0168	15	352	.002841	.0533
1.75	3,017	.000331	.0182	16	330	.003030	.0550
2	2,640	.000378	.0194	17	311	.003219	.0567
2.25	2,347	.000426	.0206	18	293	.003409	.0584
2.5	2,112	.000473	.0217	19	278	.003598	.0599
2.75	1,920	.000521	.0228	20	264	.003788	.0615
3	1,760	.000568	.0238	21	251	.003977	.0630
3.25	1,625	.000615	.0248	22	240	.004166	.0645
3.5	1,508	.000663	.0257	23	229	.004356	.0660
3.75	1,408	.000710	.0266	24	220	.004545	.0674
4	1,320	.000757	.0275	25	211	.004735	.0688
5	1,056	.000947	.0307	26	203	.004924	.0702
6	880	.001136	.0337	27	195	.005113	.0715
7	754.3	.001325	.0364	28	188	.005303	.0728
8	660	.001515	.0389	29	182	.005492	.0741
9	586.6	.001704	.0413	30	176	.005682	.0754

TABLE XI.

VALUES OF A IN SQUARE FEET, r IN FEET, AND $\sqrt{r}$ FOR CHANNELS HAVING VERTICAL SIDES.

For Use in Kutter's Formula $v = C\sqrt{ri}$ and $Q = Av$.

Depth in Feet.	Bed-width, 3 Feet.			Bed-width, 5 Feet.			Bed-width, 10 Feet.		
	A	r	$\sqrt{r}$	A	r	$\sqrt{r}$	A	r	$\sqrt{r}$
1	3	.600	.774	5	.714	.845	......	......	......
1.5	4.50	.750	.866	7.5	.937	.968	......	......	......
2	6	.857	.926	10	1.111	1.054	20	1.429	1.195
2.5	7.50	.937	.967	12.5	1.250	1.118	25	1.666	1.290
3	9	1	1	15	1.364	1.168	30	1.875	1.396
3.5	......	......	......	17.5	1.458	1.208	35	2.058	1.434
4	......	......	......	20	1.538	1.241	40	2.222	1.490
4.5	......	......	......	......	......	......	45	2.367	1.538
5	......	......	......	......	......	......	50	2.5	1.581

Depth in Feet.	Bed-width, 20 Feet.			Bed-width, 40 Feet.			Bed-width, 60 Feet.		
	A	r	$\sqrt{r}$	A	r	$\sqrt{r}$	A	r	$\sqrt{r}$
3	60	2.307	1.518	......	......	......	......	......	......
4	80	2.857	1.690	160	3.333	1.826	240	3.529	1.878
4.5	90	3.105	1.762	180	3.672	1.916	270	3.913	1.978
5	100	3.333	1.825	200	4	2	300	4.286	2.073
5.5	110	3.553	1.885	220	4.314	2.077	330	4.646	2.155
6	120	3.750	1.937	240	4.614	2.148	360	5	2.236
6.5	......	......	......	......	......	......	390	5.343	2.311
7	......	......	......	280	5.180	2.276	420	5.676	2.382
7.5	......	......	......	......	......	......	450	6	2.450
8	......	......	......	320	5.714	2.394	480	6.316	2.513
9	......	......	......	360	6.207	2.491	540	6.923	2.963
10	......	......	......	......	......	......	600	7.500	2.738

TABLE XII.

VALUES OF A IN SQUARE FEET, r IN FEET, AND $\sqrt{r}$ FOR CHANNELS HAVING SIDE SLOPES OF 1 ON 1.

For Use in Kutter's Formula $v = C\sqrt{ri}$ and $Q = Av$.

Depth in Feet.	Bed-width, 3 Feet.			Bed-width, 5 Feet.			Bed-width, 10 Feet.		
	A	r	$\sqrt{r}$	A	r	$\sqrt{r}$	A	r	$\sqrt{r}$
1	4	686	828	6	.766	.875	......	......	....
1.5	6.75	932	965	9.75	1.054	1.027	......	......	....
2	10	1.155	1.075	14	1.314	1.147	24	1.533	1.238
2.5	13.75	1.365	1.168	18.75	1.553	1.246	31.25	1.831	1.353
3	18	1.567	1.252	24	1.780	1.334	39.00	2.110	1.452
3.5	......	....	....	29.75	1.997	1.413	47.25	2.375	1.541
4	......	....	....	36	2.207	1.486	56	2.628	1.621
4.5	......	....	....	......	....	....	65.25	2.871	1.694
5	......	....	....	......	....	....	75	3.107	1.763

Depth in Feet.	Bed-width, 15 Feet.			Bed-width, 20 Feet.			Bed-width, 40 Feet.		
	A	r	$\sqrt{r}$	A	r	$\sqrt{r}$	A	r	$\sqrt{r}$
2	34	1.646	1.283	......	....	....	......	......	....
3	54	2.300	1.516	69	2.422	1.556	......	......	....
3.5	64.75	2.601	1.612	......	....	....	......	......	....
4	76	2.888	1.700	96	3.066	1.751	176	3.431	1.852
4.5	87.75	3.165	1.779	110.25	3.369	1.835	200.25	3.798	1.949
5	100	3.431	1.852	125	3.661	1.913	225	4.155	2.038
5.5	......	....	....	140.25	3.944	1.986	250.25	4.504	2.122
6	126	3.941	1.985	156	4.220	2.054	276	4.844	2.201
7	......	....	....	......	....	....	329	5.501	2.343
8	......	....	....	......	....	....	384	6.132	2.476

Depth in Feet.	Bed-width, 60 Feet.			Bed-width, 80 Feet.			Bed-width, 100 Feet.		
	A	r	$\sqrt{r}$	A	r	$\sqrt{r}$	A	r	$\sqrt{r}$
4	256	3.590	1.895	......	....	....	......	......	....
5	325	4.384	2.095	425	4.514	2.125	525	4.600	2.145
6	396	5.145	2.268	516	5.321	2.307	636	5.437	2.331
6.5	432.25	5.515	2.348	562.25	5.715	2.391	692.25	5.848	2.418
7	469	5.877	2.424	609	6.102	2.470	749	6.252	2.500
7.5	506.25	6.234	2.497	656.25	6.484	2.546	806.25	6.652	2.579
8	544	6.584	2.566	704	6.860	2.619	864	7.046	2.654
8.5	582.25	6.928	2.632	752.25	7.230	2.689	922.25	7.435	2.726
9	621	7.267	2.696	801	7.595	2.756	981	7.819	2.796
10	700	7.929	2.816	900	8.312	2.883	1100	8.575	2.928
11	......	....	....	1001	9.009	3.001	1221	9.313	3.051
12	......	....	....	......	....	....	1344	10.03	3.167

Table XIII.

VALUES OF A IN SQUARE FEET, r IN FEET, AND $\sqrt{r}$ FOR CHANNELS HAVING SIDE SLOPES OF 1 ON 1½.

For Use in Kutter's Formula $v = C\sqrt{ri}$ and $Q = Av$.

Depth in Feet.	Bed-width, 3 Feet.			Bed-width, 5 Feet.			Bed-width, 10 Feet.		
	A	r	$\sqrt{r}$	A	r	$\sqrt{r}$	A	r	$\sqrt{r}$
1	4.50	.681	.83	6.5	.755	.87	......	......	......
1.5	7.87	.935	.97	10.87	1.045	1.02	......	......	......
2	12	1.175	1.08	16	1.310	1.15	26	1.510	1.23
2.5	16.87	1.405	1.19	21.87	1.560	1.25	34.375	1 807	1.34
3	22 50	1.628	1.28	28.5	1.802	1.34	43.5	2.090	1.44
3.5	......	......	......	35.87	2.036	1.43	53.375	2.358	1.54
4	......	......	......	44	2.266	1.51	64	2.620	1.62
4.5	......	......	......	......	......	......	73.375	2.873	1.70
5	......	......	......	......	......	......	87.5	3.121	1.77

Depth in Feet.	Bed-width, 20 Feet.			Bed-width, 40 Feet.			Bed-width, 60 Feet.		
	A	r	$\sqrt{r}$	A	r	$\sqrt{r}$	A	r	$\sqrt{r}$
3	73.50	2.386	1.54	......	......	......	......	......	......
4	104	3.021	1.73	184	3.399	1.84	264	3.547	1.88
4.5	120 37	3.332	1.82	210.37	3.742	1.93	300.37	3.941	1.99
5	137.5	3.615	1.90	237.50	4.094	2.03	337.50	4.325	2.08
5.5	155.37	3.901	1.97	265.37	4.435	2.11	375.37	4.702	2.17
6	174	4.179	2.04	294	4.770	2.18	414	5.071	2.25
6.5	......	......	......	323.4	5.097	2.26	453.37	5.434	2.33
7	......	......	......	353.5	5.418	2.33	493.50	5.789	2.40
7.5	......	......	......	......	......	......	534.37	6.139	2.47
8	......	......	......	416	6.043	2.46	576	6.483	2.54
9	......	......	......	481.5	6.646	2.58	661.50	7.155	2.67
10	......	......	......	......	......	......	750	7.808	2.79

Table XIV.

VALUES OF C FOR GIVEN SLOPES, i, AND HYDRAULIC MEAN RADII, r, IN FEET.

	r	Coefficients of Roughness for n =							
		.009	.010	.012	.015	.020	.025	.030	.035
	Feet.	C	C	C	C	C	C	C	C
	.1	110	95	74	54	36	27	21	17
	.2	130	114	90	67	46	34	27	22
Slope $i = .01$ =	.4	151	133	107	82	57	44	35	29
	.6	162	143	116	90	64	49	39	33
	.8	170	151	123	95	68	53	43	35
1 in 100 =	1	175	156	128	99	72	56	45	38
	2	191	171	142	112	83	66	55	46
52.8 feet per mile.	3	199	179	149	119	89	71	59	51
	4	204	184	154	123	93	76	63	55
	6	210	190	160	129	99	81	68	59
	10	217	196	166	136	105	86	74	65
	.1	110	94	73	54	36	27	21	17
	.2	129	113	89	66	45	34	27	22
	.4	150	131	105	80	56	43	34	28
Slope $i = .001$ =	.6	161	142	115	88	63	48	39	32
	.8	169	150	122	94	68	52	42	35
1 in 1000 =	1	175	155	127	99	71	56	45	38
	2	191	171	142	112	83	66	54	46
5.28 feet per mile.	3	199	179	149	119	89	71	59	51
	4	204	184	154	124	93	75	63	54
	6	211	190	160	130	99	81	68	59
	10	218	197	167	136	105	87	74	65
	.1	104	89	69	50	34	25	19	16
	.2	126	110	87	65	44	32	25	21
	.4	148	129	104	79	55	42	33	27
Slope $i = .0004$ =	.6	157	140	113	87	62	47	38	31
	.8	166	148	121	93	67	51	42	35
1 in 2500 =	1	172	154	125	98	70	55	45	37
	2	190	170	141	112	83	65	54	45
2.11 feet per mile.	3	199	179	149	119	89	71	59	51
	4	204	184	154	124	94	76	63	55
	6	211	191	161	130	99	81	69	60
	10	219	199	168	138	107	88	75	66
	.2	....	....	76	57	39	29	23	19
	.4	....	....	95	72	50	38	31	25
	.6	....	....	105	81	57	44	35	30
Slope $i = .0001$ =	.8	....	....	114	88	63	48	39	33
	1	....	....	120	93	67	52	42	35
1 in 10,000 =	2	....	...	138	109	81	64	53	45
	3	....	....	149	119	89	71	59	51
.528 feet per mile.	4	....	....	155	125	94	76	64	55
	6	....	....	164	134	102	84	71	61
	10	....	....	174	143	111	92	78	69
	15	....	....	181	150	118	98	85	75

79. 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,

$$Q = Av, \qquad (11)$$

or, substituting for v its value from equation (10),

$$Q = C \times A \times \sqrt{r} \times \sqrt{i}. \qquad (12)$$

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.

80. Surface and Mean Velocities.—The surface velocity is that which is most easily obtained by simple methods. Numerous experiments have been made by Du Buat, Francis Brunning, Humphrey and Abbott, and others, to obtain the ratio of the mean to the surface velocity. From these it has been found that this ratio varies chiefly between $v = .780V$ and $v = .920V$, in which $v =$ the mean velocity of the entire cross-section of the channel and $V =$ its central or maximum surface velocity. This ratio varies with the section of the channel and the roughness of its sides, as well as with the depth. It is found that the ratio of v to V should be at a maximum when the breadth equals twice the depth, also, for several sections, for that which is largest. When breadth is equal to three times the depth, $v = .91V$; when breadth is equal to five times the depth, $v = .88V$; when breadth is equal to eight times the depth, $v = .83V$.

81. 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. For increased accuracy several passings of the float over this distance should be measured, or, better still, the time of passing of floats over several different lengths of 100 feet should be determined. The mean of these observations will give the central or maximum surface velocity, which multiplied by the proper ratio above will give the mean velocity v. The mean surface velocity may be obtained by throwing in a number of floats on different portions of the surface of the stream and timing their passage over a fixed distance. The resulting velocity per second multiplied by .8 will give approximately the mean velocity of the entire stream cross-section.

The velocity of a stream may be ascertained with still greater accuracy by determining the mean velocity, not of the surface as above, but of the entire body of the stream, by timing upright rods so weighted that their bottoms shall float within a few inches of the bed of the channel. These rods may be either of wood or of tin, of uniform size, and may be jointed so as to be of convenient use in different depths of water. In using the Pitot tube the stream should be cross-sectioned as in other velocity determinations, and the mean velocities may then be ascertained by the tube for various sections in the channel.

82. Current Meters.—Current meters are mechanical contrivances so arranged that by lowering them into a stream the velocity of its current can be ascertained with accuracy by a direct reading of the number of revolutions of a wheel, and a comparison of this with a table of corresponding velocities. Various forms of current meters have been designed and used, the three 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; the

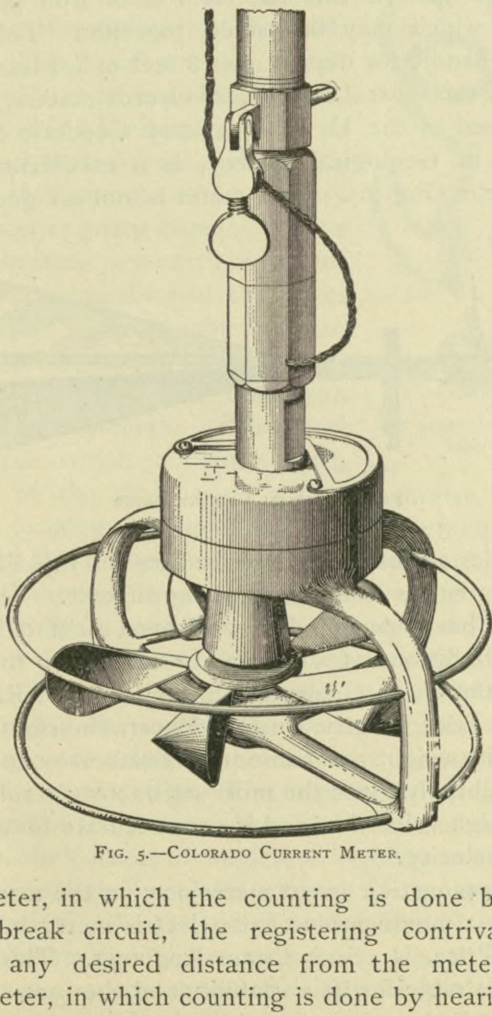

FIG. 5.—COLORADO CURRENT METER.

electric meter, in which the counting is done by a simple make-and-break circuit, the registering contrivance being placed at any desired distance from the meter; and the acoustic meter, in which counting is done by hearing through an ear-tube the clicks made by the revolutions of a wheel and counting the same. There are several makes of current meters of nearly all these varieties.

Of direct-acting meters, that which has been found most effective in turbid waters, and is employed by the hydrographers of the U. S. Geological Survey, is the Colorado Meter (Fig. 5). In this the stem is of iron pipe, several lengths of which may be joined together. This meter is difficult to handle for depths over 8 feet or for less than 1 or 2 feet. Of the several varieties of electric meters, one, which is chiefly used in the U. S. Coast and Geodetic Survey and in the U. S. Geological Survey, is a modification of the Haskell Meter (Fig. 6). This meter is not so good for very

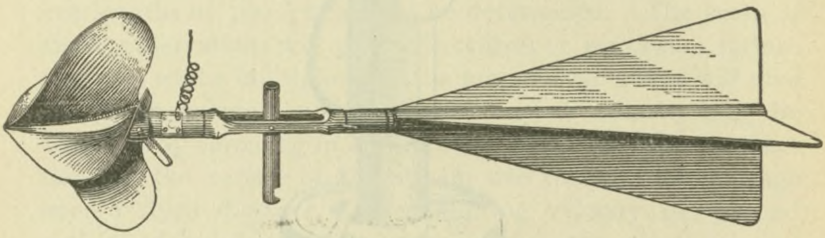

FIG. 6.—HASKELL CURRENT METER.

high velocities as that next described, as the rapidity of revolution is so great as to make counting difficult. The electric meter which has been found to work most satisfactorily under nearly all conditions of depth and velocity by the hydrographers of the U. S. Geological Survey and U. S. Engineer Corps is the Price Electric Current Meter, the chief objection to which is its weight and consequent cumbersomeness. This meter undoubtedly gives the most satisfactory results in large streams of high velocities, and is very accurate for streams of nearly any velocity.

The only acoustic current meter now on the market is the Price Meter, a modification of the electric meter invented by Mr. W. G. Price, U. S. Assistant Engineer. This meter is especially desirable for its portability and the ease with which it can be handled, as it weighs but little over a pound. In very shallow streams it gives the most accurate results of any meter, and is held at the proper depth by a metal rod in the hands of the observer, as is the Colorado Current Meter. This meter is designed especially to stand hard knocks which may be received in turbid irrigation waters, and it can be used in high velocities, as only each tenth revolution is counted. It consists (Fig. 7) of a strong wheel, composed of six conical-shaped cups, which revolve in a horizontal plane; its bearings run in two cups holding air and oil in such manner as to entirely exclude water or gritty matter. Above the upper bearing is a small air-chamber, into which the shaft of the wheel extends. The water cannot rise into this air-chamber, and in it is a small worm-gear on the shaft, turning a wheel with twenty teeth. This wheel carries a pin which at every tenth revolution of the shaft trips a small hammer against the diaphragm forming the top of the air-chamber, and the sound produced by the striking hammer is transmitted by the hollow plunger-rod through a connecting rubber tube to the ear of the observer by an ear-piece. The plunger-rod is in 2-foot lengths, and is graduated to feet and tenths of feet, thus rendering it serviceable as a sounding or gauging rod.

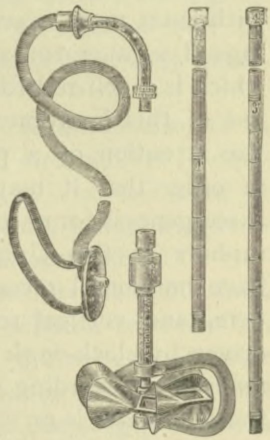

FIG. 7.—PRICE ACOUSTIC CURRENT METER.

83. **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.

Fluctuations in the heights of streams may be measured with even more accuracy than can be obtained by the readings of a gauge-rod as above described by using a Nilometer, which is a self-recording gauge. The chief objection to the use of this instrument is that its maintenance requires the the attention of a person of considerable mechanical skill in order that it may be kept in proper order. There are three general forms of Nilometer employed by the hydrographers of the United States Geological Survey. These have horizontal recording cylinders, vertical recording cylinders, and vertical record disks. All of these devices are driven by clock-work and are designed to run a week before renewal of recording paper. The record of stream height by the Nilometer is on a scale less than the actual range of the water, and the recording pencil is connected by a suitable reducing device with a float which rises or falls with the stream. This float is usually placed in a small well near the stream-bank, its bottom communicating with the stream-bed by a pipe of such size as will not become readily clogged. The fluctuations of the water in this pipe correspond with those in the stream and turn the recording wheel through the agency of a cord wound around the wheel and having its lower end attached to the float.

84. **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 electric 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.

85. **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.

86. **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.

87. **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. Among the advantages of the weir as a measuring device are its simple construction, accuracy, cheapness, and ease of operation. Its results are easily interpreted by use of tables; it gives quantities of flow in second-feet directly; it is not necessary to maintain a constant head above it; and it causes a trifling loss of head.

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 and V-weirs, both of which latter have inclined sides with slopes of about one fourth horizontal to one vertical.

88. Rectangular Measuring Weir.—In using the ordinary weir, (Fig. 8), this should be placed at right angles to the 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

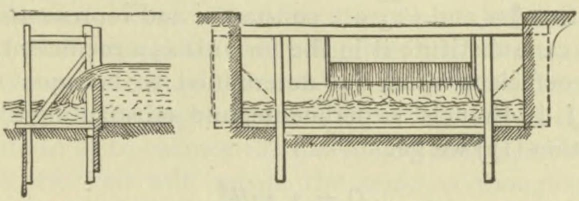

FIG. 8.—RECTANGULAR MEASURING WEIR.

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.

89. Francis' Formulas.—The form of equation indicated by theory for the discharge of a weir is

$$Q = Av; \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (11)$$

or, substituting for the mean velocity v of a given film its value $\frac{2}{3}\sqrt{2gh}$, and for A the area its equivalent $l \times h$, we get

$$Q = l \times h \times \tfrac{2}{3}\sqrt{2gh}, \quad . \quad . \quad . \quad . \quad (13)$$

which formula may be transposed so as to become

$$Q = l \times \tfrac{2}{3} h^{\frac{3}{2}} \sqrt{2g} \quad . \quad . \quad . \quad . \quad . \quad . \quad (14)$$

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 reduction of volume by the crest contraction can be compensated for by the coefficient m, and inserting this factor in formula (14) we have

$$Q = \tfrac{2}{3} m \sqrt{2g}\, l h^{\frac{3}{2}} \quad . \quad . \quad . \quad . \quad . \quad . \quad (15)$$

The factors $\frac{2}{3}m$ and $\sqrt{2g}$ 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 Mr. J. B. Francis' experiments, and substituting this value in equation (15) we get

$$Q = 3.33 l h^{\frac{3}{2}} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (16)$$

Owing to this falling away of the surface at 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 $l' = (l - 0.1nh)$. Hence

$$Q = 3.33(l - 0.1nh)h^{\frac{3}{2}}, \quad . \quad . \quad . \quad . \quad (17)$$

which is Francis' formula.

90. Conditions of using Rectangular Weir.—If the weir be placed so as to meet the following conditions, formulas (16) and (17) will give the best results. These 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 weir opening should be at least equal to twice 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 at least seven times larger than the weir opening. The approaching channel should be straight and of uniform cross-section. The weir should be erected in a plane at right angles to the stream and perpendicular to its bed, and the edges of the opening should be sharp up-stream and cut away down-stream.

91. 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. This is based on the theory that the effective length l of a rectangular weir being less than its true length owing to contraction, if the area of the weir be increased in proportion to its depth (since contraction increases in this ratio) and so as to balance the loss due to contraction, the flow through the weir will remain the same as though the weir were rectangular without contraction. 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

$$Q = 3.36\tfrac{2}{3} lh^{\frac{3}{2}} . \quad . \quad . \quad . \quad . \quad . \quad . \quad (18)$$

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. This weir (Fig. 73) may be used as a divisor, and for fairness of measurement is especially adapted to use on irrigation canals.

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

$$Q = 2.65th^{\frac{5}{2}}. \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (19)$$

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

$$Q = 2.65h^{\frac{5}{2}}, \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (20)$$

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

92. **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.

93. **Tables of Weir Discharge.**—Tables XV and XVI are taken from Prof. L. G. Carpenter's instructive bulletin on water measurement These tables give directly the discharge of rectangular and trapezoidal weirs of given lengths and for given depths.

94. **Measurement of Canal Water.**—In order that water flowing in open canals 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

TABLE XV.

DISCHARGE OVER RECTANGULAR WEIRS OF VARIOUS LENGTHS, AND WITH VARIOUS DEPTHS OF WATER, WITH AND WITHOUT CONTRACTION.

Formula, $Q = 3.33\frac{1}{3}(l - 0.2h)h^{\frac{3}{2}}$.

Depth of Water on Crest.		Discharge in Cubic Feet per Second.						
		With Two Complete Contractions.						Correction to be ADDED to each of the preceding to give discharge with NO contraction.
In inches.	In feet.	$l = 1$ ft.	$l = 1.5$ ft.	$l = 2$ ft.	$l = 3$ ft.	$l = 5$ ft.	$l = 10$ ft.	
0.3	.025	.0133	.0200	.0267	.0400	.0677	.133	.0000
0.6	.050	.0369	.0556	.0743	.1116	.1863	.3726	.0004
0.9	.075	.0674	.1015	.1350	.2040	.3410	.6830	.0010
1.2	.1	.1033	.1550	.2078	.3132	.5240	1.0519	.0021
1.5	.125	.1438	.2175	.2912	.4385	.7332	1.4695	.0037
1.8	.15	.1879	.2847	.3816	.5743	.9627	1.9312	.0058
2.1	.175	.2355	3575	.4795	.7235	1 2115	2.4315	.0085
2.4	.2	.2861	.4352	.5843	.8824	1.4787	2.9690	.0119
2.7	.225	.3399	.5177	.6956	1.0513	1.7627	3.5412	.0160
3.0	.25	.3959	.6042	.8126	1.2293	2.0227	4.1462	.0208
3.8	.275	.4543	.6946	.9350	1.4157	2.3771	4.7803	.0264
3.6	.3	.5149	.7287	1.0725	1.6103	2.7057	5.4441	.0328
3.9	.325	.5775	.8863	1.1952	1.8129	3.0483	6.1368	.0401
4.2	.35	.6420	.9871	1.3423	2.0226	3.4032	6.8547	.0483
4.5	.375	.7079	1.0905	1.4732	2.2385	3.7691	7.5956	.0574
4.8	.4	....	1.1974	1.6160	2.4623	4.1489	8.3655	.0674
5.1	.425	....	1.3070	1.7689	2.6926	4.5400	9.1585	.0785
5.4	.45	....	1.4189	1.9221	2.9874	4.9410	9.9725	.0905
5.7	.475	....	1.5333	2.0790	3.1703	5.3529	10.8994	.1036
6.0	.5	....	1.6500	2.2392	3.4177	5.7743	11.6672	.1178
6.3	.525	....	1.7689	2.4029	3.6709	6.2069	12.5469	.1331
6.6	.55	....	1.8899	2.5698	3.9295	6.6489	13.4474	.1496
6.9	.575	....	2.0129	2.7395	4.1928	7.0995	14.3658	.1671
7.2	.6	....	2.1381	2.9128	4.4621	7.5607	15.3072	.1859
7.5	.625	....	2.2646	3.0881	4.7351	8.0291	16.2641	.2059
7.8	.65	....	2.3929	3.2663	5.0130	8.5064	17.2399	.2271
8.1	.675	....	2.5234	3.3478	5.2965	8.9939	18.2374	.2496
8.4	.7	....	....	3.6313	5.5536	9.4882	19.2497	.2733
8.7	.725	....	....	3.8170	5.7747	9.9901	20.2786	.2984
9.0	.75	....	....	4.0052	6.1702	10.5002	21.3252	.3248
9.3	.775	....	....	4.1961	6.4704	11.0190	22.3905	.3525
9.6	.8	....	....	4.3884	6.7734	11.5434	23.4684	.3816
9.9	.825	....	....	4.5833	7.0810	12.0764	24.5649	.4121
10.2	.85	....	....	4.7806	7.3929	12.6135	25.6790	.4440
10.5	.875	....	....	4.9792	7.7075	13.1641	26.8056	.4774
10.8	.9	....	....	....	8.0257	13.7177	27.9477	.5123
11.1	.925	....	....	....	8.3473	14.2779	29.1044	.5886
11.4	.95	....	....	....	8.6725	14.8451	30.2766	.5864
11.7	.975	....	....	....	9.0012	15.4192	31.4642	.6258

TABLE XV.—*Continued.*

DISCHARGE OVER RECTANGULAR WEIRS

Depth of Water on Crest.		Discharge in Cubic Feet per Second.			
		With Two Complete Contractions.			Correction to be ADDED to each of the preceding to give discharge with NO contraction.
In inches.	In feet.	$l = 3$ feet.	$l = 5$ feet.	$l = 10$ feet.	
12.0	1.0	9.3333	16.0000	32.6667	.6667
12.3	1.025	9.6679	16.5859	33.8809	.7091
12.6	1.05	10.0058	17.1784	35.1099	.7531
12.9	1.075	10.3471	17.7777	36.3532	.7988
13.2	1.1	10.6890	18.3825	37.6110	.8460
13.5	1.125	11.0370	18.9916	38.9781	.8949
13.8	1.150	11.3866	19.5080	40.1615	.9455
14.1	1.175	11.7396	20.2308	41.4573	.9977
14.4	1.2	12.0935	20.8569	42.7654	1.0516
14.7	1.225	12.4507	21.4893	44.0856	1.1072
15.0	1.25	12.8103	22.1269	45.4184	1.1646
15.3	1.275	13.1733	22.7713	46.7663	1.2237
15.6	1.3	13.5375	23.4189	48.1224	1.2846
15.9	1.325	13.9047	24.0727	49.4927	1.3473
16.2	1.35	14.2744	24.7318	50.8753	1.4117
16.5	1.375	14.6450	25.3936	52.2651	1.4779
16.8	1.4	....	26.0625	53.6710	1.5460
17.1	1.425	....	26.6355	55.0870	1.6160
17.4	1.45	....	27.4122	56.5122	1.6878
17.7	1.475	....	28.0950	57.9515	1.7615
18	1.5	....	28.7814	59.3999	1.8371
18.3	1.525	....	29.4719	60.8584	1.9146
18.6	1.55	....	30.1675	62.3290	1.9940
18.9	1.575	....	30.8681	63.8116	2.0754
19.2	1.6	....	31.5727	65.3042	2.1588
19.5	1.625	....	32.2809	66.8059	2.2441
19.8	1.65	....	32.9935	68.3185	2.3315
20.1	1.675	....	33.7093	69.8393	2.4207
20.4	1.7	....	34.4269	71.3710	2.5128
20.7	1.725	....	35.1546	72.9146	2.6054
21	1.75	....	35.8827	74.4662	2.7008
21.3	1.775	....	36.6151	76.0286	2.7984
21.6	1.8	....	37.3520	77.6020	2.8980
21.9	1.825	....	38.0709	79.1614	3.0196
22.2	1.85	....	38.8341	80.7716	3.1034
22.5	1.875	....	39.5812	82.3717	3.2093
22.8	1.9	....	40.3321	83.9816	3.3174
23.1	1.925	....	41.0860	85.5995	3.4275
23.4	1.95	....	41.8436	87.2271	3.5399
23.7	1.975	....	42.6045	88.8635	3.6545
24	2	....	43.3665	90.5061	3.771
27	2.25	....	....	107.44	5.06
30	2.50	....	....	125.16	6.59
36	3	....	....	162.79	10.39

TABLE XVI.

DISCHARGE OVER CIPPOLETTI'S TRAPEZOIDAL WEIR OF VARIOUS LENGTHS AND WITH VARIOUS DEPTHS.

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

Depth of Water on Crest.		Discharge in Cubic Feet per Second over Weir.							
In inches.	In feet.	$l=1$ ft.	$l=1.5$ ft.	$l=2$ ft.	$l=3$ ft.	$l=4$ ft.	$l=5$ ft.	$l=7$ ft.	$l=10$ ft.
.3	.025	.0135	.0202	.0269	.0404	.0539	.0673	....	.1347
.6	.05	.0367	.0566	.0754	.1131	.1508	.1885	....	.3771
.9	.075	.0690	.1035	.1380	.2071	.2761	.3451	....	.6902
1.2	.1	.1064	.1596	.2128	.3192	.4256	.5319	....	1.0639
1.5	.125	.1488	.2232	.2976	.4464	.5952	.7440	....	1.4881
1.8	.15	.1956	.2934	.3912	.5868	.7824	.9780	....	1.9560
2.1	.175	.2464	.3697	.4929	.7393	.9858	1.2322	....	2.4644
2.4	.2	.3010	.4515	.6020	.9029	1.2039	1.5049	....	3.0098
2.7	.225	.3592	.5388	.7184	1.0777	1.4369	1.7961	....	3.5922
3.0	.25	.4208	.6312	.8417	1.2625	1.6833	2.1041	....	4.2083
3.3	.275	.4855	.7282	.9709	1.4564	1.9419	2.4273	....	4.8547
3.6	.3	.5531	.8297	1.1063	1.6594	2.2126	2.7657	....	5.5314
3.9	.325	.6238	.9358	1.2477	1.8715	2.4954	3.1192	....	6.2384
4.2	.35	.6972	1.0459	1.3945	2.0917	2.7890	3.4862	....	6.9724
4.5	.375	.7730	1.1595	1.5460	2.3190	3.0920	3.8649	....	7.7299
4.8	.4	....	1.2777	1.7035	2.5553	3.4071	4.2588	....	8.5177
5.1	.425	....	1.3993	1.8658	2.7987	3.7316	4.6645	....	9.3290
5.4	.45	....	1.5246	2.0328	3.0492	4.0656	5.0820	....	10.1640
5.7	.475	....	1.6534	2.2045	3.3067	4.4089	5.5112	....	11.0225
6.0	.5	....	1.7854	2.3805	3.5708	4.7610	5.9512	....	11.9025
6.3	.525	....	1.9210	2.5614	3.8420	5.1227	6.4034	....	12.8068
6.6	.55	....	2.0599	2.7465	4.1198	5.4930	6.8663	....	13.7326
6.9	.575	....	2.2018	2.9357	4.4036	5.8715	7.3393	....	14.6787
7.2	.6	....	2.3472	3.1293	4.6939	6.2585	7.8231	....	15.6463
7.5	.625	....	2.4955	3.3274	4.9911	6.6548	8.3185	....	16.6370
7.8	.65	....	2.6462	3.5283	5.2924	7.0565	8.8206	....	17.6413
8.1	.675	....	2.8007	3.7343	5.6014	7.4686	9.3357	....	18.6715
8.4	.7	....	......	3.9437	5.9156	7.8874	9.8593	13.8030	19.7186
8.7	.725	....	......	4.1565	6.2347	8.2930	10.3912	14.5477	20.7824
9.0	.75	....	......	4.3733	6.5599	8.7466	10.9332	15.3065	21.8665
9.3	.775	....	......	4.5942	6.8912	9.1883	11.4854	16.0796	22.9708
9.6	.8	....	......	4.8177	7.2265	9.6354	12.0442	16.8619	24.0885
9.9	.825	....	......	5.0453	7.5679	10.0906	12.6132	17.6585	25.2264
10.2	.85	....	......	......	7.9154	10.5538	13.1923	18.4692	26.3846
10.5	.875	....	......	......	8.2669	11.0225	13.7781	19.2893	27.5562
10.8	.9	....	......	......	8.6234	11.4978	14.3723	20.1212	28.7446
11.1	.925	....	......	......	8.9850	11.9800	14.9749	20.9649	29.9499
11.4	.95	....	......	......	9.3516	12.4688	15.5860	21.8204	31.1720
11.7	.975	....	......	......	9.7233	12.9644	16.2054	22.6876	32.4019
12.0	1.	....	......	......	10.1000	13.5667	16.8333	23.5667	33.6667
12.3	1.025	....	......	......	10.4808	13.9744	17.4679	24.4551	34.9359
12.6	1.05	....	......	......	10.8666	14.4888	18.1110	25.3554	36.2220
12.9	1.075	....	......	......	11.2575	15.0100	18.7624	26.2674	37.5249

TABLE XVI.—*Continued.*

DISCHARGE OVER TRAPEZOIDAL WEIRS.

Depth of Water on Crest.		Discharge in Cubic Feet per Second over Weir.				
In inches.	In feet.	$l = 3$ feet.	$l = 4$ feet.	$l = 5$ feet.	$l = 7$ feet.	$l = 10$ feet.
13.2	1.1	11.6524	15.5365	19.4206	27.1888	38.8412
13.5	1.125	12.0513	16.0684	20.0855	28.1198	40.1711
13.8	1.150	12.4553	16.6071	20.7588	29.0624	41.5177
14.1	1.175	12.8644	17.1525	21.4406	30.0168	42.8812
14.4	1.2	13.2764	17.7019	22.1274	30.9784	44.2548
14.7	1.225	13.6936	18.2581	22.8226	31.9517	45.6453
15.0	1.25	14.1148	18.8197	23.5246	32.9344	47.0492
15.3	1.275	14.5410	19.3880	24.2349	33.9289	48.9699
15.6	1.3	....	19.9603	24.9503	34.9305	49.9007
15.9	1.325	....	20.5394	25.6742	35.9439	51.3484
16.2	1.35	....	21.1238	26.4047	36.9666	52.8095
16.4	1.375	....	21.7123	26.1404	37.9966	54.2808
16.8	1.4	....	22.3075	27.8844	39.0382	55.7688
17.1	1.425	....	22.9082	28.6352	40.0893	57.2704
17.4	1.45	....	23.5128	29.3910	41.1474	58.7820
17.7	1.475	....	24.1242	30.1552	42.2173	60.3105
18.0	1.5	....	24.7396	30.9245	43.2943	61.8490
18.3	1.525	....	25.3604	31.7005	44.3808	63.4011
18.6	1.55	....	25.9866	32.4833	45.4766	64.9666
18.9	1.575	....	26.6182	33.2727	46.5818	66.5455
19.2	1.6	....	....	34.0685	47.6959	68.1370
19.5	1.625	....	....	34.8702	48.8183	69.7405
19.8	1.65	....	....	35.6782	49.9495	71.3565
20.1	1.675	....	....	36.4913	51.0878	72.9826
20.4	1.7	....	....	37.3111	52.2355	74.6222
20.7	1.725	....	....	38.1376	53.3926	76.2752
21.0	1.75	....	....	38.9691	54.5568	77.9383
21.3	1.775	....	....	39.8074	55.7304	79.6149
21.6	1.8	....	....	40.6515	56.9121	83.3030
21.9	1.825	....	....	41.5009	58.1013	83.0018
22.2	1.85	....	....	42.3577	59.3008	84.7154
22.5	1.875	....	....	43.2179	60.5031	86.4358
22.8	1.9	....	....	....	61.7211	88.1730
23.1	1.925	....	....	....	62.9442	89.9203
23.4	1.95	....	....	....	64.1720	91.6743
23.7	1.975	....	....	....	65.4116	93.4452
24.0	2.0	....	....	....	66.6560	95.2228
25.5	2.125	....	....	....	72.999	104.285
27.0	2.25	....	....	....	79.541	113.63
28.8	2.4	....	....	....	87.619	125.17
30.0	2.5	....	....	....	93.156	133.08

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.

95. **Requisites of a Measuring Apparatus or Module.**—Prof. L. G. Carpenter enumerates the following as the conditions most desirable for a module or apparatus for measuring irrigation water :

Its discharge should be capable of conversion into the common measure, which is cubic feet per second. The ratio of discharge indicated from two outlets should be the actual ratio. The same module should give the same discharge wherever placed; it should be capable of being used on canals of all sizes and of being set to discharge any fraction of its capacity for the process of distributing pro rata. Attempts to tamper with or alter its discharge should leave traces easy to recognize; and it should be simple enough to be operated by men of ordinary intelligence, so that calculations should not be required to regulate the discharge of different modules or to determine the amount thereof. It should occupy but small space, and the discharge should not be affected by variations of the water-level in the supplying canal. It should be inexpensive, and cause the least possible loss of head. Nearly all modules attempt to maintain a constant pressure of water above the opening, the orifice remaining unchanged.

96. **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 Mr. 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.

97. **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 which 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.

98. **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 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. 9, *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

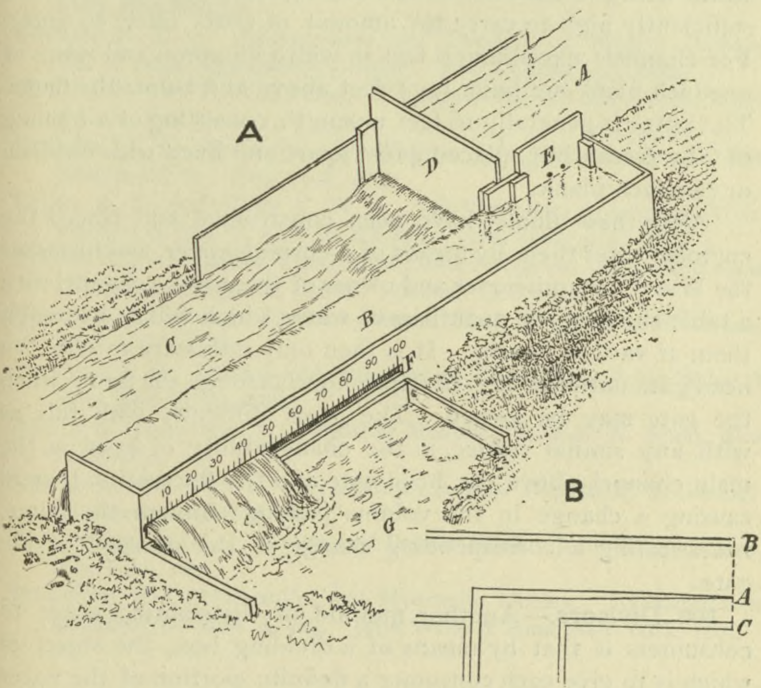

FIG. 9.—FOOTE'S MEASURING WEIR, A. WATER DIVISIOR, B.

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.

99. 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.

100. 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. 9, *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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CHAPTER VII.

SUBSURFACE WATER SOURCES AND SEWAGE FOR IRRIGATION.

102. 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.

103. 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.

104. 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.

105. Examples of Artesian Wells.—Some great wells have been sunk in different parts of the world. The celebrated Grinnell well in Paris commenced with a 20-inch bore and is gradually reduced to an 8-inch bore at the bottom; its depth is 1806 feet and its yield has been as great as 1.5 second-feet. A well has recently been bored in the neighborhood of Wheeling, West Virginia, to the great depth of 4500 feet but is dry. At Sperenberg, near Berlin, is a well 4170 feet deep, and at Schladabach, near Leipsic, is a well 5740 feet in depth. 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 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 527 respectively in North Dakota and South Dakota, and 534 in 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 at the average rate of 13.2 acres per well. Their average depth is 210 feet, average cost $245, and average discharge 0.12 second-feet.

106. Capacity and Cost of Artesian Wells.—The capacities of flowing wells are relatively small as compared with the volumes of water required in irrigation. Of the eight thousand wells reported from the arid region comparatively few are of sufficient capacity for use in irrigation. The great majority are shallow in depth and small in bore and discharge. They range from 100 to 200 feet in depth, from 2 to 4 inches in internal diameter, and discharge rarely as much as 0.1 of a second-foot; though this volume, if stored in a suitably located reservoir, should irrigate a moderate-sized farm. On the other hand there are, especially in South Dakota and Southern California, some very large flowing wells. In the former State there are reported to be at least twenty-five wells with discharges ranging from 1 to 6 second-feet, and in Southern California about thirty wells of similar capacities The largest well in South Dakota delivers continuously about 6.68 second-feet.

The cost of sinking artesian wells is an exceedingly variable quantity, and is dependent upon the depth and bore of the well and the material through which it is sunk. In some localities, under favorable conditions, 6-inch wells of moderate depth are sunk and lined at prices ranging between $2.75 and $3.50 per foot. In North Dakota, where wells are sunk through sandstone, wells of 2 to 4 inches diameter are sunk for about 80 cents for the first hundred feet up to $1.50 for the second hundred feet. One well 1084 feet in depth cost $4000 and yields 4 second-feet. In South Dakota are many

2-inch wells of depths from 250 to 300 feet, which have been sunk at the very low price of $75, the price increasing thence to $300 for a 3-inch well 300 feet deep. In other places, where more gravel and stone are encountered, 2-inch wells have cost $300, and 3- and 4-inch wells from $400 to $1000. One well 850 feet in depth, 4 inches in diameter, cost $1800, and discharges 3 second-feet, irrigating about 100 acres. In Colorado in sandstone and hard clay the cost of well-drilling averages $2.00 per foot.

107. Storage of Artesian Water.—Having decided from a study of the geology and a knowledge of the success attained by other wells, the general locality in which the well is to be drilled, its specific location should if possible be on the highest point of the land to be irrigated, and in such a position that it may be outside of and tributary to the reservoir in which the water is to be stored. Since artesian wells flow continuously during twenty-four hours of the day and three hundred and sixty-five days in the year, it is desirable to store as much of the water which flows during the non-irrigating period as possible, in order that the greatest duty may be gotten from the well. The volume flowing continuously from almost any well is usually too small to enable it to flow over the land in sufficient volume for the purposes of irrigation, so that a necessary adjunct to nearly every well is a storage reservoir of greater or less dimensions. In the case, however, of a well which discharges about 1 second-foot, or nearly enough to irrigate 100 acres from unstored flow, such a well may be made capable of irrigating ten times this area if the water flowing at other times than the irrigating periods can be stored. Small reservoirs sufficiently large only to retain enough water to produce the requisite head for flowing over land may be built as are watering-tanks on railways, or they may be cheaply excavated in the highest ground on the farm and properly lined. Larger ones may be constructed by making use of the natural configuration of the country and building a dam across a hollow or ravine (Chap. XV).

108. Size of Well.—The volume of the well does not depend upon its size. A 6-inch well will not necessarily discharge twice as much water as a 3-inch well—perhaps not as much. The amount of flow depends directly upon the volume of the water-bearing strata and the pressure due to its initial head or source. Providing this is sufficiently great, then the discharge of the well is dependent on its diameter. Other things being equal, a large well will cost more to drill, but will be more easily and cheaply cleaned and kept in operation than a smaller one, which is apt to clog. Further, during and after drilling an accident may ruin a small well, while a larger one may be recased with diminished bore and still remain serviceable. For purposes of irrigation it may in general be said that a well less than 4 inches in diameter should not be drilled, and it is probable that one with a bottom bore greater than 8 inches will not be economical.

Nearly all wells which terminate in soft rock, sand, or gravel discharge more or less of these materials. To prevent this from clogging the well it is not uncommon to place perforated pipe in the bottom of the well through the water-bearing stratum. There are many styles of such pipe, but in general it may be stated that pipe with circular perforations of uniform diameter is not the most serviceable, as it is apt to become clogged. Some of the patented perforated pipes with slots having less aperture on the outer than on the inner surface are preferable. In some cases experience may show that it is not desirable to insert perforated pipe, but to let whatever comes to the well be discharged and collected in the storage reservoir.

109. Manner of having Wells Drilled.—There are many responsible firms who make a business of drilling and boring artesian wells, and for those who are unfamiliar with the business of well-sinking, it is better to contract with some such firm to perform the work required. On the other hand, the sinking of a well is not a difficult operation for those who have any idea of the process, though by contracting they are certain of having the well sunk as they desire, within a fixed price, and are relieved of the risk of accidents.

In the oil and gas regions the drilling of wells to tap oil- and gas-bearing strata, which is a process entirely similar to that of drilling wells for water, is a matter of everyday occurrence, and nearly all who desire to sink wells perform the work on a sort of half-contract system. The principal apparatus comprises an engine, boiler, carpenter's rig, and set of drilling-tools, and the common practice is for the owner to provide all except the tools and fuel and let the drilling of the well at so much a foot to a contractor who furnishes these and does the work of putting down the well. In Ohio and Pennsylvania wells drilled in this manner by half-contract cost from 50 to 80 cents per foot for moderate-sized wells up to $1.50 to $2.00 for large and deep wells.

110. Varieties of Drilling-machines.—Wells may be drilled by various methods, among the chief of which are by cables, poles, and hydraulic process. Provided the well is to be drilled by contract, it is of little importance what method is employed, since the contractor is responsible for the proper completion of the work, and the style of rig is a matter for his own choice. In the Dakotas and some other of the plains regions it has been found that wells drilled with pole machines have proved most satisfactory and performed the cheapest work, aside from the amount of time taken in coupling and uncoupling the rods. In the oil- and gas-bearing regions cable machines are most popular. There are many patterns of hydraulic, jetting and rotary rigs which are adopted by different well-boring firms. The latter are dependent upon a rotary motion given to a piston-rod working by hydraulic power and turning a tubing with cutting-edge. In hydraulic jetting machines, which can be used cheaply only in gravel or sand, there is employed a short drill-bit having a hollow shank through which a jet of water is forced from pipe rods, thus creating an upward current which carries out the drillings. Some of these hydraulic and jetting machines have met with remarkable success.

The chief advantage of pole rigs over cable rigs is in the certainty of the revolutions given to the drill, as the rods form a rigid connection between the drill and the machine above, and the motion is uniform in the direction of tightening the screws of the joints. This tends to preserve the connection, and keep the drill under perfect control. Cable rigs are chiefly preferred because of the ease with which they can be operated and the speed with which the tools can be lowered and removed and the bailing apparatus substituted in their

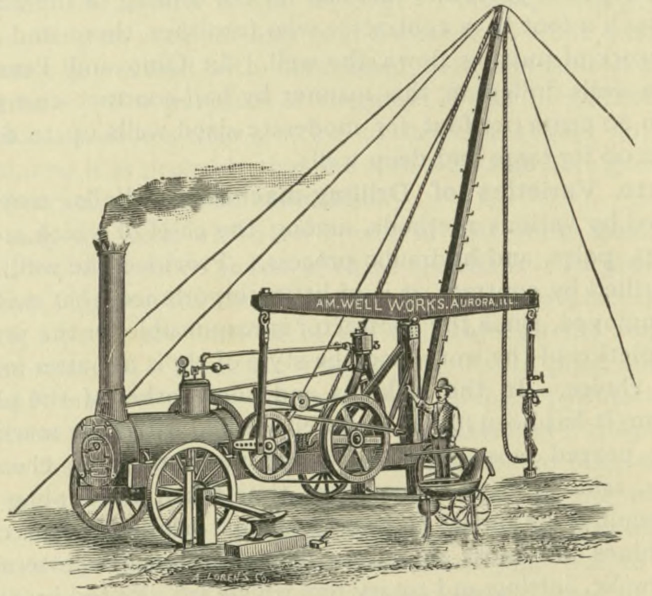

FIG. 10.—PORTABLE ARTESIAN-WELL DRILLING RIG.

place. The chief disadvantages as compared with the pole rigs is in the greater friction produced by the corrugated surface of the cable, the uncertainty as to whether the striking bar reaches the bottom of the drill, the likelihood of cutting or bending the cable, and the danger of breaking under the strain when tools become fast. As the cable is rotated both to the right and left there is also liability of uncoupling the joints at the tools, and there is a possibility that the cable may not produce the proper rotation in the drill, and thus not bore the hole truly circular. There are now on the market a number of excellent portable well-drilling rigs, both of the old reliable walking-beam type (Fig. 10), and also jetting and hydraulic rotary rigs. These can frequently be purchased outright at prices which will render them cheaper than any other method of having wells drilled.

111. **Process of Drilling.**—The general process of drilling consists in having a long, heavy drilling-bar, the lower end of which is dressed to a cutting-edge, which is dropped into a hole in the rock and by its weight cuts or breaks the stone where it strikes. At each blow this rod is turned a little, thus making the hole round. The drill is hung from the end of a cable or series of jointed poles which are raised and dropped by machinery. After the drill has worked for a short time it is removed, and the drillings, or small pieces of rock which have collected in the bottom of the hole and deaden the blow of the drill, are removed. This is done by pouring water in the hole if it be dry, and the fluid mud thus formed is lifted to the surface by a long, narrow bailer with a valve at its lower end. These operations of drilling from three to five feet, then cleaning out the mud and drilling again, are alternated until the desired depth is reached. If casing or lining is to be introduced and the hole is not drilled truly cylindrical, it is reamed out by a steel tool of desired diameter, weighing about 125 pounds and attached in place of the drill.

The apparatus which goes to make a drilling-machine comprises an engine and boiler of about 20 horse-power, a set of drilling-tools, and cable or poles. These latter are generally spoken of as the rig. It is also necessary to provide tubing or casing to line the well through such permeable strata as might cause the loss of water or through such strata as may provide water which is undesirable for the purposes required. It is sometimes necessary to line wells with tubing throughout their entire length, and in such cases it is usual to begin with a large bore, say 8 inches, and after sinking this to a given depth, say 200 or 300 feet, to reduce the diameter of the tubing by an inch or two.

The "set of tools" which compose the drill—for the latter is not a solid bar, but several pieces—weigh about 2500 lbs., and consist of a steel "bit" or "drill," of the size of the bore desired, screwed into the lower end of the "augur stem," which latter is a steel rod 30 feet long and 3 inches in diameter. To the upper end of this are screwed "jars," and above them the "sinker bar," which is 15 feet long and 3 inches in diameter, and of steel. The jars by slacking together in falling cause the sinker bar to act on and through them to the drill as a hammer. The term "rig" generally includes, in addition to the set of tools, the woodwork and necessary iron fittings forming a derrick to carry a sheave at a sufficient height, perhaps 50 to 80 feet, to swing the drilling-tools clear of the ground; also, both wheels and shaft on which the drill cable is wound; the sand reel for winding up the smaller rope used in cleaning out the drillings; a walking-beam to give vertical motion, and a band-wheel for transmitting power from the engine to the moving parts.

After the engine has been started and the walking-beam is made to rock up and down at the rate of 20 to 30 strokes a minute, lifting the tools with it, the length of stroke being adjústable from 15 inches to 3 feet, the rope is then twisted by means of a stick, first in one direction for a while and then in the opposite direction alternately. This twisting of the rope turns the drill, and the driller who handles the rope knows by the "feel" how the tools are working, the texture of the rock and the occurrence of an accident. Occasionally the temper and set-screws are turned out a little, thus lowering the tools. After the drilling has gone on to a depth of 4 or 5 feet the tools are hoisted clear of the floor, the bull-rope swung off to one side and the bailer or sand pump is swung over the hole from the sand reel, and is allowed to drop by its own weight, and upon reaching the bottom is filled with mud and sand through the valve at its lower end and is then drawn up and emptied; this process being repeated if necessary to clear the hole before drilling is again resumed. The rate of drilling depends wholly upon the character of strata encountered, but averages from 15 to 50 feet per working day.

112. **Capacity of Common 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 water plain 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.

The extent to which common wells may be used as a source of supply for irrigation is not appreciated in the United States, where as yet irrigation is practised only in a large way and irrigators are but just coming to a realization of the advantages of intensive cultivation, whereby but a few acres are worked by a single farmer, but in the most thorough manner possible. In a few portions of the far West, notably in Central and Southern California, where Italians and Chinamen are engaged chiefly in market-gardening, wells are employed to some extent for the supply of water. In such cases the water is raised by one of several processes (Chap. XIX), chiefly by windmills, and by mechanical lifts worked by horse-power, and similar to the Persian wheel of Asia.

It is to India that we must look in order to gain an idea of the extent to which wells may furnish irrigation water. In the Central Provinces of India 120,000 acres are irrigated from wells. In Madras 2,000,000 acres are irrigated from 400,000 wells. In the Northwest Provinces 360,000 acres are irrigated from wells. Some of these wells are sunk to depths as great as 80 to 100 feet, in some cases through hard rock, and are capable in ordinary seasons of irrigating from 1 to 4 acres each. These wells may really be said to supplement irrigation from canals and reservoirs, for after the waters of the latter have been used and have seeped into the soil they are caught by the well and are again used for irrigation. Thus wells as an adjunct to canals may be said to add materially to the duty of the latter.

113. Tunnelling 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 Khojah 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.

114. Underground Cribwork.—Submerged cribs have been planned for the American Water Company on Cherry creek in Colorado, and have been used by the Citizens' Water Company on the South Fork of the Platte river in Colorado. The former enterprise contemplates a submerged open crib dam sunk in the gravel bed of Cherry creek, and resting on blue clay which is 73 feet below the surface of the stream. This cribwork is to be 70 feet in height, with its crest 3 feet below the bed of the stream. This is not a dam, as it will not extend across the entire channel of the stream, but will stop 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 to be placed 4 feet apart across stream, and be planked on both faces with interstices of 3 inches on the upper face. The water caught in this cribwork will be pumped to the surface.

The Citizens' Water Company develop 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

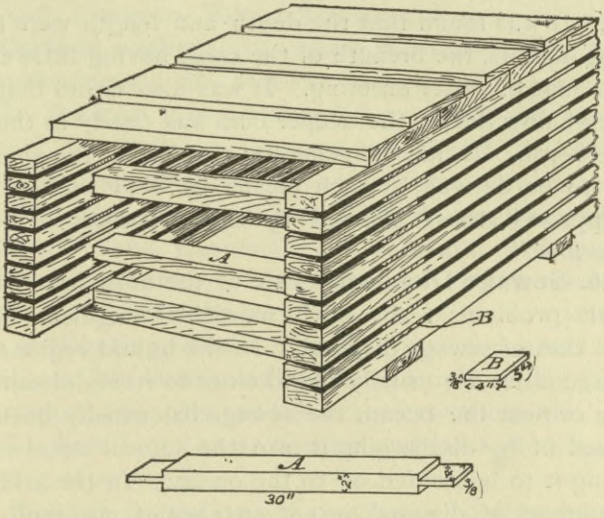

FIG. 11.—GATHERING-CRIBS, CITIZENS' WATER CO., DENVER.

bed of the stream. The cribs (Fig. 11) 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.

115. 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 plane. 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. It may be generally stated that the amounts of water to be derived by such means are very limited and do not approach those claimed by the advocates of so-called "underflow."

116. Sewage Disposal.—One of the most important and difficult problems with which municipal engineers have to deal is that of sewage disposal. In the humid regions, where the large cities are usually found close to rivers of some magnitude or near the ocean, the sewage has usually been easily disposed of by discharging it into the natural waterways and allowing it to be carried off to the ocean. In the arid region this method of disposal is not so easy of accomplishment. because of the lack of waterways into which to discharge it. Difficulty has also been encountered in some of the older inhabited portions of the world and as a result, other and newer methods of disposing of sewage are attracting attention. A method which is rapidly gaining in favor is by flowing the sewage over the soil and permitting it to filter downward through this and find its way to the natural watercourses. It has been found that nature performs the chemical and mechanical action of removing the heavy matter and purifying the more liquid portions of the sewage even more satisfactorily than it can be done artificially. This has naturally led to the utilization of such sewage water in irrigating crops and this method of disposal of sewage is of especial interest to the people of the arid West.

It is found that by this means the disposal of sewage may not only be rendered a simple matter, but that instead of being an item of great expense to the municipality it may even in a few instances be rendered a source of income. The use of sewage for irrigation has been practised for many years quite extensively in various portions of Europe, notably at Paris, which thus disposes of one third of its sewage; at Berlin, which uses all its sewage for irrigation; at Edinburgh, Birmingham, Florence, Milan, Madrid, and many other cities and hundreds of smaller towns which maintain sewage farms. In our own country this method has met with some little favor. In the East it is employed successfully at Meriden, Connecticut, and Pullman, Ill.; and in the West the following ten cities dispose of their sewage by irrigation: Colorado Springs, Trinidad, Fresno, Pasadena, Redding, Los Angeles, Santa Rosa, Helena, Cheyenne, and Stockton, with populations varying between one thousand and fifty thousand inhabitants. It will thus be seen that this is no new problem either in engineering science or municipal government. It is well tried, and has been practised for a couple of centuries in some European countries, and in every case where the soil is suitable has been found an economical and satisfactory method of disposing of sewage.

117. Sewage Irrigation.—Sewage may be disposed of by discharging it on land in practically three ways, namely, by intermittent downward filtration, by broad irrigation, and by a combination of these two methods. Intermittent downward filtration is simply a mode of purifying sewage by applying it to land without making any attempt to utilize it in irrigation, or in other words, in the watering and cultivation of crops. It requires a much smaller area of land than where it is used in irrigation. It depends for its utility on the fact that sewage passed through porous soil becomes aerated and rapidly purified through the oxidizing action of the air which the soil holds in its pores, and for its successful operation requires that the sewage shall not be passed through the same piece of land continuously, but at long enough intervals in order to permit the soil to become aerated. By thus intermittently resting the soil the sewage from 500 to 1000 people per acre can be purified. This system in itself is of little interest to irrigators, and therefore will not be further described.

The utilization of sewage by broad irrigation requires the employment of a much larger tract than for intermittent downward filtration, one acre of land being sufficient to utilize the sewage of from 150 to 500 people. This system has been largely and successfully used, especially where the soil is porous and underlain by a deep porous subsoil. When the farm is properly laid out and carefully managed the effluent water is pure enough to be practically harmless when returned to the natural drainage channels. One of the most serious objections to the disposal of sewage by irrigation is the fact that the farmer must take the sewage at all times, even though he have more than he wants and it hurts his land. It has been found, however, that a combination of the above methods in which intermittent filtration is used as a supplement to broad irrigation practically overcomes this disadvantage, and is the most satisfactory method of disposing of and utilizing sewage on land. This is done by laying out a small portion of the land as a filter-bed by providing it with ample underdrainage, and on this the sewage is discharged at such times as it is not needed in irrigation.

118. The Fertilizing Effects of Sewage.—It is well known that the excreta of human beings is far richer in nitrogenous substances than that of any other animal, and that it has a far greater fertilizing value. Human excreta loses its fertilizing value rapidly by decomposition, and it is therefore necessary to apply it to the soil within a few days, and to this end the only practical way of moving it is by water carriage. It can thus be moved as much as fifty miles without deteriorating, while the volume of carrying water need never exceed 10 cubic feet daily per individual, or about the proportion of water to excreta which usually finds its way into city sewage.

On the sewage farms of Paris the most varied products, from vegetables of all kinds to flowers and fruits, are profitably grown. The cultivation of vegetables is predominant, cabbages and cauliflower being especially prolific. The sewage water is employed as a manure or for watering grain, mangel-wurzel, and meadows. Lucerne is cut as often as four or five times a season, and mangel-wurzel produces as much as 40 tons per acre. The municipal engineers of Paris state that the rent value of lands irrigated by sewage have increased in value since their reclamation from 100 to 400 per cent. At Colorado Springs enormous crops are reported to be raised from sewage irrigation, while the lessee of the right to use the city sewage is not troubled with the vexatious problem of priority of rights. The sewage of the city of Fresno, Cal., is used in irrigating a large tract of land on which all varieties of vegetables are profitably grown. The best evidence obtainable indicates that the fertilizing effects of sewage are not as great as claimed, and that the crops raised by its use in humid regions are scarcely more abundant than those gotten by use of chemical fertilizers. In the arid regions it produces better crops than water alone, but scarcely better than those gotten by the use of water with artificial fertilization. Thorough cultivation of the soil greatly increases its value as a fertilizer.

119. **Effects of Sewage Irrigation on Health.**—Fears have been entertained that sewage farms would prove dangerous to the health of the neighboring districts, and that the crops grown on them would be unwholesome. These fears, however, have undoubtedly proven groundless, as shown by experience in many portions of the world. It is found that the combined action of soil and vegetation furnishes the true solution of the problem of sewage disposal on land. It satisfies the sanitary conditions, and at the same time gains for agriculture a source of manure and water which would otherwise run to waste. The principle on which this system rests is, that when pure water charged with materials in suspension and solution is flooded over permeable soil the upper bed of this acts as a filter, and all matter in suspension is separated by mechanical action. After this superficial mechanical filtration the water reaches the roots of the plants, which absorb with benefit the fertilizing substance remaining in solution. Lastly, as the waters which have escaped the absorbent action of the plants or the retentive action of the soil continue their descent through a sub-

soil either naturally or artificially permeable, they undergo in this an oxidizing action which changes them from organic substances into nitrates or nitrites—purely mineral substances, which present no danger of fermentation, and are harmless when sufficiently diluted.

Chemical analyses of the waters flowing from the Paris sewage farms show no sensible trace of decomposable nitrogen and but little more than a trace of nitrogen in the state of a mineral ammonia. On the other hand, where no vegetation exists on the surface a very perceptible and dangerous quantity of nitrogen has been obtained. It was also discovered that the descent through the porous soil insures a satisfactory aeration, as sewage water flowed on the surface and containing scarcely any oxygen issues from a bed of stony earth but six feet in depth with a gain of from 400 to 600 per cent of oxygen, so that there was a complete revivification of the sewage water, which was not merely clarified, but actually purified. Water has been drawn from wells sunk in the middle of lands irrigated by sewage, and this water has been found to be perfectly clear and identical in appearance and taste with waters of the subterranean water-plain which supplies wells elsewhere in the neighborhood of Paris. The experience at the Pullman, Ill., farm is similar, as there the superintendent of the farm lives in a handsome house in the centre of the irrigated area, and is in no way affected or annoyed by the sewage.

Thorough tilling of the soil after each flowing of sewage is essential, and it is by the creation of a proper tilth that the aeration of the sewage and incorporation of the solid matter with the soil is accomplished. It is this process of absorption of the water, incorporation of the deposits with the soil, and its utilization by plants that guarantees the salubrity of the surrounding country. Villages which have sprung up in the neighborhood of sewage farms in Europe show no signs of disorders or diseases of any kind. A more surprising fact is that there is practically no stench from the flowing of sewage over the land. When put on the land with no more dilution than the flushing water, there is at that time a perceptible and disagreeable odor, but as soon as the soil has been cultivated this entirely disappears.

120. Duty of Sewage.—Chemical analysis of sewage water indicates that the theoretic amount which may be used in irrigating crops with benefit to agriculture, and which the soil and crops will deprive of nitrogen, alkalies, and phosphoric acid, which elements most affect the purity of water, is 4.5 acre-feet per acre. However, where several crops are produced in a single season on the same soil, and more especially where these crops consist of alfalfa and similar plants which require heavy watering by flooding methods, as much as 12 to 15 acre-feet per acre may be applied without harmful effect. On the sewage farms of Gennevilliers, in the suburbs of Paris, the average annual application of sewage during ten years varied between 15 and 22 acre-feet per acre. This soil, it seems, is especially well adapted to the purpose, as it consists of a bed of alluvium from 20 to 30 feet in depth, and composed of sand and gravel interspersed with a little vegetable mould. It is believed that this natural filter-bed will remain in good condition even after the formation of mud many feet in depth. It has been found, however, that the average depth of deposit for 10 years does not exceed 0.5 of an inch. It appears, also, that these deposits are not foul or dirty, as they contain as much as 50 per cent of siliceous matter which renders them friable and permeable, and the cultivation of the soil each year incorporates this deposit with the soil, resulting in the maintenance and increase of the arable earth.

At the Colorado Springs sewage farm the sewage of 12,000 people has been beneficially disposed of in irrigating 15 acres of meadow and alfalfa and 10 acres of vegetables. This is approximately at the rate of 1 acre to 500 inhabitants. At Pasadena, California, the sewage of 6000 people is beneficially used in irrigating about 40 acres, or at the rate of about 1 acre to 150 inhabitants. At Los Angeles, California, the entire sewage, which averages 105 second-feet flowing constantly, has been used in irrigating 1700 acres; this is the sewage of 50,000 people, so that it was employed at the rate of about 30 individuals per acre, also about at the rate of 40 acre-feet per acre per annum. At Santa Rosa, California, 20 acres of land are employed in disposing of the sewage of 5200 people, and at Helena, Montana, 40 acres in disposing of the sewage of 14,000 people. At the Meriden, Connecticut, broad irrigation and intermittent filtration farm, one of the most modern laid out in this country, the sewage of 15,000 people, amounting to about 3 second-feet running continuously, is being successfully treated and disposed of on less than 14 acres—a rate of 150 acre-feet per acre.

It must be remembered, however, that the above figures do not represent the ultimate duty of sewage as such. They show rather the limits in amount of sewage which may be disposed of, that is, the extreme amounts which may be utilized on a given area without harmful effects, rather than the minimum amount which may be utilized with beneficial effects to the crops. In other words, they show the limits to which sewage may be disposed as a sanitary problem, rather than the limit of crop which may be irrigated with a given amount of sewage as an irrigation problem. It is not unlikely that where sewage is used rather for its value as an irrigating material than otherwise that the sewage of as few as from 50 to 100 people may suffice to irrigate an acre, and that not over 4 to 6 acre-feet in depth per annum, allowing for waste in winter, will produce satisfactory and beneficial effects in irrigation.

121. Methods of Laying Out Sewage Farms and Applying Sewage.—In preparing land for sewage irrigation it must be remembered that sewage cannot be disposed of continuously on the same piece of land with benefit to crops, but that it must be rotated from one plot to another so as to give each a rest and permit of the soil being cultivated and the crops handled. With this end in view it has been found that the most satisfactory way of laying out a sewage farm is to divide it into many very small tracts or plots of about one acre in extent each, so arranged and subdivided by distributing channels that the sewage may be applied to them separately and independently. Experience has shown that first of all the soil must be of suitable texture, and care should be taken in choosing a location in which may be found a deep and light surface soil, underlain if possible by a deep and porous subsoil, preferably of sand and gravel. If the slopes of these are such as to furnish good natural drainage, no difficulty is likely to arise in utilizing such land for an indefinite period of time under proper treatment.

The sewage farm at Meriden, Connecticut, consists of three feet of fine material at the surface, below which is a deep layer of sand and gravel which acts admirably as a filter-bed. This land, however, was at first very much overworked, only a small area being utilized in disposing of the sewage. It has since been successfully revivified and put in condition to suitably operate for many years, by thoroughly cleaning the surface and scraping and ploughing to a depth of 14 inches. Moreover, a number of ditches 3 feet wide were sunk down into the subsoil, the finer material being removed and replaced by gravel. In this way the surface was connected with the gravel or natural filter-bed by the simplest form of artificial drainage. In cleaning the filter-bed it was found that the sludge which had dried hard by exposure to the atmosphere could be raked off into piles and carted away. It was also found that over the greater part of the area the depth of this was scarcely one eighth of an inch after three years of usage, though this increased to nearly 6 inches at the outlet of the sewage drain. In making additional sewage plots at the same farm a depth of about two feet of the close-grained surface soil has been removed and the remainder ploughed to a depth of about 14 inches, reaching a few inches into the gravel beneath. The cost of preparation of this farm has been nearly as great as one thousand dollars per acre, but this is owing largely to the very small area employed and the inferior quality of surface soil. At the Paris sewage farms the water is brought from the city in closed sewers and then in an open drain, and finally is distributed through the 55 acres of irrigated fields in about 3.5 miles of open surface channels.

After a suitable soil has been chosen and the land has been underdrained or otherwise suitably prepared, it should be divided by open drains, preferably lined, into plots of from 200 to 400 feet on a side. The sewage should be brought to the limits of the farm in closed sewer conduits, which must be properly ventilated. It is desirable at the outlet of the conduit at the entrance of the farm to construct a small storage reservoir, suitably lined, since it may be necessary to retain the sewage of at least twenty-four hours, and certainly of a night, at times when it is not possible to use it. A screen should be placed at the head of the farm distributaries in order to keep out such matter as it is not desirable to use in irrigation, and this may be removed at certain intervals, either to waste land, where it may be ploughed under, or may be disposed of by cremation or other process. The most satisfactory mode of constructing such a reservoir so that no odor shall emanate from it is to cover it with a rough board roof and build a ventilating chimney, which can be constructed cheaply of lumber and should not be less than 50 feet in height. Such a chimney is sufficient to prevent any nuisance, either from the reservoir or from sewage flowed therefrom on to the fields. By such means sewage which is used on freshly irrigated land scarcely emits the slightest odor, while none is perceptible immediately after ploughing.

The most satisfactory way of applying sewage for irrigation is through furrows between rows of vegetables, the simple furrow method of irrigation (Chap. XIV) being employed. In some cases, however,—notably at Trinidad, Colorado,—the embankment or check method has been employed, more especially in the cultivation of grain and forage crops. After applying sewage to crops it is left only so long as to permit it to become dry enough to work, when the land is thoroughly tilled and all solid matter turned over before the next application of sewage, while such a variety of crops must be employed as to make the irrigation season as long as possible.

During the non-irrigating period, the winter months, the sewage may be flowed in rotation over various plots of land and be permitted to filtrate through this and find its way back to the natural drainage channels. It is desirable, however, to use precaution and not overcharge the land, and this may be prevented by tilling it a few times during the more open days of winter. As soon as the crops are to be sown in spring it is desirable, should too great an accumulation of solid matter appear on the surface, to rake this off before planting the season's crops. Experience has proven that sewage reaches the lands at a sufficiently high temperature, even in the coldest weather, to permit it to remain unfrozen and to find its way by filtration into the soil.

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PART II.

CANALS AND CANAL WORKS.

CHAPTER VIII.

CLASSES OF IRRIGATION WORKS.

123. Gravity and Lift Irrigation.—Irrigation works may be divided into the above two great classes. Gravity works include all those by which the water is conducted to the land with the aid 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-power;
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.

124. **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 available for purposes of navigation.

125. **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 sufficient 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. They 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 supply 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 supplies from the seepage water with which the soil in nearly every country is permeated (Chap. VII).

126. Inundation Canals.—Inundation canals might also be called flood-height canals, as they are dependent for their water supply on the height of flood rise in the river from which they are diverted. This variety of canals is employed most satisfactorily in connection with rivers which have built up their beds by the deposition of sediment, and therefore practically flow on the summits of ridges. The most notable of these are the Indus in India and the Nile in Egypt. There are a number of such streams in this country, as the Sacramento, and the lower portions of the Yuba and Feather rivers in California, the beds of which are in some places at considerable heights above the surrounding country. The lower Mississippi belongs to the same class of streams.

Inundation canals rarely require any permanent headworks for the control and admission of water to their channels, their heads consisting of a simple cut through the river bank or ridge which separates the river from the low-lying, surrounding country. As they depend upon flood rises for their supply, the beds of these canals are generally at some height above the beds of the rivers from which they are diverted and usually at the level of mean or low water, so that when these streams are not in flood they do not receive any supply and are therefore not perennial canals. Heads on such canals are usually situated in the true bank of the main river from which they draw their supply, but at such a position that they are not in a cutting bank or one against which in its meanderings the river impinges, lest it destroy the bank at this point, and thus the headwork. It is not uncommon, however, to locate them in sloughs or bayous which are found adjacent to rivers of this character. Such sloughs are generally larger than the canals which they thus feed, and the velocity of the water being less in them, as a consequence the silt which might otherwise be deposited in the canals is left in the sloughs.

There are many such inundation canals taken from the river Indus. The difference in elevation from ordinary water stage and inundation level of this river is from 8 to 12 feet. The velocity of the river is in some places greater than the nature of the banks will stand, and therefore the heads of the inundation canals have to be opened afresh every year. These canals vary in bed-width from 6 to 50 feet, and are from 10 to 60 miles in length. They carry depths of water during flood periods of from 5 to 10 feet and their slopes range from 1 in 4000 to 1 in 10,000.

The inundation canals taken from the Nile in Egypt have usually a much lighter slope, ranging from 1 in 20,000 to 1 in 33,000, and some of the more modern of these have permanent control works at their heads with regulating bridges and escapes at places where they are not subject to destruction by floods. The flood rise of the Nile is about 15 feet, and the period of flood, which commences gradually and subsides slowly, is from August to October, and after its subsidence crops are sown in November and reaped in spring. The water of the Nile is not always delivered to the ground as in ordinary irrigation during the growing of crops, but as it must be gotten when the flood is at its height, and this is the case with most inundation canals, it is permitted during the period from August to October to rest in basins formed by levees or embankments separating one basin from another; and by standing in these basins it deposits its silt and enriches the ground and at the same time soaks the ground so thoroughly that after the subsidence the soil retains sufficient moisture to mature crops. In the older methods of irrigation from the Nile the water was supplied directly from the river into the upper basin, and flowed from this through to the lower basins and then back to the river. Latterly, however, some great canals have been built which skirt the bluffs of the river, and instead of pouring the water into the upper basin and letting it run from one basin to another until it reaches the lower and is finally discharged from this back into the river, it is admitted to many basins separately from the canal, thus furnishing fresher water more evenly charged with sediment than the basins would have received by the old method.

The greatest of the Egyptian inundation canals is the new Ibrahimiyah canal, which is diverted from the Nile near Siout, and skirting the western edge of the valley irrigates about twenty great basins and a number of smaller ones, containing in all an area of about 600,000 acres, and in addition 500,000 acres of high-level and Fayoum crops not divided into basins, making a total of 1,100,000 acres irrigated by it. This though really an inundation canal, acts practically as a perennial canal for a time, as its bed is about 6 to 8 feet below ordinary low Nile, when it has a minimum discharge of about 1500 second-feet. This great canal is about 160 miles in length, has a bottom width in its upper reaches varying between 160 and 230 feet, and a maximum depth of 33 feet with a surface slope of 1 in 22,000. Its maximum discharge has been as great as 32,000 second-feet, and it is quite open to the river without any headworks for the control of the water entering it. The first regulating work on its line is at Derout, where the water can be drawn from the canal by great regulating works into five branches or passed into the river through a large escape. At the terminus of this canal, at the lower end of the basin system near Qushesha is the largest masonry escape in the world for discharging the water which comes from the canal and basins. Its maximum capacity is 80,000 second-feet, and it consists of 60 vents of 10 feet each, the maximum height on it being calculated at nearly 15 feet. It is closed by a series of great double gates operated by travelling cranes from the piers above.

127. **Perennial Canals.**—Perennial canals derive their supplies 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-sections must be relatively large, that they may carry a given discharge with the least velocity. They are usually navigable, and in most cases their water supply is abundant.

128. Dimensions and Cost of some Perennial Canals.—In Table XVII, on page 118, 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.

129. 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 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 escape or sand 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.

TABLE XVII.

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	....

CHAPTER IX.

ALIGNMENT, SLOPE, AND CROSS-SECTION.

130. 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 the permanency of the former are the most important items to be ascertained in the preliminary investigation of any irrigation project.

131. Diversion Works.—The diversion works of a canal include the works for directing the water of the stream into the canal entrance, which may be by weir or by training works; the mode of controlling the amount of water admitted to the canals, which may be by regulating gates at its head or by simply making an open cut unregulated as in inundation canals; of scouring sluices, to prevent the deposition of silt at the canal head; of escapes or sand gates in the upper reaches of the canal line for the disposal of surplus water and the removal of sediment; and finally of the diversion line of the canal itself, which is that portion of the canal line required to bring the water to the neighborhood of the irrigable land.

One of the first problems to be settled in the preliminary design and location of a canal is the choice of the point of diversion or site for the headworks. These are almost invariably located high up on the supplying stream in order to command the largest area possible and to receive water from the stream before the latter has become charged with silt which it derives from cutting away its banks in the more low-lying and level country. Occasionally however, where the area of land to be commanded is limited, it may be desirable to locate the headworks at some lower point on the stream. By locating the headworks high up on the supply stream it is usually possible to reach the water-shed or interfluve by the shortest possible diversion line. The chief disadvantage of such a location is that the first few miles of diversion line are sure to be intersected by sidehill drainage, the passage of which may entail great difficulty, and if the slopes of the adjacent country are heavy much expensive hillside cutting.

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 land. The endeavor should always be to so locate the diversion canal as to reduce its length to a minimum, so that the canal shall command irrigable land and thus derive revenue at the earliest possible point.

132. Alignment.—Having determined the source of water supply, and its relation to the irrigable lands, the next 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 summits of the water-sheds dividing the various streams in order that it may command land on each side by its branches.

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.

133. **Method of Survey.**—The surveys which it is necessary to make in properly designing, locating, and constructing a canal may be distinguished as hydraulic, preliminary, and location surveys and construction surveys. The first of these has already been described in Chapters II to V inclusive, and it includes all those problems connected with the quantity, character, and physical properties of the water supply. The construction survey will not be described here other than to state that it is similar to such surveys on railway or other engineering work, and includes careful cross-sectioning of the canal line; the making of detailed estimates of the volume of material to be moved; and the staking out of the line on the ground in accordance with these estimates, in order that the contractors or laborers may know just where and how to work.

The preliminary and location surveys are of a slightly different character, and will therefore be described more fully. Circumstances will determine whether this work shall be divided into two surveys—a preliminary and a location survey, or whether once going over the ground may not suffice for both operations. These surveys may be divided into two general classes: (1) linear or trial-line surveys, and (2) contour topographic surveys. Where a contour topographic map is made, this will usually answer all the purposes of both classes of survey. It must be made to cover such an area of ground as will include all possible diversion lines as well as main lines, branches, and distributaries, in fact, the whole area which comes within the scope of the irrigation projecct. Such a survey and map once made, it is the preliminary survey, and on it the location may be laid down with accuracy and practical finality, and it will in all likelihood be necessary to deviate from such a location but little as construction progresses.

134. **Linear or Trial-line Survey.**—Such a survey may be conducted in the following manner: Assuming that the irrigable lands are situated on a bench or at an elevation above a stream bed which flows between deeply cut banks, and that suitable headworks may be found at several points along the stream, it will be best to start from the upper end of the irrigable area and run a level line back up-stream on the grade chosen until it reaches the stream bed. Should it strike this at a point unfavorable for the construction of headworks, a second trial line may be started at some suitable point above or below this and run back in the opposite direction with the same grade, or, if it has been started up stream, with the insertion of a fall or two; and this operation may be repeated until such a line is obtained as will begin at the most desirable site for headworks and reach the land with the shortest line and at the same time encounter fewest obstacles.

The first trial line or two of levels run as above serve practically as preliminary lines. They should therefore be run in long sights, there rarely being any necessity for great care and accuracy. In all work of this sort it is unnecessary to conduct any other form of survey than a mere level line, the chief object being to ascertain the relations of the elevations of the proposed sources of water supply and the irrigable lands.

Having now determined with approximate accuracy the position of the final location line, this should be fixed by the running of more careful trial lines, accompanied by a transit survey for location, and by cross-sectioning with the aid of a couple of rodmen, both by stadia and level, for a little distance on either side of the line. A decided advantage in this class of work will be obtained by the use of a plane table instead of a transit, since on it can be sketched as the work progresses contour lines which will permit of a greater range of usefulness for the material obtained by the survey.

From the end of the diversion lines thus developed a rough preliminary level and transit or plane-table line should be run back into the country, approximately on a grade contour-line skirting the higher slopes or at the head of the plain or valley, as may be. This line will furnish the data on which to estimate approximately the area of land controlled by this high-level line. These facts ascertained and the relations of the amount of water, its duty and the area of land which it will irrigate, and the total area under command having been determined, trial location lines may be run in similar manner for the selection of the final main line and its branches. The combination of level followed up for location by transit or plane-table with stadia accompaniment for cross-sections will then furnish all the details for final location. In all of the operations thus described substantial bench-marks should be established as frequently as possible, as well as tie points for the horizontal control. When the final location trial lines are being run, more permanent bench-marks should be left at frequent intervals, and in the course of all the surveys connections should be made as frequently as possible with the land-survey system of the country, if the work be in the western United States, in order that the relations of the irrigation project and various lines and ditches to this system of land surveys and the surrounding country may be at once established (Art. 136).

135. Contour Topographic Survey.—In practically all cases where the time and means at the disposal of the engineer will permit, the best results will be obtained by making a detailed contour map of the area under consideration. In nearly all instances it will be best to precede such a survey by a hasty trial-level line or two as already described in the preceding article, in order to ascertain the feasibility of bringing the water to the lands, and in general the area that will be commanded. These questions having been satisfactorily settled, and the approximate location of diversion lines along steep hillsides or canyon walls having been thus ascertained, the contour survey should be made to include only so much of the diversion line as is in question, and of the irrigable area as will lie below the high-line levels.

Where the location of the diversion line is at once fixed by the roughness of the topography, a satisfactory way of making the contour map is to run out a couple of limiting grade contours with level and plane table, the lower being at the lowest possible limit of diversion line and the upper at the highest. In the case of the Santa Anna canal (Art. 143) the vertical distance between these two was found to rarely exceed 70 to 100 feet. These lines should be run in conjunction with stadia cross-sectioning which shall be platted directly on the plane-table sheet, showing the same at once on a scale of 100 to 500 feet to the inch, with a contour interval of 2 to 5 feet. With such a map as this the engineer will be able to lay down almost at once the final location of his line.

Where the diversion line terminates in more open, gently sloping country and the remainder of the canal line is less closely fixed, the following methods should be employed for the survey of the remainder of the irrigable area: Beginning at the end of the survey for diversion line, and following around on the approximate high-line grade, a careful transit and level line should be run to the limits of the highest portions of the irrigable area. From this chained transit and level lines should be run with equal accuracy down the summits of the divides or interfluves separating the main drainage lines, and as these reach the lower limits of the area under examination they should be connected by cross lines. The whole may then be plotted up on a suitable scale, say 500 to 1000 feet to one inch and in 5 or 10 foot contours, and the sheet on which these lines are plotted may then be taken into the field either as a whole or divided into sections, placed on plane-table boards, and the topography filled in thereon. This should not be done by running out the various contours, but by irregularly cross-sectioning or dividing up the area by lines which shall so cut up the territory as to enable the contour crossings of the lines run on the plane table to be connected from one line to the other with an accuracy well within the contour interval, and thus permit of the whole being filled up as a final map. This secondary work resting on the main chained and taped lines is preferably done in a more cheap and expeditious manner by using the plane table instead of the transit, the stadia instead of the chain or tape and in some cases the water-level, or vertical or gradienter angles in place of the spirit-level, though the latter methods for elevations must be adopted and used with the greatest precaution.

On such a map it then becomes an easy matter to lay down the approximate location of the main canal lines and their various branches, the sites of falls, regulating works, and escapes. Thereafter it may be necessary, prior to construction, to make other more detailed contour maps of limited areas, with a view to determining in special cases the more exact location of the heads of branches, the crossing of drainage lines, the position of escapes, and at similar points on the canal line.

136. Right of Way on Public Land; also State Desert Land Grants.—In order to obtain right of way for canals, ditches, or reservoirs on private lands in the West, arrangements must be made, as elsewhere in the United States, by purchase or otherwise, with the individual owner. In order to obtain right of way on public lands, being those which have not yet passed from the ownership of the government by means of homestead, pre-emption claim, or other method; surveys, maps, and construction must be made to accord with certain regulations issued by the General Land Office, whereupon a grant of the land affected by the right of way will be made. This grant, however, is only for purposes of canal or reservoir construction, is dependent on the fulfilment of the conditions required, and is not transferred in fee.

Among other of the requirements of the regulations above referred to, which must be fulfilled in order that right of way may be granted, are the following, which are of chief interest to engineers: The survey in all its parts must be connected with section and township corners of the public-land survey; especially the termini of the canal, ditch, or lateral must be connected with the nearest corner. The notes and all data for the computation of traverses connecting with such corners must be entered in the field notes. The method of running the grade lines must be described, as well as the size, graduation, and make of instrument used. Stations and courses must be numbered, and field notes must show whether the side or middle line of the canal has been run. Maps for filing with the general and local land offices must be drawn on tracing linen, in duplicate, on the scale of 2000 feet to the inch for canals and 1000 feet to the inch for reservoirs, or on a larger scale in special cases. These maps should show all subdivisions of the public surveys, the source and amount of water supply of the reservoir or canal, the details of alignment of canals, and flood lines of reservoirs. Permanent monuments must be set at the intersection of the water-line of the reservoir with the public-land lines, also on either side of the intersection of the canal lines, in such manner as to comply with the requirements of witness corners as laid down in the "Manual of Surveying Instructions," issued by the General Land Office in 1894. The map must also bear a statement of the width of each canal or branch at high-water line, the capacities of reservoirs in acre-feet, and height of proposed dam. They must also show, in ink of a distinctive color, other canals or reservoirs than those for which application of right of way is made. In addition all field notes must be in duplicate, properly dated, and be filed like the maps; and both field notes and maps should bear the certificates of the engineer and president of the company or owner of the canal.

By Act of Congress of August 18, 1894, certain lands in the West which are officially designated as "desert lands" are donated to each State and Territory to the extent of 1,000,000 acres each, on the fulfilment of specified conditions by them as to the manner of their use and disposal. Regulations containing the requirements under which these grants are to be obtained have been issued by the General Land Office, and among other points of interest to engineers are the following:

The State is required to file a map of the land selected which shall show the mode of contemplated irrigation and the source and quantity of the water supply. This map must be on tracing linen, in duplicate, and on a scale not greater than 1000 feet to the inch. With it must be filed the field notes in duplicate, and both map and field notes must show the connection of the various elements of the irrigation projects with the public-land survey corners. An accompanying statement must show that the proposed plans will be sufficient to thoroughly irrigate and reclaim the land. The whole to bear the affidavit of the engineer who prepared the plan.

137. **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 em-

bankment 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.

138. 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. 12. 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-

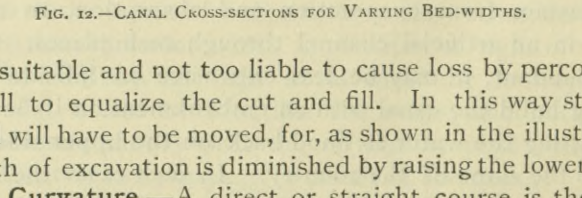


FIG. 12.—CANAL CROSS-SECTIONS FOR VARYING BED-WIDTHS.

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.

139. Curvature.—A direct or straight course is the most economical alignment, 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. On the other hand 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.

140. 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.

141. 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. I). 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 300 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. XXIII). 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 222), 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 229). 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. 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. XXI). 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 distribu-

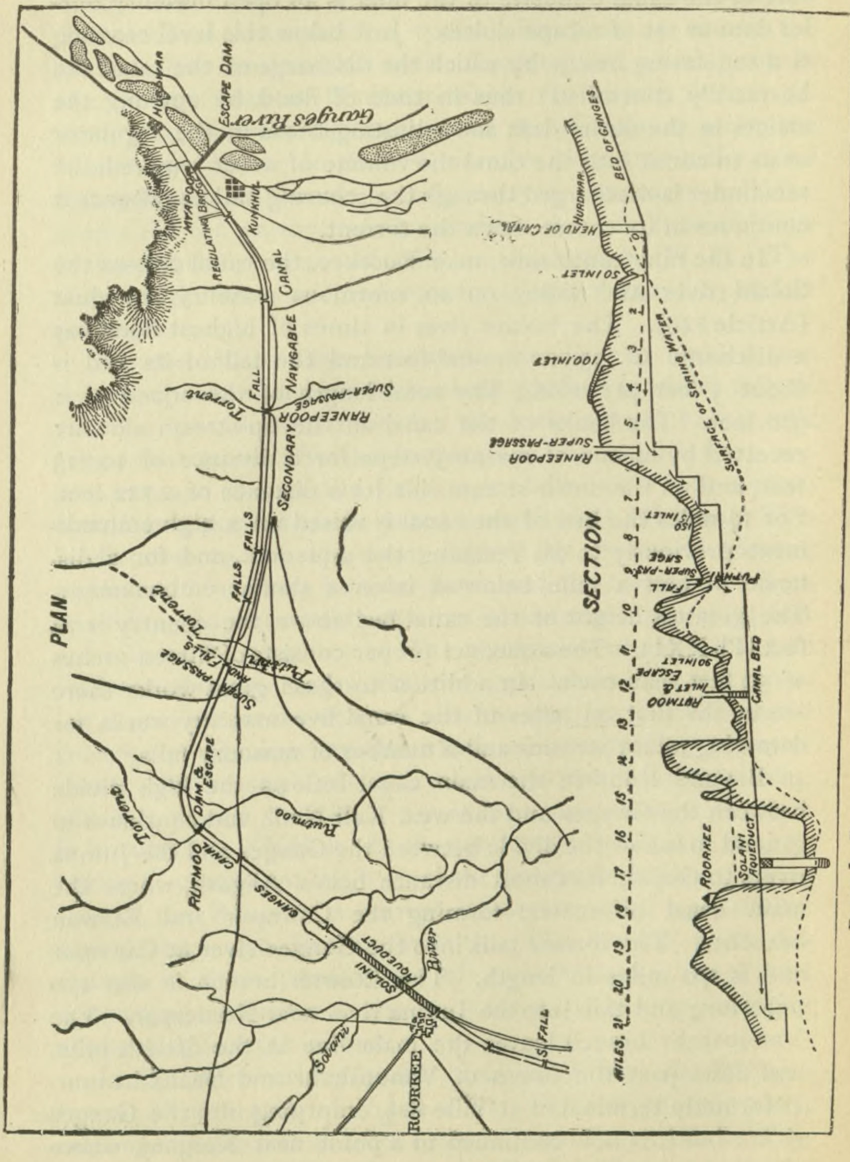


PLATE I.—PLAN AND CROSS-SECTION OF GANGES CANAL, HURDWAR TO ROORKEE, INDIA.

taries 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.

142. 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 contour topographic map had been made of the entire region, and from this the canal line was laid down (Fig. 13).

The headworks of the Turlock canal consist of a masonry dam which is constructed as a common diversion weir for the 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 336 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

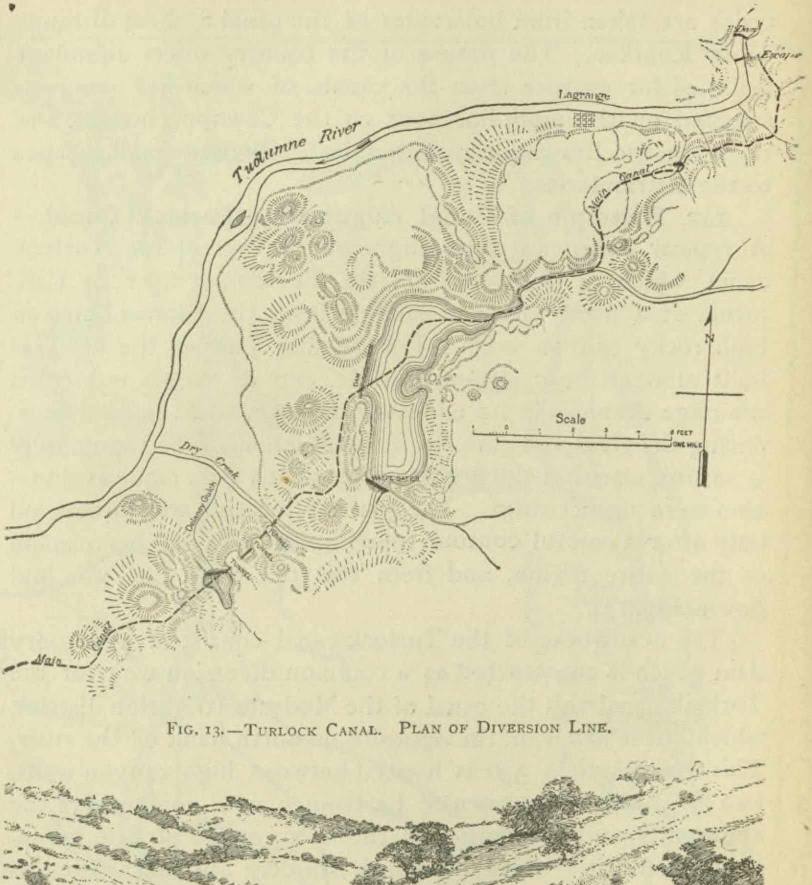


FIG. 13.—TURLOCK CANAL. PLAN OF DIVERSION LINE.

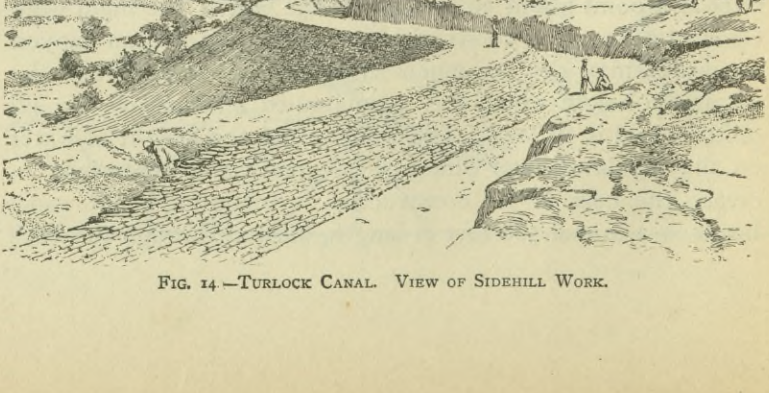


FIG. 14.—TURLOCK CANAL. VIEW OF SIDEHILL WORK.

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

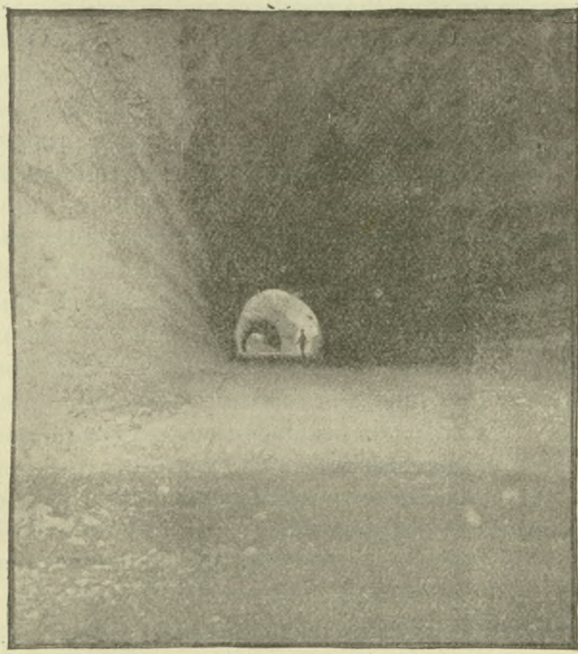


FIG. 15.—TURLOCK CANAL. VIEW THROUGH TWO TUNNELS.

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 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. 14). 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. 21). 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. After the second year this slate-rock so disintegrated by air-slacking that much of it fell away, and the sides of the cutting caved in, requiring extensive rebuilding on firmer lines and flatter slopes.

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 $1\frac{1}{2}$ 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. 15) 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 205).

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 7½ 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 7½ 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.

143. Examples of Canal Alignment, Santa Ana Canal. —One of the most remarkable and typical American alignments is that of the Santa Ana canal, diverted from the Santa Ana river above the head of the San Bernardino valley, California. This canal is especially interesting because of the extreme care and thoroughness exercised by its chief engineer, Mr. Wm. Ham. Hall, in making the preliminary and location surveys for the very difficult diversion line encountered in the first few miles of this work. It is also notable, aside from the alignment, for some very interesting details of construction, chiefly in the flumes, siphons, and sand-boxes encountered at various portions of the line.

The Santa Ana canal is designed to irrigate 45,000 acres in the northern part of San Jacinto valley, which is separated from the canal head by four main divides between the Santa Ana river, Mill creek, Yucaipe, Timoteo and San Jacinto creeks. Through a low neck in the higher ridge immediately north of San Jacinto valley the irrigating company had previously built Moreno tunnel, 2320 feet in length, and of the capacity desired. This tunnel had been well lined with brick and plastered with cement, and therefore became a controlling factor in the location of the alignment from the headworks chosen, as they must of necessity deliver through this tunnel. The source of water supply is the natural flow of the Santa Ana river reinforced by the Bear valley storage reservoir in San Bernardino mountains. The country between the tunnel and the point at which the Santa Ana river leaves the mountains in a canyon at an elevation of 1850 feet, is exceedingly broken. The air-line between these two points is 12 miles, while a grade contour connecting them would be 42 miles in length.

The site of the headworks was selected in a solid rock point of the canyon side where the river bed has an elevation of 2320 feet. The diversion line was designed to carry 240 second-feet of water for the first portion of the line, and thereafter 300 second-feet to a reservoir site on the line of the work, after which it will diminish to 200 second-feet to the Moreno tunnel, so that the latter figure represents the maximum volume which will be carried for direct irrigation. With these controlling factors most of the work has been constructed for a present use of half the capacity as far as the head of the Alessandro pipe line at the terminus of the Moreno tunnel, the total length thus finished to date being 29,095 feet. In addition the canal has been excavated for about 11,500 feet further, and the topographic and preliminary surveys carried to Moreno tunnel, the end of the main work. Beyond this tunnel the work will in future be divided into the Alessandro and Moreno canals and several pipe lines.

The headworks, as already stated, are located opposite a face of solid rock, and the intake of the canal is a tunnel 220 feet in length through this vertical point of rock. The tunnel debouches at a point where a side canyon enters, and to avoid this the canal is built into the hill as a walled cutting, over which the side torrent is carried. From this point the canal rapidly "climbs" the cliffs and mountain sides, for the canyon bed drops away at the rate of 125 to 160 feet a mile, and on its line the tunnels, flumes, and pipes are given hydraulic gradients averaging less than 10 feet per mile. In the course of this line nine spurs are pierced by tunnels having an aggregate length of 4329 feet; three canyons are crossed by pressure pipes or siphons having a total horizontal length of 2127 feet; there are sixteen stretches of flume having an aggregate length of 14,100 feet; one piece of walled canal 152 feet in

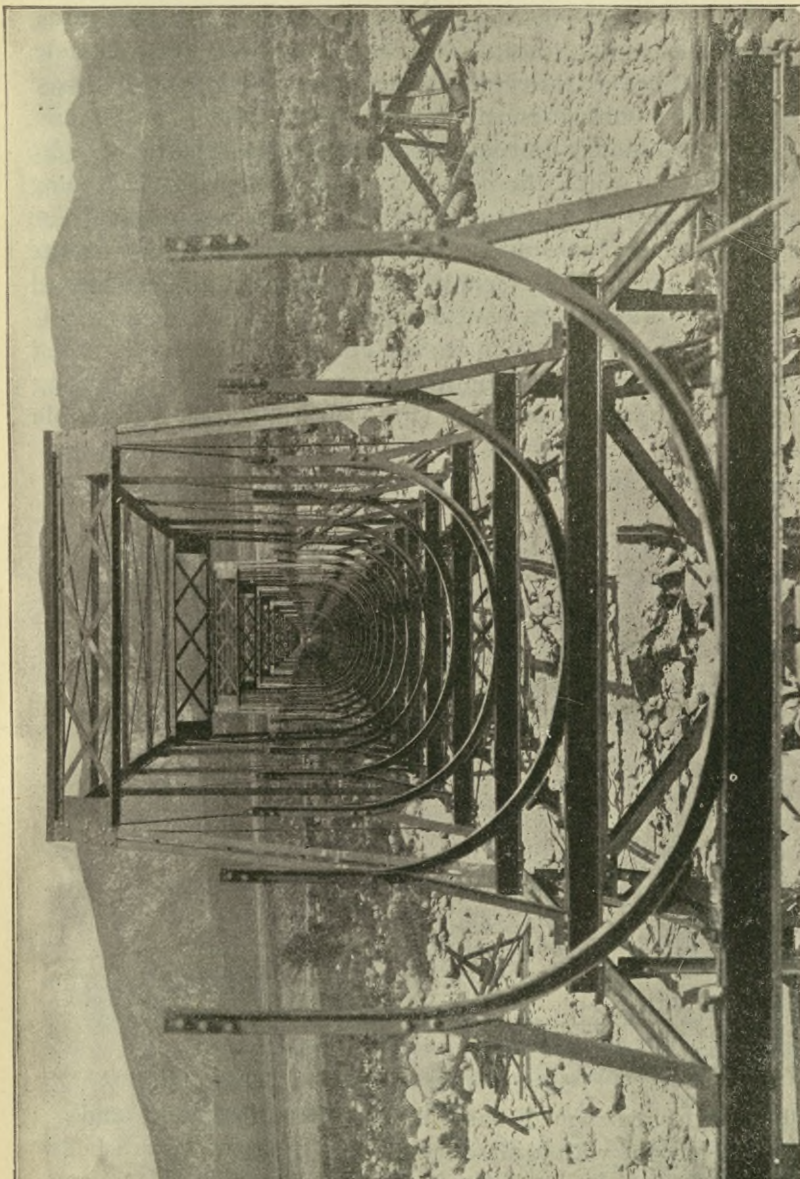


PLATE II.—MILL CREEK FLUME AND STEEL BRIDGE, SANTA ANA CANAL.

length; and eight masonry walled structures, as sand gates, junction bays, and escapes, having a length of 213 feet. In addition there are thirty-nine structures in the nature of trussed girders or combination Fink truss spans resting on timbers piers and masonry footings with an aggregate length of 3345 feet, besides which there are eight stretches of canal having a total length of 8214 feet, lined with rubble in mortar. The largest stream encountered on the completed part of the diversion line is Mill creek, which is crossed in a flume carried on a steel bridge 1072 feet in length (Plate II).

The headworks, which are not yet constructed, will consist of a diversion weir, scouring-sluices, and canal regulating gates. The weir as designed (Fig. 16) will consist of a rubble

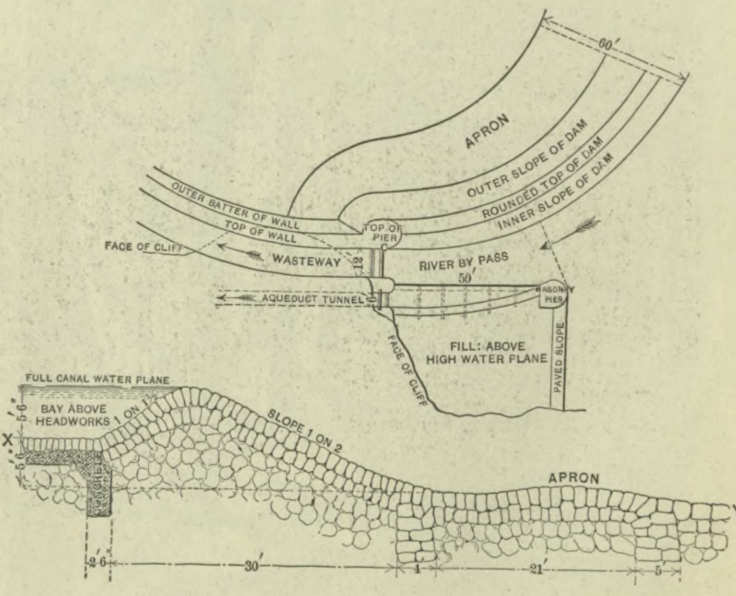


FIG. 16.—PLAN OF HEADWORKS AND CROSS-SECTION OF DIVERSION WEIR, SANTA ANA CANAL.

masonry, ogee-shaped dam about 11 feet in height and 30 feet in length, with a masonry apron 21 feet in length below this. As designed, this weir is peculiarly planned to cross the river, not at right angles to its course, but at such an angle as to force the water over against the tunnel entrance. Between the near end of this weir and the tunnel entrance is constructed a sort of by-pass to the river, divided into two channels—one closed by a series of scouring-sluices for the discharge of sediment and surplus water, and the other for the control of the water supply entering the canal head. Practically at right angles to this by-pass is a line of regulating gates, which admit water to a bay constructed of masonry and terminating in the entrance to the tunnel on a line with the scouring-sluice gates. The by-pass is thus held between the weir on the outside and the inner wall of the chamber or bay, and leads directly to two aqueducts—one the tunnel through solid rock, and the other the wasteway in solid rock and masonry. The scouring-sluice gates, the intake weirs at the bay entrance, and the head regulating gates afford the means of controlling the flow of water.

A rolling drop of one foot is built at the lower end of the bay to facilitate entry to the head gate, while a similar drop of two feet is inserted below the scouring-sluice. This latter is to be closed by an iron gate 14 feet wide, trussed behind to move on rollers operating by hydraulic power, while the regulating gate is to be of similar construction—both to be operated from platforms constructed overhead. These works have been so planned that the general level of the top of the weir shall not be over three feet higher than the minimum height of the bowlder bed of the canyon, and it is expected that the water will be taken from the surface of the stream, and that the scouring-sluice gate will cause an outrush past the intake, with a view to keeping out the sediment and other suspended matter.

The tunnels are sometimes in solid rock, when only the water channel is masonry-lined to produce a smooth surface, and at other times in loose material, when the entire surface is lined. These tunnels have the cross-section shown in Fig. 17, being 5 feet 9 inches in maximum height, with a curved invert below and an arched roof above. The walled canal has a cross-section practically the same as the water channel

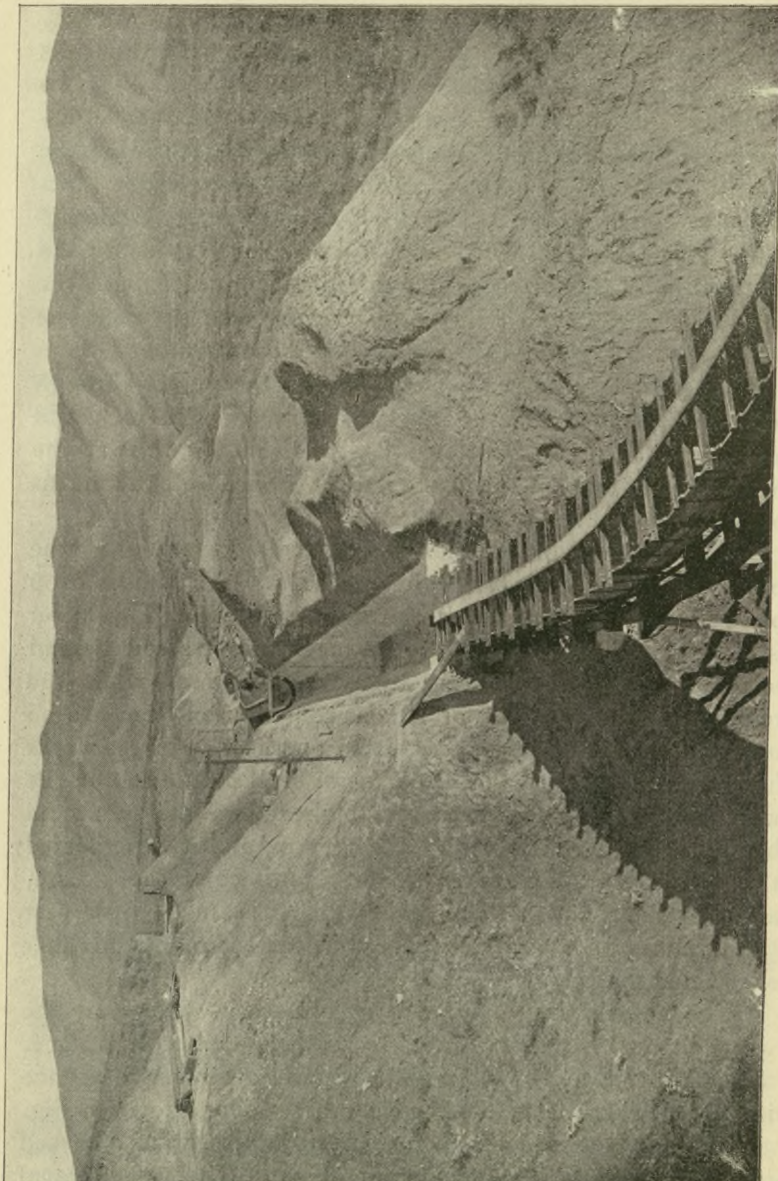


PLATE III.—VIEW ON LINE OF SANTA ANA CANAL.

of the tunnel. The lined canal has a peculiar cross-section, the essential point of which is similar to that of the tunnel and the flumes, namely, an unusual depth in proportion to width (Fig. 20). The flumes are of wood and of the kind which may be called the stave and binder combination, consisting of wood bound together and held in a rounded bottom, straight-sided form, by steel ribs and binding rods, acting in conjunction with wooden yokes or ties across the top (Plate III). The pipe lines and inverted siphons are likewise of wood, and are constructed of staves 2 inches in thickness, 52 inches internal diameter, with carrying capacities each of 120 second-feet, two parallel pipe lines being used on the main line, and they are of the common wooden stave and circular binder-rod pattern (Art. 226). The first sand-box or escape bay (Fig. 57) is located 700 feet from the initial point, while others are planned for different points lower down on the works, should such prove necessary. This first sand-box is 60 feet long by 13 feet wide, and its lower end slopes transversely so that it is 7.3 inches deep on the upper side and 10.6 deep on the lower or discharging side, and it is so constructed as to cause a slacking of the velocity and settlement of sediment at this point. There are six wasteways or escapes in that portion of the diversion line which is now built—four at the upper ends of tunnels 3, 4, 5, and 6 respectively, and at distances of 3200, 6350, 8840, and 12,930 feet from the intake.

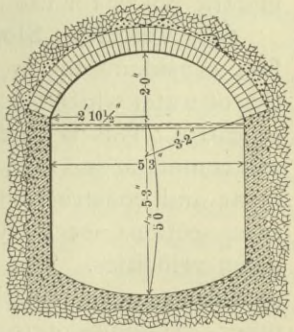


FIG. 17.—CROSS-SECTION OF TUNNEL, SANTA ANA CANAL.

What is expected to be a valuable and profitable feature of this canal is a proposed drop or fall for the utilization of water power. Below the main canal the same company owns two other canals, which will derive their waters from it, and are so situated below it that they will require a discharge of about 60 second-feet falling through a height of 350 feet in a horizontal length of 700 feet. This drop is to be effected through pipes which will render available approximately 2000 horse-power, which will be utilized by turbines in generating electric power for use in the valley below.

144. **Velocity, Slope, and Cross-section.**—One of the first considerations in designing a canal system is the quantities of water which the main line and its branches are severally to carry. This is chiefly dependent on the areas which they will command and the water duty. These determined, the alignment and construction are affected most by the slopes and cross-sections necessary to discharge the quantities required at given velocities. The three factors, velocity, slope, and cross-section, 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 the exercise of 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.

145. 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.

146. 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.

147. Examples of Canal Velocities and 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. 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 in 2112.

148. 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, therefore trapezoidal cross-sections are always employed except in other materials than firm 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, and the trapezoidal section which gives the maximum discharge for any area of waterway is semi-hexagonal, or one in which the hydraulic mean depth equals half the depth of water. Such cross-sections as these, 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. 138), 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 would 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 $22\frac{1}{2}$ feet. Such a depth as this unless in very haid 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.

149. 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. 18). The conditions which govern the choice of one of these three forms are dependent primarily on the alignment and grade of the canal, and second-

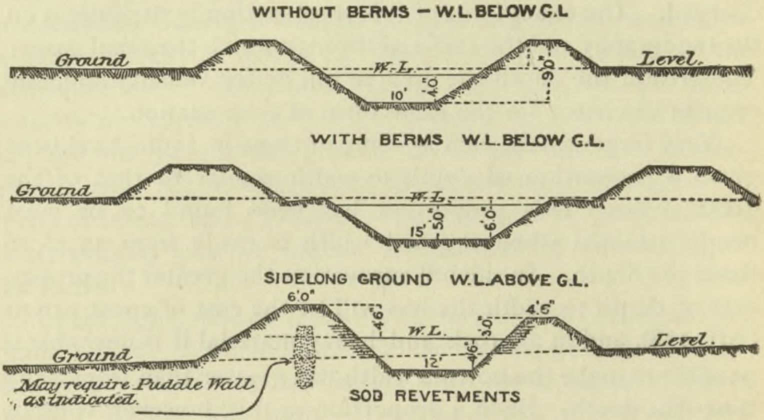


FIG. 18.—VARIOUS CANAL CROSS-SECTIONS.

arily 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 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 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. 18.) 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.

150. 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 between 1 on 1 and 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 and coarse gravel. In ordinary firm soil mixed with gravel or in 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 3 or 4 may be necessary.

The top width of the canal bank is generally from 4 to 10 feet, according to the material and 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.

151. 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. 19) is given by practically designing the canal as

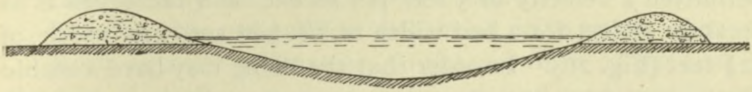


FIG. 19.—CROSS-SECTION OF CALLOWAY CANAL SHOWING SUBGRADE.

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.

152. Cross-section of Lined Canal.—There are many advantages to be gotten from lining a canal channel excavated in earth, especially where the soil is porous and water valuable. In many portions of India and in Europe, particularly where the canal passes through sandy soil, or where for some reasons a high velocity is desirable or unavoidable, the canals are lined for portions of their lengths, usually with hand-placed, dry-laid stone paving. In some portions of Southern California canals are similarly lined, though there, owing to the high value of water, canals are usually lined with rubble paving set in cement or mortar in order to prevent loss by absorption. When a canal is thus lined its cost is in the end not greatly increased, for the saving in cross-sectional area due to the ability to increase the velocity, as well as the great saving of water in sandy and gravelly soils, largely offsets the cost of lining; moreover, it is possible to give a lined channel a cross-section more nearly approaching that called for by theory (Art. 148), namely, a greater relative depth to width, because of the stability added to the banks by the lining.

A typical lining of this sort is that given the Santa Ana canal in California, in alluvial soil, sand, and gravel. This canal is almost wholly in excavation (Plate III); the water is permitted a velocity of 5 feet per second, and the depth is as great as $7\frac{1}{2}$ feet for a bed width of $6\frac{1}{2}$ feet and top width of $12\frac{1}{2}$ feet (Fig. 20). In order that the lining may have a stable

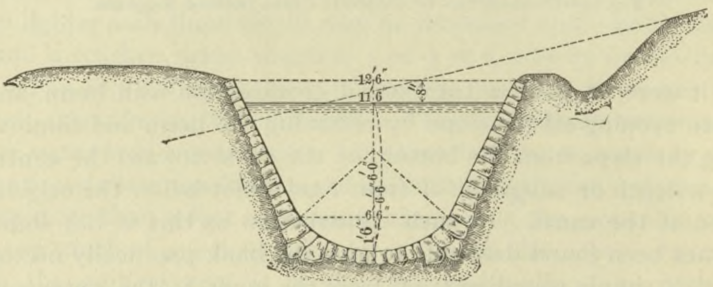


FIG. 20.—CROSS-SECTION OF LINED CHANNEL, SANTA ANA CANAL.

footing and the bottom be less liable to bulge, this is curved downward with a versed sine of $1\frac{1}{2}$ feet, forming thus a sub-grate of that depth. The banks are 2 feet higher than the water surface, and are built on side slopes of 2 on 1. The earth excavation had a bottom width of 7 feet and the same slopes as above, and was trimmed at bottom to the lining, which consists of cobbles and bowlders laid in mortar and grouted and faced with cement plaster.

153. 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.

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

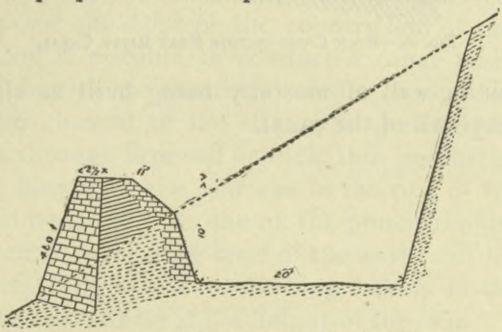


FIG. 21.—ROCK CROSS-SECTION. TURLOCK CANAL.

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. 21). In this instance a retaining-wall of hand-placed stones, with an outer slope of 4 on 1 and a top width of $2\frac{1}{2}$ 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. The slopes have proven too steep, as the friable shale has disintegrated and caused the made banks and the sides of excavated banks to crumble and fall. On the Bear River canal in Utah, the cross-section shown in Fig. 22 was given in order to avoid too much excavation in extremely rocky side-

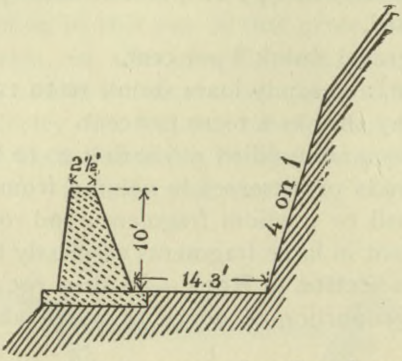


FIG. 22.—ROCK CROSS-SECTION BEAR RIVER CANAL.

hill, a retaining-wall of masonry being built as the lower or embankment-wall of the canal.

CHAPTER X.

HEADWORKS AND DIVERSION WEIRS.

155. **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 of its construction and maintenance. As one of the principal objects of the diversion weir is to raise the level of the water and force it into the canal head, one of the controlling factors in determining the site of the headworks is the height of the weir. This again is dependent on the effect of various weir heights on the location and cost of the remainder of the headworks and of the diversion canal. Also on the flood discharges, amounts of sediment carried by the stream, the foundation, the depth of water in the canal, and similar factors.

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. In selecting the site for the headworks it is desirable to choose a portion of the stream in which its channel is straight and its cross-section uniform for some distance. If the site is in a wide portion of the stream the weir should be located at a point where the stream shows no signs of shifting its channel.

156. 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.

157. 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. The integrity of weirs is in constant danger of destruction; 1, by actual breaching by the force of the current; 2, by undermining by the falling water; 3, by outflanking, especially where the banks are unstable or not protected by substantial wing-walls; 4, by undermining on the upper side by parallel currents owing to the weir not being at right angles to the course of the stream. This may be remedied by the building of suitable training-works (Art. 184).

158. 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.

Foundations of the first class, viz., in rock or bowlders, are found in the foothills and at canyon exits, and where the slope of the stream-bed exceeds 8 feet per mile. Those of the second class are found in valleys and on the plains where the stream-bed has slopes less than 8 feet per mile.

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.

159. 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.

160. 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.

161. 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 a large portion of its length and for the full height of the weir.

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. A rapidly rising flood is more dangerous than a slowly rising flood, not only because of its greater velocity, but because it causes a greater head or fall over the weir as the water has not had time to back up below and form a water-cushion. For the same reasons a falling or diminishing flood is less dangerous than a rising flood.

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.

162. 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. 23), which has been employed at the heads of the Del

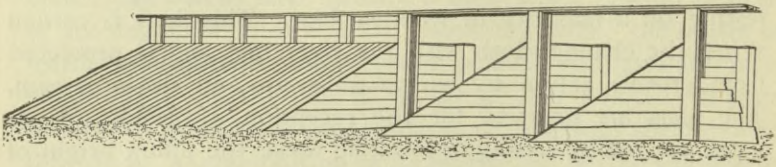


FIG. 23.—OPEN WEIR. MONTE VISTA CANAL.

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 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 (Pl. IV). An example of these is the weir at the head of the Calloway canal (Fig. 24), which consists of 100 bays, each separated by a simple open triangular framework of wood founded on piles, the width of 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 2½ 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.

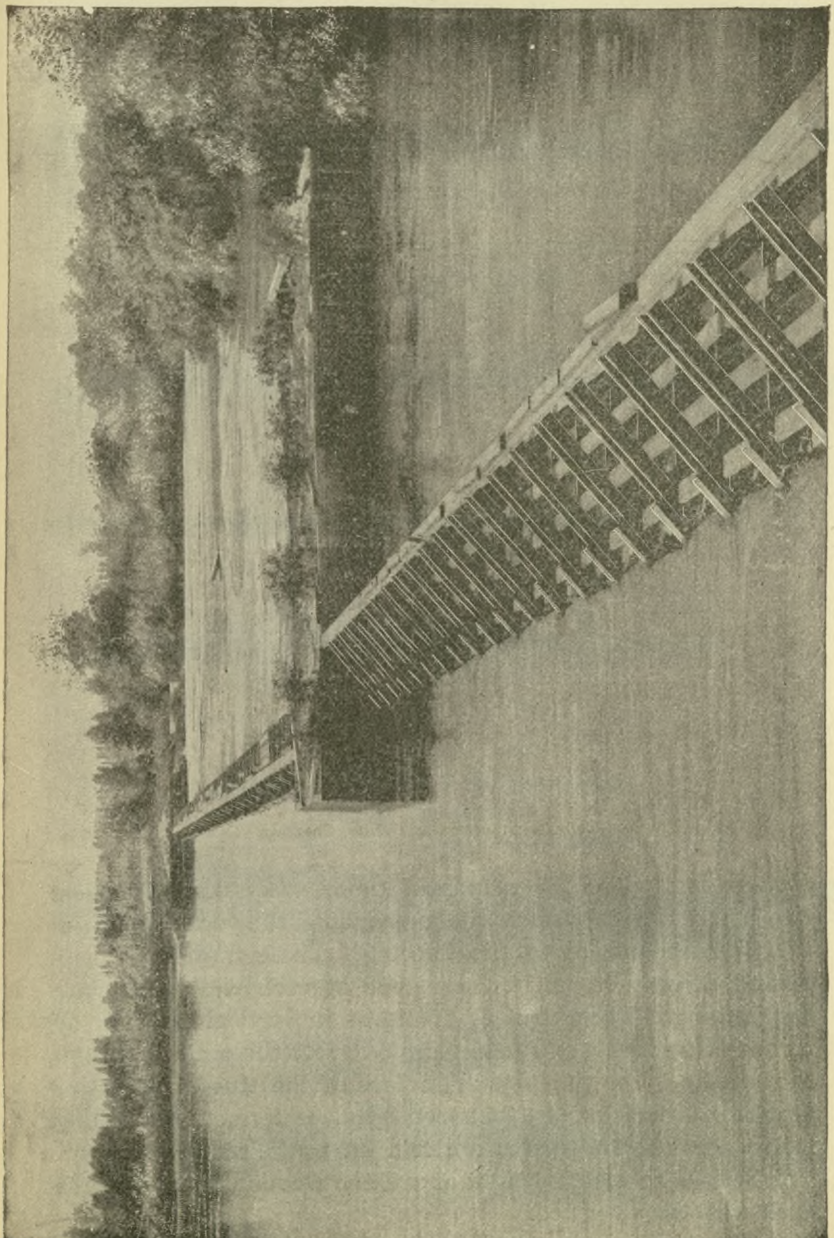


PLATE IV.—KERN RIVER DIVERSION WEIR. HEAD OF CALLOWAY CANAL.

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 4 feet apart. These frames consist of 2 pieces, the up-stream piece being 15 feet 2 inches long and set at an angle of 38 degrees, while the other supports it at right angles and is 9 feet 4

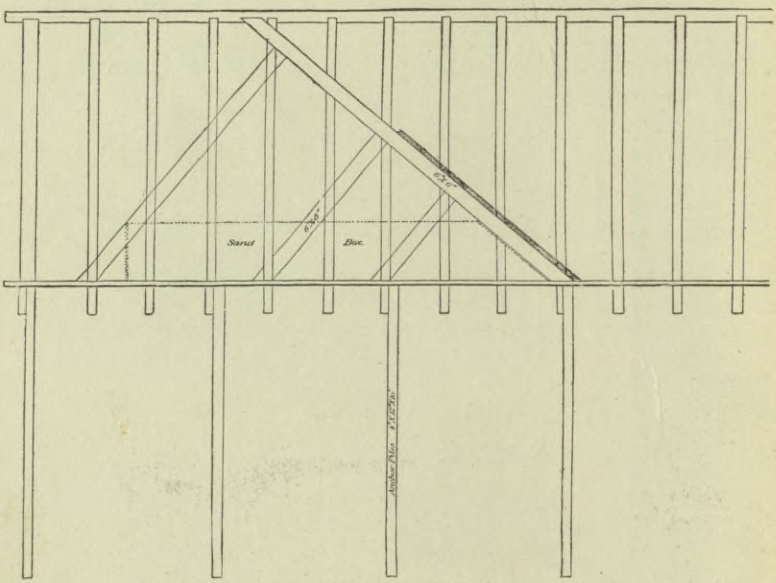
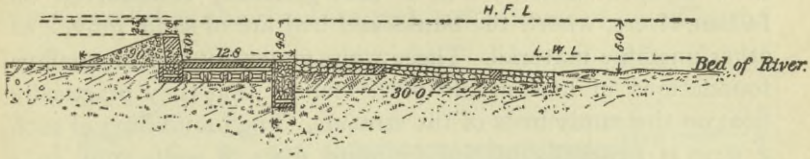


Fig. 24.—Cross-section of Open Weir, Calloway Canal.

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.

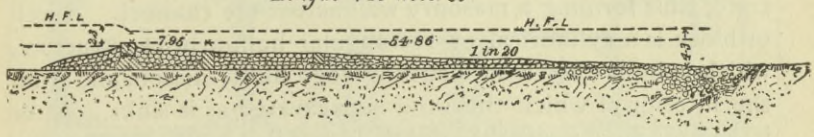
NARORA WEIR—LOWER GANGES CANAL.

Length 1260 metres



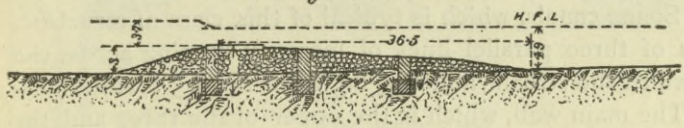
OKHLA WEIR—AGRA CANAL.

Length 743 metres



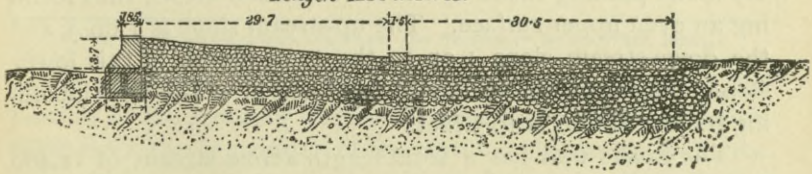
DEHREE WEIR—SOANE CANAL.

Length 3825 metres



BEZWARA WEIR—KISTNA CANAL.

Length 1150 metres.



GODIVERY WEIR.

Length 6274 metres.

PLATE V.—CROSS-SECTIONS OF INDIAN WEIRS.

163. 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. A well or block is a cylindrical or rectangular hollow brick structure, which is built upon a hard cutting edge like a caisson, and from the interior of which the sand is excavated as it sinks. After it has reached a suitable depth it is filled with concrete, the whole depending for its stability on the friction against its sides. This form of construction is illustrated in Pl. V, 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 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. 25) one in the centre and two 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

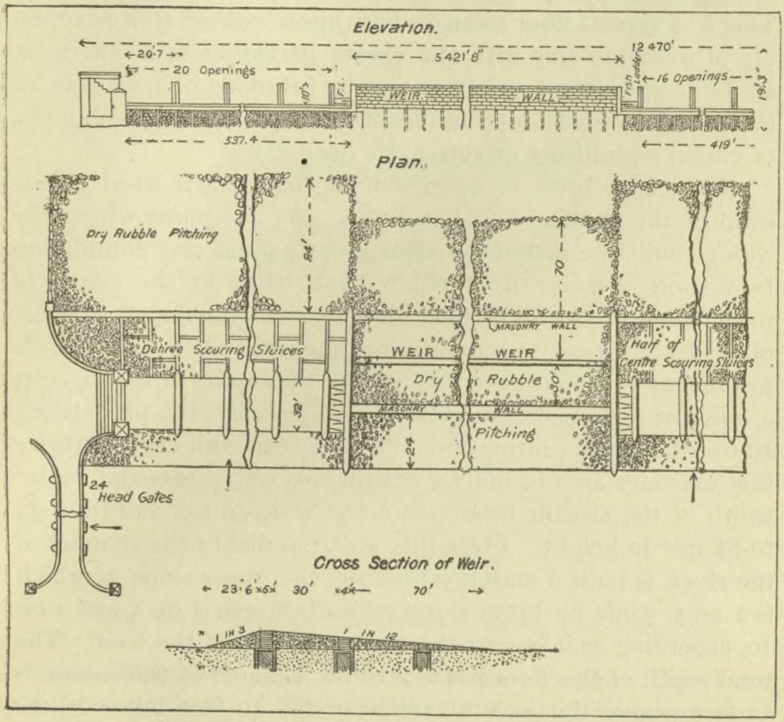


FIG. 25.—HALF-ELEVATION AND PLAN, AND SECTION OF SOANE WEIR, INDIA.

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. 26) 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 stream from 4 to 6 feet wide on top and from 3½ to 8½ 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 2½ feet up-stream from the central wall and 9 feet down-stream, their total length parallel to the channel being 15½ feet and their width on top 6 feet. The crests of these pillars are 6½ 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 6½ feet above the stream-bed. The openings between these piers are closed by means 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.

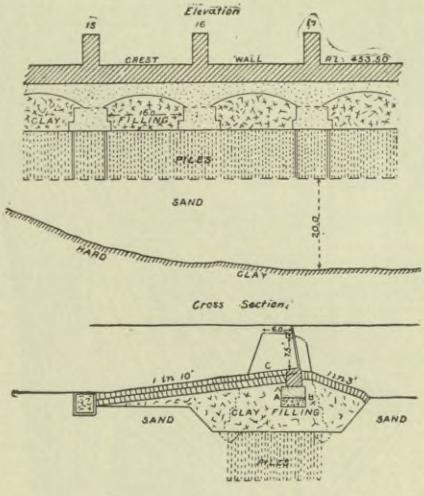


FIG. 26.—ELEVATION AND CROSS-SECTION OF SIDHNAI WEIR, INDIA.

164. Movable Iron Weirs, French Type.—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 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. 27).

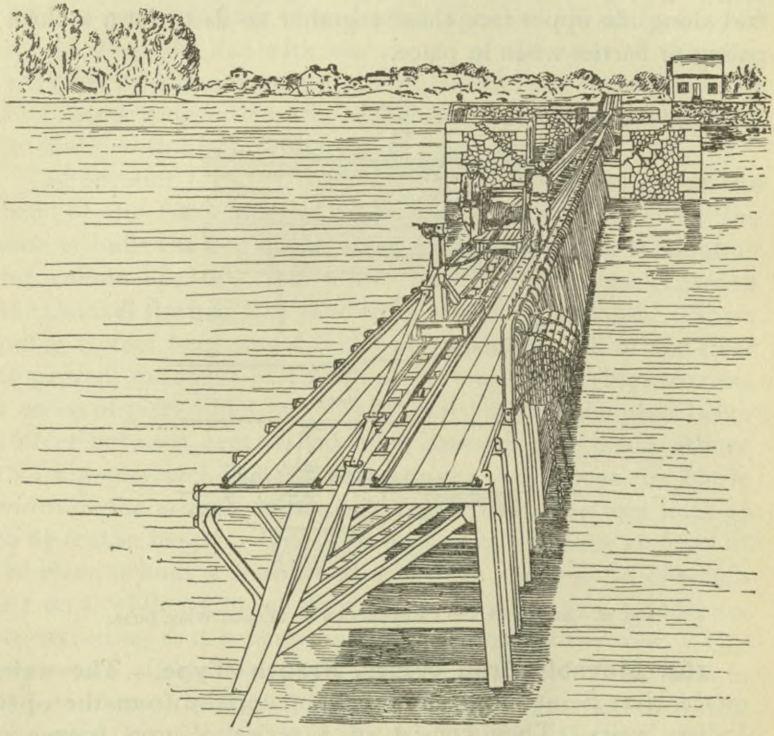


FIG. 27.—VIEW OF OPEN WEIR ON RIVER SEINE, FRANCE.

165. 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

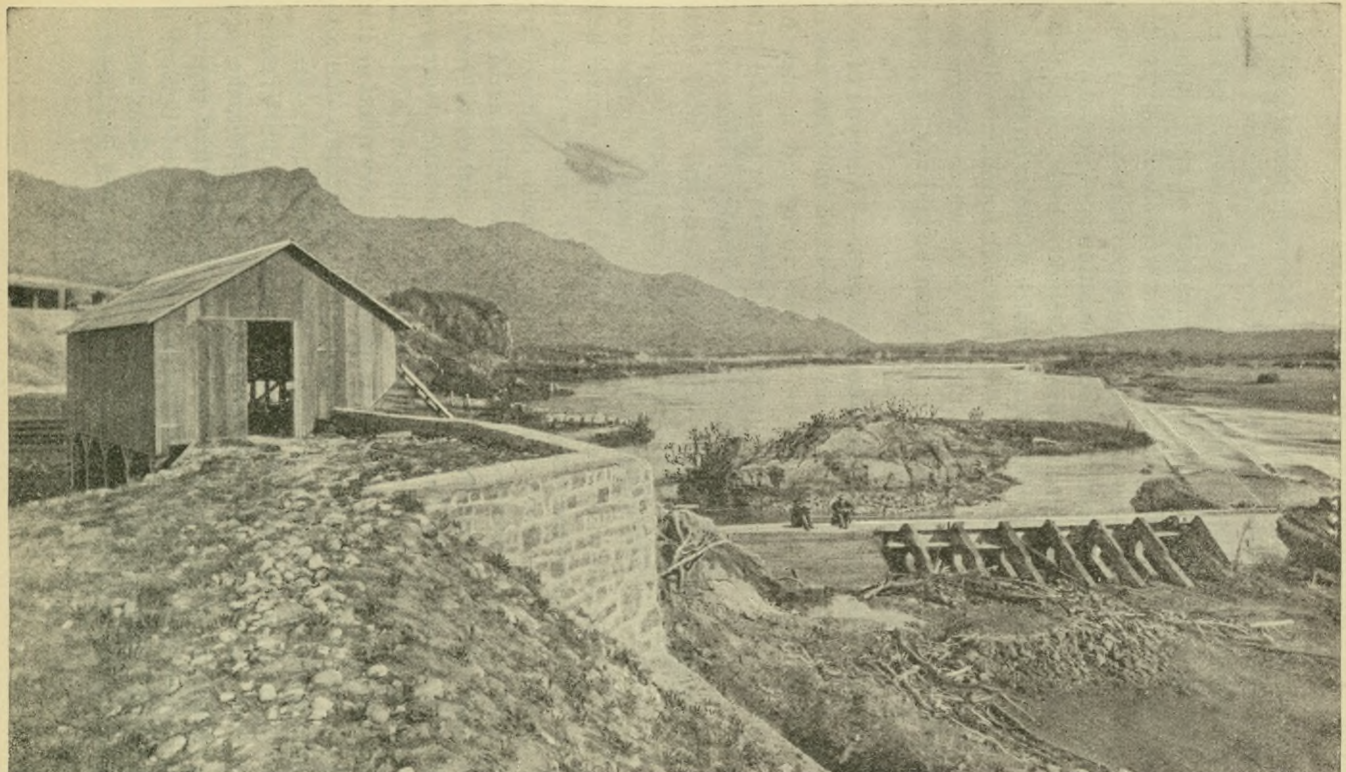


PLATE VI.—VIEW OF WEIR AND SCOURING SLUICES, HEAD OF ARIZONA CANAL.

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.

166. 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. VI) 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. 28). The first section consisted of two rows of cribs, the upper faces of which were given a slight

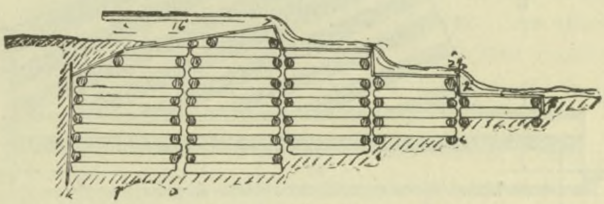


FIG. 28.—CROSS-SECTION OF ARIZONA WEIR.

batter, and on them silt has since deposited and helps to weight the structure. Immediately below the crest and with its upper surface 2½ feet lower is another row of cribs which drop off 2½ feet to the third row of cribs, below which at a distance of 2½ 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 is 370 feet in length on its crest, which is 17½ 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. 29). The up-stream face has a slope of 1 on 2 while, that of the down-stream face is 1 on ½, 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

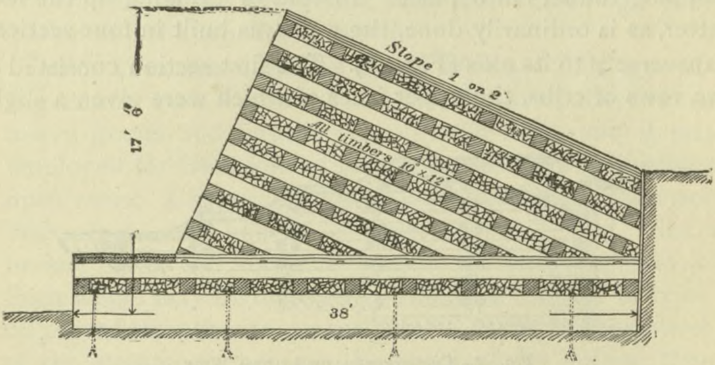


FIG. 29.—CROSS-SECTION OF BEAR RIVER WEIR.

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. 30). 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 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.

167. Rock Fill and Crib Weir.—This form of construction in diversion works contains many of the features of loose rock dams which are fully described in Chapter XVI. The chief reason for describing such structures under the above title in this place is because of their uses as diversion and overfall weirs; while loose-rock dams, though they may be used for diversion purposes, are not designed with the idea of permiting water to flow over their crests.

The one noteworthy example of this form of structure is the weir which was built across the Bruneau river at the head of the Owyhee canal in Idaho in 1893. After three years there is no appreciable displacement or settlement in this structure, and the loose-rock foundation has served apparently as perfectly as solid rock, in spite of the pressure of great floods passing over the weir crest. A rock-fill form of construction was used because of the abundance of broken stone in the narrow gorge at the weir site, the impossibility of building a wasteway, and the necessity of constructing in twenty feet of

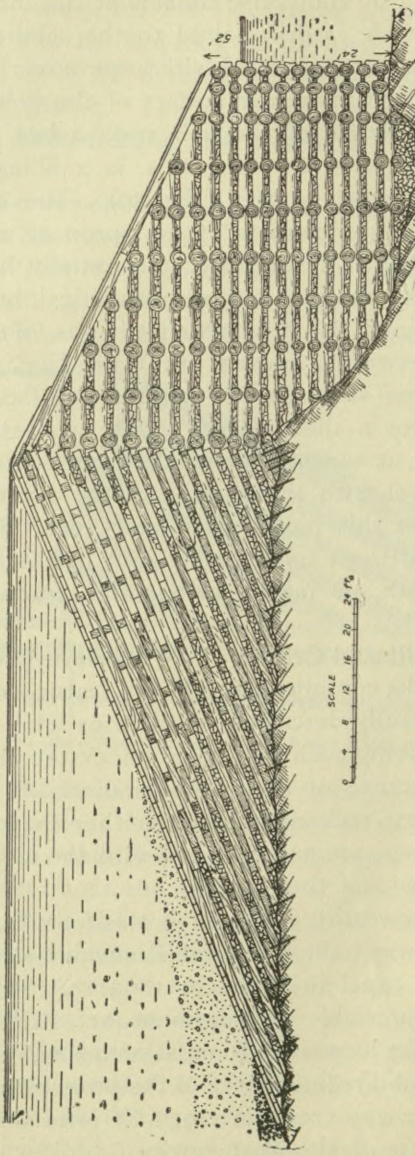


FIG. 30.—CROSS-SECTION OF HOLYOKE WEIR.

water, which would have made the foundation for nearly any other form of structure exceedingly expensive. The weir consists of a loose-rock bed or foundation 176 feet long across the canyon, 170 feet wide at the base, 110 feet wide on top, 20 feet in maximum height, with its top reaching 5 feet above the water (Fig. 31). On top of this rock-fill as a

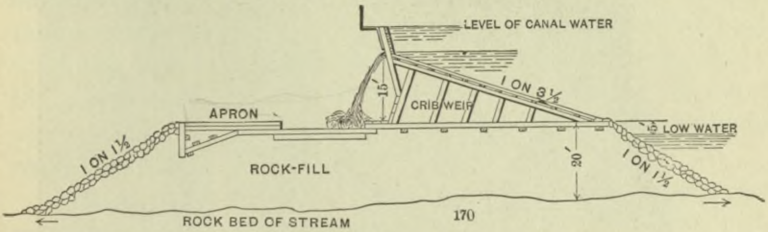


FIG. 31.—ROCK-FILL AND CRIB WEIR, BRUNEAU RIVER, IDAHO.

foundation an ordinary timber crib dam has been constructed, the crest of which is 15 feet above its foundation at the crest of the rock-fill, and on a level with the bottom of the canal. On the crest of the timber cribwork are erected a set of scouring-sluices for the entire length of the weir crest (Plate VII), which consists of posts projecting above the weir crest to a height of about seven feet and carrying on their ends iron brackets. Upon these brackets are laid a footwalk, from which flashboards are inserted into the openings between the posts. The weir terminates at either end in massive masonry abutments or wing walls, which reach 10 feet above the crest of the crib weir and connect it with the canyon slopes. The crest of the crib weir is 176 feet long, its up-stream slope is 1 on $3\frac{1}{2}$, and its down-stream slope is nearly vertical, being at right angles to the other slope. The upper surface of the crib weir is faced with 4-inch plank supported against water-pressure by posts resting on mudsills of 12 by 14 timber crossing each other at right angles, and forming squares 8 feet between centres. These sills are bedded in trenches in the loose rock of the foundation crest. From the crest of the wooden crib weir the water falls 15 feet to an apron of heavy timber which covers as a flooring the down-stream half of the

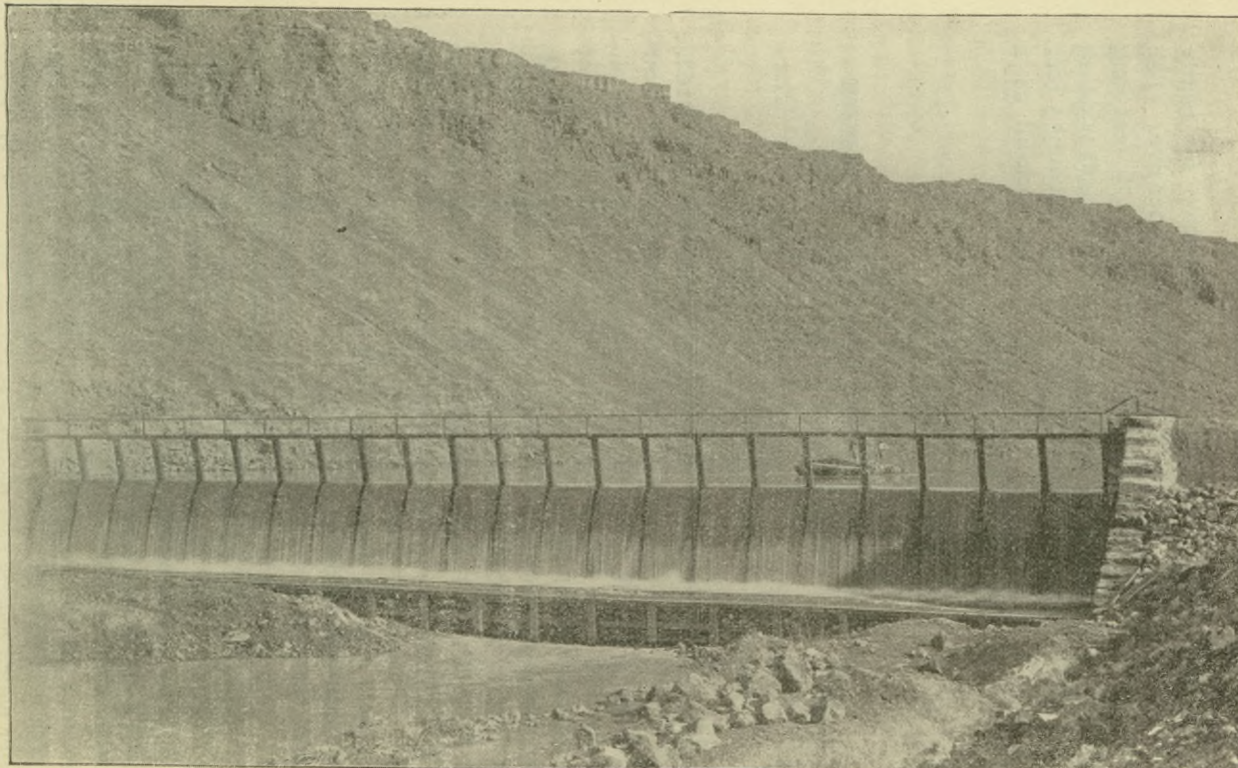


PLATE VII.—VIEW OF ROCK-FILL AND CRIB WEIR, BRUNEAU RIVER, IDAHO.

loose-rock foundation, while the cribwork itself rests on the up-stream half of the same.

During construction the discharge of the river was passed through the interstices of the rock filling, leaving its top perfectly dry, so that the operation of building the timber cribwork was carried on without regard to the water flowing beneath. After the completion of the structure the foundation was made water-tight by dumping gravel from a flat boat upon the upper face of the rock-fill. About 200 feet below the main weir is a subsidiary rock-fill which breaks the water up so as to form a water cushion over the toe of the main weir.

168. 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. 32) is of gravel and loose ma-

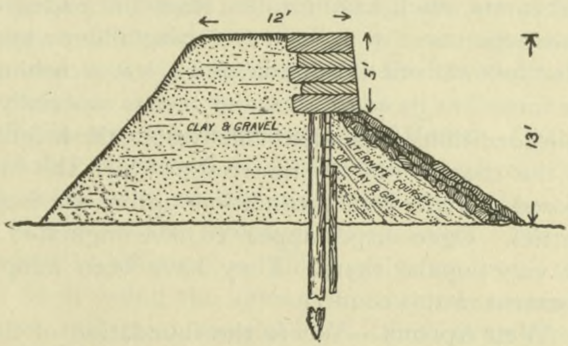


FIG. 32.—CROSS-SECTION OF LITTLE KUKUNA WEIR.

terial, 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 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 $1\frac{1}{2}$.

169. 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.

170. 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 length 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.

171. 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. V. 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 velocity and diminish its erosive action. In our own country a similar long sloping rollerway is that on the Holyoke weir (Fig. 30).

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. 33). 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

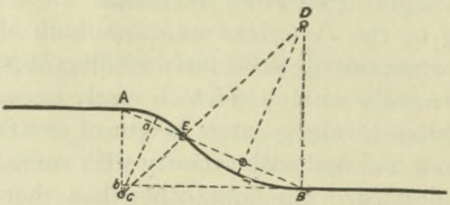


FIG. 33.—DIAGRAM OF OGEE CURVE.

projected on B at D. From the points C and D as centres, draw the curves of the ogee

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

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

Excellent examples of rollerway weirs with ogee-shaped curves are illustrated in Plates VIII and IX, and examples of storage dams with wide crests and curved overfalls in Articles 311 and 312.

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.

172. 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.

In this country a few 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 336). 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 downstream 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 180) and the Betwa weir (Article 335). 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. 106. 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.

173. **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.

174. **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. 34) 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 upper 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

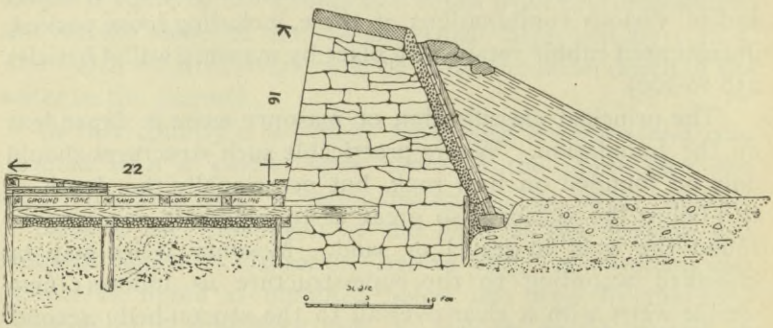


FIG. 34.—CROSS-SECTION OF NORWICH WATER POWER COMPANY'S WEIR.

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.

175. 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 thickness being 5 feet. The crest of the weir is from 15 to 10 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.

176. 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, while the remainder is founded on a stratum of alluvial soil containing bowlders. The piers (Plate VIII) 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

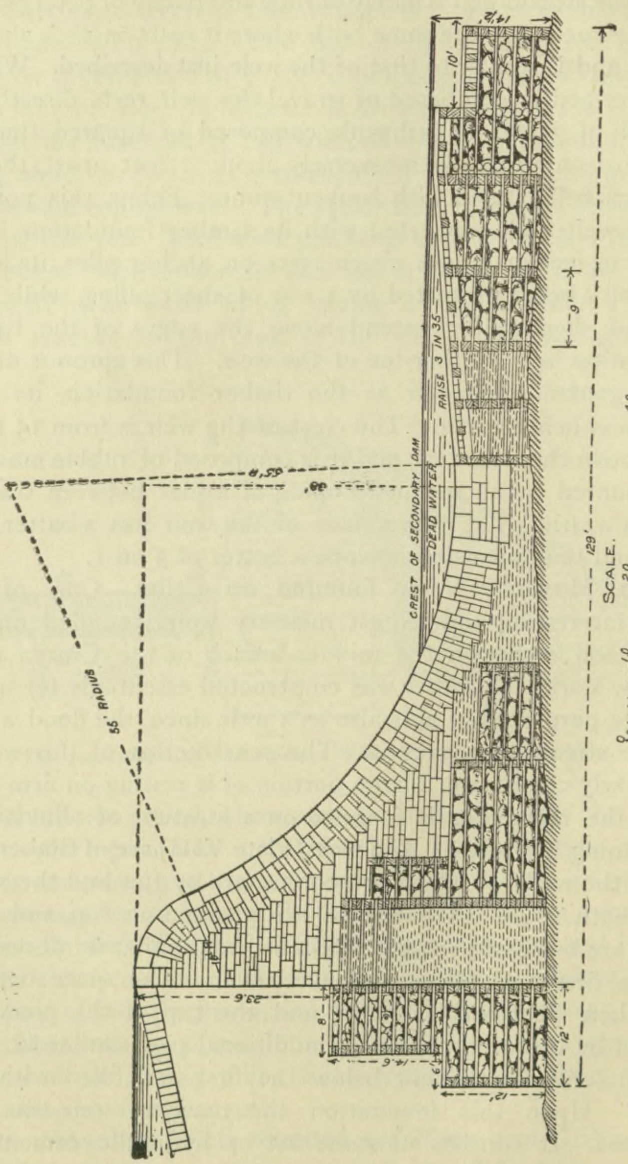


PLATE VIII.—CROSS-SECTION OF CROTON DAM.

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 $23\frac{1}{2}$ 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 $11\frac{1}{2}$, and the amount of this rise is $2\frac{1}{2}$ 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.

177. 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. V are illustrated several examples of these structures, while that built across the Soane river is described in Article 163. 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 9½ 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

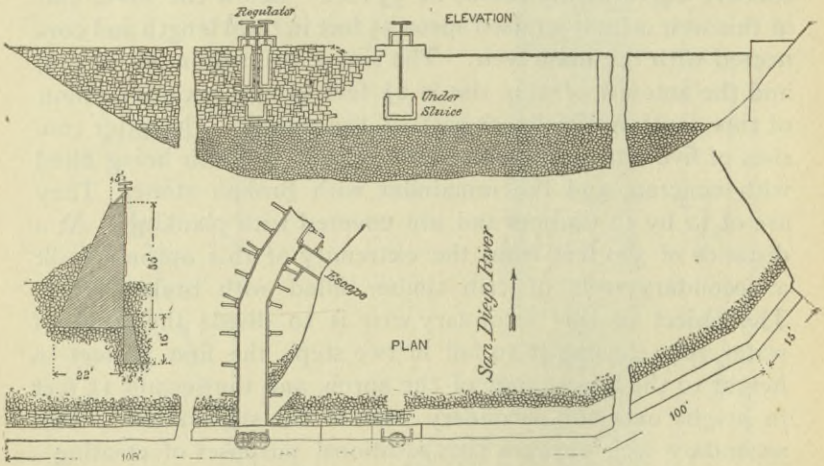


FIG. 35.—PLAN, ELEVATION, AND CROSS-SECTION OF SAN DIEGO WEIR.

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.

178. 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. 35) 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 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.

179. Henares Weir.—This weir is at the head of the Henares canal in Spain. As shown in cross-section (Fig. 36) it

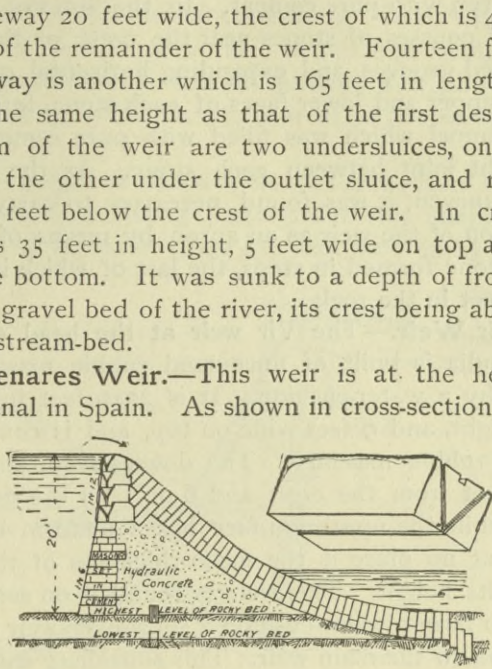


FIG. 36.—CROSS-SECTION OF HENARES WEIR, SPAIN.

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 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.

180. 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, 43½ 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 24½ 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.

181. Cohoes Iron Ogee Rollerway Weir.—A most interesting type of rollerway weir with ogee shape so as to conform as closely as possible to the curve of falling water is that of the great State dam at Cohoes, N. Y., across the Mohawk river. The most interesting feature of this remodelled weir is the use of iron and steel to give the required surface instead of cut masonry. As a result an equally substantial face and apron have been obtained at considerably less expense than would have been possible with finely dressed ashlar, which is commonly used under similar conditions. Otherwise the weir is practically a masonry structure, as this iron facing has been placed over and bolted to the old masonry weir, and the latter has been filled up with concrete cement to the surface of the iron facing.

The total length of this weir is 1611 feet (Plate IX). The facing is backed by strong plate-girder ribs shaped to the final contour of the weir (Fig. 37), and secured by bolts and

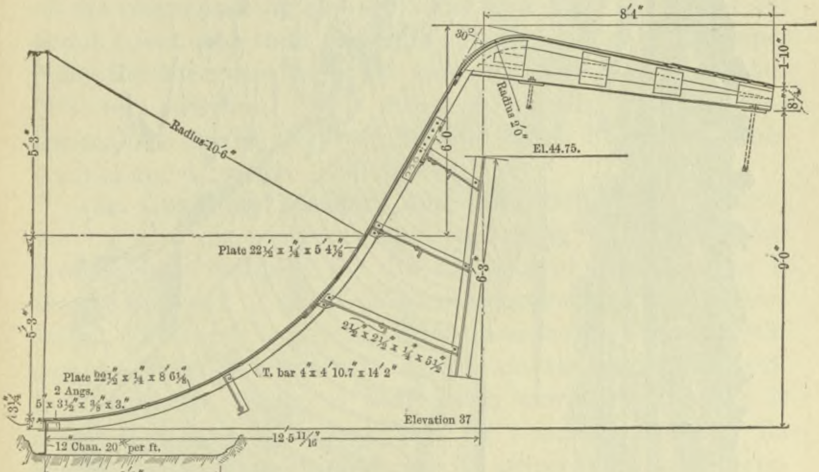


FIG. 37.—CROSS-SECTION OF IRON WEIR, COHOES, N. Y.

braces to the masonry work of the old structure. Sheet iron is secured in grooves in these ribs so as to fill it up as a flush and smooth face, and concrete has been rammed in, forming a solid backing. In constructing this weir the lower half of the apron was first built and the space between the iron plates and the masonry filled with the concrete, then the upper half was built in the same manner and the concrete carried up behind the iron plates at the top. At the foot of the curved rollerway is a short continuation of heavy plate iron about 5 feet in length, fastened to the lower girder of the rollerway on one side and to wooden beams let into the rock of the

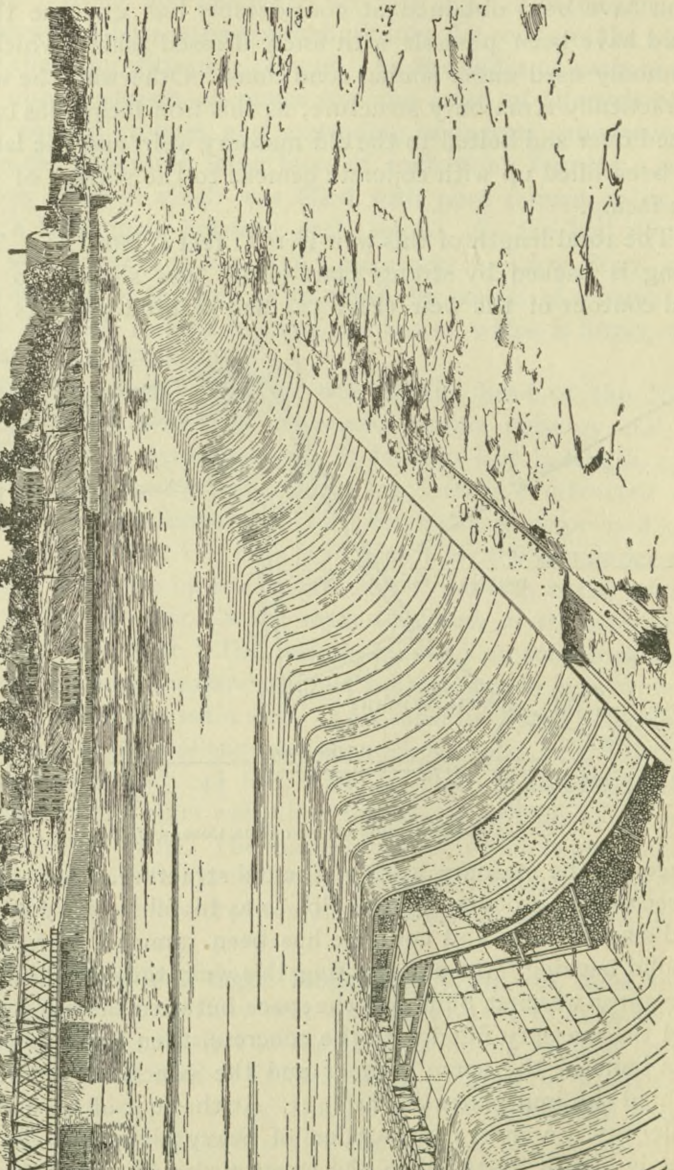


PLATE IX.—IRON-FACED ROLLERWAY WEIR, COHOES, N. Y.

river bed at the outer end. This acts as an apron, the surface of which is flush with that of the rock bed of the river.

The framing of this iron weir face is 12½ feet in length horizontally from the crest of the weir to the lower end of the rollerway, while the top of the coping is 8 feet 4 inches in length and slopes back with a fall of 1½ feet. The total height of the weir, from the crest to the foot of the rollerway is 10 feet 6 inches. The coping girder is 8¼ inches deep at the back or up-stream end and 18 inches in depth at the crest-curve, which latter has a radius of 2 feet from the upper end of the crest-curve or coping. The weir face is straight for about 5 feet and then curved for the remainder of its slope, being the lower portion of the face of the weir, with a radius of 10 feet 6 inches. It is thoroughly braced by angle-irons backed into the masonry behind, and forms altogether a substantial and attractive looking structure.

182. Goulburn Masonry and Iron Drop-gate Weir.— One of the most modern and interesting weirs which has recently been designed for the diversion of storage water is that at the head of the Goulburn irrigation system in Australia. This weir is a clear overfall weir for its whole length, and at each of its abutments there heads a main line of canal. In addition to acting as a diversion weir this structure is intended to act as a storage dam for a small portion of its height, its available storage capacity being about 1000 acre-feet, though it is expected that this can be filled up several times in a season. On the crest of the masonry structure are built up iron pillars between which slide lifting gates which can be raised and lowered by hydraulic power, and add thus to the diversion height of the weir and furnish storage capacity equivalent to this height. The highest flood known in the river was estimated to discharge 50,000 second-feet, and the wasteway capacity furnished between the crest of the masonry work and the soffit of the overhead bridge is capable of passing a larger volume than this.

The Goulburn weir is founded on alternate beds of sandstone, slate, and pipe-clay, standing almost vertically on edge.

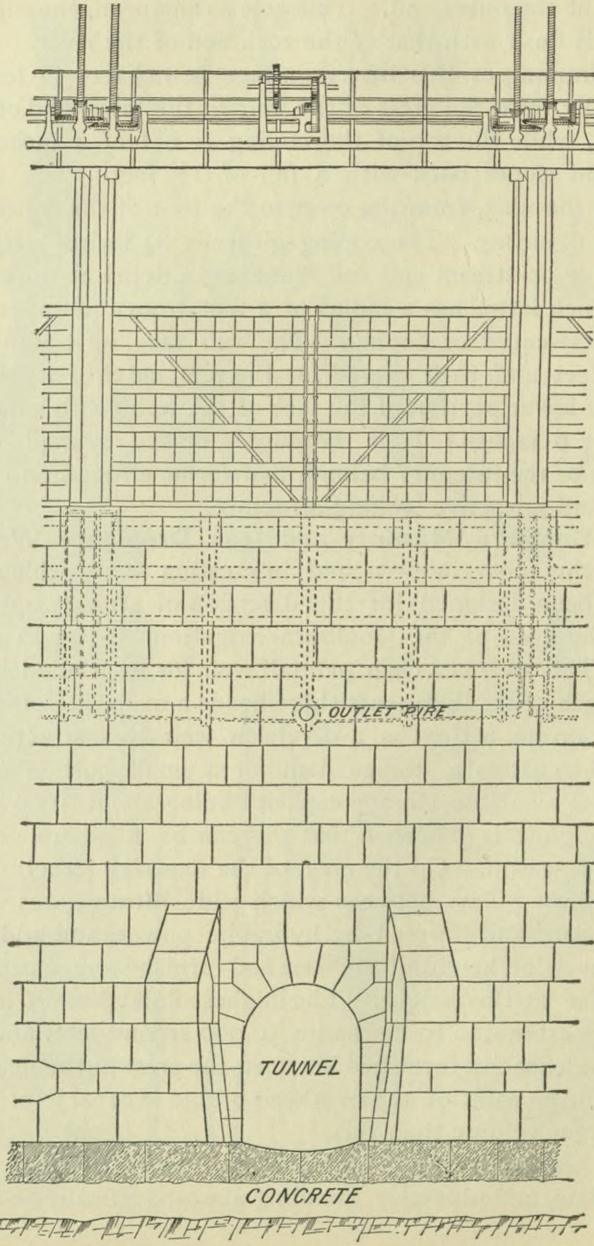


FIG. 38.—DOWN-STREAM ELEVATION, GOULBURN WEIR, AUSTRALIA.

This weir is of sufficient height to raise the summer level of the river about 45 feet, or to a total of 50 feet above the river

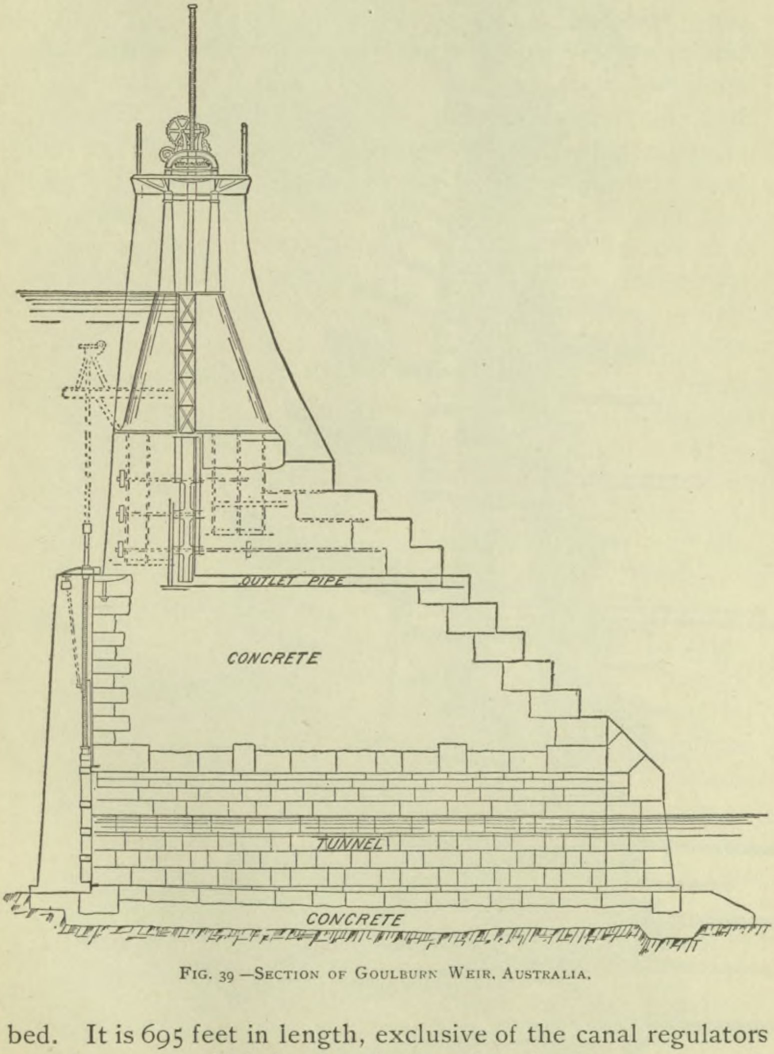


FIG. 39.—SECTION OF GOULBURN WEIR, AUSTRALIA.

bed. It is 695 feet in length, exclusive of the canal regulators at either end, which have a further length of masonry work of 230 feet. The body of the work is of combined concrete

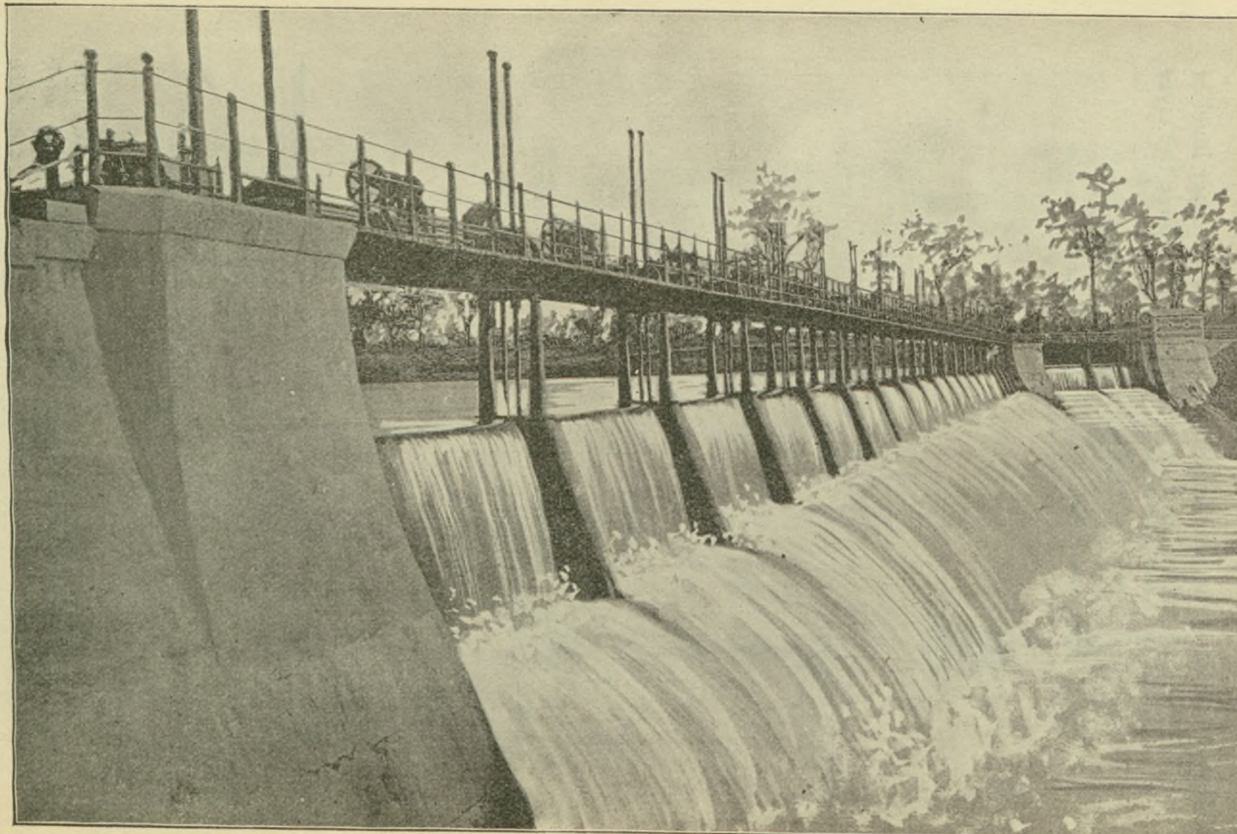


PLATE X.—VIEW OF GOULBURN WEIR, AUSTRALIA.

masonry, composed of broken stone, sharp grit, and Portland cement backed with stepped granite. In the portion of the weir across the natural waterway of the river are six temporary tunnels (Fig. 38), each with a sectional area of 44 feet, which were designed to carry the ordinary flow of the stream during the process of construction, about 3750 second-feet, and which were filled in with masonry after tne completion of the remainder of the work. The waterway in the upper portion of the weir above the masonry crest is composed of 21 floodgates (Plate X) each having a clear opening of 20 feet horizontally and 10 feet vertically. These are lowered into chambers or recesses in the body of the structure (Fig. 39), and can be so adjusted as to maintain the water-level in front of the canal off-takes at the normal full supply level. The chambers are lined with skeletons of cast-iron ribs between strong cement mortar, and the wall in front of each chamber that takes the pressure of the water is strengthened by a series of rings of wrought iron built into the concrete which are strong enough to take the entire thrust, so that should the concrete become fissured from any cause the skeleton would still take the pressure, the masonry merely acting in detail. The gates are framed with wrought-iron T beams filled with cast-iron plates, and weigh 7 tons each. They are worked by screw gearing actuated by 30½-inch Leffel turbines, which can be worked either together or separately. The available head for working them varies from 3 to 13 feet, according to the volume of water in the river, and they give from 3 to 27 horse-power. Hand gearing is provided for each gate in case of emergency.

183. 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. 40), 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

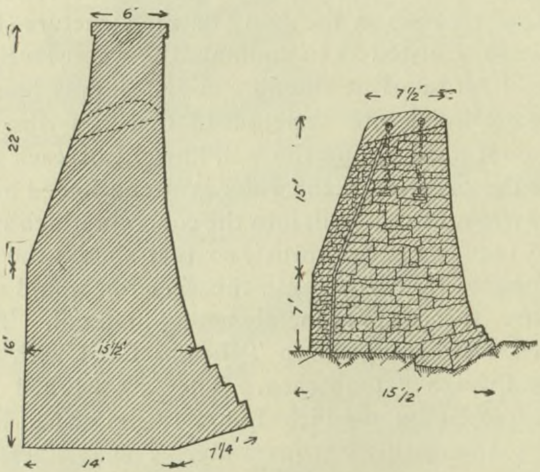


FIG. 40.—CROSS-SECTIONS OF NEWARK DAM AND WEIR.

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. 41). 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 upstream 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,

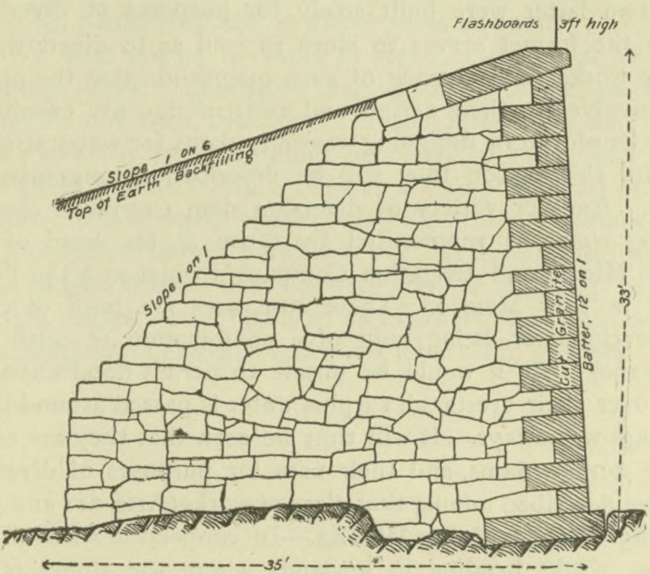


FIG. 41.—CROSS-SECTION OF LAWRENCE WEIR.

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. A recently constructed masonry weir across the Mohawk river at Little Falls, N. Y., is similar in its general design to the Lawrence weir. This weir is curved in plan to follow a favorable rock outcrop for foundation; is 16 feet in maximum height; 336 feet long on curved crest; has a downstream batter of 6 on 1 and an upstream paved slope of 1 on $1\frac{1}{2}$. Its top width is 6 feet, and width at base 23 feet.

184. **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. 311 and 312).

185. **River-training Works.**—In connection with all irrigation works heading in lowlands where the slopes of the stream-beds are slight and their banks of sand, silt, or other easily eroded material, river training or improvement works must be constructed in order to maintain the stream in the channel which will cause it to do the least injury to the diversion-works. During periods of high flood such rivers erode their banks and may change their channels to such extent as to leave the head or diversion works high and dry or to undermine and destroy them, unless the river is so trained as to secure for it a permanent channel for some distance above and below the diversion works.

Such training-works have been extensively constructed at the heads of many of the great Indian canals which head in lowlands or deltas. The most extensive examples of these are to be found at the heads of the Lower Ganges and Agra canals, and an interesting example of the necessity of training-works is found at the site of the siphon which carries the Cavour canal in Italy under the river Sesia. Here, in order to prevent the destruction of the siphon, it was found necessary to guide the river by training-works and make its channel more permanent and its slopes more uniform.

We need not go to India or Italy for examples of river-training works. The most magnificent of these are to be found in our own country. On the Mississippi and many minor rivers it has been found necessary to guide and maintain their channels with a view to the improvement of navigation. These works consist, according to circumstances, of walls or embankments parallel to the channels of the streams so as to give them a uniform waterway, or of jetties run out at right angles to the banks so as to direct their flow, and of combinations of embankments and jetties and groynes.

CHAPTER IX.

SCOURING SLUICES, REGULATORS, AND ESCAPES.

168. 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 scouring sluices 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

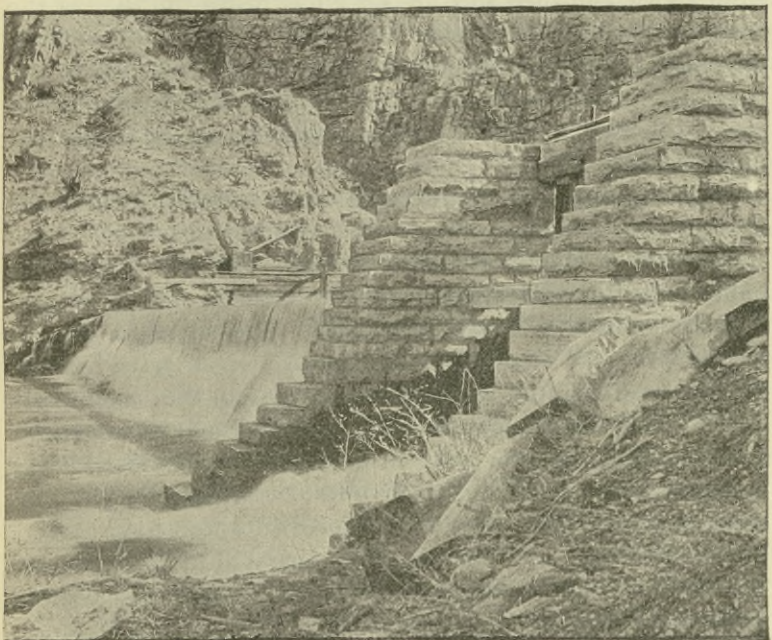


FIG. 42.—VIEW OF HIGHLINE CANAL WEIR.

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. 42 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.

187. 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. 23) 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 $2\frac{1}{2}$ 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.

188. Falling 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. 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.

189. Bear-trap Movable Sluice Gates.—One of the most satisfactory rapidly operated movable sluice gates or shutters is the American type of bear-trap gate, and the more recent forms of its development which have been employed on river improvement work, notably on the Great Kanawha river, by the Engineer Corps of the Army. This form of gate is especially satisfactory from the fact that its operation is almost automatic. When raised it forms an obstruction or weir across the entire width of the channel, or across a part of it, as a scouring sluice, and when lowered it is either lifted above the water surface or dropped against the bed of the stream so as to offer no obstruction to its free flow. About the only example of the adoption of this general principle at the head of irrigation-works is to be found in India, where the weirs at the head of the Arrah canal, a part of the Soane system, are constructed on the Chanoine modification of the original bear-trap system. This form of weir shutter has also been used on river-im-

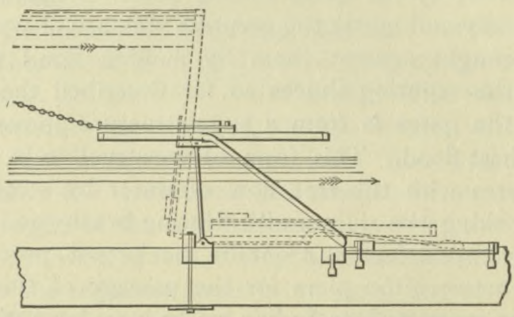


FIG. 43.—CHANOINE MOVABLE SHUTTER, RAISED, LOWERED, AND CLOSED.

provement works in this country, and is illustrated in Fig. 43, which is self-explanatory.

The essential features of the old bear-trap gate are two leaves built across the sluiceway and fastened by horizontal hinges at the bottom. When the sluiceway is open the leaves lie in a horizontal position, the up-stream leaf overlapping the other for a portion of its length. When the sluiceway is closed the two leaves form a triangle, of which the bottom of the sluiceway is one side and the leaves the other two, the apices being at the two hinges and where the leaves abut against each other. The space within the triangle is a chamber which may be filled by inlet pipes closed by means of valves under the control of the operator of the gate. To raise the gate the outlet from the chamber below is closed and the inlet opened, when the water fills the chamber and presses the lower surface of the leaves. As the water has also access to the upper surface of the upper leaf, the pressure from below upon it is neutralized; on the lower leaf there is no counter pressure, and therefore the pressure from below tends to raise it, and also the upper leaf which rests upon it. This is done in a properly proportioned structure, and the gate is carried to a height limited by the dimensions of the leaves. To lower the gates the water from above is cut off, the outlet valves are opened, and the chamber emptied. The pressure of the water is thus thrown on the upper leaf, and forces it back into a horizontal position against the bed of the sluiceway.

This form of gate has been found difficult of operation, owing chiefly to the difficulties of properly proportioning the angles and lengths of the leaves. Important developments have been made in the general design of this gate by Carro, Parker, Lang, and lastly Scott. These improvements have resulted in the addition of a leaf, or more correctly, in practically dividing the upper leaf into two parts or joints (Fig. 44), so that as the sluice-gate is lowered the upper leaf

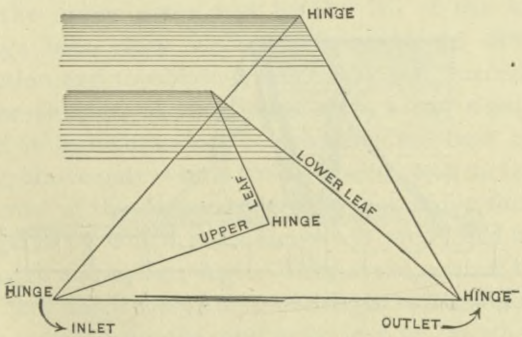


FIG. 44.—BEAR-TRAP GATE, PARKER MODIFICATION.

which is jointed near its centre folds inward, that is, into the chamber, while the upper and lower leaves are hinged together at the top. This is practically the Parker gate, which thus has four hinges—one at the foot of each gate, one at the apex, and one in the upper leaf. This gate has eliminated nearly all the difficulties of the old bear-trap, as there is no overlapping at the apex, while the height obtainable for the same length of sluice is over twice as great; there is no sliding friction, and the gate cannot be brought to a sudden stop when it approaches its full height, but comes to a rest gently. The conditions which give the most satisfactory length of leaf for the Parker gate are, according to Lieut. H. M. Chittenden, U. S. A., lower leaf multiplied by lower section of upper leaf minus upper section of upper leaf equals the base, and that when raised to full height they shall not rise above a curve corresponding to the particular condition of back-water. The proportions as shown in Fig. 44, however, practically fill all conditions essential to successful operation.

190. 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.

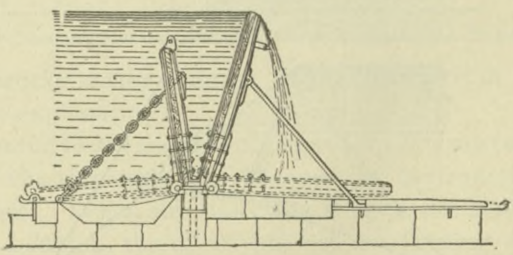


FIG. 45.—CROSS-SECTION OF MAHANUDDY AUTOMATIC SHUTTERS, INDIA.

45). 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.

191. Soane Falling 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. XI.). A low masonry wall 12 inches high has been built up on the down-stream 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 pressure, 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.

192. 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

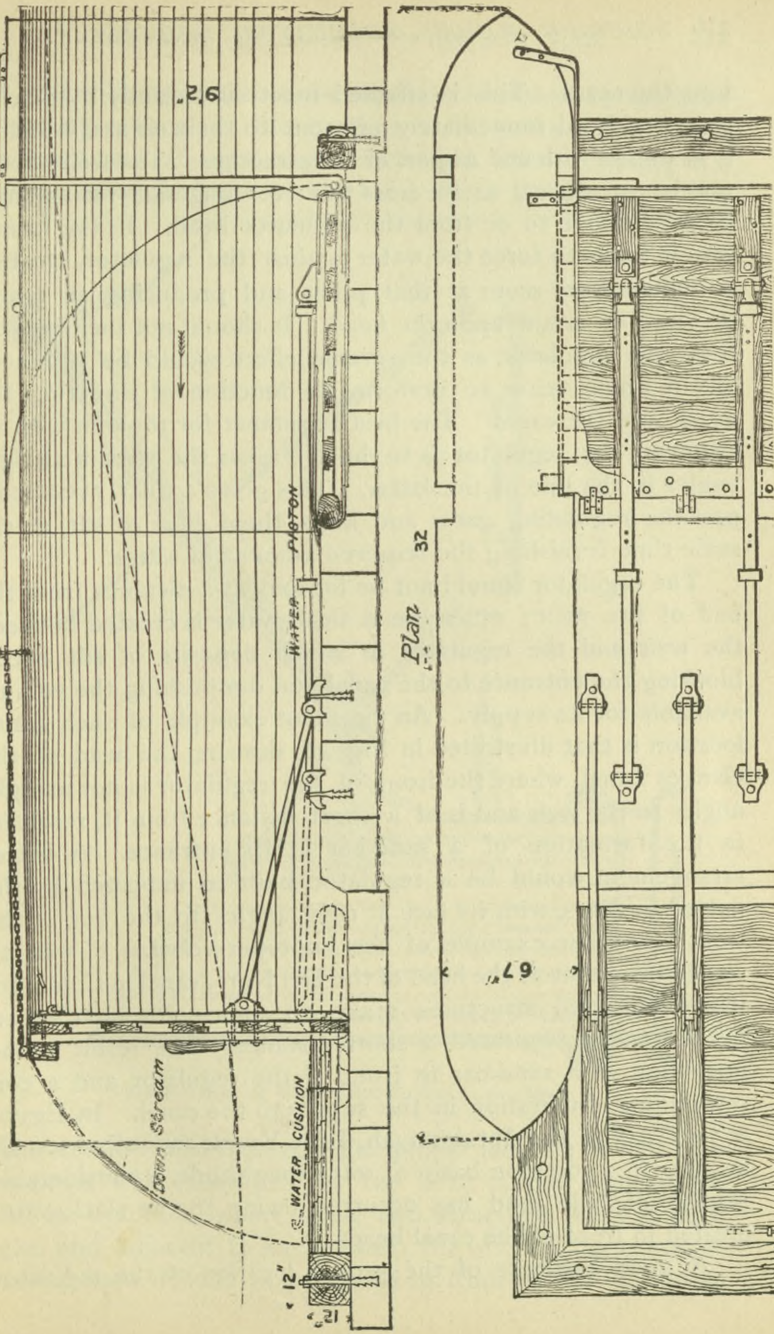


PLATE XI.—FALLING SLUICE GATE. SOANE CANAL, INDIA.

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 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 latter. 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. 46, 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 Ibrahimiyah, 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

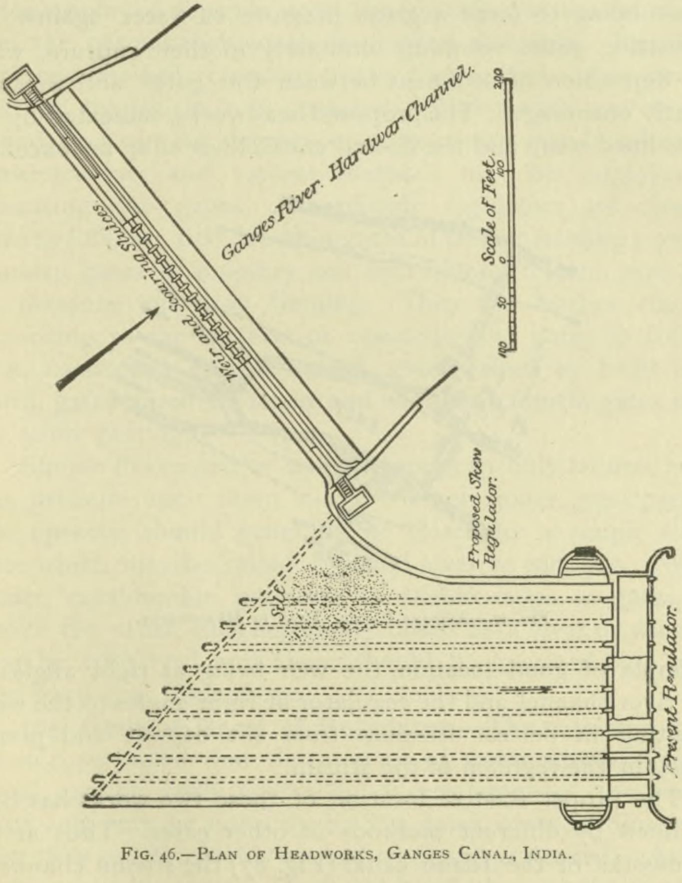


FIG. 46.—PLAN OF HEADWORKS, GANGES CANAL, INDIA.

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. 47 shows the plan of the old and new Arizona headworks. The present 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 rupture, while the deposition of sediment between the gates and weir was greatly encouraged. The proposed headworks, indicated by the cross-lined weirs and the dotted canal, lines offer an excellent

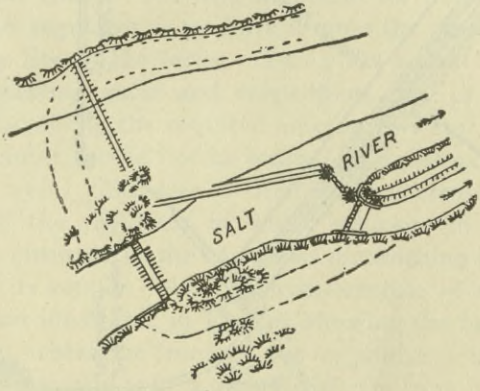


FIG. 47.—ARIZONA CANAL. PLAN OF HEADWORKS.

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. 87) 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 an under sluice.

193. **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.

194. **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.

195. 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 di rectly 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.

196. 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. 24) 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.

197. 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. 50, 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.

198. 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

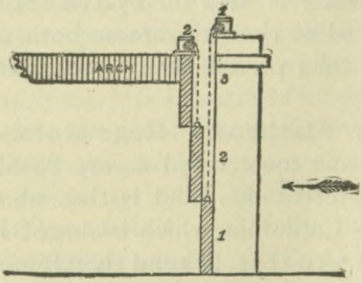


FIG. 48.—REGULATOR GATES, GANGES CANAL.

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 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.

199. 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. 49, the gate is constructed

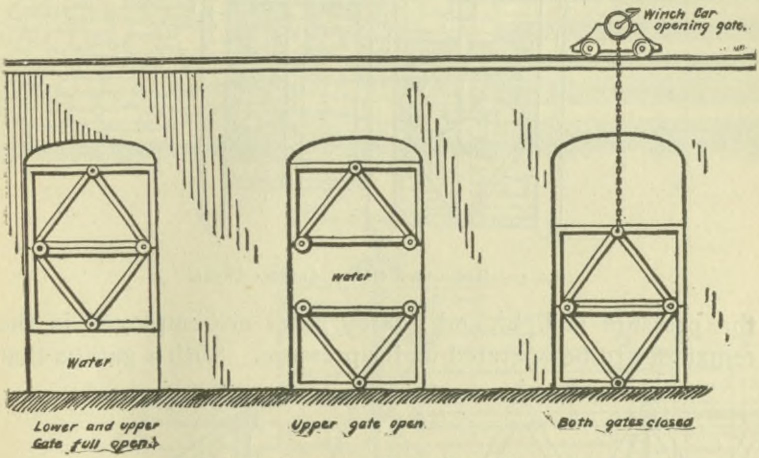


FIG. 49.—REGULATOR GATES, SOANE CANAL.

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.

200. 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 simple lifting gates are generally inserted in a few of the openings, to be used when

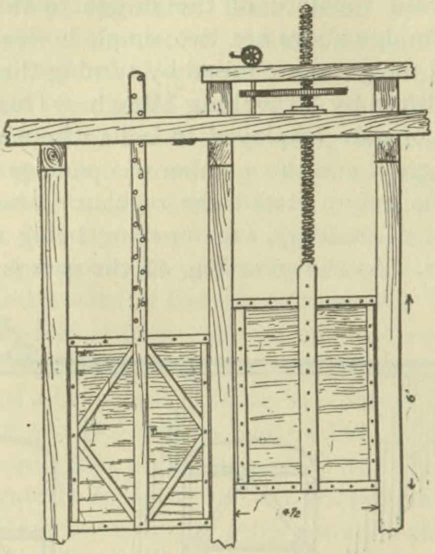


FIG. 50.—REGULATOR GATES, ARIZONA CANAL.

the pressure is light, and geared gates are employed in the remainder to be operated under pressure. Such a gate is that

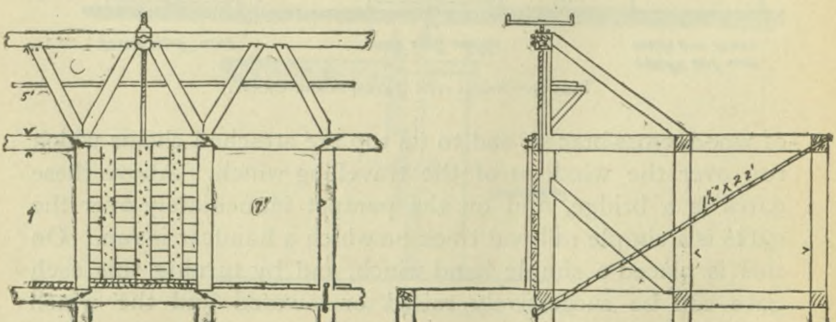


FIG. 51.—REGULATOR GATES, DEL NORTE CANAL.

at the head of the Arizona canal (Fig. 50), which is constructed of wood framed with iron. Above it projects a heavy steel

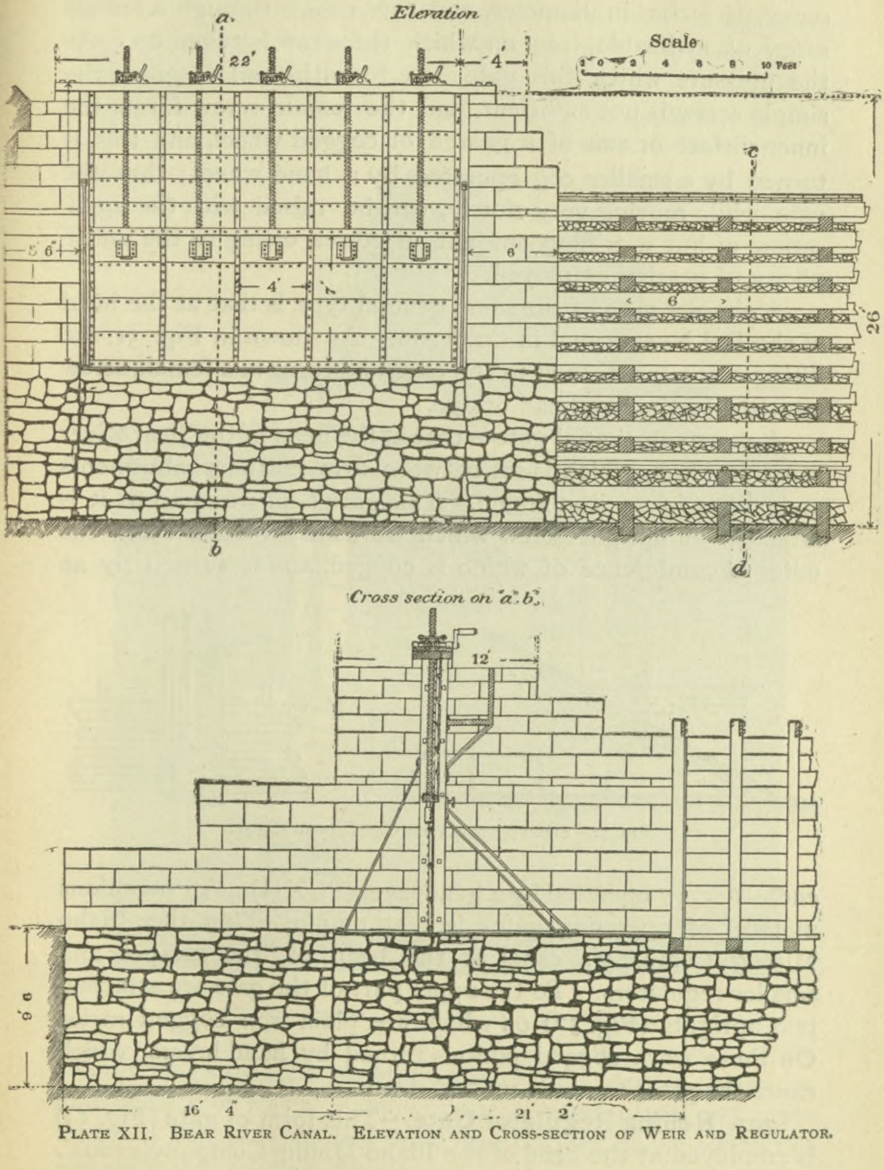


PLATE XII. BEAR RIVER CANAL. ELEVATION AND CROSS-SECTION OF WEIR AND REGULATOR.

screw, 1½ inches in diameter, and this passes through a female screw of malleable iron on which the wear is taken up. As the pressure which this gate has to withstand is great, the simple screw is not sufficient, and the female screw forms the inner surface or axis of a geared or cogged wheel, and this is 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. 51, 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

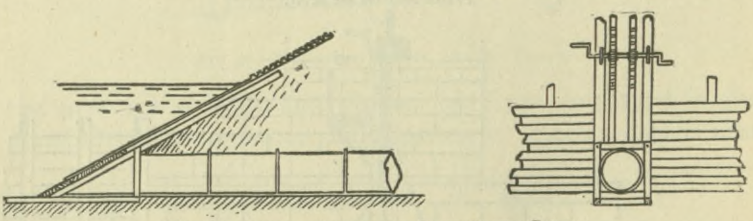


FIG. 52.—SLIDING REGULATOR GATE, IDAHO CANAL.

endless screw operated by a hand lever (Pl. XII). 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. 52) 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.

201. Rolling Regulator Gate.—This form of gate (Fig. 53) 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. 27). 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

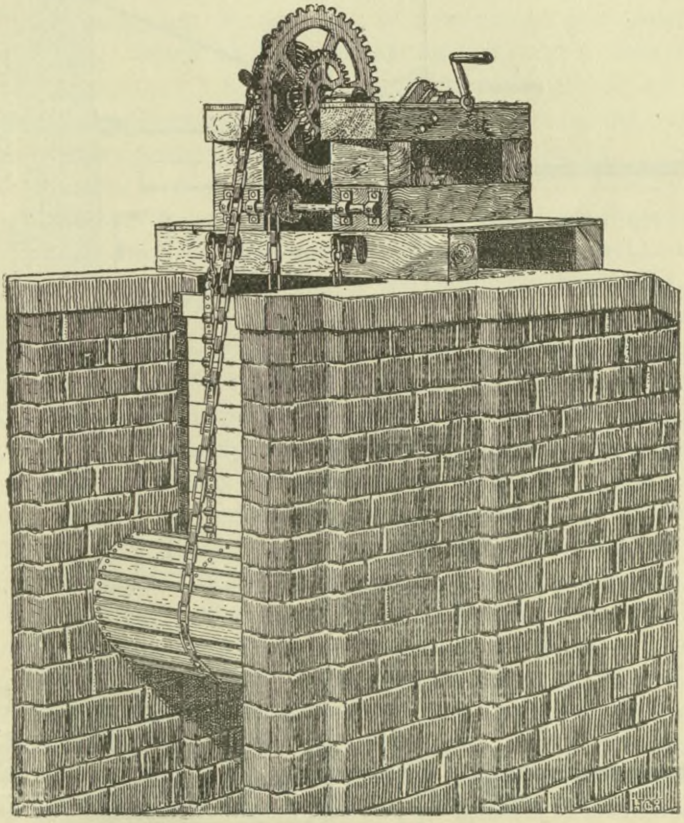


FIG. 53.—ROLLING REGULATOR GATE, IDAHO CANAL.

the openings are separated by masonry piers 3 feet in thickness, 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

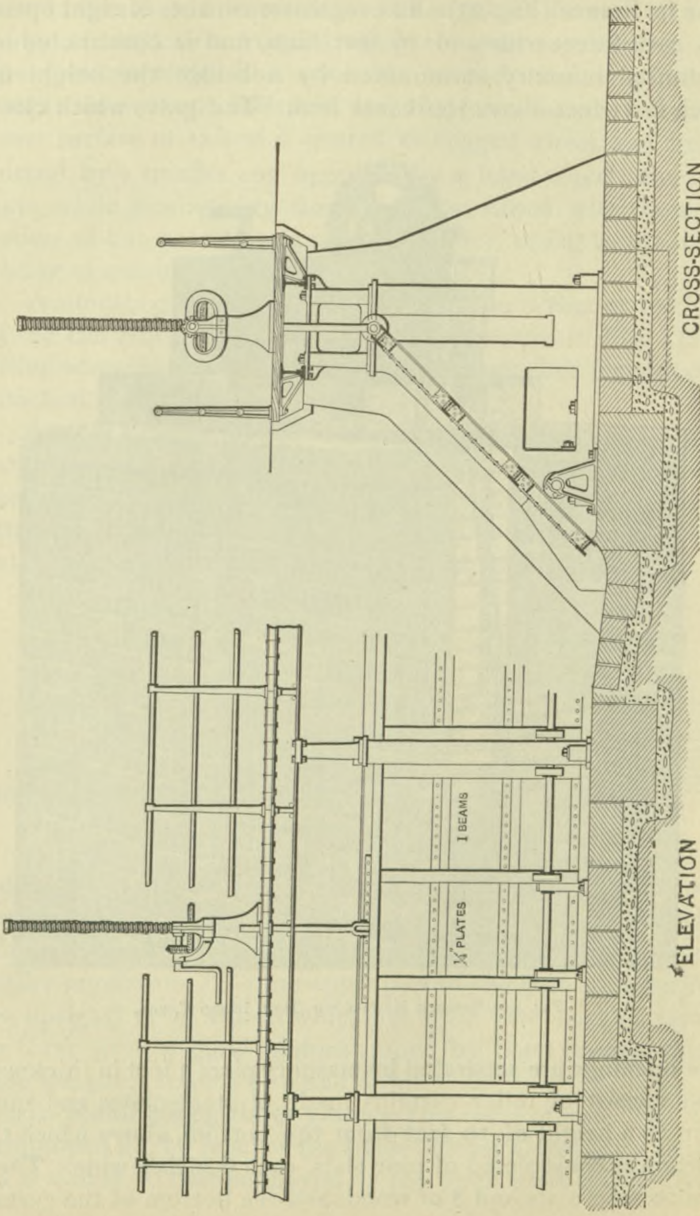


FIG. 54.—INCLINED, FALLING REGULATOR GATES, GOULBURN CANAL, AUSTRALIA.

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.

202. Inclined, Horizontally Pivoted Falling Gates.—The regulator heads to some of the branch canals on the Goulburn Irrigation System are of interesting and peculiar construction. Each of these heads consists of a series of fourteen gates each 7 feet high by 10 feet wide, placed across the channel and arranged to admit of the surface of the water at the offtakes being maintained at any desired height. They are of wrought iron, fitted with rollers at the head, worked in vertical recesses in cast-iron piers and on roller bearings carried on horizontal shafts, supported on pedestals secured to the stonework in the masonry bed of the regulator head and to the piers (Fig. 54). These roller bearings are a little below the lower third of the gates. The motion of the gates is peculiar, being vertical at the head, forward in raising and backward in lowering on the roller bearings so that the pressure on them may be nearly balanced in all positions and so that a minimum power may be required to work them. Each gate is manipulated by separate gearing, consisting of a screw shaft with eye and pin connection to the gate head, worked by a bevel-wheel gearing into a pinion operated by windlass from the bridge above. One of the most desirable results of this form of regulator head is in the fact that the water is drawn off the surface of the canal, and thus is freer of silt than if it were taken from the bottom. Each offtake consists of a series of notches with knife-edge measuring weirs, each 6 feet wide.

203. Hydraulic Lifting Gate.—At the head of the Folsom canal in California the regulating gates (Pl. XXXVII) 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. XIV) 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

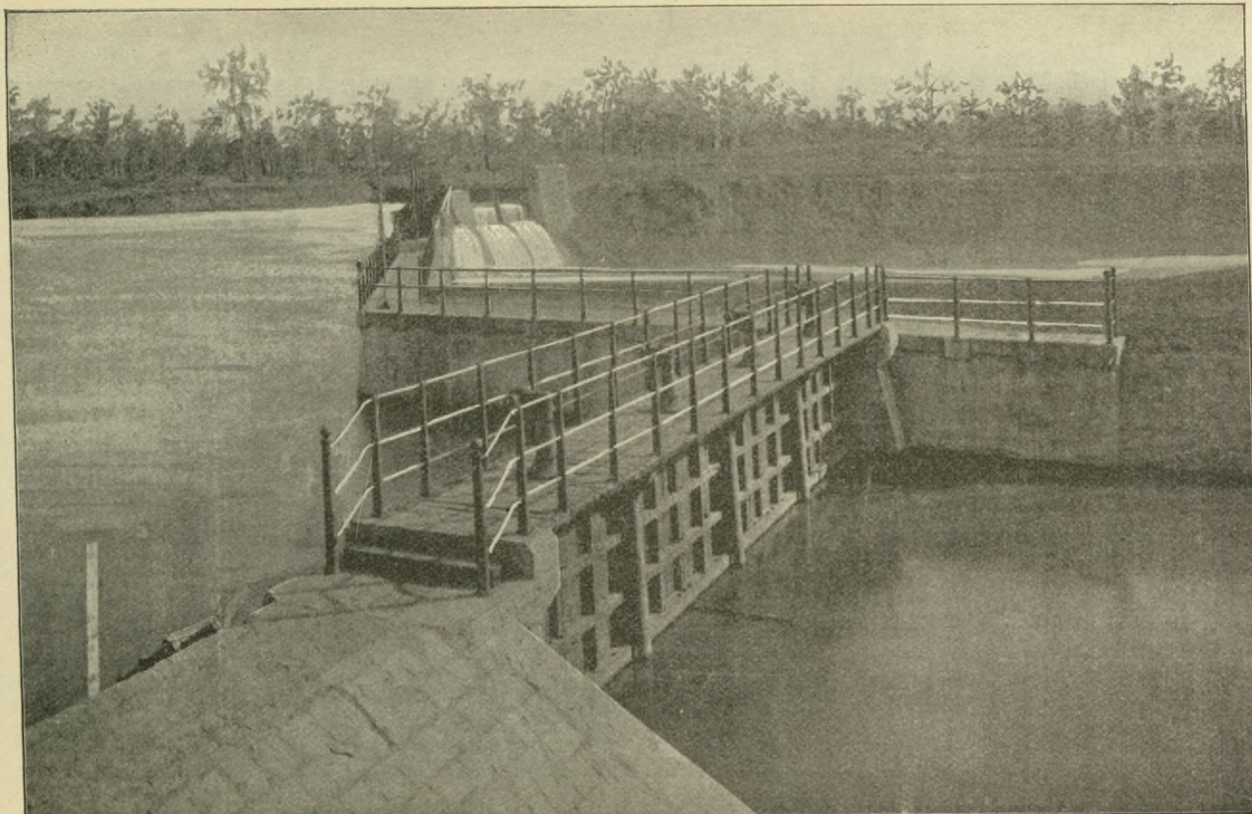


PLATE XIII.—VIEW OF REGULATOR AND WEIR, GOULBURN CANAL, AUSTRALIA.

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.

204. **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 drainageway 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

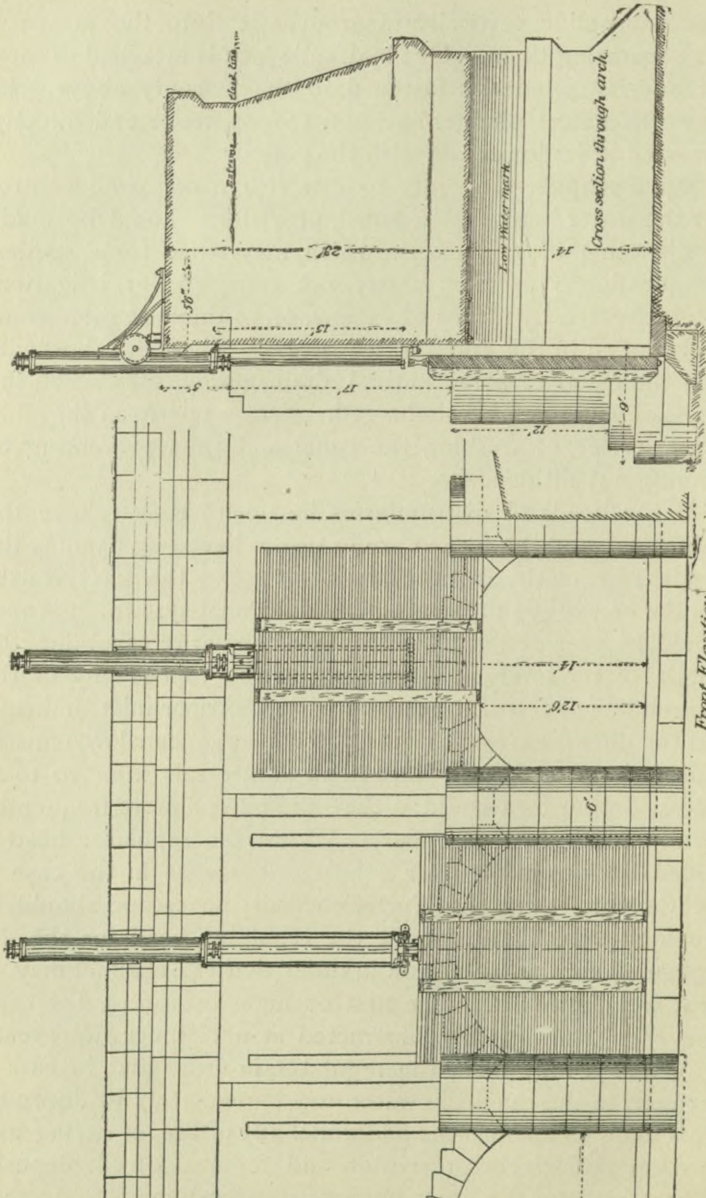


PLATE XIV.—CROSS-SECTION AND ELEVATION OF REGULATOR GATES, FOLSOM CANAL.

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.

205. 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 escapeway 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.

206. 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. 24). 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. 52). 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

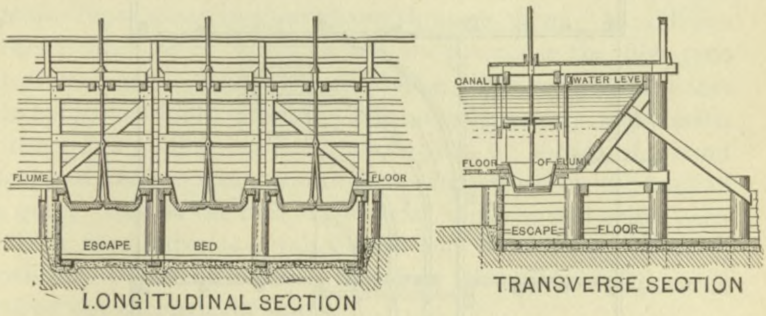


FIG. 55.—ESCAPE-FLUME ON GOULBURN CANAL, AUSTRALIA.

gate is a receiving box which discharges up and down stream into two inclined wooden flumes which lead the water into the creek.

Escapes are provided on the Goulburn canal in Australia by building them in the floors of the flumes which cross occasional drainage lines. The first of these is in the seventh mile, and another at a point a little farther on, where the canal leaves the neighborhood of the river bank. These escapes consist of three openings in the floor of the flume, fitted with valves opening downward and worked from a gangway above by screw and lever gearing (Fig. 55). These escapes serve the purpose of regulating the supply by disposing of surplus water, and also act as sand gates.

Provision has also been made with a view to disposing of extreme flood waters which might enter the canal from the river by the construction of several escapes within the first seven miles of the canal, at points where it approaches near to the river bank and is in firm ground. These escapes are simple outlets consisting of masonry sills laid in the ground at a height of 3 inches above full supply level of the canal, the entire space over the sills being left open as a free escape, and the aggregate length of these flood escapes, measured along the sills, is 265 feet.

207. Sand Gates.—Sand gates are practically escape gates,

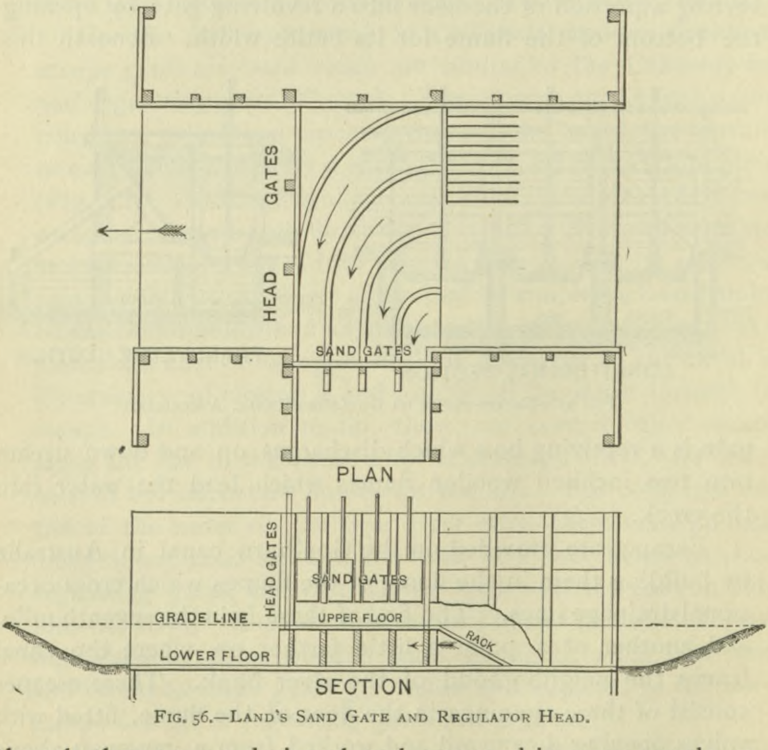


Fig. 56.—Land's Sand Gate and Regulator Head.

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.

Considerable attention has been paid in more recent irrigation practice in the West to the insertion of sand gates near the canal heads, as the evil effects have become appreciated of permitting heavy matter in suspension to enter and become deposited in canals (Chap. IV). One form of sand gate which has been found to work satisfactorily, and which is called Land's Sand Gate after its inventor, consists of a flume built across the canal, and containing both the regulating gates and the sand gates, the latter being placed on the lower side of the canal above the regulating gates. Above the regulating gates the flume is given a double flooring, the lower of which is about 2 feet below the level of the grade-line of the canal, while the upper is on a level with the bottom of the regulating gates and the grade-line. In the lower floor are placed curved guiding-racks (Fig. 56) which retard the velocity of the stream, causing the deposition of sediment in front of and below the regulating gates and direct the discharge toward the sand gates so that when the latter are opened the deposited matter can be readily flushed out.

An excellent though expensive form of sand gate is that introduced in the Santa Ana canal. The main feature of this is the sand box or enlargement of the canal in which the sediment is caused to be deposited. These sand boxes are placed on the line of the flume, and into them the water is

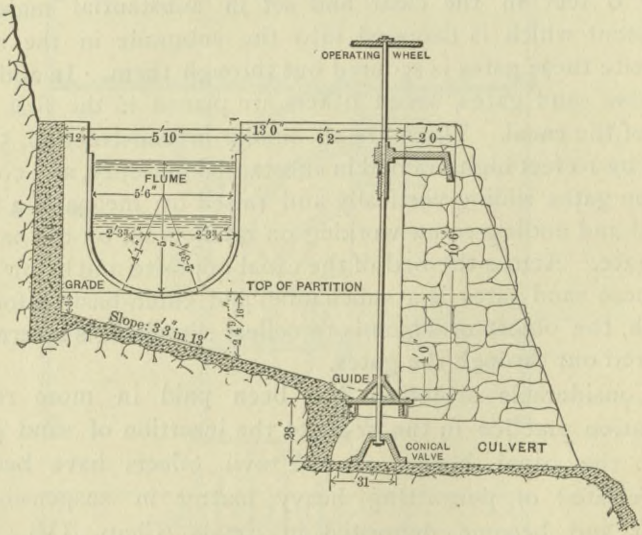


FIG. 57.—CROSS-SECTION OF SAND BOX, SANTA ANA CANAL.

discharged, and because of their increased cross-section the velocity of the water is checked and the sediment deposited. The first of these, located 700 feet from the headworks, is excavated in rock, and is so constructed as to form a solid rock and masonry chamber. Within the wall line it is 60 feet long by 13 feet wide, and its floor slopes transversely (Fig. 57) so that its depth on the upper side is 7 feet 3 inches, while on the lower side it is 10 feet 6 inches. This sloping floor is broken transversely into longitudinal profiles by three partitions, the level tops of which are 6 feet 6 inches below the surface level and are 15 feet apart from centre to centre. These partitions are 6 inches wide on top, and their sides fall each way with a slope of 2 on 3. They are so arranged as to divide the bottom of the chamber into four compartments, the lowest portions of which are 30 inches square each, and are spaced 15 feet apart between centres, and lie next the base of the outer wall. To each of these sumps the bottom of the chamber slopes, and in each is located a sand valve, which opens down into a culvert leading to an escape-way. For the full capacity of the canal the water section of the sand box is 65 square feet while that of the flume is 24.7 square feet, and the velocity of the current is reduced from 9.7 to 3.7 feet per second. The sand valve employed is a truncated hollow cast-iron cone with a projecting flange on its base, this latter being pointed down, while the truncated end is drawn up by a vertical rod into a short cast-iron cylinder 18 inches in diameter, which forms the sluiceway on which the valve falls and closes.

CHAPTER XII.

FALLS AND DRAINAGE WORKS.

208. 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.

209. 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 171) with the object of diminishing the velocity and consequent erosive action of the water; 3, the water may plunge into a water-cushion (Article 172). 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.

210. 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 J. H. 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

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

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.

211. 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.

212. 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 $\frac{1}{2}$ foot broad by $\frac{3}{4}$ of a foot deep; upper end $\frac{1}{4}$ foot broad by $\frac{3}{4}$ of a foot. 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.

213. Notched Fall Crest.—Great advantages have been found in India in adopting a notched form of canal fall, which practice has shown overcomes almost wholly the difficulties of excessive velocity and erosive action below the fall. The breast-wall or crest of the fall is cut away into a number of notches, the bases of which are at a level with the canal bed, the crest of the breast-wall being above full-supply level (Plate XV). At the foot of each notch is a lip projecting beyond the outer face of the breast-wall, which has an influence in retarding the stream and determining the form of the lower face of the falling water. The notches are all so designed as to discharge at any given level the same amount of water as the canal above carries at that level, so that there is approximately no increase in velocity in the canal as the water approaches the fall, while a uniform depth and flow are maintained. The water flows from the notches in a fanlike shape, and meets the water surface below in a steady stream, which contrasts favorably with the violent ebullitions which accompany clear overflows. The action on the banks of canals below the falls is very slight, and permits of the wings being of but moderate length. Mr. R. B. Buckley states there is no

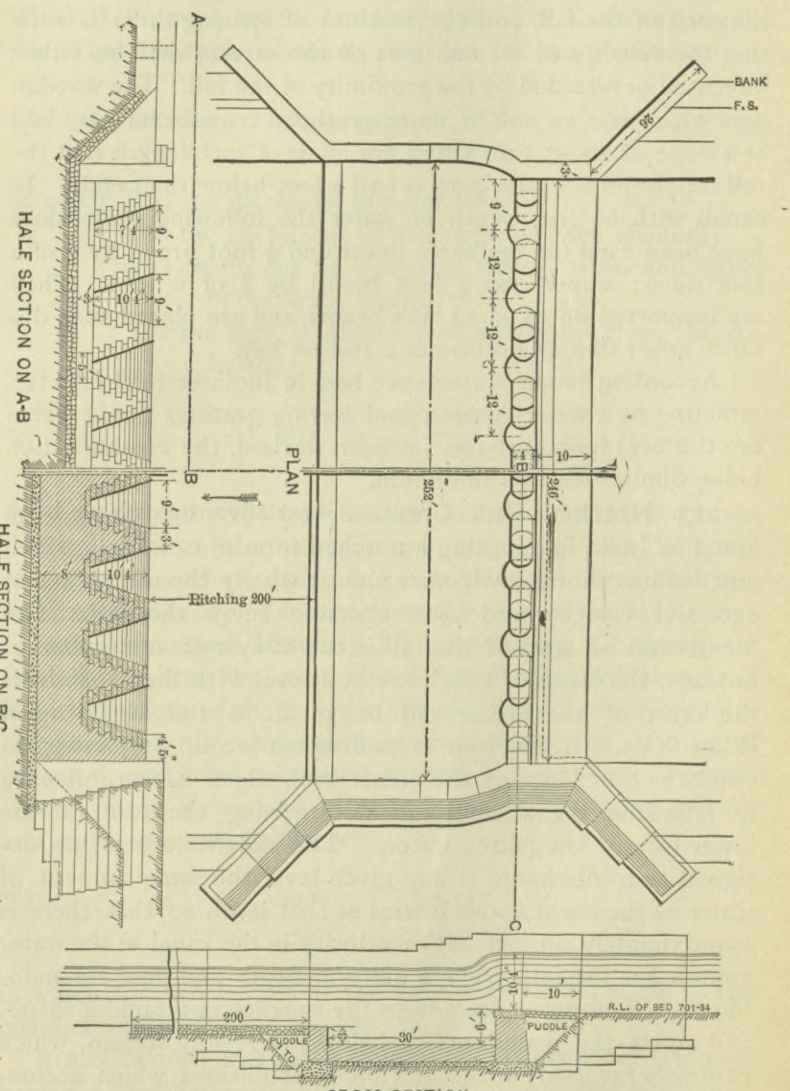


PLATE XV.—NOTCH FALL, CHENAB CANAL, INDIA.

question of the superiority of this over all other forms of falls where it can be adopted. The form of the basin below the fall is practically that of a shallow water-cushion, it being widened in addition to check the ebullition of the water and reduce it to a steady forward velocity. One objection to this form of fall is the depth of foundation required, which adds considerably to the difficulty of construction except where the soil is very firm. In many cases it may prove cheaper to protect the lower level by mere strength of additional material than by the construction of a deep water-cushion.

214. 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

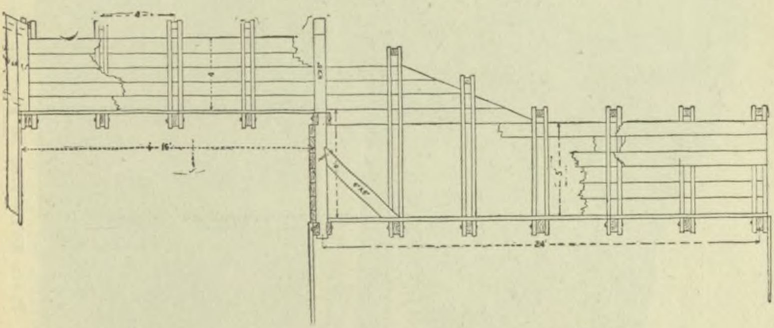


FIG. 58.—LONGITUDINAL SECTION OF FALL, ARIZONA CANAL.

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 XVI) 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 18 to 21 feet in length on the crest. They consist (Fig. 58) of wooden fluming, the flooring of which is 12 feet in length

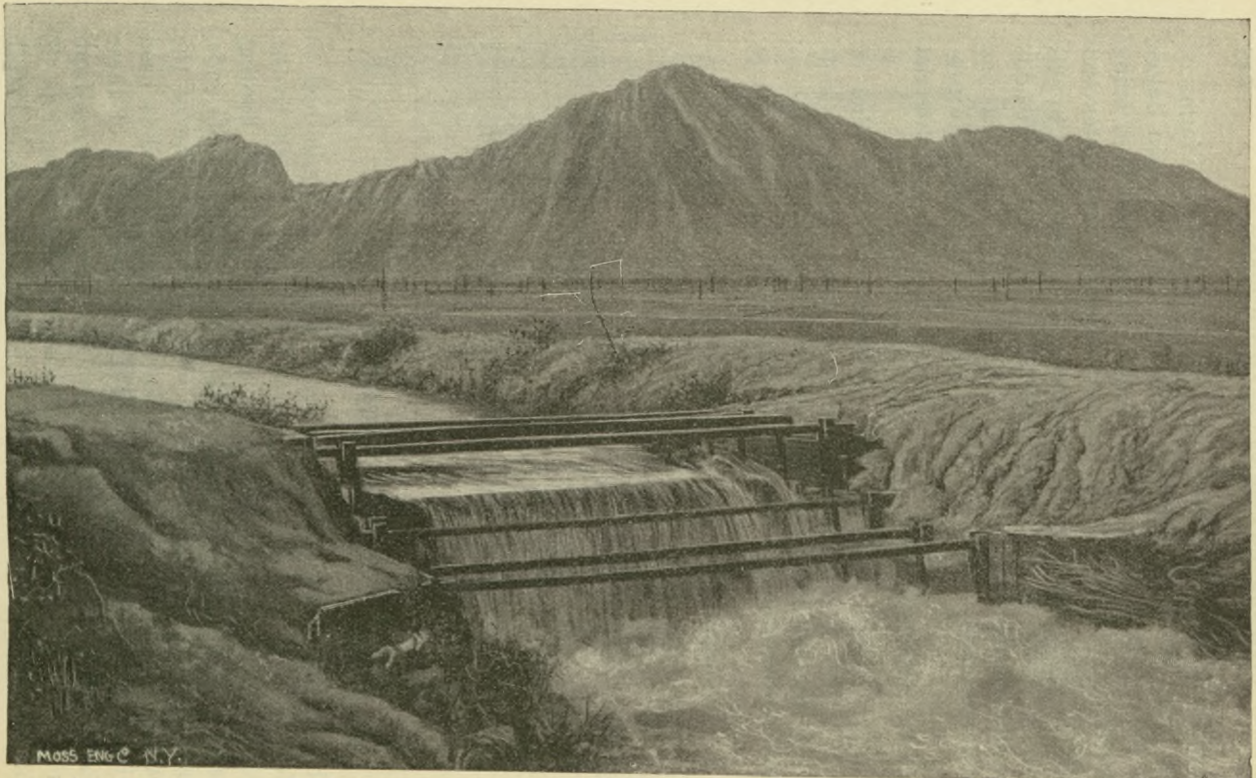


Plate XVI.—View of Fall on Arizona Canal.

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

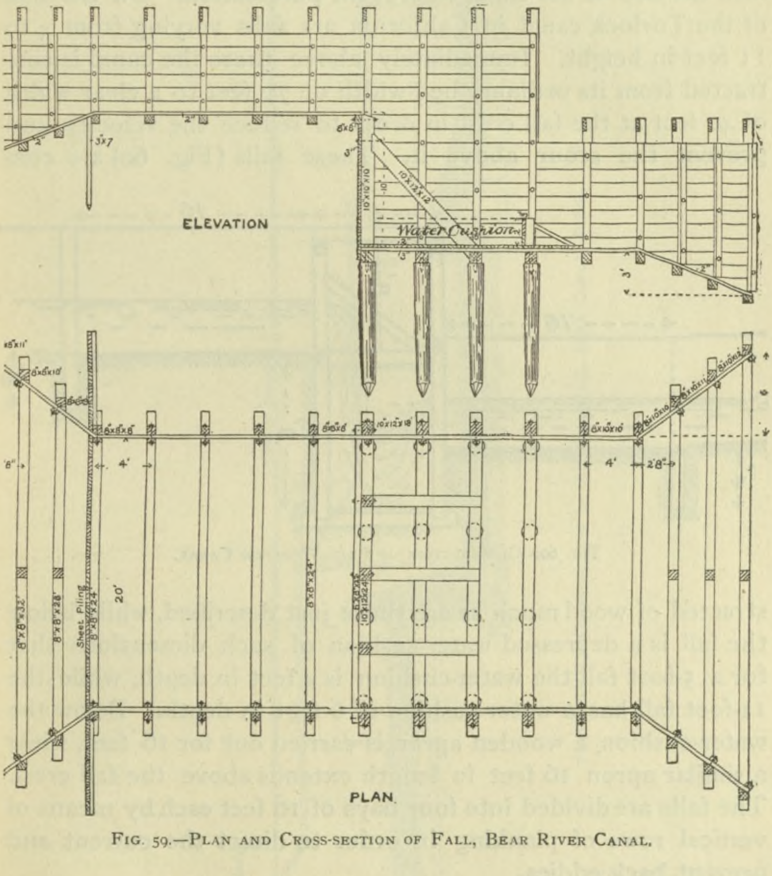


FIG. 59.—PLAN AND CROSS-SECTION OF FALL, BEAR RIVER CANAL.

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.

215. 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. 59). 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. 60) are con-

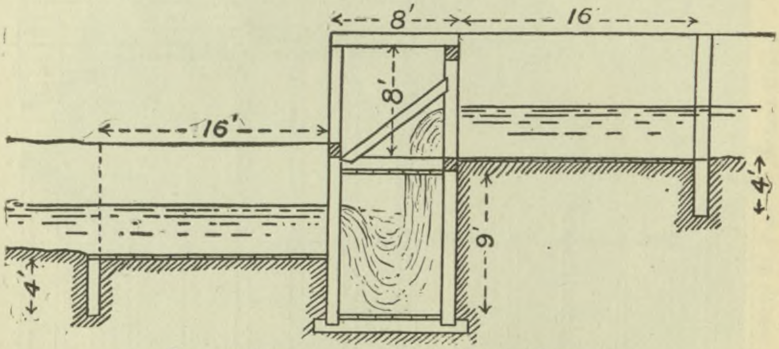


FIG. 60.—CROSS-SECTION OF FALL, TURLOCK CANAL.

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.

216. 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 customary, however, in the case of wide canals to divide the falls into bays of 10 feet each, or thereabouts,

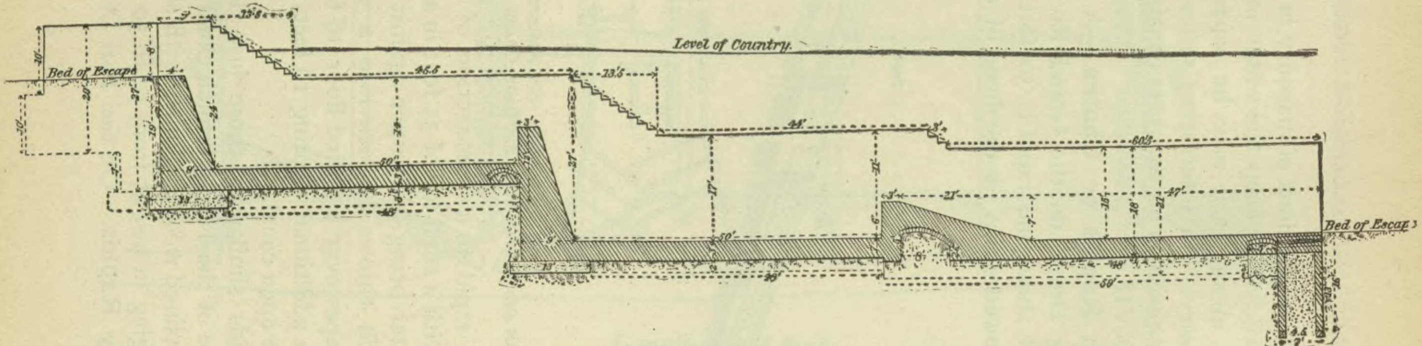


PLATE XVII.—CROSS-SECTION OF KUSHUK FALL, AGRA CANAL, INDIA.

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 XVII.

217. **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

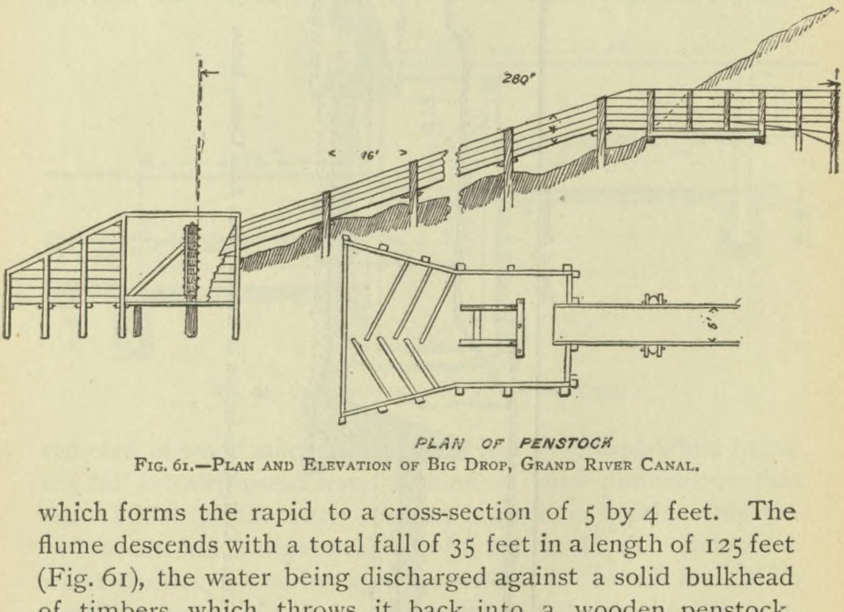


FIG. 61.—PLAN AND ELEVATION OF BIG DROP, GRAND RIVER CANAL.

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. 61), 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.

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.

218. **Masonry Rapids.**—On the Bari Doab canal in India

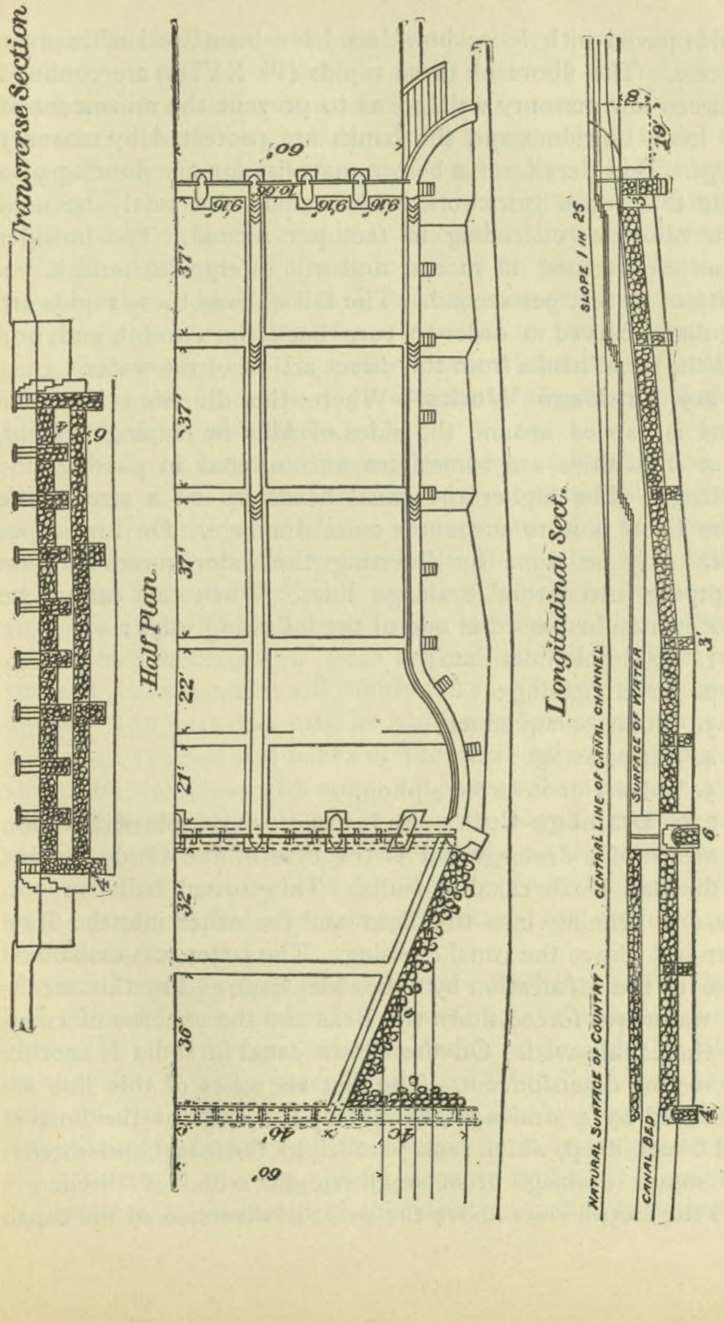


PLATE XVIII.—PLAN OF RAPIDS, BARI DOAB CANAL, INDIA.

rapids paved with loose bowlders have been used with great success. The floors of these rapids (Pl. XVIII) 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 curved in order to turn back the current and protect the canal banks from the direct action of the water.

219. 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.

220. 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.

221. 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.

222. 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 221, 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

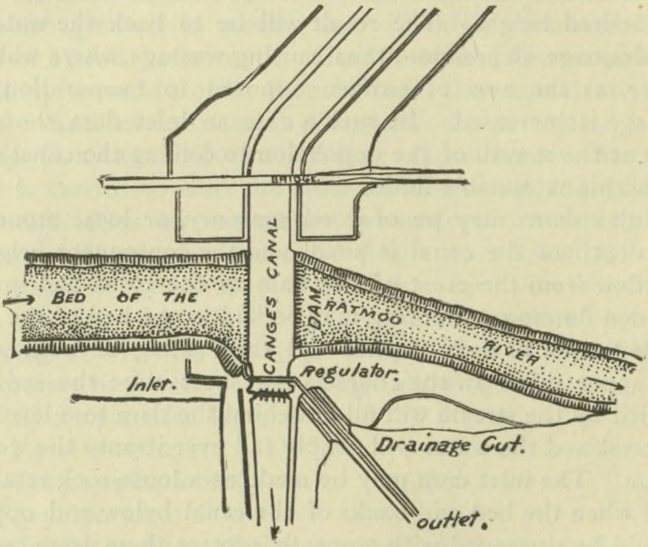


FIG. 62.—PLAN OF RUTMOO CROSSING, GANGES CANAL, INDIA.

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. 62). 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 flash-boards fitting into grooves.

223. 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 water-way 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.

A satisfactory method of connecting flumes with canal banks is that employed in the San Luis canal in Colorado, which consists in building a vertical drop in the flume at its junction with the earth at either end. These drops are let down from 2 to 4 feet, and are then filled up to the level of the canal and flume-gate with earth. At either end of the drop against the earth embankment and at the end of the flume proper is placed sheet piling, while the earth is carefully tamped back of and about the drop. Another plan employed with satisfaction on the Bessemer canal, but a little more expensive of construction, is to build out at the end of the flume a couple of parallel rows of sheet piling at right angles to the end of the flume and the direction of the canal, and another row meeting this at the junction with the flume is run out at 45°, thus enclosing an angle in the canal bank between two rows of sheet piling filled in with earth. Another way of making the connection satisfactorily, and used in conjunction with some of the methods above described, is that of putting in an inclined drop or apron running into the canal bank, with sheet piling at either end. Be the means employed what they may, the greatest care must always be exercised in making the connection of flume ends with earth.

224. 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.

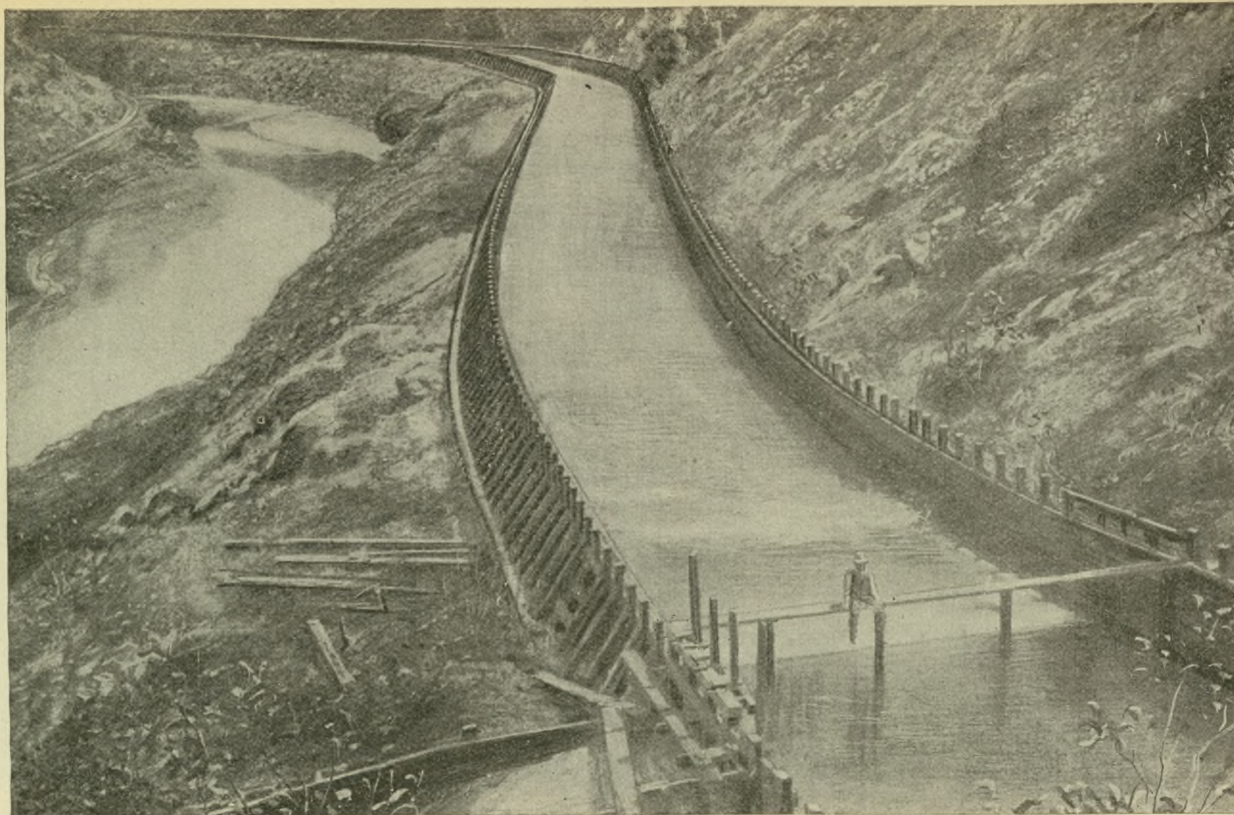


PLATE XIX.—HIGHLINE CANAL, COLORADO. VIEW OF BENCH FLUME. ESCAPE IN FOREGROUND

XIX), the great San Diego flume in California. The former is a little over half a mile in length. 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 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. The Santa Ana flume was built both to give a permanent form of water-way and prevent losses by absorption; and also because the rocky canyon wall on which it is aligned is in places too steep to permit of excavation or other form of channel within reasonable limits of expense.

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.

225. 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 standards 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 excellent example of bench flume is that of the San Diego Flume Company (Fig. 63), which is 6 feet wide in the clear and 4 feet high; the bottom and sides are planked with 2-inch

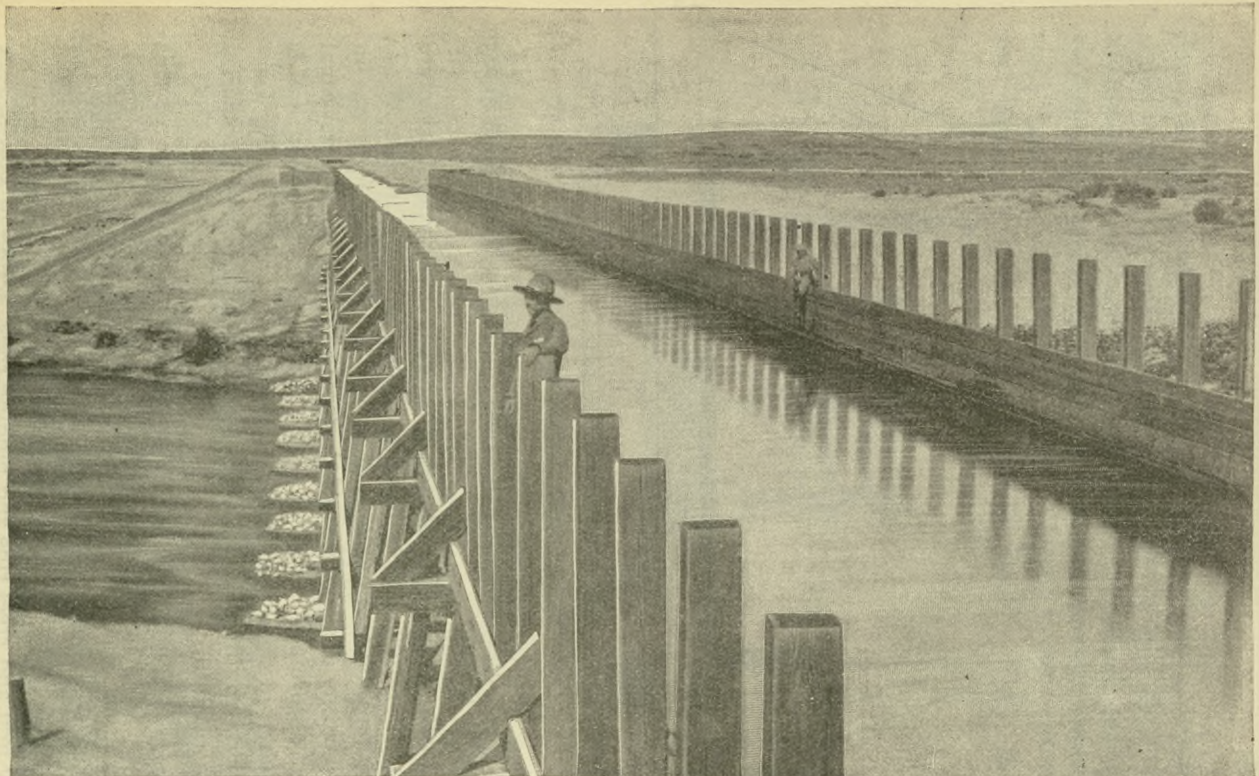


Plate XX.—View of Pecos Flume.

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 the flume, consisting of 4 by 4 scantling placed at intervals

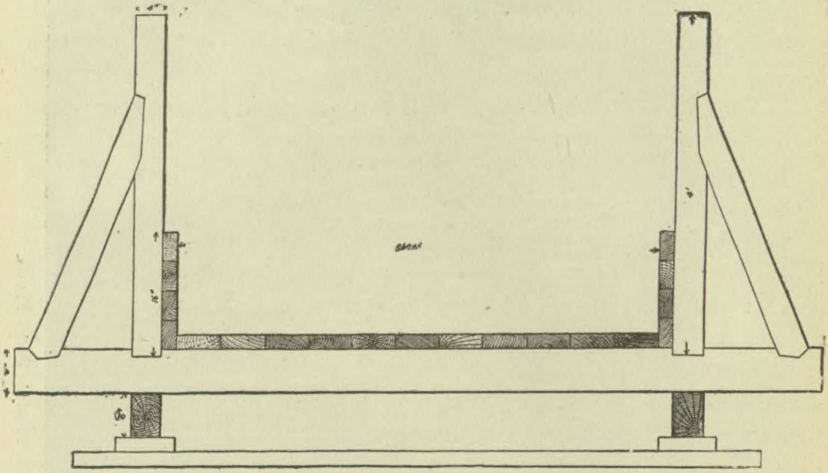


FIG. 63.—CROSS-SECTION OF SAN DIEGO FLUME.

of 4 feet and braced by diagonal uprights 2 by 4 inches and 3 feet in length.

226. **Stave and Binder Flumes.**—This type of structure has in various forms recently come into use in the West. Perhaps the first example of its use in a modified form was in the Stony creek culvert of the Central Irrigation District canal in California (Pl. XXIV). It is curved in cross-section, and is related in shape and some features of its design to the Laybourn iron flume, which has attained more or less popularity in Colorado.

The most recent and thoroughly elaborated type of this flume is that recently constructed on the Santa Anna canal. As indicated by the title, this flume is very much like the lower half of a wooden water-pipe (Art. 267). It consists of wooden staves bound and held together in a rounded bottom by iron and steel ribs and binding-rods, acting in conjunction with wooden yokes or ties across the top (Fig. 64). In its simplest form this flume is semicircular, with the top edges braced apart by the stiff yoke or cross-head, so that there is no tendency of the shell to buckle inward. As developed on the Santa Ana canal, the sides are formed of broader boards which are carried up vertically in a line tangent to the ends of the bottom half-circle and to the desired height. Upon ap-

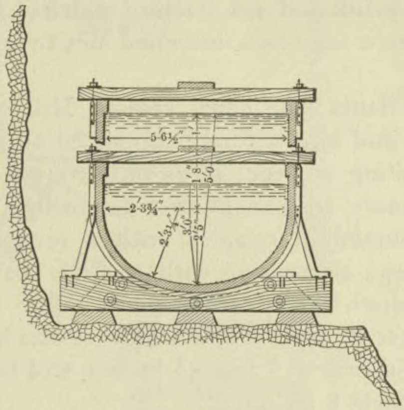


FIG. 64.—CROSS-SECTION OF STAVE AND BINDER FLUME, SANTA ANA CANAL.

plying the binding compression on this form there is a tendency to buckle inward, which is obviated by stiff ribs made to serve as binders and introduced at intervals in the shell; and to these the side boards are held by a special device which prevents their buckling, while at the same time they are permitted to move edgeways, so that the flume shell is made compressible and adjustable within the framework in order that it may be kept water-tight at all times.

The flume illustrated has an oval bottom and vertical straight sides; it is 5 feet 6 inches wide inside, 5 feet 6 inches deep below the top of the side boards, and intended to carry 5 feet in depth of water. These flumes have a stiff iron T-rib resting on a wooden bolster or sill every 8 feet, and between each pair of these ribs are two binding-rods, thus making a binder every $2\frac{2}{3}$ feet of length. As finished, the sides are but one board and a cap-piece high, making the water 3.8 feet in depth, though it is intended to enlarge the flume, as shown in the illustration, by raising the sides 2 feet, which will double the capacity. The binding-rods are of $\frac{5}{8}$-inch round mild steel, screw-threaded for 4 inches at each end, on which nuts are fastened to hold down the cross-yokes. The latter are of Oregon pine, 3 by 4 inches; the T-iron ribs are $2\frac{1}{4}$ by $2\frac{1}{4}$ inches. The side planks are held to the ribs by two lug-pieces to each plank and rib, each of which is fastened firmly to the plank by a lag-screw intended not to go through the plank.

Where the Santa Ana canal crosses Mill creek it is carried in a stave and binder flume supported on a through-rivet steel bridge resting on steel piers with concrete foundations. This is an ordinary 19-span Pratt truss bridge, each truss 48 feet long, arranged in couples, with a rocking pier at the point of juncture, alternating with 10 Fink spans 16 feet each in the intermediate braced trestle-piers. The total length of this bridge is 1072 feet. The width of the bridge between centre lines or chords is 8 feet $5\frac{1}{2}$ inches, and the Pratt trusses have each 6 panels 8 by 9 feet.

227. **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. XX). 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.

228. Iron Aqueducts.—But few of these have been constructed, though it is probable that they will continue to grow

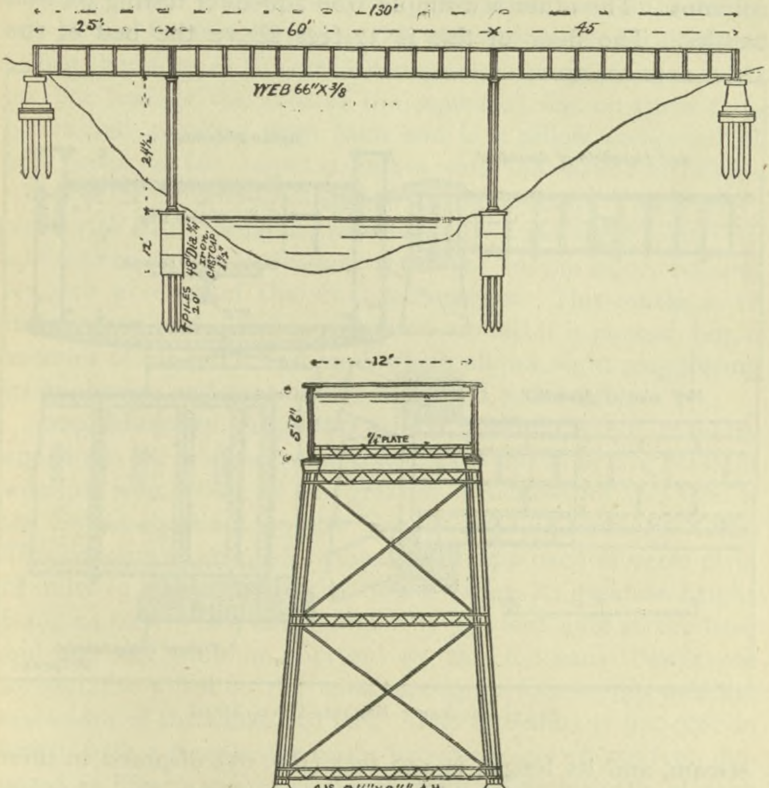


FIG. 65.—BEAR RIVER CANAL. ELEVATION AND CROSS-SECTION OF IRON FLUME ON CORINNE BRANCH.

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

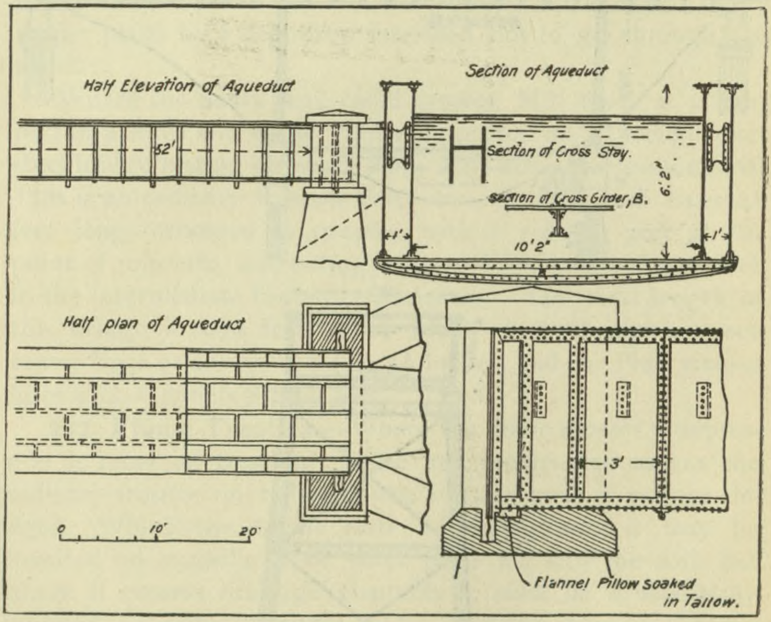


FIG. 66.—AQUEDUCT, HENARES CANAL, SPAIN.

stream, and its length is 130 feet (Fig. 65), 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 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 $\frac{3}{8}$ 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. 66), and each girder is calculated 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 mixture of tar, pitch, and sand, which allows slight play during its expansion and contraction and yet is water-tight.

229. 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. XXI). 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

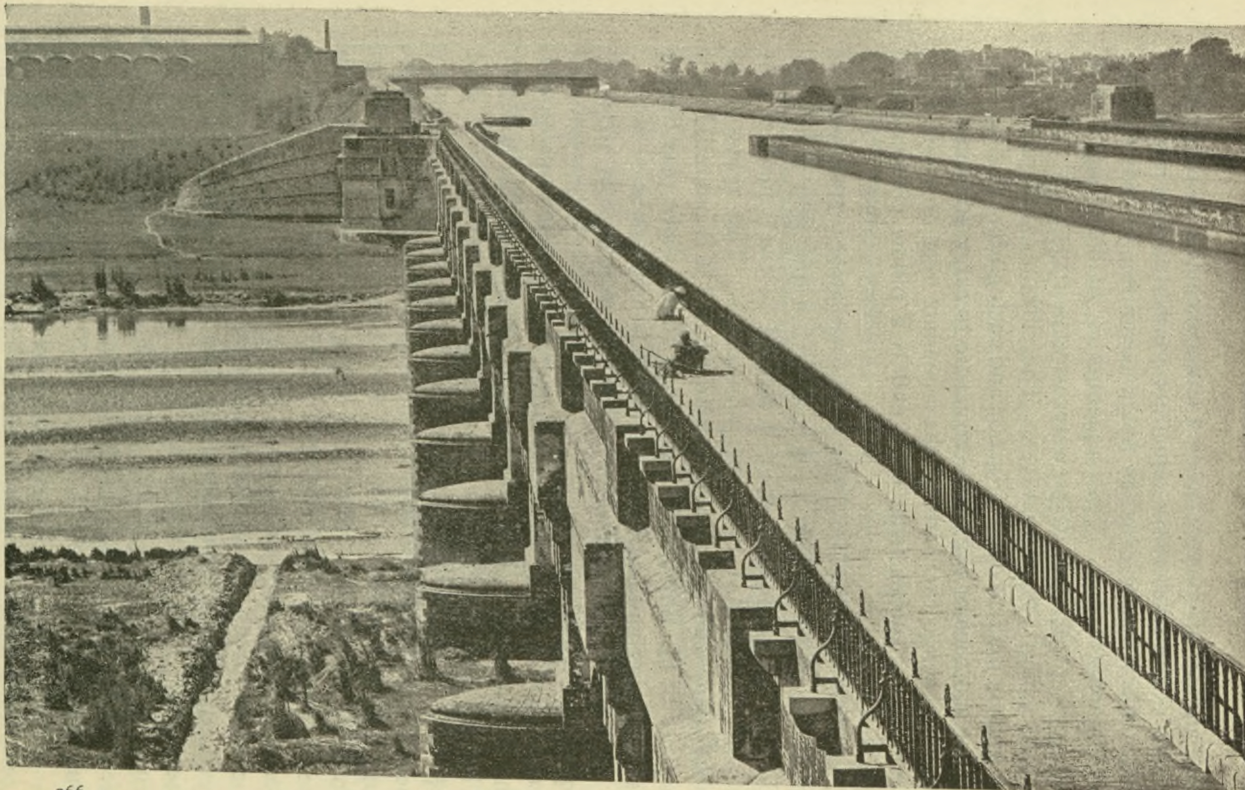


PLATE XXI.—VIEW OF SOLANI AQUEDUCT, GANGES CANAL, INDIA.

in India (Plate XXII). 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.

230. 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 superpassages 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

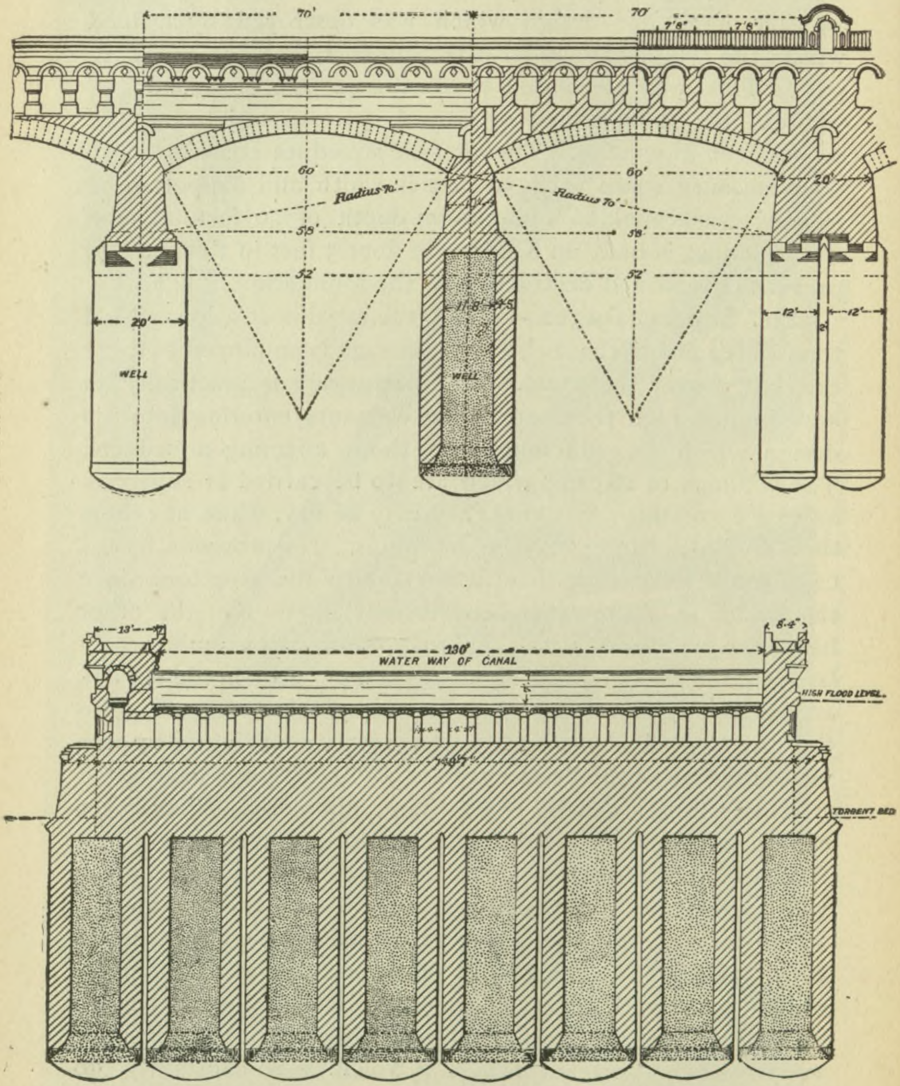


PLATE XXII.—ELEVATION AND CROSS-SECTION OF NADRAI AQUEDUCT, LOWER GANGES CANAL, INDIA.

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. XXIII) is built of masonry founded on wells, and its flooring, which is given a steep slope in order that the velocity shall 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.

231. 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 (Arts. 264 and 265), though the formula for flow through open channels may also be used in some cases (Art. 77). Inverted siphons or, as they are sometimes called, pressure pipes, are frequently employed in crossing deep depressions in place of flumes on high trestles. This method is most satisfactory in

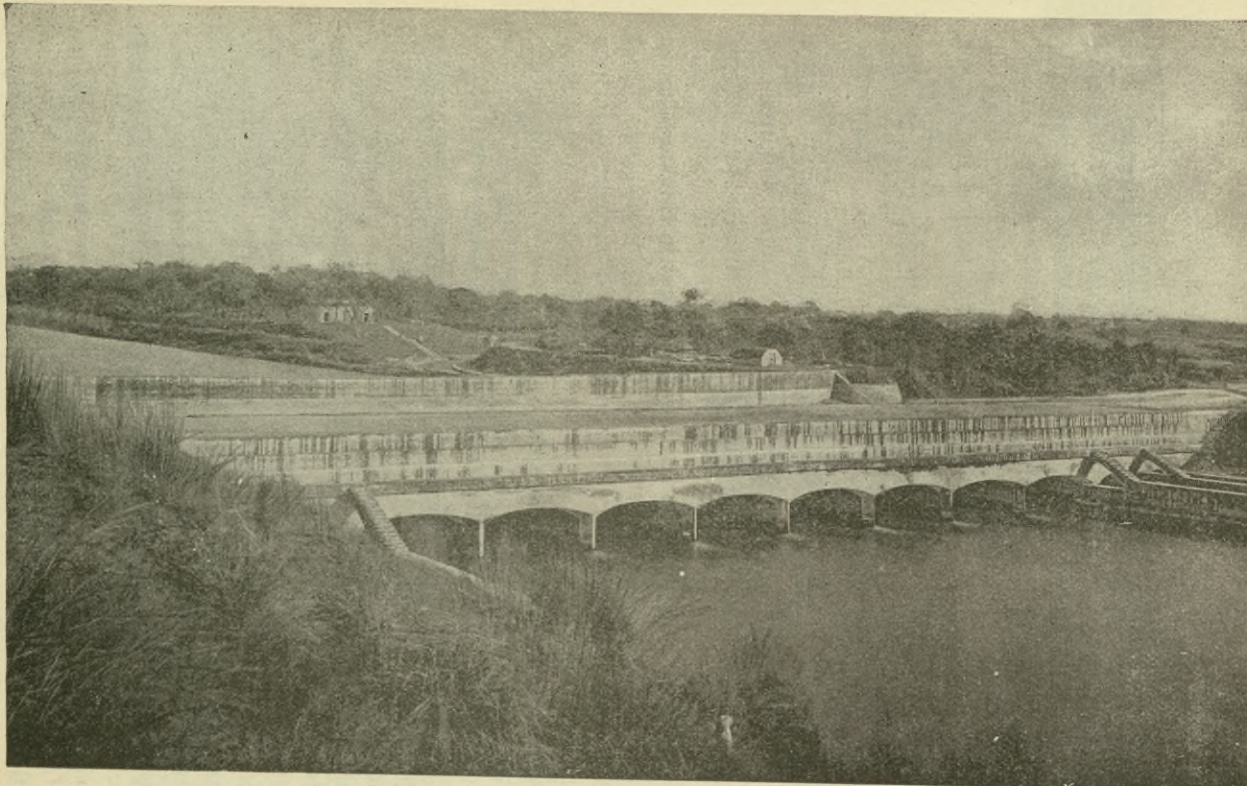


PLATE XXIII.—VIEW OF RANIPUR SUPERPASSAGE, GANGES CANAL, INDIA.

crossing depressions carrying little drainage water. In such cases wooden pipe is usually employed (Pl. XXV), though wrought- and cast-iron pipes are also frequently used.

232. 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. 67). 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.

A most interesting wooden siphon is that which carried 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. XXIV. This siphon

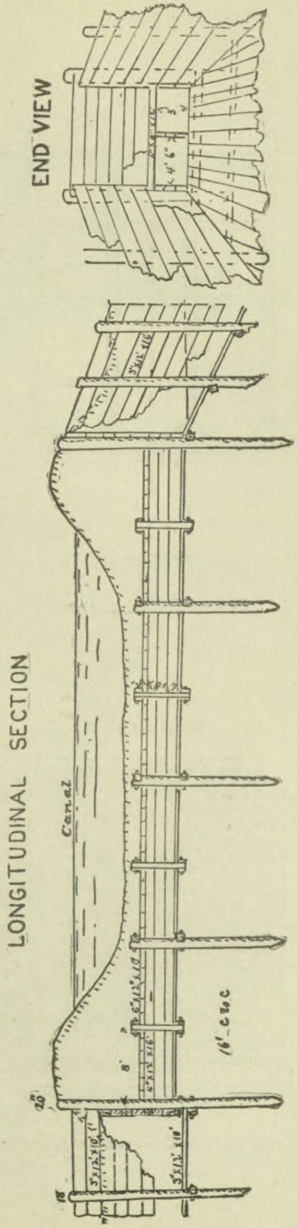


Fig. 67.—Sections of Wooden Siphon, Del Norte Canal.

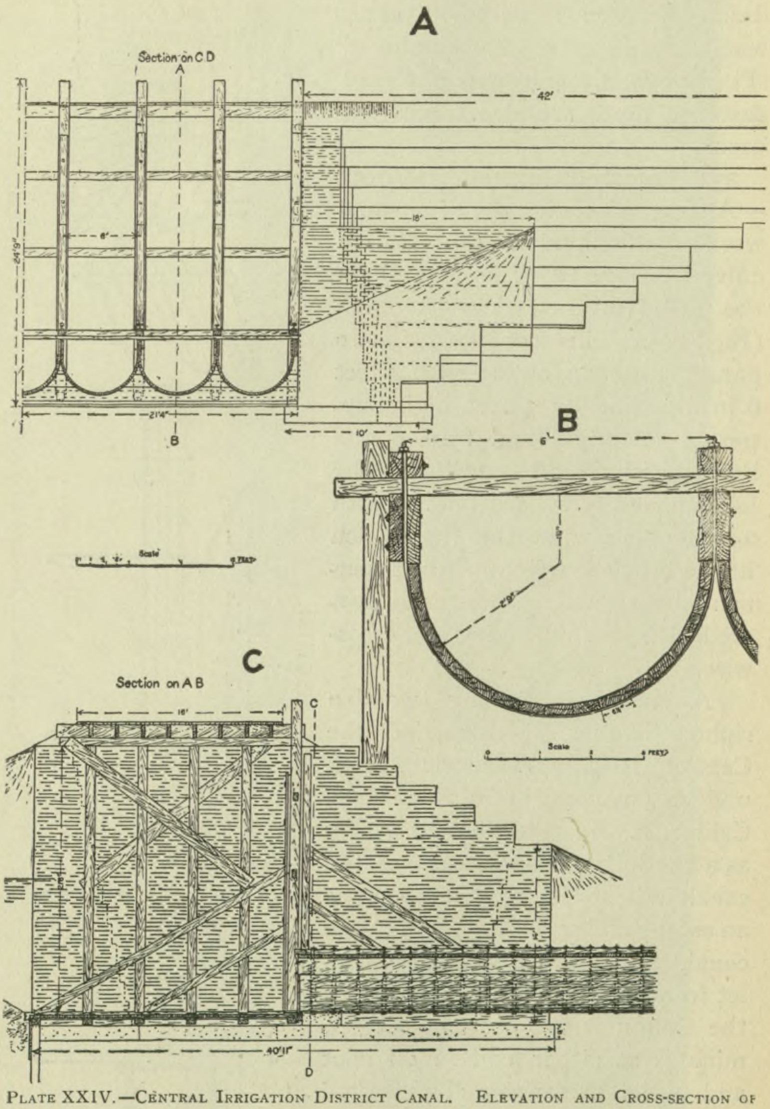


PLATE XXIV.—CENTRAL IRRIGATION DISTRICT CANAL. ELEVATION AND CROSS-SECTION OF STONY CREEK CULVERT.

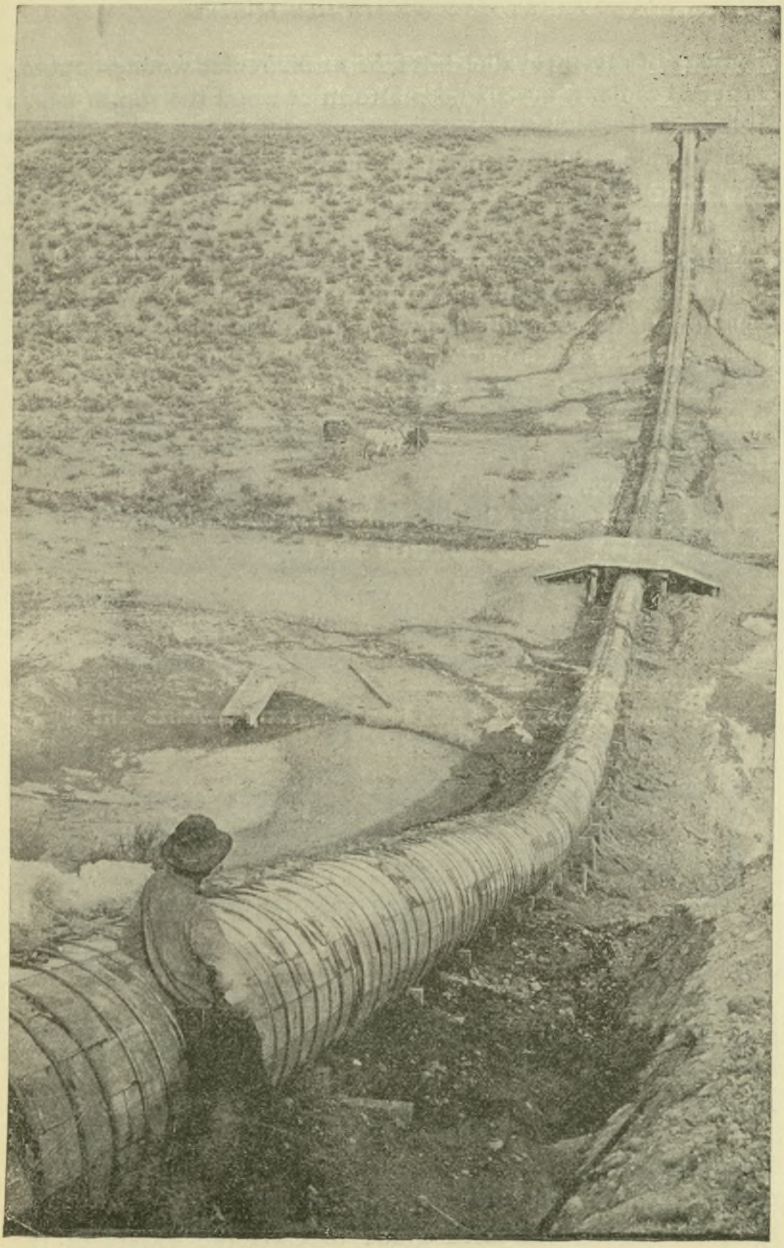


Plate XXV.—Idaho Irrigation Company's Canal. View of Wooden Siphon on Phyllis Branch.

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 flashboard 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

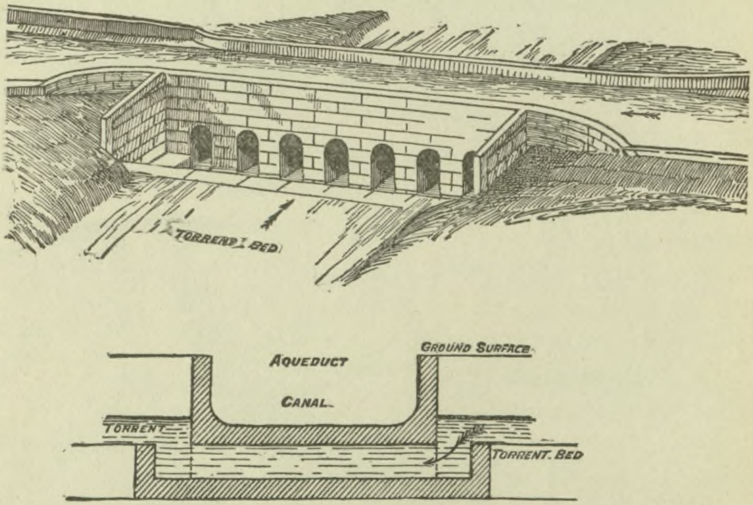


FIG. 68.—SOANE CANAL. CROSS-SECTION OF KAO NULLA SIPHON-AQUEDUCT.

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.

233. 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. 68). 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. 69) 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

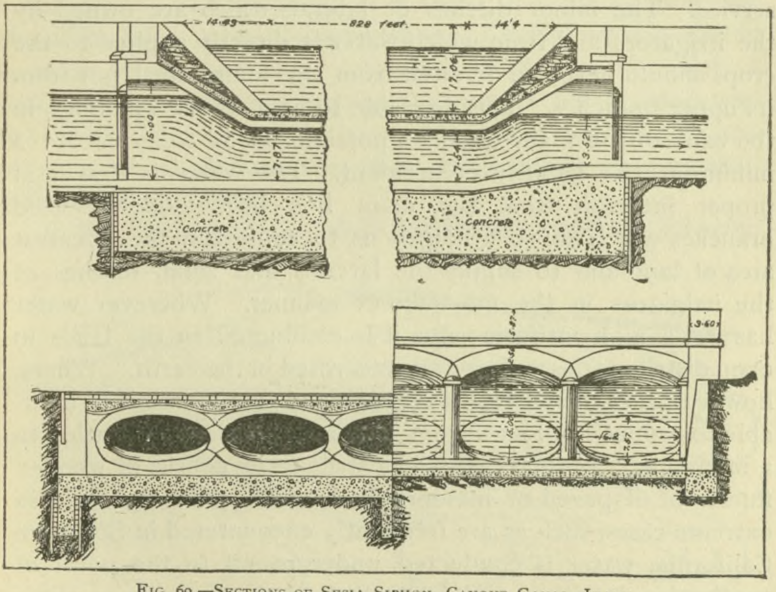


FIG. 69.—SECTIONS OF SESIA SIPHON, CAVOUR CANAL, ITALY.

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 XIII.

DISTRIBUTARIES.

234. **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 absorption 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.

235. **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 Article 135 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. 70 is shown an ideal distributary system. The con-

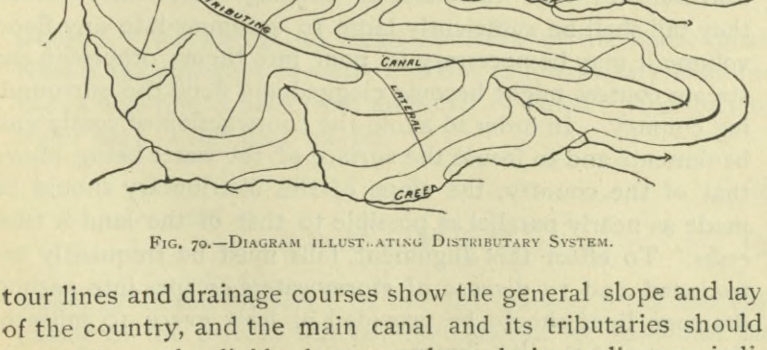


FIG. 70.—DIAGRAM ILLUSTRATING DISTRIBUTARY SYSTEM.

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.

236. 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.

237. 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

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

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

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

and

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

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

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

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

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

or substituting in the above equation, we get

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

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

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

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 each $= 1$, and for L or l very great, E^o or $E^w = 0$.

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

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

$$E^w = 1 - \frac{.03 \times 6^1}{1} = .820\cdot$$

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

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

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.

238. 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.

239. **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.

240. Capacities of Distributaries.—In planning a distributary system care should be taken to design each of the laterals below a given distributary head so that its carrying capacity shall be equal to the duty which it has to perform. This duty is dependent on various factors, which are fully discussed in Chapter V. The total area to be commanded by each distributary and its laterals should be known, the duty of water for the particular soil and crops estimated, and then careful allowance made for the area of waste land (Art. 65), being that which will remain uncultivated or will be occupied by roads, buildings, etc. Consideration having been given to all these factors, the capacities required of the different channels can then be readily determined and their dimensions fixed. The simple form of computation which this investigation takes is

A' = gross area commanded by the distributary;
a = area of waste land;
b = proportion of culturable land which is to be irrigated during an average year.

Then

$$A = (A' - a) \times b, \quad . \quad . \quad . \quad . \quad . \quad (1)$$

in which A is the net area to be irrigated, and is equivalent to the same symbol in the formula in Article 58, so that the discharge of the distributary, Q, becomes

$$Q = \frac{A}{D} = \frac{(A' - a)b}{D}. \quad . \quad . \quad . \quad . \quad . \quad (2)$$

This simple form of equation is one which has been used by Mr. R. B. Buckley in India in designing distributaries, and has been found to give the most satisfactory results. Where rotation periods are to be imposed on a canal (Art. 66), careful attention must be paid to these and to their effect on the necessary discharging capacities of the distributaries, and the latter must therefore be designed in accordance with the effect which these rotation periods or tatils will have on their required maximum discharges.

241. Distributary Channels in Earth.—The cross-section of the main or larger distributaries should be relatively the same as for main canals (Articles 148 to 150). 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 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.

242. 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. In this 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. 71 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

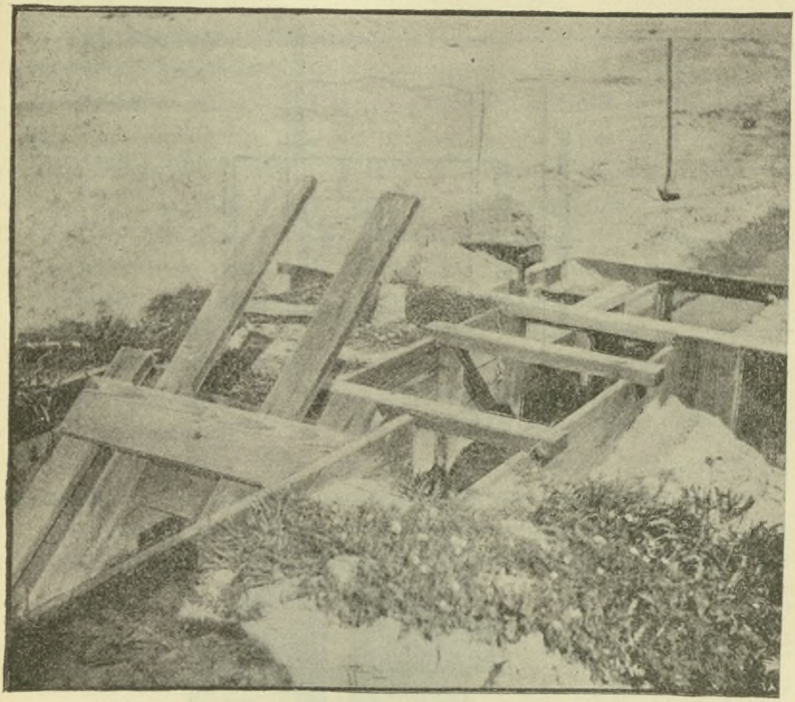


FIG. 71.—VIEW OF DISTRIBUTARY HEAD, CALLOWAY CANAL.

designed much as are the main heads on the same canal (Fig. 52).

On the Del Norte canal in Colorado a few of the distributaries 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. 72).

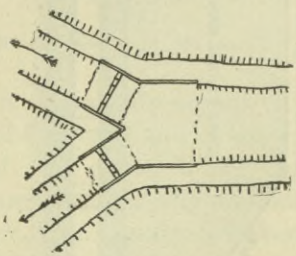


FIG. 72.—PLAN OF BIFURCATION, DEL NORTE CANAL.

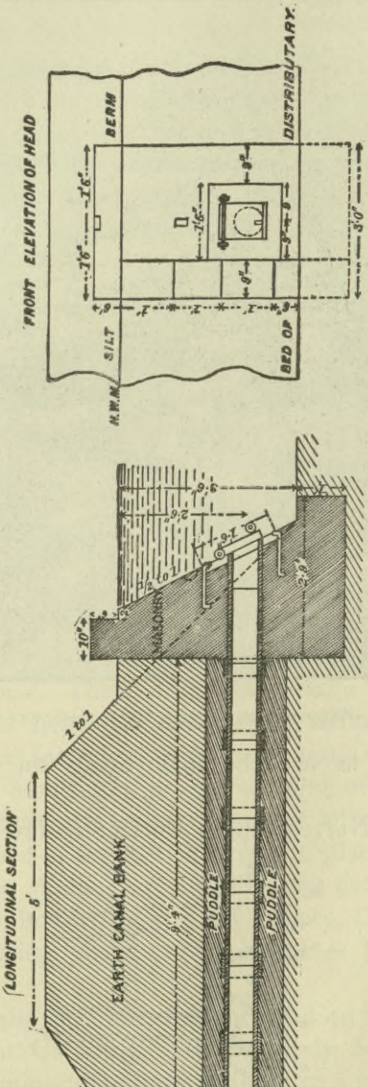


PLATE XXVI.—STANDARD MASONRY OUTLET FOR DISTRIBUTARIES, PUNJAB, INDIA.

243. 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. XXVI is shown the type of distributary head used on the [illegible] 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 172). Distributaries are passed over or under each other or the country drainage in flumes or siphons as are main canals (Chapter XII).

In the West the practice of constructing measuring weirs in the heads of distributaries and laterals, and of rating the

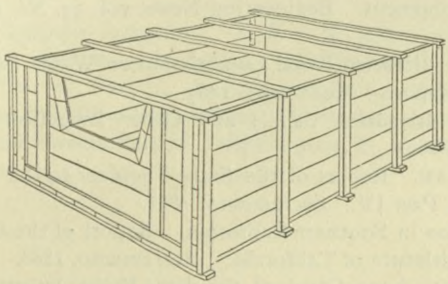


FIG. 73.—CIPPOLETTI MEASURING WEIR.

discharges of these at various depths, is rapidly growing in favor. The laws of Colorado and of some other States require the construction of such rating flumes, and it is the business of the Water Commissioners, who are under the direction of the State Engineer, to ascertain the amount of water flowing through these under various heads, dependent on the height to which the regulating gates are raised. The Water Commissioners tabulate these discharges and furnish them to the canal owners, so that when a certain volume of water is called for by irrigators, the gate may be opened the required amount and then locked. At first these rating flumes were generally constructed as simple open flumes without measuring weirs, but now the latter, usually with the trapezoidal cippoletti opening (Fig. 73), is more commonly employed.

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CHAPTER XIV.

APPLICATION OF WATER, AND PIPE IRRIGATION.

245. **Relation of Water to Plant-growth.**—The prevailing idea of irrigation has been the moistening of the soil by spreading water on its surface so as to produce by artificial means an effect similar to that caused by rain. This definition goes but a part of the way. It fails to take account of the effect of water on the physical properties of soil, and the physical and chemical processes which accompany plant-growth. More broadly, then, irrigation may be defined as the application of water to soil at such times, in such amounts and so accompanied by cultivation of the soil as to produce the condition best suited to plant-growth.

After light and heat, water is the most important of the various factors which influence plant-growth. Next to it in importance may be placed the physical condition of the soil, and lastly, plant-food. It is on these five factors, light, heat, water, soil texture, and plant-food, that the machinery for plant-growth is dependent, and in about the order stated. When these are furnished in the right amounts and at the right times, the best results may be expected in plant-growth, and of these the latter three may be controlled to a material extent by the irrigator. In our arid regions heat and light are usually furnished in abundance during the irrigating season, and fortunately the clouds rarely furnish water at such times as it is not desired. Therefore, having at his disposal a satisfactory irrigation system, the quality and amount of the crops which may be produced depends chiefly on the knowledge of the farmer as to the proper mode of utilizing the water and soil.

Plants do not naturally grow well in arid regions, because the amount of moisture evaporated from the surface of the leaves and stems by the domesticated grain and vegetable plants commonly cultivated is much greater than the amount which their roots can absorb, and as a result the plant dries up. Nearly all of our cultivated plants have originated in humid climates, and as all plants evaporate, or, as it is technically known, "transpire," large amounts of water in the process of their growth, one of the first considerations in adapting our plants to Western needs is the desirability of modifying and developing them so as to encourage the growth of those varieties best suited to the conditions. The physiological effect of irrigation is to furnish for absorption by the roots of plants sufficient moisture to balance the amount transpired by the leaves. In the humid region both air and soil are moist; in the arid region both are warm and dry. The effect of this dryness of the air is to increase the transpiration, and by keeping up a greater tension in the plant to force or hasten its growth. It is for this reason that plants cultivated in the arid region by the aid of irrigation produce larger and more rapid growths than those of the humid regions—much as is the case with hot-house plants. In order to best satisfy the conditions of plant-growth, the supply of moisture should never fail from the time the seed is placed in the soil until the crop is fully matured; and there are certain times when the cessation of the supply will do more harm than at others—for instance, in the case of corn, when grain is forming in the ear. Irrigation does away with the element of chance by supplementing moisture, by making the crop larger and more certain, and by reducing the number of acres which the farmer may cultivate.

For easier understanding of the subject of plant-growth, it may be stated that moisture absorbed by the suckers or hairs of the rootlets rises to the green parts, especially the leaves, where it assimilates carbon from the air and becomes concentrated by its evaporation or transpiration. What remains descends and is distributed through the plant, where the carbon increases the growth of its various organs by adding to the cellular structure of the plant. Transpiration takes place chiefly during the heat of the day, and is greater when the air is dryer and the soil more moist. The amount of this transpiration is enormous at times. An ordinary crop of meadow grass, cutting two tons to an acre, will transpire 6½ tons of water on a dry day and about 527 tons of water during a growing season; an average crop of wheat will evaporate 260 tons of water in a growing season. Therefore hay evaporates an amount of water equal to 5½ inches in depth of the water over the surface, and wheat an amount equal to about 2⅝ inches in depth of water in an irrigating season. The soil must therefore be maintained in a condition to supply this incessant consumption of moisture, and as soon as the moisture becomes deficient, the current or flow of sap becomes slackened and the plant remains stationary or dies. One hundred pounds of meadow-grass contains on an average 70 pounds of water, and 100 pounds of red clover over 80 pounds of water, to every 100 pounds of fresh material. Such moist plants as lettuce, cucumber, cabbage, etc., contain as much as 95 to 98 pounds of water to every 100 pounds of fresh material. These facts indicate the importance of water in plant-growth and the amount which must be supplied them, especially when it is recalled that much of the moisture reaching the soil evaporates from its surface or percolates through it.

All plants obtain their water solely through their roots, therefore a well-developed root system is of the highest importance to their welfare. Consequently any process which will aid this development, such as the mode of application of water or attracting the roots in various directions by fertilization or tillage, should be carefully considered. Saturated soil is detrimental to their growth, while half-saturated soil is most favorable to their growth, and therefore to that of the plant.

246. Relation of Soil Texture to Plant-growth.—Among the more important methods employed for conserving the moisture which reaches the soil from natural or artificial sources, and of making it available to the plant through its roots, are 1, cultivation and fertilization of the soil so as to enable it to absorb the moisture which reaches it; 2, decreasing evaporation from the soil surface, through producing a proper tilth or mulching; and 3, decreasing the evaporation from the plants. Evaporation is essential to plant-growth, yet excessive evaporation is wasteful of moisture. Under certain conditions where it is desirable to reduce evaporation, which is less rapid in moist air than in dry air and in a calm than in strong winds, we may increase the amount of moisture in the air and thus diminish the evaporation from the plant by sheltering them by growths of trees suitably disposed, especially in regions in which hot, dry winds are prevalent.

It is wasteful to allow water to flow off soils on which large sums have been expended in the introduction of irrigating systems. Every drop of water which flows from the soil is an indication that the latter is not in a proper physical condition. Either the surface slope should be changed by grading and terracing, or the soil be made more open and porous by proper preparation and cultivation. Also, wherever this cultivation has been properly performed water is more rapidly carried into the soil, and this not only diminishes the loss by flow-off, but also by evaporation from the surface. Where moderate deep ploughing will not accomplish the object desired, subsoiling must be resorted to. There is much wisdom in the practice in vogue among many of the older and more wasteful irrigators who flood their land in the fall after their crops have been harvested. The common assumption is that this adds so much moisture to the soil that it is retained until the following spring, when the plants require it. The real explanation of the benefits of fall irrigation is that the soil which is helped by this process is not in the proper physical condition. Nothing rectifies an unfit physical condition better than water. Therefore subsoiling or deep ploughing, whereby the soil is stirred up for a depth of 12 to 18 inches, breaks up its texture, and if water is properly applied thereafter beneficial effects will surely be felt by the succeeding crop. Such subsoiling should rarely be performed in the spring, and just before planting, as it works an injury to the first crop, though a benefit to the succeeding crops. It should therefore be done at a considerable time before the crop is put into the ground. After the ground has once been deeply subsoiled it should be frequently stirred up so as to maintain a mulch of loose, dry soil on the surface, and thus check evaporation and aid in the absorption of moisture. This after-cultivation should rarely be to a depth exceeding 3 to 5 inches.

There are many curious facts connected with the effect of water, cultivation, and fertilization on plant-growth. It is well known, however, that climatic conditions, changing seasons, which chiefly include light, heat, and moisture, have more effect upon the production of crops than is obtained through any degree or kind of fertilization. It is not rare to have a crop over a wide area fall off one-third or even one-half by reason of unfavorable climatic conditions, even though fertilizers may be applied most liberally. Again, soil may be analyzed and be found wanting in certain chemical requisites for plant-growth. These chemicals may be supplied by fertilizers and yet produce little beneficial effect on plants, while some other fertilizer or some other methods of cultivation may increase the crop a hundredfold, in some way which is unexplained by chemical analysis. This is undoubtedly due to the physical effects of water brought more intimately in contact with the soil in changing its texture, or to physical effects of fertilizers on soil rather than on its chemical constituents. This is probably by changing the relation of soil to moisture and heat so as to better adapt it to the needs of the particular plant. Thus, a worn-out soil is not necessarily deficient in plant-food, and soils which are barren in that they will produce little plant-growth have been shown by chemical analysis to contain an abundant supply of food material.

The texture of soil is largely determined by the amount of clay and the manner in which the clay particles are arranged in the soil. Early vegetable and fruit soils have from 4% to 10% clay, but are too light for wheat. The best tobacco lands have from 15% to 20% clay, while the subsoil in good wheat and grass land has from 20% to 50% clay. Fertilizers and water and the temperature at which the latter is applied have great effect on the texture of clayey soils by changing the arrangement of the soil grains. Thus the subsoil of a good grass land from decomposed limestone has 40% to 50% clay, yet impervious pipe-clays on which nothing can grow owing to their physical texture have been analyzed and found to have the same percentage of clay. This is because the grains are evenly arranged and the spaces between them through which water moves have so uniform a size that water can scarcely circulate. On the other hand, in the limestone soil having the same proportion of clay the grains are differently arranged or are granulated and held close against the grains of sand, thus leaving large spaces in the soil through which water and air can move readily. Experiments have been made which show that a few drops of ammonia will make a very coarse, sandy soil almost impervious to water, as will also carbonate of soda or black alkali (Art. 44). Lime and organic matter may do the same, yet the effect of lime is commonly to render heavy soils looser and more friable.

One of the most potent influences on the physical texture of soil and its relations to plant-growth is the amount of air-space in the soil and the relation of this to the amount of moisture contained therein. When water in soil amounts to over 80% of its water-holding capacity it is detrimental to plant-growth. This is because its roots are immersed in water and the soil is poor in oxygen or air. On the other hand, when only a part of the space in the soil is filled with water and the air-supply is sufficient ordinary plants do best; that is, when the water in the soil amounts to from 40% to 60% of its water-holding capacity, or in other words, when the spaces in the soil contain half air and half water. The water-holding capacity of a soil depends on the amount of space between the soil grains, and averages 40% to 60% of the total soil volume; therefore the amount of water in soil should average about 25% of the total soil volume, and there should be about the same percentage of air-space, dependent upon the character of the soil and the crop to be grown. Thus the amount of water in soil most favorable to wheat growth is 12 to 20 pounds in every 100 pounds of weight. At the other extreme, the water-holding capacity of heavy clay soils is about 44 pounds per 100 pounds of saturated soil. The plant may wilt in a soil containing 10% to 12% of moisture, becauset his amount may be so small as to make water movement to the roots too slow. In a soil of different texture, the same plant may not suffer when the amount of moisture in the soil is as low as even 6%.

247. **Theory of Cultivation by Irrigation.**—From what has preceded it is evident that the correct mode of applying water to soil in irrigating various crops is yet but a matter of merest experiment. It is dependent on many varying factors, among which are the physical and chemical properties of the soil; the temperature of both air and water; and the condition of the crop growth, that is, the time when it requires irrigation.

As has been shown, the irrigator must strive to accomplish three results in the most perfect manner; 1, he should not apply either too much or too little water, but just sufficient to about half fill the soil spaces; 2, this water should be applied in such manner as to be most evenly distributed throughout the soil in order to encourage root-growth in all directions; 3, the soil should be so cultivated as to create the loosest texture, and thus enable it to hold the largest proportion both of air and water without settling and becoming heavy or soggy. The physical texture of the soil is beneficially affected by deep subsoiling or ploughing, followed by rain or irrigation, at a considerable period of time before the planting of crops—preferably in late fall, when water is abundant. After seeding the soil should not again be deeply ploughed, but should be frequently stirred for a few inches in depth to produce a surface mulch, especially prior to irrigation, that the water may be absorbed and not evaporate too freely. The first cultivation after seeding cannot take place until after the plant has attained sufficient growth to render it possible to avoid injuring it. Certain crops, as meadow-grass and hay, cannot be so cultivated without injury to them. Others which are planted in rows, as potatoes and corn, and orchards and vineyards, offer excellent opportunities for such after cultivation. A well-cared-for orchard or vineyard should never show a sign of weed or other plant growth at any period in the year. Water cannot be applied after seeding to hay, grain, and similar crops, until they have attained such a height above the ground that the cracking of the drying soil at the surface will not seriously injure the crown growth and delicate stalks.

This brings us to the theory of time of application and amount of water, which again can be only determined by experiment and yet is dependent upon certain general principles. Experiments made at the Agricultural Experiment Station of Utah with early and late watering of hay and grain crops indicates for the latter that early and late watering produces decidedly larger crops of grain and but little less of straw. In the climate of Utah early irrigation means watering in the middle of May instead of toward the middle of June, as is customary, and late irrigation means watering but a week before harvest time. The effect of usual irrigation instead of beginning early in the season and ending late in the season seems to be that the early irrigation affects soil temperature as well as its physical properties. Grain plants absorb a large amount of moisture during the time when they are taking on stem and leaf. There is but little moisture relatively in the grain, and this is formed rapidly and during a short period of time. The application of water shortly before harvesting forces the grain, makes it ripen rapidly, and produces a greater ratio of grain to straw, and a larger yield of the former as well as of the whole plant. On the other hand, the influence of early and late watering on potatoes has the opposite effect. In the experiments referred to this crop suffered materially from the effects of early watering, due, it is believed, to watering before the plant demanded it thus reducing the temperature of the soil and of the air around it in such a manner as to practically delay the season. In comparisons of night and day irrigation it transpired that the temperature of soil irrigated at night was higher than that irrigated in the day. It is well known that irrigation lowers the temperature of the soil and therefore retards plant growth for the time being, so that there appears to be an advantage in this point for night irrigation; yet for grain crops the yield of the grain is greater for day than for night irrigation, due, probably, to checking of growth of foliage. On the other hand, the yield of straw is greater as a result of night irrigation.

The method of applying water is also indicated to some extent by the foregoing considerations. Where water is applied to plants by flooding, especially where evaporation is great and the amount applied relatively small, it results in a shallow growth by attracting the roots near to the surface. If water is applied from small orifices of subsurface pipes it encourages the root-growth in that direction only, and prevents their spreading in other directions. Irrigation by deep furrows at some distance apart one from the other tends to draw root-growth toward the furrows, though in certain plants as potatoes, corn, celery, and others which are naturally grown in ridged rows, this form of irrigation is best suited to root development. For many other varieties of plants, and especially for fruit-trees and vines, the method of application which is probably best suited to root development is by means of many small but deep furrows carrying small volumes of water which shall completely enter and uniformly saturate the soil in all directions.

As to time and amount of water, each soil, crop, and climate is a law unto itself, and experience, tempered by a knowledge of the physical and chemical effects of moisture and the texture of soil on plant-growth must indicate to the irrigator the course best suitable for his particular conditions. In Chapter V this subject has been treated in a general manner and additional facts are pointed out in the following articles. Each condition calls for a particular depth of watering and a particular time for applying water which, if properly fulfilled, will produce superior results.

248. 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.

249. **Preparation of Ground for Irrigation.** — The amount and kind of preparation required to put ground in the most suitable condition for irrigation is indicated in the above general discussion. In every case where the best results are desired the greatest care should be taken in properly grading and laying out the irrigable lands. A little time and money devoted in the beginning to a proper preparation of the land will be more than repaid in the saving of water and the ease and cheapness with which it can be applied. Land once properly prepared can always be cheaply and easily maintained in the best condition. The real secret of successful irrigation is intensive cultivation, by which is meant careful and tireless attention to a very small area of land by one individual. A single farmer can produce larger and better crops and obtain greater profit from thorough and careful cultivation of 10 to 20 acres than from superficial cultivation of 100 acres. Where land is properly prepared one man can quickly and thoroughly handle water on ten acres whereas two or three men would not produce as satisfactory results on the same area illy graded and prepared.

Where water is to be applied by the flooding method, great care should be taken to produce a perfectly uniform slope and surface. This should be done by the use of some of the grading tools which are now on the market, in connection with levels taken to determine within an inch or two as maximum limits the slope of the land. If the surface is particularly uniform, deep ploughing followed by harrowing and then dragging over the surface a heavy log or beam or some other device for levelling the land will suffice. At other times the slope may be too great to permit of irrigation by flooding, because it would afford such a velocity as to cause erosion of the soil. This is to be corrected by grading the soil so as to form checks or in extreme cases by terracing, which is but an exaggerated form of check. If the surface is uneven the water will stand about in pools, so that certain portions of the land will receive too much and become supersaturated while other places will be high and dry. It is only, therefore, by the creation of a uniform surface that water can be satisfactorily applied by the flooding method.

Where the soil is to be prepared for irrigation by furrows, and especially where these furrows are to be small and narrow,

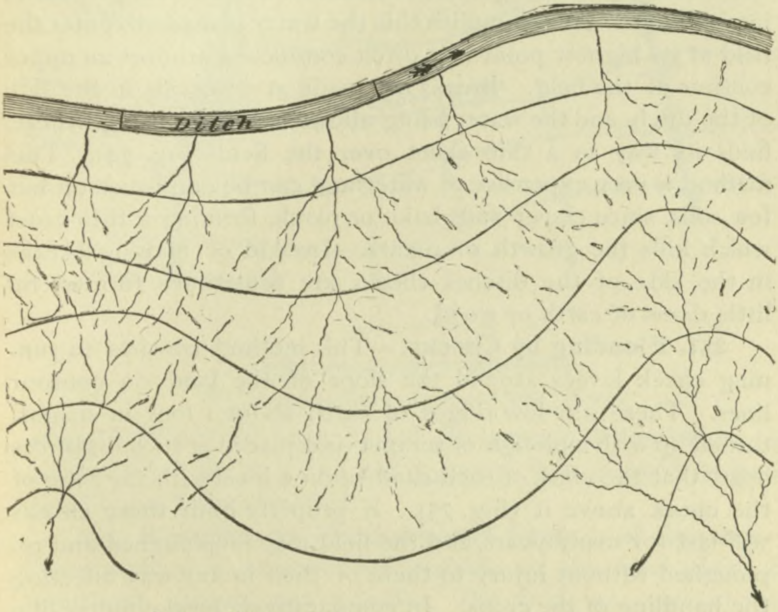

FIG. 74.—DIAGRAM ILLUSTRATING FLOODING OF MEADOWS.

as in the drill method of irrigation, even greater care must be taken than in the flooding method in producing the proper slope and surface level. If the slope of the land is too steep the furrows and drills will, because of the velocity of the water, be rapidly eroded. If the slope is too slight the water may take so long in flowing across the fields as to be all evaporated or absorbed before it reaches the further end. Too steep slopes may be rectified by running small ditches or flumes down the slope of the ground and inserting falls in them to overcome excess of slope, and by turning the water from these into lateral furrows and drills which run at such an angle as possess the proper fall.

250. Sidehill Flooding of Meadows.—This method is the most wasteful of water, but it is that most commonly practised 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. 74). 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.

251. 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. 75). If properly built these checks will last for many years, and the field may be ploughed and reploughed 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. 75) and admitted by gates into each separate

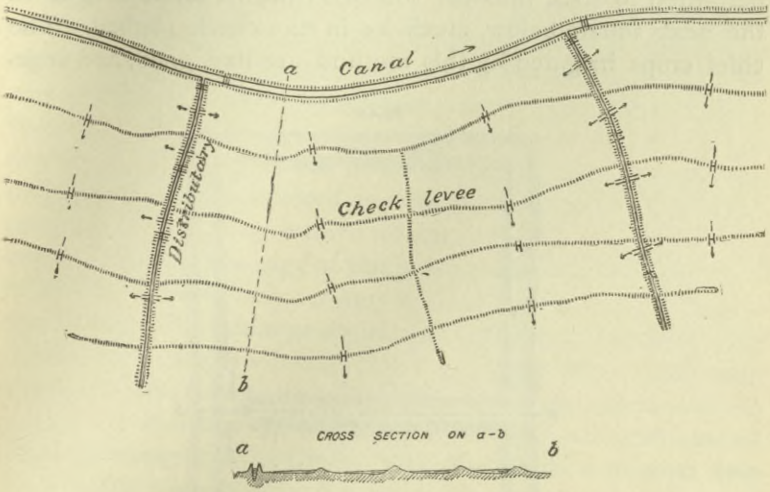

FIG. 75.—IRRIGATION BY SYSTEM OF CHECK-LEVEES.

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.

252. 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. 76), and these are separated by ridges or levees of from 10 to 12 inches in height in 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 vege-

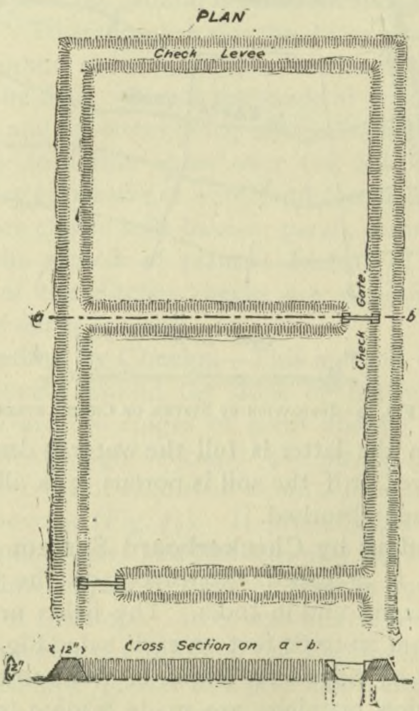

FIG. 76.—FLOODING BY SYSTEM OF SQUARES.

tables. 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

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.

253. **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.

254. **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 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 (Art. 257), and the water penetrates by sidewise soakage to the crops. Grain 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 fur-

PLATE XXVII.—IRRIGATING ORCHARD BY TERRACED BASINS ON HILLSIDE.

row irrigation by rolling the field after the grain is planted with a heavy roller on the surface of which are annular projections of a few inches in height and from ½ to 1 foot apart. These make grooves in the surface of the soil in a direction

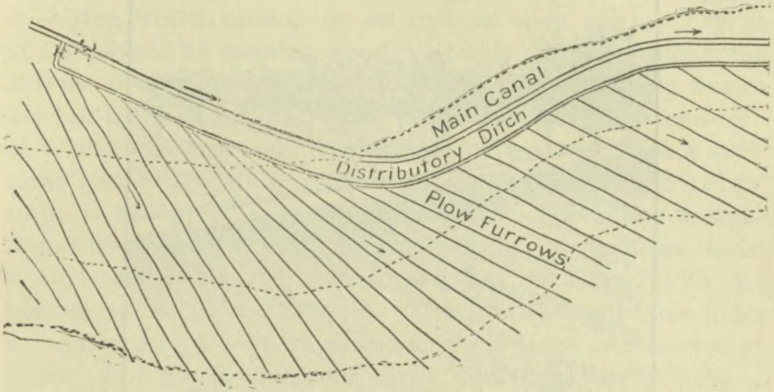

FIG. 77.—FURROW IRRIGATION OF GRAIN.

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.

255. Combined Flooding and Furrow Irrigation of Orchards.—Where trees 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 ditches, generally in a double row, one on either side of and at a short distance from the trees or vines (Fig. 78). 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.

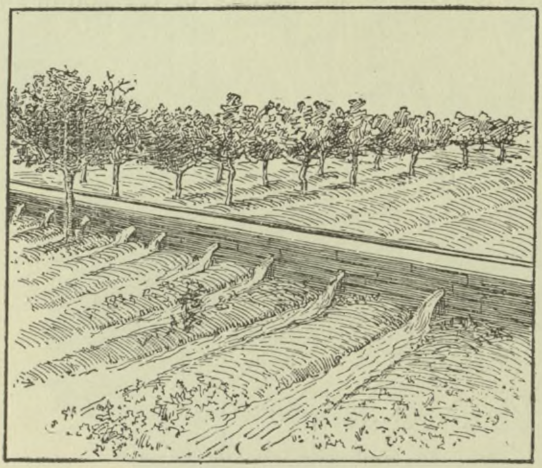

FIG. 78.—FURROW IRRIGATION OF ORCHARDS.

256. 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. 78) 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. 78). 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.

257. Ditch and Furrow Checks. — Water flowing in minor ditches must be checked and turned into the field channels and furrows by some temporary and inexpensive means. Likewise water flowing through the smaller furrows must be turned from these into other furrows and drill-rows by some similar temporary expedient. The plan of erecting wooden structures at such points is not only expensive, but inconvenient, as permanent structures interfere with the working of the fields. The form of check which works about as satisfactorily as any on the larger field ditches is the canvas dam (Fig. 78*a*), which consists of a simple piece of scantling from 5 to 7 feet long, according to the width of the ditch, on which is nailed and held by a lath a piece of 10 or 12 ounce canvas from 50 to 60 inches wide, preferably large enough to afford ample protection to the sides of the ditch, and about 3 feet in length. At the bottom of this piece of canvas, laths and a rope should be fastened for properly manipulating it.

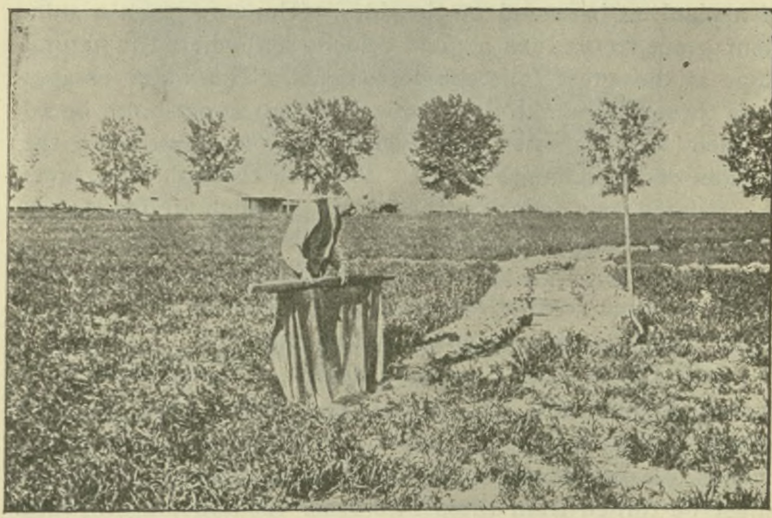

FIG. 78a.—USING CANVAS DAM.

The scantling is laid across the ditch banks, and the canvas conforms to the inner surface of the ditch, and is held in place by a stake of wood driven through the rope loop at the bottom. This canvas dam obviates the necessity of injuring the sides of the ditch by earth, temporary earth or wood dams.

The older and more common mode of checking water in ditches or furrows is by throwing dirt into these from either bank until the flow of water is blocked. This method is still probably the most satisfactory for use in small furrows and drills, which a spadeful of earth or a small stone will block. Another form of dam than that of canvas, and which is not unsatisfactory under certain conditions, is that of a thick piece of sheet metal, which may have a couple of handles at the top, and be fastened to a small wooden scantling. This piece of sheet iron, which should be curved or pointed at the bottom, can be forced vertically into the ground in a manner similar to that of driving a shovel into the bed of the ditch, and this will produce a satisfactory temporary check.

258. Subsurface Irrigation.—Irrigation from beneath the surface or sub-irrigation is theoretically one of the most economical and satisfactory methods of applying water to plants. The idea is to replace seepage from above by absorption from below, which, to be perfect, should not wet the surface. As a result the water thus applied to the soil should theoretically have the same temperature, and thus not set the plant-growth back, and as long as the water does not reach the surface it is presumable that just the right amount has been applied and not sufficient to saturate the soil. This is effected by laying pipes underground, and these derive their supply from distributaries, which are usually of vitrified pipe. The cost of preparing land for this mode of irrigation is relatively great, but is more than repaid by the saving in water charges, since the duty of water reaches as high as 500 to 1000 acres per second foot. This method has been most extensively employed among the valuable fruit-lands of Southern California, which are usually divided into orchard lots of from 10 to 20 acres each. The company distributing pipe terminates at the highest point in each of these lots, and from this the sub-irrigation pipes of the farmer are conducted through the orchards.

In practice this method has not proven as satisfactory as had been anticipated. Roots clog the orifices of the sub-irrigation pipes, and uniform watering of the soil such as is required to produce the best form of root-growth is practically impossible. This is true even when roots do not clog the orifices, and, moreover, growth of these roots has also the effect of bursting and destroying the pipes. Yet in some localities where this method has been introduced and great care and attention have been paid to the maintenance of the sub-irrigation pipes very satisfactory results are still achieved. Sub-irrigation, while attractive theoretically, is considered a failure even in Southern California, where it has been most thoroughly tried and no expense spared to make it effective. If the soil is sufficiently open to give the required drainage most of the water will seep off instead of ascending toward the plants. The water also tends to wet only that part of the soil which is cold and sour, and does not sufficiently wet the warm, rich surface soil. In Kansas experience in sub-irrigation is similar to that in California. For the first two years everything works most satisfactorily, and as this method has only been practised in Kansas for a few years sub-irrigation is still favorably considered in that locality. It is claimed to work well with fruits, but best with edible roots, as beets, carrots, and potatoes. It is claimed that it does not exclude air from the soil, and that with deep-rooting plants it is less work to operate. In fact, there appears to be still a great difference of opinion as to the advantages and disadvantages of this mode of irrigation; but it is well known that it does not work well in sandy or gravelly soils, because of the tendency of the water to drain through these, and this may account for the disfavor into which this method has fallen in Southern California, where this class of soils predominate. This system works best in loamy and silty soils, where the effects of capillarity are greatest, and this fact may account for the favor which this system has met with in Kansas.

On the whole, sub-irrigation is troublesome and expensive to operate, and as it does not accord too well with the theoretical requirements of plants and soil, it is probable that it will be less adopted in the future than it has in the past. The experiments on sub-irrigation at the Utah Agricultural Station developed the fact that for that climate and soil sub-irrigation failed to supply sufficient moisture for growing crops, as the lateral movement of water due to capillarity was too slow to furnish the requisite moisture for transpiration. At the same time the atmosphere about the plants to the height of 12 inches, as well as the temperature of the soil, were warmer from sub-irrigation than by surface irrigation.

259. Sub-irrigation Pipes.—These are made of sheet-iron or steel or of some porous or glazed material, the latter 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.

260. 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.

261. 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. 79). At the top of the standpipe 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

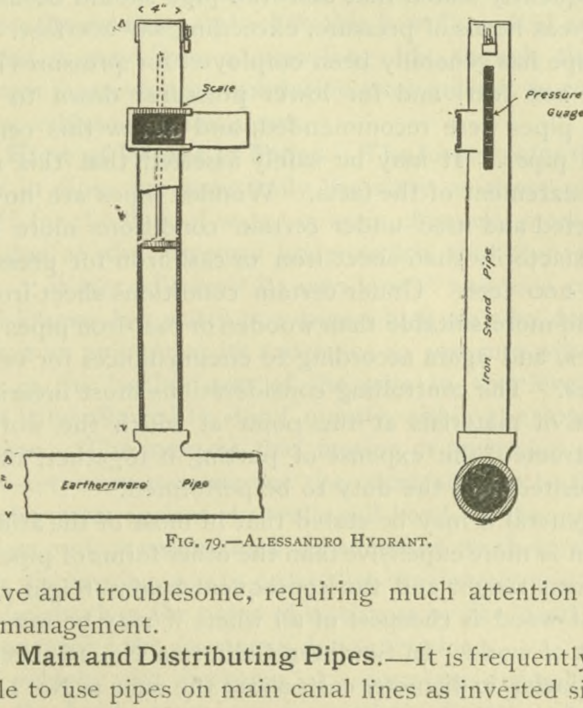

FIG. 79.—ALESSANDRO HYDRANT.

expensive and troublesome, requiring much attention for its proper management.

262. **Main and Distributing Pipes.**—It is frequently found desirable to use pipes on main canal lines as inverted siphons, or pressure pipes to carry water over depressions which would otherwise have to be crossed by flumes or similar structures. Since methods of building suitable pipes of wood have been introduced in the West this form of structure has come into more popular use because of its economy of water, its relative cheapness, and its durability; not only for passing drainage lines on main canals, but also in many places on distributaries.

The pipes more generally used are of cast iron, sheet steel or iron, wood, cement, or vitrified, according to the amount of pressure which they may have to withstand and the relative cost of the various materials in the locality in which they are to be used. The pipes in more general use range from 4 to

60 inches in diameter, or in extreme cases are even larger, though it has usually been found desirable to use two pipes when the volumes to be carried call for excessive diameters. It is frequently stated that cast-iron pipe should be used only under great heads of pressure, exceeding say 200 feet. Sheet-metal pipe has generally been employed for pressures between 50 and 200 feet, and for lower pressures down to 20 feet wooden pipes were recommended, and below this cement or vitrified pipes. It may be safely asserted that this is not a correct statement of the facts. Wooden pipes are now being constructed and used under certain conditions more cheaply and satisfactorily than sheet iron or cast iron for pressures as high as 200 feet. Under certain conditions sheet-iron pipes are found more suitable than wooden or cast-iron pipes for low pressures, and again according to circumstances for very high pressures. The controlling considerations must invariably be the cost of materials at the point at which the work is to be constructed, the expense of putting it together, the durability desired, and the duty to be performed.

In general it may be stated that in most of the arid region cast iron is more expensive than the other forms of pipe, owing to its great weight and the consequent heavy freight charges, and that wood is cheapest of all where it may be sawed near the seat of work. In Southern California No. 16 sheet-metal pipe 20 inches in diameter costs about the same as No. 14 sheet-metal pipe 18 inches in diameter and about the same as No. 12 sheet-metal pipe only 12 inches in diameter, while medium cast-iron pipe for the same cost must be as small as 8 inches in diameter. Vitrified pipe of the same cost would have a diameter of 20 inches and wooden pipe a diameter of 18 inches. In general it may be asserted that in this locality, and the same is true though in a greater degree of other portions of the arid region, that for a given diameter or capacity wooden pipe costs about the same as No. 14 sheet-iron or steel pipe of the same diameter, and this costs less than heavier sheet pipe and much less than cast-iron. Experience gained in the West indicates that the life of a wooden pipe well constructed and properly tended is quite as long as that of well-constructed and asphaltum-coated sheet-metal pipes. It may be expected under proper conditions to have a life of at least forty years. On the other hand, the life of a sheet-metal pipe depends largely on the coating, and when this is well applied and well maintained it may have a very long life, though the least carelessness in preventing oxidation may cause it to rust and become worthless in a few years.

263. **Flow of Water in Pipes.**—The formulas for the flow of water in pipes are practically the same as those given in Chap. VI for the flow of water in open channels, modified by friction due to the pressure under which the pipe may be placed. "Water will seek its own level," and as a result of this well-known law water in a pipe which may be depressed to as great an amount as its resistance to pressure will stand, will rise in the further arm of the pipe to the level of its source if it be allowed to stand quietly within the pipe without motion. The moment that motion or flow takes place it becomes necessary to overcome the resistance due to the friction of the water against the walls and bends of the pipe, and this balance of pressure must be obtained by shortening its lower or discharging arm in order that the source may be at a greater height than the point of discharge by the head necessary to overcome the resistance due to friction.

The velocity and discharge of a pipe are dependent not only on the head or pressure for a given diameter, but also upon the frictional resistance which the interior of the pipe offers to the flow of water. Accordingly the discharge for a given head may be increased for the same diameter of pipe by using a pipe with smoother lining with the straightest possible alignment and fewest obstructions to flow in the way of bends and joints. In conducting water through pipes from storage reservoirs, account must always be taken of the reduction of pressure in the pipe due to the lowering or draining of the water from the reservoir, and such pipes should always be calculated on a basis of the pressure due to the height of their inlets, and not to the height of water in the reservoir.

Should the pipe rise at any point above the hydraulic grade-line (*BE*, Fig. 80), or should it have vertical bends of any considerable height, air will accumulate at the summit of these bends at *A*. This air is compressed from one side by the head h, and on the other side by the head H; then if $h = H$ and the surface of the water at *D* does not meet the outlet of the pipe *E*, there will be no discharge. Under less ex-

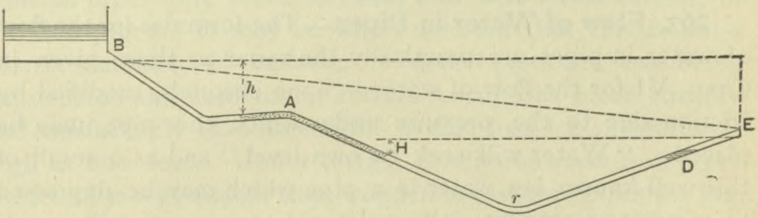

FIG. 80.—FLOW OF WATER IN PRESSURE PIPES.

treme cases the discharge will be greatly reduced, and it is therefore necessary, where there are such vertical bends in pipes, to insert ventilators, or air-valves as they are called, to release the air-pressure at such points. Likewise at the lowest point in a pipe crossing, say a ravine, at the point *r*, mud, dirt, and other obstructions are apt to accumulate, and it is necessary to insert there mud-valves or blow-offs for clearing the pipes of such obstructions.

264. **Formulas of Flow in Pipes.**—If there were no resistance to the flow in pipes, due to friction or pressure, the velocities of flow would be similar to those for falling bodies. Experience has shown, however, that velocity of flow in pipes is equal to only two thirds of the height, the remaining third being lost in overcoming the resistance to flow of water in entering the pipe; hence under such circumstances we have the formula

$$v = \sqrt{\frac{4gh}{3}}, \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (1)$$

in which g is the acceleration due to gravity or 32.2 feet, and h is the head or height of fall.

One of the controlling factors in determining the flow of water through pipes under pressure is the hydraulic grade-line, which is a straight line drawn from the entry to the exit of the pipe (*BE*, Fig. 80). Water flowing through a pipe which has several vertical bends in it may rise nearly to the level of this hydraulic grade-line, though the pipe should never except under the most unfavorable circumstances rise in any portion of its length above this line, otherwise there will be a decided loss of pressure at this point and a diminished flow below it, calling for an increased diameter from that point on to carry the discharge required.

Where the vertical bends in a pipe are kept well below the hydraulic grade-line or where the pipe is practically in the grade-line, in other words, its alignment is straight and direct, it will be under little or no pressure, and the formulas of flow of water in open channels are directly applicable to all computations necessary to determine the velocity and discharge in such pipes. These formulas are given in Chapter VI, and may therefore be used in computing the discharges and other factors required in designing pipes which are not under pressure.

There are many formulas for the flow of water in pipes under pressure, notable among which are those of Weisbach, D'Arcy, Flynn, and others whom it will be unnecessary to mention here. It is not desirable in such a work as this to go into the theory and derivation of such formulas of flow of water in pipes. These subjects have been fully and thoroughly treated of in many accessible publications, among which are those of Weisbach, Fanning, Bovey, and Flynn. For the purposes of this work it is sufficient to give but a few of the simpler of such formulas, and the necessary tables to aid in their use. The most satisfactory modification of the more abstruse formulas are those published by Mr. P. J. Flynn in private pamphlets or in the Transactions of the Technical Society of the Pacific Coast and in his admirable treatise on "Irrigation Canals and other Irrigation Works," and these, with some modifications of tables for their use, are given in the following article.

265. Tables of Flow in Pipes With and Without Pressure.—The Chezy modification of Kutter's formula given in Art. 76,

$$v = C\sqrt{ri}, \quad . \quad . \quad . \quad . \quad . \quad . \quad (2)$$

is really the most satisfactory which can be used to express the flow in pipes without pressure. This formula is sufficient only for particular conditions of pipe surface, unless the value of C be made to include the coefficient of roughness and other modifying elements, according to Kutter's method. So considering it here, as was done in the case of flow in open channels, the tables given in Art. 77 then furnish the material from which to compute nearly all of the elements of flow in pipes either clean or tuberculated.

The D'Arcy formula for flow in clean pipes is

$$v = \left(\frac{ri}{.00007726 + \frac{.00000162}{r}} \right)^{\frac{1}{2}} \quad . \quad . \quad . \quad . \quad (3)$$

The value of the coefficient in this formula depends on the hydraulic mean depth r, and is not affected by slope as is the case with Kutter's formula. D'Arcy's formula is based on careful experiments made with clean pipes and is therefore quite accurate for pipes of moderate diameter, but not, however, for pipes of large diameter. Kutter's formula, on the other hand, is derived not only from experiments with small, but also with very large, channels; and as it takes into consideration the roughness of surface as well as the slope, it agrees more accurately with the actual discharge than does D'Arcy's for large pipes.

Flynn has modified D'Arcy's formula (3) to the following simplified form:

$$v = \left(\frac{155256}{12d + 1} \right)^{\frac{1}{2}} \times \sqrt{ri}. \quad . \quad . \quad . \quad . \quad . \quad . \quad (4)$$

He has also taken D'Arcy's formula for flow in old cast-iron pipes badly tuberculated, and from it has derived the simplified form

$$v = \left(\frac{70244}{12d + 1}\right)^{\frac{1}{2}} \times \sqrt{ri}, \quad . \quad . \quad . \quad . \quad . \quad (5)$$

in which d equals the diameter of the pipe, and i is the sign of the slope or fall of water in any height h, divided by 1, which is the horizontal projection of the hydraulic grade-line joining the two extremities of the pipe. It may be further stated that r, which is the hydraulic mean depth, is equal to one fourth of the diameter in the case of circular pipes. These formulas may again be reduced to the more simplified Chezy form

$$V = C\sqrt{ri} \quad \text{and} \quad Q = AC\sqrt{ri}, \quad . \quad . \quad . \quad . \quad (6)$$

and the values of these factors have been tabulated by Mr. Flynn in such form as to aid the rapid solution of all problems relating to the flow of water in pipes.

In order to simplify Kutter's formula for computation of velocity of flow in pipes under varying conditions of roughness and diameter, Mr. Flynn reduces it to the following form, after computing Tables XVIII and XIX to facilitate its use :

$$v = \left\{ \frac{K}{1 + \left(44.41 \times \frac{n}{\sqrt{r}}\right)} \right\} \sqrt{ri}. \quad . \quad . \quad . \quad . \quad (7)$$

TABLE XVIII.

VALUES OF K FOR USE IN FLYNN'S MODIFICATION OF KUTTER'S FORMULA.

n	K	n	K	n	K	n	K	n	K
.009	245.63	.012	195.33	.015	165.14	.018	145.03	.021	130.65
.010	225.51	.013	183.72	.016	157.60	.019	139.73	.022	126.73
.011	209.05	.014	137.77	.017	150.94	.020	134.96	.023	124.90

TABLE XIX.

VALUES OF THE $\sqrt{r}$ FOR CIRCULAR PIPES OF DIFFERENT DIAMETERS.

Diameter. Ft. In.	$\sqrt{r}$ in Feet.	Diameter. Ft. In.	$\sqrt{r}$ in Feet.	Diameter. Ft. In.	$\sqrt{r}$ in Feet.
5	.323	2	.707	4 4	1.041
6	.354	2 2	.736	4 8	1.080
8	.408	2 4	.764	5	1.118
10	.456	2 6	.790	5 4	1.155
1	.500	2 8	.817	5 8	1.190
1 2	.540	2 10	.842	6	1.225
1 4	.577	3	.866	6 6	1.275
1 6	.612	3 4	.913	7	1.323
1 8	.646	3 8	.957	7 6	1.369
1 10	.677	4	1.	8	1.414

TABLE XX.

AREAS, ETC., OF CIRCULAR PIPES OF DIFFERENT DIAMETERS AND UNDER PRESSURE.

Based on D'Arcy's formula of flow through clean cast-iron pipes, in which $v = C\sqrt{r} \times \sqrt{i}$, and $Q = AC\sqrt{r} \times \sqrt{i}$.

Area in square feet $= A$; also, $C\sqrt{r}$ and $AC\sqrt{r}$.

Diam.	A	$C\sqrt{r}$	$AC\sqrt{r}$	Diam.	A	$C\sqrt{r}$	$AC\sqrt{r}$
ft. in.	sq. ft.			ft. in.	sq. ft.		
1	.005	11.61	.063	2 4	4.276	85.39	365
$1\frac{1}{2}$	.012	15.58	.191	2 8	5.585	91 51	511
2	.021	18.96	.413	3	7.068	97.17	686
3	.049	24.63	1.208	3 4	8.726	102	895
4	.087	29.37	2.563	3 8	10.559	107	1136
6	.196	37.28	7.306	4	12.566	112	1414
8	.349	43.75	15.270	4 4	14.748	117	1729
10	.545	49.45	26.952	4 8	17.104	121	2082
1	.785	54.65	42.918	5	19.635	126	2476
1 2	1.069	59.34	63.435	5 4	22.340	130	2912
1 4	1.396	63.67	88.886	5 8	25.220	134	3388
1 6	1.767	67.75	119.72	6	28.274	138	3912
1 8	2.182	71.71	156.46	6 6	33.183	144	4782
1 10	2.640	75.3	198	7	38.485	149	5757
2	3.142	78.8	247	7 6	44.179	154	6841
2 2	3.687	82.1	302	8	50.266	160	8043

TABLE XXI.

AREAS, ETC., OF CIRCULAR PIPES OF DIFFERENT DIAMETERS AND UNDER PRESSURE.

Based on D'Arcy's formula for flow of water through old cast-iron pipes, lined with deposit, in which $v = C\sqrt{r} \times \sqrt{i}$, and $Q = AC\sqrt{r} \times \sqrt{i}$.

Areas in square feet equal A; also, $C\sqrt{r}$ and $AC\sqrt{r}$.

Diam. ft.	Diam. in.	A sq. ft.	$C\sqrt{r}$	$AC\sqrt{r}$	Diam. ft.	Diam. in.	A sq. ft.	$C\sqrt{r}$	$AC\sqrt{r}$
	1	.005	7.81	.042	2	6	4.909	59.45	292
	1½	.012	10.48	.128	2	8	5.585	61.55	344
	2	.022	12.75	.278	2	10	6.305	63.49	400
	3	.049	16.56	.813	3		7.068	65.35	462
	4	.087	19.75	1.725	3	4	8.726	69	602
	6	.196	25.07	4.915	3	8	10.599	72.40	764
	8	.349	29.43	10.27	4		12.566	75.7	951
	10	.545	33.26	18.13	4	4	14.748	78.9	1163
1		.785	36.75	28.87	4	8	17.104	81.9	1400
1	2	1.069	39.91	42.67	5		19.635	84.8	1665
1	4	1.396	42.83	59.79	5	6	23.758	89.1	2116
1	6	1.767	45.57	80.53	6		28.274	93.1	2632
1	8	2.182	48.34	105.25	6	6	33.183	96.9	3216
1	10	2.640	50.66	134	7		38.485	100.6	3872
2		3.142	52.96	166	7	6	44.179	104.1	4602
2	2	3.687	55.26	204	8		50.266	107.6	5410
2	4	4.276	57.44	246					

TABLE XXII.

VALUES OF $C\sqrt{r}$ FOR VARIOUS DIAMETERS AND COEFFICIENTS OF ROUGHNESS n, FOR CIRCULAR PIPES FLOWING FULL.

Based on Flynn's modification of Kutter's formula, $v = C\sqrt{r} \times \sqrt{i}$, and $Q = AC\sqrt{r} \times \sqrt{i}$.

d ft.	d in.	$n = .011$ $C\sqrt{r}$	$n = .013$ $C\sqrt{r}$	$n = .017$ $C\sqrt{r}$	d ft.	d in.	$n = .011$ $C\sqrt{r}$	$n = .013$ $C\sqrt{r}$	$n = .017$ $C\sqrt{r}$
	6	30.9	24.6	16.98	3	4	124	103	75
	8	38.7	31	21.6	3	8	132	110	80
	10	45.8	36.9	25.8	4		140	116	86
1		52.8	42.6	30	4	4	148	123	91
1	2	59.1	47.8	33.9	4	8	155	129	95
1	4	65.2	52.9	37.6	5		163	135	100
1	6	71	57.8	41.3	5	6	173	144	108
1	8	76.8	62.6	44.9	6		183	153	114
1	10	82.1	67	48.2	6	6	193	161	121
2		87.4	71.4	51.6	7		202	169	127
2	4	97.3	79.9	57.9	7	6	211	177	133
2	8	107	87.9	64	8		220	184	139
3		116	95	70					

266. Sheet-iron and Steel Pipes. — Sheet-metal pipes were first used for conveying water under pressure for hydraulic mining in the far West, and when the people of that region turned their attention to agriculture, they immediately came into favor for conveying irrigation water. There are several varieties and makes of these pipes constructed either of iron or steel, and the prices for either are about the same. Steel is preferable to wrought iron chiefly for great pressures, since for lesser pressures its greater strength than wrought iron requires such a reduction in its thickness, if this strength is to be utilized, as would render it liable to collapse. Its surface, however, is more smooth and less liable to scale when bent. Wrought iron, on the other hand, is more rigid because of requiring greater thickness. It is therefore less liable to be dented or otherwise injured, and being more porous it takes the asphalt coating better than does steel. The plates from which these pipes are made are usually annealed, and in the case of wrought iron a tensile strength of about 45,000 pounds per square inch and in steel about 60,000 pounds per square inch is called for.

The sheet metal from which pipes are usually made comes in widths of from 3 to 4 feet, and are of varying lengths, so that when these plates are rolled into the cylindrical form of the pipe there is generally more than one seam in pipes having diameters above one foot. This, however, offers no objection; it is rather an advantage, as it adds to the rigidity of the pipe. There are many makes of pipe, the more prominent of which are lap-welded, converse lock-jointed pipe; straight, double-riveted, and spiral-riveted pipe. These are made of varying thicknesses of plate, ranging from 18 B.W. gauge down to No. 10 gauge, and even thicker. Safe working stresses, in pounds, for the various gauges of metal more usually employed are as follows:

No.	Thickness.	Pounds Square Inch.
16	.06	6,000
14	.08	7,500
12	.11	9,000
10	.14	12,000
3/16	.19	14,000

Straight-riveted pipes are double-riveted along the seams and as delivered several lengths are riveted together or lap welded, making the section as delivered from 20 to 25 feet in length. The distance apart between rivets in the rows varies from .33 to .40 inches, and the distance between any two rows is about 3/4 of an inch. Spiral-riveted pipe, as its name implies, is made by curving the plates spirally into a cylindrical form, and it is believed that this method of riveting gives a little added stiffness, owing to the manner in which the riveting and the seams are disposed around the circumference of the pipe. Laminated pipe has some advantages over other forms, in that it consists of two sheaves of sheet metal one within the other. This is made by rolling together and uniting at the edges two sheet-metal plates each of half the thickness necessary for an ordinary pipe. The inuer shell is telescoped into the outer while immersed in hot asphalt, producing a thickness between the sheets of about 1/16 of an inch, and thus making what is claimed to be an impassable barrier to corrosion.

All of these forms of sheet-metal pipe depend for their length of life on their resistance to rust and the coating of asphalt which is given them. This coating has a varying thickness of 1/6 to 1/20 of an inch, and is made as nearly impervious as possible on both the inner and outer surfaces. The composition of this coating is various proportions of asphaltum fluxed with crude oil and heated nearly to burning-point. In this hot fluid the pipes are inserted. There is a decided difference in the various makes of pipe as to the amount and character of the flux used with the asphaltum, and each maker has his own special variety and mode of application.

267. **Wooden Stave Pipes.**—There are a number of varieties of this make of pipe, among the first to find favor being that known as the Colorado wooden pipe, the invention of Mr. C. P. Allen, of Denver. There have since been put on the market various modifications of this pipe, each possessing special advantages, and among the more important of these are the Dwelle pipe, the Miller pipe, and the Excelsior wooden pipe.

The chief differences in these various forms of pipe consist in the method of binding the edges of the staves together, that is, the form of the groove or lug with which they meet and the mode of uniting or fastening the ends of the metal binding-rods. These pipes are now made of various sizes from 10 inches up to 72 inches in diameter, while even larger diameters might be used if desired.

The walls of these pipes are formed of wooden staves bound together by iron or steel rods or 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 perfect cylindrical pipe. To join the ends of the staves a thin metallic tongue is inserted in the case of the Colorado pipe, which tongue is a trifle longer than the width of the stave and cuts into the two adjoining staves. The confining bands are of round or flat iron or steel 3/8 to 3/4 inch in diameter, sometimes greater or less than this according to the pressures to be withstood and the diameter of the pipe, and are shipped from the factory as rods. In the Colorado pipe these rods are provided at one end with a square head, at the other with a thread and nut. They are bent on the ground on a bending table to the proper form, and are coated with mineral paint or asphaltum varnish and cut about 6 inches longer than the outside circumference of the pipe, on which they are slipped loose. As the construction of the pipe progresses these binding-rods are screwed up gradually a little at a time until they are brought to a uniform tension on the whole length of the pipe. These binding-rods are spaced apart various distances, depending on the pressure which the pipes have to withstand.

In the Colorado pipe the coupling or saddle in which the rod ends are fixed is of cast iron. In the Dwelle pipe the threaded portion of the binding-rod is upset and an eye formed at the other end which fits into a special casting, and instead of a metallic tongue at the stave ends a flat, V-shaped groove is used, a similar groove being also employed on the edges of the staves. The Excelsior pipe reduces considerably the weight in the coupling. The Colorado method of joining the stave ends is perhaps the best, while other pipes have their advantages in various parts.

The best materials from which to make such pipes are Oregon pine and California redwood, though native pine and Texas pine are equally satisfactory where their cost is less, providing care be taken in choosing lumber that shall have fewest knots possible. California redwood has a great advantage over any other, in that it has a great crushing strength which prevents rounded bands from penetrating the wood sufficiently to injure the fibre, and this prevents injury to the staves in case of excessive strain due to the swelling of the wood when saturated. The staves are usually prepared for the larger diameter of the pipes from carefully selected 2 by 6 inch joists, which are dressed down less than one half an inch in either direction. The distances apart of the binding-rods vary from 5 inches apart between centres under great pressures up to 12 inches under lesser pressures, and in putting these pipes together the staves can be so dressed and the binders so placed as to accommodate pipes to moderate curve both vertically and horizontally. On the Santa Ana canal one curve has a radius of 148 feet. This pipe is 52 inches in diameter and is made of staves of from 2 to 2½ inches thickness, according to the pressure, which in one place is 160 feet, where the binding bands are but 2½ inches apart between centres. The diameter of these pipes may be reduced during construction by inserting tapering staves at proper places, and the reduction can thus be made without any abrupt change of diameter, but gradually. The Dwelle pipe is now made in a new form called the coupled stave-pipe, for which certain advantages are claimed in that the interlocking ends of the staves are made to come within a short space and are covered with a coupling sleeve which is practically a short section of pipe put in by sawing the short staves into wedge forms which are driven in as the pipe is tightened.

268. Construction of Wooden Pipe Lines.—Wooden pipe lines of all kinds should be so aligned and located as to be kept well below the hydraulic grade-line in order that the pipe shall not only be kept full of water, but under pressure at all times. The object of this is that the pipe when in operation shall have the wooden shell at all times thoroughly saturated with water, otherwise it will rapidly deteriorate. Care must be taken in aligning a wooden pipe to introduce as few and as large curves as possible, owing to the difficulty of constructing these. Because of the large number of joints in wooden pipe only the best lumber and that which is well seasoned must be employed, and the greatest care must be taken in its erection so that all joints shall fit perfectly. After the staves have been dressed they must be kept under cover to avoid warping or checking. Care must be taken in tightening the binding bands, for no matter how tightly these may be fastened when the pipe is built, they will be comparatively loose within a couple of days, and water will be spurting from every seam.

Experience differs as to the desirability of burying wooden stave-pipes in the earth, and of coating the wood. Pipes which are above the surface are exposed to danger of destruction from fires. Such coatings as coal-tar and asphaltum only increase the danger of destruction from this cause, and in general this class of coating is not recommended. For the interior of flumes and of pipes asphaltum has certain preservative advantages; but the great disadvantage of such coating on either surface is that it prevents free soakage of the wood and the passage of the water from the interior to the exterior of the pipe, for on the constant saturation of the wood is largely dependent its preservation. Taken on the whole, a simple coating of whitewash is perhaps the most satisfactory which can be put on wooden pipes, and probably the only one which fills all requirements, and is superior to leaving them without covering at all. Other things being equal, it is believed that burying the pipes in the ground and leaving them uncoated will prolong their life. It is believed that wooden stave-pipes above ground, kept saturated, will last longer than the metal binding-rods underground.

The tensile strength of the binding-rods is affected (1) by the pressure in the pipes; (2) by the pressure arising from the expansion of the wood; (3) by the tensile strength of the rod itself; and (4) by the compressive strength of the wood. Mr. C. K. Bannister gives the following simple formula for determining the distance between adjacent bands.

Let d = distance in inches between two bands;

t = maximum tensile strength of each band in pounds;

p = pressure of water in lbs. per sq. in. measured at bottom of pipe;

r = internal radius of pipe;

C = coefficient to allow for strain caused by swelling of wood; also includes factor of safety in binding-rods.

Then

$$d = \frac{t}{Cpr}.$$

Practice indicates that C is generally equal t about 4 or 5 as a factor of safety.

In some cases it will be found desirable to anchor wooden pipes in order to keep them in place and to prevent their creeping. More recent experiences indicate, however, that where such pipes are carefully constructed, and more especially where their ends terminate in substantial penstocks of heavy timber work or masonry, there will be no tendency to creep, and the mode of construction of the pipe itself creates such a stiff shell of large diameter as to practically prevent any such tendency.

269. Measurement of Water in Pipes.—Where water is pumped an excellent measure of the volume discharged from pipes can be gotten by noting the capacity of the pump per stroke and the speed and time of running. Elsewhere in pipes the only satisfactory method of directly measuring their discharge is by means of some of the various patented water-meters. Measurement by computations depending on the diameter of the pipe and head of water and consequent theoretic velocity and discharge is not at all satisfactory. There are a number of water-meters on the market, nearly all of which give satisfactory results, though some of them are apt to be clogged by sediment in turbid waters and others are too heavy and cumbersome or too expensive to be satisfactorily used on irrigation work, except where water has an extremely high value and is sold by careful measure.

Perhaps the form of water-meter which is most likely to find favor among irrigators is the Venturi meter, the invention of Mr. Clements Herschel. These meters are made in all sizes, from those suitable for measuring pipes a few inches in diameter up to meters capable of utilization on pipes 6 or 8 feet in diameter. This meter consists of two pipes forming the tube, and of the register, the former of which consists of two funnel-shaped pipes of different tapers, while the latter is a delicate electrical recording apparatus with dials, etc., which record the volumes discharged. This meter is not affected by water-hammer, dirt, sticks, or other substances. The principle on which it works is that of measuring the differences of pressure due to friction in passing through the throat of the pipe, as it is called—that is, the narrow portion where the two inverted funnels meet; for it is well known that the pressure is less at the throat or contraction than at the upstream end. A peculiar feature of this instrument is that this difference in pressure does not produce any appreciable loss of head, as is the case with other meters; and the final result of the principle on which it depends for its action is summed up in the rule that "the faster the flow of the water through the Venturi tube the greater is the difference in pressure at the upstream end and at the throat of the tube; and the slower the flow the less the difference in pressure." Upon this principle the action of the meter is based, and by utilizing these differences of pressure the amount of flow is shown by the register.

Except for the delicate registering apparatus this instrument is not necessarily expensive. It consists practically of a couple of cast-iron pipes moulded to fixed shapes. Cheaper forms of Venturi meters are being constructed for use in measuring sewage of towns, and still cheaper forms are being designed in brick and cement, and even in wood, for measuring the discharges in open as well as closed irrigating channels.

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PART III.

STORAGE RESERVOIRS.

CHAPTER XV.

LOCATION AND CAPACITY OF RESERVOIRS.

271. **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.

272. **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.

273. 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.

274. **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.

275. 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. 81, is the most favorable.

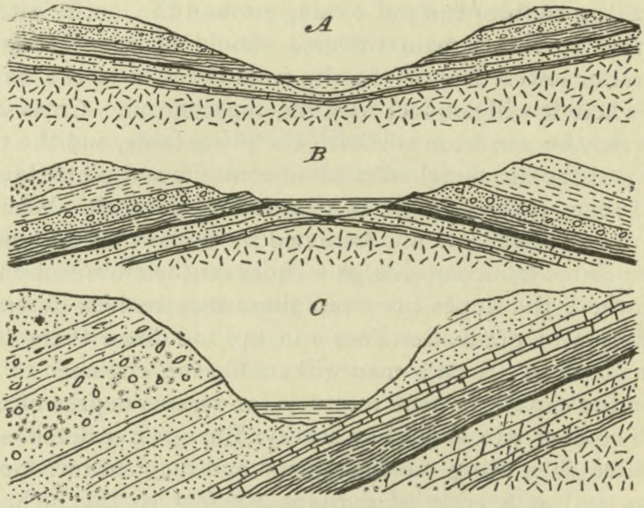


FIG. 81.—DIAGRAMS ILLUSTRATING GEOLOGY OF RESERVOIR SITE.

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. 81, *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. 81, 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 XXIII.

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	$19.11
Bear Valley	"	"	40,550	64	300	5.30
Hemet Valley	"	"	10,500	122	250	.....
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
Monument Creek	Colorado	"	860	40	855	29.10
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
Lake McMillan	New Mexico	Loose rock with earth backing	138,000	52	1,686	1.27
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 of the 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.

276. **Cost and Dimensions of some Great Storage Reservoirs.** — In Table XXIII 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 XVI.

EARTH AND LOOSE-ROCK DAMS.

277. **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.

278. Dimensions of Earth Dams.—An earth dam may fail (1) from lack of stability of cross-section; (2) from disintegration by erosion of the material composing it. It may fail from lack of stability 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 unsatisfactory 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 erosive action of water. Failures from this cause are usually due (1) to insufficient wasteway; (2) to the mode of drawing off water through pipes or tunnels; (3) to carelessness in construction.

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. XXVIII), 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.

279. **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 unless a masonry core-wall is used, when a firm bonding to prevent creep of water can be gotten between this and solid rock up to the level of the crest of the dam. 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, or concrete in cement.

280. Foundations of Masonry Core and Puddle Walls. —The foundations for a masonry core-wall should always rest only on firm, homogeneous rock, as described for foundations of masonry dams (Art. 313). In any case, where only loose material or disintegrated rock is to be found, some other form of earth dam than one having a masonry core is to be recommended. The core-wall should have a perfect bond with the clean bed-rock, both under the dam and up the sides of the hill to a level with its crest; otherwise it is always a menace to the integrity of the structure, and may afford the means for creep of seepage water which will eventually destroy the structure.

Where a puddle wall of clay or earthy gravel is employed instead of the masonry core, equal precautions with regard to its foundation are as necessary as for that of a masonry core wall, though in the case of a puddle wall it is not necessary or desirable to found it on rock. The best foundation for a puddle wall is fine, loose material, as gravel or sand containing earthy material; a firm bed of clay, or a close-grained hardpan. This should be cleaned off and trenched so as to insure a firm seat and close bond for the puddle wall.

281. **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.

One excellent way of dealing with a foundation containing a number of small springs is to begin construction from the inner toe and work progressively across the base of the dam to the outer toe, first stripping and preparing the foundation of the inner toe and carrying the construction from the centre wall outward, toward this toe, in such manner as to force the springs out from under the foundation, or if possible, to smother them. Where a cement puddle or core wall is used, in the foundation of which a number of large springs are encountered, there is often trouble in closing the foundation over these. One method of doing this is to carry up the wall, leaving a little hole or tube through which the spring may issue, and to coax this upward as the wall progresses until a point is reached above which the spring does not rise. Or the diameter of the tube may be diminished until it becomes too narrow for the passage of the water owing to its diminished head and increased friction. Perhaps the most successful way of dealing with such springs in foundations is to bottle them. This consists of puddling or filling cement about the springs until they are confined to a comparatively narrow outlet orifice, in which an iron pipe may be cemented. This pipe is then led off to the outer side of the embankment or is carried up in construction until the spring ceases to rise, and it is then capped so as to effectively bottle the water.

282. **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, especially for very high dams and those with which other masonry structures combine, as masonry wasteways or extensions of the dam, for then a safe bond can be made between the core-wall and its adjoining masonry work. Magnificent examples of such works have recently been built for the Boston water supply and the New York water supply. 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 dry 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 an impervious face of great thickness.

283. **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. 103). 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

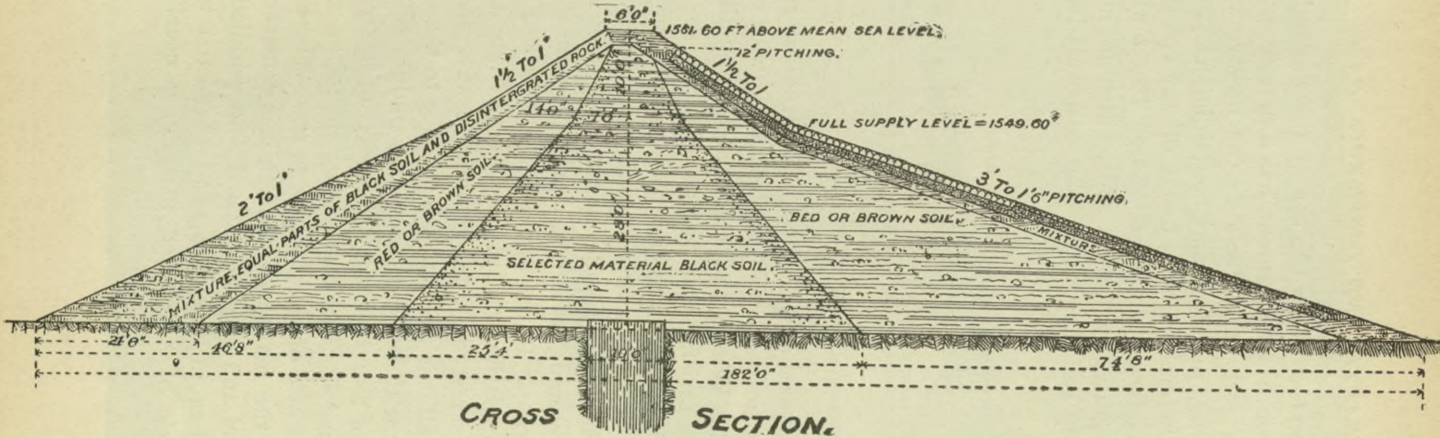


PLATE XXVIII.—CROSS-SECTION OF ASHTI DAM, INDIA.

top, which is 20 feet below the top of the embankment. The Boston Waterworks Department has a practice of backing the core-wall by a sort of buttress on the up-stream side of fine selected material, puddled, and containing clayey matter, the remainder of the embankment being of any coarse and heavy material, especially on the lower side. The outer slopes are made of loose gravel to prevent slips, which are more likely to occur in clayey soil (Fig. 82). The core-wall has occasional

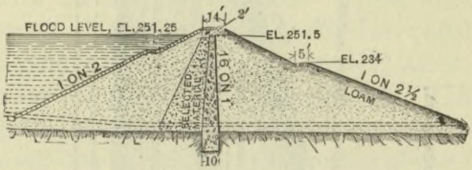


FIG. 82.—CORE-WALL AND EARTH EMBANKMENT, BOSTON WATERWORKS.

buttresses of masonry to prevent longitudinal creep of water and the up-stream surface is well plastered with cement.

284. **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 impossible to obtain in the arid region. It is difficult to prepare, and requires careful manipulation in placing it. 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

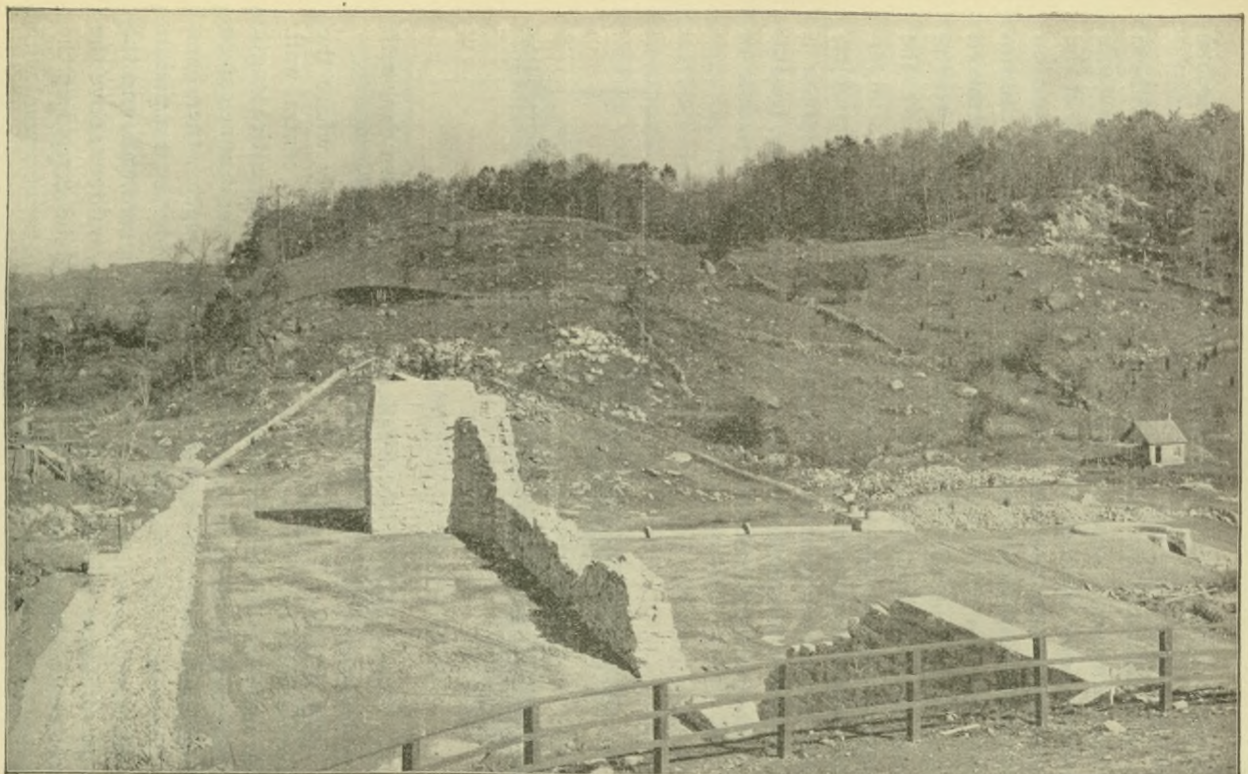


Plate XXIX.—Earth Dam and Masonry Core-wall during Construction, Carmel, N. Y.

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, which are the greatest menace to any structure depending for its imperviousness on a puddle wall or face. One of the serious objections to a puddle face is its liability to slip if the reservoir is drawn down so quickly as not to give it time to dry, for this is a slow process in so close-grained a material as a puddle of

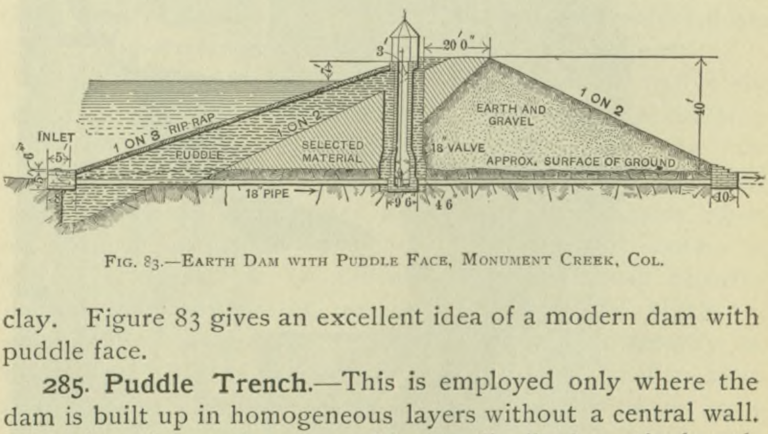


FIG. 83.—EARTH DAM WITH PUDDLE FACE, MONUMENT CREEK, COL.

clay. Figure 83 gives an excellent idea of a modern dam with puddle face.

285. 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 XXVIII, 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. The use

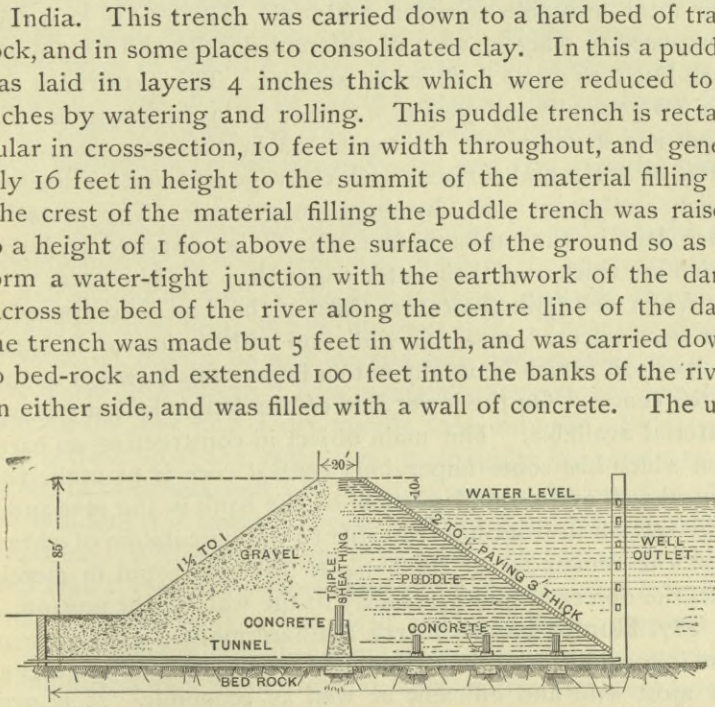


FIG. 84.—CROSS-SECTION OF EARTH DAM, SANTA FÉ, SHOWING MASONRY CROSS TRENCHES.

of a puddle trench is open to the same general objections as a puddle or core wall, but in a less degree. One of its great advantages where filled with masonry is that it furnishes an excellent bond for the outlet pipes and culvert, and as a barrier against creep of water along these. About the only good example of this form of construction is to be found in the Santa Fé dam (Fig. 84), in the bottom of which are four parallel masonry puddle trenches. This dam was one of the first in which goats were used to do the tramping. The material was levelled by a drag in 3-inch layers.

286. Construction of Embankment.—As frequently 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 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 coarser 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.

287. Homogeneous Earth Embankment.—This type of dam is considered by the writer and by 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 dry puddle face, only the whole structure is built up at the same time in irregular layers of 6 to 15 inches in thickness, and well tramped over or dry 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. Excellent results in compacting such dams has recently been gotten on the

Santa Fé dam (Fig. 84) by keeping a band of goats tramping constantly back and forth over the surface during the entire period of construction, 100 goats doing the work for about 20 wheel scrapers.

288. 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.

Recent experiments by Prof. E. W. Hilgard with alkali soils in California (Art. 44) shows that black alkali—carbonate of soda—puddles clay soils so as to make them impervious and untillable, and sandy soils so as to produce a tough hardpan through which water cannot pass. Puddle walls in dams watered with a one-tenth per cent solution of carbonate of soda will be rendered water-tight, and so tough and solid as to resist pressure and erosion to a remarkable degree.

289. **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. XXVIII). 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. XXVIII). 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.

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

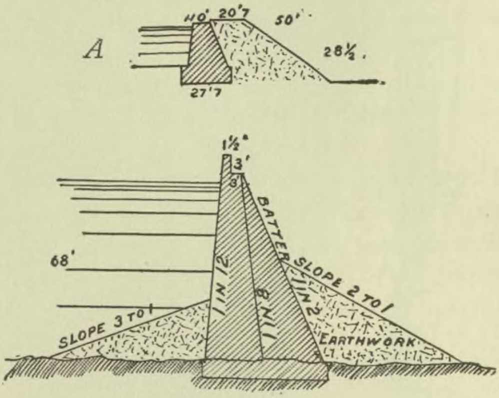


FIG. 85.—CROSS-SECTIONS OF KABRA DAM (*A*) AND EKRUK DAM (*B*), INDIA.

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. 85, *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.

The masonry may be put in as in the case of the Ekruk tank in India (Fig. 85, *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

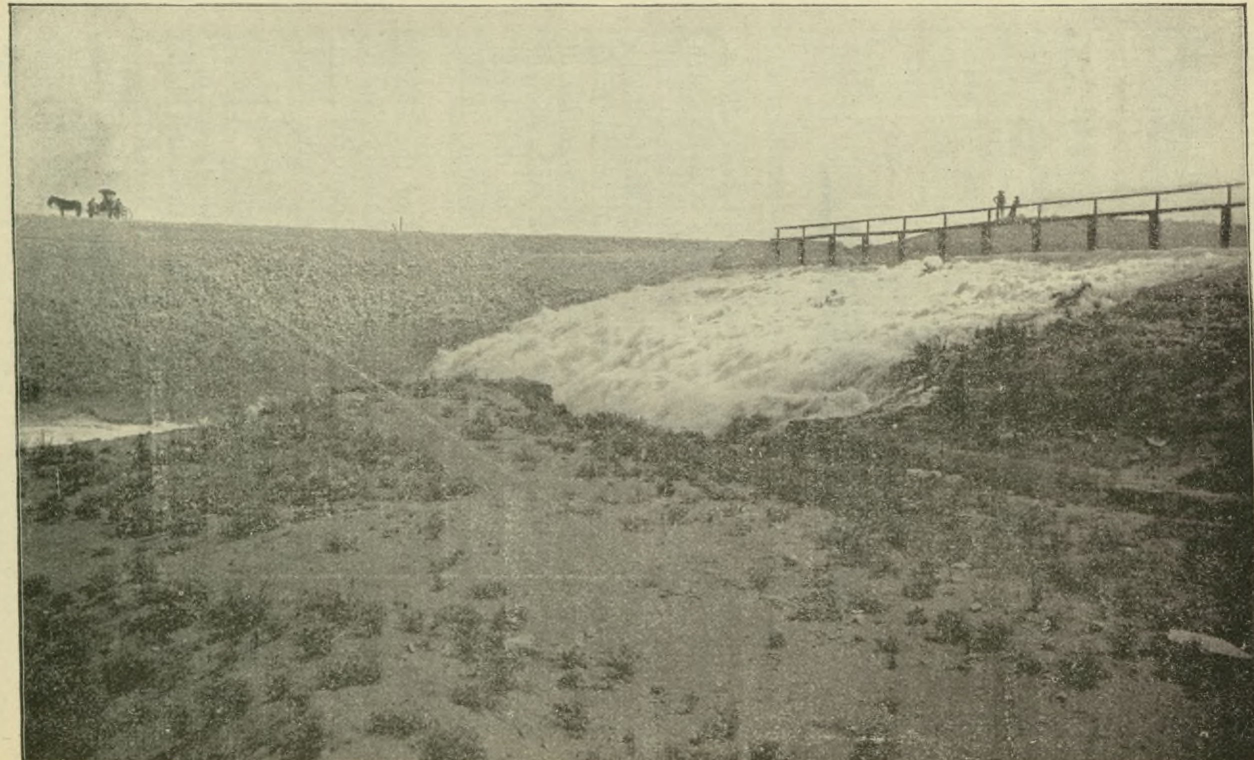


PLATE XXX.—VIEW OF PECOS DAM, UNDERSLUICE AND WASTEWAY.

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.

291. Earth and Loose-rock Dams.—Pecos Dams.—The dam at the head of the Pecos Irrigation Company's canal, at Eddy, New Mexico (Pl. XXX), 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

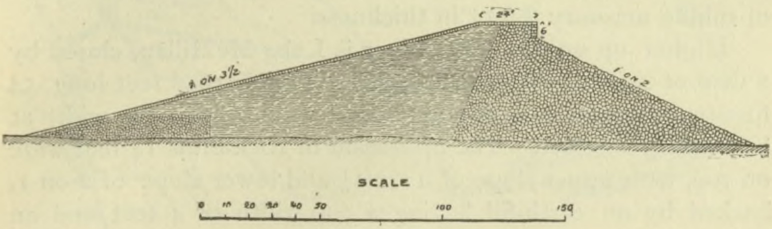


FIG. 86.—CROSS-SECTION OF PECOS DAM.

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. 86) 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.

Higher up on the Pecos river is Lake McMillan, closed by a dam of similar design, but larger. This is 1686 feet long, 54 feet in maximum height, and 20 feet wide on top, the width at base being 290 feet. The up-stream or rock-fill is 14 feet wide on top, with upper slope of 1 on $1\frac{1}{2}$ and lower slope of 2 on 1, backed by an earth-fill having a top width of 4 feet and an outer or lower slope of 1 on $3\frac{1}{2}$.

292. **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. 87). 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 proposed 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. 88) is to be constructed of loose rock with an earth facing. It will be 220 feet in length on its crest and 43 feet in maximum height, its top width being 10 feet,

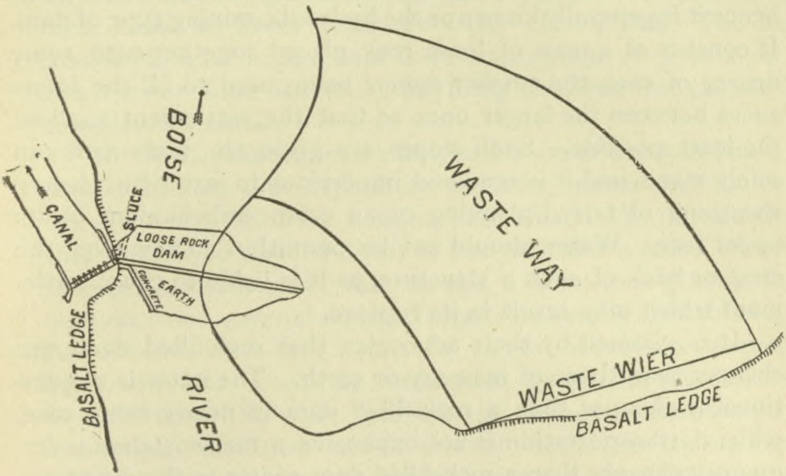


FIG. 87.—PLAN OF IDAHO DAM.

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

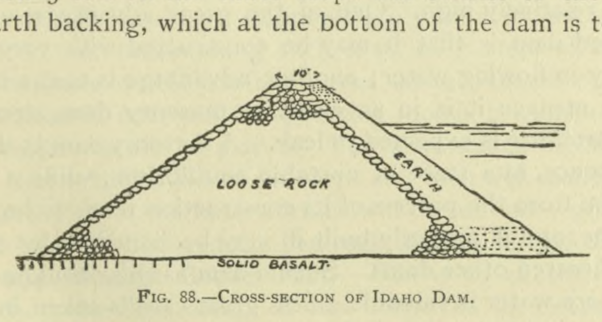


FIG. 88.—CROSS-SECTION OF IDAHO DAM.

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.

293. **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 3 to 2 on 1 and a lower slope of 1 on 1; anything less than this cannot be considered secure.

Mr. R. B. Stanton, as a result of his experiences in lining a composite dam with asphaltum concrete, makes the following excellent suggestion for building such a dam. He recommends that the site be cleaned down to bed-rock and levelled off with proper toe catches, and that on this a loose-rock gravity dam be built from material dumped in place carefully by cableways. As for other loose-rock dams, the largest stones up to several hundredweight should be surrounded by smaller pieces so as to make the whole mass compact and reduce settlement to a minimum. The inner face of this dam should be laid up carefully on a suitable slope from a thickness of several feet at the bottom, the materials being laid dry by hand and the joints being well filled with spalls. This upper slope, starting from the bottom, should be stepped back 3 or 4 inches for every 5 or 6 feet of rise, making such a series of steps all the way to the top, and on this surface a true asphalt concrete such as is described in Art. 320, 6 inches to 1 foot in thickness according to height, should be placed. The advantages of this system are that with the aid of the steps and with well-made materials no creeping of the inner surface will occur, and a perfect joint will be made between the bed-rock, side walls, and dam.

294. 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. 89) 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.

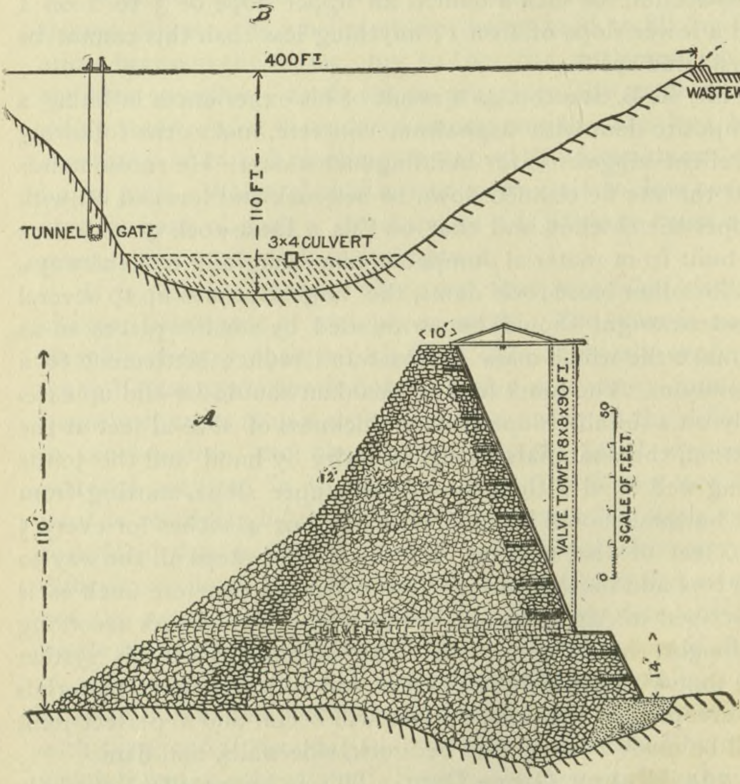


FIG. 89.—ELEVATION AND CROSS-SECTION OF WALNUT GROVE DAM.

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 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.

295. Crib Dams.—The general form of construction and several examples of crib weirs were given in Articles 165 and 166. Structures of similar design have occasionally been built of sufficient height to form storage reservoirs. The employment 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 cribwork 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.

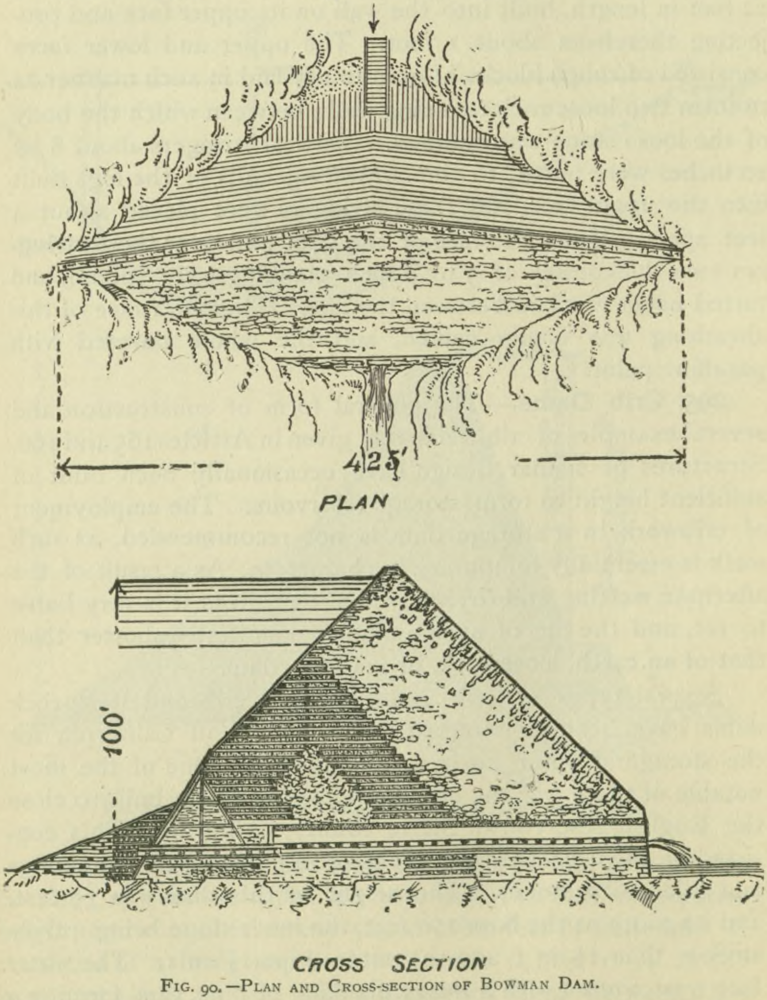


FIG. 90.—PLAN AND CROSS-SECTION OF BOWMAN DAM.

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. 90) 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.

296. 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. 91) 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

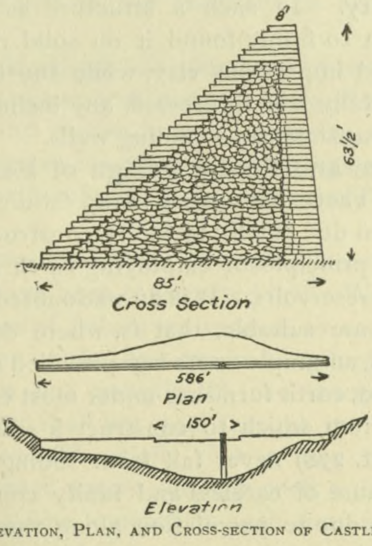


FIG. 91.—ELEVATION, PLAN, AND CROSS-SECTION OF CASTLEWOOD DAM.

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 1½ to 2½ 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 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.

297. Failure and Faulty Design of Earth and Loose-rock Dams.—There have been many failures of earth dams, which have been due chiefly to faulty construction and design, and not to the principle of employing earth as a material for closing storage reservoirs. It is an undoubted fact that where the conditions are suitable, that is, where desirable material can be obtained, an ample wasteway provided and proper foundations procured, earth furnishes under most circumstances the best material from which to construct a safe reservoir dam. Such dams (Art. 278) never fail from sliding or overturning, but always because of careless and faulty construction acting through erosion due to percolation along the outlet pipes or foundation, or along some structure within the dam, or by erosion from overtopping.

There are some notable instances of large earth dams built on leaky ground which have remained intact for many years. This is notably the case with some great earthen dams built in California for the water supply of San Francisco which have been in constant use for over twenty-five years. The ground at the site of one of these was full of water-bearing gravel beds and a puddle trench carried down 98 feet below the bed of the valley failed to meet an impervious strata. Accordingly the height of the dam was reduced to 50 feet above the bed of the valley, and when the reservoir was completed springs appeared and have continued to flow harmlessly for many years. The San Leandro dam at Oakland, California, is 120 feet high above the valley bed, and rests on leaky ground. This dam is twenty years old, and has leaked more or less around both edges, along the hillsides as well as from the bed below. The same is the case with a small old earth dam built for the water supply of Los Angeles, California, from below which a spring of water is constantly issuing. It is evident that the upward pressure beneath these structures is not sufficient to overturn or move them.

Earth dams are among the oldest structures in the world. There are many thousands of them in India, Europe, America, and elsewhere, and in the former countries some of these have been in existence for many centuries and are still as safe and substantial as when built; in fact, they have improved with age.

Loose-rock dams are yet to a certain extent experimental, and while many have been constructed in the West, many have failed. The causes of failure are various, as are the modes of construction. One of the first causes of failure is an unstable foundation. It is not essential to the integrity of such a structure that water shall not pass through it. It is desirable to have it water-proof, chiefly that it may hold water for storage; but if water passes through such a dam, as it may, the structure should be founded on such firm and unerodable material, and should itself have such natural slopes, that the passage of water shall cause no erosion within it or about it, and therefore no settlement. Another cause of failure is to be found in such steep slopes that the loose rock will not maintain them or that waste water overtopping the structure will cause a tendency to assume natural slopes which will reduce the height of the dam and thus lead to its destruction. Unless a loose-rock dam be given exceedingly low slopes which are not likely to be changed by violent water action, as ample a wasteway must be provided as for an earth or masonry dam. Without further knowledge than is now possessed as to the causes of failure in such structures and their proper design, every precaution should be taken in their construction to make them as safe against settlement by carefully building them up in horizontal layers, as secure against erosive action of water through and around them, and as free from the danger of overtopping by supplying ample wasteway, as is desirable in the construction of an earthen or masonry dam.

CHAPTER XVII.

MASONRY DAMS.

298. 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 309 and 310. 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.

299. 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, Art. 298, 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. 92.

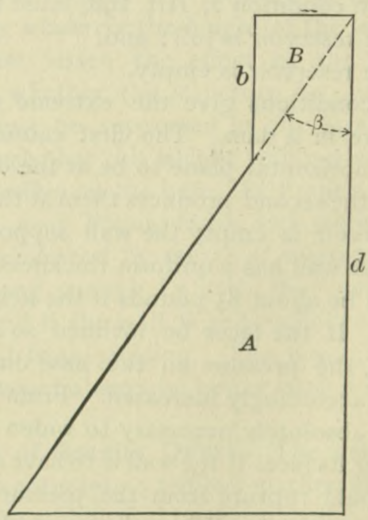


FIG. 92.—THEORETICAL TRIANGULAR CROSS-SECTION OF DAM.

300. 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

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

TABLE XXIV.

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 $c = \frac{x}{w}$.

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.

302. **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 most dams 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:

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

303. 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.

304. 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. 93 suppose the lines of reaction R and

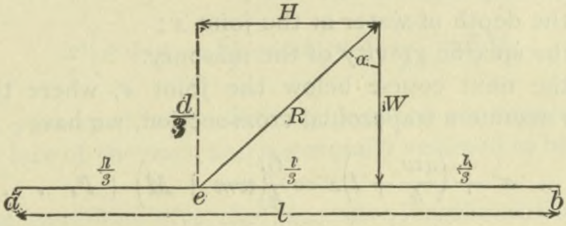


FIG. 93.—DIAGRAM ILLUSTRATING WEGMANN'S FORMULA.

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;
$M = \frac{d^3}{6r} =$ 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

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

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

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

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.

305. 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. 94.

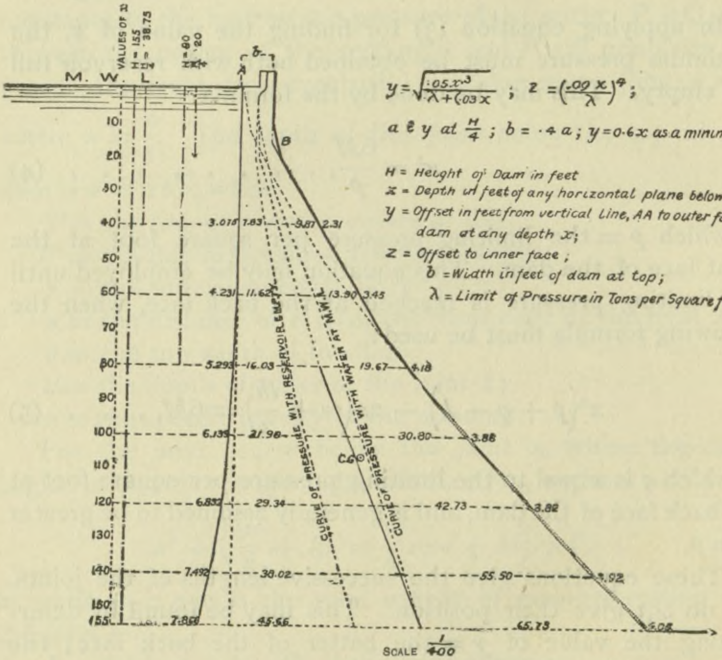


FIG. 94.—MOLESWORTH'S PROFILE TYPE.

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

This formula gives a dam of excellent cross-section nearly approaching that gotten by Wegmann's and others, and one in which the resultants of pressures, reservoir full and empty, lie well within the middle third. The computations by this formula are simple, and for that reason it is given here.

$y =$ 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;

$\lambda =$ 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 $= \frac{a}{2}$.

306. 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 289. 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.

307. **Profile of Dam.**—In Fig. 95 is given a comparison of the profiles obtained by several of the more common formulas,

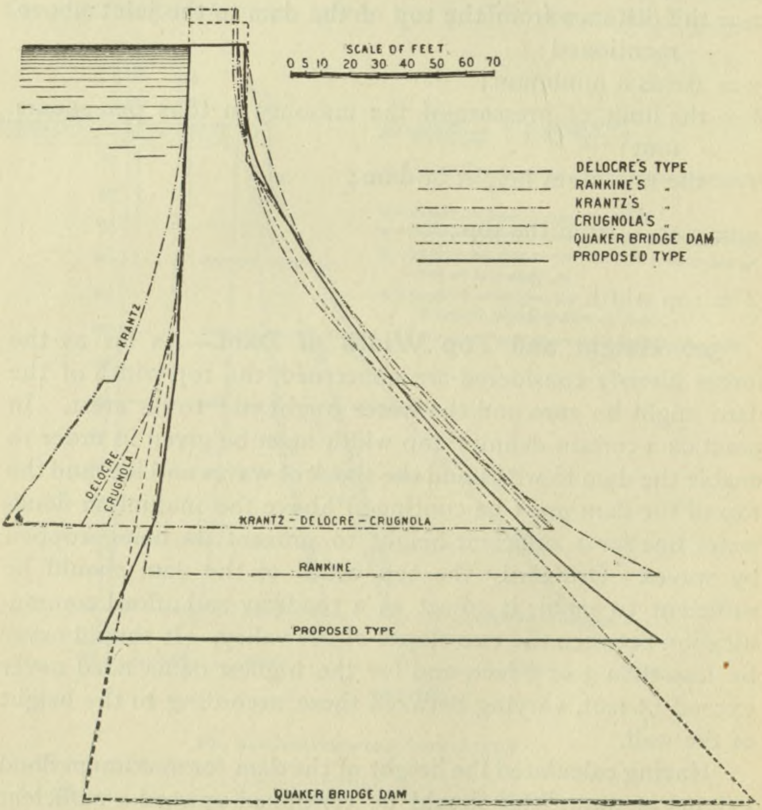


FIG. 95.—COMPARISON OF PROFILE TYPES.

while that which is shown in full lines is the practical profile type No. 3, adopted by Wegmann. 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 $2\frac{1}{3}$.

TABLE XXV.

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.			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.				Coefficient of Friction necessary for Equilibrium.	Factor of Safety against Overturning.
									Reservoir full.		Reservoir empty.			
			Left in feet.	Right in feet.	Total in feet.				In feet of Masonry.	In tons of 2000 lbs.	In feet of Masonry.	In tons of 2000 lbs.		
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 $2\frac{1}{4}$.

308. Stability against Upward Water Pressure; also Causes of Failure.—The question of the effect of springs under foundations of masonry dams is still an open one, and has led to much discussion among the more experienced builders of such structures. A cross section which would enable a dam to resist upward water-pressure should be much heavier than one called for by the usually accepted theories which disregard such pressure. It would, in fact, be nearly twice as heavy, and therefore call for about twice as much masonry in the structure as would the theoretical cross-section. All the evidence appears to be against danger of rupture from such causes. There have been constructed and are still standing many masonry dams designed on cross-sections too light to withstand theoretical pressures from below, and of all these structures but three of any moment have failed, namely, the Puentes, Habra, and Bouzey dams; and the causes of failure of each of these has been due to faulty construction rather than to errors in not designing an ample profile for withstanding upward pressure. In a recent discussion of this subject before the American Society of Civil Engineers Mr. Wegmann enumerates the causes of failure in these three structures as follows:

The Puentes dam, built in Spain a century ago, was 164 feet high. During construction a deep pocket of earth was discovered under the foundation, and instead of going down to solid rock a pile foundation was employed in this place. This was forced outward by water-pressure and caused a rupture in the cement portion of the dam. The Habra dam, constructed twenty-five years ago, failed through the poor material used in its construction. This consisted of porous sandstone not of uniform character, while the sand used in the cement was of poor quality; the hydraulic lime was made of calcareous stone of imperfect quality containing quicklime, which possibly expanded, and thus assisted in making the structure porous. The dam was not water-tight, for water flowed through it like a sieve, until finally a severe rain-storm caused the structure to be overtopped by a flood 13 feet in height above the crest, which resulted in its destruction. The Bouzey dam was completed ten years ago and was founded on porous conglomerate rock, consisting of siliceous stones joined together by poor cement material. The foundation was not carried down to solid rock, and to prevent leakage under the dam a guard-wall $6\frac{1}{2}$ feet thick was carried below the main structure for a depth of 20 or 30 feet to solid rock. As the maximum height of water in the reservoir was to be 75 feet, this guard-wall could scarcely be expected to prevent water from percolating under the dam, and thus cause sufficient upward pressure to injure its stability by sliding. The structure was finally ruptured, not by settling or overturning, but by sliding or bulging forward, which produced numerous fissures in the foundation and four vertical fissures in the wall. In this latter case upward pressure under the base caused the disaster, but this doubtless would not have occurred had the whole dam been founded on solid rock, as should have been the case.

In the construction of most masonry dams fissures are encountered in the foundation rock from which water issues or is expected to issue under pressure. These should invariably be carefully examined, cleaned out, and followed down to such a depth as to reach homogeneous rock, or, after being cleaned out for a considerable depth, should be gradually narrowed up with cement so as to coax leakage water to a small orifice, which may terminate in a tube and be led out of the structure or be capped and bottled.

309. **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 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 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 sus-

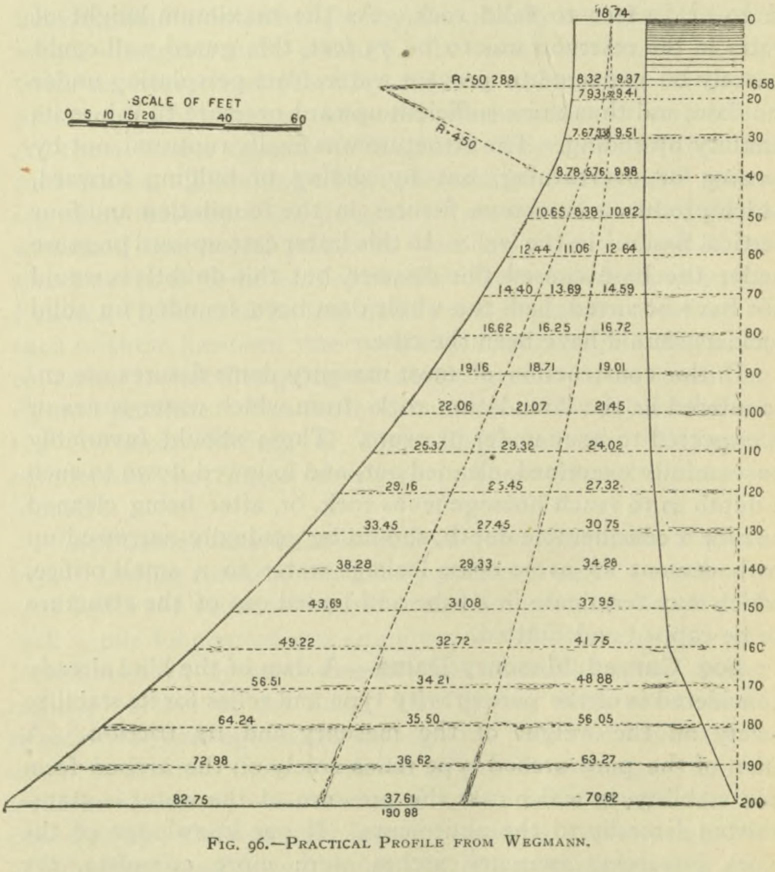


FIG. 96.—PRACTICAL PROFILE FROM WEGMANN.

tained 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. 333 and 339).

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.

310. 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 may 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. XXXIV), 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.

311. Wide-crested or Overfall Dams.—Gravity dams, (Art. 299) are designed with cross-sections sufficiently ample to enable them to resist by their weight the hydrostatic forces which tend to overturn or slide them. Curved dams (Art. 309) are designed to resist the same forces, but their weight is aided in that effort by curving them in plan in such manner as to add to their stability by bringing into play arch action. They are sometimes accordingly diminished in cross-section from gravity requirements. Overfall dams have to resist, in addition to the hydrostatic forces tending to shear or overturn them, the dynamic and erosive action due to having large volumes of water pour over them, often from great heights. Their cross-sections must therefore be increased over those called for by gravity requirements by an amount sufficient to enable them to resist the pounding and erosive action of the falling water. This result is usually aided by diminishing these forces to some extent by giving their lower slopes such rollerway curves (Art. 171) as will cause the water to slide or roll down them rather than fall and be broken up on them; also, by diminishing the effective height of overfall by water-cushions, as is done for weirs (Art. 172).

An overfall dam therefore has to withstand greater destructive forces than any other type, and should accordingly be con-

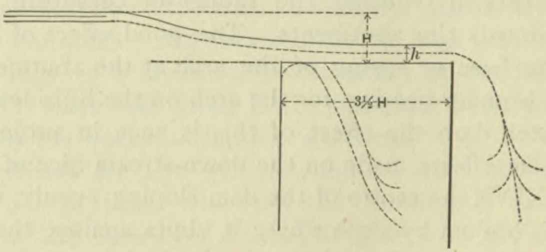


FIG. 97.—FLOW OVER WIDE-CRESTED DAM.

structed of even better materials, with greater care, and be if possible more firmly founded than a simple gravity dam of corresponding height. The cross-sections of existing overfall dams exhibit on the one hand the conservatively massive English type illustrated by the Vyrnwy dam (Fig. 108), and on the other the bold Western type illustrated by the Turlock dam (Fig. 110), which is scarcely heavier than gravity alone would require. In both of these examples well-designed rollerway curves and deep water-cushions greatly reduce the erosive effect of the falling water. Between these extremes the Folsom dam (Plate

XXXVIII) is amply heavy and has a good lower curve, but too wide and sharp a crest to produce the best results. The Betwa dam (Fig. 109), while having a cross-section at variance with the theoretic demands upon it, is accordingly made very heavy, with a massive buttress below to reinforce it. The form of overfall given the new Croton weir (Fig. 105) is more nearly that required by theory but the advisability of breaking its lower face up into steps is doubtful, for this prevents its roller-way curve from causing the water to slide gently down it, while it induces a strong erosive action on the face of the dam, though it reduces the action at the toe. Probably the best cross-section yet given an overfall dam is that given the Colorado river dam (Fig. 111), the curve of the lower face of which is such as to best aid in producing a sliding motion in the falling water, while its crest width is neither too great nor too little.

312. Design of Overfall Dams.—As yet theory has made little advance in solving the proper cross-section for overfall dams. We believe that they should be somewhat more massive than gravity sections, and that their lower slopes should be so modified as either to cause the water to slide, as in the Colorado river dam, or fall clear of the base, as in the Betwa dam. We know that the provision of a water-cushion tends materially to reduce the erosive and dynamic action of the falling water by reducing the height of fall. On the other hand, the form of the crest and of the lower slope are still points of difference among hydraulic engineers. There are those who favor a very wide crest with a sharp lower edge as giving the proper direction to a body of falling water; there are others who favor the curved crest and lower face; and still others who advocate the stepped outer slope.

Again, instances are not lacking in which an unprecedented flood has proven too great for the wasteway of a simple gravity dam, and large volumes of water have passed harmlessly over its crest. This has happened during construction to the Bhatgur, Tansa, and, in our own country, to the San Mateo dams; while since completion the more frail Sweetwater curved dam, and lightest of all the Bear valley dam, have been harmlessly topped by flood-waters. To assume from this, however, that such sections are sufficiently substantial for overfall dams would be unsafe, and no careful engineer will intentionally tempt fate as yet by adopting them for overfall dams. Again, each case will require special treatment and its own dimensions, according as the dam is to be topped by a few feet of water as in the Croton overfall weirs, or by a flood which banks up 30 feet above the crest as with the Folsom dam.

One of the simplest problems connected with the design of wide-crested overfall dams is the determination of the velocity and discharge for a given depth of water passing over the crest, yet the theoretic determination of these quantities is one of the most difficult problems of solution, since it is affected by the curves at both the upper and lower edges of the crest as well as by the crest width. The crest velocity may be accepted as being due to the differences in head over the dam crest between still water above the dam and high-water surface over the crest. Thus (Fig. 97), v is dependent on $H - h$, and may be expressed

$$v = \sqrt{2g(H-h)}, \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (1)$$

and for discharge per unit of length in second-feet

$$Q = Bh\sqrt{2g(H-h)}. \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (2)$$

$Q = 0$ when $h = 0$ or $h = H$. In formula (2) the discharge Q is a maximum when $H = 3/2h$; therefore, for maximum discharge,

$$Q = .385BH\sqrt{2gH}, \quad . \quad . \quad . \quad . \quad . \quad . \quad (3)$$

which agrees very closely with the results obtained from experiment.

A rounded inner edge to the crest prevents crest contractions and increases the discharge greatly in proportion to the degree of rounding. The same effect is produced by curving the outer edge, the tendency of which is to diminish the width of the dam crest and to make it conform more nearly to the discharge which would be gotten from a knife-edge weir. Width of crest and degree of rounding will affect accordingly the velocity and discharge, and will determine how far out the column of water will fall for a given height over the weir crest. Thus a very sharp crest well rounded on the outer face will cause the water to flow or slide down this face and alight upon it well within the toe; a broader crest less curved will throw the water further out and make it alight nearer the toe; while a very wide crest with sharp outer edge may throw the water out entirely clear of the dam. Francis' formula of flow for a sharp-edged weir crest (Art. 89), that is, one in which B equals zero, gives

$$Q = 3.90H\sqrt{H}. \quad . \quad . \quad . \quad . \quad . \quad . \quad (4)$$

If the crest is very wide, and there be practically no curve at either edge, the stream assumes a rectilinear direction, so that the formula for maximum flow in a canal applies, or

$$Q = 2.92H\sqrt{H} \quad . \quad . \quad . \quad . \quad . \quad . \quad (5)$$

which will not apply for crest widths less than $3.5H$. Accordingly, for intermediate widths, the constants in formulas (4) and (5) will have value intermediate between those there given. (See Art. 342.)

313. **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, how-

ever, 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.

314. Preparing Foundation.—After the foundation of a masonry dam has been excavated down to a solid or homogeneous rock the greatest care should be taken in properly cleaning it and roughing its surface in order to make a most perfect bond with the masonry superstructure. Perhaps the best way of describing the care to be taken in these particulars is by rehearsing the account given by Mr. Walter McCulloh, detailing the excavation for the foundation for the Sodom dam of the Croton water supply.

The rock of the foundation was rotten, disintegrated, and shaly for a depth of from 4 to 15 feet. In preparing the foundation, drilling was done by steam and hand, and light charges of 40 to 60 per cent dynamite used in blasting until the rock appeared firm. Then all seams and fissures were followed up with block-hole and black-powder blasting, and by barring out until a solid and practically tight bottom was secured. The foundation thus prepared was swept clean with wire stable brooms and washed thoroughly with streams from hose-pipes. In the process of washing it may be stated that streams under high pressure are desirable in order to remove every particle of loose material, and the use of hot water or steam is found to facilitate the cleansing of the foundation.

When the bottom of the foundation for the Sodom dam was

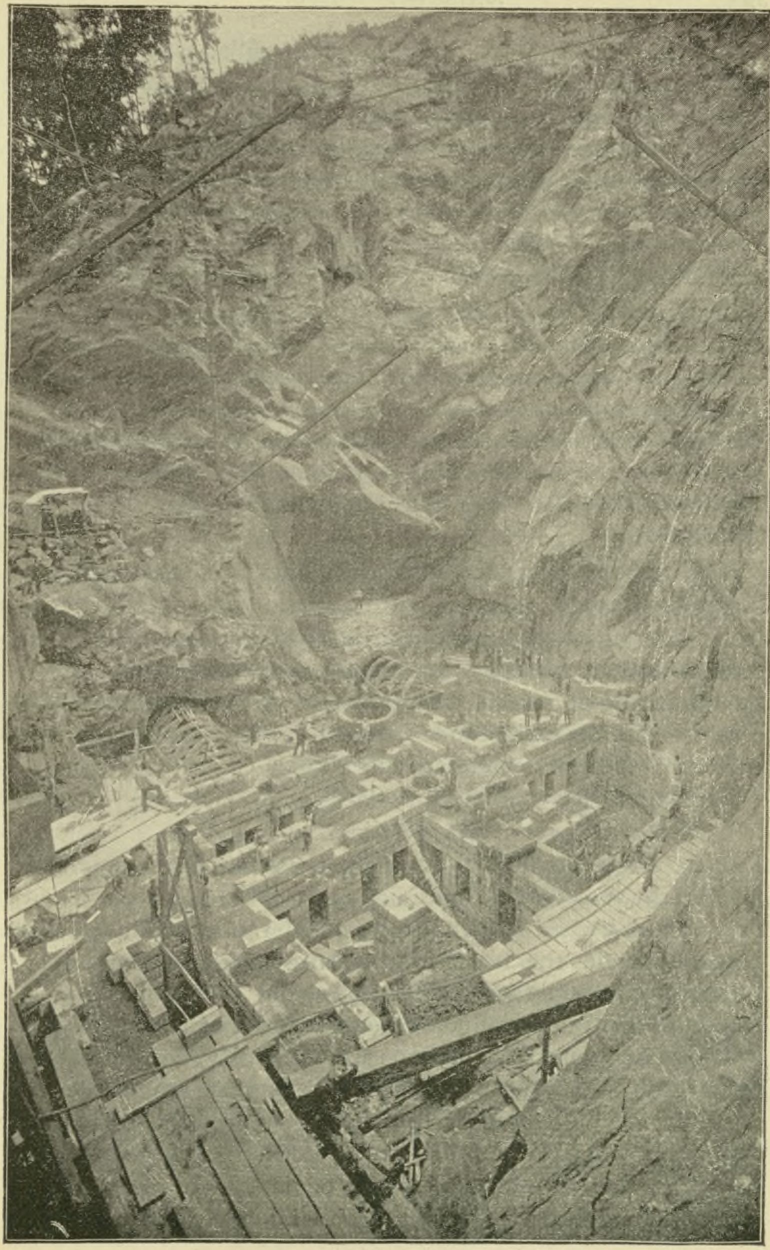


Plate XXXI.—Excavating Foundation for New Croton Dam and Gate-house.

ready, all pockets, holes, and seams were filled with rich Portland cement concrete, forming a series of level beds on which to build up the rubble superstructure. The use of concrete beds was discontinued later, as it was found that a tighter bed could be formed with rubble and small stones. A large amount of water made its way through the loose rock above the bottom and in some cases through seams in the bottom itself, but generally where the rock appeared solid the seams were not followed any deeper. Springs washed the mortar out of the concrete, but in making the rubble beds the water was led around and prevented from doing damage, by forcing the streams from place to place, until finally a small well 2 feet in diameter and from 1 to 2 feet deep would be formed about the place where the water boiled up. When the mortar about each well was thoroughly set, it was bailed out and quickly filled with dry mortar, and on top of this a large stone would be placed, and the spring was effectually bottled. This same process was followed over the entire bottom wherever water had to be contended with, and after the first 6 feet of rubble foundation was laid, no difficulty was experienced.

Pl. XXXI illustrates admirably the great depth to which foundations are sometimes carried. This shows the foundation of the gate chambers of the new Croton Aqueduct at Cornell's, which will extend in places to a depth as great as 80 feet below the level of the river bed.

315. Material of which Constructed.—Reservoir dams may be built of ashlar masonry; of rubble or concrete with or without 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 328 and 335, and the new Croton dam in New York (Art. 329). 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 it is not generally approved in this country.

316. 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 Beetaloo dams in Australia, respectively 60 and 110 feet in height (Articles 330, 331). The Periar and Beetaloo 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. XXXIII) weigh about 300 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.

317. 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.

318. 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.

319. **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 mortar, 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 $2\frac{1}{2}$ 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.

After cement masonry has been allowed to rest until it has had time to dry and harden it must be gone over with sharp picks or chisel-edged tools to remove the scale, roughen the surface, and make it clean and fresh in order that the new mortar may adhere to it. This is especially true of new work laid on old which has set for several months. Masonry should not be built in winter during freezing weather unless exceptional precautions are taken to cover and protect it from frost. Recent experiments with masonry built in freezing weather on Sodom dam showed that the effect of freezing was not as serious as had been anticipated, though no mortar was set in temperatures less than $+20°$ F., and hot brine made of five pounds of salt to one pound of water was used for mixing. This and the fact that the sand and stones were heated is believed to have helped the quality of this cement. In addition, salt was scattered over the new work at night, and at times a layer of sand was spread over to protect it. Masonry laid under these circumstances showed in the spring but slight damage to the surface. A thin scale not exceeding one eighth of an inch in thickness was left, which was easily scraped off, and under this the mortar was in good condition.

Many mechanical devices are employed both for the mixing of cement mortar and the conveying of mortar and stones to the work, and for laying the same. Descriptions of these, however, are to be found in special works on the subject. The chief object of mechanical mixers is to thoroughly incorporate the dry materials so as to bring them into intimate contact. Of the mechanical conveyors the more common are bucket elevators and inclined tramways. Perhaps the most useful for the construction of dams, which because of their location are usually inaccessible, is the overhead cableway, by which materials are conveyed from the side hills and delivered to any point on the dam. One of the most notable of such cableways was that used in the construction of the Colorado river dam. This cable was suspended on two towers, the higher of which was about 70 feet tall and was situated at a point 65 feet above the crest of the dam. The main cable was $2\frac{1}{2}$ inches in diameter and 1850 feet long, being 1350 feet between points of support, and on this cable the carriage and its operating cables were supported.

Various methods are employed to preserve the batter lines of a masonry dam during construction. On the Bhatgur dam this was done by means of large wooden forms constructed to scale from drawings, and set and tested by transit instruments. These forms gave the masons outlines which they had no difficulty in following. On the Sodom dam the true batter-points were established for each course of the facing stone at every twenty feet of the length of the dam by the use of instruments. These batter-points were cut in the stone and the foreman required to work to them for each succeeding course, and at change of batter short profiles fifty feet apart were set out by the engineers to insure the correct laying of the first course at the new rate of batter.

320. Asphalt Lining and Cement Wash.—It is frequently necessary to line the inner slopes of earth embankments or loose-rock or masonry dams, and the entire inner surfaces of small artificial storage reservoirs such as are used in storing the discharge for a day or two, either of sewage or artesian water. It is common to use such small distributary reservoirs in connection with pumped water, especially where pumped by windmills, or for water which is to be furnished through pipes for subirrigation.

Linings for large storage dams, however, are usually applied with the object of preventing leakage through them which may endanger their integrity, or for the purpose of saving water as in the case of a loose-rock dam. Some of the modes of making loose-rock dams impervious have been described in Arts. 291 to 294, in connection with their construction. These consist chiefly in the deposition of silty or earthy material from scows on the upper side of the loose rock dam, or lining the inner surface with planking which may be rendered impervious by the application of paraffine or asphaltum paints. Several of the more recently constructed earth dams with masonry cores, and gravity masonry dams have had one or more coatings of rich cement wash or natural bitumen applied somewhat like whitewash or as a thick paint to their inner surfaces in order to make them less pervious. This has proved effective in preventing sweating on the outer surfaces of the dams or core walls.

The most satisfactory and most impervious lining for earth embankments is asphaltum. Its toughness and flexibility enables it to conform without rupture to slight cracks and settlements of the underlying material, thus indicating where repairs may be necessary, while such repairs may be easily made in the asphaltum. The bottoms of earth distributary reservoirs, the inner slopes of a number of earth embankments in California and Colorado, and the bottoms and sides of earth canals and tunnels have been lined with this substance, which has remained in satisfactory condition, always easy to repair, for a number of years, and experience shows that where properly applied such linings have proven successful under the most trying conditions.

A number of methods have been devised for making the asphaltum adhere to the surface beneath, and thus prevent slipping or crawling in hot weather. One of these is by anchoring heavy burlap at the top of the slope, stretching it tight upon and pressing it into the first coat of asphalt. Upon this a second coat of asphalt is spread. Such application of burlap to the Linda Vista Reservoir in Oakland, California, which is 35 feet deep with slopes of 1 on 1, has prevented the creeping of the asphaltum for four years. Another and equally successful mode of fastening the asphaltum is by the use of anchor spikes cut from strap iron about an inch wide and 6 to 8 inches long, which are driven through the asphalt into the banks, and over this a second coating of asphalt is applied.

One of the most interesting examples of the use of asphaltum for lining dams is in the case of the two West Ashland Avenue reservoirs in Denver, Colorado, on which this lining was applied by Mr. J. D. Schuyler. The maximum depth of one of these reservoirs is 32 feet, with side slopes of 1 on $1\frac{1}{3}$. Beginning at the bottom of the slopes, the asphaltum was laid in horizontal strips about 10 feet wide with an average thickness of $1\frac{3}{4}$ inches, and was spread with hot rakes and tamped with hot square tampers and ironed with heavy hot smoothingirons much as is the asphaltum used in street pavements. While this sheet of asphalt was warm, strap-iron anchor spikes 1 inch wide, 1/8 inch thick, and 7 to 8 inches long were driven through the asphaltum into the bank in rows 1 foot apart and 1 foot between centres in the row, every other row being flush with the concrete, the alternate rows being allowed to project for the support of scantling on which the workmen stood. When the final coat was applied the projecting spikes were driven flush, and all painted over with bitumen.

The asphaltum used on these reservoirs consisted of 78% La Petra asphalt with 22% of Las Conchas flux from the Lower California coast. This was boiled in open kettles for twelve hours at a temperature a little over 300° and frequently stirred. 20% by weight of this was mixed with 80% of sand previously heated to the same temperature. The weight of this mixture after being applied was about 127 pounds per cubic foot. Upon this lining was applied a second or paint coat of pure Trinidad asphaltum fluxed with residuum oil and poured on hot from buckets, and ironed over with cherry-red-hot irons. A more satisfactory cohesion is gotten between the two coats by applying the second quickly after the first is laid and while it is still warm and clean, the thickness of this second being from 1/8 to 1/4 of an inch. Mr. Schuyler reports the cost of this lining as about 15 cents per square foot.

An interesting use of asphaltum lining has recently been made by Mr. R. B. Stanton in the construction of a small placer mining dam in an inaccessible portion of the mountains of Southern California. This lining is a true concrete, in which asphaltum is used as a mortar or binding material with broken stone. The stone was obtained from a porphyry dike near by, and was in sizes of two inches and under, all the fine material and dust being used so as to form a nearly theoretically perfect concrete, where every large space is filled with some small material, and thus only the smallest amount of asphaltum was necessary to bind the whole together. The rock was heated and mixed in a pan, and a hot paste composed of four parts of California refined asphaltum and one part crude petroleum was boiled in another pan, poured over the hot rock, and well mixed with shovels and hoes. This concrete was put on in layers four inches in thickness, in horizontal strips four to six feet wide, and where the strips joined the old edge was coated with hot paste. Over the whole a second coating of hot asphaltum paste, mixed in the same proportions and boiled for a much longer time, was applied, which when cool was hard and brittle like glass, yet tough and elastic when warm. This paste was applied and ironed down to a thickness not exceeding one eighth of an inch. This lining has stood for two years without showing a single crack. In one place the bank settled 6 inches under a strip 4 feet wide and the lining followed the settlement without break, and though applied on a slope of $1\frac{1}{2}$ on 1 it has shown no tendency to creep, which is one of the great objections to a lining composed of asphaltum and sand.

It is believed by Mr. Stanton that this asphaltum concrete forms the most perfect lining for reservoirs of any size in any climate, while it is remarkably cheap, having cost but 15 cents per square foot in an inaccessible locality. He suggests that such concrete lining should be of broken stone, not gravel or sand, and of all sizes below two inches, retaining even the dust, and that the materials for such concrete can be had at almost any reservoir site, as it is unnecessary to use clean sharp sand, since mixed broken gravel, dirt, and sand, if clear of vegetable matter, will give equally satisfactory results. Asphaltum and asphalt concrete are considered among the best materials for lining not only earth or loose-rock dams, but for canals in pervious materials, scouring sluices, or wasteways, where high velocities are necessary and the banks and bottom must be perfect.

321. 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 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

FIG. 98.—VIEW OF SAN FERNANDO SUBMERGED DAM.

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 550 feet apart and the bed-rock about 45 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. 98). 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.

322. 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.

323. 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.

324. Examples of Masonry Dams.—In Table XXIII on

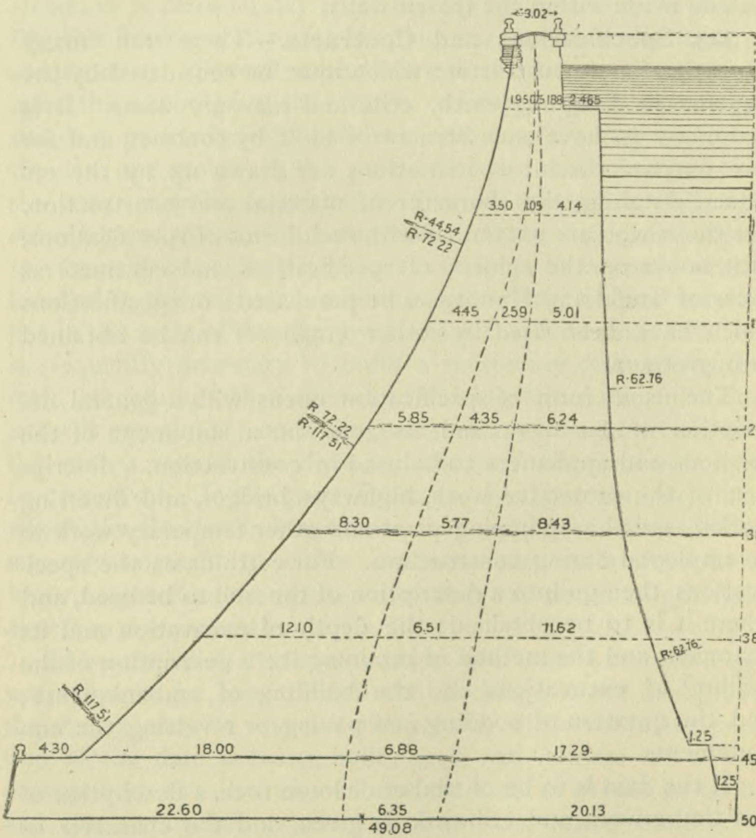


FIG. 99.—CROSS-SECTION OF FURENS DAM, FRANCE.

page 337 are 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 with out a few examples of the larger and more typical of the modern ones, 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,

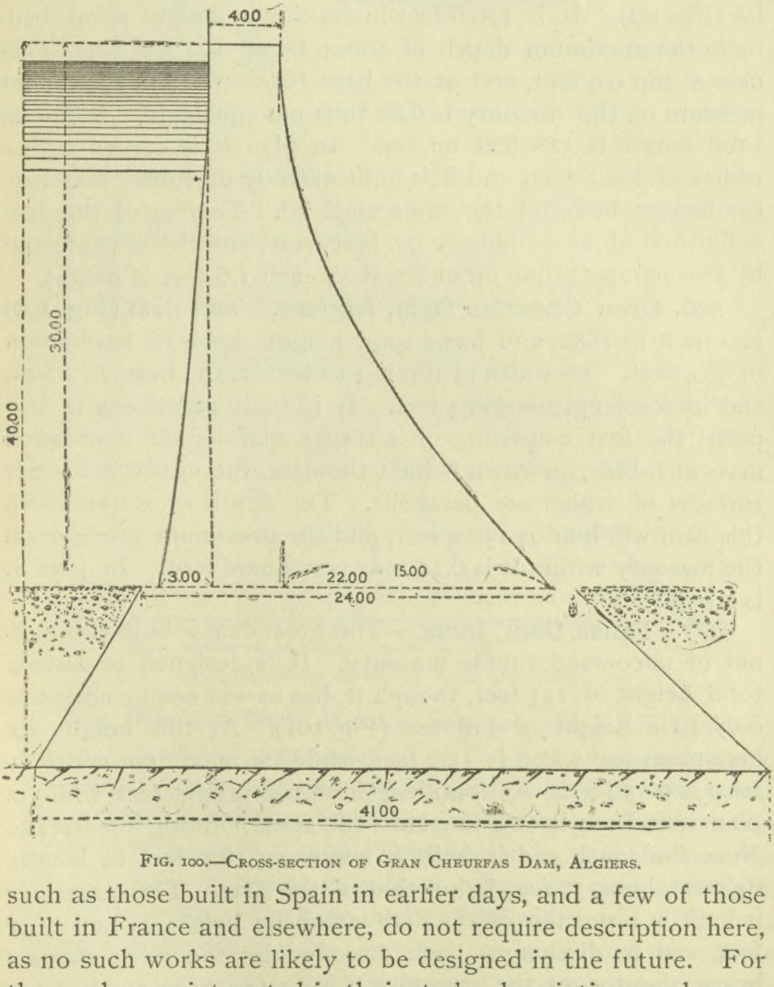


FIG. 100.—CROSS-SECTION OF GRAN CHEURFAS DAM, ALGIERS.

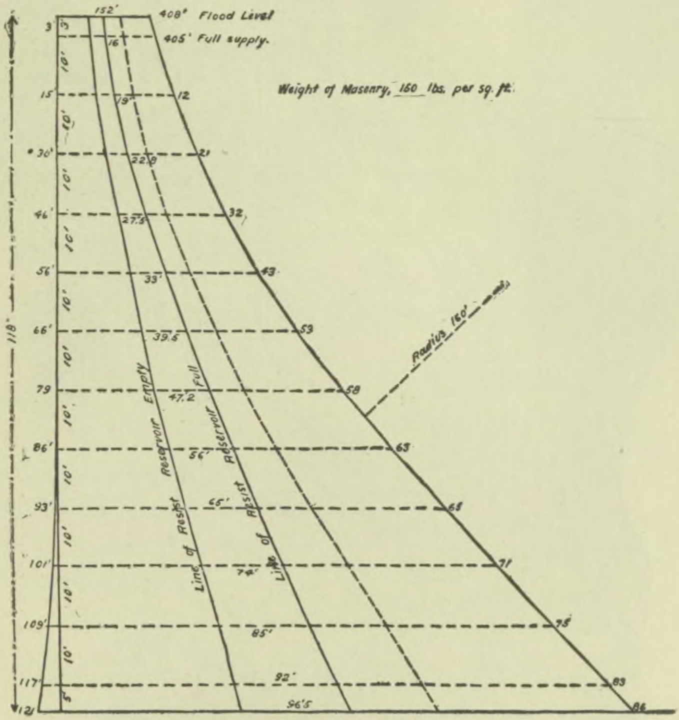
such as those built in Spain in earlier days, and a few of those built in France and elsewhere, do not require description here, as no such works are likely to be designed in the future. For those who are interested in their study, descriptions and cross-sections of these can be found either in Wegmann's "Design and Construction of Masonry Dams," Krantz's "Reservoir Walls," or in the 12th and 13th Annual Reports of the U. S. Geological Survey.

325. **Furens Dam, France.**—This is one of the largest and first of the great dams built according to modern formulas (Fig. 99). It is 170.6 feet in maximum height above bedrock, 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 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.

326. **Gran Cheurfas Dam, Algiers.**—This dam (Fig. 100) 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 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.

327. **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. 101). At this height its maximum top width is 15.2 feet, while its maximum width at 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

is a large outlet tunnel, which discharges into the conduit which carries the water to Bombay for the supply of that city.



Note.. Pressures reservoir empty, in lbs. per sq. Inch

FIG. 101.—CROSS-SECTION OF TANSA DAM, INDIA.

328. **Bhatgur Dam, India.**—This dam (Pl. XXXII) is 4067 feet in length, and is constructed of the best uncoursed rubble masonry in cement, excepting in the upper central portion, where the pressure is less than 60 lbs. per square inch. There it is of concrete with blocks of rubble imbedded in it. 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. 102). 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

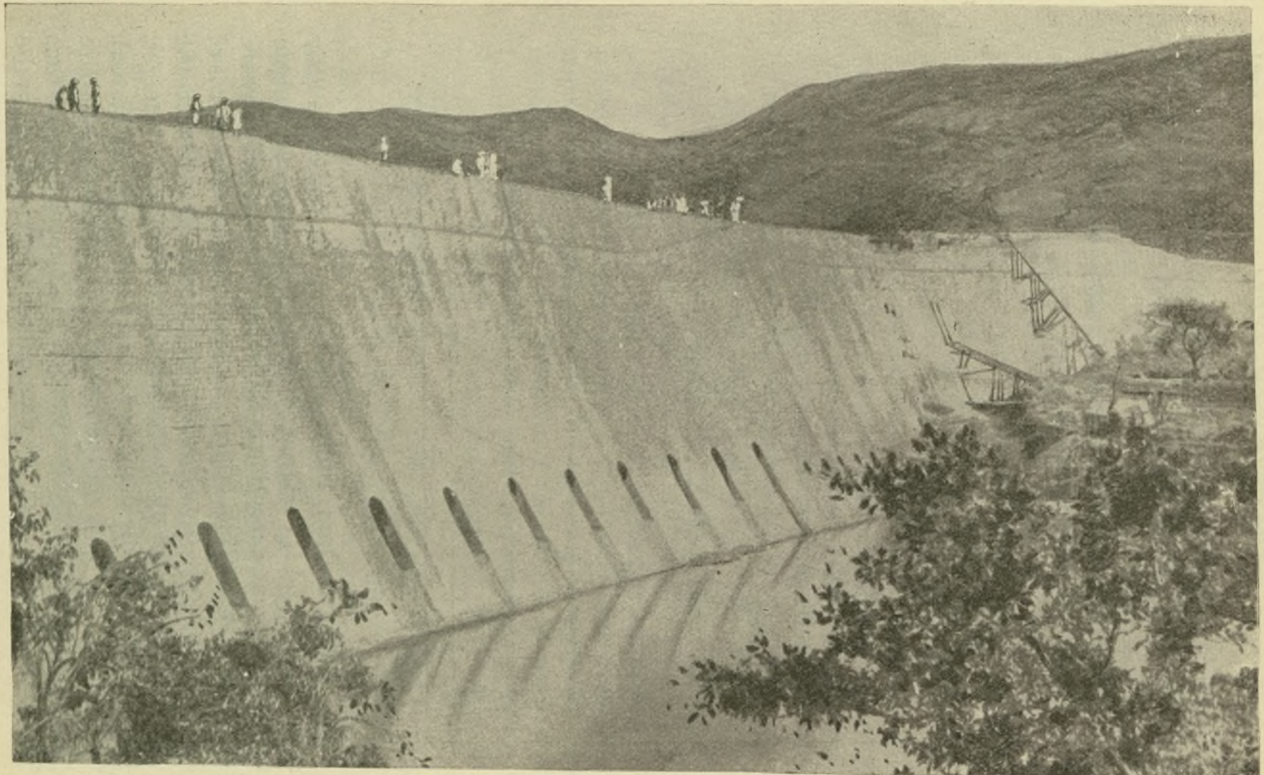


PLATE XXXII.—VIEW OF BHATGUR DAM, INDIA.

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

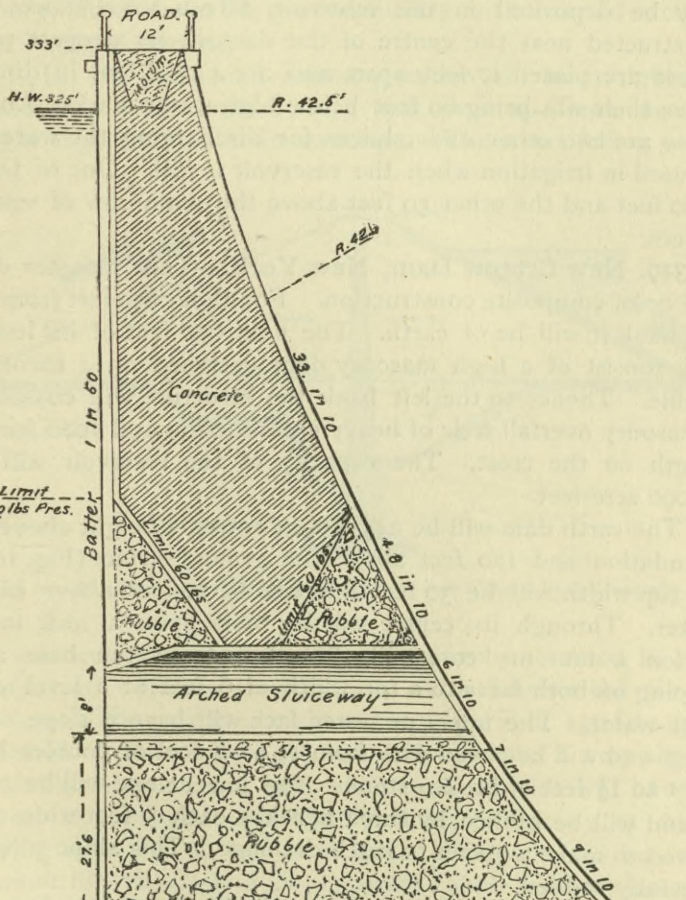


FIG. 102.—CROSS-SECTION OF BHATGUR DAM, INDIA.

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.

329. **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. 103). 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 $1\frac{1}{2}$ to 2 feet of cobbles laid on 1 to $1\frac{1}{2}$ feet of broken stone. The lower slope will be 1 on 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 crest of the dam will be 14 feet above the high-water level or crest of the overfall weir. 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

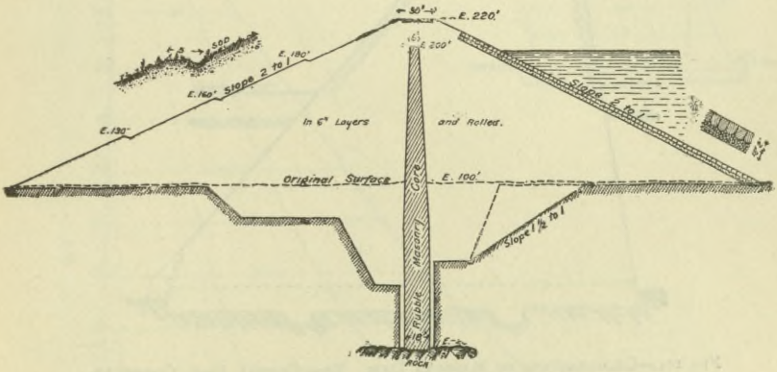


FIG. 103.—CROSS-SECTION OF EARTH EMBANKMENT. NEW CROTON DAM, CORNELL'S.

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 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. 105.)

330. Periar Dam, India.—This dam, which is constructed throughout of concrete, is 1230 feet long on top, has a maximum height of 173 feet. Its crest is surmounted by a parapet 5 feet in height, the maximum depth of water which the dam will

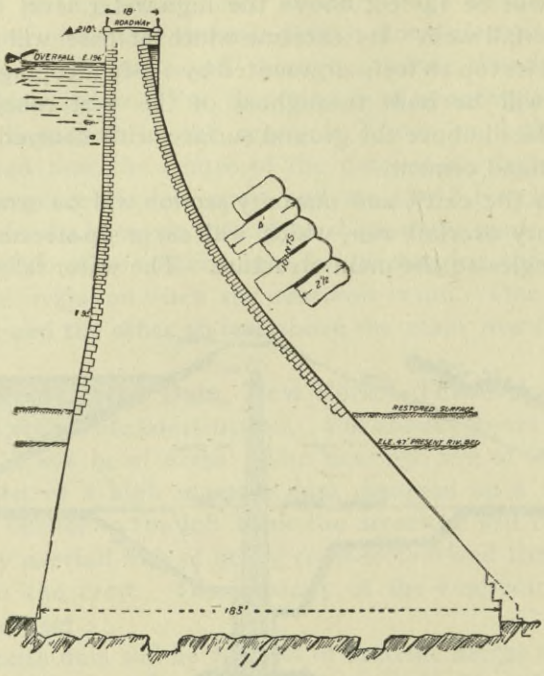


FIG. 104.—CROSS-SECTION OF MASONRY DAM. NEW CROTON DAM. CORNELL'S.

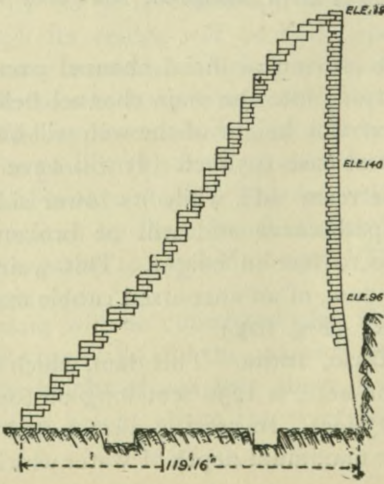


FIG. 105.—CROSS-SECTION OF OVERFALL WEIR. NEW CROTON DAM, CORNELL'S.

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

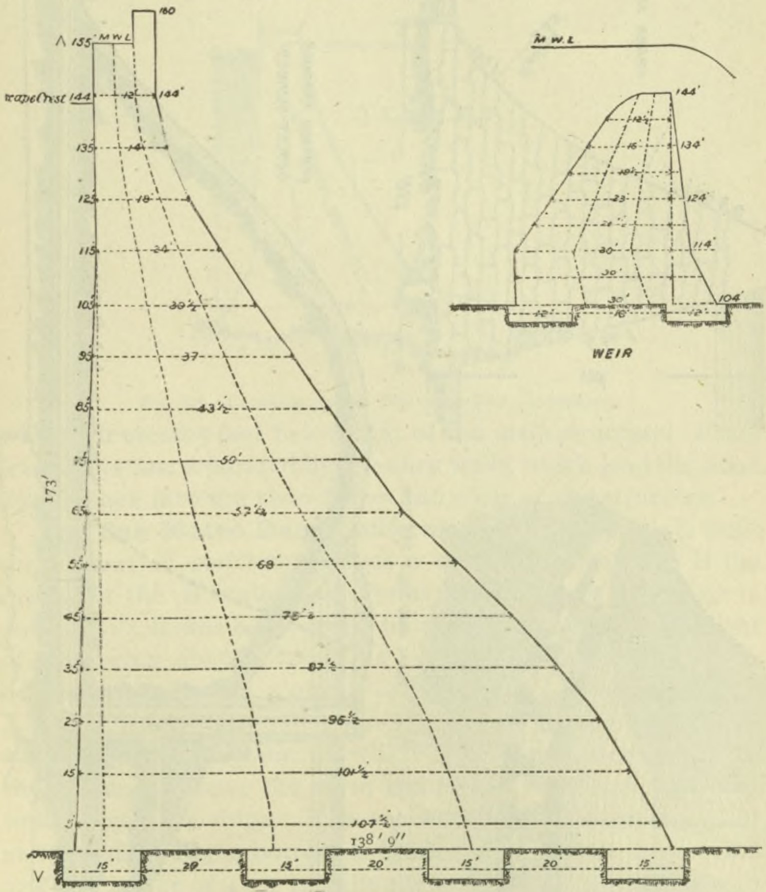


FIG. 106.—CROSS-SECTIONS OF PERIAR DAM AND WASTE WEIR, INDIA.

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.

331. Beetaloo Dam, South Australia.—This structure (Fig. 107) 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,

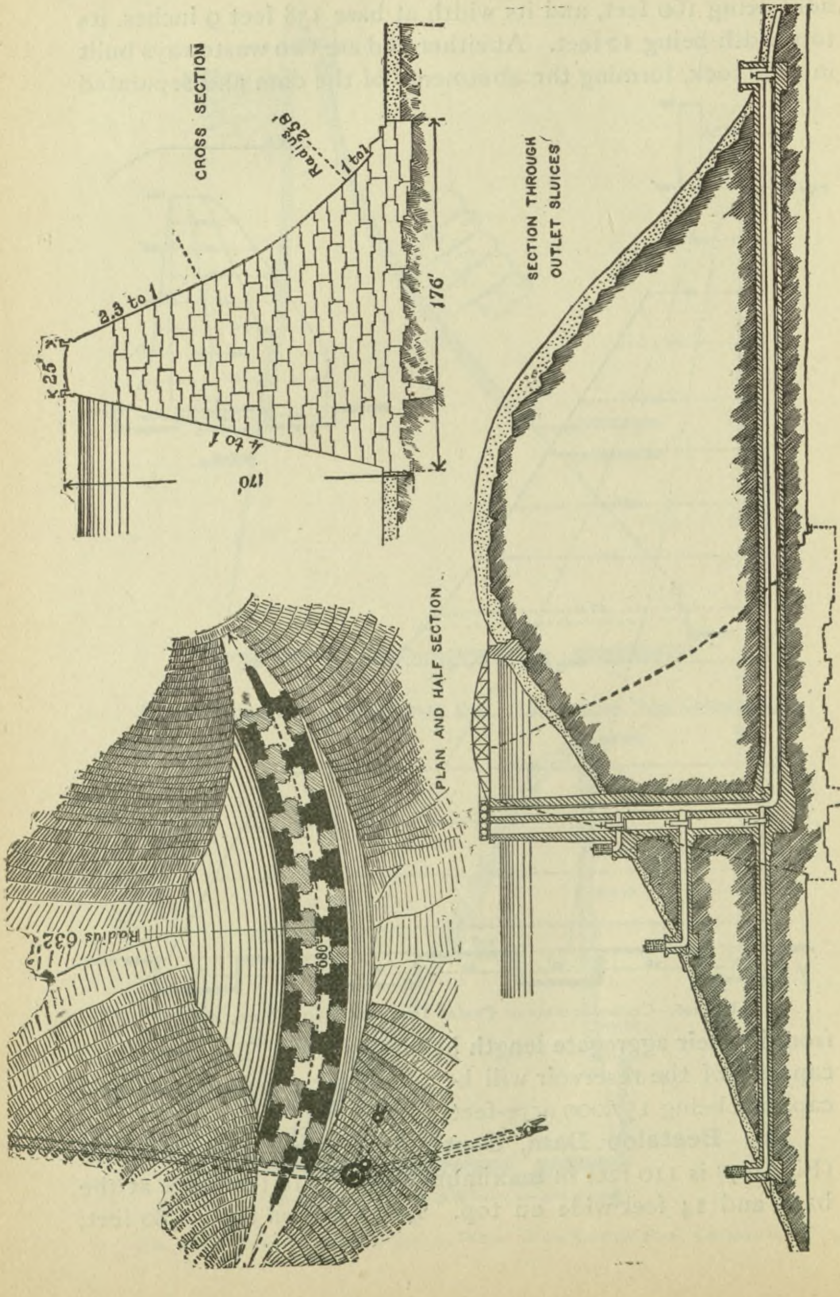


PLATE XXXIII.—SAN MATEO DAM. PLAN, CROSS-SECTION AND OUTLET SLUICES.

and it is curved in plan, the convex side facing up-stream. It is constructed throughout of concrete, and in one end of the dam is built a set of three wasteways, their total length being 200 feet

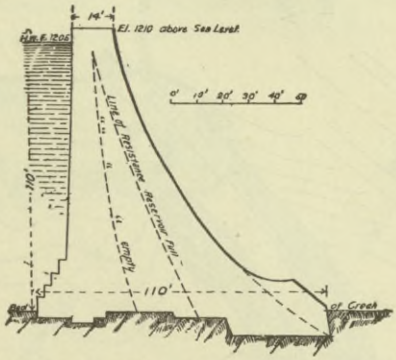


FIG. 107.—CROSS-SECTION OF BEETALOO DAM, AUSTRALIA.

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.

332. 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 316, 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. XXXIII, its maximum height is 170 feet, its crest being 5 feet above high-water mark, at which level is a wasteway built a 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.

333. Sweetwater Dam, California.—This dam (Pl. XXXVI) 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 XXXIV and XXXV, it is 90 feet in maximum

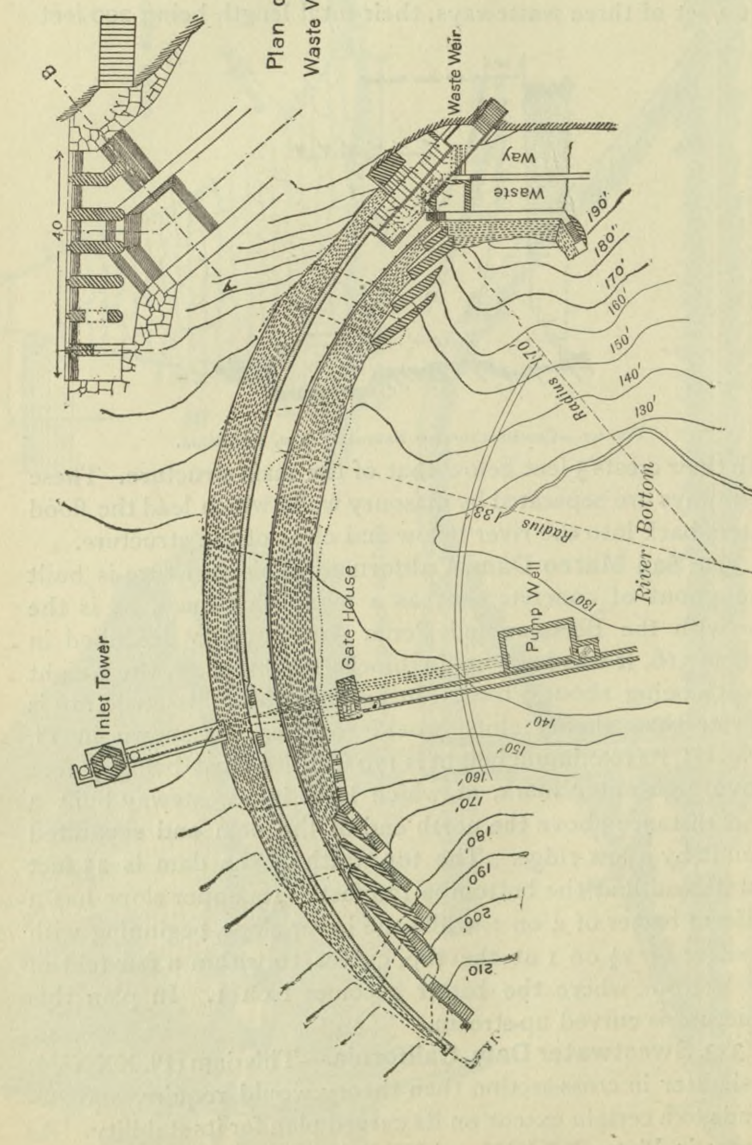


PLATE XXXIV.—PLAN OF SWEETWATER DAM.

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

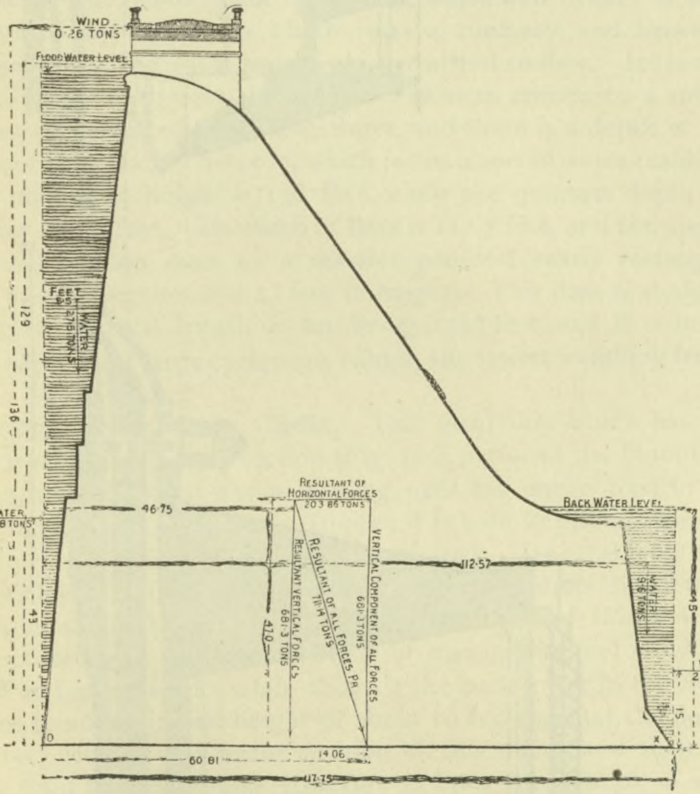


FIG. 108.—CROSS-SECTION OF VYRNWY DAM, WALES.

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

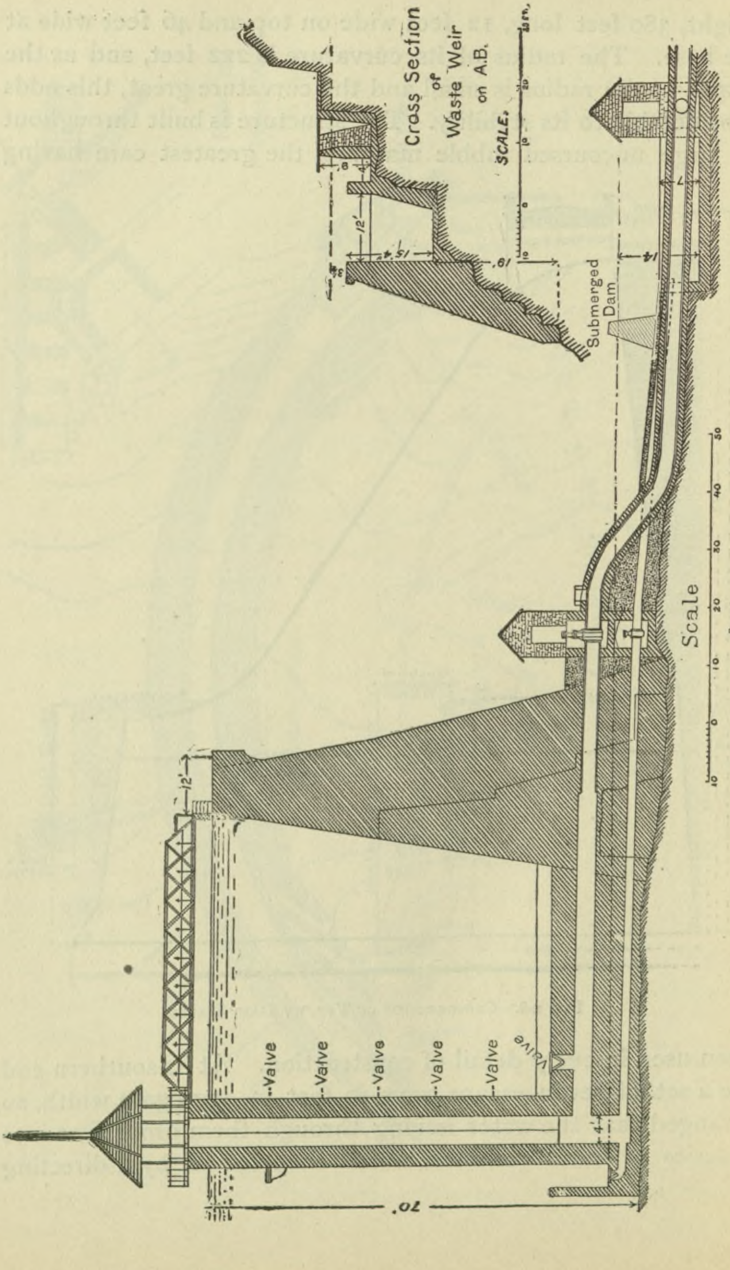


PLATE XXXV.—CROSS-SECTION OF SWEETWATER DAM.

wall so as to clear the dam. Near its base is a discharge sluice, operated from a water tower in the reservoir.

334. **Vyrnwy Dam, Wales.**—This structure is peculiar in cross-section (Fig. 108), 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 throughout of large cyclopean rubble, the stones weighing from 2 to 8 tons apiece.

335. **Betwa Dam, India.**—This structure, which has an unusually heavy cross-section (Fig. 109), performs the functions of a weir, the flood waters passing over the entire crest to an extreme depth of 6½ 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 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 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

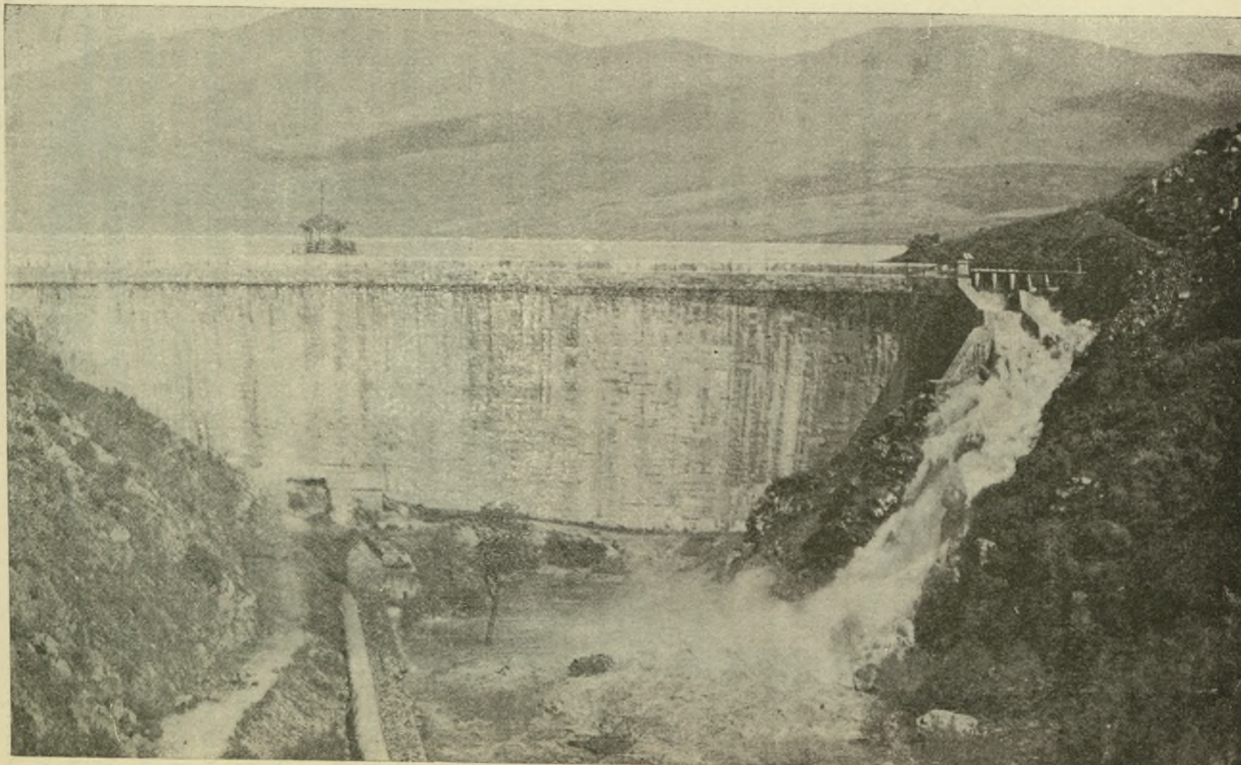


PLATE XXXVI.—VIEW OF SWEETWATER DAM.

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.

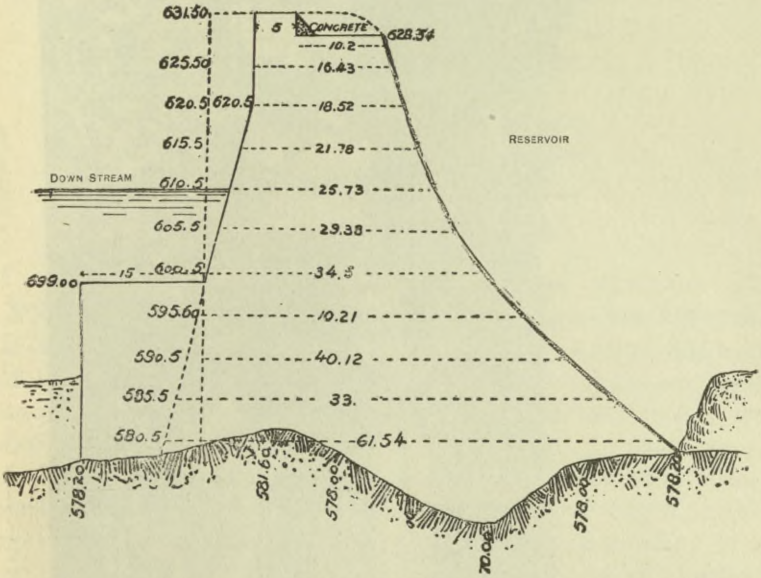


FIG. 109.—CROSS-SECTION OF BETWA DAM, INDIA.

336. **Turlock Dam, California.**—This structure (Fig. 110) 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 feet, thus giving a water cushion on which the floods may fall. The main weir is curved in plan with a radius of 300 feet ; it is 320 feet in length on top, 90 feet in width at the base, 24 in width on top, and 128 feet in maximum height, and is built

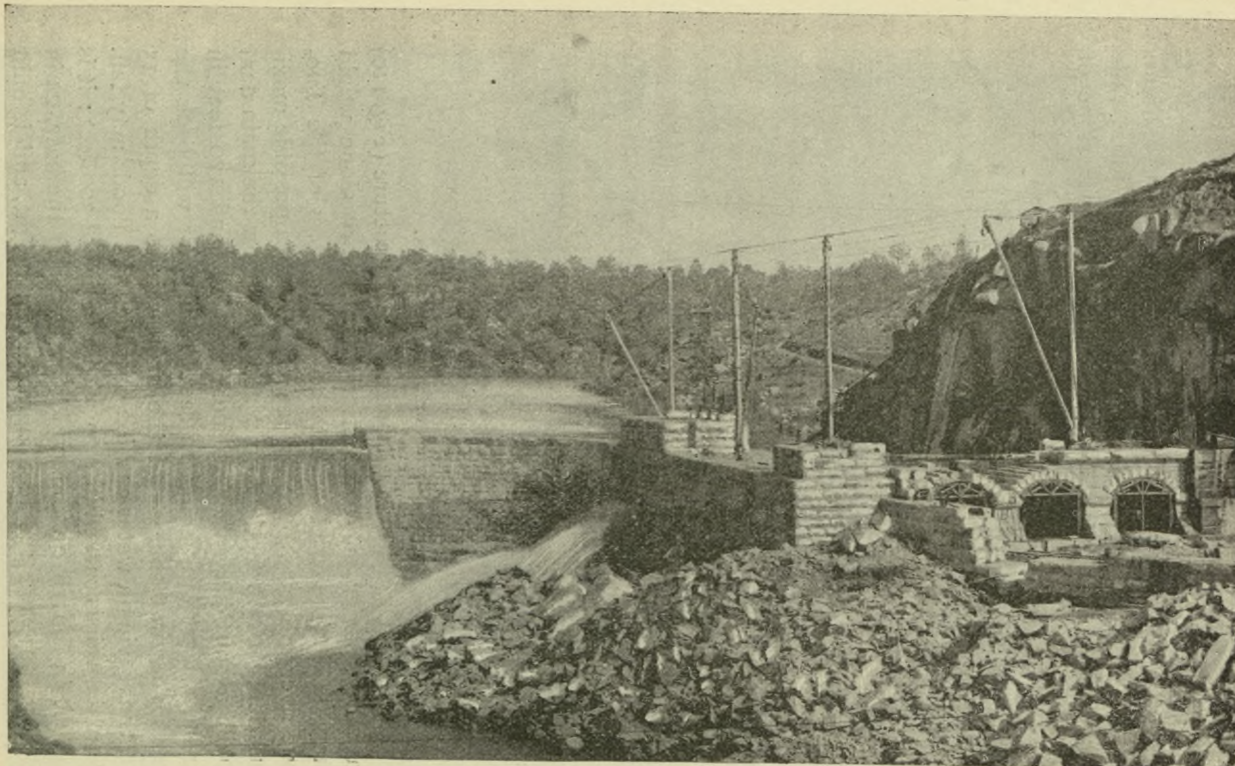


Plate XXXVII.—Folsom Canal, View of Weir and Regulator.

throughout of uncoursed rubble masonry. There is no escape-way, while there are a couple of undersluices which served to pass water during construction. The lower fall has an inclination of about .465 feet per foot.

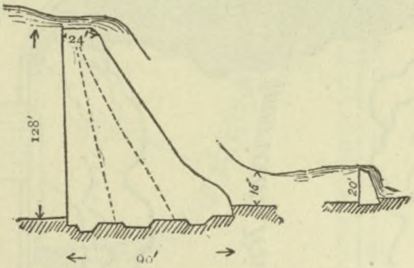


FIG. 110.—CROSS-SECTION OF TURLOCK DAM.

337. **Folsom Dam, California.** — This structure (Pl. XXXVII), 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. XXXVIII). 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 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.

338. **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

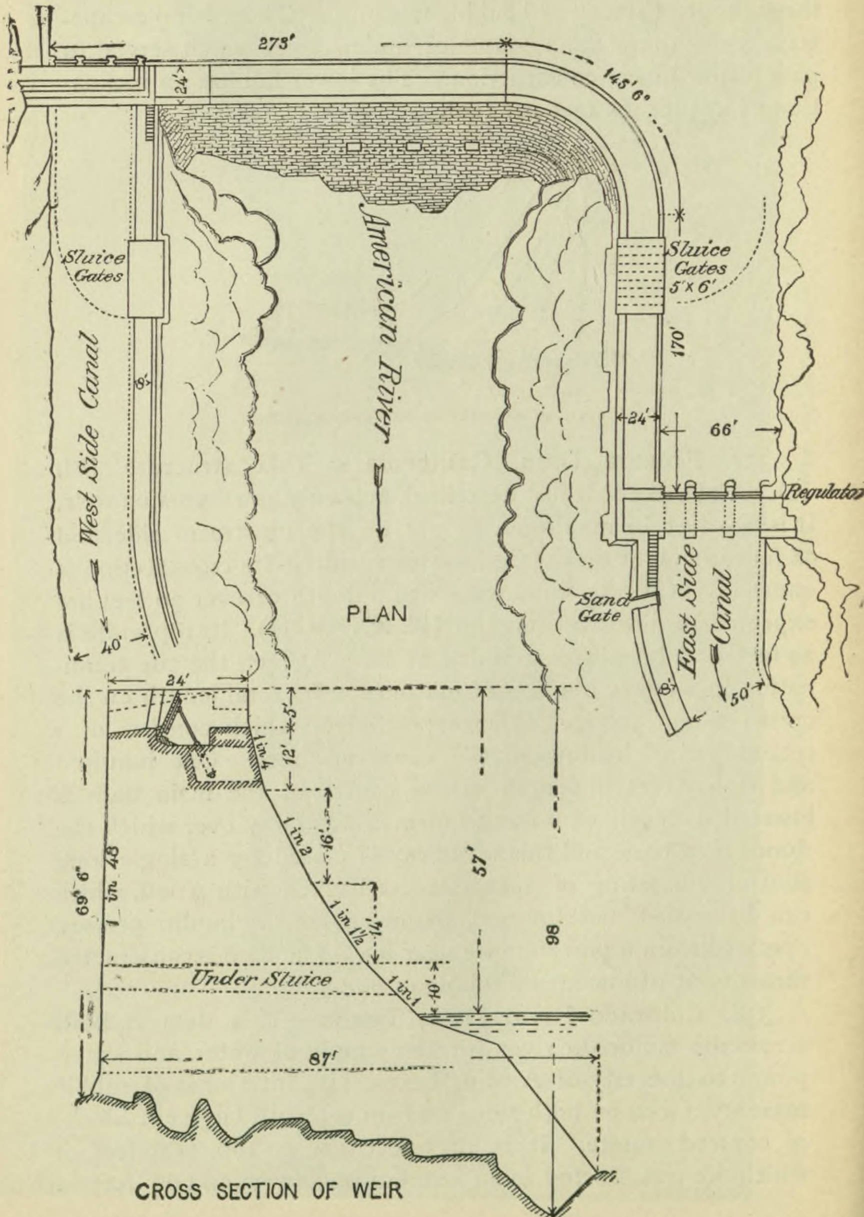


PLATE XXXVIII.—FOLSOM CANAL, PLAN AND CROSS-SECTION OF WEIR.

maximum height, its upper face being vertical. The lower face has an easy ogee-shaped curve (Fig. 111), calculated to pass the

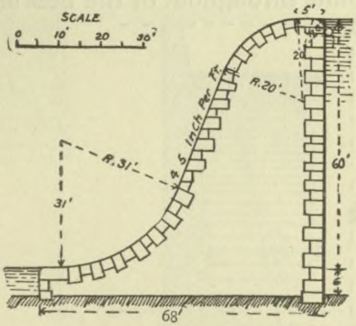


FIG. 111.—CROSS-SECTION OF COLORADO RIVER DAM.

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.

339. **Bear Valley and Zola Dams.**—The most notable curved dams are the Bear Valley dam in California, and the Zola dam in France, the cross-sections of which are unusually light, as they depend chiefly on their curved plan for their stability. The former (Fig. 112) 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. The structure is 450 feet in length on top, and in plan it is curved with a 300-foot radius (Fig. 113). It is built throughout of the best uncoursed rubble

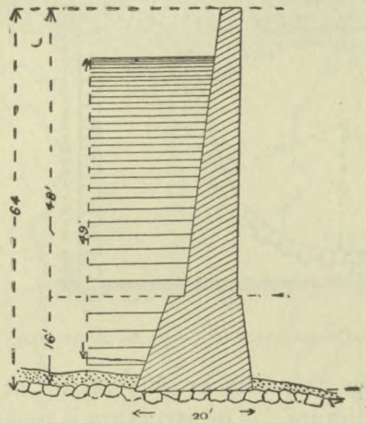


FIG. 112.—CROSS-SECTION OF BEAR VALLEY DAM.

granite masonry, and depends almost wholly on its curved plan and the excellence of its construction for its stability, since the

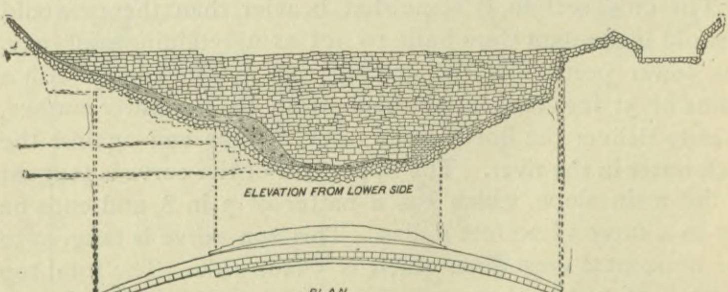


FIG. 113.—PLAN AND ELEVATION OF BEAR VALLEY DAM.

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 Zola dam (Fig. 114) 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

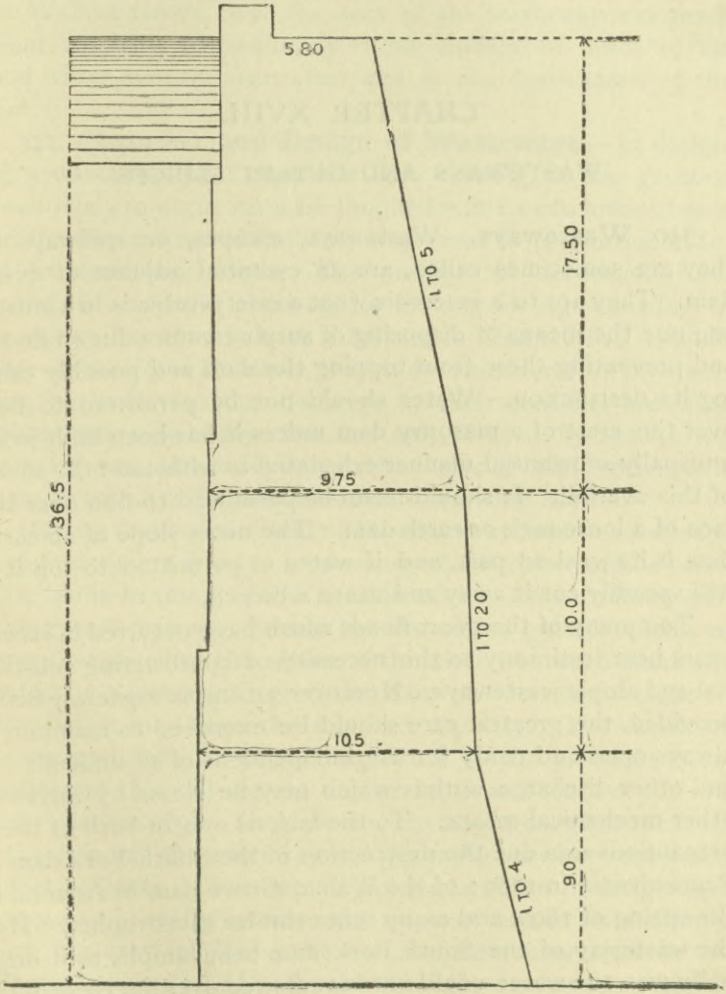


FIG. 114.—CROSS-SECTION OF ZOLA DAM, FRANCE.

curvature and the excellence of its construction for its stability. The material of which it is built is uncoursed rubble masonry

CHAPTER XVIII.

WASTEWAYS AND OUTLET SLUICES.

340. **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 insufficient, resulting consequently in the passage of much of the flood water over the dam crest and in the destruction of the work.

341. 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 must 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 289). 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.

342. 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. 89) may be applied with approximate results, and we have

$$Q = 3.33(l - .1nh)h^{\frac{3}{2}}. \quad . \quad . \quad . \quad . \quad . \quad . \quad (1)$$

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

$$Q = 5.35clh^{\frac{3}{2}}, \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (2)$$

in which c 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. (see also Art. 312),

$$Q = 3.012lh^{1.53}. \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (3)$$

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.

343. **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 334 to 338). 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 327 to 329).

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 as in the Carmel reservoir (Art. 345).

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 reser-

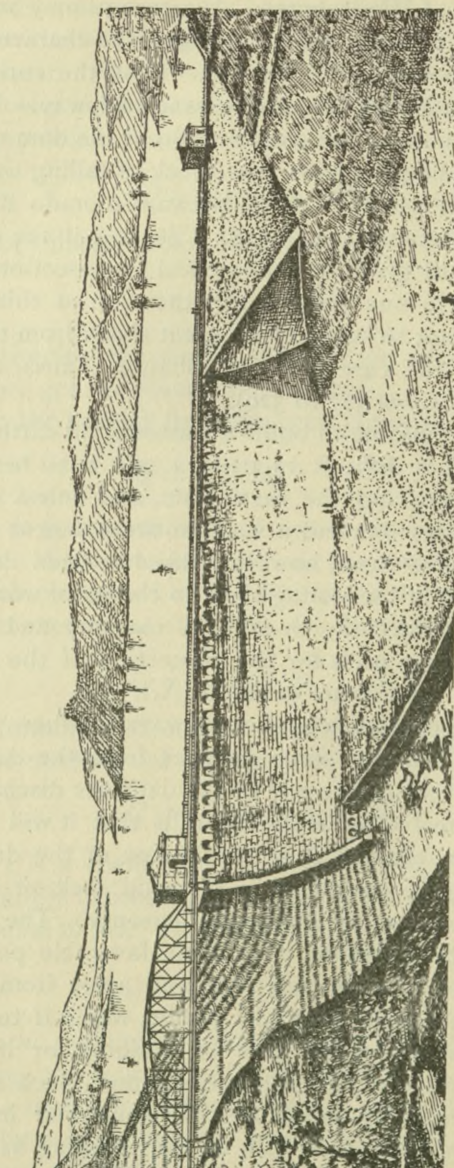


Fig. 115.—View of New Croton Dam and Wasteway.

voir dam in New Jersey, the Ashti and Periar dams in India, and the Pecos and Idaho dams in the West.

344. **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 X. 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 171) 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 172). Other of the more usual and more popular forms are the wide-crested overfall dam (Articles 311 and 312) and the stepped overfall weir used in the Croton reservoirs (Figs. 105 and 115).

345. **Examples of Wasteways**.—Brief descriptions and illustrations of wasteways were given in Articles 328 to 331. The wasteway of the Sweetwater dam is peculiar. It is built as a continuation of the main dam and, as shown in Plates XXXIV and XXXV, 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. 106) 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 292. 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.

Interesting types of wasteways to earth dams are those for some of the Croton water-shed reservoirs, and that for the Santa Fé dam. The Carmel reservoir of the Croton watershed is closed by an earth dam, 260 feet in length of the centre being occupied by a masonry overfall weir and gate-house. This masonry wasteway is bonded with the earth dam through the masonry core walls, and by protecting wing or retaining-walls of masonry. The maximum height of the waste weir is 65 feet, and the crest of the earth dam is 12 feet higher. The waste weir has a cross-section similar to that of the new Croton dam (Fig. 105).

Above the Santa Fé earth dam is an old masonry dam, the crest of which has been cut down to the level of the wasteway of the earth-dam, which is 10 feet lower than its crest. The old dam serves to check and cause the deposit of sediment above it, and this is to be sluiced off through an undersluice and tunnel terminating in the wasteway of the dam (Fig. 116). When the flood flow exceeds the capacity of the tunnel it will pass over the old dam and be discharged through the main wasteway which is at one end of the earth dam, is semi-circular with a radius of 150 feet, and is 471 feet long on its crest. Its

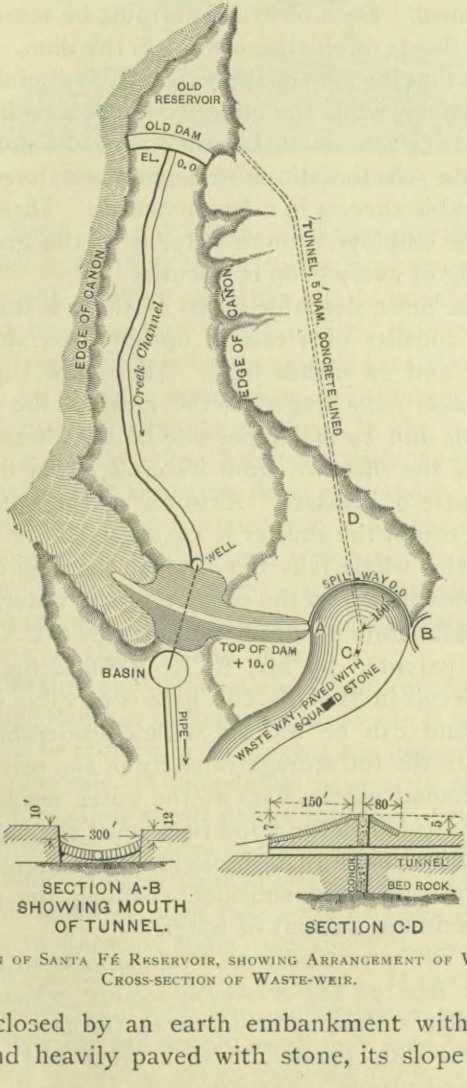


FIG. 116.—PLAN OF SANTA FÉ RESERVOIR, SHOWING ARRANGEMENT OF WASTEWAY AND CROSS-SECTION OF WASTE-WEIR.

summit is closed by an earth embankment with a masonry core wall, and heavily paved with stone, its slope being very gentle.

346. Automatic Shutters and Gates.—The use of flash boards 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. 117. 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 pressure. 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. XXXIX) 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. XXXIX),

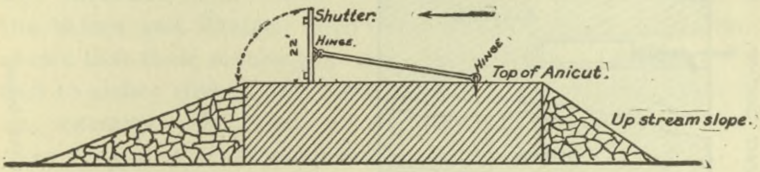


FIG. 117.—CROSS-SECTION OF SHUTTER ON SOANE WEIR, INDIA.

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, it runs out from holes in the bottom and the weights then become heavier than the gate and raise it.

347. **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.

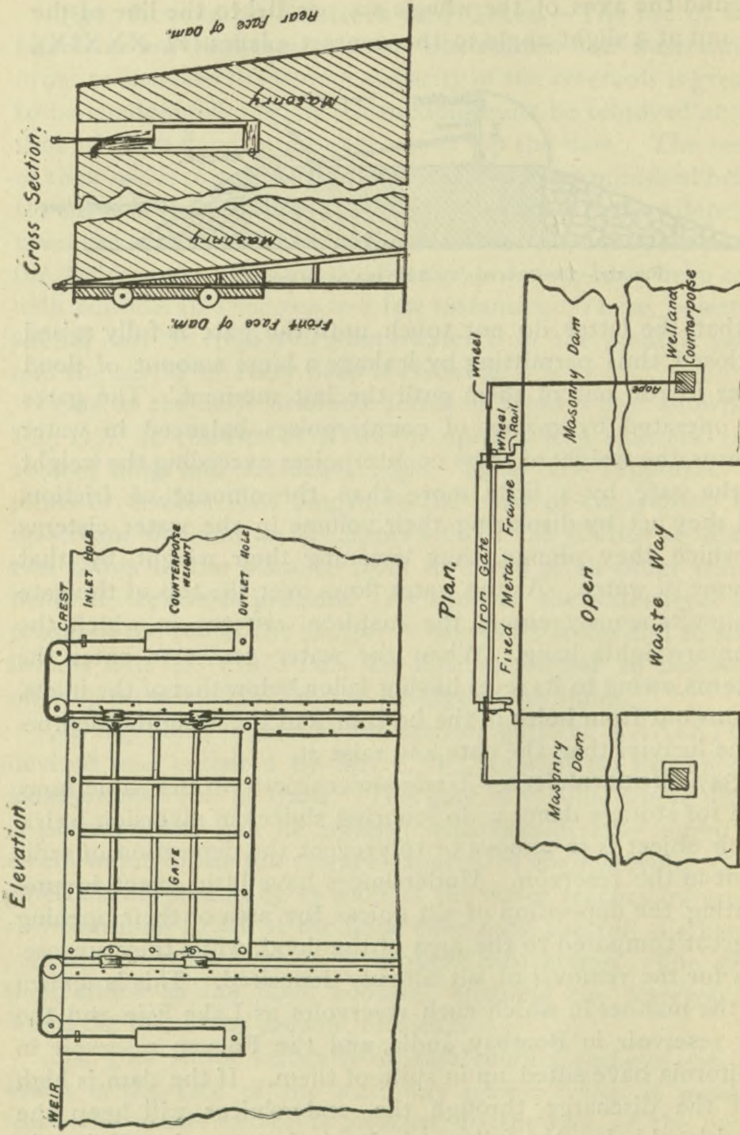


PLATE XXXIX.—PLAN, ELEVATION, AND CROSS-SECTION OF REINOLD'S AUTOMATIC WASTE GATE, INDIA.

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.

348. **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. XXXII). 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 became 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. XXXVIII). 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.

349. **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. 118). 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.

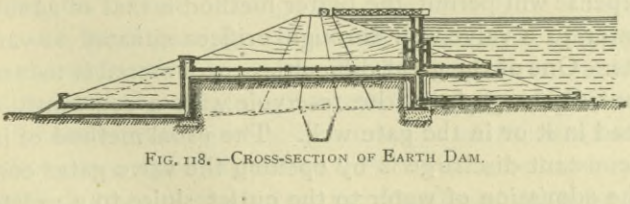


FIG. 118.—CROSS-SECTION OF EARTH DAM.

350. 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. 119 is of the kind em-

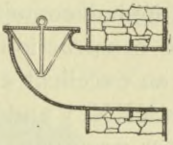


FIG. 119.—VALVE-PLUG, SWEETWATER DAM.

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 XXXV 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 cap stan 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.

351. 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 XXXIV and XXXV, showing the gate tower to the Sweetwater reservoir. In Fig. 120 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. XXXIII), and the proposed Citizens' Water Company dam in Colorado, the valve tower is situated at a point quite independent 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.

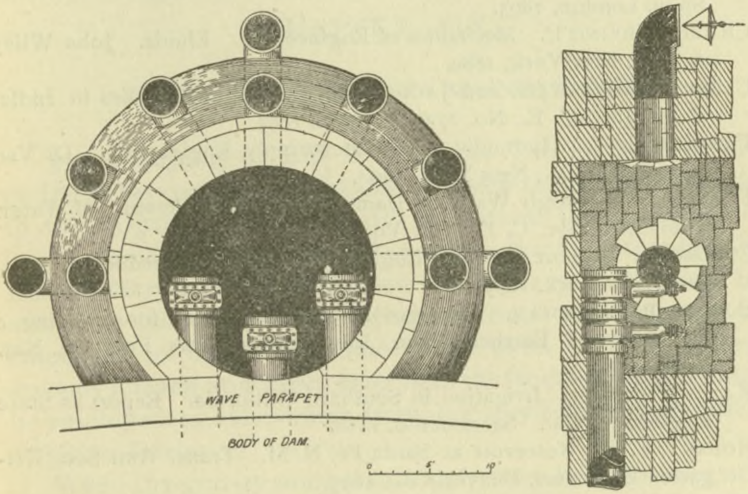


FIG. 120.—VALVE CHAMBER AND VALVES.

The mode of drawing water from the Beetaloo reservoir, Australia, is interesting in that no outlet-valve tower is used. The outlet pipe is carried through a tunnel in the hillside, and on the inner or reservoir surface this pipe curves up the slope of the hill. Its entrance or extremity is 63 feet above the outlet tunnel. Twenty-six feet lower down is a second inlet valve, and a third is placed opposite to and level with the tunnel entrance. These inlet valves are of common flap pattern, covered with wire strainers, and are operated from the crest of the dam by shafts or rods $1\frac{1}{2}$ inches in diameter, which rest on pulleys. These rods are pulled up or pushed down to open or close the valves by having a long screw at their upper ends, working in a female screw turned by a $2\frac{1}{4}$-foot hand-wheel.

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CHAPTER XIX.

PUMPING, TOOLS, AND MAINTENANCE.

353. **Pumping or Lift Irrigation.**—The methods of irrigating so far considered are those in which the water is brought to the irrigable land by gravity or natural flow. There are large volumes of water situated at such a low level that gravity will not carry it to the fields, and this water must be raised by means of pumps or other lifting devices. Pumping may be employed to utilize the water from wells or from natural streams flowing at a lower level than the land irrigated, or may be employed to raise water from low-service canals to others at higher levels.

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 subsoil becomes saturated, the ground-water level is raised, and much of the water delivered by gravity systems thus finds its way by seepage from the fields into the soil and may be pumped up and reemployed in 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, and a large proportion of the irrigation in Europe, China, Japan, India, and Egypt is by means of lifting (Art. 112). In Oriental lands lifting is performed almost wholly by animal or man power, through various ancient devices operated chiefly by bullocks or men. In Italy quite a deal of pumping is done by machinery, chiefly to raise water from existing low-level canals to high-service canals.

In our country the value of pumping as a means of irrigation is scarcely yet appreciated. A few windmills and water-wheels are utilized for this purpose, and a small amount of pumping is done by steam-power, though the value of the water supply to be derived from these modes of lifting is sure to increase greatly in the near future, when its cheapness and adaptability come to be fully recognized. A perusal of this chapter will show that pumping as a mode of supplying irrigation waters, far from being more expensive than water derived from gravity supplies, generally furnishes water more cheaply than do gravity supplies, both in the matter of first cost of the pumping or gravity plant or the equivalent cost of water right (Tables I, XVII, and XXIII), and in the matter of the cost of maintenance and operation equivalent in the gravity supply to the annual water rental or rate paid. Moreover, the source of water supply is more directly under control of the irrigator, and he is troubled by none of the vexatious questions of priority of right, tatils, etc.

354. **Motive Power and Pumps.**—Pumps are machines for elevating water, and consist of two principal parts: (1) the pumping or water-elevating mechanism, and (2) the motive power by which this is operated.

Pumps may be divided into four general classes, according to the principle on which they raise the water. These are:

1. Lift-pumps.
2. Force or plunger pumps.
3. Rotary and centrifugal pumps.
4. Mechanical water-elevators.

Lift and force pumps may be combined, and may be either reciprocating or rotary, in which latter case they come under class three. All may be single or double acting.

The motive power may be:

1. Animal-power.
2. Wind-power.
3. Water-power.
4. Hot-air or gas engines.
5. Steam-engines.

The above classes of pumping machinery are nearly all interchangeable. Thus lift, force, and centrifugal pumps and mechanical elevators may nearly all be operated by any of the various motive powers, though not by all of them.

Distinguished from these there are three additional classes of pumping mechanisms in which the motive power and pump are inseparable. These are:

1. Injectors, vacuum-pumps, and pulsometers, in which steam is the motive power.

2. Hydraulic rams and hydraulic pumping-engines, in which water is the motive power.

3. Siphons and siphon elevators, in which atmospheric pressure is the motive power.

Lifting-pumps operate by drawing water through a suction-pipe as the pump bucket ascends; the water is forced through a valve in the bucket as the pump descends, and is then again lifted as the bucket reascends. This variety of pump is dependent for its operation on the creating of a partial vacuum below the pump bucket by atmospheric pressure.

Force-pumps draw water through a suction-pipe as do lift-pumps, but the water is raised above the bucket by the action of a piston or plunger which forces it through a delivery valve. Force-pumps may be single or double acting, and nearly all steam-pumps are of the latter variety, the discharge of these being practically continuous, for as the water is drawn in at one end it is forced out at the other.

Centrifugal pumps depend for their action on a disk to which are attached propeller-blades revolving inside a chamber. These propeller-blades create a partial vacuum which lifts water into the chamber by suction, whence it is forced by the following propeller-blade. Rotary pumps are practically revolving piston-pumps, differing from the latter chiefly in that they are not direct-acting.

Mechanical water-elevators include the various patented devices in which water is raised by means of disks or buckets arranged on a revolving chain; also chain-pumps, Archimedean screws, Persian wheels, norias, the common well-sweep, which is the pæcottah of India and the sakia of Egypt; and many other curious devices.

All the motive powers except wind may be used to operate any of the various classes of pumps. Ordinarily, however, animal power is used to operate the lighter forms of pumping machinery which are intended to elevate small quantities of water, and the common lift and force-pumps and mechanical elevators of various kinds. Wind is employed almost exclusively for the operation of lift and force pumps, as it is too uncertain in its action to work well with centrifugal pumps or mechanical elevators.

The various motive powers may be divided into two general classes, according to the manner in which they are attached to the pumps. These are:

1. Direct-acting pumps.
2. Fly-wheel and belting pumps.

Direct-acting pumps have no rotary motion, their action being reciprocating, and both steam and water cylinders being mounted on a solid bed-plate, so that the piston-rod which produces the power has attached to it the plunger which elevates the water. Fly-wheel or indirect-acting pumps may have the motive power at some distance from and independent of the elevating pump, and be connected therewith through shafting or belting or some other mechanical device. They are not so satisfactory or reliable in their operation where used for irrigation, as they are liable to get out of alignment and out of order, and therefore require more skilled attendance than do direct-acting pumps, though, on the other hand, their efficiency is generally higher.

355. **Choice of Pumping Machines.** — The pump and motive power which are to be employed in each particular case depend wholly on the services to be performed and on various local modifying conditions. The variety of pump must be chosen according as greater or less volumes are to be elevated to greater or less heights. The motive power must be selected according to the pump chosen, the work to be done, and the fuel available, be this air, water or wood, coal or gasoline.

Where means are limited and the area to be irrigated is but a few acres, the motive power chosen will usually be either animal or water. The first is cheapest of installation but least economical, and the second is next cheapest, where a sufficient water supply is available for the operation of an ordinary mid-current undershot wheel or hydraulic ram. Where the area is small but the means at the disposal of the irrigator less limited, animal power will usually be left out of consideration, and the choice rest between wind, water, hot air, gasoline, or steam pumping-engines. If the wind be reasonably steady and the facilities good for the construction of a storage tank, that power, though not less expensive to install than some others, is least expensive and troublesome to maintain and operate, yet not the most reliable. Where water is abundant, it furnishes, through rams, water-wheels, turbines, or water engines, the next least expensive power to maintain and operate, though not the cheapest to install. The class of water motor selected will depend wholly upon the volume of motive power available and the height to which the water is to be raised. Hot-air engines and gasoline engines furnish the most reliable power for pumping water, and are less difficult to operate than steam-engines. Gasoline engines are especially economical where coal or wood as fuel are expensive, though hot-air engines have a wide adaptability in the variety of fuel which they may utilize. Steam-engines, where coal is cheap, furnish the most satisfactory motive power, but are generally not so economical to operate, especially where small areas are to be irrigated. For the pumping of large volumes, water and steam are the only competing motive powers.

The irrigation engineer who proposes installing a pumping plant should consider all the various circumstances which affect the case under consideration. He should carefully weigh the necessity for having a permanent and steady supply, the inaccessibility of the plant for repairs or replacement of broken parts, the relative cost and accessibility of different kinds of fuel or of water, and the degree of intelligence and skill possessed by those who are to operate the machine employed.

356. **Animal Motive Power.**—There are numerous modes of utilizing animal power in pumping water. Among the oldest and best known of these are the common domestic hand pump, the well-sweep, and the curb and bucket, which are all too limited in capacity for use in irrigation. More extended in use is the Persian wheel, and some of its American adaptations which have recently come into use in this country. There are several varieties of this apparatus skilfully designed and constructed which are more efficient than the old oriental wheel. These have large metal buckets hung on heavy linked chains which revolve over the wheel and dip into the source of water supply beneath, and are operated by iron cogged gearing turned by horses or bullocks attached to a shaft from the centre of these and walking around in a circle. These have capacities varying between 500 cubic feet per hour for one horse up to 2000 cubic feet per hour for four horses for a depth of 20 feet, the first cost for plant ranging from $200 to $500.

There are on the market a number of mechanical devices for utilizing animal power in pumping water, consisting chiefly of various forms of sweeps to be drawn by horses walking in a circle, or treadmills for utilizing horse, bullock, or sheep power, through gearing and shafting. Most of these are simple in construction and operation, are not liable to get out of order, and are with their pump connection capable of lifting sufficient water with a two-horse device to irrigate three to five acres per season without storage, while this amount could be at least doubled if a storage tank of sufficient capacity were provided for retaining water raised during periods when it is not wanted for immediate use. Among the more prominent makers of these devices are the American Well Works of Aurora, Ill., and Byron Jackson of San Francisco.

Of the older mechanical devices for lifting water by irrigation there may be enumerated, as among the more prominent, the mot of India, which consists of a rope passing over a pulley down into the well, and to the end of which a bucket or other receptacle is attached. This is raised by two bullocks walking away with the rope, usually down an incline, thus raising the bucket to the top of the well, where it is emptied into a distributing ditch. The Persian wheel is perhaps the most commonly employed of the various devices, both in India and generally throughout Asia and Egypt. It consists of a vertical wheel, to the outer rim of which are either attached buckets which dip into the well or over which is hung a rope which hangs below the lower periphery of the wheel and to which buckets are attached, and as these reach the upper circumference of the wheel they spill their contents into a trough which leads the water to the fields. Another old-fashioned water-lifting device is the paecottah of India, which is the sakia of Egypt and the bascule of Europe and the common well-sweep of America. By its use from 500 to 2000 cubic feet of water are raised in a day. With the mot two bullocks working 10 hours a day will raise 3¾ acre-feet in a season of 90 days. With the Persian wheel two bullocks will lift 2000 cubic feet of water a day. Still other devices of this kind, and worked like the well-sweep by man power, are the latha or scoop of India and China, the double-zigzag balance of Asia Minor and Egypt, the well chain, the noria, the tympan, and the Archimedean screw.

357. **Windmills.**—Windmills are being extensively used in the San Joaquin valley in California, and on the great plains east of the Rocky mountains and in other portions of the West, for pumping water for irrigation. They have been most extensively employed for pumping water for domestic use, but as the necessity for irrigation has become better appreciated, and as water has become more scarce and valuable, windmills have come into more extended use in providing water for irrigation. The chief objection to windmills for this purpose is their unreliability, as they are wholly dependent upon the force of the wind for their operation. This objection is not so serious on the great plains between the Rocky mountains and the Mississippi river, where there is almost always a sufficiently steady and powerful wind to keep mills constantly turning. In other places they are less certain in their action, and may

fail the farmer at the very time when he is most in need of a water supply.

Because of their uncertainty of operation, windmills should never be used for purposes of irrigation without providing as an adjunct an ample tank or reservoir for the storage of sufficient water to irrigate a considerable area. As the wind may blow at any time during the twenty-four hours, and is just as likely to blow at night as in the day, when the water cannot be used in irrigating the fields, a storage capacity sufficient certainly to impound water pumped during the night-time should be provided; though for any security, and in order to irrigate a reasonable area, ample capacity should be provided to store the water of several days' pumping when irrigation may not be necessary. This storage capacity may be obtained by using one of the various forms of elevated tanks which are supplied by windmill makers; or, better still, if the windmill can be located at a high point on the farm, an artificial reservoir may be excavated at this point and suitably lined, which shall have capacity to contain a much larger amount of water.

358. **Capacity and Economy of Windmills.**—The amount of work which a windmill will perform depends on two prime considerations: (1) the force and steadiness of the wind; and (2) the size of the wind-wheel.

It requires on an average a wind velocity of not less than 6 miles an hour to drive a windmill, and on an average winds exceeding this velocity are to be had during 8 hours per day. Hence, about one third of the total wind movement is lost for work. The reports of the U. S. Weather Bureau indicate that the average wind movement of the entire country is 5769 miles per month, or about eight miles per hour. These averages are somewhat exceeded in Dakota, where the average hourly velocity is ten miles; also in Nebraska, Kansas, and neighboring States; while they are too great for other portions of the arid West. The following table gives roughly the force of the wind for ordinary velocities:

TABLE XXVI.

WIND VELOCITY AND POWER.

Miles per Hour.	Feet per Second.	Pressure per Square Foot, in pounds.	Miles per Hour.	Feet per Second.	Pressure per Square Foot, in pounds.
6	7.5	.12	30	44.0	4.4
10	14.7	.5	35	51.3	6.0
15	22.0	1.1	40	58.8	7.9
20	29.3	2.0	45	66.0	10.0
25	36.7	3.1	50	73.3	12.3

The following table is derived from Mr. A. R. Wolff's excellent work on the windmill, and shows the capacity and economy of an experimental windmill having various diameters of wheels, with an assumed average velocity of wind of 16 miles per hour and with eight hours per day as the average number of days during which the results given may be obtained.

TABLE XXVII.

CAPACITY OF WINDMILL.

Size of Wheel, feet.	Revolutions of Wheel.	Gallons of Water raised per Minute to an Elevation of						Horse-power developed.
		25 Feet.	50 Feet.	75 Feet.	100 Feet.	150 Feet.	200 Feet.	
10	60 to 65	19.2	9.6	6.6	4.7			0.12
12	55 " 60	33.9	17.9	11.8	8.5	5.7		0.21
14	50 " 55	45.1	22.6	15.3	11.2	7.8	4.9	0.28
16	45 " 50	64.6	31.6	19.5	16.1	9.8	8.0	0.41
18	40 " 45	97.7	52.2	32.5	24.4	17.5	12.2	0.61
20	35 " 40	124.9	63.7	40.8	31.2	19.3	15.9	0.78
25	30 " 35	212.4	107.0	71.6	40.7	37.3	26.7	1.34

Mr. Wolff estimates the cost of operating a windmill for a 25-foot lift, including interest on first cost and charges for maintenance, as ranging from seven tenths of one cent per hour for a 10-foot wheel to 24 cents for a 16-foot wheel and 43 cents for a 25-foot wheel.

Aside from the uncertainty of action in windmills, it is evident from the foregoing that the windmill is one of the most economical of prime movers. Its operation calls for no expense for fuel, practically none for attendance in self-regulating mills, and little or none for repairs. In comparison a steam-engine calls for large expenditures for fuel, repairs, and attendance, while most classes of water motors call for heavy expense in providing and maintaining a supply of water, as well as for attendance and repairs. On an average it appears that the economy of a windmill is at least 1.5 times that of a steam-pump, while it has an additional economy over the latter because of the attendance and repairs demanded by the steam-boiler. On the other hand, a windmill usually calls for additional expense where it is used for irrigation in making its supply more certain by providing storage capacity.

Some interesting comparisons of the efficiency of various windmills have recently been made by Mr. J. A. Griffiths in Australia. The mills experimented with were situated in the ordinary manner directly over a well, reciprocating pumps being attached directly from a crank on the main axle of the sail-wheel, the latter being erected on the usual wooden mill tower. Mr. Griffiths assumes the following equation of energy required to stop and start a stream as being

$$E = \frac{AWv}{2g}, \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (1)$$

in which A is the sectional area in square feet of a stream of air weighing W pounds per cubic foot, and moving with a uniform velocity of v feet per second. In computing the efficiency of windmills the unit adopted is usually 100 square feet corresponding to a circle of 11.3 feet diameter, or about that of the smallest windmills ordinarily employed, and the unit of velocity is 10 miles per hour. A uniform stream of air of 100 square feet sectional area, weighing .075 pound per cubic foot and moving with a velocity of 10 miles per hour, contains an actual kinetic energy of 1,323,267 foot-pounds per hour, or .6683 of an English horse-power, and from this coefficient the energy of any other stream may be calculated. In the following table are given horse-powers of wind acting upon an area of 100 square feet for velocities ranging from 5 to 30 miles an hour:

Table XXVIII.

ENERGY OF WIND ACTING UPON A SURFACE OF 100 SQUARE FEET.

Velocity of Wind.	At Sea-level.	At 1000 Feet above Sea-level.	At 2000 Feet above Sea-level.
Miles per Hour.	H.P.	H.P.	H.P.
5	0.0835	0.0780	0.0724
10	0.6683	0.6237	0.5792
15	2.2550	2.1050	1.9550
20	5.3470	4.9900	4.6340
25	10.4400	9.7460	9.0500
30	18.0400	16.8400	15.6400

The highest net efficiency observed in Mr. Griffiths' experiments at 7 miles per hour was twenty-five per cent; also, that the velocity of the wind when leaving the mill was .909 of the approaching velocity. He further observed that the loss of velocity was proportionately less than the loss of energy, and that with a working efficiency of 10 per cent or less the loss of velocity was scarcely appreciable.

In designing a windmill for pumping, two things have to be considered—the torque, or statical turning moment, and the speed of the wheel in relation to that of the pump. The former should be as large as possible so that the mill will start with the faintest wind, and the latter must not be too fast for the pumps in a small mill or too slow in a large one. Hence the size of a mill is an important element in the arrangement of its vanes. The angle between any portion of a vane and the plane of the wheel is termed the weather angle, and to obtain the greatest torque at starting the weather angle should be the complement of the best incidence angles, or between 70 and 55 degrees. In practice it is found that the weather angle is never as great as this, being in the best examples about 43 degrees.

The following table gives the results of Mr. Griffiths' experiments for the five American-made windmills tested.

TABLE XXIX.

CAPACITIES AND EFFICIENCIES OF SEVERAL WINDMILLS.

	Stover solid Hand-control Rudder.		Perkins solid Automatic Rudder.	Althouse Folding Rudderless.	Althouse Folding Rudderless.	Carlyle Automatic Rudder.
Outer diameter of sail-wheel ....feet	11.5		16.0	14.16	10.16	9.83
Inner diameter of sail-wheel....feet	4.5		6.0	4.5	3.83	4.16
Gross area of sail-wheel....square feet	104		201	157	81	80
Weather angle at outer ends of vanes....	43°		36°	30°	28°	50°
Diameter and stroke of pump....inches	3 × 4		3 × 10¾	3 × 10	3 × 4⅝	3 × 4
Average head of water during tests....feet	29.2	61.2	39	66.3	38.7	30.7
AT MAXIMUM EFFICIENCY.						
Velocity of wind....miles per hour	5.8	6.5	6.0	7.0	8.5	6.0
Velocity of mill....revolutions per minute	13.0	13.3	7.5	12.6	20.5	12.5
Actual horse-power....	0.011	0.025	0.024	0.065	0.028	0.012
Horse-power per 100 square feet of gross area....	0.011	0.042	0.012	0.41	0.035	0.015
Maximum net efficiency....per cent	8.7	14.4	8.9	19.3	9.0	10.4
IN 100 AVERAGE HOURS, CALM LOCALITY.						
Average quantity of water lifted....gallons per hour	153.0	135.0	259.0	267.0	115.0	145.0
Average continuous horse-power developed per 100 square feet	0.022	0.040	0.025	0.057	0.028	0.028
Average continuous gross horse-power developed....	0.023	0.024	0.50	0.089	0.023	0.022
Average net efficiency....per cent	2.1	3.9	2.5	5.5	2.7	2.7
IN 100 AVERAGE HOURS, WINDY LOCALITY.						
Average quantity of water lifted....gallons per hour	287.0	271.0	525.0	540.0	237.0	270.0
Average continuous horse-power developed per 100 square feet	0.041	0.080	0.051	0.115	0.057	0.052
Average continuous gross horse-power developed....	0.043	0.083	0.102	0.180	0.046	0.042
Average net efficiency....per cent	0.28	1.11	0.71	1.59	0.78	0.72

359. **Varieties of Windmills.**—A wind-wheel is designed for the utilization of wind-power much as is a water-wheel for that of water-power, but differs from it in that the former is wholly immersed in a sea of air while the latter is acted upon by a limited current of water. The common paddle-wheel will not be revolved by the wind, because its force is exerted equally on diagonally opposite paddles ; hence the paddles on one side only should be exposed to the wind—a result which is obtained without the use of a screen by pivoting the paddles upon their axes so that on one side they present a broad face to the wind and in passing to the other side they turn on their pivots so as to present a feather edge to the wind.

Windmills consist of arms, cross-bars, and clothing therefor. They are made plane, warped, or concave. The older type of sail-mill is common in Europe, especially Holland, and have usually four arms, occasionally more. The narrow part of the sail is usually covered with a wind-board, as it is called, and the broader with wind slats of wood or with a covering of sail-cloth. American mills differ from European mills in that they are chiefly of the propeller type, and instead of a small number of sails of considerable width are made with a great number of blades or slats of slight widths, and otherwise have an entirely distinct appearance from the European mill, as the wheel presents a closed surface as compared with the large open spaces between the arms of the sail-mill. As a result of this mode of construction the American mill is lighter in weight as well as appearance than the European mill ; and though the wide angle of the vane is not as advantageous as in the sail-mill, the surface presented for a given diameter is sufficiently great to more than compensate for this difference, and it would appear that the American mill is superior to the European type from the fact that it is rapidly replacing them in Europe.

The several types of American mills are distinguished both by the form of the wheel and the mode of regulating or governing its position and direction so as to obtain a uniform power and rate of revolution under varying wind velocities. There are two principal types of these mills, namely, (1) sectional wheels with centrifugal governor and independent rudder, and (2) solid wheels with side-vane governor and independent rudder. Besides these there are a number of special types, including various combinations of solid and sectional wheels with various arrangements of rudder, or in some cases no rudder is employed, and the wind pressure upon the wheel is relied on to bring it into direction.

The windmill is usually placed directly over the well on a wooden or iron tower or scaffolding, having usually four upright inclined pillars which straddle the well. This tower should be sufficiently high to raise the wheel at least 10 feet above all obstructions, as buildings, trees, etc. At the top of the scaffolding is a platform or a turntable with an open center, through which the pump rod descends vertically to a reciprocating force-pump in the well. A horizontal crank-shaft supported in bearings on the upper movable part of the turntable is connected with the pump by a swivel-joint in order to permit of the rotation of the mill top necessary in adjusting the sails to the various horizontal directions of the wind. An overhanging end of the crank-shaft carries the wind-wheel, which in some forms is on the lee side of the tower, in which case it maintains its direction perpendicular to the wind by pulling the turntable around, and is rudderless. In others the pressure of the wind on a rudder vane with sufficient area and leverage to overbalance the wind keeps it perpendicular to the wind, and on the windward side of the tower. In addition there is also a controlling or regulating gear to stop the mill when the reservoir is full or repairs are necessary, and to prevent damages by gales. This gear is operated by hand or is automatic.

Of side-vane governor mills the Corcoran and Eclipse are excellent examples. Of centrifugal governor mills the Halladay and Althouse are good examples. The latter is folding and rudderless. Of special mills the Buchanan is a good example, being dependent for its regulation on the tendency of the wheel to go into the direction it turns as the velocity of the wind increases. The Stover is a solid sail wheel with vanes so regulated that the mill may be reefed or even stopped or otherwise regulated to go slowly in heavy winds. The Perkins mill has a solid wheel with automatic rudder, which also acts as an automatic regulator, though in slow winds it must be half reefed. The Carlyle special mill has a rudder arranged to reef the sail in storms, and so attached by an adjustable cam as to cause the centre of gravity of the rudder to rise as it falls toward the wheel. The Leffel windmill depends for regulation on the fact that the centre line of the wheel shaft stands off from and parallel to the plane of the rudder, while the wheel of the mill is of peculiar type, being made of metal blades with a helical curve. The Cyclone and Woodmouse are two excellent types of modern American mills. The Advance is perhaps the best make of automatic regulating rudder mill having both side steering-vane and governing rudder.

360. Value of Windmills as Irrigating Machines.—Windmills average in cost from \$50 to \$400, according to size and make. Any one intending to use a mill should purchase it from a reliable maker, and choose a design according to the work which the mill is expected to perform and the average wind velocity in his locality. Thus, on the Rocky mountain plains a windmill will run at least twelve hours in a day, while in some of the mountain valleys between the Rocky mountains and the Pacific slope eight hours per day is an average run. Experience with average mills already constructed shows that a 5-inch pump will discharge about 250 cubic feet an hour, a 6-inch pump about 380 cubic feet per hour, and an 8-inch pump about 650 cubic feet per hour. On the average basis of duty of water a 5-inch pump will therefore irrigate about 6 acres if running constantly or about 2 acres if running one third of the time.

The average mill will, according to experience, do sufficient work to irrigate from 1 to 3 acres. If, however, a storage reservoir or tank be supplied this may be emptied and filled several times during an irrigating season, which contains several irrigating periods, and a mill supplied with such a reservoir may therefore irrigate from three to five times the area above indicated. There are numerous windmills in the West which irrigate from 10 to 15 acres from wells 30 to 150 feet in depth with the aid of storage tanks, and these plants, including mill and tank, average in cost from $150 to $350. Taking $250 as a mean and their capacities at $12\frac{1}{2}$ acres, the cost of these plants is about $20 per acre irrigated and the cost for maintenance practically *nil*. A 25-foot mill will, according to Table XXVII, in a working day of 8 hours, pump one third of an acre-foot to a height of 25 feet, or one sixth of an acre-foot to a height of 50 feet. In an irrigating season of 120 days it will raise 40 acre-feet to the lower elevation, and were storage provided for half this volume and the reservoir filled before the beginning of the irrigating season, such a mill would theoretically, under average wind conditions, be capable of furnishing enough water to irrigate from 20 to 30 acres. It is doubtful, however, if such a duty will be practically obtained even with the most ample storage facilities.

361. **Water Motors.**—Water acts as a motive power by its weight or by its impulse. In the former case it falls slowly through a given height, and in the latter it passes through the machine with a constantly decreasing velocity. The work P which it performs because of a given fall h is

$$P = Qwh, \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (2)$$

in which Q is the whole quantity of water falling in one second of time and w the weight of a unit of volume. If v is the velocity with which it enters, then the work which it performs because of impulse, before coming to rest, is

$$P = Qw\frac{v^2}{2g}, \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (3)$$

but water started from rest will attain a velocity v only after it has passed through a height $h = \frac{v^2}{2g}$; hence in the latter case the formula may be written as is (2).

Hydraulic motors are machines designed to utilize the energy possessed by falling or moving masses of water, and may be divided into the following two classes: (1) water-wheels and (2) water-engines. These may be again divided into several classes, in some of which both water-wheels and water-engines act either through power due to fall or due to impulse, or a combination of both. Water-wheels may be subdivided into two classes: (1) vertical water-wheels, and (2) horizontal water-wheels. Of the former we have the more common of the old-fashioned wheels:

1. Undershot water-wheels.
2. Breast-wheels.
3. Overshot water-wheels.
4. Hurdy-gurdies.
5. Pelton water-wheels.

The latter is a modern adaptation of the old-fashioned hurdy-gurdy, and is properly an impulse wheel. Horizontal wheels are turbines of various types, and in these, like vertical wheels, water may act both by pressure or impulse, or by a combination of the two.

Water-engines may be divided into three classes:

1. Bucket-engines.
2. Rams.
3. Water-pressure engines.

The first is an antiquated form of motor by which work is performed by allowing water to enter buckets, thus causing these to descend vertically. Rams utilize the impulse due to the weight of a large body of water in forcing a smaller body to a desired height. Water-pressure engines utilize the fluid properties of water in a manner somewhat resembling the operation of steam-engines—producing a reciprocal motion through the pressure of water confined in an upright pipe.

Assuming one has to elevate a given quantity of water to a fixed height, having at one's disposal an hydraulic force which it is desired to apply to the work, we may divide the problem into two parts: (1) The fall may be distinct from the water which is to be lifted, or (2) the question of fall may be inseparable from the water to be elevated. In the first class are included all water-wheels and bucket-engines; in the second are included rams and most forms of water-pressure engines. In either case it may be necessary to raise a large volume to a small height or a small volume to a large height, and each of these offices may be performed through a large volume of power from a small height, or a small volume of power from a great height. It is this intimate connection between volume and head of power and of resulting work which calls for the exercise of ingenuity and discretion on the part of the engineer in choosing the water motor by which this work is to be done. Thus, a ram is nearly always called upon to utilize a large volume of power from a small height to raise a small volume of water to a great height. The Pelton water-wheel generally utilizes a small volume of power from a great height to elevate varying quantities of water to varying heights.

362. Undershot Water-wheels.—The word water-wheel is usually applied to the various old-fashioned vertical wheels, undershot, breast, and overshot wheels. Undershot wheels may be classified as midstream wheels, the common undershot wheels, and Poncelet wheels. In midstream wheels the motive power is due to the velocity or impulse of the current of water in the stream in which the wheel is set, and such wheels are employed almost exclusively for the elevation of water for irrigation. They are very simple in construction and operation, and may be advantageously employed where water is abundant, even in streams having the very slowest velocity of flow.

Midstream wheels produce the greatest power for the smallest diameter of wheel when the float-boards are made straight, but not radial. They vary from 12 to 16 feet in diameter, but are not infrequently larger, the float-boards varying from 9 to 12 in number, while two of them at least should always be immersed at the same time. These float-boards project from 24 to 30 inches from the wheel-rim, and dip into the water about half of their depth. In rivers where the water-level fluctuates, the axle of the wheel is made movable on its supports to render it capable of being raised or lowered at pleasure to suit the height of water-level, and this is effected by resting one or both extremities of the axle on floats. The horse-power of a midstream wheel may be calculated by the following formula from Mr. P. R. Bjorling:

$$HP = (v - v_1)\ .0028Av, \quad . \quad . \quad . \quad . \quad . \quad (4)$$

in which v is the velocity of the stream in feet per second, v_1 the mean velocity of the float-boards in feet per second, and A the immersed area of the float-boards in square feet.

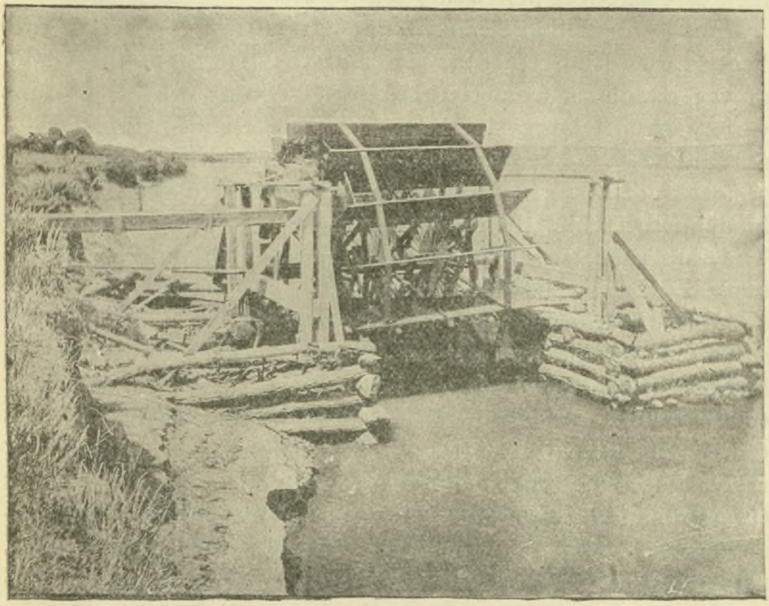

FIG. 121.—UNDERSHOT WATER-WHEEL OR NORIA.

Numerous wheels of this class have been successfully employed in pumping water for irrigation in various portions of the West. In some cases these wheels have attached to their outer rim a row of buckets (Fig. 121), which dip into the water as the wheel revolves, are thus filled, and then as they reach the upper portion of their revolution spill their contents into a trough which leads to the irrigating ditches. Such wheels are called norias, and are of very ancient origin, having been used for ages in all portions of the world, most extensively, perhaps, in Egypt and Italy. At other times midstream wheels are suspended directly in the stream current, and by means of gearing or belting are connected with pumps which elevate the water for irrigation. Such contrivances have been employed in the West, one of which, on the Platte river, has a 10-foot wheel, 14 feet broad on the face. It runs a 3½-inch centrifugal pump, and is said to elevate 2½ second-feet of water to a height of 16 feet, or 5 acre-feet per twenty-four hours.

The average diameter of the midstream water-wheel of the West varies from 10 to 20 feet and the length of the blade of the paddle is from 6 to 10 feet. Some wheels of this variety but of large size have been successfully employed—notably on the Green river in Colorado—which are from 20 to 30 feet in diameter. These are hung on wooden axles 5 inches in diameter, while their paddles dip 2 feet into the stream. They are used as norias, for on their outer circumference are buckets of wood, having an air-hole in the bottom closed by a suitable leather flap-valve which permits the bucket to fill rapidly by forcing out the air. These buckets are 6 feet in length and 4 inches square, and have a capacity of a little less than a cubic foot each. The largest of the wheels on the Green river have 16 paddles and lift 10 cubic feet of water per revolution, and as they make two revolutions a minute, though they spill a large portion of their contents, each wheel handles about 4000 cubic feet per day, or approximately 1/10 of an acre-foot.

Common undershot water-wheels, as distinguished from midstream wheels, are the best where a fall of convenient height cannot be obtained, and the velocity of the water is yet relatively great. These are confined in a channel which is made about the width of the wheel and is wider at the inlet than at the wheel so as to give freedom of access to the water and to increase its velocity. These wheels operate most satisfactorily where the fall is from 1/2 to 2 feet in the course of the race. The paddles are similar to those for midstream wheels, though sometimes they are curved and of iron. The number of float-boards or paddles for such a wheel may be determined by the formula

$$n = \frac{4d}{3} + 12, \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (5)$$

in which n is the number of float-boards and d the diameter of the wheel. These wheels vary in diameter from 10 to 20 feet, and are usually constructed of from 30 to 40 paddles, varying from $1\frac{1}{2}$ to $2\frac{1}{2}$ feet in depth, their length being from 3 to 6 feet. The power developed by such a wheel may be ascertained by the formula

$$HP = .0006qh, \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (6)$$

in which q is the quantity of water in cubic feet per minute, and h the head of water in feet.

Poncelet wheels act rather on the turbine principle, their paddles being curved. They are usually immersed to the height of their axes, and the water is screened from them with the exception of a few inches near their under surface, so that it impinges by impulse against the under side of the wheel and acts much as does a turbine. The power they are capable of generating may be expressed by the formula

$$HP = .068qh. \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (7)$$

Breast-wheels are placed where there is a considerable fall in a manner similar to Poncelet wheels, so that the level of water is about at the height of their axes. They have usually curved paddles or buckets, and the water impinges against them both by weight and impulse at a point just below the axial line.

363. Overshot Water-wheels.—Overshot wheels are more economical in their use of water, and are therefore employed where water is scarce. In these the water is delivered above the wheel by means of a flume, race, or penstock, and they are so constructed that the water may be delivered either on the near or the far side of the wheel, according to the arrangement of the outlet gates controlling the supply. On the outer circumference of the overshot wheel is a series of buckets into which the water pours and by its weight causes the wheel to revolve. As the wheel turns each bucket fills as it passes the inlet orifice and empties as it approaches the bottom, so that on one side are always a certain number of buckets filled with water. This class of wheels may be employed in falls of from 6 to 60 feet, and with streams having from a few up to 50 second-feet discharge. In order to lose as little of the fall as possible the bottom of the wheel should approach close to the lower water surface, but should not dip into it, as by drowning the wheel its power is diminished. These wheels are made of such width as to permit of their buckets being large enough to hold a considerable weight of water.

The buckets of overshot wheels may be made of straight boards or sheets of metal having two or three bends in them, or may be curved. The number of buckets may be calculated by the following formulas given by Bjorling: For wheels from 12 to 20 feet in diameters,

$$n = 2.1d; \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (8)$$

and for wheels 25 to 40 feet in diameter,

$$n = 2.3d. \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (9)$$

The depth of shrouding for these wheels is about 12 inches, and the bucket opening is about 1/3 of a square foot for each cubic foot of bucket contents, or is about 7 inches in width. The quantity of water in cubic feet per second being Q, the effective horse-power of such a wheel may be gotten by the formula

$$HP = .085Qh. \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (10)$$

Overshot water-wheels may be employed to operate through gearing or belting any of the usual forms of reciprocating or centrifugal pumps, and will elevate volumes of water and to heights proportioned to the power they are capable of developing.

364. **Turbine Water-wheels.**—Turbine wheels may be divided into three classes, according as they are acted on (1) through impulse, (2) through pressure, and (3) through reaction. Impulse wheels have plain or concave vanes or float-boards, on which the water strikes more or less perpendicularly. Pressure-wheels have curved float-boards along which the water glides. Reaction wheels consist of an arrangement of pipes from which water issues tangentially. To this latter class really belong Pelton wheels, which are vertical reaction wheels.

While pressure and reaction wheels are similar in construction, they differ in that in the former the passages between the vanes are not completely filled with water, while in reaction wheels the water fills and flows through the whole section of the discharge-pipe. In impulse wheels the water spreads over the vanes in all directions, while in pressure and reaction wheels it flows only over one side. Turbines are again distinguished as (1) outward, (2) inward, and (3) mixed or parallel flow turbines. The former receive the water at the centre and deliver it at the periphery of the revolving wheel, the regulating apparatus consisting of a ring inserted between the outer periphery of the guide-blades and the internal periphery of the revolving wheel. In inward-flow turbines the motion of the water, as the name implies, is practically the reverse of that for outward flow. Turbines possess an advantage over vertical water-wheels in that they may be used with any fall of water from one foot to several hundred feet. The efficiency of turbines differs with the height of the fall. It is less for small wheels and high falls and greater for large wheels and small falls. Overshot wheels frequently attain a greater efficiency than turbines with falls of 20 to 40 feet, while with falls of 10 to 20 feet their power is about the same; but turbines possess a great advantage over other, except Pelton, water-wheels in having the same efficiency under different falls and volumes, and in that they can be regulated. Sometimes the whole power may be required of the turbines, at other times only a part may be required; sometimes water is scarce, and at other times abundant; but the regulating apparatus is such that the efficiency remains nearly the same. The chief differences between turbines and vertical water-wheels are that the turbines may be drowned, but vertical wheels must be elevated above the water in the tail-race; the turbine takes its supply at the bottom of the fall and the water-wheel at the top or beginning of the fall, and therefore the former obtains nearly the whole pressure due to the head or height of the fall; turbines work without material loss of energy when drowned and move with a greater velocity than vertical water-wheels, and hence may be reduced in size and weight for equal power.

The outward-flow turbine is more popular in Europe, and is generally known as the Fourneyron type of turbine. The horse-power of this class of turbine may be roughly determined by the formula

$$HP = \frac{qh}{700}, \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (11)$$

in which q is the quantity of water flowing through the pipes of the turbine per minute. Inward-flow turbines are more popular in the United States. They are the reverse of the Fourneyron type in that the guide-wheel surrounds the revolving wheels and after the water has passed the buckets it is gradually deflected downward by the curved under side of the revolving wheel. Mixed and parallel flow turbines are generally known as the Jonval or Girard type of turbine. They may be fixed at any convenient distance above the tail-race, and must have sufficient water above the guide-blades to allow it to enter freely without eddies. The horse-power of this type of turbine may be roughly determined by the formula

$$HP = \frac{Qh}{.079}, \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (12)$$

in which Q is the quantity of water available in cubic feet per second.

Of the American makes of water-wheels probably the two most extensively employed are the Victor turbine and the Leffel turbine, though a number of other types are manufactured. These wheels have been extensively employed for all the various purposes to which power may be applied, and a number of pumping plants for irrigation operated by such turbines have been erected in the West. These turbines come in sizes and powers ranging from a few inches in diameter under a head of but a few inches, and capable of developing as little as one horse-power, up to the enormous sizes which have recently been built for the Niagara Water Power Company, and other similar concerns, which are capable of developing as much as 2000 horse-power, and which may be operated under several hundred feet of head, and have diameters as great as 6 or 8 feet. They range in price likewise from one hundred to several thousand dollars.

There has recently been erected at Prosser Falls, Washington, a turbine power and pumping plant capable of irrigating 4000 acres, besides furnishing power for factories and electric-lighting purposes. The water is pumped to an elevation of 100 feet with a power-producing fall of 20 feet, and is delivered to the turbines through a flume 10 feet deep by 12 feet clear width, and carrying 6 feet in depth of water at low stages. The turbines are 48 inches diameter, of special Victor type, and there are two of these, each capable of developing 135 horse-power under 12 feet head. They operate a pair of Duplex pumping engines of 25-inch cylinder and 24-inch stroke, each having a capacity of 4000 gallons a minute. A third pump is to be installed, and when erected the entire plant will have a daily capacity of about 45 acre-feet.

365. **Pelton Water-wheels.**—Pelton water-wheels are simpler in construction than turbine wheels and less liable to be clogged or get out of order, while they can be worked under much greater heights of fall than can turbines. They are vertical, tangential reaction wheels, and power is derived from the pressure of the head of water supplied by a pipe which discharges upon the wheel buckets on the lower side of the wheel through a small nozzle, as was the old hurdy-gurdy of California mining days operated by the discharge through a hydraulic monitor. Pelton wheels are not recommended for

heads less than 30 to 50 feet, as below these heads turbines are usually more efficient, though for the production of small powers, say 10 or 20 horse-power, a Pelton wheel will give as great efficiency as any other wheel with heads of 10 to 30 feet. But above 50 feet head and up to 2000 feet a Pelton wheel is best, as no other wheel produces anything like the same efficiency or works with equal simplicity. These wheels are adapted to a wide range of conditions of water supply, producing power under the most varying conditions with almost equal efficiency. This is accomplished by simple change of nozzle tips, by varying the size of stream thrown upon the wheel, the power of which may thus be varied from maximum to 25% without appreciable loss. The buckets being open there is no uncertainty or annoyance from derangement of the parts, or stoppage by driftwood or other substances in the water. They are relatively cheap of instalment, and may utilize the water from a small spring or creek as well as from the largest source of supply. These wheels admit, by varying their diameter, of being placed directly on the crank-shaft of power pumps without intermediate gearing or connections. The efficiency of Pelton wheels is perhaps a little higher than that of turbines, the latter attaining efficiencies of 60% to 80%, while Pelton wheels not uncommonly attain efficiencies of 70% to 85%.

The buckets which are on the periphery of a Pelton wheel —the latter, by the way, is narrow, not having the broad diameter of other vertical wheels—are of metal, cup-shaped, and divided into two compartments in such way as to develop the full force of the impinging stream, while in passing out the water sweeps the curved sides with a reactionary influence, giving it the effect of a long impact. The power of this wheel does not depend upon its diameter, but upon the volume and head of water supplied. There are practically three types of Pelton wheel—single nozzle, double nozzle, and multiple nozzle,—though there are in addition a number of patented attachments to the wheel, giving it various names, as the Hett Pelton wheel, the Caudle Pelton wheel, and others, which have no especial advantage over the more ordinary forms. The single, double, and multiple nozzle wheels, as their names imply, may have one or more nozzles from which water is directed tangentially at various parts of the wheel.

As yet few Pelton wheels have been employed in pumping water for irrigation, but it is not improbable that in the near future their value in utilizing small volumes of water from great heights for this purpose will become better appreciated. Several wheels of this character have been erected, both for the production of very small and very large power. Some are so small as to develop power from the supply of a house faucet, running little pumps for filling water tanks for domestic use. At the North Star mine in California one of the largest has recently been constructed for pumping which is operated under an effective head of 750 feet, the wheel being 18½ feet in diameter, of 300 H.P. capacity, and mounted on a bicycle-like spoked frame having a 10-inch steel shaft connected directly with air-compressing engines with a speed of 300 feet a minute. The peripheral speed of this wheel is 6000 feet a minute, and its efficiency is believed to be as high as 90%. Another Pelton water-wheel working under 810 feet head, the wheel being but 40 inches in diameter, develops 600 horse-power; and still another wheel 57 inches in diameter under 1410 feet head develops 500 horse-power.

366. Uses of Water-power.—Turbine and Pelton water-wheels, and in fact simple vertical water-wheels, may be utilized in connection with irrigating plants in a reverse manner to that just described; that is, not for pumping water for irrigation, but the water which flows in gravity canals may be utilized to develop power for various economic purposes. On the lines of nearly all canals there is wastage of power in falls introduced at the headworks, at the outlets of storage works, and at various points where falls are introduced to neutralize the surface grades. At such points as these turbines or Pelton wheels may be employed to generate power which may be utilized for manufacturing purposes, for electric lighting or transmission of power, or for pumping water from low-service canals to high-service lines which will bring under irrigation areas which would otherwise go without water supply.

The water in the Folsom canal in California is used to develop power through four pairs of turbines, each pair having a capacity of 1260 horse-power at 300 revolutions per minute, operated under 55 feet head. They develop 5000 horse-power for electric transmission, the remainder being employed for power at the site. On the Santa Anna canal it is proposed to employ some of the water energy wastefully lost in the falls in developing electric transmission and lighting in San Bernardino valley. On the Arizona canal is a great fall introduced to reduce grade, which is to be used in producing water-power. In Italy and India irrigation water is thus employed for developing power. At the Fife dam in Bombay, and notably near Cigliano, Italy, where the water flowing in a feeder of the Cavour canal is thus employed to lift water to a high-level canal for further use in irrigation. There are diverted from the stream three canals, between the two lower of which is placed an extensive turbine pumping plant which receives its water from the upper of these two canals and tails into the lower canal, whence it is distributed for the irrigation of low-lying fields. The third or uppermost of these canals through a wrought-iron pipe three feet in diameter supplies water under head of 66 feet to the pumps below, and these elevate it through a pipe of the same diameter to a total height of 140 feet, where it is emptied into a fourth canal, which distributes it for irrigating the upper fields.

367. Water-pressure Engines.—Water-pressure engines, as their name implies, develop power from the pressure of water confined in upright pipes. They are used to convert stored into active energy, as the water acts by its weight alone, in occasional instances only being assisted by momentum. Their motion is periodical and reciprocating, and they are therefore serviceable when not required to develop rotary motion as in operating direct-acting pumping engines. In using water-pressure engines care must be taken to prevent sudden checking of the descending volume of water, and to this end escape-valves.

air-vessels, accumulators, or other means of iessening shock which the engine sustains are provided.

The principal parts of such a machine are the reservoir, the supply-pipe which conducts the water from the reservoir to the working cylinder, in which latter work is performed by forcing upward a loaded driving piston. The cistern is relieved of water after the work is performed by a discharge-pipe. A regulator is placed in a connecting pipe which joins the working cylinder with the supply-pipe. These engines are made single and double acting, and may have one or two cylinders. In the single-acting engine the piston is moved by water-pressure in one direction only, and in the opposite direction by its own or added weight. In the double-acting engine the piston is moved in both directions by the force of water. There are many designs and many makes of these engines, and it may be said of them that they are better suited for pumping in mines where a great fall is readily obtainable and a comparatively small volume is to be elevated to a considerable height than they are for irrigating purposes. There are a few forms of water-pressure engines which are rotative, and these are similar to steam-engines with fixed cylinders, connecting-rods, and fly-wheels, but have no flap on the slide-valve. There are also a few designs of oscillating water-pressure engines, worked on the principle of the old oscillating marine engine; yet none of these can be recommended especially for pumping water for irrigation. There is not much difference between rotary water-pressure engines and impulse turbines other than that the former are less economical than the latter, which are generally larger and more efficient.

No exhaustive experiments have been made to ascertain the performance and economy of these engines. Where they are utilized under the most advantageous circumstances their efficiency is not infrequently as high as 70% or 80%. Water-wheels have an advantage over water-pressure engines in their simplicity and cheapness. Under small pressures water-wheels are preferable. Under great pressures pressure-engines may attain higher efficiencies, but they are much more elaborate and expensive machines as compared with most water-wheels, except the Pelton. It may be said that under great pressures, where it is desirable to make the best use of the water-power available, water-pressure engines are most economical, but where water-power is abundant, and it is desired to economize cost, turbines have the advantage.

368. **Hydraulic Rams.**—Where there is a slight fall and it is desired to raise small amounts of water, the hydraulic ram is extremely simple, useful, and economical. It works on the principle of a large volume of water having a small fall forcing by impulse blows a smaller volume of water to a higher elevation. Hydraulic rams owe their action to the shocks which are so objectionable in water-pressure engines. There are three classes of such machines: (1) those having no air-vessel in direct communication with the drive-pipe; (2) those having an air-vessel in direct communication with the drive-pipe; and (3) pumping-rams. Water is delivered to the ram from a reservoir or a stream with steady flow through a supply or drive pipe, at the end of which is a check-valve opening into a chamber connected with the discharge-pipe. In the drive-pipe, near the check-valve, is a weighted pulse or clack valve which opens inwards, and the length of stroke of which is capable of regulation. Supposing the water at rest in the machine and the delivery-valve closed and the pulse-valve open: The water that passes through the drive-pipe flows with a velocity due to the height of fall out through the pulse-valve, which it almost im mediately closes. At the moment the issue of water ceases a ramming stroke is created which opens the delivery-valve and permits the water to enter the air-vessel, and at the same time, in consequence of the shock to the delivery-valve and by virtue of its elasticity, the water flows back through the drive-pipe. At the moment the backward motion begins the delivery-valve closes and the pulse-valve opens to allow the passage of water from the drive-pipe. The alternation of these effects is continuous.

A general rule for the discharge of hydraulic rams is that about one seventh of the supply volume of water can be elevated to a height five times that of the fall, or one fourteenth part may be elevated ten times the height of the fall, etc. The fall should range from 2 to 10 feet, but not above this, owing to the wear and tear produced by the ramming stroke. Among the advantages of hydraulic rams are their small first cost and very small cost for maintenance; also, that they are unaffected by tail water, and may continue working even when flooded. They are not economical in water, as it takes a large volume to do a small amount of work. Their efficiency is low, and as the height to which the water is elevated increases the efficiency decreases. The efficiency of a well-designed machine working under the most advantageous circumstances may be as high as 66%. Perhaps with a working head of 4 feet no water would be delivered at 80 feet, so that where there is great difference between the delivery and supply head only a small percentage of water is pumped. Let H = height to which water is elevated and h_1 = head consumed in friction and hydraulic resistance; then for the actual work done

$$P = w(H - h_1), \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (13)$$

in which w = weight of water elevated. The efficiency of a ram may be expressed by the formula

$$E = \frac{qh}{QH}, \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (14)$$

in which q = quantity of working water in gallons, Q = quantity of pumped water in gallons, h = head of working fall, and H = height to which water is pumped. The diameter of the drive-pipe may be gotten by the formula

$$d = .058 \sqrt{q}. \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (15)$$

As to the length of drive-pipe, this should be increased as the height to which water is to be lifted is increased, so that water shall not be forced back into the supplying channel when the pulse-valve closes. A general rule is to make its length equal five to ten times the height of fall. Its inclination should vary from 1 in 18 for small falls to 1 in 4 for high falls. The diameter of the delivery-pipe is usually from 1/3 to 1/4 the area of the drive-pipe. In placing a hydraulic ram care should be taken to protect it from frost by placing it and the pipes underground. A strainer should be secured at the upper end of the drive-pipe and a stop valve or cock placed in it in order to control its action.

There are many designs of hydraulic rams on the market, and the one most suitable for the work required should be chosen. In general it may be stated that but few of these are sufficiently powerful to pump water for irrigation purposes. Most of those which can be used for such purposes partake more of the nature of hydraulic-ram engines than of hydraulic rams proper; that is to say, they are so designed that they may be actuated by dirty water as well as clear, and are more intricate in their construction and valve arrangement than is the simple ram above described. One of the best hydraulic rams made for pumping large volumes is the Rife hydraulic engine. This is claimed by the maker to elevate water 25 feet for every foot of fall, and to deliver one third of the water used to operate them $2\frac{1}{2}$ times the height of fall; one sixth of the water 5 times the height of fall; etc. The largest of these only are capable of elevating enough water for irrigation purposes. Those having a drive-pipe 8 inches and a delivery-pipe 4 inches in diameter are capable, under a head of 10 feet and utilizing one second-foot of supply per minute, of elevating about $\frac{1}{2}$ acre-foot per day of 24 hours to a height of 25 feet. Such a machine costs $500, or at the rate of about $10 per acre irrigated for first cost of plant, and practically nothing for operation.

369. **Hot-air and Gasoline Pumping-engines.**—Hot-air pumping-engines depend for their operation on power developed by the expansion of heated air without the interposition of steam or other agency to convert the heat into motion. Gas and gasoline engines are likewise operated without converting the heat produced by combustion into steam, but de-

pend upon the expansive force produced by the explosion of gas or of gasoline converted into gas when brought in contact with air. Both of these are patented and manufactured mechanisms, to be obtained from the various makers. They are only made to develop comparatively small powers, and therefore can be utilized in pumping comparatively small volumes of water, capable of irrigating, perhaps, 5 to 50 acres. They have under certain conditions decided advantages over water and steam motors in that they can be employed where there is not a sufficient water supply to operate a water motor, utilizing, as they do, practically no water, and therefore being able to pump all that is available for irrigation. They may be employed where steam-pumps cannot be, both because of their economy in water consumption and because of the kinds of fuel which they may use; gasoline being serviceable in arid regions where fuel is expensive and difficult to obtain, and hot-air engines being capable of utilizing any variety of fuel. Again, they are small and compact, and simple of erection by comparatively unskilled machinists, and can be operated at the least expense for supervision, and with the least skill on the part of the operator.

Hot-air engines are constructed almost wholly as pumping-engines, and the motive power and pumping apparatus are combined in one machine inseparably connected. Gasoline engines come, as do other motive powers, independent of the pump, and therefore capable of utilization for doing other forms of work; but they are also made as gasoline pumping-engines, both pump and motive power being combined in one mechanism. The only form of hot-air pumping-engine on the market is the De Lamater hot-air engine. Many thousands of these machines are in use, chiefly for pumping small quantities of water in cities for manufacturing or domestic uses, only a few being employed in pumping water for irrigating. They are so simple of construction that any one capable of lighting a match may operate them. There is no possibility of explosion, as may occur through carelessness with a gasoline engine. When once started they require no further attention than the replenishment of fuel. These engines are made with capacities ranging from a few gallons a minute up to one second-foot, equivalent to .2 of an acre-foot per day of 24 hours, limited by the height of lift, which varies from a few feet to 500 feet. One of the objections to hot-air pumping-engines is their great first cost, which for the largest sizes—for example, those capable of pumping .2 second-foot—is $600, or for plant alone, $3000 per second-foot, equivalent to from $50 to $75 per acre irrigated.

Gasoline engines are used extensively in some portions of the West, notably in Kansas, for pumping water for irrigation. Among the better makes of these engines are the Weber and Oriental gasoline engines, These are made of various dimensions up to those capable of developing 50 H.P., and pumping a correspondingly large volume of water, and they are constructed as combined motive and pumping plants, or as separate motors to be attached to varying forms of pumps. The chief advantages which these machines have over other motive powers for pumping are their compactness and simplicity of installation and operation, but above all their cheapness, not so much for first cost as for ultimate maintenance, though in this latter item they do not surpass hot-air pumping-engines. The largest of these engines are capable of elevating for low heads as much as 3 second-feet of water, or 6 acre-feet per day of 24 hours, and lesser quantities to greater heights in proportion. The cost of operation of such a machine as this has been for gasoline as low as $1.25 per day, or $0.20 per acre-foot, and these figures apply to portions of the West, where gasoline is most expensive. To put it another way, these engines will pump water at a cost of about one cent per hour; and working ten hours a day, the largest size will elevate sufficient water to a height of 20 feet to irrigate about 320 acres if storage be provided. The first cost of the plant is from $400 to $600 for engines capable of irrigating 10 to 20 acres, and larger plants in proportion. This is at the rate of about $30 per acre without storage, and the cost of operation is about $1.25 per acre irrigated.

370. Pumping by Steam-power.—There are many forms of motors designed to utilize steam-power in pumping. These may be divided into the following three classes:

1. Those in which water is elevated directly by steam, as injectors and pulsometers.

2. Those which utilize the power developed by steam through an engine indirectly by gearing and belting or other separable connection.

3. Those which utilize steam-power through an engine directly, as direct-acting or fly-wheel pumping-engines. In considering steam as a motive power it is unnecessary to refer to any of the forms of steam-engines and boilers employed in developing this power, as their name is legion, and they are manufactured in all varieties, forms, sizes, and prices. Under the title of Pumps direct-acting engines will have to be considered, because the motive power is a portion of the pumping mechanism. Such is also the case with pulsometers, vacuum-pumps, and injectors.

The only feature of steam as a motive power which it is desirable to consider is the cost and economy of producing power for pumping purposes and the amount of work which a given power will perform. This consideration of power in elevating water is one which bears the same relation to its other forms as does the power produced by water or by air, and therefore the facts here developed have immediate bearing on the powers produced by water-motors as well as steam-motors. According to Mr. J. T. Fanning, the power P in foot-pounds required to produce a given flow of water by pumping is

$$P = QWh, \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (16)$$

in which $Q =$ volume in cubic feet of water to be set in motion, $W =$ the weight of a cubic foot of water in pounds, equal to 62.5 lbs., and $h = \frac{v^2}{2g}$ is the height in feet to which the rate of motion is due.

The power required to accelerate the motion of water is in addition to the dynamic power p in foot-pounds required to lift it through a height H of actual elevation ; and the equation of lifting per second disregarding frictional resistance, is

$$p = QWH, \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (17)$$

or for any time

$$p = tQWH. \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (18)$$

The frictional resistance F to flow in a straight pipe is proportional about to the square of the velocity of flow, and is computed by some formula for friction head h_1, among which, for lengths exceeding 1000 feet, is

$$h_1 = \frac{4lFv^2}{2gd}, \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (19)$$

in which l is the length of pipe in feet, while the coefficient F may be derived from the tables of resistance to flow in pipes given in Chapter XIV.

The equation of power p_1 required to overcome the frictional resistance to flow in pipes, in which h_1 is the vertical height of lift in feet equivalent to frictional resistance, is

$$p_1 = QWh_1 = QW\frac{4lFv^2}{2gd}. \quad . \quad . \quad . \quad . \quad (20)$$

As one horse-power = 33000 foot-pounds per minute,

$$HP = \frac{PS}{33000}, \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (21)$$

in which P is the power in foot-pounds required to produce a given flow and S is the number of strokes of the pump per minute.

The horse-power required to overcome the combined dynamic lift and frictional resistance to flow for a given time t, is

$$HP = \frac{tQW(H+h_1)}{33000}. \quad . \quad . \quad . \quad . \quad . \quad (22)$$

371. Centrifugal and Rotary Pumps.—For lifting large volumes of water to moderate heights the centrifugal pump excels in economy, efficiency, and simplicity of construction, and in cost both for plant and its erection. Where circumstances are suited to its employment, it is perhaps the best pump for irrigation. Being valveless, it is well adapted to raising water containing sediment or foreign matter. It is continuous in its action, and is easily erected and operated by machinists of moderate skill.

A centrifugal pump is essentially an outward-flow turbine driven in the reverse direction. Water enters the pump without any velocity of whirl, and leaves it with a whirling velocity which must be reduced to a minimum in the action of lifting. The direction of the water as it flows toward the discharge-pipe is controlled by a single guide-blade, which is the volute or outer surface of the pump chamber into which water flows on leaving the fan. A centrifugal pump cannot be put into action until it has been filled with water, which operation is effected through an opening in the casing when the pump is below water or when above water by creating a vacuum in the pump chamber by means of an air or steam jet which raises the water into the suction-tube. In action the water rotates in the pump as a solid mass, and delivery only commences when the speed is such that the head due to centrifugal force exceeds the lift, though this speed may afterwards be reduced. As the pump commences to operate the water rises in the suction-tube and divides so as to enter the centre of the pump disk on both sides. The revolving pump disk or fan, as in a turbine, is provided with vanes or blades curved so as to receive the water at the inlet surface without shock—an effect obtained by so proportioning the pump as to give a gradually increasing velocity to the water until it reaches the outer ends of the vanes and then a gradually decreasing velocity until it reaches the discharge-pipe —a result obtained in construction by conical ends to both suction and delivery pipes and a spiral casing. The water leaves the surfaces of the vanes with more or less velocity and impinges upon the mass of water flowing around inside the outer casing towards the discharge-pipe, and this casing must have a section gradually increasing to the point of discharge in order that delivery across any section of it may be uniform. This section is also designed so as to compel rotation in one direction only with a velocity corresponding to the velocity of the whirl on leaving the pump disk.

Nearly all centrifugal pumps are provided with a vortex or whirlpool chamber in which the water discharged from the revolving vanes continues to rotate, and here the kinetic energy is converted into pressure energy which would otherwise be wasted in eddies in the confining chamber. This vortex chamber is provided with guide-blades following the direction of the stream lines so as to prevent irregular motion. The working parts of a centrifugal pump accordingly consist of a series of curved disks or vanes mounted on a spindle and revolving in a chamber in a manner similar to a fan-blower. A revolution of each vane within this closed case produces a partial vacuum which draws up the water, and it is on a proper proportioning and arrangement of these vanes that the effective working of the pump chiefly depends. These pumps are so constructed that the casing may be removed to allow of the inspection and cleaning of the pump disks. The curved vanes are made of the best steel or phosphor bronze and the pump disks should be perfectly balanced in order to produce even motion in the water.

The efficiency of a centrifugal pump diminishes with the lift, and for lifts exceeding 25 to 30 feet a plunger pump produces better results. Centrifugal pumps are usually driven by belting or shafting from water or steam motors, from which they may be erected independently and at some distance, though they are also manufactured so as to gear directly upon the motor shaft. The more rapidly the pump disk rotates, the lift remaining constant, the smaller is the centrifugal force—a peculiar condition, due to the fact that as the discharge increases the velocity of the water in the casing more nearly approaches that of the water leaving the pump disk, and therefore the efficiency of the pump improves, and with it the theoretical lift diminishes as well as the centrifugal force. Another peculiar property of centrifugal pumps is that a small increase in the number of revolutions after it has begun discharging produces a very large increase in the delivery. The highest efficiency ordinarily obtained by centrifugal pumps is from 65 to 70 per cent, and experiments seem to indicate that the efficiency of a centrifugal pump increases as its size increases. Thus, a pump with 2-inch discharge-pipe will give an efficiency of 38 per cent, while a 3-inch pipe will give a 45 per cent efficiency and a 6-inch pump a 65 per cent efficiency. From this it appears evident that a centrifugal pump is to be recommended rather where large volumes of water are to be lifted.

Rotary pumps are theoretically among the most efficient, and their form is one of the favorite ones among experimenters in pump designs. At the same time they are capable of elevating but small quantities of water, and are not of much value for elevating water for irrigation. They may be termed revolving piston-pumps in distinction from direct-acting pumps, and have the advantage of not changing the direction of flow of the water during its elevation by each stroke of the pump. They can be run at a high speed, and have no complicated leather-valves or pistons to be choked or get out of order. There are numerous forms of these pumps on the market, which may be divided into two classes, according to the forms and methods of working the revolving pistons and the manner in which the butment is obtained, for it is this which receives the force of the water when impelled forward by the piston. In these two particulars consist the essential differences of various rotary pumps. The efficiency of rotary pumps is low, there being a great excess in driving-power required over useful work performed, caused chiefly by the inertia of water or difficulty of putting it in motion after it has been brought to rest and the necessity of imparting at certain moments a high velocity to a large volume of water, which calls for the expenditure of a considerable power.

372. Examples of Centrifugal Pumping Plants.—Among the better forms of centrifugal pumps now on the market are those made by the San Francisco Tool Company and the Byron Jackson Machine Works, on the Pacific coast, and those of the Morris Machine Works, the Baldwinsville Pump Works and the Joseph Edwards Pump Company in the East. These pumps are made both to be worked separately by transmitted power or by motors attached directly to the pump frame, and are of varying capacities, from those having 2-inch discharge-pipes up to those having 24-inch discharge-pipes, the largest sizes being capable of elevating as much as 15 second-feet, or the same number of acre-feet, in a day of 12 hours. Such pumps as these vary in cost according to circumstances, but the larger sizes cost for plant about $100 per second-foot of capacity for moderate lifts, while for 15 second-foot pumps they require engines capable of developing about 5 H.P. per foot of lift.

Among the more notable centrifugal pumping plants for irrigation is one for the Vermilion Canal Company in Louisiana, consisting of six 15-inch pumps, which are capable of discharging 130 second-feet of water against a head of 20 feet, and are operated by two engines, each of 250 H.P. Another centrifugal pump, working on a farm in Southern Arizona and operated by a 10-H.P. engine and boiler, has a capacity of two-thirds of a second-foot a day. The operation of this plant calls for the consumption of about one cord of wood per day of twenty-four hours, and it is capable of irrigating about three acres in a season. A similar pump in the same locality and operated by a gasoline engine of 35 H.P. will handle about 11½ acre-feet in twenty-four hours, on a consumption of about 84 gallons of gasoline. Other centrifugal pumps of small capacities and capable of watering 5 to 10 acres per day, and in the course of an irrigation season from 50 to 100 acres, are operated by one man at a cost of about $2.50 per acre irrigated for maintenance and $15 per acre for first cost of plant.

It is proposed to erect an extensive centrifugal pumping plant for the Summit Lake Water Company in California, and the estimates of the engineers for a plant capable of irrigating

40,000 acres, including distributing canals and other items, is $81,000, the cost of the pumping plant alone being estimated at about $0.75 per acre, while the cost for operation of and interest on the pumping plant during an irrigating season is estimated to be about $1 per acre, on the assumption that depth of irrigation will be 1 foot and the lift 20 feet. These figures are considerably below those of most gravity systems.

373. **Steam Pumping-engines.** — These are of various types and makes, and may be either direct-acting, or pump and motor may be indirectly connected by belting or shafting. In general it may be stated that fly-wheel pumping-engines which give high duty under the conditions of municipal water service, and other forms of indirect pumping-engines, are not the most efficient and economical for purposes of irrigation. Direct-acting pumping-engines have a reciprocating motion, and may be either single or double acting. They may also be either steam pumping-engines or the water end of a direct acting pump may be operated through gearing from water-motors.

Direct-acting steam-pumps have the water and steam ends centred in line one with the other so that the water-plunger and steam-piston are attached to the same piston-rod and work together without an intervening crank or other connection. This is the simplest and most compact form of steam pumping-engine, and is more extensively used for pumping than all other varieties of pumping machinery combined, though it is perhaps one of the most wasteful and expensive forms of steam-engines. This is largely due to the fact that they are the cheapest and most compact form of steam pumping-engine. They are manufactured by many establishments and the qualities and efficiencies of the various makes are well established by competition and experiment. This class of pumping-engine is similar in nearly all the various makes, differing chiefly in details of valve motion, and among the best known are the Knowles, Blake, Smith-Vaile, Dean, Cameron, Worthington, and Davidson valve movements.

In selecting steam pumping-engines, among the points most desirable are, strength and simplicity of working parts;

large water-valve area; long stroke and ample wearing surfaces; continuity of steam flow; simplicity of adjustment and repair; moderate steam consumption. In choosing from the various makes of pumping-engines it is well in corresponding with their makers to inform them among other points of the purposes for which they are to be used; height of lift and height to which water is to be forced; quantity of water to be elevated; motive power; and quality of fluid, as clear or muddy.

Direct-acting steam pumping-engines may be either high-pressure or compound. In the latter case they are economical in both fuel and water consumption, and their cost for operation is therefore correspondingly less, though their first cost is a little greater. The best form of direct-acting pumping-engines are Duplex pumping-engines, consisting of two direct-acting steam pumping-engines of equal dimensions, side by side on the same bed-plate, with a valve motion so designed that the movement of the steam-piston of one pump shall control the movement of the slide-valve of its opposite pump so as to allow one piston to proceed to the end of the stroke and come to rest while the other piston moves forward on its stroke. The earliest and one of the best duplex pumping-engines is the Worthington pump.

All single-acting pumps should be provided with air-chambers, while they are a decided addition to even double-acting pumps, for though the latter have a fairly steady discharge, the air-chamber insures almost perfect uniformity of delivery. The capacity of an air-chamber for a pair of double-acting pumps is about five or six times the combined capacities of the water-cylinders, while for a single-acting pump it may be ten or twenty times greater. The air-chamber performs practically the office of a stand-pipe attached directly to the pump. It neutralizes the variations of velocity of discharge in the delivery-pipes, the fluctuations of which might cause danger of ramming and wastage of work. The air-chamber obviates this by permitting the excessive delivery of water from a pump-stroke to enter it and thus compress the air, while on the return stroke the expansion of the air forces out water to supply the deficiency.

374. Examples of Steam Pumping Plants.—Several extensive steam pumping plants have been introduced in the West for providing water for irrigation. Their first cost is usually a little greater than that for centrifugal pumps, though not always so, depending upon the height of lift. Their efficiency is usually quite high as compared with all other forms of pumping plants, and their maintenance cost is usually less than that for centrifugal pumps. Their operation requires skilled labor, as does that of centrifugal pumps; but they are less liable to get out of order, and any injuries sustained can usually be readily repaired.

In Arizona is a high-pressure engine capable of irrigating 100 acres per season which cost when erected $1000, or $10 per acre irrigated, while the cost for running it is but $5 per acre. A larger and more modern plant operated near Tucson consists of two compound Smith-Vaile pumping-engines, capable of irrigating 600 acres per season at a cost for operation of $3 per day, the first cost for this plant laid down having been $4200, or $7 per acre, and the height of lift being 70 feet. Still another pumping plant, consisting of an automatic cut-off condensing engine with two 150 horse-power boilers, has been erected on the Yuma river. The pumping-engine has 18-inch stroke and 42-inch cylinders, and is of 165 H.P. capacity. This is an Allis engine having a fly-wheel weighing 7 tons and making 67 revolutions per minute, the capacity of the pump being 12 second-feet or about 24 acre-feet in a day of 24 hours. This pump delivers water through a 26-inch redwood stave main, elevating the water 80 feet, and this is stored in a reservoir having 23 acre-feet capacity. A year's test of this engine shows it to be capable of discharging 12 second-feet at a cost of $3 per second-foot for fuel.

A pumping plant designed by the writer and employed for the irrigation of 1000 acres consists of a duplicate set of duplex compound pumping-engines each capable of elevating 25 second-feet with a suction height of 15 feet and forced to an elevation of 40 feet, having a boiler capacity sufficient for both pumping plants. One pump only was to be used during the first year or two of development of the irrigation project, the second pump being used rather as a relay pump in case of accident, and to be called into service only when the highest duty for all the area under irrigation should be demanded. Such a plant erected in Arizona cost $10 per acre controlled, and for operation was estimated to cost not exceeding $0.75 per acre.

375. **Pulsometers; Siphon and Mechanical Elevators.** —Pulsometers are mechanisms for lifting water by the direct action of steam. They are most advantageously employed for rough work and in difficult situations, chiefly because of their portability, as they can be readily moved from one point to

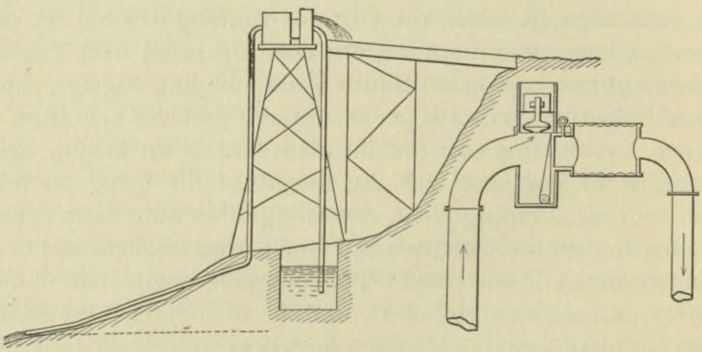

FIG. 122.—GENERAL VIEW AND DETAIL OF SIPHON ELEVATOR.

another. The pulsometer is capable of utilizing very dirty water, but its capacity is so limited as to render it practically of small value in pumping water for irrigation. It consists of a couple of pear-shaped vessels in one casting, the necks of which terminate in a single chamber. It is designed somewhat on the plan of the human heart, wherein two valve-seats are arranged with one ball-valve which oscillates between them. It also has an air-chamber, suction and delivery valves. When charged with water, steam is admitted and presses on the water surface in one chamber, forcing it through the delivery-valve into the delivery-pipe. When the steam reaches the opening leading to the discharge-pipe it comes in contact with the water already in the pipe, and is immediately condensed, forming a vacuum in the chamber just emptied. This vacuum draws the ball-valve over to the seat opposite that which it previously occupied, and prevents for the time further admission of steam, and to fill the vacuum thus formed water rises through the suction-pipe and fills the empty chamber, an operation which is repeated indefinitely. Pulsometers contain practically no movable parts, wear is reduced to a minimum, very little attention is required in their use, and there is little chance of clogging the valves by dirty material, but their efficiency is extremely low.

Lemichel et Cie of France manufacture an apparatus called a siphon elevator, which is claimed to attain an efficiency of 90%. It consists ot a siphon erected at a fall or dam in a river, at a reservoir dam, or in any situation where the lower or discharge arm can be carried below the suction-pipe, so as to give a difference of elevation for the creation of siphon action. At the highest point of the siphon are constructed air and valve chambers, the effect of which is to relieve the siphon at that point of most of the water passing through it, only a little passing on down through the longer arm of the siphon to keep up siphon action. It is this contrivance which enables water to be elevated by the siphon to heights as great as nearly 30 feet at sea-level, instead of being delivered, as by common siphons, below the point from which it is derived. The capacity of these siphon elevators varies according to their dimensions and the height to which they elevate the water, but at sea-level they have been built with capacities sufficiently great to elevate 8 acre-feet in 24 hours. This is a very large quantity when the simplicity of construction and cheapness of first cost of this mechanism is considered; and it may be safely stated that if further experiment with it shows it to be as effective as it has proven in the past, it will prove a valuable water-lifting apparatus where only trifling heights—say 10 to 15 feet—are to be overcome, and there is sufficient fall and surplus water to permit of the wastage caused by the operation of the siphon. Batteries of two or three of these siphon elevators have been erected, one above the other, whereby, with additional wastage of water for each siphon, heights two or three times that to be effected by one siphon elevator have been obtained.

The siphon elevator depends for its efficiency on the operation of the air-chamber or receiver and the regulator, which are placed at the upper bend of the siphon pipe. At the bottom of the suction-pipe is a check-valve which allows the ingress of water but prevents its escape. At the bottom of the lower arm of the siphon is a stop-cock which, when open, permits the escape of water so that when it moves a vacuum is created behind it which is filled with water as in simple siphons. In action the siphon elevator must first be filled with water, and as this descends in the lower pipe and ascends in the upper or suction pipe it passes through the receiver, where it reaches an open check-valve which intermittently cuts off its flow into the regulator. The water forces this valve forward and closes it, and its exit being thus cut off, its momentum raises a puppet-valve which is in the regulator and is maintained by a spiral spring. Through this valve the water escapes into a storage tank or irrigating ditch. While the regulator is being partially emptied into the pipe a vacuum is caused which creates a depression in the corrugated heads of the receiver as in an aneroid barometer, and the pressure on the check-valve being diminished, it is thrown open by the weight on the lever, permitting the water to fill the regulator once more and the corrugated heads to again assume their normal position. This vibratory motion occupies but a brief time, as many as 400 to 500 such pulsations taking place per minute, so that the flow of water is nearly continuous.

Of mechanical water elevators perhaps the only one which is of utility in elevating water for irrigation is the Link Belt Box water elevator, manufactured in Chicago. This machine is somewhat like the old chain-and-bucket pump, and consists of an elongated box which can be set up over the well or other water supply. At either end of this box is a wheel of peculiar construction, carrying on its periphery a metal link belt or chain, having attached to it at short intervals wooden projections of such dimensions as to completely fill the cross-section of the box. These projections, or flights, as they are called, close the space in the box between each flight, and as the chain revolves they are raised, carrying with them the water resting upon them and preventing it from running back. This machine may be operated by animal, wind, steam, or water power, as desired. The largest size made is capable of lifting about 5 second-feet or 5 acre-feet in a working day of 12 hours, with an expenditure of 7 horse-power for a 10-foot lift. The highest satisfactory lift of these machines is about 20 feet, and the cost of a machine of this capacity is about $50 per second-foot or $1 per acre controlled, a comparatively trivial outlay for first cost of pumping plant, excepting the motive power.

376. **Irrigation Tools.**—There is little to say of the tools required in the construction and management of irrigation works. Agricultural-tool makers now manufacture hoes, spades, shovels, and ploughs of special designs for the making and management 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 water runs, thus irrigating the crop much as if it were flooded.

377. **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

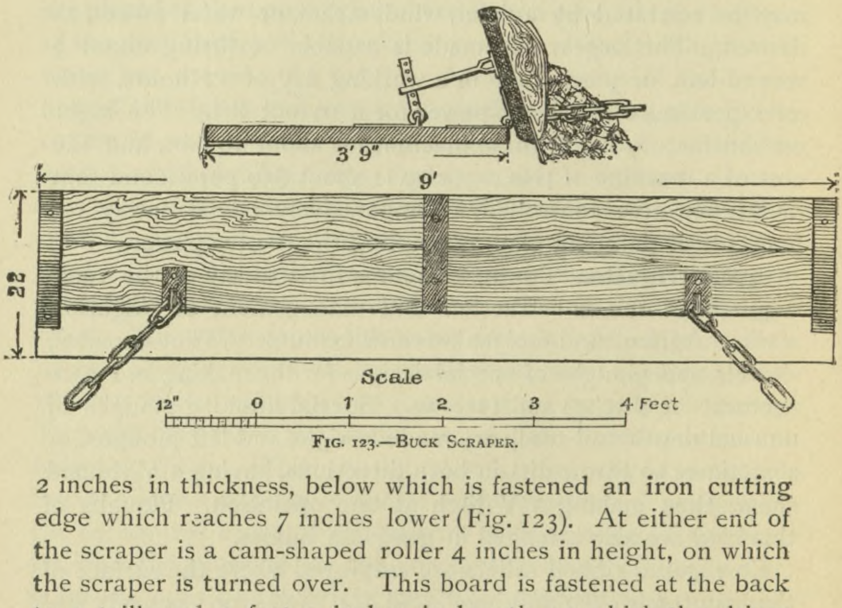

FIG. 123.—BUCK SCRAPER.

2 inches in thickness, below which is fastened an iron cutting edge which reaches 7 inches lower (Fig. 123). 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 1½ 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.

378. Grading and Excavating Machines.—Several of the road-grading machines give great satisfaction in levelling and grading land which is to be irrigated. Among the more useful of these are the Shuart and New Era graders, which expedite the work of preparing land for furrow or flooding, and thus greatly aid the operations of applying water. 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. 124). 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-six of these buckets, each having a capacity of ½ 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,

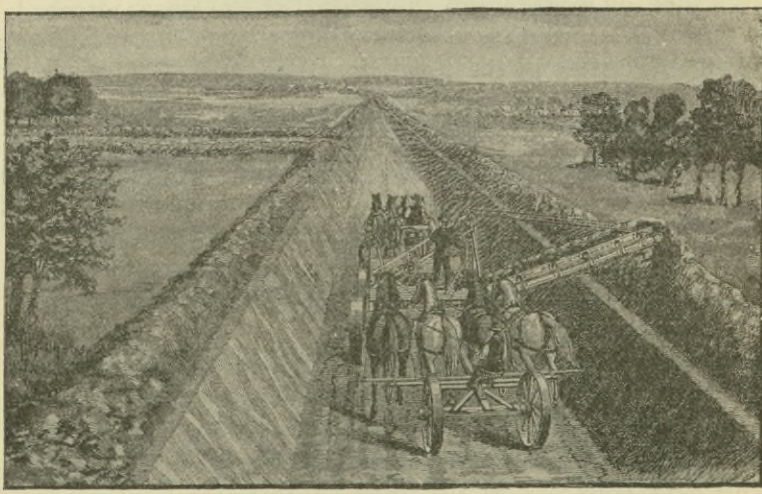

FIG. 124.—NEW ERA EXCAVATOR.

being similar in design and in construction to the ordinary dredges employed in river and harbor work, and in like operations. A small dredge which has recently been employed on irrigation canals has a single bucket, a draught of only 15 inches, and can excavate from 2 feet above the water-line down to 8 feet below it, delivering either to a shore-conveyer or to scows. A larger dredge of similar construction has a 3-foot draught and excavates as much as 70 cubic yards per hour, delivering it at a distance of 100 feet.

379. 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.

380. Sources of Impairment of Irrigation Works.—These are:

1. Erosion of the inner slope of the banks by the canal 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 willows 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.

381. **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 the 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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