CHAPTER XVI.

172. Application of Fitzgerald's Results to the Croton Watershed.—The evaporation data determined by Messrs. Fitzgerald and Kuichling are sufficient for all ordinary purposes in the North Atlantic States. In the discussion of the capacity of the Croton watershed Mr. Fitzgerald's results will be taken, as the conditions of the Croton watershed in respect to temperature and atmosphere are affected by the proximity to the ocean, and other features of the case make it more nearly like the Metropolitan drainage area near Boston than the more elevated inland district near Rochester.

If the monthly amounts of evaporation be taken from the preceding table, and if it further be observed that a volume of water 1 square mile in area and 1 inch thick contains 17,377,536 gallons, the following table (Table XI) of amounts of evaporation from the reservoirs in the Croton watershed, including the new Croton Lake, will result, since the total area of water surface of all these reservoirs is 16.1 square miles.

TABLE XI.

Jan.	0.96 ×	16.1 ×	17,377,536 =	268,600,000	gallons.
Feb.	1.05 ×	" ×	" =	293,800,000	"
Mar.	1.70 ×	" ×	" =	475,700,000	"
April	2.97 ×	" ×	" =	831,000,000	"
May	4.46 ×	" ×	" =	1,247,900,000	"
June	5.54 ×	" ×	" =	1,550,100,000	"
July	5.98 ×	" ×	" =	1,673,200,000	"
Aug.	5.50 ×	" ×	" =	1,538,900,000	"
Sept.	4.12 ×	" ×	" =	1,152,800,000	"
Oct.	3.16 ×	" ×	" =	884,200,000	"
Nov.	2.25 ×	" ×	" =	629,600,000	"
Dec.	1.51 ×	" ×	" =	422,500,000	"
	39.20		Total =	10,968,300,000	"

It will be seen from this table that the total annual evaporation from all the reservoir surfaces of the Croton watershed, as it will exist when the new Croton Lake is completed, will be nearly 11,000,000,000 gallons, enough to supply the boroughs of Bronx and Manhattan at the present rate of consumption for about forty days.

173. The Capacity of the Croton Watershed.—The use of the preceding figures and numbers can be well illustrated by considering the capacity of the Croton watershed in its relations to the present water needs of the boroughs of Bronx and Manhattan which that watershed is designed to supply. The total area of the Croton watershed is 360.4 square miles, of which 16.1 square miles, as has already been observed, is water surface. As a matter of fact the run-off observations from that watershed have been maintained or computed for the thirty-two-year period from 1868 to 1899, inclusive, covering the evaporation from the reservoirs and lake surfaces as they have existed during that period. The later observations, therefore, include the effects of evaporation from the more lately constructed reservoirs, but none of these data cover evaporation from the entire surface of the new Croton Lake, whose excess over that of the old reservoir is nearly one third of the total water surface of the entire shed. As a margin of safety and for the purpose of simplification, separate allowance will be made for the evaporation from all the reservoir and lake surfaces of the entire watershed as it will exist on the completion of the new Croton Lake, as a deduction from the run-off. The preceding table (Table XI) exhibits those deductions for evaporation as they will be made in the next table.

In Table IX the year 1880 yields the lowest run-off of the entire thirty-two-year period. The total precipitation was 36.92 inches, and only 34.21 per cent of it was available as run-off. The first column in Table XII gives the amount of monthly rainfall for the entire year, the sum of which aggregates 36.92 inches. Each of these monthly quantities multiplied by .3421 will give the amount of rainfall available for run-off, and the latter quantity multiplied by the number of square miles in the watershed (360.4) will show the total depth of available water concentrated upon a single square mile. If the latter quantity be multiplied by 17,378,000, the total number of gallons available for the entire month will result, from which must be subtracted the evaporation for the same month. Carrying out these operations for each month in the year, the monthly available quantities for water-supply will be found, as shown in the last column.

TABLE XII.

(Jan.	3.43 ×	.3421	= 1.173) ×	360.4 ×	17,378,000 −	268,600,000 =	7,077,700,000
(Feb.	3.40 ×	"	= 1.163) ×	" ×	" −	293,800,000 =	6,989,900,000
(Mar.	3.90 ×	"	= 1.334) ×	" ×	" −	475,700,000 =	7,879,000,000
(April	3.57 ×	"	= 1.221) ×	" ×	" −	831,000,000 =	6,816,000,000
(May	1.04 ×	"	= .356) ×	" ×	" −	1,247,900,000 =	982,000,000
(June	1.40 ×	"	= .479) ×	" ×	" −	1,550,100,000 =	1,449,800,000
(July	5.86 ×	"	= 2.005) ×	" ×	" −	1,673,200,000 =	10,890,000,000
(Aug.	4.16 ×	"	= 1.423) ×	" ×	" −	1,538,900,000 =	7,373,100,000
(Sept.	2.42 ×	"	= .828) ×	" ×	" −	1,152,800,000 =	4,032,900,000
(Oct.	2.83 ×	"	= .968) ×	" ×	" −	884,200,000 =	5,178,500,000
(Nov.	2.32 ×	"	= .794) ×	" ×	" −	629,600,000 =	4,343,100,000
(Dec.	2.59 ×	"	= .886) ×	" ×	" −	422,500,000 =	5,126,300,000
	36.92						

The sum of the twelve monthly available quantities will give the total number of gallons per year applicable to meeting the water demands of the boroughs of Bronx and Manhattan.

174. Necessary Storage for New York Supply to Compensate for Deficiency.—At the present time the average daily consumption per inhabitant of those two boroughs is 115 gallons, and if the total population be taken at 2,200,000, the total daily consumption will be $2,200\ 000 \times 115 = 253,000,000$ gallons. If the latter quantity be multiplied by 30.5, the latter being taken as the average number of days in the month throughout the year, the average monthly draft of water for the two boroughs in question will be 7,716,500,000 gallons. The subtraction of the latter quantity from the monthly results in the preceding table will exhibit a deficiency which must be met by storage or a surplus available for storage. Table XIII exhibits the twelve monthly differences of that character.

It is seen from this table that the total monthly deficiencies aggregate 27,795,700,000 gallons and that there are only two months in which the run-off exceeds the consumption, the surplus for those two months being only 3,336,000,000 gallons.

TABLE XIII.

7,077,700,000 − 7,716,500,000 =	−	638,800,000	
6,989,900,000 − " =	−	726,600,000	
7,879,000,000 − " =			+ 162,500,000
6,816,000,000 − " =	−	900,500,000	
982,000,000 − " =	−	6,734,500,000	
1,449,800,000 − " =	−	6,266,700,000	
10,890,000,000 − " =			+ 3,173,500,000
7,373,100,000 − " =	−	343,400,000	
4,032,900,000 − " =	−	3,683,600,000	
5,178,500,000 − " =	−	2,538,000,000	
4,343,100,000 − " =	−	3,373,400,000	
5,126,300,000 − " =	−	2,590,200,000	
	−	27,795,700,000	+ 3,336,000,000
	+	3,336,000,000	
	−	24,459,700,000	

The total deficiency for the year is therefore 24,459,700,000 gallons. Dividing the latter quantity by the average daily draft of 253,000,000 gallons, there will result a period of 97 days, or more than one quarter of a year, during which the minimum annual rainfall would fail to supply any water to the city at all. These results show that in case of a low rainfall year, like that of 1880, the precipitation upon the Croton watershed would supply sufficient water for the boroughs of Bronx and Manhattan at the present rate of consumption for three fourths of the year only. A distressingly serious water famine would result unless the year were begun by sufficient available storage in the reservoirs of the basin at least equal to 24,459,700,000 gallons. Should such a low rainfall year or one nearly approaching it be one of a two- or three-year low rainfall cycle, such a reserve storage would be impossible and the resulting conditions would be most serious for the city. If an average year, for which the total rainfall would be about 48 inches preceded such a year of low rainfall, the conditions would be less serious. The figures would stand as follows:

Total run-off =

$$17{,}377{,}536 \times 360.4 \times 22.93 - 17{,}377{,}536 \times 16.1 \times 39.2$$
$$= 132{,}640{,}000{,}000 \text{ gallons.}$$

Total annual consumption..........	= 92,345,000,000	"
Available for storage................	= 40,295,000,000	"
Deficiency.........................	= 24,459,700,000	"
Surplus...........................	= 15,835,300,000	"

The average year would, therefore, yield enough run-off water if stored to more than make up the deficiency of the least rainfall year by nearly 16,000,000,000 gallons. In order to secure the desired volume it would therefore be necessary to have storage capacity at least equal to 24,459,700,000 gallons; indeed, in order to meet all the exigencies of a public water-supply it would be necessary to have far more than that amount. As a matter of fact there are in the Croton watershed seven artificial reservoirs with a total storage capacity of nearly 41,000,000,000 gallons, besides a number of small ponds in addition to the new Croton Lake which with water surface at the masonry crest of the dam has a total additional storage capacity of 23,700,000,000 gallons. The storage capacity of the new Croton Lake may be increased by the use of flash-boards 4 feet high placed along its crest, so that with its water surface at grade 200 its total capacity will be increased to 26,500,000,000 gallons. After the new Croton reservoir is in use the total storage capacity of all the reservoirs and ponds in the Croton watershed will be raised to 70,245,000,000 gallons, which can be further augmented by the Jerome Park reservoir when completed by an amount equal to 1,900,000,000 gallons. This is equivalent, at the present rate of consumption, to a storage supply for 285 days for the boroughs of Manhattan and the Bronx.

175. No Exact Rule for Storage Capacity.—This question of the amount of storage capacity to be provided in connection with public water-supplies is one which cannot be reduced to an exact rule. Obviously if the continuous flow afforded from any source is always greater per day than any draft that can ever be made upon it, no storage-reservoirs at all would be needed, although they might be necessary for the purpose of sedimentation. On the other hand, as in the case of New York City, if the demand upon the supply has reached its capacity or exceeded it for low rainfall years, it may be necessary to provide storage capacity sufficient to collect all the run-off of the watershed. The civil engineer must from his experience and from the data before him determine what capacity between those limits is to be secured. When the question of volume or capacity of storage is settled the mode of distribution of that volume or capacity in reservoirs is to be determined, and that affects to some extent the potability of the water. If there is a large area of shallow storage, the vegetable matter of the soil may affect the water in a number of ways. Again, it is advisable in this connection to consider certain reservoir effects as to color and contained organic matter in general.

176. The Color of Water.—The potability* of water collected from any watershed is materially affected by its color. Although iron may produce a brownish tinge, by far the greater amount of color is produced by dissolved vegetable matter. Repeated examinations of colored water have shown that discoloration is in many cases at least a measure of the vegetable matter contained in it. While this may not indicate that the water is materially unwholesome, it shows conclusively the existence of conditions which are usually productive of minute lower forms of vegetation from which both bad taste and odors are likely to arise.

There are two periods in the year of maximum intensity of

* What is generally known as the "Michigan standard of the purity of drinking-water," as specified by the Michigan State Laboratory of Hygiene, is here given:

"1. The total residue should not exceed 500 parts per million.

"2. The inorganic residue may constitute the total residue.

"3. The smaller amount of organic residue the better the water.

"4. The amount of earthy bases should not exceed 200 parts per million.

"5. The amount of sodium chloride should not exceed 20 parts per million (i.e., 'chlorine' 12.1 parts per million).

"6. The amount of sulphates should not exceed 100 parts per million.

"7. The organic matter in 1,000,000 parts of the water should not reduce more than 8 parts of potassium permanganate (i.e., 'required oxygen' 2.2 parts per million).

"8. The amount of free ammonia should not exceed 0.05 part per million.

"9. The amount of albuminoid ammonia should not exceed 0.15 part per million.

"10. The amount of nitric acid should not exceed 3.5 parts per million (i.e., 'N as nitrate' .9 part per million).

"11. The best water contains no nitrous acid, and any water which contains this substance in quantity sufficient to be estimated should not be regarded as a safe drinking-water.

"12. The water must contain no toxicogenic germs as demonstrated by tests upon animals.

"The water must be clear and transparent, free from smell, and without either alkaline or acid taste, and not above 5 French standard of hardness."

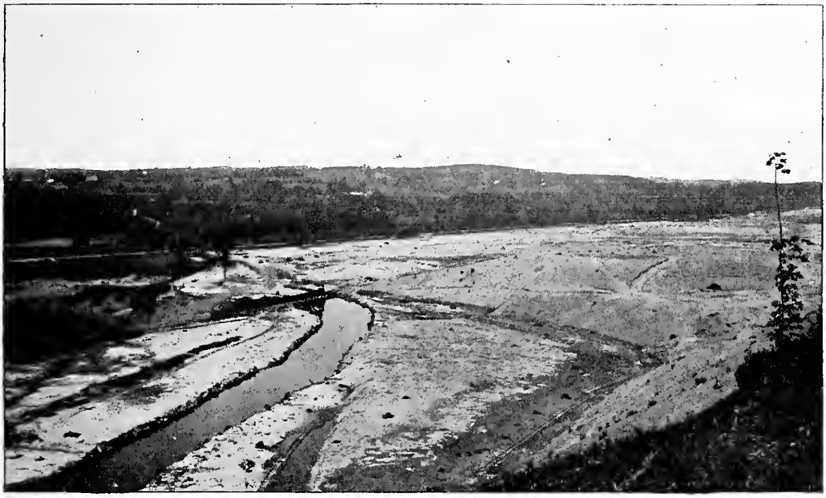
This standard is too high to be attained ordinarily in natural waters.

color, one occurring in June and the other in November. The former is due to the abundant drainage of peaty or other excessively vegetable soils from the spring rains. After June the sun bleaches the water to a material extent until the autumn, when the dying vegetation imparts more or less coloring to the water falling upon it. This last agency produces its maximum effect in the month of November.

There are various arbitrary scales employed by which colors may be measured and discolored waters compared. Among others, dilute solutions of platinum and cobalt are used, in which the relative proportions of those substances are varied so as to resemble closely the colors of the water. The amount of platinum used is a measure of the color, one unit of which corresponds to one part of the metal in 10,000 parts of water. Again, the depth at which a platinum wire 1 mm. (.039 inch) in diameter and 1 inch long can be seen in the water is also taken as a measure of the color, the amount of the latter being inversely as the depth. This method has found extended and satisfactory use in connection with the Metropolitan Water-supply of Boston, the Cochituate water having a degree of color represented by .25 to .30, while the Sudbury water has somewhat more than twice as much. The Cochituate water is practically colorless.

The origin of the color of water is chiefly the swamps which drain into the water-supply, or the vegetation remaining upon a new reservoir site when the surface soil has not been removed before the filling of the reservoir. The drainage of swamps should not, as a rule, be permitted to flow into a public water-supply, as it is naturally heavily charged with vegetable matter and is correspondingly discolored. This matter, like many others connected with the sanitation of potable public waters, has been most carefully investigated by the State Board of Health of Massachusetts in connection with the Boston water-supply. Its work has shown the strong advisability of diverting the drainage of large swamps from a public supply as carrying too much vegetable matter even when highly diluted by clear water conforming to desirable sanitary standards.

177. Stripping Reservoir Sites.—The question of stripping or cleaning reservoir sites of soil is also one which has been carefully studied by the Massachusetts State Board of Health. As a consequence large amounts of money have been expended by the city of Boston in stripping the soil from reservoir sites to the average depth in some cases of 9 inches for wooded land and 12½ inches for meadow land. This was done in the case of the Nashua River reservoir having a superficial area of 6.56 square miles at a cost of early $2,910,000, or about $700 per acre. It has been found that the beneficial effect of this stripping is fully secured if the black loam in which vegetation flourishes is removed.



Wachusetts Reservoir, showing Stripping of Soil.

This stripping of soil is indicative of the great care taken to secure a high quality of water for the city of Boston, but it is not done in the Croton watershed of the New York supply. It cannot be doubted that the quality of the Croton supply would have been sensibly enhanced by a similar treatment of its reservoir sites. Mr. F. B. Stearns, chief engineer of the Metropolitan Water-supply of Boston, states that in some cases the effects of filling reservoirs without removing the soil and vegetable matter have "continued for twenty years or more without apparent diminution." On the other hand, water discolored by vegetable matter becomes bleached to some extent at least by standing in reservoirs whose sites have been stripped of soil.

178. Average Depth of Reservoirs should be as Great as Practicable.—In the selection of reservoir locations those are preferable where the average depths will be greatest and where shallow margins are reduced to a minimum. It may sometimes be necessary to excavate marginal portions which would otherwise be shallow with a full reservoir. There should be as little water as possible of a less low-water depth than 10 or 12 feet, otherwise there may be a tendency to aquatic vegetable growth. The following table exhibits the areas, average depths, capacity, and other features of a number of prominent storage-reservoirs.

COMPARATIVE TABLE OF AREAS, DEPTHS, AND CAPACITIES OF STORAGE RESERVOIRS WITH HEIGHTS AND LENGTHS OF DAMS.

Name and Location of Reservoir.	Area, Square Miles.	Average Depth, Feet.	Maximum Height of Dam.		Length of Dam, Feet.	Capacity, Million Gallons.
			Above Ground.	Above Rock.		
Swift River, Mass	36.96	53	144	......	2,470	406,000
Nashua River, Mass	6.56	46	129	158	1,250	63,068
Nira, near Poona, India	7.25	27	100	......	3,000	41,143
Tansa, Bombay, India	5.50	33	127	131	8,770	37,500
Khadakvasla, Poona, India	5.50	32	100	107	5,080	36,737
New Croton, N. Y.	......	......	157	225	1,270	32,000
Elan and Claerwen, Birmingham, Eng., water-works (total for six reservoirs)	2.34	43	98–128	......	4,460	20,838
All Boston water-works reservoirs combined	5.82	14	14–65	......	......	15,867
Vyrnwy, Liverpool, Eng.	1.75	......	84	129	1,350	14,560
Ware River, Mass.	1.62	33	71	......	785	11,190
Sodom, N. Y.	......	......	72	89	500	9,500
Reservoir No. 5, Boston water-works	1.91	19	65	70	1,865	7,438
Titicus, N. Y.	......	......	105	115	......	7,000
Hobbs Brook, Cambridge water-works	1.00	12	23	......	......	2,500
Cochituate, Boston water-works	1.35	8	......	......	......	2,160
Reservoir No. 6, Boston water-works	0.29	25	52	......	1,500	1,500

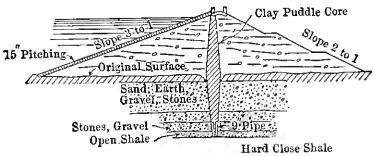
179. Overturn of Contents of Reservoirs Due to Seasonal Changes of Temperature.—It will be noticed that the average depth is less than about 20 feet in few cases only. If the water is deep, its mean temperature throughout the year will be lower than if shallow. During the warmer portion of the year the upper layers of the water are obviously of a higher temperature than the lower portions, since the latter receive much less immediate effect from the sun's rays. As the upper portions of the water are of higher temperature, they are also lighter and hence remain at or near the top. For the same reason the water at the bottom of the reservoir remains there throughout the warm season and until the cool weather of the autumn begins. The top layers of water then continue to fall in temperature until it is lower than that of the water at the bottom, when the surface-water becomes the heaviest and sinks. It displaces subsurface-water lighter than itself, the latter coming to the surface to be cooled in turn.

This operation produces a complete overturning of the entire reservoir volume as the late autumn or early winter approaches. It thus brings to the surface water which has been lying at the bottom of the reservoir all summer in contact with what vegetable matter may have been there. The depleted oxygen of the bottom water is thus replenished with a corresponding betterment of condition. It is the great sanitary effort of nature to improve the quality of stored water entrusted to its care, and it continues until the surface is cooled to a temperature perhaps lower than that of the greatest density of water.

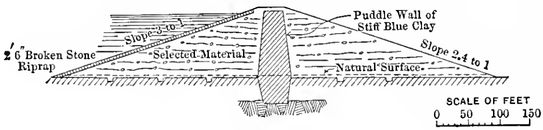
Another great turn-over in the water of a lake or reservoir covered with ice during the winter occurs in the spring. When the ice melts, the resulting water rises a little in temperature until it reaches possibly its greatest density at 39°.2 Fahr., and then sinks, displacing subsurface water. This goes on until all the ice is melted and until all water cooled by it, near the surface, below 39°.2 Fahr. has been raised to that temperature. The period of summer stagnation then follows.

180. The Construction of Reservoirs.—The natural topography and sometimes the geology of the locality determines the location of the reservoir. The first requirement obviously is tightness. If for any reason whatever, such as leaky banks or bottom, porous subsurface material, or for any other defect, the water cannot be retained in the reservoir, it is useless. Some very perplexing questions in this connection have arisen. Indeed reservoirs have been completed only to be found incapable of holding their contents. Such results are evidently not creditable to the engineers who are responsible for them, and they should be avoided.

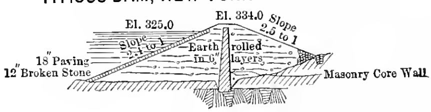
YARROW RESERVOIR, LIVERPOOL WATER SUPPLY



SAN LEANDRO DAM, SAN FRANCISCO WATER WORKS



TITICUS DAM, NEW YORK WATER SUPPLY



In order that the bottom of the reservoir may be water-tight it must be so well supported by firm underlying material that it will not be injured by the weight of water above it, which in artificial reservoirs may reach 30 to 100 feet or more in depth. The subsurface material at the site of any proposed structure of this character must, therefore, be carefully examined so as to avoid all porous material, crevasses in rocks, or other open places where water might escape. Objectionable material may frequently be removed and replaced with that which is more suitable, and rock crevices and other open places may sometimes be filled with concrete and made satisfactory. Whatever may be the conditions existing, the finished bottom of the reservoir should be placed only on well-compacted, firm, unyielding material.

The character of the reservoir bottom will depend somewhat upon the cost of suitable material of which to construct it. If a bottom of natural earth cannot be used, a pavement of stone, brick, or concrete may be employed from 8 inches to a foot or a foot and a half in thickness. The reservoir banks must be placed upon carefully prepared foundations, sometimes with masonry core walls. They are frequently composed of clayey and gravelly material mixed in proper proportions and called puddle, although that term is more generally applied to a mixture of clay and gravel designed to form a truly impervious wall in the centre of the reservoir embankment. Some engineers require the core-wall, as it is called, to be constructed of masonry, with the earth or gravelly material carried up each side of this wall in layers 6 to 9 inches thick, well moistened and each layer thoroughly rolled with a grooved roller, or treated in some equivalent manner in order that the whole mass may not be in strata but essentially continuous and as nearly impervious as possible. The masonry core-wall should be founded on bed-rock or its equivalent. Its thickness will depend upon the height of the embankment. If the latter is not more than 20 or 25 feet high, the core-wall need not be more than 4 to 6 feet thick, but if the embankment reaches a height of 75 feet or even 100 feet, it must be made 15 to 20 feet thick, or possibly more, at the base. Its top should be not less than 4 or 6 feet thick, imbedded in the earth and carried well above the highest surface of water in the reservoir.

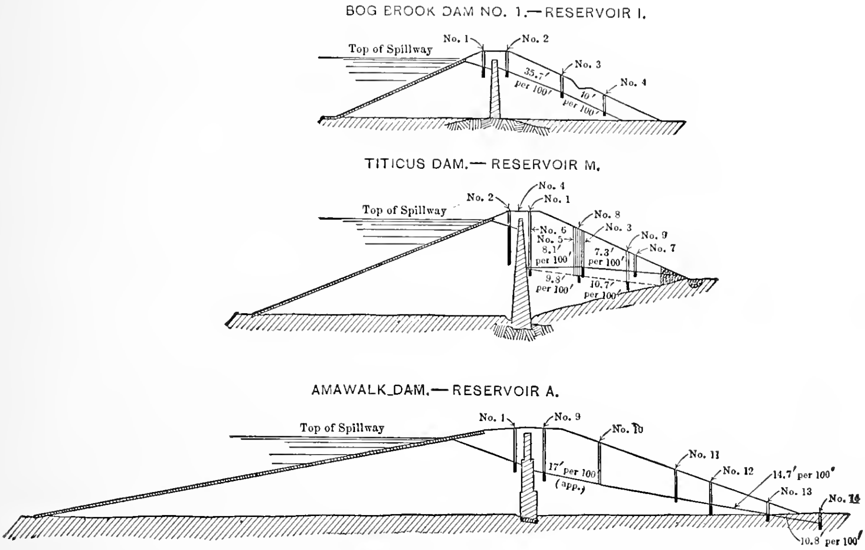
The thickness of the clay puddle-wall employed as the central core of the reservoir embankment is usually made much thicker than that of masonry. As a rough rule it may be made twice as thick as the masonry core at the deepest point and not less than about 6 feet at the top. The thickness of the puddle core is sometimes varied to meet the requirements of the natural material in which it is embedded at different depths.

Frequently, when embankments are under about 20 feet high, the core-walls may be omitted, excavation having been made at the base of the embankment down to rock or other impervious material, and if the entire bank is carried up with well-selected and puddled material.

The interior slopes of reservoir embankments are usually covered with roughly dressed stone pavement 12 to 18 inches thick, laid upon a broken-stone foundation 8 to 12 inches thick, for a protection against the wash of waves, the pavement in any case being placed upon the bank slope after having been thoroughly and firmly compacted. The sloping and bottom pavements, of whatever material they may be composed, should be made continuous with each other so as to offer no escape for the water. In some cases where it has been found difficult to make the interior surfaces of reservoirs water-tight, asphalt or other similar water-tight layers have been used with excellent results.

The care necessary to be exercised in the construction of storage or other reservoirs when earth dams or embankments are used can better be appreciated when it is realized that almost all such banks, even when properly provided with masonry or clay-puddle core-walls, are saturated with water, even on the down-stream side, at least throughout their lower portions. A board of engineers appointed by the commissioners of the Croton Aqueduct in the summer of 1901 made a large number of examinations in the earth embankments in the Croton watershed, and found that with scarcely an exception those embankments were saturated throughout the lower portions of their masses, although in every case a masonry core-wall had been built. The results of those investigations showed that the water had percolated through the earth portion of the embankments and even through the core-walls, which had been carried down to bed-rock. This induced saturation, more or less, of the material on the down-stream slopes of the embankments. When material is thus filled with water, unless it is suitably selected, it is apt to become soft and unstable, so that any superincumbent weight resting upon it might produce failure. The fact that such embankments may become saturated with water fixes limits to their heights, since the surface of saturation in the interior of the bank has generally a flatter slope than that of the exterior surface. The height of the embankment therefore should be such that the exterior slope cannot cut into the saturated material at its foot, at least to any great extent. From what precedes it is evident that the height of an earth embankment will depend largely upon the slope of the exterior surface. This slope is made 1 vertical to 2, $2\frac{1}{2}$, or 3 horizontal. The more gradual slope is sometimes preferable.

It is advisable also to introduce terraces and to encourage the growth of sod so as to protect the surface from wash. The inner paved slope may be as steep as 1 vertical to $1\frac{1}{2}$ or 2 horizontal.

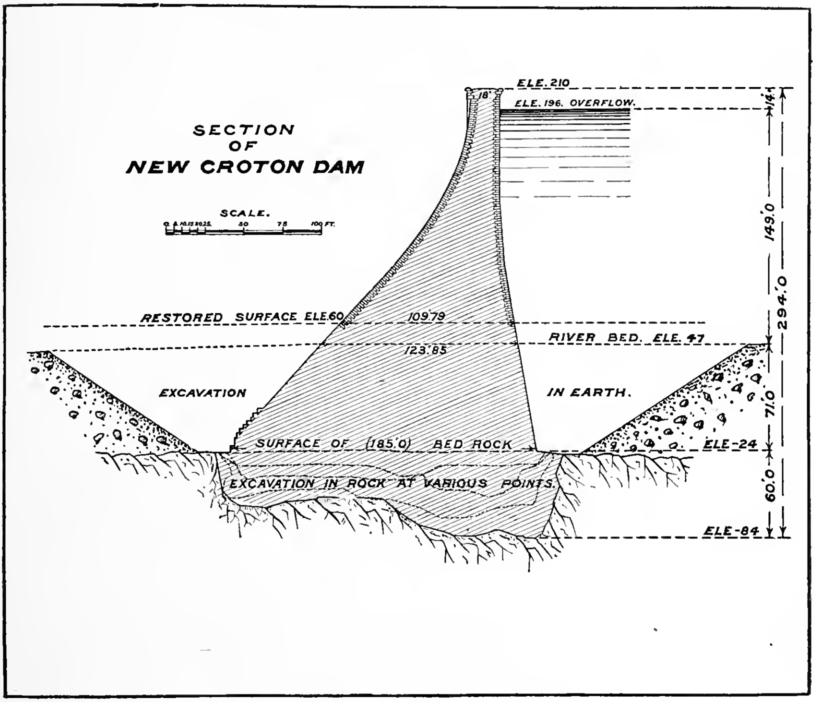


Earth Dams in Croton Watershed, showing Slopes of Saturation.

181. Gate-houses, and Pipe-lines in Embankments. — It is necessary to construct the requisite pipe-lines and conduits leading from the storage-reservoirs to the points of consumption, and sometimes such lines bring the water to the reservoir. Wherever such pipes-line or conduits either enter or leave a reservoir gates and valves must be provided so as properly to control the admission and outflow of the water. These gate-houses, as they are called, because they contain the gates or valves and such other appurtenances or details as are requisite for operation and maintenance, are usually built of substantial masonry. They are the special outward features of every reservoir construction, and their architecture should be characteristic and suitable to the functions which they perform. Where the pipes are carried through embankments it is necessary to use special precautions to prevent the water from flowing along their exterior surfaces. Many reservoirs have been constructed under defective design in this respect, and their embankments have failed. Frequently small masonry walls are built around the pipes and imbedded in the bank, so as to form stops for any initial streams of water that might find their way along the pipe. In short, every care and resource known to the civil engineer must be employed in reservoir construction to make its bottom and its banks proof against leakage and to secure permanence and stability in every feature.

182. High Masonry Dams.—The greatest depths of water impounded in reservoirs are found usually where it is necessary to construct a high dam across the course of a river, as at the new Croton dam. In such cases it is not uncommon to require a dam over 75 to 100 feet high above the original bed of the river, which is usually constructed of masonry with foundations carried down to bed-rock in order to secure suitable stability and prevent flow or leakage beneath the structure. It is necessary to secure that result not only along the foundation-bed of the dam, but around its ends, and special care is taken in those portions of the work.

The new Croton dam is the highest masonry structure of its class yet built. The crest of its masonry overflow-weir is 149 feet above the original river-bed, with the extreme top of the masonry work of the remaining portion of the dam carried 14 feet higher. A depth of earth and rock excavation of 131 feet below the river-bed was necessary in order to secure a suitable foundation on bed-rock. The total maximum height, therefore, of the new Croton dam, from the lowest foundation-point to the extreme top, is 294 feet, and the depth of water at the up-stream face of the dam will be 136 feet when the overflow is just beginning, or 140 feet if 4 feet additional head be secured by the use of flash-boards. In the prosecution of this class of work it is necessary not only to reach bed-rock, but to remove all soft portions of it down to sound hard material, to clean out all crevices and fissures of sensible size, refilling them with hydraulic cement mortar or concrete, and to shape the exposed rock surfaces so as to make them at least approximately normal to the resultant loads upon them, to secure a complete and as nearly as possible water-tight bond with the superimposed masonry. If any streams or other small watercourses should be encountered, they must either be stopped or led off where they will not affect the work, or, as is sometimes done, the water issuing from them may be carried safely through the masonry



Cross-section of New Croton Dam.

mass in small pipes. The object is to keep as much water out of the foundation-bed as possible, so as to eliminate upward pressure underneath the dam caused by the head of water in the subsequently full reservoir. It is a question how much dependence can be placed upon the exclusion of water from the foundation-bed. In the best class of work undoubtedly the bond can be good enough to exclude more or less water, but it is probably only safe and prudent so to design the dam as to be stable even though water be not fully excluded.

The stability of the masonry dam must be secured both for the reservoir full and empty. With a full reservoir the horizontal pressure of water on the up-stream face tends to overturn

dammed will permit a curved structure to be built, the curvature being so placed as to be convex up-stream or against the water pressure. In such a case the dam really becomes a hori-

Foundation Masonry of New Croton Dam.

zontal arch and, if the curvature is sufficiently sharp, it may be designed as an arch horizontally pressed. The cross-section then has much less thickness (and hence less area) than if designed on a straight line so as to produce a gravity section. A number of such dams have been built, and one very remarkable example of its kind is the Bear Valley dam in California; it was built as a part of the irrigation system.

CHAPTER XVII.

183. **Gravity Supplies.**—When investigation has shown that a sufficient quantity of water may be obtained for a required public supply from any of the sources to which reference has been made, and that a sufficient storage capacity may be provided to meet the exigencies of low rainfall years, it will be evident if the water can be delivered to the points of consumption by gravity, or whether pumping must be employed, or recourse be made to both agencies.

If the elevation of the source of supply is sufficiently great to permit the water to flow by gravity either to storage-reservoirs or to service-reservoirs and thence to the points of consumption, a proper pipe-line or conduit must be designed to afford a suitable channel. If the topography permits, a conduit may be laid which does not run full, but which has sufficient grade or slope to induce the water to flow in it as if it were an open channel. This is the character of such great closed masonry channels as the new and old Croton aqueducts of the New York water-supply and the Sudbury and Wachusetts aqueducts of the Boston supply. These conduits are of brick masonry backed with concrete carried sometimes on embankments and sometimes through rock tunnels. When they act like open channels a very small slope is employed, 0.7 of a foot per mile being a ruling gradient for the new Croton aqueduct, and 1 foot per mile for the Sudbury. Where these conduits cross depressions and follow approximately the surface, or where they pass under rivers, their construction must be changed so that they will not only run full, but under greater or less pressure, as the case may be.

184. **Masonry Conduits.**—In general the conduits employed to bring water from the watersheds to reservoirs at or near places

of consumption may be divided into two classes, masonry and metal, although timber-stave pipes of large diameter are much used in the western portion of the country. The masonry conduits obviously cannot be permitted to run full, meaning under pressure, for the reason that masonry is not adapted to resist the tension which would be created under the head or pressure of water induced in the full pipe. They must rather be so employed as to permit the water to flow with its upper surface exposed to the atmosphere, although masonry conduits are always closed at the top. In other words, they must be permitted to run partially full, the natural grade or slope of the water surface in them inducing the necessary velocity of flow or current. Evidently the velocity in such masonry conduits is comparatively small, seldom exceeding about 3 feet per second. The new and old Croton aqueducts, the Sudbury and Wachusetts aqueducts of the Metropolitan Water-supply of Boston, are excellent types of such conveyors of water. They are sometimes of circular shape, but more frequently of the horseshoe outline for the sides and top, with an inverted arch at the bottom for the purpose of some concentration of flow when a small amount of water is being discharged and for structural reasons.

The interiors of these conduits are either constructed of brick or they may be of concrete or other masonry affording smooth surfaces. In the latest construction Portland-cement concrete or that concrete reinforced with light rods of iron or steel is much used. Bricks, if employed, should be of good quality and laid accurately to the outline desired with about $\frac{1}{4}$-inch joints, so as to offer as smooth a surface as possible for the water to flow over. In special cases the interiors of these conduits may be finished with a smooth coating of Portland-cement mortar. If conduits are supported on embankments, great care must be exercised in constructing their foundation supports, since any sensible settlement would be likely to form cracks through which much water might easily escape. When carried through tunnels they are frequently made circular in outline. They must occasionally be cleaned, especially in view of the fact that low orders of vegetable growths appear on their sides and so obstruct the free flow of water.

185. **Metal Conduits.**—Metal conduits have been much used within the past fifteen or twenty years. Among the most prominent of these are the Hemlock Lake aqueduct of the Rochester Water-works, and that of the East Jersey Water Company through which the water-supply of the city of Newark, N. J., flows. When these metal conduits or pipes equal 24 to 30 or more inches in diameter they are usually made of steel plates, the latter being of such thickness as is required to resist the pressure acting within them. The riveted sections of these pipes may be of cylindrical shape, each alternate section being sufficiently small in diameter just to enter the other alternate sections of little larger diameter, the interior diameter of the larger sections obviously being equal to the interior diameter of the smaller sections plus twice the thickness of the plate. Each section may also be slightly conical in shape, the larger ends having a diameter just large enough to pass sufficiently over the smaller end of the next section to form a joint. Large cast-iron pipes are also sometimes used to form these metal conduits up to an interior diameter of 48 inches. The selection of the type of conduit within the limits of diameter adapted to both metals is usually made a matter of economy. The interior of the cast-iron pipe is smoother than that of the riveted steel, although this is not a serious matter in deciding upon the type of pipe to be used.

Steel-plate conduits have been manufactured and used up to a diameter of 9 feet. In this case the pipe was used in connection with water-power purposes and with a length of 153 feet only, the plates being $\frac{1}{2}$ inch thick. The steel-plate conduits of the East Jersey Water Company's pipes are as follows:

Diameter.	Thickness.	Length
48 inches	$\frac{1}{4}$ inch	21 miles.
48 "	$\frac{5}{16}$ "	
48 "	$\frac{3}{8}$ "	
36 "	$\frac{1}{4}$ "	5 "

The diameters and lengths of the metal pipes or conduits of the Hemlock Lake conduit of the Rochester Water-works are as follows:

36-inch wrought-iron pipe..	9.60 miles.
24 " " " ..	2.96 "
24 " cast-iron pipe......	15.82 "
Total................	28.39 "

All metal conduits or pipes are carefully coated with a suitable asphalt or tar preparation or varnish applied hot and sometimes baked before being put in place. This is for the purpose of protecting the metal against corrosion. Cast-iron pipes have been used longer and much more extensively than wrought iron or steel, but an experience extending over thirty to forty years has shown that the latter class of pipes possesses satisfactory durability and may be used to advantage whenever economical considerations may be served.

186. General Formula for Discharge of Conduits — Chezy's Formula.—It is imperative in designing aqueducts of either masonry or metal to determine their discharging capacity, which in general will depend largely upon the slope of channel or head of water and the resistance offered by the bed or interior of the pipe to the flow of water. The resistance of liquid friction is so much more than all others in this class of water-conveyors that it is usually the only one considered. There is a certain formula much used by civil engineers for this purpose; it is known as Chezy's formula, for the reason that it was first established by the French engineer Antoine Chezy about the year 1775, although it is an open question whether the beginnings of the formula were not made twenty or more years prior to that date. Its demonstration involves the general consideration of the resistance which a liquid meets in flowing over any surface, such as that of the interior of a pipe or conduit, or the bed and banks of a stream.

The force of liquid friction is found to be proportional to the heaviness of the liquid (i.e., to the weight of a cubic unit, such as a cubic foot), to the area of wetted surface over which the liquid flows, and nearly to the square of the velocity with which the liquid moves. Hence if l' is the length of channel, p the wetted portion of the perimeter of the cross-section, w the weight of a cubic unit of the liquid, and v the velocity, the total force of liquid friction for the length l' of channel will be $F=\zeta wpl'v^2$, ζ being the coefficient of liquid friction. The path of the force

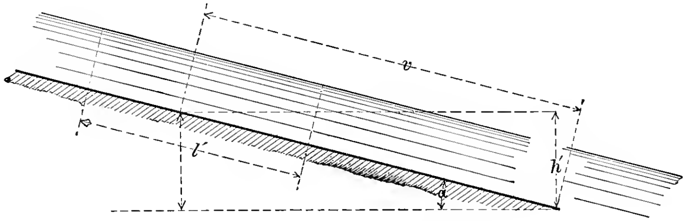


FIG. 2.

F for a unit of time is v, and the work W which it performs in that unit of time is equal to the weight wal' falling through the height h', a being the area of the cross-section of the stream.

Hence $$W=\zeta wpl'v^2 . v = wal'h'. \quad . \quad . \quad . \quad . \quad . \quad . \quad (7)$$

$$\zeta v^2 v=\frac{a}{p}h', \quad v=\sqrt{\frac{1}{\zeta}\frac{a}{p}\frac{h'}{v}}=c\sqrt{rs}. \quad . \quad . \quad . \quad . \quad . \quad (8)$$

In this equation

$$c=\sqrt{\frac{1}{\zeta}}; \quad r=\frac{a}{p}=\text{hydraulic mean radius;}$$

$$s=\frac{h'}{v}=\text{sine of inclination of stream's bed.}$$

As the motion of the water is assumed to be uniform, the head lost by friction for the total length of channel l is the total fall h, and by equation (8), since $\frac{h'}{v}=s=\frac{h}{l}$,

$$h=\frac{v^2}{c^2}\frac{l}{\frac{a}{p}}. \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (9)$$

If, as in the case of the ordinary cast-iron water-pipes of a public supply system, the cross-section a is circular,

$$\frac{a}{p}=\frac{\pi\frac{d^2}{4}}{\pi d}=\frac{d}{4},$$

and

$$h=\frac{4.2g}{c^2}\frac{l}{d}\frac{v^2}{2g}=f\frac{l}{d}\frac{v^2}{2g}, \quad . \quad . \quad . \quad . \quad . \quad . \quad (10)$$

in which $f=8g\div c^2$.

The quantity f is sometimes called the "friction factor." For smooth, new pipes from 4 feet down to 3 inches in diameter its value may be taken from .015 to .03. An approximate mean value may be taken at .02.

The last member of equation (8) is Chezy's formula, and it is one of the most used expressions in hydraulic engineering. Some values for the coefficient c will presently be given. The quantity r found by dividing the area of the cross-section of the stream by the wetted portion of its perimeter is called the "hydraulic mean radius," or simply the "mean radius." The other quantity, s, appearing in the formula is, as shown by the figure, the sine of the inclination of the bed of the stream.

In order to determine the discharge of any pipe, conduit, or open channel carrying a known depth of water, it is only necessary to compute r and s from known data and select such a value of the coefficient c as may best fit the circumstances of the particular case in question. The substitution of those quantities in Chezy's formula, i.e., equation (8), will give the mean velocity v of the water which, when multiplied by the area of cross-section of the stream, will give the discharge of the latter per second of time. It is customary to compute r in feet. The coefficient c is always determined so as to give velocity in feet per second of time. Hence if the area of the cross-section of the stream, a, is taken in square feet, as is ordinarily the case, the discharge av will be in cubic feet per second.

187. Kutter's Formula.—The coefficient c in Chezy's formula is not a constant quantity, but it varies with the mean radus r, with the sine of inclination s, and with the character of the bottom and sides of the open channel, i.e., with the roughness of the interior surface of the closed pipe. Many efforts have been made and much labor expended in order to find an expression for this coefficient which may accurately fit various streams and pipes. These efforts have met with only a moderate degree of success.

Progress View of Construction of New Croton Dam.

The form of expression for c which is used most among engineers is that known as Kutter's formula, as it was established by the Swiss engineer W. R. Kutter. This formula is as follows:

$$c=\frac{\sqrt{r}}{n}\left(\frac{\frac{1.811}{n}+41.65+\frac{.00281}{s}}{\frac{\sqrt{r}}{n}+41.65+\frac{.00281}{s}}\right).$$

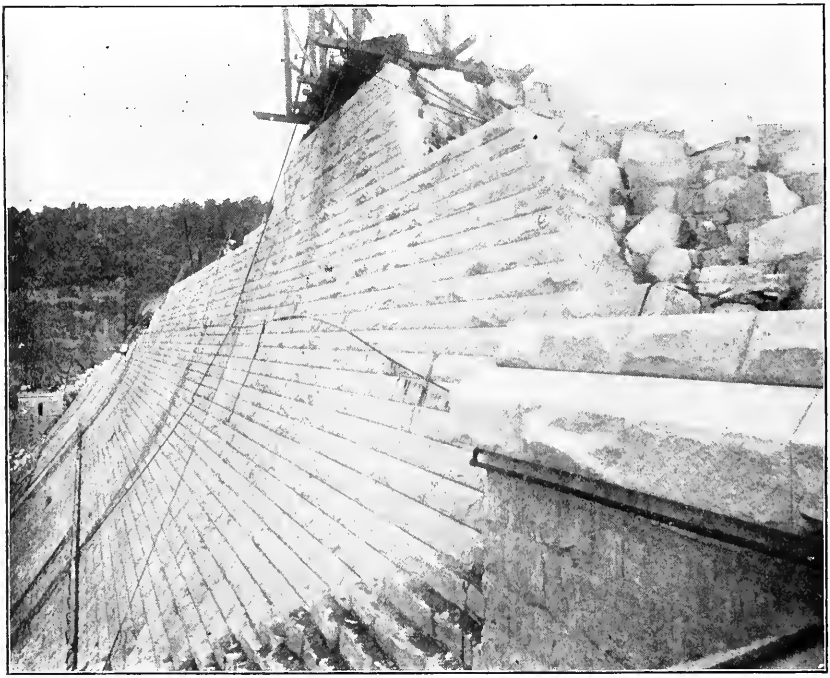
The quantity n in this formula is called the "coefficient of roughness," since its value depends upon the character of the surface over which the water flows. It has the following set of values for the surfaces indicated:

$n = 0.009$ for well-planed timber;
$n = 0.010$ for neat cement;
$n = 0.011$ for cement with one third sand;
$n = 0.012$ for unplaned timber;
$n = 0.013$ for ashlar and brickwork;
$n = 0.015$ for unclean surfaces in sewers and conduits;
$n = 0.017$ for rubble masonry;
$n = 0.020$ for canals in very firm gravel;
$n = 0.025$ for canals and rivers free from stones and weeds;
$n = 0.030$ for canals and rivers with some stones and weeds;
$n = 0.035$ for canals and rivers in bad order.

188. Hydraulic Gradient. — Before illustrating the use of Chezy's formula in connection with masonry and metal conduits, of which mention has already been made, it is best to define another quantity constantly used in connection with closed iron or steel pipes. This quantity is called the "hydraulic gradient." If a closed iron or steel pipe is running full of water and under pressure and if small vertical tubes be inserted in the top of the pipe with their lower ends bent so as to be at right angles to its axis, the water will rise to heights in the tubes depending upon the pressures of water in the pipe or conduit at the points of insertion. Such tubes with the water columns in them are called piezometers. They are constantly used in connection with water-pipes in order to show the pressures at the points where they are inserted. A number of such pipes being inserted along an iron pipe or conduit, a line may be imagined to be drawn through the upper surfaces of the columns of water, and that line is called the "hydraulic gradient." It represents the upper surface of water in an open channel discharging with the same velocity existing in the closed pipe.

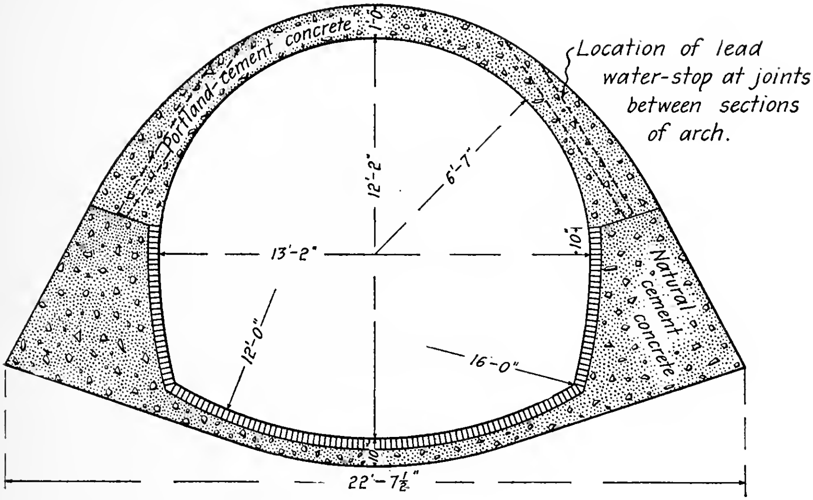
In case Chezy's formula is used to determine the velocity of discharge in a closed pipe running under pressure, the sine of inclination s must be that of the hydraulic gradient and not the sine of inclination of the axis of the closed pipe. In the determination of this quantity s by the use of piezometer tubes, if a straight pipe remains of constant section between any two points, it is only necessary to insert the tubes at those points and observe the difference in levels of the water columns in them. That difference of levels or elevations will represent the height which is to be divided by the length of pipe or conduit between the same two points in order to determine the sine s.

The hydraulic gradient plays a very important part in the construction of a long pipe-line or conduit. If any part of the pipe should rise above the hydraulic gradient, the discharge would no longer be full below that point. It is necessary, therefore, always to lay the pipe or the closed conduit so that all parts of

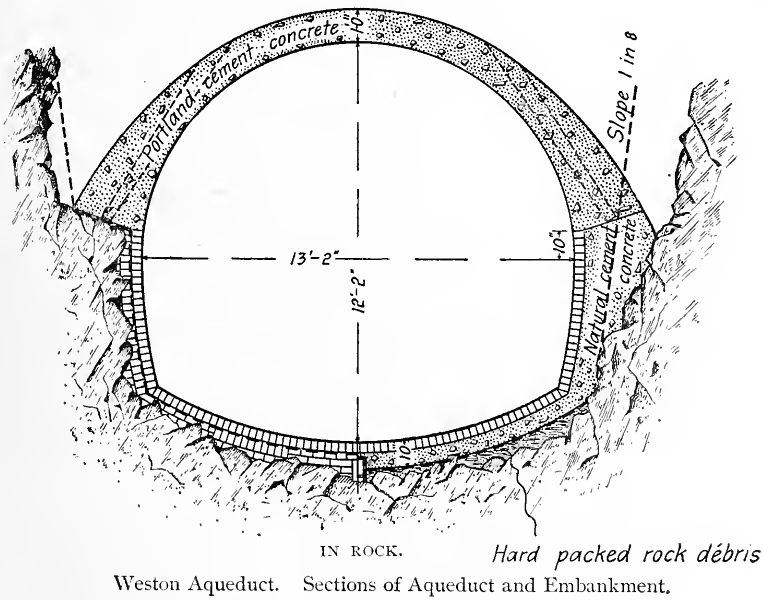


Progress View of Construction of New Croton Dam.

it shall be below the hydraulic gradient. Caution is obviously necessary to lay a pipe carrying water deep enough below the surface of the ground in cold climates to protect the water against freezing. At the same time if the pipe-line is a long one it must follow the surface of the ground approximately in order to save



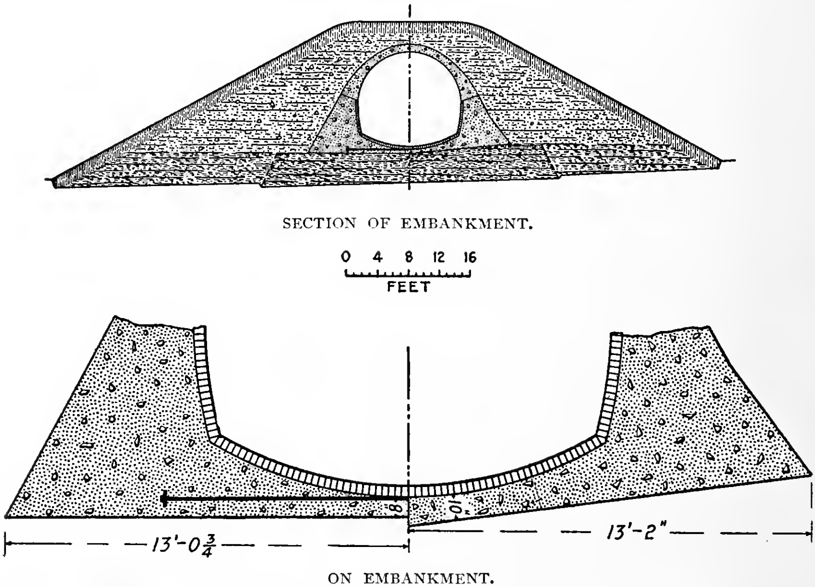
IN LOOSE EARTH.



IN ROCK. Hard packed rock débris

Weston Aqueduct. Sections of Aqueduct and Embankment.

expensive cutting. There will, therefore, generally be summits in pipe-lines, and inasmuch as all potable water carries some air dissolved in it, that air is liable to accumulate at the high points



Weston Aqueduct. Sections of Aqueduct and Embankment. Gradient, 1 in 5000.

or summits. If that accumulation goes on long enough, it will seriously trench upon the carrying capacity of the pipe and decrease its flow. It is therefore necessary to provide at summits what are called blow-off cocks to let the air escape. At the low points of the pipe-line, on the contrary, the solid matter, such as sand and dirt, carried by the water is liable to accumulate, and it is customary to arrange blow-offs also at such points, so as to enable some of the water to escape and carry with it the sand and dirt.

189. Flow of Water in Large Masonry Conduits.—In order to apply Chezy's formula first to the flow of the masonry aqueducts of the New York and Boston water-supplies, it is necessary to have the outlines of those conduits so that the wetted perimeter and hence the mean radius may be determined for any depth of water in them.

The figure shows the desired cross-sections drawn carefully to scale. Table XIV has been computed and arranged from data taken from various official sources so as to show the depth,

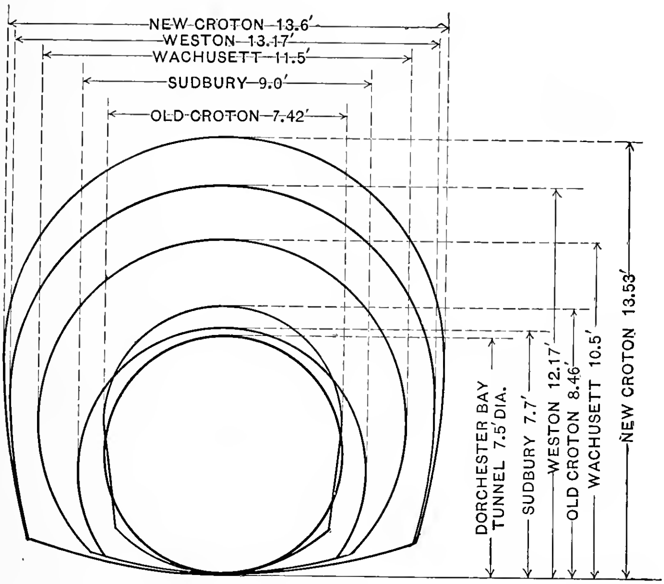


FIG. 3.

mean velocity, discharge per second and per twenty-four hours, and the coefficient used in Chezy's formula, together with the coefficient of roughness n in Kutter's formula for the conduits shown in the figure.

This table exhibits in a concise and clear manner the use of Chezy's formula in this class of hydraulic work.

190. Flow of Water through Large Closed Pipes. — The masonry conduits to which consideration has been given in the preceding paragraphs carry water precisely as in an open canal, but the closed conduits or pipes of steel plates and cast iron, like the Hemlock Lake conduit at Rochester and the East Jersey conduit of the Newark Water-works, are of an entirely different

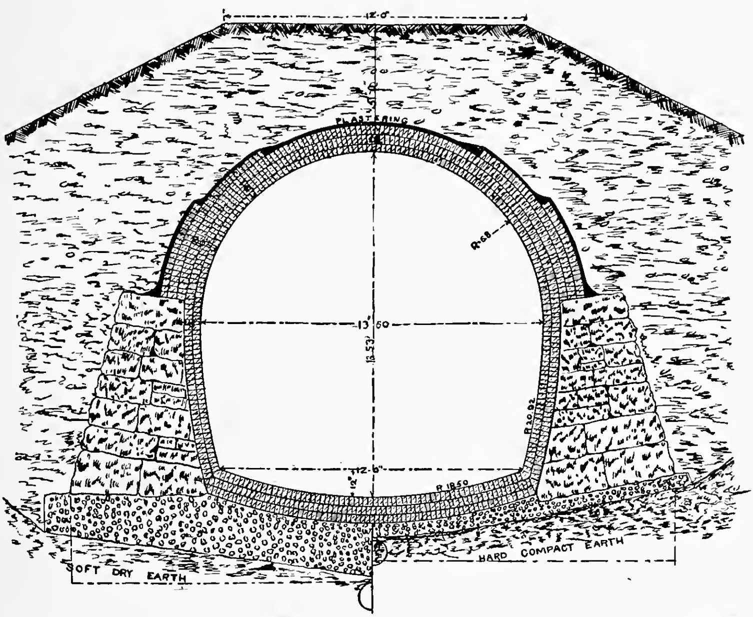
TABLE XIV.

Aqueducts.	Depth, in Feet.	Hydraulic Radius r, Feet.	Grade s.	Coefficient c.	Mean Velocity, Feet.	Discharge. Cubic Feet per Second.	Discharge. Gallons per 24 Hours.	n in Kutter's Formula.
* New Croton (1899).......	8.42	3.974	.0001326	153.3	3.52	371.6	240,200,000	
* " " (after two years' use)	......	2.338	"	131.3	2.312	......	....	.0133
† " "	......	1	"	119.3	1.374			
† " "	......	1.5	"	126.3	1.781			
† " "	......	2	"	129.8	2.114			
† " "	......	2.5	"	132	2.404			
† " "	......	3	"	133.4	2.661			
† " "	......	3.5	"	134	2.887			
† " "	......	4	"	134.4	3.095			
Old Croton (1899) clean.......	6	2.338	....	133.4	2.958	122.8	79,400,000	.0133
" " ordinary condition; not clean	6	2.338	....	123.2	......	......	73,300,000	
" " not clean.......	7.33	2.368	....	118.2	......	......	85,600,000	
Dorchester Bay tunnel.......	......	1.875	....	119				
" "	......	2.338	....	125.0	......	......	....	.014
Wachusetts, new; probably clean (approx.)......	......	......	....	144.9				
Sudbury, clean............	......	.5	.000189	116.9	1.14			
" "	......	1.0	"	127.0	1.74			
" "	......	1.5	"	133.3	2.24			
" "	......	2.0	"	137.8	2.68			
" "	......	2.5	"	140.4	3.04			

* From report by J. R. Freeman to B. S. Coler, 1899.
† From report of New York Aqueduct Commission.

type, as they carry water under pressure. Hence the slope or sine of inclination s belongs to the hydraulic gradient rather than to the grade of the pipe itself. Where the pipe-line is a long one its average grade frequently does not differ much from the hydraulic gradient, but the latter quantity must always be used. As in the case of the masonry conduits, the coefficient c in Chezy's formula will vary considerably with the degree of roughness of the interior surface of the pipe, with the slope s, and with the mean radius r. An important distinction must be made between riveted steel pipes and those of cast iron, for the reason that the rivet-heads on the inside of the former exert an appreciable influence upon the coefficient c. The rivet-heads add to the roughness or unevenness of the interior of the pipe. Table XV gives the elements of the flow or discharge in the two pipe-lines which have been taken as types, as determined by actual measurements; it also exhibits similar elements for timber-stave pipes, to which reference will be made later.

CROTON AQUEDUCT
IN EARTH.



As would be expected, the velocity of flow in these pipes may be and generally is considerably higher than the velocity of movement in masonry channels. Both Tables XV and XVI give considerable range of coefficients computed and arranged from authoritative sources, and the coefficients c for Chezy's formula represent the best hydraulic practice in connection with such works at the present time. In using the formula for any special case, great care must be taken to select a value for c which has been established for conditions as closely as possible to those in question. This is essential in order that the results of estimated discharges may not be disappointing, as they sometimes have been where that condition so necessary to accuracy has not been fulfilled.

TABLE XV.

VALUES OF COEFFICIENT c.

Pipe-line.	Diameter.	Hydraulic Radius r.	Hydraulic Gradient.	Mean Velocity.
Hemlock Lake	36″ wrought iron	9″	.000411	1.532
" "	24″ wr't and cast	6″	.00239	3.448
Rush Lake to Mt. Hope	24″ cast iron	6″	.00255	3.448
Sudbury aqueduct	48″ " "	12″	....	3.738
" "	48″ " "	12″	....	4.965
" "	48″ " "	12″	....	6.195
" "	48″ " "	12″	....	3.738
" "	48″ " "	12″	....	4.965
" "	48″ " "	12″	....	6.195
East Jersey Water Co.	48″ steel riveted pipe	12″	.002	4.62
Timber-stave pipe, Ogden, Utah	72″.5	......	....	.5
" " " " "	72″.5	......	....	1.0
" " " " "	72″.5	......	....	1.5
" " " " "	72″.5	......	....	2.
" " " " "	72″.5	......	....	2.5
" " " " "	72″.5	......	....	3.0
" " " " "	72″.5	......	....	3.5
" " " " "	72″.5	......	....	4.0

Pipe-line.	Coefficient c.	Discharge. Cubic Feet per Second.	Discharge. Gallons per 24 Hours.	Remarks.
Hemlock Lake	87.3	10.83124	7,000,000	
" "	99.7	10.83124	7,000,000	1892.
Rush Lake to Mt. Hope	96.5	10.83124	7,000,000	
Sudbury aqueduct	140.14	....	....	Pipe new 1880.
" "	142.11	....	....	
" "	144.09	....	....	
" "	139.94*	....	....	After cleaning, 1894–95.
" "	141.74*	....	....	
" "	143.16*	....	....	Before cleaning, $c=108$
East Jersey Water Co.	103.3	58.02	37,500,000	1891.
Timber-stave pipe, Ogden, Utah	72	....	....	1897.
" " "	96			
" " "	109			
" " "	115			
" " "	119			
" " "	122			
" " "	124			
" " "	126			

* These values correspond to the formula $c = 131.88v^{0.045}$.

TABLE XVI.

VALUES OF c IN $v = c\sqrt{rs}$.

	No...	1	2	3	4	5	6	7	8	9	10	11	12	13
	Age...	New	4 Years	New	New	New	New	New	New	4 Years	4 Years	New	New	5 Years
Velocity, Feet per sec.	Diam. Inches.	36	36	38	38	42	42	42	48	48	48	48	72	103
0.5	c in $v = c\sqrt{rs}$.	....	....	....	....	....	....	....	....	....	....	....	110	126.5
1		86	....	....	....	....	96	101	101.2	78	97.2	97.1	110	116.6
1.48														
1.5		90.6	....	....	....	....	103	102.8	105.4	84.6	100.8	98.7	111	112.7
2 0		95.2					107.9	104.3	108.8	89.6	103.3	100.3	110	110.3
2.44		....	....	....	....	115.9								
2.5		99.4	....	....	....	....	111	105.5	111.2	92.4	104.9	101.6	108	108.8
3		103.3	....	....	....	....	112.6	106.4	111.8	93	105.3	102.2	108	107.7
3.23		....	....	114										
3.27		....	....	116.6										
3.32														
3.5		107	....	....	....	....	113	107.2	113.4	93.2	104.8	103.6	110	106.9
3.52														
3.9		....	....	....	109.2									
3.96														
4		110.6	....	....	....	....	112.8	107.8	113.2	94	104	104.2	111	106.2
4.5		114	....	....	....	....	111.8	108.2	112.4	94.2	103.7	104.7	....	105.6
4.93		....	106.3											
5		117.2	....	....	....	....	110.8	108.4	112	94.4	103.7	105.1		
5.5		120.4	....	....	....	....	110.2	108.5	111.7	94.7	103.7	105.2		
6		123.6	....	....	....	....	110	108.5	111.6	94.9	103.7	105.2		
12.6														
Kutter's n = coefficient of roughness		.014	.013	.013	.013	.013	.013	.013	.013	.016	.014	.014	.014	.015

Exp. Nos. 1–2. Clemens Herschel, 1892. East Jersey Conduit, cylindrical joints.
Nos. 3–4. E. Kuichling, 1895. New Rochester conduit, cylindrical joints.
No. 5. I. W. Smith, 1896. Portland, Oregon, water-works.
Nos. 6–7. Clemens Herschel, 1896. East Jersey conduit, taper joints.
No. 8. Clemens Herschel, 1892. East Jersey conduit, cylindrical joints.
Nos. 9–10. Clemens Herschel, 1896. East Jersey conduit, cylindrical joints.
No. 11. Clemens Herschel, 1896. East Jersey conduit, taper joints.
No. 12. Marx, Wing, Hoskins, 1897. Pioneer El. Power Co., Ogden, Utah.
No. 13. Clemens Herschel, 1887. Holyoke, Mass., testing flume.

191. Change of Hydraulic Gradient by Changing Diameter of Pipe.—It has already been seen, in the case of closed pipes or conduits, that the hydraulic gradient with slope s governs the velocity of flow, and also that all parts of the pipe-line must be kept below that gradient. It is sometimes desirable, in order to

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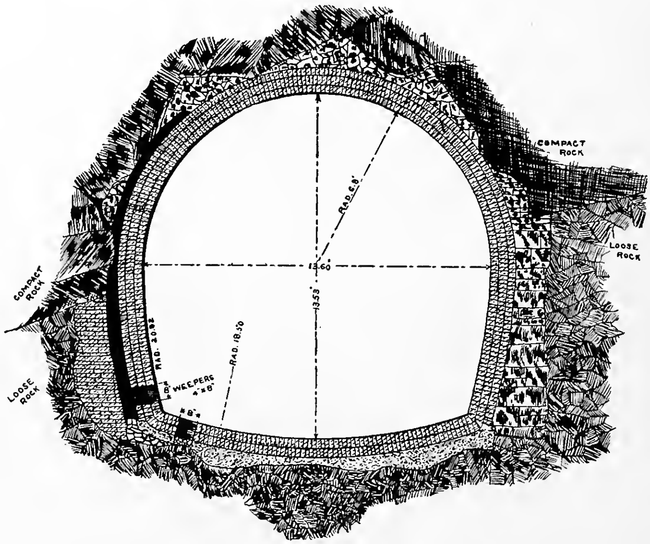
IN ROCK.



meet conditions either of topography or of flow, to raise or lower the hydraulic gradient over the whole or some portion of the pipe-line. This can easily be done to any needed extent by varying the diameter of the pipe. An increase in diameter will in general decrease the velocity of the water and increase its pressure, thus increasing correspondingly the height of the columns of water in the piezometer-tubes. As the top surface of the latter determines the hydraulic gradient, it is seen that increasing the diameter of a portion of the pipe-line will correspondingly raise the gradient over the same portion. Thus by a proper relative variation of diameters the hydraulic gradient of a given pipe-line may readily be controlled within sufficient limits to meet any ordinary requirements of this character.

192. Control of Flow by Gates at Upper End of Pipe-line.— Obviously, if the pressure in the pipe-line is diminished, less thickness of metal will be required to resist it, and a corresponding degree of economy may be reached by a decrease in the quantity of metal. In the 21 miles of 48-inch steel-plate pipe of the East Jersey Water Company there is a fall of 340 feet; if, therefore, the flow through that pipe were regulated by a gate or gates at its lower end, the lower portion of the line would be subjected to great intensity of pressure. If, however, the flow through the pipe is controlled by a gate or gates at its upper end, enough water only may be admitted to enable it to flow full with the velocity due to the hydraulic gradient. By such a procedure the pressure upon the pipe over and above that which is necessary to produce the gradient is avoided. This condition is not only judicious in the reduction of the amount of metal required, but also in reducing both the leakage and the tendency to further leakage, which is largely increased by high pressures. This feature of control of pressure in a long pipe-line with considerable fall is always worthy of most careful consideration.

193. Flow in Old and New Cast-iron Pipes—Tubercles.—The velocity of flow through cast-iron mains or conduits or through the cast-iron pipes of a distribution system of public water-supply depends largely upon the condition of the interior surface of the pipes as affected by age. All cast-iron pipes before being shipped from the foundry where they are manufactured are immersed in a hot bath of suitable coal-tar pitch composition in order to protect them from corrosion. After having been in use a few years this coating on the interior of the pipes is worn off in spots and corrosion at once begins. The iron oxide produced under these circumstances forms projections, or tubercles as they are called, of greatly exaggerated volume and out of all proportion to the actual weight of oxide of iron. When the pipes are emptied these tubercles are readily removed by scraping, but before their removal they greatly obstruct the flow of water through the pipes. Indeed this obstruction is so great that the discharging capacity of cast-iron mains must be treated in view of its depreciation from this source.

Table XVII exhibits the value of the coefficient c to be used in Chezy's formula for all cast-iron pipes having been in use for the periods shown.

TABLE XVII.

TABLE OF VALUES OF f AND c.

Authority.	Pipe-line.	Diameter, Inches.	Hydraulic Radius r, Inches.	Velocity, Feet per Second.	Coefficient c.	Coefficient f.
Darcy	New pipe	3.22	.8	0.29 10.71	78.5 100.0	.0418 .0257
Darcy	Old cast-iron pipe lined with deposit	9.63	2.41	1.00 12.42	72.5 74.0	.0489 .0468
Darcy	Pipe above cleaned......	9.63	2.41	0.91 14.75	90.0 98.0	.0316 .0269
Brush	Cast-iron pipe tar-coated and in service 5 years.	20	5	2.00 3.00	114.0 110.0	.0197 .0214
Darrach	Cast-iron pipe in service 11 years	20	5	2.71 5.11	67.5 83.0	.0568 .0376
Darrach	Cast-iron pipe in service 7 years	36	9	1.58 2.37	60.0 66.0	.0716 .0586

Obviously it is not possible to clean the smaller pipes of a distribution system, but large cast-iron conduits may be emptied at suitable periods and have their interior surfaces cleaned of tubercles or other accumulations. At the same time, if necessary, a new coal-tar coating can be applied.

Table XVIII exhibits the values of the coefficient c to be used in Chezy's formula for new and clean coated cast-iron pipes. It represents the results of actual hydraulic experience and is taken from Hamilton Smith's "Hydraulics." A comparison between this table and that which precedes will show how serious the effect of tubercles may be on the discharging capacity of a cast-iron pipe.

In using Chezy's formula, $v = c\sqrt{rs}$, in connection with either Table XVII or XVIII, the slope or sine of inclination s of the hydraulic gradient may be readily computed by equation (10), which gives the head lost by friction in a closed circular pipe as

TABLE XVIII.

VALUES OF c IN FORMULA: $v = c\sqrt{rs}$.

Velocity v, Feet per Second.	Diameters in Feet ($d=4r$).												
	.05	.1	1	1.5	2	2.5	3	3.5	4	5	6	7	8
1	.....	80.0	96.1	102.8	108.8	112.7	116.7	120.2	123.0	127.8	131.8	134.8	137.5
2	77.8	88.9	104.0	110.9	116.2	120.3	123.8	127.0	129.9	134.3	138.0	141.0	143.3
3	82.4	93.7	108.7	115.6	120.8	124.8	128.3	131.4	134.2	138.6	142.3	145.4	147.6
4	85.6	97.0	112.0	118.9	124.0	128.1	131.5	134.6	137.4	141.9	145.5	148.6	151.0
5	87.6	99.3	114.4	121.3	126.5	130.6	134.1	137.1	140.0	144.7	148.1	151.2	153.6
6	89.1	101.0	116.3	123.2	128.6	132.6	136.3	139.4	142.3	146.9	150.5	153.5	
7	90.0	102.4	118.0	125.0	130.4	134.6	138.2	141.5	144.5	149.0	152.7		
8	90.6	103.3	119.3	126.4	132.0	136.3	140.0	143.3	146.3	151.0	154.9		
9	90.7	104.0	120.4	127.7	133.3	137.7	141.6	145.0	148.1	152.8	156.7		
10	90.8	104.5	121.4	128.8	134.5	139.0	142.9	146.4	149.7	154.6			
11	90.9	104.7	122.0	129.7	135.6	140.2	144.2	147.7	151.0				
12	91.0	104.8	122.5	130.4	136.4	141.1	145.2	148.8	152.3				
13	91.0	105.0	122.9	131.0	137.1	141.9	146.1	149.8	153.2				
14	91.0	105.0	123.2	131.5	137.6	142.5	146.7	150.5	154.0				
15	91.0	105.0	123.6	131.8	138.0	142.9	147.2	151.1	154.6				
20(?)	.....	.....	123.9	132.9									

$h = f\frac{l}{d}\frac{v^2}{2g}$. It is only necessary in a straight pipe or one nearly straight to compute the quantity $s = \frac{h}{l} = \frac{f}{d}\frac{v^2}{2g}$.

194. Timber-stave Pipes.—In the western part of the country long conduits or pipe-lines are frequently constructed of timber called redwood. Staves of suitable thickness, sometimes $1\frac{3}{4}$ inches, are accurately shaped and finished with smooth surfaces so as to form large pipes of any desired diameter. These staves are held rigidly in place with steel bands drawn tight with nuts on screw-ends, so as to close tightly the joints between them. Such wooden conduits are rapidly and cheaply built and are very durable. They have the further advantage of requiring no interior coating, as the timber surface remains indefinitely unaffected by the water flowing over it. The latter part of Table XV shows coefficients for Chezy's formula which may be used for such a class of timber conduits. As the interior surfaces of such closed conduits are always very smooth, the coefficients are seen to be relatively large, and such pipes are, therefore, well adapted to maintain unimpaired discharging capacity for great lengths of time.