CHAPTER VIII.

RESERVOIRS.

Different kinds of Reservoirs—Available Capacity—Storage Reservoirs: Capacity; various methods and rules—Compensation Reservoirs—Depositing Reservoirs: theory, dimensions and proportions; area relatively with filter beds—Service Reservoirs their various uses; capacity required—Natural Reservoirs or Lakes: salmon stairs—Semi-natural Reservoirs: theory and construction of earth-work and masonry dams; puddle walls; springs on the site of the dam; outlet arrangements; wasteweir and byewash; syphon weir; separating weir—Reservoirs entirely artificial: economical proportions; cost; covering; regulator—Water Towers.

RESERVOIRS, as they will be understood in this chapter, are natural or artificial basins appropriated or formed for the purpose of reserving or storing water. They are sometimes used with a view to equalise the flow of rivers and streams, by retaining the water in times of excessive rainfall and floods, and yielding it again in times of drought; they are also used in a similar way for purposes of irrigation. In the present instance, however, only those have to be regarded which form part of water-works for supplying cities and towns; but even such reservoirs will often be found playing an important part in the economy of rivers and streams.

In works for supplying towns with water there will be included storage, depositing or settling, compensation and service reservoirs; the several uses of these have already been referred to in a previous chapter. All reservoirs are for the purpose of storing water for a greater or less length of time, but the storage reservoirs mentioned above are more particularly those in which the water is retained from the rainy season to the dry. Again, storage reservoirs will always act to a certain extent as depositing or settling basins, and sometimes as compensation reservoirs; but there are settling basins or reservoirs which are used where storage is unnecessary, as in the case of a supply being taken from a large yet turbid river, whose waters are fit neither to be immediately distributed in the town nor to be directly filtered with economy. Compensating reservoirs, acting only as such, in connection with works for domestic supply, are usually those into which the turbid waters of flood-time are diverted, so that the compensation water to be given to the streams and mills below shall not be a tax upon the clear water which is available for domestic purposes. Service reservoirs are always clear water reservoirs, and from them the town supply is immediately drawn. Summit reservoirs are a particular kind of service reservoirs, being always connected with a pumping main.

The first point that has to be determined in connection with a proposed reservoir is its available capacity, by which is meant the net capacity, or that which can be actually utilised for the purposes of the reservoir. This will be the volume contained between the highest and lowest working levels; for in all reservoirs where matters suspended in the water are likely to be deposited, a certain space has to be left at the bottom, from which the water is never drawn in the ordinary course of proceedings, but only for cleansing and other occasional purposes. In storage reservoirs this 'bottom' is frequently made about one-sixth of the depth of the reservoir at its deepest part. The available capacity will, in all cases, be regulated by the fluctuation in the supply to and the demand upon the reservoir. In some cases both the demand and the supply will fluctuate, as in the case of storage or impounding reservoirs, where the demand will be somewhat increased in the summer months; and the supply, which depends upon the rainfall, will vary greatly with the different seasons of the year, and with different years. In other cases the demand only will fluctuate, as for instance where a service reservoir, drawn upon at rates varying with different periods in the twenty-four hours, is fed night and day by a constant stream from a conduit or pumping main. Lastly, the capacity may be determined by the length of time for which either the inflow or the outflow is altogether stopped; as, for instance, where, through any accident to a conduit or to pumping machinery, the service reservoir remains unsupplied until the necessary repairs can be effected.

The chief use of storage reservoirs, in works for the supply of towns, is to retain a volume of water which shall meet the entire demand for the longest period or immediate succession of periods during which the demand is in excess of the supply. They have thus not only to balance the supply and demand between different seasons of any one year, but also to compensate for the deficiency of supply in one year out of the excess of previous years. The rainfall of three consecutive dry years (*see* page 39) is now generally made the basis of calculation, and the drainage area is determined accordingly. The reservoirs must then be such as to provide for the demand taken over not only the dry years but a long series of years; and it will have been the duty of the catchment area to raise the minimum flow to such an amount that the tax upon a reservoir of reasonable size shall be not more than it can withstand. The old method of calculating the catchment area, from the general mean annual rainfall, would require, if properly followed out, a storage capacity to retain the whole fall of excessively wet years and distribute it over a long series of medium and dry years. The drainage area would of course under such circumstances be less, but the reservoirs, to be safe against the effects of drought, would have to be made of a capacity which would be under ordinary circumstances unjustifiable on grounds of economy. The whole question, as we have seen in Chapter VI., is one of the adjustment between the catchment area, the extent of the variations in the rainfall, and the capacity of the reservoirs, in such a manner that the latter shall actually retain the required quantity of water at the commencement of a period of defect in the supply.

The various elements which make up the total demand have been detailed in a previous chapter, and consist of the requirements of the population at so many gallons per head per day, and the quantity to be passed down as compensation to the streams. These several items of the demand must be regarded not only as having a mean annual value, but as varying more or less with the different seasons of the year. In London the consumption during the summer months is about 10 per cent. above the average daily consumption taken over the whole year, while the consumption in the winter may be set down at about the same percentage below the average.

In cases where actual rainfall observations on the gathering ground have been made for a long series of years, or even where an approximation to this has been formed by comparison between newly-established gauges on the ground itself, and old-established gauges in the neighbourhood (as directed in a former chapter, p. 34), a simple calculation may be made of the storage capacity required to meet a given demand. The question of how far such calculation will, with a reasonable margin, form a reliable base for estimation of the storage capacity, will of course depend upon the actual rainfall observations being sufficiently extensive to represent, within fair limits, the variations which may yet occur. Supposing this to be the case, and, moreover, that observations have been made upon the proportion borne by the actual flow off the ground to the amount of rain falling—which proportion will of course rapidly diminish as the absolute amount of rain is less—there can be formed a table or diagram of the available supply, showing its several fluctuations. The demand, with its fluctuations to be provided for, will be known already, and it may be shown upon the diagram or inserted in the table. Then, working backwards in point of time, and taking the difference between the inflow and the draught, we may determine, by a simple process of addition and subtraction, what quantity will be required in store at the commencement of the several short periods—generally months—into which the time is divided. The maximum attained will be the net capacity of the reservoir, beyond which a margin of safety must be allowed, depending upon the considerations already referred to. The storage capacity thus determined will be equal to the demand for a certain number of days, which number will be greater as the mean annual demand is greater in proportion to the mean annual supply; and it will also be found greater as the mean annual fall is less.

The storage capacity may be taken as a certain number of days of the excess of the demand over the dry-weather flow, that is to say, the average of the quantity which may be expected to pass into the reservoir during the drought. This may be ascertained from a series of comparisons of actual gauging with the rainfall during the same period. Should there be rainfall records of more severe droughts, the corresponding minimum flow of the streams will be still less in proportion to the fall of rain; for the loss by evaporation will be proportionately greater, as the ground surface is hotter and more parched. In districts that are generally selected as catchment areas for town supplies, a dry-weather flow of from one-quarter to three-quarters of a cubic foot per second (that is, from 135,000 to about 400,000 gallons per day) per thousand acres may be expected. Great caution however, must be exercised in making allowances under this head, and a careful study of judiciously-arranged, gaugings should invariably be made. In his evidence before the Royal Commission on Water Supply (1869), Mr. Bateman gave the following particulars of the dry-weather flow from different areas, during the remarkable drought of 1868.

Mean average rainfall	Name of Water-works	Area of drainage ground	Period of Drought in 1868		Rainfall during drought	Amount of rainfall collected	Equal to a flow from each 1000a.	Loss by evaporation* during drought
		acres		days	inches	inches	cubic feet per second	inches
50·	Manchester Corporation	19,000	April 27 to September 24	150	9	2·77	·776	6·23
	Same (taken over four short periods)	18,000	June 1 to June 29 . .	28	1	—	·612	
			June 29 to August 3 .	35	1·34	—	·43	
			July 27 to August 3 .	7	·21	—	·329	
			June 1 to August 3 . .	63	2·34	—	·5114	
52·38†	Dublin	14,080	May 1 to September 18 .	141	11·298	4·37	1·303	6·928
	Glasgow— Loch Katrine District .	45,800	May 30 to August 8 .	70	4·65	1·39	·839	3·26
49·	Gorbals District . .	2,560	March 29 to October 3 .	189	18·19	6·45	1·43	11·74
46·	Liverpool . . .	10,000	April 28 to October 16 .	161	12·20	1·525	·398	10·675
44·	Blackburn . . .	880	March 27 to September 25	182	12·8	·75‡	0·199	

* The loss by evaporation and absorption is taken as the difference between the rainfall and the quantity collected, no allowance being made for infiltration into the stratum, because the drainage areas in the table are almost entirely impervious, and from those few parts into which the rain does infiltrate the water is yielded again as springs within the catchment basin.

† Mean of 8 years from 1861 to 1868, and probably from 5 to 7 inches above the true mean.

‡ Mr. Bateman could not vouch for the accuracy of this item; indeed, he believed that double the amount would be nearer the truth, The drainage area, however, is a bare hill-side, wholly grass land, with scarcely a spring upon it; and this will account for the large proportion of rainfall that was absorbed.

After having ascertained the greatest excess of the demand above the supply, it remains to be decided how many days' storage will be required. As before stated, the number of days' storage required will be greater as the mean annual rainfall is less. Thus, in many parts of the West of England where the rainfall is greatest, 100 or 120 days' supply will be sufficient, while 240 or 250 days' supply will be required in the East, where the total rainfall is less and the droughts are longer. It would appear that the storage capacity should vary inversely as the square root of the rainfall taken for three consecutive dry years; and, moreover, that if the constant number 1,000 be divided by this root, a fair idea of the number of days' storage required will be obtained. The discovery of this law is due to Mr. Hawksley, and was arrived at only after many years of extensive observation.

Suppose, for example, that a population of 100,000 is to be supplied with 25 gallons per head per day, which is equal to a total of 2,500,000 gallons per day, and suppose, further, that the supply is to be taken from a catchment area, over which the mean annual rainfall is, say, 42 inches. Deducting one-sixth, or 7 inches, there will be 35 inches for the fall during three consecutive dry years (*see* p. 39), which we will make the basis of the calculation. The square root of 35 is 6 nearly, and 1,000 divided by 6 is equal to 166, say 170 days' storage. In addition to the above $2\frac{1}{2}$ million gallons per day, there will be, suppose, 1,000,000 gallons per day to be given as compensation to streams (*see* p. 89); and 160,000 gallons per day to be lost by the evaporation of, say, one-tenth of an inch per day from the surface of the reservoir of, say, 70 acres. The total demand will therefore be 3,660,000 gallons per day. The dry-weather flow into the reservoir we will assume as a little less than half a cubic foot per second from each 1,000 acres taken over an area of, say, 2,500 acres. This will give 600,000 gallons per day, which, deducted from the total demand, will leave 3,060,000 gallons per day. This quantity, multiplied by 170 for the number of days' storage, will be 520,000,000 gallons, the required available capacity of the reservoir.

The capacity of storage reservoirs is frequently considered in relation to the area of the drainage ground; but it may be seen that the proportion between the two will depend upon the rainfall, and the loss by evaporation and absorption. If the storage capacity be determined according to the principle just laid down, it will vary inversely as the square root of the mean rainfall of three dry years, whereas the drainage area will vary inversely as the available rainfall, that is, after the loss by evaporation and absorption has been deducted. The proportion, therefore, will depend upon those physical conditions which affect the loss by evaporation and absorption, and which have been referred to elsewhere. The storage capacity per acre of drainage area, giving a supply to the following towns, is as under :—

City or Town supplied	Storage capacity per acre of gathering ground	City or Town supplied	Storage capacity per acre of gathering ground
Glasgow	cubic feet		cubic feet
Gorbals District. .	52,437	Dublin	25,565
Loch Katrine . .	30,183	Manchester . . .	24,333
Liverpool . . .	48,500	Sheffield	32,000

We have hitherto supposed the estimation of the demand upon storage reservoirs to include an item for compensation to streams. As already stated, however, reservoirs are sometimes set apart solely for retaining the compensation water; and arrangements are made, either self-acting or otherwise, for passing the turbid flood-water into the compensation reservoir, and allowing only the clear water of the stream—if there be sufficient of it—to flow into the reservoir from which the town is supplied. It cannot be expected that the clear water shall always bear to that which is turbid the proportion of the town demand to the demand for compensation; but even when the turbid water is in excess, arrangements can be made for 'decanting,' from the compensation to the town reservoir, the required quantity after it has become clear by settlement and exposure to the atmosphere. By such arrangements, freedom of the water from mechanical or suspended impurities will be ensured with the greatest possible efficiency. The capacities of the two reservoirs should bear about the same proportion to one another as the respective demands upon them (*see* Chap. VI., p. 89), except where the turbid waters are in excess, and where in consequence special regard must be had to purification by settlement. It must be remembered, however, that the capacity of the town reservoir is determined on the supposition that the whole of the dry-weather flow passes into it. The compensation reservoir must, therefore, have additional capacity, because it will be deprived of its due share of the dry-weather supply. In other words, the compensation reservoir must have a capacity to meet the demand for the required number of days' storage, no allowance being made for inflow during the time. Usually, however, the circumstances of the case—chiefly the favourableness of the site—will determine the relative capacities of the compensation and town reservoirs.

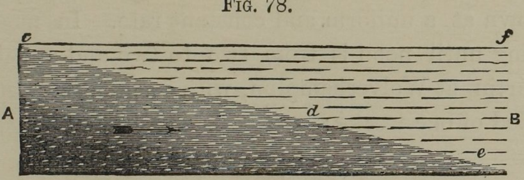
As already stated, storage or impounding reservoirs are necessarily settling reservoirs; indeed, all reservoirs are so to a greater or less extent. The area and capacity of which storage reservoirs as such have to be made is generally in excess of that required for efficient settlement, so that seldom, if ever, will an addition have to be made to the capacity of a storage reservoir, in order specially to allow of the complete subsidence of suspended matter. For even when there is an exceptionally small quantity of water in the reservoir—say, only a week's supply—it will in most cases be at rest to all intents and purposes; in other words, the velocity of the current towards the outlet will be far lower than will be likely to interfere with the settling process. The case is different where large storage capacity is not required, and where reservoirs have to be provided expressly for the purpose of purifying water by subsidence, as, for instance, where the supply is derived from a turbid river whose flow is greatly in excess of the requirements of the works. Here the object will, of course, be to provide for the settlement of the mechanical impurities with the least possible expense.

In assigning dimensions and proportions to depositing reservoirs, the first operation is to ascertain or assume the velocity at which the suspended particles will descend and leave behind, or rather above them water in the required condition, that is, fit to be at once supplied to the consumers—as, for instance, in the case of the precipitation of whiting from chalk water undergoing the softening process—or fit to be filtered economically, as in the case of most river water. This rate will determine the net area of the reservoir. Thus, the demand in cubic feet per day, divided by the velocity of deposit in feet per day, will give the area. The velocity of descent being supposed at its maximum, this area is the absolute minimum, and is quite independent of the depth—disregarding of course the influence of waves. But this area would be sufficient only if the process could be continued without interruptions, such as by filling and emptying the reservoir. Where the arrangements are such that these processes are necessary, and where the whole available depth of water is clarified before drawing off is commenced, it is advisable to divide the net area as found above into independent reservoirs or compartments, and then add one of such compartments. This additional compartment will of course be smaller, as the number of parts into which the original area is divided is greater. Thus, if to clarify the total average supply in a particular case required, say, 6 acres of quiescent water wherein the depositing process might proceed without interruption, we should, by having two reservoirs of 3 acres each, require an additional one, or reserve, of 3 acres, making altogether 9 ; but by forming three reservoirs or compartments of 2 acres each, an additional or reserve compartment, making altogether 8 acres, would suffice to enable 6 acres of water to be constantly depositing their grosser suspended particles

Existing samples of depositing reservoirs present great diversity in their dimensions, as compared with the average daily supply; and this is the case even where the waters appropriated are not only very similar, but, as in the case of certain companies supplying London from the Thames, practically identical. In this latter instance, we find the area of depositing reservoirs varying from about 16 square feet to about 100 for each 1,000 gallons supplied daily. These differences may be accounted for partly by the varying facilities and difficulties of acquiring the necessary site, but mainly by the differences of opinion as to the most economical point to carry the clarification by deposit, previous to the process of filtration. In the case of the deposition of whiting in the course of Clarke's softening process, there is less room for diversity of opinion. Here it is found that when the hard water is clear, the whiting deposits pretty constantly at about 12 inches per hour. But it is an essential to rapid deposit that the water should be quite clear; with slightly turbid water the process becomes tedious, and where there is considerable turbidity, arising, as with flood waters of most rivers, chiefly from the presence of organic matter and finely divided clay, the carbonate of lime will not precipitate. In order to apply Clarke's process to rain-water with anything like successful results, it is necessary that the water should be filtered before the lime is applied. Sometimes the pipe through which the water is discharged is connected by a movable joint to a length of pipe whose free extremity is attached to a float in such a manner that clear water may be drawn from near the surface while the settling process is yet going on at a lower level. On Plate 34 is shown an example of this in the Aberdeen Waterworks, and on Plates 44, 45, and 48 improved arrangements are shown for effecting a similar object in connection with the depositing reservoirs of the softening process at Canterbury. Where such appliances are adopted, a saving may be made in the dimensions to be assigned to the reservoirs ; and perhaps the margin to be allowed in practice beyond the areas estimated in the manner before described may be considered as secured by the time saved in partly drawing off the water before the clarification of the lower part is complete.

It is maintained by some that total absence of current or agitation is essential to the efficiency of a depositing reservoir. This is proved to be a mistake, by the fact that storage reservoirs in which there is a current are efficient settling reservoirs; and further, settling reservoirs themselves, designed to allow of the water standing at rest, frequently in practice have to be worked with a current, that is to say, water is admitted from the river at one end while it is being pumped from the other, and this without sensibly detracting from the efficiency of the subsiding process. Indeed, regarding it in the light of modern science, it is difficult to conceive a liquid as being in a condition of perfect rest. Total absence of current, if it could be obtained, is no doubt desirable in the highest degree; but as it cannot, there arises the question, what is the maximum velocity consistent with efficient settlement, or to what extent do different velocities affect the settlement. The rapidity with which solid particles are deposited in water is no doubt directly proportional to the specific gravity of the matter, and inversely proportional to the minuteness of the division of the particles, and also to the velocity of the currents in the water. Again, with any given conditions of turbidity there are no doubt fixed rates of deposit for particular velocities. Suppose it be ascertained that the mechanical impurities in water, in a certain condition of turbidity, will subside to the required clearness at the rate of n inches per day (measured vertically) from the surface of such water) when there is a mean velocity of v feet per second, $q \div v$ will then be the sectional area of a channel, or, if the term be preferred, an elongated reservoir, along which the required number of cubic feet of water, q, will pass, allowing, at the same time, subsidence to take place at the rate of n inches per day. Suppose Fig. 78 to be longitudinal section of such a channel or elongated reservoir, into which the turbid water is admitted at A: $c\ d\ e$ is the surface above which the water is found of the required clearness, and it may be readily ascertained at what depth this surface will be below the surface of the water ($c\ f$) at any given time. Thus at the end of one day it will, according to the notation adopted above, be n inches below the surface. The length of the channel A B must be such that the depth of clear water shall be sufficient to allow of its being drawn off by some means causing the least agitation possible in the channel, especially near the bottom. The best arrangement for this purpose is a weir of such a length that only a very thin sheet of water, and therefore one having a low velocity, shall flow over it. If the level of the water in the channel be liable to variation, the weir must of course be adjustable, and a properly arranged floating pipe may be made to answer the purpose. The most economical proportions for this kind of subsiding channel or reservoir may be readily ascertained for any particular case when the effect of the velocity of the current upon the rate of subsidence is known. It is not unfrequently found that settling reservoirs designed with the intention of allowing the turbid water to remain for a time at rest, as far as practicable, are worked with a determined current, that is to say, water is drawn from one end while it is being admitted at the other. But it will be seen that, in many cases, to effect the same amount of settlement will require with the slow current system much less reservoir space than is necessary where the turbid water is allowed to stand at rest. Depositing reservoirs are not unfrequently made of a capacity equal to three days' demand; that is to say, there are three reservoirs worked alternately, each of a capacity equal to about one

Fig. 78.



day's demand. They are generally made from about 10 to 15 feet in depth. A depth of less than about 10 to 12 feet is not advisable, principally on account of the fact that water in shallow reservoirs is liable to become unduly heated in the summer months, and, moreover, vegetation is promoted.

In determining upon the dimensions and proportions of subsiding reservoirs, and the same relatively with those of the filter beds, if any, it should be remembered that the whole system of purification is capable of adjustment to a state of the most economical efficiency. No time (and here time represents reservoir area or capacity) should be wasted in allowing those particles to be deposited in the reservoir which could be more readily removed by filtration, and, on the other hand, there must be avoided the chance of too quickly fouling and choking the filters, when a little more subsidence would most likely lessen or postpone this evil, and with better results in other respects. This, however, is a point upon which no rules can be laid down, and concerning which the experience of the engineer can alone be his guide.

In the following table are given the proportions of depositing and filtering areas, for the different companies supplying London:—

	For each 1000 gallons supplied daily				
	Subsiding reservoir		Filter bed	Total area appropriated to the purification of the water	Proportion of subsiding reservoir area to filter bed area
Thames Companies:—	Square yards	Capacity in gallons	Square yards	Square yards	
West Middlesex	11·24	6,450	4·25	15·49	$2\frac{1}{2}$ to 1
Grand Junction	3·81	2,640	2·58	6·39	$1\frac{1}{2}$ to 1
Southwark & Vauxhall	2·84	2,200	2·92	5·76	1 to 1
Lambeth	1·50	772	·78	2·28	2 to 1
Chelsea	1·73	1,240	1·15	2·88	$1\frac{1}{2}$ to 1
East London	29·38	12,140	3·20	32·58	9 to 1
New River	20·85	7,740	2·13	22·98	10 to 1

It has been proposed that depositing reservoirs should always be of such capacity as to afford a storage room to meet the demand when the river is running in excessive flood, and when in consequence the suspended and dissolved impurities attain their maximum. This would allow the purer waters of the river to be selected, and would result in the reservoirs and filter beds fouling less rapidly. An eminent chemist has expressed the opinion that to enable a proper selection of Thames water to be made at the points where the supply for London is at present abstracted, two or three weeks' storage would be required.*

Service reservoirs next require attention, and unlike storage, compensating, or subsiding reservoirs, they are common to most works for the supply of water to towns, whether catchment, river, or well schemes, and whether gravitation or pumping. Service reservoirs are constructed with different objects, according to the circumstances of the case. When water is brought into a town from a considerable distance, whatever be the source, the conduit or channel in which it is conducted, has to be of a certain discharging capacity, which is reduced to a minimum if it be made according to the average demand, the conduit being of course in constant operation. But it must be remembered that the water is not consumed in the town at a uniform and constant rate. In most cases the draught upon the mains from eight o'clock in the morning until noon is from two to two and a half times the average consumption. Some experiments by Mr. H. Martin, at Wolverhampton, with constant service, gave the following results, the experiment extending over the period of one week†:—

Time	Percentage of gross consumption	Time which would be occupied in delivering gross consumption	Time	Percentage of gross consumption	Time which would be occupied in delivering gross consumption
Between 6 and 7 A.M.	3·735	26·77 hours	Between 2 and 3 P.M.	6·388	15·64 hours
„ 7 „ 8 „	5·209	19·19 „	„ 3 „ 4 „	7·862	12·72 „
„ 8 „ 9 „	6·192	16·14 „	„ 4 „ 5 „	5·209	19·19 „
„ 9 „ 10 „	6·438	15·53 „	„ 5 „ 6 „	6·290	15·90 „
„ 10 „ 11 „	7·076	14·13 „	„ 6 „ 7 „	3·685	27·13 „
„ 11 „ 12 „	7·764	12·88 „	„ 7 „ 8 „	5·012	20·00 „
„ 12 „ 1 P.M.	5·995	16·68 „	„ 8 „ 9 „	3·047	32·81 „
„ 1 „ 2 „	5·946	16·82 „	„ 9 P.M. 6 A.M.	14·152	68·26 „
				100·000	

* Dr. Frankland's evidence upon the Metropolis Water (No. 2) Bill, 1871.

† From Mr. Martin's evidence in 'The Board of Health Report on Supply of Water.'

Supposing that the water is flowing into the reservoir at a uniform rate, 4·17 per cent. per hour, these fluctuations would demand a capacity equal to about twenty-five per cent. of one day's consumption. But it would not be advisable to run so fine as this, and even if there were no other considerations, at least half a day's demand should be provided. Under the intermittent system the required capacity of the service reservoir will depend, to a certain extent, upon how the system is worked. If the supply to the town be distributed uniformly over, say, twelve hours of the day, and if at the same time the flow into the service reservoir be uniform and constant for the twenty-four hours, it is seen at once that the reservoir must contain, theoretically at least, half a day's supply, beyond which, of course, in practice a margin should be allowed. Again, where a town, either under the constant or intermittent system, is supplied from a reservoir into which the water has to be pumped by steam power, it is often found advisable to pump only during the day and not during the night. If in such a case the water were delivered to the town at a uniform rate spread over the time during which the engines were at work, a reservoir—in this case a summit reservoir—would be required merely to provide against the accidental irregularities in the draught from the mains, and to reserve for the night a sufficient quantity of water for immediate use in case of fire.

The foregoing considerations, with regard to the capacity to be assigned to a service reservoir, are regardless of provision against inconvenience to the town that might result from the supply being for a time interrupted by accident or failure. If the reservoir be fed by a long conduit, there should be storage sufficient to meet the demand during the time which would be occupied by the restoration of the conduit to its normal condition. This amount of storage capacity should increase with the magnitude and complexity of the conduit. It is not advisable, under any circumstances, to have much less than about two days' supply; while, on the other hand, reasonable prudence would seem to require for long conduits, say, up to about 50 miles, a week's supply, or even more.

With a view to prevent, or rather lessen, the inconveniences of a breakdown in the principal mains leading through a town, the town may, where the situation is favourable, be divided into districts, each district having an independent service reservoir of a capacity sufficient to meet the demand in the district while the damage is being repaired.

Towns are very frequently dependent upon engine power for their due and regular supply of water, and unless there be power in reserve, and a liberal allowance of minor duplicates always available in case of emergency, it should be the duty of the service reservoir to provide for the demand while the necessary repairs or renewals can be effected.

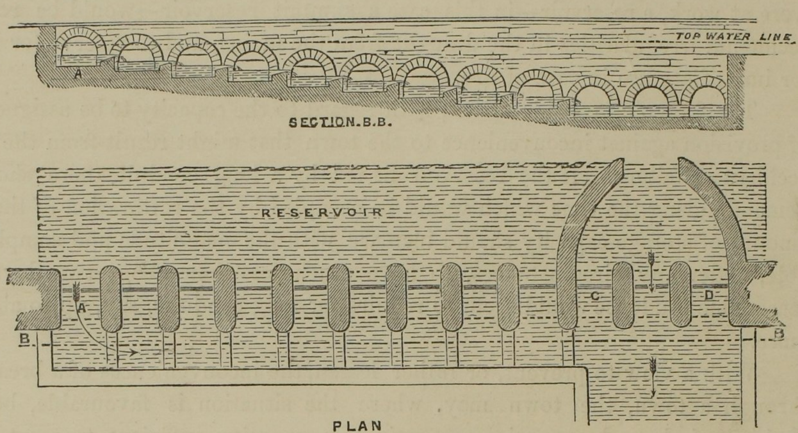
When the water supplied to a town undergoes filtration, it is necessary to provide a service reservoir, generally known in such cases as a clear water reservoir or tank, even if there be no other consideration which would require it ; for the filters will yield their water continuously and uniformly at the average rate, while the consumption in the town will vary from hour to hour; so that the reservoir, as before, is required to supply the deficiency and store the excess. It is true that if the filter be sufficiently extensive, or if the water be passed through the filter at an excessive velocity, the variations in the draught from the town may be met, but with undue expense by the former expedient, and by the latter with a sacrifice of efficiency.

Having inquired into the circumstances which affect the capacity—in most cases the principal feature—of a reservoir, we will proceed to notice the different kinds of reservoirs, classified according to their formation, namely, natural reservoirs, or lakes, reservoirs formed by damming across valleys, and reservoirs more entirely artificial, including such as are generally introduced as town reservoirs.

Natural reservoirs, or lakes, as we have seen in a former chapter, are utilised principally by an addition to their available capacity, that is the space contained between their highest and lowest working levels. A difficulty frequently presented against drawing upon a lake below its natural low-water line, where pumping power is not to be employed, is that of finding a suitable outlet, for the natural outlet is not unlikely to be so situate as to offer many disadvantages to leading a conduit from it; and in such a case the margin of the valley has to be pierced—never a very insignificant operation. Then, again, to raise the water level will require the formation of works at the natural outlet of the lake, and here provision will in most cases have to be made for the passage of fish. On Plates 27 and 28 are illustrated the outlet works connected with the Loch Katrine scheme for supplying Glasgow, and they may be regarded as excellent types of works of the kind. In the case of Loch Katrine the difference between the highest and lowest working levels is 7 feet, and of Loch Vennachar 11 feet 9 inches. The Loch Vennachar outlet works are by far the larger of the two, as a much greater quantity of water has to be passed down.

Fig. 2, Plate 28, shows a section of the salmon stairs, of which, as may be seen from Fig. 1, there are four placed at different levels, so that the fish may have access to the loch when the water in the latter is at different heights. Under favourable circumstances a salmon will leap to a height of about 7 feet; but apart from the question of economy, the lower the fall the better. Moreover, the sluices must be so disposed that at no time shall the velocity of the current through them be greater than the fish can make headway against. Mr. Leslie has proposed to employ only one flight of stairs to accommodate all levels of water in the lake, by placing it parallel with the series of sluices, as shown in Fig. 79. The fall at each step should be made about 18 inches. The water is allowed to flow through only *one sluice at a time*, and the number of leaps to be made by the salmon, will be thus reduced to the minimum in the season of drought when the water is at its lowest level in the lake, and the salmon have scarcely any inclination to *run*. The sluices are made to open by lowering, and are shut by being raised. Taking, therefore, the sluice A, when the water would be at its highest in the lake, the salmon, having ascended the other pools, would take its last leap over the sluice A into the reservoir. In like manner, as the level of the water changed, the sluices would be successively opened. The depth of water in each pool should be not less than 3 feet. The two sluices C and D are simply intended as ordinary outlet-sluices to let off flood waters, or in the event of any other emergency, and should be protected by wing walls, as shown on the accompanying plan.

Fig. 79.

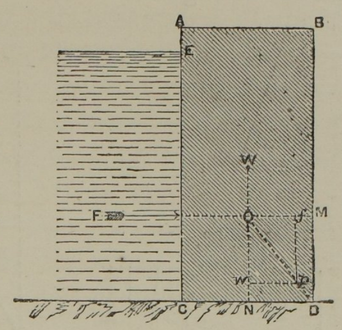


The most common method of forming storage reservoirs is that in which an embankment or dam is thrown across a valley, thus completing the basin already in part formed by nature. The sites of such reservoirs, as might be supposed, vary considerably in point of favourableness for capacity. The principal object is, of course, to provide the greatest capacity with the least possible cost in damming; and then comparative uniformity of depth is the next thing to be desired, in order that there may not be an undue proportion of shallow water where vegetation and organic impurities generally will be fostered. Advantageous sites are frequently found at the confluence of two or more streams, where one dam placed just below the point of junction will serve to impound all the waters. The site will be still more favourable if, after the streams have united, the sides of the valley close in and form a gorge, reducing the bank to comparatively small dimensions.

Simultaneously with the favourableness of the site for capacity, and for the formation of the bank in point of dimensions, the geological features must be carefully regarded, in order that a water-tight reservoir may be constructed. If any porous strata be intersected, it will be necessary to study their dip, for if it be away from the valley, such strata will only drain the reservoir of its contents; but if the valley be on a synclinal axis, the porous strata, if any, dipping towards the reservoir, will, on the other hand, serve to augment its waters by the inflow of springs which most likely will be perennial. Cracks and fissures in rocks are frequently sources of leakage from reservoirs, and special means should be taken to stop all such as are discovered, by the introduction of concrete and puddle. The reservoirs of the Manchester Waterworks, situated on the lower coal measures and the millstone grit, presented many difficulties in this respect. The mountain limestone also is full of fissures, by which the water is almost sure to be drained away. Where excavations are conducted in the interior of a reservoir, care must be taken not to cut through a sound, water-tight bottom, and expose a pervious stratum into which the impounded water may escape.

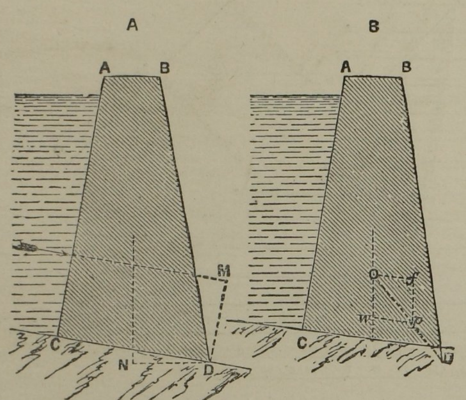
We have now to inquire into the means that are available for damming across the valley at the point selected. The material almost universally employed in the formation of large dams in this country is earthwork; and earthwork dams are of the most ancient type, as proved by the numerous reservoirs constructed by the Hindoos in times remote, and already referred to in a former chapter. In France, Italy, and Spain, however, dams of masonry are very common, and in the case of the Furens reservoirs for the supply of St. Etienne, and that of the Barrage of Puentes, in Spain, reach to a height of 50 metres (about 164 feet), whilst the highest earthen dam in this country is that of the Entwistle reservoir of the Bolton Waterworks, which retains a depth of only 120 feet of water.

Fig. 80.



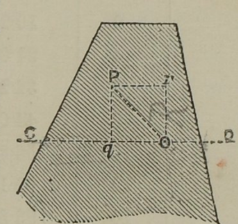
Let A B C D (Fig. 80) be a rectangular wall of masonry, having to sustain the pressure of water against E C. This pressure we know will be perpendicular to E C, and will be equal to the weight of a prism of water, whose base is the area pressed, and whose height is the vertical distance of the centre of gravity of that area below the surface of the water. This total pressure may be represented by a single force F, acting at a depth of two-thirds E C below the water surface. The effort of the water to overturn the wall about the point D, then, will be measured by the product of F and the distance D M. But the resistance of the wall to overturning about the point D will be measured by the product of its weight, and the distance D N, taken horizontally from D to a line drawn vertically through the centre of gravity of the wall. The pressure of the water and the weight of the wall should be taken for a definite length—say one foot—of the latter; and if the resistance of the wall be greater than the effort of the water, as above found, obviously the wall will not overturn. The case may be represented graphically by drawing W N vertically through the centre of gravity of the wall, and F M perpendicular to E C, through the centre of pressure. If the parallelogram O f p w be then drawn so that the sides O f and O w represent the pressure of the water and the weight of the wall respectively, the diagonal O p will represent the resultant pressure, both in direction and magnitude; and if this be produced so as to cut the line of the base C D, the wall will stand or be overturned, according as the intersection is within or beyond the point D. The foregoing simple methods of investigation may be applied to any wall or part of a wall, whatever be the inclination of the up-stream face, A C (Figs. 81, A and B), and whatever be the section of the wall or dam, so long as that, by the method of moments, the distance D M (Fig. 81, A) is measured parallel with the face A C, and the distance D N is taken horizontally. The application of the parallelogram of forces (Fig. 81, B) is equally simple.

Fig. 81.



The resultant pressure p O, (Fig. 82) acting at O in any sectional plane C D of the dam, may be resolved into two other forces O q and O r. The former tends to cause the part of the dam or wall above C D to slide bodily over the lower part, and is met, either by the resistance of the section of the bank at C D to shearing, or if at that part there be a joint, or rather bed, in the work, then by the frictional resistance, which is equal to the pressure O r multiplied by the co-efficient of resistance for the case. The pressure O r is distributed over the section C D in such a manner that, if we represent this pressure by R, the length C D by l, and the distance O D by u, the pressure r, at the point D will be given by one of the following formulæ :—

Fig. 82.



$$r = 2\left(2 - \frac{3u}{l}\right)\frac{R}{l}$$

$$\text{or } r = \frac{2\,R}{3\,u}$$

the first being used when u is greater, and the second when it is less than $\frac{1}{3}\,l$. The profile of the wall should be such that the pressure r shall never be greater than the limiting stress, whether the reservoir be empty or full.

The theory of masonry dams forms the subject of a very interesting and rather elaborate memoir by M. Delocre, of the Administration des Ponts et Chaussées.* The accompanying figures show the results of M. Delocre's formulæ applied by him to dams up to 50 metres in height; the specific gravity of the masonry being 2·00, and the maximum stress to which the material is subject being 6 kilogrammes per square

* See *The Engineer*, Vol. xxvi.

centimetre or $5\frac{1}{2}$ tons per square foot. In Fig. 83 the dam is shown stepped on both faces, each step being two metres in height.

FIG. 84.

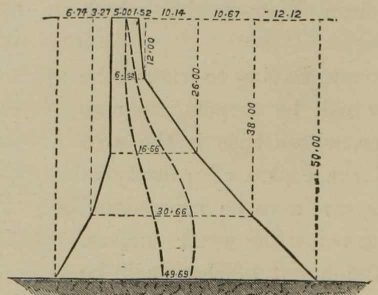


FIG. 83.

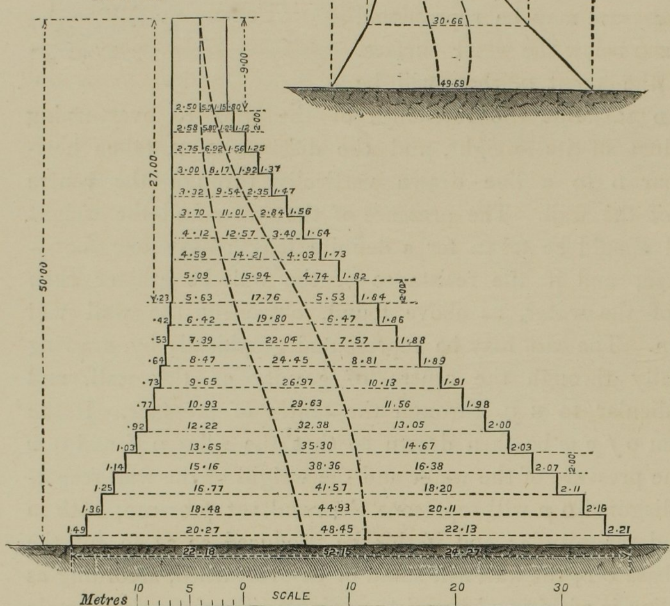
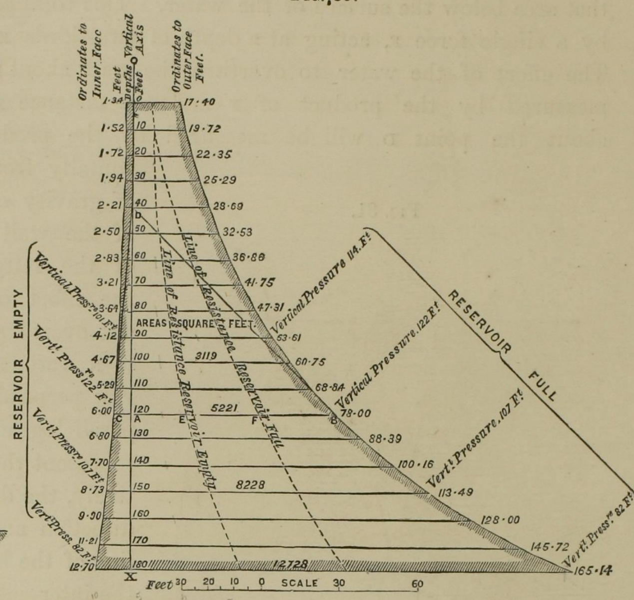


FIG. 85.



The following tables give the cubic contents for the two types:—

Depth measured from top	Quantity of masonry		Depth measured from top	Quantity of masonry		Depth measured from top	Quantity of masonry	
	Per linear metre	Per linear yard		Per linear metre	Per linear yard		Per linear metre	Per linear yard
	cubic metres	cubic yards		cubic metres	cubic yards		cubic metres	cubic yards
2	10·253	13·411	20	144·454	188·946	36	456·055	596·529
4	21·012	27·484	22	170·530	223·053	38	514·940	673·541
6	32·278	42·229	24	199·503	260·950	40	579·283	757·702
8	44·052	57·620	26	231·380	302·645	42	649·912	860·085
10	56·330	73·680	28	267·020	349·262	44	726·830	950·694
12	69·120	90·409	30	307·312	401·964	46	810·020	1059·506
14	83·608	109·359	32	357·247	467·279	48	899·515	1176·566
16	100·993	132·099	34	401·828	525·591	50	995·300	1301·852
18	121·275	158·628						

Depth measured from top	Quantity of masonry		Depth measured from top	Quantity of masonry		Depth measured from top	Quantity of masonry	
	Per linear metre	Per linear yard		Per linear metre	Per linear yard		Per linear metre	Per linear yard
	cubic metres	cubic yards		cubic metres	cubic yards		cubic metres	cubic yards
9	45·00	58·860	25	213·32	279·022	39	559·38	731·669
11	56·60	74·033	27	248·84	325·483	41	629·98	824·014
13	70·44	92·135	29	288·44	377·279	43	706·70	924·364
15	86·78	113·508	31	332·52	434·936	45	789·84	1033·111
17	105·86	138·465	33	381·42	498·897	47	878·70	1149·349
19	127·88	167·267	35	435·36	569·451	49	976·60	1277·393
21	153·02	200·150	37	494·62	646·963	51	1080·00	1412·640
23	180·44	236·015						

When the sides of the valley close in at any level to a width and for a length equal to the thickness of the wall at that level, the part of the dam below has no longer to resist the effect of the water-pressure to overturn the dam, for in such case this pressure is transmitted to the sides of the valley. The increase in the thickness of the dam from this level downward may therefore be modified, as long as the intensity of stress does not exceed the stipulated maximum; for, as far as the stability of the dam is concerned, its thickness at any level need not be greater than the corresponding width of the valley. The sections given in Figs. 84 and 85 being very nearly profiles of equal resistance, may be used for dams of any height up to 50 metres, by taking a corresponding height from the upper parts of the figures ; provided, of course, that the density of the material of the dam and the limiting working stress are the same as in the example for which those figures were calculated. In the figures are shown the curves of pressure when the reservoir is full and when empty.

In Fig. 85 is given a profile for a dam 180 feet high, worked out by the late Professor Rankine from independent calculations. It may be used for any less height in the manner described for M. Delocre's profiles.

The dam of the Furens reservoir, as actually constructed, is shown in Fig. 86. Fig. 88 is the Barrage de Puentes before referred to ; the section is very nearly 50 per cent. greater than that of Fig. 86, and the maximum stress on the material is even greater than in the latter case, being 7·9 kilos per square centimetre. Fig. 87 is the Barrage of Almanza, 20·7 metres in height. Its contents are about 1½ per cent. less than for a corresponding

Fig. 86.

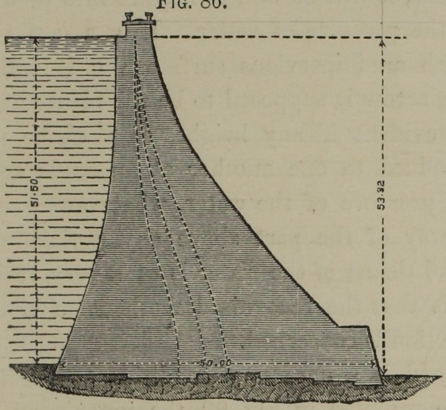


Fig. 87.

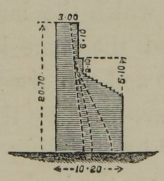
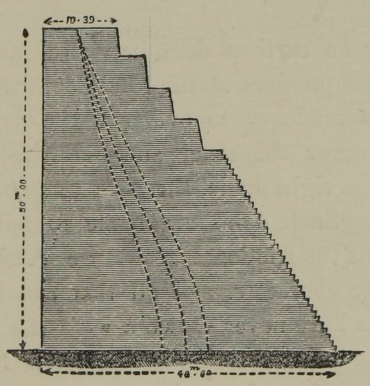
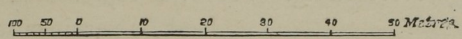


Fig. 88.





height of the type given in Fig. 83, but the maximum stress is 14 kilos per square centimetre instead of 6, as in the latter. This prominently illustrates the desirability of selecting a true profile.

Large dams should never be constructed of masonry where they cannot be conveniently carried down to a foundation on the solid rock. The dam of the Furens reservoir is on mica schist, and the greatest care was taken to secure an excellent foundation. The Barrage de Puentes, on the contrary, was built upon piles driven into a bed of alluvium, and in the year 1802 it gave way at the base, as might almost have been predicted from the circumstances of the case. Upon earthen foundations it will generally be found advisable to form the dam of earthwork ; indeed, the cost of masonry under such circumstances would almost always preclude its adoption.

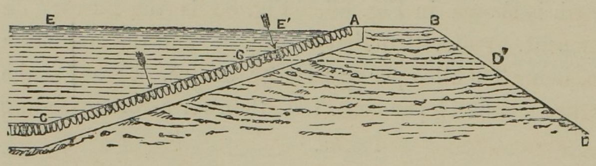
In determining the proportions of earthwork dams it must be remembered that a limit is assigned in one direction by the slope at which the material can be made to stand. The down-stream slope should seldom be trusted at less than 2 (horizontal) to 1 (vertical), while the up-stream slope, having its angle of stability somewhat reduced by the presence of the water, should never be less than 3 to 1. These slopes will be found almost the universal practice in this country, and for general cases there appears to be no reason for modifying them. The influence of the water on the material forming the inner part of the dam demands careful study. Some clays which, when excavated, are solid and firm, will have their consistency so changed when exposed to water that they will barely stand at 6 or 7 to 1. Sometimes the outer slope is made 2½ to 1, and, when the inconvenience and dangers of a failure are of such magnitude as they are with large bodies of water, an increase of this kind in the margin of safety will never be condemned. The slopes being determined, the next operation is to fix the width or thickness of the bank at the top water-line. This, again, is regulated alone by practical considerations, for the dam must be carried a certain height above the water-line, and at this height a certain width must be allowed for a footway, which cannot well be less than 6 or 8 feet, is frequently made from about 12 to 20, and is suggested by Mr. Rawlinson to be so much as 30 feet. Moreover, the width on top must be at least sufficient to afford protection to the puddle-wall where this is introduced, and should not be less than two and a half to three times the top thickness of it. We shall thus find the thickness of the dam at the top water-line to vary from about 30 feet to more than double that amount.

The height to which the dam should be carried above top water-line will depend upon the length of 'fetch,' that is, the greatest length of the reservoir along which the wind may blow down upon the dam. This distance will determine the greatest height of the waves; and the following formula, which accords fairly with observations on lochs and other similar sheets of water, and is taken from Mr. Stevenson's work on Harbours, may be of some guide.

$$H = 1{\cdot}5\sqrt{D} + (2{\cdot}5 - \sqrt[4]{D})$$

in which H is the height of the waves in feet, and D the length of fetch in miles. It must be remembered that the waves of greatest height may be generated when there is a considerable depth of water flowing over the waste-weir, and a further allowance must be made for this, beyond which again there should be a reasonable margin of safety. The difference of level between the waste-weir and the top of the dam should never be less than 3 feet, and it will, mostly, be found necessary to make it much more than this. A very good practice is to steepen the inner slope above the water-line, as shown in Fig. 91, and on Plate 3; or, in addition, to continue the pitching-up into a dwarf wall, as shown on Plate 25, Fig. 7.

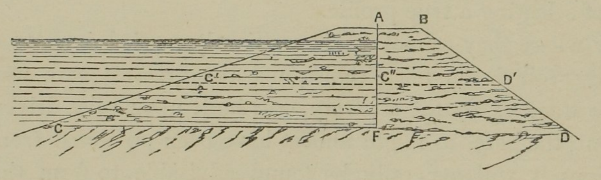
Fig. 89.



Having seen some of the limits which practical considerations assign to the forms and dimensions of earthwork dams, it will be well to inquire into their stability against the pressure of water. If A B C D (Fig. 89), representing an earthen dam, be viewed as a compact mass, with an impervious surface A C, it will be seen that the water-pressure of the total depth E C, of which the large arrow is supposed to be the resultant, has not the slightest tendency to overturn the dam, and the same is evident if any height, as E′ C′, be regarded. Indeed, whenever the inner and outer faces A C and B D are inclined to one another at an angle equal to or greater than a right angle, the dam cannot be overturned by the pressure of the water, whatever be the weight of the dam. Such dams are liable to fail only by a sliding away of the part above any horizontal section, as C′ D′. But the frictional resistance which opposes the horizontal thrust of the water is of a very variable and indeterminate character, and all that can be said on the point is that the recognised methods of constructing earthen dams have never shown a weakness as far as the sliding action is concerned.

But the impermeability of the outer face A C above mentioned is a necessary condition to the foregoing remarks concerning the resistance to overturning. For if, as represented in Fig. 90, the inner portion of the bank is pervious to water, and the water penetrates to the impervious surface A F before it is resisted, then the conditions of equilibrium are very different. The entire pressure of the water has to be withstood by the part A B F D; and not only so, but this part has also to sustain part of A C F, which would otherwise fall outward by its own gravity. Moreover, the penetration of the water as far as A F reduces the section of the material which has to resist the horizontal thrust of the water, say above C′ D′, in the proportion of C′ D′ to C″ D′. It does not follow, however, that the dam represented in Fig. 90 must fail because the water thus penetrates it for a certain distance. All that results from this is that its stability is much reduced and should be investigated.

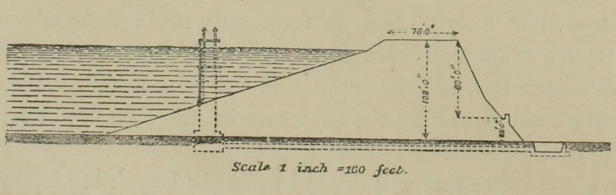
Fig. 90.



The most simple method of forming a bank is that which has been practised in India from the earliest times. As stated in a previous chapter, that country abounds with reservoirs, some of which are of enormous extent; but in all cases the entire section of the bank is uniformly well consolidated and impermeable, no special precautions against water-penetration being taken either on the inner face or in the heart. The earth is carried in small quantities, mostly in baskets, by men, women, and children, trodden and punned by the feet of the workers, and also by animals (sometimes by elephants), until the whole is left a thoroughly dense and compact mass.

The Cummum tank in the presidency of Madras is, according to native testimony, one of the oldest of these works, and its water surface is eight square miles in extent. The dam or 'bund,' as it is called (Fig. 91), is 102 feet high, and is formed to retain a little over 90 feet in depth of water. The inner slope is 3 to 1 below the water-line, but a little steeper above it. The outer slope is in two grades — ½ to 1, and about 1 to 1, as shown in the figure. Both the inner and outer slopes are heavily pitched with stone.*

Fig. 91.

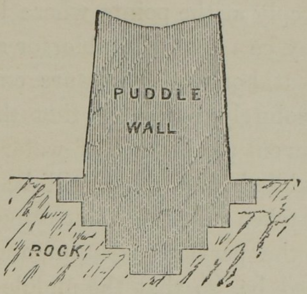


The most approved practice of forming dams in this country is that in which a wall of puddled clay is placed in the centre, and carried down in a trench so

* Appendix, by Mr. C. Brumell, C.E., to the Report on the Dale Dyke Reservoir Failure, 1864.

as to form a sound and water-tight junction with an impervious stratum. Sometimes the wall is founded on a layer of concrete on the bottom of the trench, as shown in Fig. 7, Plate 25, and also on Plate 17. The bottom of the trench should be cut longitudinally into horizontal trenches as shown in Plate 2, Fig. 1. Puddle-walls are made from 6 to 12 feet in width at the top, having a batter back and front of from 1 to 2 inches in a foot; but it would appear to be more consistent with the variations in the water-pressure at different depths to adopt a less top width and a sharper batter, such as is shown on Plate 2, Fig. 2, and Plate 17, Fig. 1. At no part should the thickness of the puddle-wall be less than a third of the head of water against it. Measured by this rule it will be observed that all the puddle-walls of the embankments shown on Plate 3 are very thin. Where a pervious bed underlies the seat of the dam the trench for the puddle-wall should at least retain the full width of the latter until the water-tight stratum is reached. Below this level, however, the width may be contracted, as the object is then chiefly to form a water-tight junction between the wall and the retentive stratum, as before mentioned. If this stratum be rock or other material of such a nature that the puddle cannot be incorporated with it at the surfaces in contact, the trench should be cut transversely as well as longitudinally into steps or benches. This method is seldom adopted in this country, but is practised generally in America. Fig. 92 is a cross section of the base of a puddle-wall formed in this manner, and it will be at once seen that should the water attempt to force its way between the puddle and the rock it will have to continually change its direction at the several right angles, and thus will be prevented from acquiring a dangerous velocity, by which the puddle would be scoured away.

Fig. 92.



Clay used for puddle should be free from loam, and should be divested of all perishable materials, pieces of stick, roots of plants, or other vegetable matter. Where subject to great hydrostatic pressure, it should be free from stones; but where resisting the pressure of only a few feet of water, and at the same time having to sustain heavy weights, it may be mixed with an equal bulk or less of screened gravel.

Next to the puddle-wall, both back and front, and in each case of the same thickness as the wall, should be placed the finest and most retentive material that is available. If the material in contact with the puddle were coarse and porous, there would be danger to the wall from its exposure to the direct pressure of the water on the inside, and from its liability to dry and crack on the outside. Very particular attention should be paid to this selected material, and it should be disposed in thin layers inclined down towards the puddle-wall. These should never be more than 2 feet in thickness, and they are often limited to 6 inches; they should be watered and well consolidated by punning or otherwise. (*See* the Specification for the Dundee Water-works, at the end of this work.) In the Bann reservoir, the inside of the puddle-wall was lined with peat 3 feet in thickness, so that, should any leak occur, the loose fibres, it was supposed, would be sucked in, and thus tend to close the hole or fissure.

Frequently it will be convenient to form the whole dam of a fine retentive material. In such a case the whole of the inner portion should be made water-tight, and the puddle-wall may then be regarded as a reserve precaution against failure. But, on the other hand, the outer and inner portions of the dam will sometimes have to be formed of stones, fragments of rock, and the like, and in such cases the selected material and puddle-wall will have to bear the whole of the water pressure on the inside.

The inner slope of the dam should always be protected against the wash of the ripples and waves by rubble stone, or other suitable pitching. Where the inner part of the bank is of a retentive nature, a layer of puddle may be spread over the whole of the inner slope, thus forming an apron so as to effectually exclude the water from the dam, and enable it with greater certainty to resist the water pressure after the manner indicated in Fig. 89 rather than that indicated in Fig. 90. When this expedient is adopted the puddle should be protected against puncture by having a layer of selected material above and below it. If the dam be founded on a porous stratum, the apron should be carried down in a cheek trench, to prevent the water rising under the inner slope of the dam. Some have considered that instead of placing a puddle-wall in the centre of the dam, all the puddle should be placed on the inner slope. The length of the inner slope, however, being generally more than three times the height of the dam, the same amount of puddle can be disposed in a central wall into a mass of more than three times the thickness, while the head of water to be resisted is the same in both cases for equal depths below the surface. Moreover, the larger the surface which has to be rendered water-tight, the greater is the liability for a faulty part to be overlooked; and, again, unequal settlement in the bank would tend to disturb the continuity of a surface layer of puddle. The back of the dam, or the down-stream portion of the selected material, may be of any heavy material that will stand the weather, for this part has to act simply by its weight; broken stones, débris of rock, or the roughest and least retentive of the material available for the formation of the dam, may be here introduced, and it should be disposed in layers inclined down towards the puddle-wall, so that the resistance to sliding may be increased. With this latter view, also, it is a very good practice to bench out the seat of the dam, as shown in Fig. 1, Plate 17.

In connection with the important subject of reservoir-making, the geology of the district has to be closely regarded, not only as far as the water-tightness of the reservoir itself and the facility for obtaining suitable materials for the dam are concerned, but also that a suitable foundation for the dam may be secured. In all works of importance the nature of the substratum should be ascertained by boring. The foundation must be firm and solid, for the weight of earthwork and water to be imposed upon it will be considerable. Loose and compressible material, and such as is likely to ooze away, as it were, under the superincumbent load—bog, silt, some clays and marls—should be avoided or removed, so as to expose a firm stratum below. No vegetable or other decayable matter, stumps of trees for instance, should be permitted to remain on the seat of the bank, any more than in the bank itself. These precautions are necessary in order to avoid an undue subsidence, which would threaten the safety of the dam. But the resistance to sliding must not be ignored; valleys often incline very steeply at the points where banks are placed; the strata may also dip rapidly down the valley; the conditions may be very favourable for a landslip, and the intrusion of the water, so as to act as an unguent, may, with the great horizontal pressure exerted, suffice to disturb the equilibrium and bring about the catastrophe. It is not absolutely necessary that the whole seat of the dam should be perfectly impervious; the puddle-wall will prevent the leakage of water past the dam if it be properly carried down to a water-tight stratum. If not, the puddle in the trench will be gradually washed away and the entire dam must sooner or later fail. Care must be taken that the water cannot find its way round the end or ends of the puddle-wall, as, with certain dispositions of the pervious stratum, and without due precaution, it is most likely to do; the trench must be extended, if need be, so that the puddle-wall may be tied in to the impervious stratum at the ends up to the level of the top water as well as carried down into it, even if the configuration of the valley seems to suggest stopping short of this. Sometimes it will be necessary to return the ends of the trench up along the sides of the valley before the pervious stratum can be completely shut off by a wall of puddle. This is illustrated in the Vale House reservoir of the Manchester Waterworks, Plate 25, Fig. 1; and it may be here mentioned that the stratification in this is shown on the plate just as it was discovered by the excavations, and the example is, therefore, of the greater interest.

Again, springs may rise on the site of the dam, and great care must be exercised in dealing with them. They are likely to cause damage in several ways; if they are in the rear of the puddle-wall they may lessen the stability of the material of this part of the dam so that it will not stand at its original slope. For the spring may be intermittent and not flow until the dam has been carried up to a considerable height; and the time of the rising of the spring will most likely be the season in which the reservoirs will be filled, so that the stability of the dam will be reduced at the very time when it will be most taxed, and the chances are that failure will ensue. Or the running of the spring may gradually wash away and undermine the base of the back of the dam, when a like result must follow. But if the back of the dam be porous, consisting of broken stone, debris of rock, or the like, or any material whose stability will not be affected by the presence of the water, a spring would be harmless if it did not scour away the substratum, and of this there need not be any fear if the water run from the outer toe of the dam clear and free from suspended matter.

All puddle must be protected against the washing and scour of a continual current, and precautions of this kind are frequently necessary for the puddle in the trench. Hydraulic concrete is perhaps the best material to use for the purpose, and it is well to protect with it the face of the puddle in trenches cut in a porous or water-yielding stratum.

Springs on the site of reservoir dams must of course be treated according to circumstances. With due precautions, danger from them may generally be avoided, and it will be only in very intractable cases that a retreat will have to be made and another site selected. Concrete might perhaps be used with advantage, much more frequently than it has hitherto been; and, combined with proper channels for drainage, the water on the seat of the dam, if not retained, would at least be prevented from doing injury.

Fig. 7, Plate 25 shows how a spring, which was met with in the puddle trench of the Vale House reservoir, and could not be suppressed, was led off by iron piping. and thence in a rubble drain laid under the back of the dam (not shown in the drawing). The wall of rubble in mortar, shown lining one side of the puddle trench, was carried up to protect the puddle against being washed away by any leakage between it and the solid ground. In the case under consideration, the shale into which the puddle trench is carried was found much disturbed and dislocated by a fault which runs up the valley. At this dislocation water was met with, and was safely provided for by the method above described.

Where it is deemed prudent to select another site for the dam, and other circumstances are not unfavourable it is of course obvious that the movement should be made down stream, in order that the spring may be diverted into the reservoir and thus turned to good account.

The danger of water rising under a dam is not unfrequently incurred by injudiciously excavating from the interior of the reservoir in a manner before mentioned, exposing a porous or fissile stratum which may afford to the stored-up water direct communication with the seat of the dam. Nevertheless, the principle of excavating from the interior of a reservoir cannot be regarded as a bad one; for not only is the material at hand, and on the same property, but the excavation at the same time increases the reservoir capacity. All that can be urged is that preliminary observation should be made to ascertain whether such excavations can be conducted without danger; and, further, caution must be exercised in directing them.

Not less important than the construction of the dam itself are the arrangements for drawing off the water. In assigning dimensions to these parts, it is advisable to make the discharging capacity sufficient not merely for the maximum anticipated uniform discharge for ordinary purposes, but also that, in case of need or emergency, the water in store might be run off very rapidly. The simplest method of forming the outlet, and at the same time most dangerous and unwarrantable, is to run a culvert or naked pipe in the made ground. The source of danger is obvious; there will be unequal pressure on account of the greater height and weight of material in the centre of the dam; there will in consequence be unequal settlement; and, whether pipe or culvert, there is almost a certainty of failure from the leakage that is sure to take place. After the experience of the many disasters resulting from this practice it is now never followed, and the discharging channels are invariably carried for their whole length in the solid ground, or at least below the natural surface. Sometimes they are entirely surrounded with puddle; but when such is the case, the depth of puddle which is underneath the pipe or culvert should be kept as shallow as possible consistent with the other objects for which it is introduced. For the puddle is but made ground after all, and is liable to settlement and compression; so that with too great a depth of puddle under the culvert or pipe and the unequally distributed load of the earthwork above, there will still be unequal settlement as dangerous as before. The worst practice would be to increase the depth of puddle under the pipe or culvert at the part where the superincumbent pressure will be greatest, and of this there was an actual example in the case of the Dale Dyke reservoir of the Sheffield Waterworks, memorable for the dire calamity which is of almost historical interest. Figs. 21 and 22, Plate 3, are reduced from drawings appended to the official report on the Dale Dyke catastrophe, and represent plan and sections of part of the trench in which the outlet pipes, two in number, were laid. Where there is such considerable deflection, pipes are obviously exposed to many dangers; if they are bolted together with flanges, or if they have long turned and bored socket joints, with close-fitting cylindrical or conical surfaces (see Chap. XIII.), they are almost sure to be fractured. If turned and bored joints be used at all in such positions, they should be made with spherical surfaces, so as to admit of motion within certain limits (Plate 12, Fig. 34). If they have the ordinary run-lead joint, fracture is not so likely to arise from deflection of the train of pipes, but the joints are liable to draw, at least sufficiently so to become leaky, that is, unless precaution be taken to avoid this. In the case of the Dale Dyke reservoir, the joint used (Plate 12, Fig. 39), was designed to prevent this. Culverts of masonry or brickwork will of course suffer more with the same deflection than pipes with proper joints; indeed, culverts under dams should not be bedded in puddle, but in concrete on the solid bottom of the trench (see Plates 3 and 21). Timber has been used under the culvert (see the Barden and Silsden reservoir, Plate 3), but the prudence of its introduction here is very questionable.

It must not be forgotten that culverts and naked pipes have to resist a considerable pressure from without. If this pressure were perfectly uniform around the cross section of the pipe or culvert, the iron pipe might be considered to be on equal terms with the masonry culvert. But this is not the case, the pressure is greater vertically than laterally, and the culvert, from its usually greater thickness of material in proportion to the diameter, has a corresponding advantage over the pipe in resisting the external pressure of the earth. Under a bank of one of the service reservoirs of the Liverpool Waterworks, two lines of pipes, each 44 inches diameter, and each numbering ten or twelve pieces, were laid within 16 feet of the top. 'Fully one-third of the pipes so placed,' says Mr. Hawksley, the engineer, 'which were excellent castings, were broken, although they had borne a pressure of 300 feet internally.'*

In order to prevent the deflection of a train of pipes when laid in puddle under a dam, they are sometimes supported at intervals on piers of masonry carried up from the solid bottom of the pipe-trench (Plate 24, Fig. 1.) This method of course accomplishes the immediate end in view, but it exposes the pipes to the danger of being fractured, as cylindrical beams having to sustain the load between the piers. In the works for supplying Melbourne, Australia, with water, two lines of pipes, each of 33 inches diameter, 1⅝ inch thick, and cast in 6-feet lengths (the largest which, under the circumstances, could be obtained from the founders in Sydney), were laid under a reservoir dam about 25 feet high, each length being supported on a pillar of ashlar. Both the mains became fractured near the centre of the dam; and as it was out of the question to run off the water from

* Trans. Inst. of Civil Engineers, Vol. XVIII. p. 387.

such an extensive reservoir, nearly full at the time, the following expedient was adopted:—'Rings of boiler plate half-inch thick, and 2 feet 6 inches in length, were turned to an outer diameter of 32⅝ inches. At one end of each ring, in the interior, a junction strip, 6 inches wide and half-inch thick, was riveted. A ring was then passed up each main to beyond the fractured part, which in one case was found to be situated outside the puddle-trench. This ring was secured within the pipe by driving strips of hard wood between the ring and the pipe, and rusting up the whole with iron cement. A second ring was then passed up, and was joined to the first by tap bolts, passing through the first junction strip and the second ring. This second ring was then fixed in the same way as the first, and the processes were repeated until there was a complete tube of boiler plate in each main for the width of the puddle-trench and the aprons. The whole of the pipes were then well painted. The section of the pipes still remained equal to, or rather larger than, the section of the main.'* A similar expedient was adopted with one of the discharge pipes of the Woodhead reservoir of the Manchester Waterworks.

The conditions, then, it would appear, which should be fulfilled if the rupture of a naked pipe is to be prevented, are that it should be surrounded with puddle for its whole length, piers of masonry being avoided; that the depth of puddle below the pipe should be uniform from end to end of the pipe-trench, and as shallow as is consistent with ensuring a sound water-tight coating; that if it be necessary to carry the pipe-trench, or any part of it, to a greater depth than this, where crossing over the puddle-trench, for instance, or where it is necessary to go down for a solid bottom, the lower part of the trench should be filled in with concrete or masonry, or other equally rigid material, so as in this way to avoid too great a depth of puddle under the pipe, and to make that depth uniform.

Culverts require a solid bearing for their whole length, and, where crossing the puddle-trench, should be supported on concrete or masonry carried up from the bottom (see Plates 3, 19, and 21) otherwise a fracture is almost sure to take place. There is much difference of opinion as to the use of ashlar or rubble for culverts under dams; but the selection of one or other form of masonry should depend chiefly upon whether the water in the culvert will be under pressure or not. If the culvert have to resist only the external pressure of the earth—as an arch—it should be of ashlar, but if the water in it be under considerable pressure, either hydrostatic or hydraulic rubble may, perhaps, be preferred, as the weakness of ashlar on the beds is avoided. This question of pressure leads to the considerations of the proper position, &c., for the valves and sluices. Wherever the valves or sluices be placed, the length of pipe or culvert on the up-stream side of them will be exposed to the full hydrostatic pressure due to the level of the water in the reservoir. Iron pipes can be made to withstand this, and, were it not for other considerations to be hereafter noticed, the valves for iron outlet-pipes might be placed at their lower or down-stream end, and as in Fig. 4, Plate 3; but to expose culverts of masonry to such internal pressure is of course out of the question. It must be remembered, however, that, above the centre of the dam, there is an external pressure equal to, if not greater than the internal pressure, and due to the weight of earth and water, so that, as far as danger of bursting is concerned, sluices may be placed in a culvert as low down as the centre of the dam. Of this method, Figs. 1, 5, 12, and 15, Plate 3, are good examples. What is to be feared from placing the valve or sluice in a pipe or culvert anywhere below the puddle wall is, that should there be any leak from settlement, rupture, or joint blowing or drawing, or from imperfect workmanship, the water would force itself out with a pressure far too great for the puddle coating to resist, and the dam would be in danger as imminent as if a spring had risen under its seat. When the valves or sluices are placed at the inner end of the outlet, the conditions of pressure will depend upon circumstances. Sometimes the main or conduit pipe is carried uninterruptedly from the interior of the reservoir so as to take advantage of the full head of water in the reservoir. In such cases the pressure in the pipe will not be materially diminished within the length of the pipe under the dam. Generally the pipe or culvert discharges itself into a basin or an open conduit. The conditions of hydraulic pressure within the pipe or culvert will then be under the control of the sluices or valves at the inner end. But unless the pipe or culvert be throttled or contracted, the pressure below the puddle wall—and this is where leakages would be most serious—can never, in dams of the ordinary proportions, be more than half the depth of water in the reservoir, and the arrangements are very rarely such that even this can obtain.

Leakages are very likely to creep along the outside of a pipe laid in puddle, especially when from settlement or subsidence the puddle is drawn away from the pipe. The flanges of the pipe will arrest this to some extent; but to guard effectually against the consequences of creeping, there should be one or more collars or shields on the pipe (Plate 24, Fig. 1, and Plate 39). The projecting rings round the culvert shown on Plate 2, Fig. 2, will act in a similar manner.

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the relative economy of these two methods, which will, of course, depend principally upon the nature of the ground and the depth of the outlet below the surface, the tunnel, if it run under the dam, will in one respect be more efficient, for it will not so much disturb the foundation of that part of the dam, and, further, a line of subsidence will be avoided. If the tunnel be driven round the end of the dam, and not under it, these arguments for efficiency will partly hold good, but economy will most likely have already decided in favour of the tunnel.

Perhaps the most complete and safe method, especially where the water in the outlet will be under considerable pressure, is to carry pipes in a culvert or tunnel so that there may be complete access to the former for inspection and repairs. By these means the bursting pressure will be resisted by the pipe or pipes, and the culvert will sustain the crushing force. Moreover, should any leak occur from the pipes, the water will be safely conducted away without effecting any damage. This method is illustrated on Plate 17, and it will be as well here to remark that the culvert is not run into the made ground, as might be supposed from Fig. 1, but is carried into the solid, as indicated by Fig. 3, Plate 16, and Fig. 2, Plate 17, the trench being filled up to the natural surface of the ground with puddle.

The apparatus—valves, sluices, &c.—for regulating the discharge of the water is generally at the inner end of the outlet, and the advantages of this arrangement have already been pointed out. Where there is not a very great depth of water, a common sluice valve is sometimes fixed at the end of the pipes, and worked by a rod, either running up the slope of the bank or passing vertically through guides to a platform at the end of a light gangway or footbridge. In the Crombie Den reservoir of the Dundee Waterworks (Plate 19) the discharge is regulated by a simple iron sluice, worked by a rod from the top of the masonry tower or well (Figs. 12 to 19).

There is always more or less difficulty, or at least inconvenience, in working sluices, valves, plugs, and the like under great pressure; and to lessen this as much as possible, and also to be able to draw off water at a comparatively short distance below the surface, where it is always clearer, various means are adopted. For the Whittle Dean Waterworks an arrangement was made by Sir William Armstrong which may be thus described.

The end of the discharge or outlet pipe, which is 24 inches in diameter, is curved upwards, and terminates in a flange. Placed upon this flange, one above the other, are three lengths of flanged pipes, the upper one being domed in at the top. The pipes are connected together by loose bolts passing through the flanges, and having a play of about 9 inches in the direction of their length. To the upper length is connected a screwed rod, and a cross-head working in suitable guides. The apparatus, when at rest, perfectly excludes the water from the outlet pipe, and when water is to be discharged the upper length is raised by the screw so as to leave an opening between it and the second length. The quantity discharged is regulated by the width of this opening and the head of water above it. When the water level in the reservoir falls below this opening, the upper length is further raised, and the bolts then catch the second length, leaving an opening between it and the third or next lower length, through which, of course, the water escapes as before. As the upper lengths are still further raised, the water is let out through the lower joint, and the discharge is thus under complete control. If it be required to discharge water from the lower opening when the reservoir is full, this arrangement will enable such to be done without the full pressure of the water having to be resisted in the process. In the large reservoirs of India, to which reference has been already made, the water is usually let off through a masonry culvert communicating with a shaft of masonry, as shown in Fig. 91. Into the shaft the water is admitted by pipes closed by wooden plugs, which are raised or lowered as required by chains passing up to a platform at the top of the shaft. This method was adopted in principle for the Vehar reservoir of the Bombay Waterworks (Plate 4).

The water is drawn from this reservoir, through a tower provided with four inlets, fixed at vertical intervals of 16 feet apart. These inlets are 41 inches in diameter, and are provided with conical plug seats faced with gun-metal. The three inlets not in use are kept closed by conical plugs, fitted by grinding (A, Fig. 6). These plugs are suspended exactly over their seats from the balcony above, and are raised or lowered at will by cranework at the top of the tower. The inlet in use is surmounted by a wrought-iron straining cage, covered with No. 30 gauge copper wire (F, Fig. 6), and fixed to a conical ring, fitting into the inlet orifice in the same manner as the plugs, and equally capable of being raised or lowered at pleasure. This strainer presents a surface of 54 square feet. The gauze is affixed to a cage, so as to admit of its being changed from a boat, when clogged, in ten minutes after the cage has been drawn up to the surface, or a plug may be substituted for the cage and lowered to its place in the same time.

At the bottom of the inlet well, and exactly over the orifice of the supply main, another conical seat is fixed, into which a similar straining cage (but with No. 40 gauge copper wire), and presenting a surface of 90 square feet, is inserted. The water thus passes through two strainers before it leaves for Bombay. The primary object of this arrangement was to obtain in the town distribution the benefit of the additional head of such an extensive reservoir, nearly full at the time, the following expedient was adopted:—'Rings of boiler plate half-inch thick, and 2 feet 6 inches in length, were turned to an outer diameter of 32⅝ inches. At one end of each ring, in the interior, a junction strip, 6 inches wide and half-inch thick, was riveted. A ring was then passed up each main to beyond the fractured part, which in one case was found to be situated outside the puddle-trench. This ring was secured within the pipe by driving strips of hard wood between the ring and the pipe, and rusting up the whole with iron cement. A second ring was then passed up, and was joined to the first by tap bolts, passing through the first junction strip and the second ring. This second ring was then fixed in the same way as the first, and the processes were repeated until there was a complete tube of boiler plate in each main for the width of the puddle-trench and the aprons. The whole of the pipes were then well painted. The section of the pipes still remained equal to, or rather larger than, the section of the main.'* A similar expedient was adopted with one of the discharge pipes of the Woodhead reservoir of the Manchester Waterworks.

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The end of the discharge or outlet pipe, which is 24 inches in diameter, is curved upwards, and terminates in a flange. Placed upon this flange, one above the other, are three lengths of flanged pipes, the upper one being domed in at the top. The pipes are connected together by loose bolts passing through the flanges, and having a play of about 9 inches in the direction of their length. To the upper length is connected a screwed rod, and a cross-head working in suitable guides. The apparatus, when at rest, perfectly excludes the water from the outlet pipe, and when water is to be discharged the upper length is raised by the screw so as to leave an opening between it and the second length. The quantity discharged is regulated by the width of this opening and the head of water above it. When the water level in the reservoir falls below this opening, the upper length is further raised, and the bolts then catch the second length, leaving an opening between it and the third or next lower length, through which, of course, the water escapes as before. As the upper lengths are still further raised, the water is let out through the lower joint, and the discharge is thus under complete control. If it be required to discharge water from the lower opening when the reservoir is full, this arrangement will enable such to be done without the full pressure of the water having to be resisted in the process. In the large reservoirs of India, to which reference has been already made, the water is usually let off through a masonry culvert communicating with a shaft of masonry, as shown in Fig. 91. Into the shaft the water is admitted by pipes closed by wooden plugs, which are raised or lowered as required by chains passing up to a platform at the top of the shaft. This method was adopted in principle for the Vehar reservoir of the Bombay Waterworks (Plate 4).

The water is drawn from this reservoir, through a tower provided with four inlets, fixed at vertical intervals of 16 feet apart. These inlets are 41 inches in diameter, and are provided with conical plug seats faced with gun-metal. The three inlets not in use are kept closed by conical plugs, fitted by grinding (A, Fig. 6). These plugs are suspended exactly over their seats from the balcony above, and are raised or lowered at will by cranework at the top of the tower. The inlet in use is surmounted by a wrought-iron straining cage, covered with No. 30 gauge copper wire (F, Fig. 6), and fixed to a conical ring, fitting into the inlet orifice in the same manner as the plugs, and equally capable of being raised or lowered at pleasure. This strainer presents a surface of 54 square feet. The gauze is affixed to a cage, so as to admit of its being changed from a boat, when clogged, in ten minutes after the cage has been drawn up to the surface, or a plug may be substituted for the cage and lowered to its place in the same time.

At the bottom of the inlet well, and exactly over the orifice of the supply main, another conical seat is fixed, into which a similar straining cage (but with No. 40 gauge copper wire), and presenting a surface of 90 square feet, is inserted. The water thus passes through two strainers before it leaves for Bombay. The primary object of this arrangement was to obtain in the town distribution the benefit of the additional head of water (due to the depth of the dam), which would have been lost had the water been strained, as is the more usual arrangement, at the outside foot of the dam. It was also thought advisable to avoid the use of such heavy sluice valves as would be required for closing inlets of 41 inches in diameter, in positions in which it would be difficult to get at them for the purpose of effecting any necessary repair. Without this arrangement the utmost head attainable would have been insufficient for a distribution by gravitation alone.*

It is very advisable to provide means of access to the outlet valves for the purposes of inspection and repair, and, indeed, this is usually done. In the Crombie Den reservoir of the Dundee Waterworks, the inner iron sluice before referred to serves for ordinary purposes ; but should it become deranged, or in any way require inspection, the outer sluice is let down, and the water is run off from the well. This outer sluice is of African teak, and works in guide-frames of oak, as shown by Figs. 12, 17, 18, 19, Plate 19. At the Bideford Waterworks (Plate 17), and also in the Port Glasgow (Plate 24), there is a tower or well of cast-iron, into which the water is admitted or excluded by valves placed inside it. At the Gorbals Works, Glasgow, there is a similar arrangement, but the iron cylinder is built into an ornamental tower of masonry. When, therefore, the water is run off from the tower, the valves may be at least partially inspected. For the inspection to be complete, and for repairs to be executed, the outer ends of the pipes must be closed against the water by a plug or similar contrivance, and then, if necessary, the valves can be entirely removed. Very complete arrangements for accomplishing this are illustrated in the case of the Rotherham Waterworks (Plate 22). The ends of the three discharge pipes are turned up and finished with bell-mouths, and a tube or hollow rod passes down to each pipe from the platform above. When it is desired to close the orifice of any pipe, a ball of wood sliding upon the corresponding rod is lowered by chains, and the corresponding valve being opened inside the well, the ball is pressed by the weight of water above, thus effectually plugging the orifice. The valve can then be thoroughly examined, and, if need be, removed. In order to raise the ball, the valve must first be closed ; water is then poured down the hollow rod to the pipe between the valve and the ball, until the hydrostatic pressure under the ball is greater than that above it. The ball may then be drawn up by the chains without difficulty. It will be noticed that there are two wells or rather one well divided into two compartments by a wall of brickwork. The outer or wet well or compartment, as may be seen from Plates 21 and 22, is rendered necessary on account of the outlet being sunk to a considerable depth below the surface of the ground, communication with the reservoir being effected by means of three tunnels driven at the levels of the respective discharge pipes. It will be seen, further, that the outer compartment is made to act as a screening chamber.

In the Dean Head reservoir of the Halifax Works (Plates 31 and 32) there is a hollow octagonal tower of masonry enclosing a circular shaft of cast-iron, which is further carried down into the solid ground to the upper end of the inlet tunnel. The water is admitted into the masonry tower through openings provided with gratings. At levels corresponding with these openings, and also with a culvert and a tunnel leading from the reservoir, are openings in the iron shaft. These latter openings are closed by means of internal cylinders which slide up and down against brass facings, and are worked by rods from above. The junction between the bottom of the iron shaft and the inlet tunnel is effected by means of a short length of cast-iron pipe passing through a cast-iron saddle which forms the crown of a masonry arch abutting on the solid ground on each side (*see* Figs. 1 and 5, Plate 32). The joint between the pipe and the saddle is run with lead.

It is not essential that the outlet pipe or culvert should run for its whole length at the level down to which the water in the reservoir is required to be drawn off. The principle of the syphon may be employed (Plate 2, Fig. 2, and Plate 17, Fig. 1) in discharging the water when the quantity in store is running low. This will sometimes enable a more favourable site to be chosen for the outlet works; but unless some such advantage can in this way be gained it is not worth while to incur the liabilities to derangement which are frequently associated with the employment of syphons. An air-pump must be provided to exhaust the air which is sure to accumulate at the summit of the syphon, otherwise it will become choked and its action will cease.

Among the works necessary in connection with reservoirs formed by embanking across valleys are the waste-weir and the bye-wash—the bye-wash being the channel for discharging past the dam any water escaping over the waste-weir. The principal feature of the waste-weir is its length, which should be such as to allow the greatest flood that may be anticipated from the gathering ground to be discharged with not more than a given rise of water above the weir crest. This limiting depth of water flowing over the weir will depend to a certain extent upon the difference of level allowed between the weir crest and the top of the dam, but from other practical considerations it should not be more than from 2 feet 6 inches to 3 feet. The flood discharge which must be provided for has then to be determined. Where the opportunity is afforded, observations should be made concerning the proportion between the volume passing down the stream or streams feeding the reservoir, and the quantity of rain precipitated during the same period. The rate at which the floods will pass down the

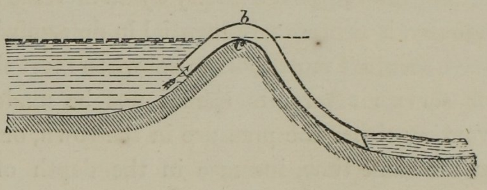
* Trans. Inst. of Civil Engineers, Vol. XVII.

streams will never be greater than the maximum rate of rainfall (*see* Chapter III.), and it will seldom, if ever, approach this limit. For, in the first place, all the absorbent material with which the rain comes in contact must be saturated, and then the several channels of drainage within the catchment area must come into train, as it were, before the main stream will be delivering an amount equal to the rain that is falling. Before this time the excessive fall will in all probability have moderated. The ratio of the maximum fall to the maximum discharge will be greater as the period of the excessive fall is shorter and the drainage area larger; and it will, of course, vary directly with the porosity of the formation, the amount of vegetation, and the flatness of the slopes of the drainage basin. As in many other details connected with the subject of drainage and water supply, actual observation alone can furnish reliable data; but there is a practice amongst engineers with regard to this matter, namely, to provide not less than 3 feet in length of waste weir for every hundred acres of catchment area. Allowing 2 feet of water above the weir crest, there will be a discharge of about 30 cubic feet per second, which corresponds with a fall of one-third of an inch per hour. In many cases it will be wise to provide for at least 50 cubic feet per second from each 100 acres of drainage area; for anything beyond this, provision will be required only where the drainage area is unusually small, and where excessive falls for very short periods, such as frequently accompany heavy thunderstorms, will make a sudden and dangerous addition to the volume of the streams.

Weirs are sometimes arranged so that as the water rises above them a greater length of overflow becomes available. Thus the waste weir of the Cummum tank before referred to is 232 feet in length, and along the crest there is a range of posts, 92 in number, 4 feet high and 10 inches wide; so that until the water reaches above the top of the posts only about 156 feet of overflow is available, and thus a large amount of water may be saved. These posts are also intended to support boards by which the depth of water in the tank may be regulated at will. Another practice finding much favour in India, where every drop of water is valuable, is to form on the waste weir a dam of earth and sods, by which the water level is raised some 3 or 4 feet without danger to the works. But when heavy floods set in, this temporary dam is overtopped and soon cleared away by the water, and the whole depth is then available to relieve the reservoir.

Mallet's syphon weir is an apparatus for regulating the water in case of floods, and consists of a syphon whose transverse section is a parallelogram, and which is carried over the crest of the weir as illustrated in Fig. 93. While the depth of water flowing over is less than the height *b*, *c*, of the syphon tube, the water flows as over an ordinary weir, with the rate of discharge due to the length; but when the level of the water rises to the height of the top plate, the syphon begins to perform its duties as such, and the discharge is greatly increased, becoming that due to the head measured from the surface of the water to the bottom of the long leg of the syphon. The maximum discharge may be regulated by a throttle valve in the bend of the syphon.

FIG. 93.



Overflow weirs should be placed, if convenient, in the solid ground away from the main dam. Great care should be bestowed upon their construction, for they are amongst the vital points of reservoir works. Figs. 25 and 26, Plate 3, show the waste weir of the unfortunate Dale Dyke reservoir, as originally constructed. It is very deficient in the length of overfall. The waste weir of the Vehar reservoir (Fig. 4, Plate 4), is formed by a dam of masonry, and a plank screen is erected in front to prevent, or at least lessen, the loss of water through the wind and the waves (see also Plates 17, 21, 25, and 31). Waste pits of masonry embedded in the dam itself have been sometimes adopted, but the practice is a dangerous one, and seldom can a sufficient length of overfall be in this way provided. See Fig. 18, Plate 3.

The water passing over the waste weir is conducted to the natural bed of the stream below the dam, or to another reservoir, as the case may be, by means of the 'byewash.' The byewash will generally have to be made with a very steep mean gradient, and to avoid the excessive scour which would result if an uniform channel were constructed, it is in most cases advisable to carry the byewash down by a series of steps, by which the velocity will be reduced. This is very well illustrated in the case of the Rotherham Works (Plates 20 and 21).

Reservoirs are sometimes constructed without any special works at the upper or inlet end; but it is in most cases advisable to provide some means for regulating the admission of water to the reservoir, and for entirely diverting it when deemed necessary. For this purpose sluices are fixed at the head of the reservoir, and an adjacent weir leads to a flood-water channel which joins the byewash at the waste weir (see Plate 20 and Plates 31 and 32). Were it not for the flood-water channel, all the water flowing down from the gathering ground would have to pass through the reservoir; and, should the reservoir happen to be full when the stream is running in flood, the excess of the supply over the demand, thus uselessly running into and out of the reservoir, would leave behind it all its solid impurities capable of depositing, to the detriment perhaps of the quality of the water in store, and at the same time unnecessarily increasing the accumulation of silt. Suitable works at the inlet