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SANITARY ENGINEERING

WITH RESPECT TO

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AND

SEWAGE DISPOSAL

BY

LEVESON FRANCIS VERNON-HARCOURT

M.A., M.INST.C.E.

AUTHOR OF
"RIVERS AND CANALS," "HARBOURS AND DOCKS," "ACHIEVEMENTS IN ENGINEERING,"
AND "CIVIL ENGINEERING AS APPLIED IN CONSTRUCTION"

COMMANDER OF THE IMPERIAL FRANZ-JOSEF ORDER OF AUSTRO-HUNGARY
EMERITUS PROFESSOR OF CIVIL ENGINEERING AND SURVEYING, UNIVERSITY COLLEGE, LONDON
AND BRITISH MEMBER OF THE INTERNATIONAL CONSULTATIVE COMMISSION FOR
THE SUEZ CANAL WORKS

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39 PATERNOSTER ROW, LONDON

NEW YORK, BOMBAY, AND CALCUTTA

1907

PREFACE

TOWARDS the end of 1901, when "Civil Engineering as applied in Construction" had been printed off, and was within a few weeks of publication, Messrs. Longmans, having looked through the headings of the subjects dealt with in that book, noticed that the very important subject, "Sanitary Engineering," occupying the four last chapters of the book, had not been included in the volumes forming their "Civil Engineering Series," and quite unexpectedly wrote to propose my supplying this omission. Bearing in mind that several books relating to waterworks and to sewage works had been published in recent years, and that, owing to my very limited leisure, the preparation of "Civil Engineering" had extended over a long period, I felt considerable hesitation about entertaining their suggestion. Eventually, however, recognizing that Messrs. Longmans were right in considering that no Civil Engineering Series could be regarded as complete without a volume on "Sanitary Engineering," and believing that the two subjects Water-Supply and Sewage Disposal, forming in conjunction the basis of sanitary science, might with advantage be dealt with in a single book, I came to the conclusion that I should not be justified in declining the proposal. Accordingly, during the last five years, I have devoted all the leisure time I could spare to the preparation of this book, in the earnest hope that by treating, as fully as practicable within the limits imposed, of two subjects so intimately connected and of such vital importance, the book may prove acceptable and useful to my professional brethren, and, being written in as little technical language as possible, may possess an interest for the public at large, whose health and prosperity are so greatly dependent upon the due observance of the rules of sanitary science, and the efficient carrying out of the works described.

My sincerest acknowledgments are due to the Institution of Civil Engineers for the aid afforded me by their *Proceedings* and very valuable technical library, without which it would be almost impossible to treat properly and fully of any branch of engineering science; and I have endeavoured to record in the foot-notes the sources from which information has been derived, both as a tribute of assistance received, and also as a guide to the reader where further details may be obtained. I also desire to express my indebtedness to my assistant, Mr. Edward Blundell, for the care with which he has prepared the numerous drawings selected by me for illustrations in the text.

In conclusion, a few remarks may be made on the theory, deduced from some experiments with models, that "a dam collapses first by the tension on the vertical sections of the tail," which the *Daily Mail,* in March, 1905, under the heading of the "Nile Dam in Danger," referred to as an "alarming scientific discovery," and to which allusion has been made in the concluding paragraph of Chapter VII., page 153, in the latter part of which chapter the failures of masonry dams have been dealt with. In the first experiment, where the wooden model was divided into a series of horizontal slices, the friction of wood upon wood was naturally quite insufficient to enable this dam, made up of a number of unconnected slices, to resist a pressure corresponding to the relative water-pressure; but after strengthening this dam against shearing stresses by pasting tissue paper over the front and back faces of the dam, so as to obtain a resistance corresponding to the friction usually allowed for masonry on masonry, the dam was able to support a pressure nearly equivalent to the calculated amount before collapsing. In the second experiment, however, where a similar wooden model was divided into a series of vertical slices, with a similar strengthening with tissue paper, the collapse of the dam occurred with a considerably smaller pressure, first opening up very close to the tail, and then shearing over, from which the conclusions were drawn, that "The current theory of the stability of dams is both theoretically and experimentally erroneous," and that "Theory shows that the vertical, and not the horizontal, sections are the critical sections." The conclusions are based on the correctness of the differences in stability exhibited by the two experimental results; but careful consideration serves to show that the two experiments differ very materially in their conformity to the actual conditions of masonry dams. In the first experiment, in spite of the modification introduced by the division into a number of slices, the weight of the several pieces still acted in the ordinary manner, increasing towards the base of the dam with the horizontal pressure, and therefore proportioning the resistance to the pressure. When, however, in the second experiment, the dam was divided up into a number of vertical slices, the ordinary conditions of a masonry dam were completely changed; and instead of its being necessary, provided of course the toe rests upon solid rock, for the centre of gravity of the whole dam to be raised before it could be overturned, the dam in the model was converted into a series of independent, narrow, vertical slices, which could be each tipped over by a moderate horizontal pressure, without any practical raising of the centre of gravity of each narrow slice, just as rows of children's narrow bricks placed on end, near together, on the floor, are readily overturned by a slight side touch. In such a case, the failure would naturally take place near the toe, where, owing to the small height of the slices and the comparatively moderate horizontal pressure, there is little friction of the adjacent surfaces to be overcome, and the tops of the slices are below the centre of pressure and possess little support against a horizontal force. Whilst, however, it is of interest to trace where the fallacies in the experimental investigations have crept in, actual experience suffices to prove the erroneousness of the theory which has been based upon them.

The correct form for masonry reservoir-dams exposed to a considerable water-pressure was determined, about the middle of last century, by careful mathematical calculations, carried out by a French engineer, in accordance with which the Furens dam was erected in 1861–1866, as mentioned on page 142, and has proved perfectly stable, and since which time several other high masonry dams have been constructed on similar lines, with equally satisfactory results; whereas, according to the conclusions drawn from the above experiments, those dams are unstable, are exposed to tensional strains reaching between 5 and 10 tons per square foot, and are subjected to shearing stresses far in excess of anything imagined. Turning now to the failure of masonry dams, the Austin dam in Texas, shown in section on page 146, should, according to the above theory, have failed by an opening at its toe under vertical tensional strains, and then have turned over; whereas, on the contrary, the dam was pushed bodily downstream in an upright position by the horizontal water-pressure. The Bouzey dam also, in the first instance, and the Gros-Bois dam were pushed slightly forward by the water-pressure. Dock walls, moreover, do not fail by vertical dislocations in front, or by overturning if the toe rests on a solid foundation; but they are pushed forward in an upright position by the pressure at the back. Unless any new theory, with regard to the stresses on masonry dams and their stability, can be proved to be more in accordance with the actual results of experience, especially in the case of failures, than the theory on which the designs of the principal masonry dams have been based during the last forty years, and have successfully borne the test in actual practice, the new theory cannot reasonably be accepted in preference to the old one; and whenever a theory, however authoritatively advanced, is evidently at variance with actual occurrences, it may safely be disregarded.

Looking, finally, at this statical problem from a purely common sense point of view, it may fairly be assumed that the authors of the new theory would admit that a masonry dam resting upon a solid rock foundation, and not subjected to any water-pressure, is not exposed to any vertical tensional stresses. The filling of the reservoir behind the dam introduces a practically horizontal water-pressure against the back of the dam, which, even when assumed to be concentrated at the centre of pressure, is very much less than the vertical pressure due to the weight of the dam, when supposed to be acting at the centre of gravity of the dam (see diagram, page 138). How, then, can it be conceived that the addition of this moderate horizontal pressure, as compared with the weight of the dam, could possibly develop the excessive vertical tensional stresses stated, which do not exist at all when the reservoir is empty? The effect of this horizontal pressure is obviously to increase the compressive stresses towards the downstream face of the dam, and to introduce horizontal shearing stresses, which, when a dam or retaining wall rests upon a slippery foundation, or is not properly connected with the stratum at its base, is liable to force the dam or wall bodily forwards.

The Assuan dam is clearly more liable to injury from scour near the toe by the rapid current of the river, and from vibrations resulting from the rapid efflux of the floods of the Nile through it, than to failure from vertical tensional stresses near its toe, to which much higher and slighter dams, such as the Chartrain, Villar, Ban, and Periyar dams (pages 165 and 172), would obviously be far more exposed if the new theory was correct, and yet exhibit no signs of being subjected to excessive strains.

L. F. VERNON-HARCOURT.

6, Queen Anne's Gate, Westminster, S.W.,
October 10th, 1906.

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ERRATA

Page 202, Fig. 122, *for* 'Lock Katrine' *read* 'Loch Katrine.'

Page 384, line 5, *for* 'The upper layers' *read* 'The upper layer.'

PART I.

WATER-SUPPLY

CHAPTER I.

INTRODUCTION; ANCIENT WATERWORKS; AND AVAILABLE RAINFALL.

Objects of sanitary engineering—Importance of Water-Supply—Ancient Waterworks, springs, tanks, wells, storage of river-water, Grecian conduits—Roman Conduits and Aqueducts, early examples, restorations for modern supply, later works—Various Ancient Aqueducts, dimensions of most notable works, introduction of pointed arches—Change effected in Conduits by Introduction of Cast Iron—Ancient and Modern Waterworks compared, improvements introduced—Origin and Measurement of Rainfall, derived mainly from the sea, rain-gauges—Distribution of Rainfall, rainy and rainless districts, decrease of rain inland, effects of monsoons, influence of sun-spots—Differences in Rainfall, high rainfalls in India and elsewhere; examples of small rainfall in India, in Europe, in the British Isles, in America, in Africa, in Australia, and New Zealand—Variations in Rainfall, periods of greatest rainfall in India, in Europe, in Africa—Fluctuations in Rainfall, highest and lowest annual rainfall at some Indian stations, in Great Britain—Average Annual Rainfall, exactness depending on period of record, rules for averages—Available Rainfall, conditions affecting it, difference of, on impermeable and permeable strata—Evaporation, great influence of, estimated losses from, variations in influence—Percolation, conditions affecting it—Flow of Streams and Rivers, causes producing differences in discharge at upper and lower portions of basin, instances of rivers flowing over arid plains.

SANITARY engineering, comprising the provision of pure and ample water-supplies, and the efficient removal of sewage and other refuse from large towns, involves some of the most important and widespread works which engineers have to carry out in the present day, owing to the rapid increase and great concentration of population resulting from the remarkable facilities of inter-communication furnished by the wonderful development of railways and steamships. Both water-supply and sewage disposal are essential for the well-being of large communities congregated together; and they are so intimately connected, that it appears expedient to deal with these two subjects consecutively, as combining to form the main basis of sanitary science.

Importance of Water-Supply.—Water is a more primary necessity of life than even food; it is indispensable for the maintenance of health and cleanliness; and it is required for various manufacturing processes. In an impure state, however, it is an insidious vehicle of disease when introduced into the human system; and the senses are incapable of detecting many of the very minute but deadly germs of organic contamination. The moderate supply of water needed for drinking and cooking cannot generally be kept distinct from the much larger supply required for various other domestic and municipal purposes; and, consequently, an ample supply of pure water is essential for the healthiness of any large number of persons collected together within a limited area, and more especially amongst the densely crowded populations of large cities and towns.

In olden times, towns, villages, and religious institutions were almost always established near the banks of rivers, with the special object of readily securing a good supply of water. Moreover, when the communities of Greece and Italy became highly civilized, and congregated in large numbers into their capitals and principal towns, large works were undertaken for the conveyance of pure supplies of water from a distance; and remarkable evidence is furnished of the degenerate condition into which these nations lapsed, from a process of gradual decline and the incursion of barbaric hordes, by the decay into which these fine works were suffered to fall during the Middle Ages.

Ancient Waterworks.—Since water is indispensable to human existence, men have been forced, from the very earliest times, to secure a supply of it. Primitive communities scattered over large areas, small in numbers and free to select their place of abode, have had their wants amply provided for by the springs and watercourses found in most parts of the world, and from which water is readily obtainable without the execution of any works. Springs, indeed, in remote antiquity, owing to the clear, cool, sparkling, and pure water they generally afford, were highly prized and carefully guarded; and on account of their source being unknown, they were held in great veneration, and sometimes supposed to be of divine origin.

When, however, population increased, and extended into districts where natural supplies were deficient, it became necessary to have recourse to artificial aids; and the earliest works for obtaining water were naturally undertaken in hot, dry countries. The simplest expedient consisted in digging out hollows for the collection of rain. Thus, in India, large tanks were excavated, in very early times, for storing up the rainfall in the wet season, to provide a supply of water during the dry season; pools formed on high ground in the outskirts of ancient Jerusalem, supplied the town with water; and in many cities, water was obtained by leading the rainfall, flowing off the roofs of the houses, into underground cisterns.

Wells were also sunk into the ground in many countries, at remote periods, for collecting and storing the underground waters, which were raised from these shallow wells by simple mechanical contrivances, which may be still seen in use in Egypt and India. Wells of great antiquity exist in Egypt; the use of wells in the time of Abraham and his descendants is recorded in the Bible; and artesian wells were sunk in China in very early times. Wells, also, were found amongst the ruins of Nineveh; and they furnished some of the earliest supplies of water in ancient Greece and Italy.

The Egyptians appear to have originated the storage of river-water, by diverting some of the flood discharge of the Nile into extensive enclosed depressions to serve as reservoirs, of which Lake Mœris was a well-known instance, with an area of about 47 square miles, from which water was drawn for summer irrigation during the low stage of the river; and the same system was adopted in Babylonia for utilizing the waters of the Tigris and Euphrates.

The Greeks devoted much attention to the supply of water, and evinced considerable skill in its conveyance from sources in the neighbouring mountains to their cities; but as they were able to carry their conduits along the contour lines of the mountain slopes in cutting, and through spurs and ridges in tunnel, these works, though well adapted for their purpose, did not furnish conspicuous objects, like the Roman aqueducts crossing valleys, and have left comparatively few visible remains.

Ancient Conduits and Aqueducts for the Water-Supply of Rome.—When Rome was the capital of the known world, it was abundantly supplied by several conduits with water from sources at a distance; and the most notable feature of these ancient works was the boldness with which, in Italy and the countries under Roman domination, these conduits were conveyed across intervening valleys on arched masonry or brick aqueducts of imposing dimensions, the remains of which constitute the most celebrated engineering records of olden times.

Rome was for the first time supplied with water from a distance in 312 B.C., when Appius Claudius had a conduit constructed, named after him Aqua Appia, carried almost wholly underground, which, having a length of about $10\frac{1}{2}$ miles, conveyed water from the base of the Alban Hills to the city. Another conduit, constructed forty years later, brought water from the river Anio near Tivoli, about $40\frac{3}{4}$ miles in length, and known by the name of Anio Vetus; whilst twelve more conduits were subsequently constructed at various periods, carried mainly underground, but passing over the valleys and low-lying plains traversed in their course, on arched aqueducts, several fine remains of which are still in existence. The most important of these conduits were the Aqua Claudia, 45 miles long, commenced in the reign of Caligula, and completed under the Emperor Claudius about 48 A.D., bringing water from springs at the base of the Sabine Hills, and the Anio Novus, constructed during the same period, which derived its supply from the river Anio at a distance of 62 miles from the capital. These two conduits met 6 miles away from Rome; and from this point their waters were conveyed separately in two distinct channels, one above the other, on an arched aqueduct attaining a maximum height of 109 feet.

Reservoirs, termed *castella*, were built at intervals alongside these conduits, for the supply of water to the districts passed on their route, and to provide for drawing off the water for repairs. The turbid river water was led into large, covered, settling tanks called *piscinæ*, constructed just outside Rome, so that the silt carried in suspension might be deposited before the water was discharged into the main reservoirs, whence it was passed for distribution into numerous small reservoirs formed in various parts of the city.

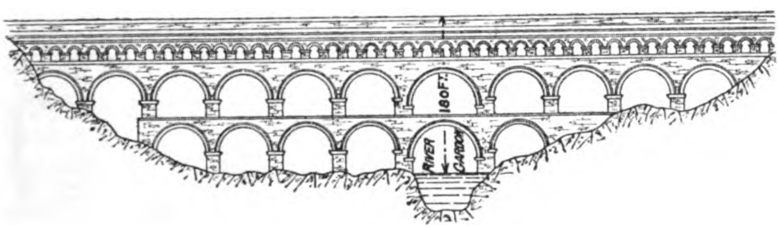
Two of the ancient conduits have been restored and utilized for the water-supply of modern Rome, the first to be brought into use again being the Aqua Virgo, about 12 miles long, and carried for the most part underground, which was originally constructed by order of Agrippa, completed in 27 B.C., and restored in 1453, furnishing an abundant supply from a very pure spring, and now known as the Acqua Vergine. The second ancient conduit brought again into operation in comparatively modern times, was the Aqua Trajana, constructed in the reign of Trajan for carrying water to Rome from Lake Bracciano (at an elevation of 528 feet above sea-level) and adjacent springs, a distance of 32⅓ miles, and restored in 1611 under Paul V., from whom it got its present name of Acqua Paola. A third ancient source of supply which has again been drawn upon for the wants of modern Rome, consists of springs in the Sabine Hills, whose waters were originally conveyed to Rome by the Aqua Martia, constructed about 146 B.C., under the direction of the prætor Quintus Martius Rex, having a length of 56 miles, for about 6½ miles of which it was carried on an arched aqueduct. The water from this source was only reintroduced into Rome in 1870, on the completion of the new conduit, 33 miles long, constructed on the modern system, with cast-iron pipes across the low-lying country in the last half of its course; and this conduit, having been opened shortly before Pius IX. lost possession of Rome, is known as the Acqua Pia.

A fourth conduit, formed after the ancient models in relatively recent times, is the Acqua Felice, which has supplied water to Rome from the Alban Hills for over three hundred years, having been completed in 1586. This conduit, with a total length of 22 miles, including 6¼ miles of arched aqueduct, was to a great extent constructed with stones obtained from the ruins of the arched aqueduct of Aqua Claudia, which was about 10 miles long, and the remains of which still stretch for about 6 miles across the Campagna.

Various Ancient Aqueducts.—Many arched aqueducts were erected across valleys by the Romans in the provinces under their sway, the finest of which is the Pont du Gard, in the south of France, with three tiers of arches, reaching an elevation of 180 feet above the river Gardon, whose valley it crosses in a length of 873 feet,[1] Fig. 1, forming a portion of the conduit constructed in the reign of Augustus, which conveyed spring water to Nimes from sources in the hills near Uzès, a

PONT DU GARD AQUEDUCT.

Fig. 1.



distance of 35 miles. There is another aqueduct on this conduit near Vert, built with a series of 256 arches in a length of 6435 feet; but its maximum height is only 50 feet. Interesting ruins of aqueducts built by the Romans exist at Mainz and near Metz in Germany, at Tarragona and Merida in Spain, and near Mitylene and Antioch in Asia. The ancient aqueduct constructed to bring water into Segovia, 2400 feet long and 102 feet maximum height, with two tiers of arches, was restored in 1483 by order of Queen Isabella, and supplies the town with water at the present day; whilst a Roman aqueduct at Evora, in Portugal, is in a good state of preservation. Remains exist of a long aqueduct which supplied Carthage with water, partly buried in sand, the period of whose construction is uncertain; and a portion of this conduit has been opened out, and conveys water to Tunis.

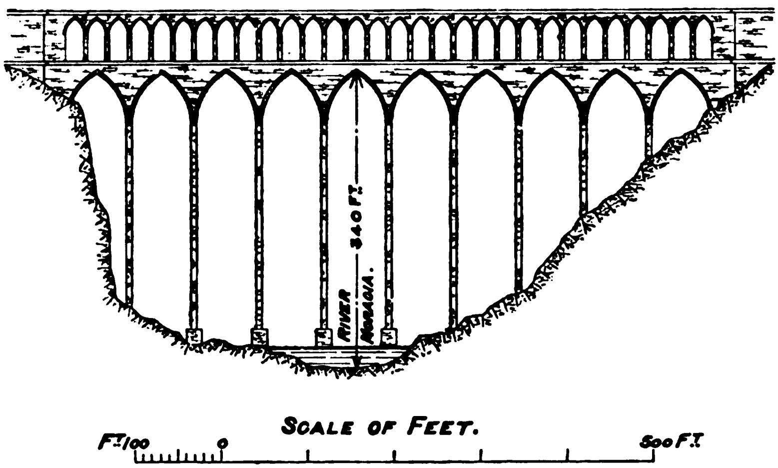
The early aqueducts were built with circular arches; but two notable instances may be cited of later aqueducts, in which pointed arches were introduced. The aqueduct near Spoleto, in Italy, though originally built on a massive plan about the beginning of the seventh

1 "Encyclopédie de l'Ingénieur," J. R. Delaistre, Paris, 1812, p. 51, plate 23.

century by one of the early dukes of Spoleto, is believed to have received its present light style with high, pointed, brick arches, from a restoration in the fourteenth century,[1] Fig. 2. The ten arches have each a span of 66 feet, and reach a maximum height of 340 feet above the bed of the river. This aqueduct is 810 feet long, and attains a height of 410 feet above the bottom of the gorge through which the river Moragia flows; and it now serves as a bridge as well as an aqueduct.

SPOLETO AQUEDUCT.

Fig. 2.



The aqueducts, also, conveying water from sources in the valley of Belgrade, in Rumelia, near the eastern extremity of the Little Balkans, across intervening valleys, to supply Constantinople, 15 miles distant, were constructed at a late period of the Roman Empire, with pointed arches in two or three tiers.

Change effected in Conduits by the Introduction of Cast Iron.—The ancient Greeks and Romans appear to have employed pipes made of earthenware, wood, and occasionally lead, for the conveyance and distribution of their supplies of water; and they were not ignorant of the value of inverted siphons for following depressions in the ground or dipping under obstacles, as remains of leaden siphons have been discovered. Lead, however, though sometimes used by them for passing under streams, or even rivers, and other impediments, is not a suitable material, owing to its small strength and its cost, for conveying considerable volumes of water under pressure in large pipes

[1] "Encyclopédie de l'Ingénieur," J. R. Delaistre, plate 4.

for long distances; and, moreover, the Romans were chary in the use of lead for water-pipes, being aware of its liability to contaminate the water brought in contact with it. Accordingly, the Romans, though often making their conduits contour the hillsides, like the Greeks, when the conditions were favourable, became obliged to resort to raised aqueducts with the requisite fall for conveying their water-supplies across deep valleys, and for bringing the water at an adequately high level into towns surrounded by low-lying plains, as in the case of Rome.

The general adoption, however, of cast iron as a material for water-pipes early in the nineteenth century, enabled supplies of water to be readily carried under pressure, in inverted siphons, down deep valleys or across low-lying lands; whilst the height at which the supply can be delivered merely depends upon the elevation of the source above the town to be supplied, or the available fall. The great modification in the method of constructing conduits thus introduced, which is well exemplified by comparing the modern Acqua Pia with the remains of the ancient Aqua Martia, has dispensed with the necessity of building raised, arched aqueducts, except in very special cases, as, for instance, where in crossing a deep, narrow valley it may be deemed expedient to avoid a considerable dip of the line of pipes, involving a considerable increase in length accompanied with a high pressure, or where, in traversing a river, it may be desired not to bury the pipes out of sight, and to reduce the pressure.

The waterworks engineer of the present day is, accordingly, very rarely called upon to erect structures at all comparable to the magnificent aqueducts of Roman times; but these works, even in their decay, command admiration as the engineering triumphs of an earlier civilization.

Ancient and Modern Waterworks compared.—Almost all the sources of supply employed at the present day were utilized by civilized nations in ancient times, such as tanks and reservoirs for the direct collection of rainfall, springs, watercourses and rivers, wells, and lakes. Reservoirs also for the collection of the flow from springs, were also resorted to; but storage reservoirs, in which the flood flow of streams is impounded by dams across valleys, constitute a comparatively modern system of securing a supply of water, except in the primitive form adopted for irrigation tanks in India, with low earthen embankments; whilst the construction of masonry dams across valleys for the formation of impounding reservoirs, though apparently first introduced in Spain between three and four centuries ago for storing water for irrigation, was not established upon correct principles before the second half of the nineteenth century.

The supply provided by the Romans appears to have been generally ample for the population; but besides the impracticability, with the available materials, of following the depressions in the ground with pipes under pressure, the size of the conduits and aqueducts seems to have been determined more with a view to accessibility for repairs, than with regard to the volume of water to be discharged by them with the given fall. Moreover, the primitive nature of the machinery for raising water in olden times, rendered it important to introduce the flow, if possible, at a high enough level to supply the town direct, involving selecting a source on high ground, and a raised aqueduct across low plains such as those which surround Rome.

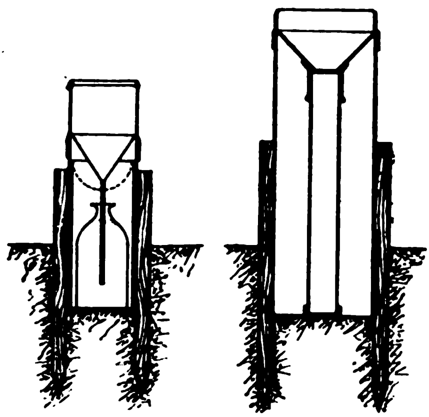
Settling tanks and covered service reservoirs were provided in the Roman works; but the distribution was only very partially carried out, and numerous fountains furnished the chief provision for a general public supply. The purity of the supply in ancient times depended upon the nature of its source; for beyond the removal of the heavier suspended matter where settling tanks were provided, nothing was done to improve its quality. The distribution, however, to every house in a town, only became economically practicable when cast iron could be brought into general use for water-pipes; whilst the principal method relied on in the present day for the purification of water-supplies, namely, the slow passage of the water through filter-beds, was introduced for the first time on a large scale in 1828, for a portion of the London water-supply, and has not even yet been very generally adopted by some of the principal civilized communities of the world.

Origin and Measurement of Rainfall.—Whatever may be the source from which a supply of water is obtained, the actual origin of the water is rain, resulting from the aqueous vapour drawn up by the sun's heat, mainly from the sea, but also to some extent from the land, so as to form clouds, from which the moisture, when condensed, falls as rain or snow. The volume of rain, however, which falls on the land varies greatly with the locality and the season of the year; and, consequently, regular records of the depth of the rainfall in a given period have to be obtained, measured commonly in inches and hundredths of an inch by means of rain-gauges, fixed at a number of suitable open sites, in order that a reliable estimate may be formed of the possible supply which may be collected from any given area.

Rain-gauges are generally made with a copper cylinder supporting a funnel near the top, which receives the rainfall and discharges it into a bottle, from which it is poured daily into a measuring glass, or into a narrow cylinder with a cross section one-tenth the top area of the funnel, in which the depth of the water is measured by a wooden scale, Fig. 3. The smaller rain-gauge in the illustration is the form generally used in Great Britain, with its funnel 5 inches in diameter, and its top 1 foot above the ground, to avoid splashes off the ground in heavy rain.[1] The larger rain-gauge, with a brass funnel 8 inches in diameter, is the type adopted by the Weather Bureau of the United States; and its top is generally raised 3 to 6 feet above the ground.

RAIN-GAUGES.

Fig. 3.



Distribution of Rainfall.—The sea covering a large portion of the surface of the globe, being the source from which rain is principally derived, the districts bordering the ocean are the most rainy, especially if they face the direction of the prevalent winds, and if these winds have traversed a great expanse of water. Moreover, mountain ranges near the sea-coast arrest the progress of the heavily charged clouds coming from the ocean and travelling towards the interior of the country, causing them to discharge their moisture as rain in rising up the mountain slopes, under the influence of the reduction in temperature at higher elevations. On this account some of the rainiest districts are found on the slopes of the Andes and the Rocky Mountains, which border the western coasts of America, and directly face the vast expanse of the Pacific Ocean, and also along the eastern coast of South America, north of Rio Janeiro, up to the West India Islands. Heavy rainfall occurs on the western side of Africa near Sierra Leone, and along the coast of Guinea, and also on the east coast, and over the high lands between the Victoria Nyanza and Abyssinia; whilst the west coast of India bordered by the Ghats, the slopes of the Himalayas nearest to the Bay of Bengal, and portions of the Malay Archipelago, are very rainy.

On the contrary, rainless regions are generally situated in the interior of large continents, cut off from the ocean by high mountain ranges, in passing over which the clouds are gradually deprived of the moisture collected by evaporation from the sea before reaching those districts, such, for instance, as the vast deserts of Northern Africa, Arabia, Persia, and Central Asia. The rainfall, moreover, over the plains in the interior, and on the leeward side of a mountain range, is always

[1] "Hints to Meteorological Observers," William Marriott, 5th edition, 1902, p. 20.

considerably less than on the mountain slopes, though the land may be watered by the drainage descending from the hills in the form of streams and rivers, resulting from the melting of the snows on the mountain-tops, and the rainfall on the inner high, and consequently cold slopes, from the attenuated clouds which have passed over the summit ridge. The rainless districts of Mexico and Peru appear to furnish exceptions to the above rule; but the arid table-land of Mexico lies in reality between two mountain ridges, which intercept the moisture derived from the Pacific and Gulf of Mexico respectively; whilst the humid winds from the Pacific pass unchanged over the low lands of Peru, often in the form of mist, till at length, on reaching the Andes, their burden of moisture is deposited.

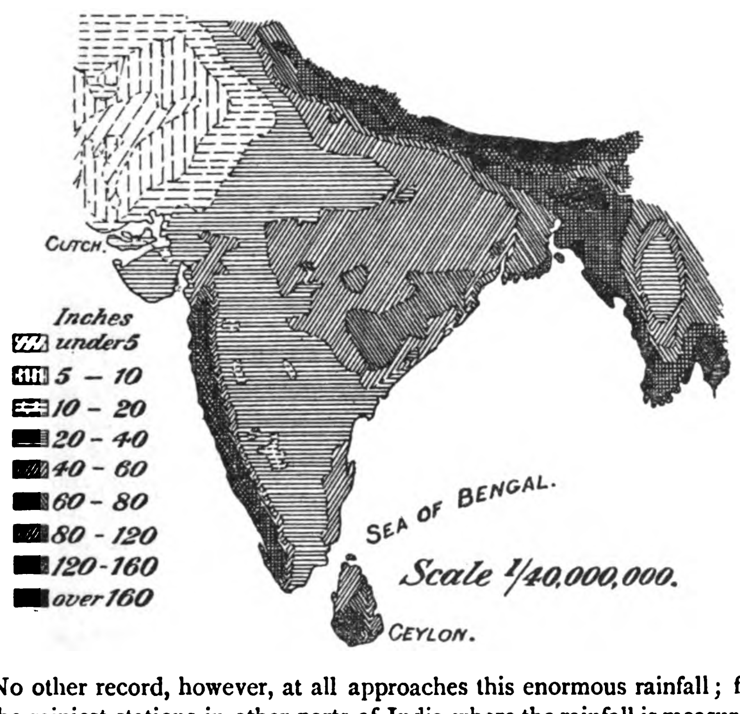
Under ordinary conditions, the rainfall decreases from the sea-coast towards the interior, owing to the gradual abstraction of moisture from the air in its passage over the land, and also from the tropics towards the poles, in consequence of the decrease of evaporation with the diminution of temperature. On the other hand, the rainfall increases with the elevation, within limits depending upon the locality, the maximum rainfall occurring at a height of about 4500 feet above sea-level in India, and at a height of only about 1900 feet in England. In tropical countries, the rainfall generally varies with the position of the sun, the rains being very heavy whilst the sun is nearly vertically overhead; whereas droughts occur during the cooler season. As the direction of the wind, however, has a great influence upon the rainfall, the rainy season in countries subject to periodical winds, especially near the sea-coast, depends largely on these winds. Thus, on the sea-coast of India, where the monsoons blow with considerable regularity in two different directions during definite periods of the year, the rainy season occurs on the western coast during the prevalence of the south-west monsoon from May to October, and on the eastern coast during the earlier period of the north-east monsoon, which blows from October to February.

Sun-spots appear to affect appreciably the heat of the sun, which is greatest when the sun-spots reach a maximum, a result occurring in cycles of from 10 to 12 years; and it has been found, from observations in various places, that the rainfall is generally greater in the years when the sun attains its greatest heat. In India, when the sun-spots are at a minimum, the rainy south-west monsoon is feeble, which occasions a great irregularity in the distribution of the rains, a heavy rainfall taking place in some districts, whilst a drought prevails over wide stretches of country.

Differences in Rainfall.—As the rainfall in any locality depends upon the position of the site in relation to the sea, the prevalent winds, and the physical characteristics of the surrounding country, it necessarily exhibits very great differences in various parts of the world; and it also usually varies considerably in different parts of the same country, and occasionally in places only short distances apart, owing to abrupt changes in the conditions resulting from the intervention of mountain ranges. Observations show that the rainfall varies from zero over the very extensive deserts of Asia and Africa, up to an annual average of 445 inches at Cherra Punji, situated on the southern slope of the Khasi Hills in Assam, facing the Bay of Bengal, at an elevation of 4455 feet above the sea, which has the highest recorded rainfall in the world.

RAINFALL MAP OF INDIA.

Fig. 4.



No other record, however, at all approaches this enormous rainfall; for the rainiest stations in other parts of India where the rainfall is measured regularly, are Malcolmpeth, Satara, on the Western Ghats to the south of Poona, with an average yearly rainfall of about 281 inches; Lalakhal, Sylhet, in Assam, with 266 inches; Buxa, Darjiling, 213 inches, and Matheran, at an elevation of 2200 feet on the Ghats near Bombay, 209 inches;[1] whilst the wettest places in other parts of the world appear to be Matuba, in the island of Guadeloupe, in the West Indies, and San

[1] "Rainfall of India," Meteorological Department, Calcutta, 1890–1900.

Luiz de Maranhão, on the northern coast of Brazil, with average rainfalls of 292 and 280 inches respectively in a year.[1] Records show rainfalls at some other places exceeding 200 inches on the average annually; whilst many localities are subject to a yearly rainfall of between 200 and 100 inches.

Though the rainfall in India on the seaward and windward slopes of the hills is excessive in some places, as indicated above, and is very large over considerable areas thus situated, it becomes quite small in the interior of the country, and very slight in some districts,[2] Fig. 4. Thus the average annual rainfall is 28 inches at Delhi; 18 inches at Lahore; $18\frac{1}{2}$ inches at Bellary, in the centre of Southern India; 12 inches at Peshawar, near the Khyber Pass; 10 inches at Quetta, about 18 miles beyond the summit of the Bolan Pass, at an elevation of 5500 feet; 8 inches at Haiderabad, in the Lower Indus valley; and only $4\frac{1}{3}$ inches at Jacobabad, on the northern boundary of Sind; whilst at the hill station of Leh, in Kashmir, at an elevation of 11,500 feet, on the northern slopes of the Himalayas, the average annual rainfall is only 3·2 inches.[3] Further north, beyond the limits of India, the rainless deserts of Central Asia are reached, as all the moisture of the humid winds coming from the Arabian Sea and the Bay of Bengal, is deposited in their passage over the intervening mountain ranges.

In Europe, the greatest rainfall occurs along the western coasts bordering the Atlantic Ocean, especially where there are ranges of hills in the neighbourhood of the sea, as in Norway, the west of Scotland, Cumberland, the hilly districts of the south of France and north of Spain, and Portugal; whilst in proceeding eastwards towards the interior, the rainfall in general decreases, having an annual average of between 35 and 20 inches over the plains of France and Germany, and from 20 down to 2 inches in Russia and Siberia. The rainfall, however, increases again on reaching mountainous regions, such as the Alps, Fig. 5.

The rainfall of the British Isles exhibits considerable differences according to the conditions of the localities, attaining an annual average of 173 inches at The Stye, at an elevation of 1077 feet; 140 inches at Seathwaite, at a height of 422 feet, in the hilly district of the Cumberland lakes; and 171 inches at the Ben Nevis observatory, 4407 feet above sea-level; whilst it is 67 inches at Kenmare, on the south-west coast of Ireland, and 96 inches on the Mangerton Mountains in the Killarney district, at an elevation of 1760 feet, where the hills are not high; 61

[1] "The Physical Atlas of Natural Phenomena," A. Keith Johnston, p. 66.

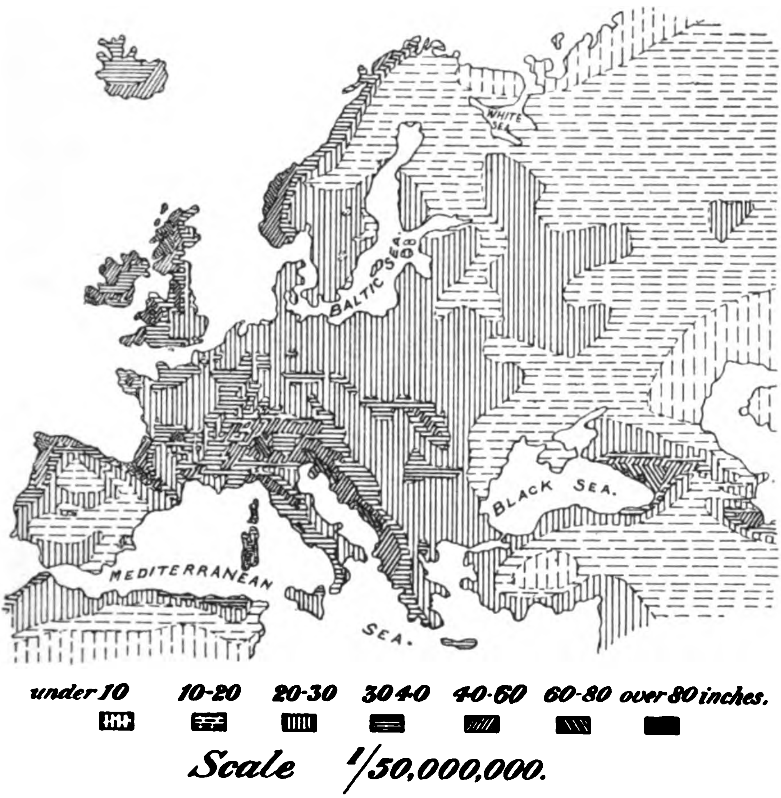
[2] "The Climates and Weather of India, Ceylon, and Burma," H. F. Blanford, p. 66.

[3] "Rainfall of India."

inches in the Isle of Skye; and 49½ inches at the Woodhead reservoir, on the high inland moorlands at the borders of Cheshire and Derby-

RAINFALL MAP OF EUROPE.

Fig. 5.



shire,[1] Fig. 6. On the eastern side of England, however, there are many parts in the flat counties of Bedford, Cambridge, Norfolk, and Lincoln, where the average annual rainfall is under 23 inches. The influence, indeed, of flat land and shelter from the wettest and prevalent winds, which in England come from the west, in making the rainfall small, is exemplified by two of the places of minimum rainfall in England, namely, Hunstanton and Grimsby, being situated on the shores of the Wash and the Humber estuary, opening on to the North Sea; and the absence of an increase of rainfall on the eastern coast from its

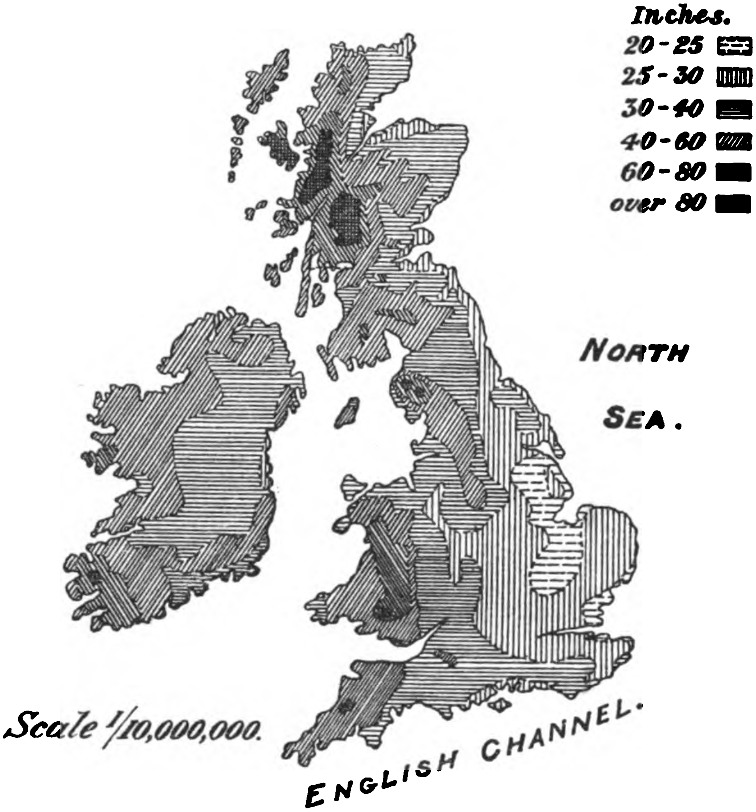
[1] "British Rainfall," G. J. Symons.

bordering the North Sea, must be attributed to this sea being too narrow for the dry easterly winds, coming over from the Continent, to gather up much moisture in passing over it, as well as to the flatness of the land on to which they blow.

In North America, the rainfall is fairly uniformly distributed over

RAINFALL MAP OF GREAT BRITAIN AND IRELAND.

Fig. 6.



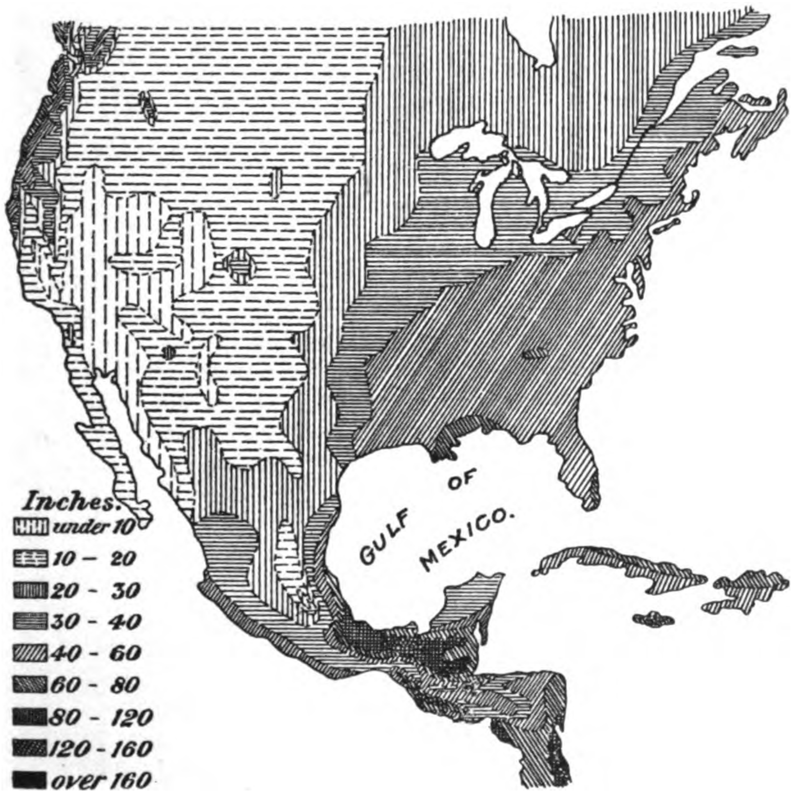
the eastern portion, the lands bordering the Atlantic Ocean being naturally the most rainy, especially Florida and the West India Islands; whereas the northern belt of land between the Pacific Ocean and the Rocky Mountains has a large rainfall; and the plains to the east of this range are arid, about half across the continent, and as far eastwards as Winnipeg in Canada, from the scarcity of rain,[1] Fig. 7. Central

[1] "Atlas of Meteorology," J. G. Bartholomew and A. J. Herbertson, vol. iii. plate 21.

America, including the isthmus of Panama, being bounded by the Pacific Ocean, the Gulf of Mexico, and the Caribbean Sea, owing to its relatively small width and its nearness to the equator, is necessarily rainy; whilst the north-eastern part of South America for a considerable

RAINFALL MAP OF NORTH AMERICA.

Fig. 7.



Scale 1/50,000,000.

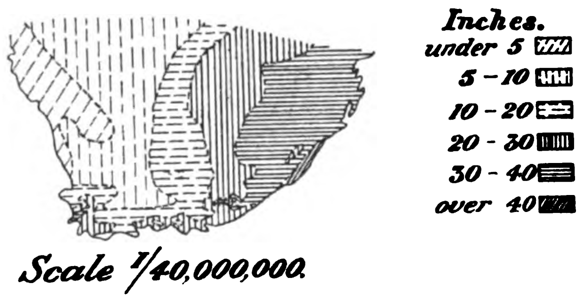
distance into the interior, and the mountainous regions bordering the Pacific, are very rainy.

The northern portion of Africa is rainless, except near the sea-coast; whilst the central equatorial regions are rainy throughout the year. In South Africa, the eastern half, the southern coast, and the district round Cape Town, are rainy; whereas the western half, with the exception of this district, is dry, Fig. 8.

The northern and eastern portions of Australia, for some distance inland from the sea-coast, and also the district bordering the south-

RAINFALL MAP OF SOUTH AFRICA.

Fig. 8.



western coast, are wet; whilst the southern and interior regions, and especially the central part of Western Australia, are very dry,[1] Fig. 9. New Zealand, situated in mid-ocean and comparatively narrow, is naturally rainy, the wettest portion being the western part of the south island, where the sea-coast is bordered by hills; and the driest portion is the eastern part of this same island, under the shelter of the western range of hills,[2] Fig. 10. Tasmania also, being of moderate extent and hilly, has a fairly large rainfall, Fig. 9.

Variations in Rainfall.—The rainfall varies at different seasons of the year, and its total yearly amount varies from year to year. Within the tropics, and in countries visited by periodical winds, there are generally wet and dry seasons; but in other parts of the world the rainy and dry periods are not clearly defined, depending mainly on the prevalence of certain winds, and varying with the locality. In India, the rainy season depends upon the monsoons, and the exposure of the locality; for though the north-east monsoon is a dry wind in comparison with the south-west monsoon, it condenses the moisture collected in blowing across the sea, on to the south-eastern Coromandel coast from October to December; whilst the western and northern coasts of India and Burma are exposed to the wet south-west monsoon, which prevails from April to October, and whose influence, from its strength and humidity, extends into the interior. Accordingly, the main rainfall over almost the whole of India and Burma occurs between May and September, the month of greatest rainfall being July. Along the

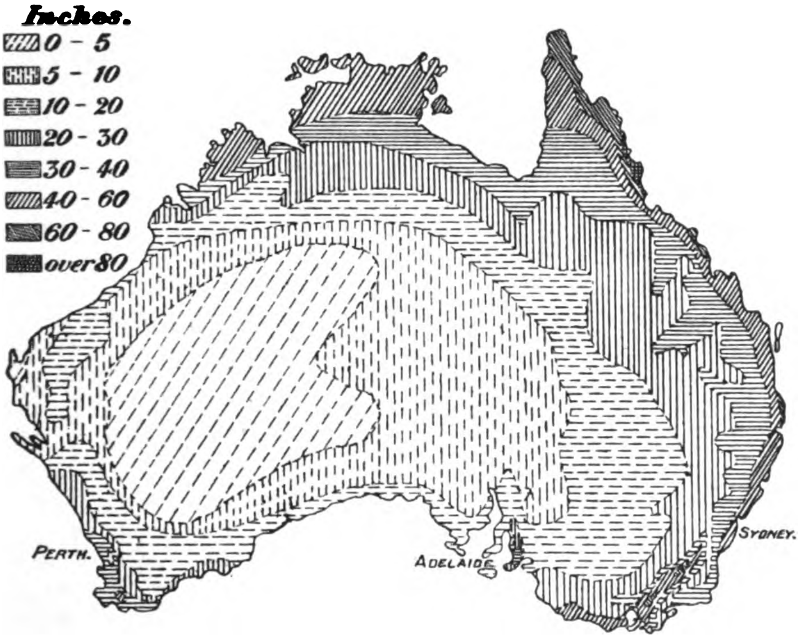
[1] "Atlas of Meteorology," J. G. Bartholomew and A. J. Herbertson, vol. iii. plates 26 and 27.

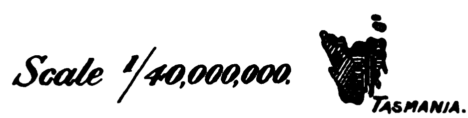
[2] *Ibid.*, vol. iii. plate 21.

south-east coast of India and in Ceylon the rainfall is heaviest between October and December, the month of maximum rainfall being October

RAINFALL MAPS.

Fig. 9.—Australia and Tasmania.





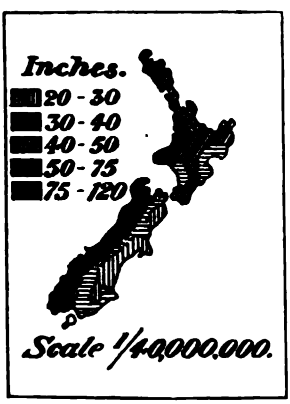


Fig. 10.—New Zealand.

at Point de Galle and Colombo, and November at Trinkomali, Cuddalore, and Madras. Except on the rainy slopes of the Himalayas and the hilly parts of Ceylon, there is hardly any rain during the three or four first months of the year; and this is also the case during the last two months of the year, with the sole exception of coasts under the influence of the north-east monsoon.

The average maximum monthly rainfall in Europe varies in the time of its occurrence according to the situation, the only months in which the maximum nowhere occurs being February, March, and April.[1] On the extreme western coasts, comprising Scotland, Ireland, France, and Portugal, December and January are the rainiest months on the average. In the eastern midlands of England, the wettest period is July and August, in the midland counties it is September, and in most of the rest of England and Scotland it is October; whilst November is the month of maximum rainfall in the extreme north of England, the south-west of Wales, and the north-western half of Ireland. The month of minimum rainfall is March in the north of France and south of Great Britain, it is April further north, May in the northern part of Scotland, and June in the Orkney and Shetland Isles and in Iceland. The distribution, however, of the rainfall throughout the year in temperate climes, with their changeable winds, varies considerably in different years.

Though the equatorial regions of Africa are rainy throughout the year, the northern and southern limits of the wide rainy belt shift their position according to the seasons, the most southern position being reached between December and February, so that the rainy district on the borders of the Sudan is restricted during this period. In South Africa, the most rainy season of the wet eastern half is from January to March, and it is very dry in this district from May to August; whereas the rainy period of the dry western portion, and of the more wet districts of Cape Town and the south coast, is between April and September.

Fluctuations in Rainfall.—The total yearly amounts of rainfall in a series of years in any locality exhibit large fluctuations, falling considerably below the average in very dry years, and rising much above the average in exceptionally wet years.

In India and Burma, the highest annual rainfall at the stations where records are kept, which is commonly less than double the minimum rainfall, reaches at some stations three times, and in a few places four times the minimum and more, the proportionate range being generally less at places with a large rainfall. Thus, for instance, the maxima and minima recorded annual rainfalls in inches are, 254 and 142 at Akyab, 227 and 84 at Cochin, 161 and 77 at Darjiling, 115 and 36½ at Bombay, 93 and 37 at Calcutta, 78½ and 48½ at Trinkomali, and 57 and 12⅓ at Poona.[2] The ratio, however, between the highest and lowest yearly rainfalls is occasionally even greater than in the examples given above, especially at places of small average rainfall. For example, the maxima and minima recorded yearly rainfalls in inches are, 43·5 and

[1] "Encyclopædia Britannica," 9th edition, vol. xvi. p. 152.

[2] "The Climates and Weather of India, Ceylon, and Burma," H. F. Blanford, Appendix I.; and "Rainfall of India," Meteorological Department, Calcutta, 1890–1900.

8·2 at Delhi, 37·8 and 5·3 at Lahore, 27·9 and 5 at Peshawar, 21·6 and 4·2 at Quetta, 16·1 and 1·9 at Multan, 12 and 0·7 at Jacobabad, 9 and 0·4 at Leh, 20·2 and 0·4 at Haidarabad, Sind, and 28 and 0·5 at Karachi; whilst unusual instances of large proportionate differences at places with higher rainfalls are furnished by Madras with 88·4 and 18·5 inches, Surat with 89·3 and 16·7 inches, Trichinopoli with 95·3 and 17·1 inches, and Mount Abu with 130·3 and 11·4 inches. Comparing the minima falls with the averages, it appears that the rainfall at the above exceptional stations is liable to be reduced in the driest years, from between a little less than half the average at Trichinopoli down to one-twentieth of the average at Haidarabad, the reduction below the average amounting to about 60 per cent. at Peshawar, Quetta, Madras, and Surat; 70 per cent. at Delhi, Lahore, and Multan; 85 per cent. at Jacobabad and Mount Abu, and 88 per cent. at Leh; and reaching 93 per cent. at Karachi, and 95 per cent. at Haidarabad.

In the British Isles, considerable fluctuations occur in the amounts of the annual rainfall, though not at all approaching in magnitude the exceptional instances given above of rainfall in India. Comparing the rainfall of the wet year 1872 with that of the exceptionally dry year 1887, it appears that the rainfall of 1887 was in many places only about half that of 1872; whilst at Combs reservoir in Derbyshire, Newton reservoir in Cheshire, and Durnford Bridge in Yorkshire, it did not amount to a third of the rainfall of 1872. The rainfall of 1887 was 49 per cent. below the average at Durnford Bridge and Newton reservoir, but only 15 per cent. deficient at Wimbledon; and in Scotland in that year, it ranged from 50 per cent. below the average at Aberfoyle in Perthshire, to only 6 per cent. at Kelso.[1] The lowest recorded rainfall in the British Isles in 1887 was 12·78 inches at Sleaford, Lincolnshire, and the highest 130·90 at The Stye, Cumberland; whereas the rainfall at these places in 1872 reached 34·87 inches and 243·98 inches respectively.

Fortunately for the supply of water, wet years alternate to some extent with dry ones, so that, in the British Isles, five years of rainfall below the average rarely occur in succession; and it is unusual for the rainfall to be below the average during three consecutive years.

Average Annual Rainfall.—After observations of rainfall have been recorded at any place for a series of years, the mean value of the annual rainfalls enables the rainfall of the place to be readily compared with that of other places, and affords a standard for denoting years as wet or dry, those with a rainfall above the mean being described as wet years, and those below the mean as dry ones. The exactness of the average rainfall deduced from the records, necessarily depends on the

[1] "British Rainfall," G. J. Symons, 1872 and 1887.

length of time during which the records have been kept continuously; and slight modifications in the averages may be introduced by the additional data of subsequent years. Where observations of rainfall have been kept for only a short time, it may be possible to obtain a more correct average by ascertaining the average rainfall of a place with a long record, and subject to similar meteorological conditions, during the shorter period, comparing this result with the real average of the whole record of the place, and then modifying the average of the station with the short record in the same proportion. From an investigation of a number of long reliable records of rainfall, it has been ascertained that a record kept for thirty-five years furnishes an average which is within 2 per cent. of the exact mean, and that a twenty years' record gives an average in which the deviation from the real mean does not exceed $3\frac{1}{4}$ per cent.[1]

The average annual rainfall supplies the basis on which the calculations of the supply to be derived from a given gathering ground are founded, though this is in excess of the amount that can be relied upon, owing to the liability of the rainfall occasionally to be more or less below the average for three or more years in succession. From a comparison of the rainfall at forty-five stations in the British Isles during a period of forty-three years, it appears that, on the average, the rainfall of the wettest year is nearly one and a half times the mean, the rainfall of the driest year is two-thirds of the mean, the rainfall of each of the driest two consecutive years is three-quarters of the mean, and the rainfall of each of the driest three consecutive years is four-fifths of the mean.[2] Very similar proportions, moreover, have been found to apply to the rainfall in various parts of the world.[3]

Available Rainfall.—The rain that falls on any gathering ground is by no means all available for water-supply; for some is restored again to the clouds by evaporation; some sinks into the ground, and may under certain conditions only come again to the surface a long distance away; and some is absorbed by vegetation. The loss from these causes depends upon the period of the year, the nature and slope of the ground, and the condition of the surface of the soil. Evaporation is very active in hot, dry weather, and on flat, impermeable strata; whereas the rain flows down sloping impermeable strata into the nearest watercourse with little loss. On the contrary, the rain sinks into permeable strata, and though thus secured from loss by evaporation, it may only appear

[1] "Average Annual Rainfall," A. R. Binnie, *Proc. Inst. C.E.*, vol. cix. pp. 117 and 119.

[2] "British Rainfall, 1883," G. J. Symons, pp. 29-32.

[3] "Lectures on Water-Supply," A. R. Binnie, 2nd edition, Chatham, 1887, pp. 10 and 23.

as a spring, or flow into the river draining the basin at a much lower part of the valley. The loss from absorption by vegetation is often much more than compensated for by the protection trees and plants afford against evaporation on flat, compact ground, though grain crops, and especially certain grasses, when growing, absorb considerable quantities of water in hot districts.

The actual flow off a drainage-area, as measured by the discharge of the river draining it, mainly depends upon the nature of the strata forming the basin; for it reaches 75 per cent. of the rainfall where the strata are impermeable and sloping, and falls to only 15 per cent. of the rainfall in the case of very permeable strata.[1] The proportion the flow bears to the rainfall is, moreover, larger in wet districts and in cold weather.

Evaporation.—The influence of evaporation is most strikingly manifested by the gradual lowering of the level of the Caspian Sea, in spite of the flow into it of the waters of the Volga from a basin of 563,000 square miles, as well as the Ural and other rivers, and the absorption of all the discharge from the 750,000 square miles draining into Lake Chad. Evaporation varies with the climate, the season of the year, the state of the weather, the wind, and the nature of the soil on which the rain falls. It attains a maximum during hot, fine weather, with a dry wind, on the surface of a sheet of water, or shortly after rain on a flat, impervious stratum; and it is greater from a surface of water than from the ground in summer, and from the ground in winter. In England, the loss from evaporation has been estimated as equivalent to a depth of from 12 to 20 inches of water in the year from the ground, and about 3 feet from the surface of reservoirs; whereas from reservoirs in India, it has been reckoned as a depth of 4 to 6 feet over the whole area in a year. In the United States, it has been calculated that the evaporation from surfaces of water ranges from a minimum of about 18 inches in a year on the North Pacific Coast, up to a maximum of about 100 inches on the Southern Plateau.[2] The annual evaporation at Melbourne from a water surface has been found to amount to $40\frac{2}{3}$ inches; whilst in South Africa, it is 39 inches at Port Elizabeth on the sea-coast, and at Van Wyk's Vley reservoir, in the interior, it reaches 80 inches.[3]

Evaporation is very slight in cold, damp weather, and, consequently, has far more influence on summer rains than those of winter; and, accordingly, the rainfall in the cold season in temperate regions is of much more importance for water-supply than rain in the warmer months.

[1] "Rivers and Canals," L. F. Vernon-Harcourt, 2nd edition, 1896, p. 13.

[2] "Public Water-Supplies," F. E. Turneaure and H. L. Russell, New York, 1901, pp. 55-59.

[3] *Proc. Inst. C.E.*, vol. xc. pp. 262-263.

This is rendered very evident by the large floods of rivers in temperate climates, except those of glacier origin, always occurring in the cold season (from November to April in the northern hemisphere), even where, as for instance in France, the rainfall is greatest during the warmer half of the year.[1]

Percolation.—Rain falling on a porous soil sinks into the ground, and is thereby removed from the influence of evaporation, unless the plane of saturation approaches within 2 or 3 feet of the surface. The amount of percolation depends upon the porosity of the stratum, and the weather. It is very large through sand, considerable through chalk, less through sandstone, and moderate through limestone; whilst on clayey soil and the denser rocks it is quite small. Percolation does not take place in warm summer weather; whereas it is large in cold weather, and especially with small falls of snow followed by a thaw. Springs and other underground waters are supplied by the percolation of rain, and are replenished during the cold season in temperate regions, when evaporation is inactive.

Flow of Streams and Rivers.—The discharge of watercourses, which constitutes the available rainfall of the basins which they drain, with the exception of any springs flowing straight into the sea, or any water which may be drawn off from underground sources, varies with the conditions which, as already pointed out, affect the flow of the rainfall off the ground. The strata forming the upper portion of the basins of rivers on high ground, are generally impermeable, the fall of the upper river is large, and the rainfall greater than on the lower ground. Accordingly, the flow of streams draining the higher portions of river basins is usually very irregular, the streams rising rapidly in high flood during rainy weather, and running almost dry in dry weather. In the lower part of a river-basin, on the contrary, the ground is commonly somewhat alluvial, and therefore permeable, the fall of the river is reduced, and the discharge being derived from a much larger area, is much more uniform, and less liable to sudden variations from great fluctuations in rainfall usually limited in extent. Rivers, consequently, in the lower part of their course, besides having necessarily a much larger discharge, possess a more regular flow; and even in tropical countries, the main rivers draining large basins subject to varied meteorological conditions, still maintain a discharge in the dry season. Moreover, sometimes rivers rising in mountainous districts with a large rainfall, eventually in their course to the sea traverse almost rainless districts, bringing water to these arid tracts, which would be uninhabitable without them, of which the Nile and the Indus furnish typical instances.

[1] "Rivers and Canals," pp. 10–11, and 150–151, and plate 1, figs. 3–6.

CHAPTER II.

SOURCES OF WATER-SUPPLY.

Sources of water—Direct Collection of Rainwater, method of storing, precautions necessary against pollution, objections to use in towns, suitable for country districts, and for washing—Springs, origin and situations, fluctuations in flow, cause of salubrity of spring-water, substances in solution, medicinal and thermal springs, means of increasing flow, instances of cities and towns supplied by springs, value as supplemental supply—Wells, method of obtaining underground supplies, compared with springs, advantages, water from wells similar to spring-water, outlet provided by artesian wells—Mountain Streams, purity, great irregularity of flow, utilized by storing flood flow by dam across valley—Rivers, common source of water-supply, towns built on banks of rivers for sake of water-supply, exposed to pollution by increase of population and refuse from manufactories, ample supply from rivers in temperate regions, underground flow in sandy river-bed in dry season in tropics, abandonment of Seine and other rivers for domestic supply, only source of supply in some localities, turbid condition in flood-time, settling tanks and filtration essential—Lakes, natural reservoirs, their influence on the inflow, instances of their use as reservoirs and their value, large storage provided by raising water-level, provisions necessitated for intake by pollution of great lakes of North America—Storage Reservoirs, mode of forming, their advantages—Different Sources of Water-Supply compared; value and defects of tanks; subsoil drainage in low sandy districts, with examples; merits and disadvantages of springs; advantages of deep wells over springs; drawbacks of mountain streams and rivers; value and rarity of lakes for reservoirs; importance and availability of storage reservoirs; greater opportunities for choice of source for large cities—Instances of cities supplied from two or more sources.

THOUGH water is primarily derived from rain, which is sometimes stored as it falls, usually a water-supply is obtained from sources above or below the ground, where the rainfall has been collected by natural causes in fairly large quantities. Thus, besides the direct collection of rainfall in the absence of other sources, water may be obtained from mountain streams, the flow of which is generally stored in reservoirs to provide against times of drought, or from rivers in the lower part of their course where they possess a more uniform flow, or from lakes which furnish natural reservoirs in some river valleys. Water also may be collected from springs, which form the outlets of underground flow, or it may be drawn up from wells sunk into water-bearing strata. The volume of water obtainable from each of these sources depends upon the extent of land from which they draw their supply, and the available rainfall over this area.

Direct Collection of Rainwater.—At a distance from rivers or watercourses, in the absence of springs, and where a supply from wells is inadequate or subject to pollution, it is necessary to collect the rain as it falls. This storage is specially necessary in tropical countries where there is a wet and a dry season, so as to obtain during the rains a sufficient supply to last through the dry months. The collection is very easily effected by digging hollows in the ground, and lining them with an impermeable layer where the strata are permeable; but unless these tanks are covered over, the loss from evaporation is somewhat excessive in the dry season in specially dry districts, where the water is most needed. The relative loss from evaporation in an open tank may, indeed, be reduced by increasing the volume collected and its depth by allowing the rainfall over the adjacent ground to flow into the tank; but unless this area is kept perfectly free from pollution, the purity of the water is liable to be thereby impaired.

A large proportion of the rain falling on the roofs of houses and sheds covered with slates or tiles, can be readily stored by leading the water flowing from the gutters, down the rainwater pipes, into a covered tank or cistern of masonry, brickwork, concrete, wood, or metal, preferably placed underground; for though this necessitates pumping up the supply, it maintains the water at a more uniform and suitable temperature, and excludes the light, thereby hindering animal and vegetable growth in the water. The impermeability of slates and compact tiles, and the sharp slope of the roof, reduce loss from evaporation to a minimum, except with slight falls of rain interspersed with sunshine during hot weather; but rain from lead or zinc roofs should not be collected, for fear of contamination. The minimum capacity for these tanks in Great Britain has been estimated at one-fourth of the annual rainfall over the collecting area;[1] and the minimum annual rainfall must be taken as the basis of the supply that can be relied upon, unless sufficient storage is provided to make up for deficient years.

Various precautions are necessary to maintain the purity of a rainwater supply. The first flow of rain off a roof after dry weather collects various impurities in washing the roofs, and should be diverted from the tank by an automatic tipping can, a two-way valve running the water to waste until turned, or other arrangement. Then the water, after going

[1] "Practical Hydraulics," T. Box, 5th edition, p. 80.

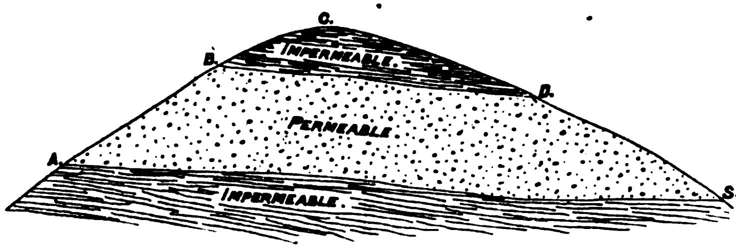
through a strainer, to arrest leaves and other *débris*, should be led into a small settling tank from which it overflows, or passed through a filter, before reaching the main tank. The tank must be perfectly watertight to be secure from infiltrations from the soil; its overflow pipe must be kept away from house drains; it must be cleaned out at least twice a year; and the suction pipe of the pump must be sufficiently raised above the bottom to avoid stirring up sediment at the bottom of the tank by pumping.

Rainwater, being the product of distillation by the sun's heat, is the purest form of water when falling from the clouds, and would therefore be most suitable for domestic use if it could be kept free from pollution. In towns, however, it is subject to contamination even in its descent, by smoke, dust, and impure air; and, consequently, in crowded centres of population, rainwater should, if possible, be only used for washing, as being softer than the water supplied from most other sources. Moreover, in towns, there is much more difficulty in maintaining the purity of the rainwater, both in its collection and during its storage, than in rural districts; and the supply from such a source, especially in places of small rainfall, or subject to long droughts, cannot be made adequate or convenient for the various needs of a town supply. Nevertheless, some towns in a few European countries and in the United States are supplied in this way, of which latter, New Orleans, which is to a great extent supplied from rainwater tanks often made of cypress, placed a few feet above the ground, is a notable instance, and where the purity of the water is in many cases unsatisfactory. The use of rainwater as a source of supply should be confined to country districts and isolated dwellings, where other sources of supply are not available, or too costly; for the precautions necessary to ensure safety are sufficiently onerous to render them liable to be neglected. Rainwater, however, furnishes a very useful supplemental supply of soft water for washing; and whenever it has to be relied upon as the sole supply, boiling, in addition to the precautions detailed above, should secure it from danger as drinking water. Unfortunately, pure rainwater, owing to deficiency in aeration, is less palatable than some waters having a considerably lower standard of purity.

Springs.—A very convenient source for a moderate water-supply is provided by springs, which generally furnish cool and very palatable water of a wholesome quality. Rain percolating through a porous stratum descends till it is arrested by an underlying impermeable stratum, over which it flows to the lowest point of the outcrop, where it emerges at the surface as a spring. The yield of such a spring depends upon the area drained by it, and the rainfall percolating into the ground; and it is greatest during rainy weather in the cold season, and where the permeable stratum constitutes the surface layer throughout its extent. If the permeable stratum is partially capped by an impermeable stratum, Fig. 11, the direct collection of the rainfall is restricted to the surface area of the outcrops AB, DS, which is supplemented by any flow off the impermeable stratum BCD, which reaches the permeable outcrops; and the combined flow from these two sources appears at the lowest point,

SECTION OF STRATA SHOWING SPRING.

Fig. 11.

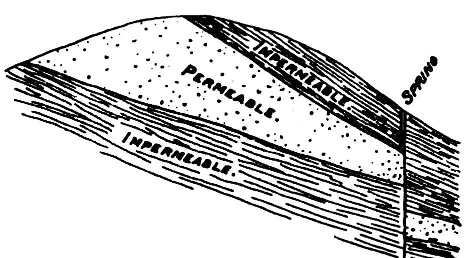


S, of the surface of the underlying impermeable stratum as a spring, sometimes issuing on the slope of a hill or the face of a cliff.

Springs are also found in the bottom of large valleys, resulting from the flow through permeable strata dipping down from a higher elevation, and producing marshy meadows or peaty bogs. Sometimes even, where a permeable stratum is overlaid by an impermeable stratum, a spring

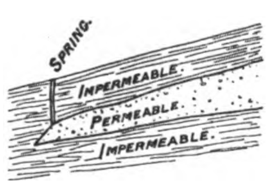
SPRING ISSUING FROM FAULT.

Fig. 12.



SPRING ISSUING THROUGH FISSURE.

Fig. 13.

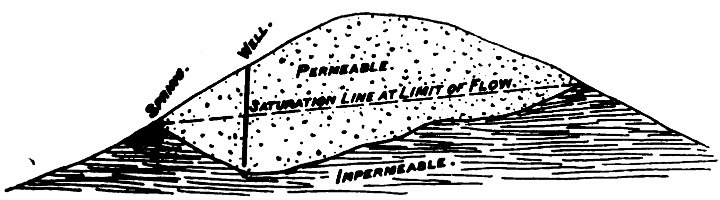


will issue at the surface, on account of the downward flow along the permeable stratum being stopped by a fault, and the water being forced by the head in the permeable stratum along a passage up the line of the fault, through the impermeable stratum, to the surface, Fig. 12; or, occasionally, the water under pressure in the lower part of an underlying permeable stratum, devoid of a natural outlet, finds an exit through a weak point, or a fissure, in the superincumbent impervious stratum, and appears at the surface as a spring, Fig. 13. A remarkable example of a spring resulting from a fault is furnished at Holywell, in Flintshire, where a well-known spring issues at the outcrop of a fault in the coal-measures.

As springs are fed by the rain which percolates through porous or fissured strata, their flow necessarily fluctuates with the rainfall and the seasons, especially where the area of collection is small, and the spring is in its immediate vicinity. When the outcrop of the permeable stratum is considerable in extent, and also at some distance from the spring, the flow of the spring is more regular, and follows less closely on the rainfall, as it is impeded by friction in passing along the stratum. The discharge in temperate regions is generally greatest in the spring, and least in the autumn, owing to the activity of evaporation in the warm season; whereas, when a spring is mainly fed by mountain snows, its maximum flow occurs inhot weather and its minimum in the winter.

INTERMITTENT SPRING, AND WELL.

Fig. 14.



Small springs are liable to cease flowing in dry, hot weather, owing to the gradient line of flow of the plane of saturation, in the absence of replenishment, falling below the level of the outlet, Fig. 14. When, under such conditions, wells have been sunk into the permeable stratum below the level of the outlet of the spring, the reappearance of the intermittent spring, or bourne, after a drought is foreshadowed by the rise of the underground water-level in these wells.

The salubrity in general of spring-water, except when exposed, at a shallow depth, to direct surface contamination, is due to the thorough filtration and oxidation the water has undergone in passing through a considerable thickness of porous soil before issuing as a spring. Spring-water, however, though usually very free from organic matter, often contains inorganic salts and gases in solution, collected in its passage through the permeable strata, and depending on their composition. Sometimes in traversing certain strata, the water becomes so highly impregnated with chalybeate, saline, or sulphurous inorganic compounds, as to be only suited for medicinal purposes, in which respect such springs are often much prized, of which the waters at Bath, Harrogate, Homburg, and various other spas furnish examples. Springs from the chalk, limestone rocks, and the New Red Sandstone, contain a large proportion of calcium salts in solution; but these salts, though rendering the water unsuitable for washing and certain manufactures, do not appear to be at all injurious to health. Thermal springs, which issue from the ground in some localities, indicate by their temperature that the water has come from permeable strata at a considerable depth. The springs at Buxton, in Derbyshire, have a temperature of 82° Fahr.; whilst the three noted springs rising in the valley of the Avon at Bath, and known to the Romans, have temperatures of 104°, 117°, and 120° Fahr. respectively, and discharge about 185,000 gallons a day. The chief constituents in solution in the Bath springs are calcium and sodium sulphates, with moderate amounts of magnesium and sodium chlorides, calcium carbonate, and potassium sulphate, and small quantities of silica, iron carbonate, and potassium nitrate.

Springs furnish a very valuable source of supply for detached houses and villages; and the only objection to their use for the water-supply of towns, is the limited yield that many of them afford. In selecting a spring for water-supply, its minimum flow should be ascertained by measuring its discharge after a period of drought, and making allowance for any excess of rainfall affecting this discharge, over the minimum recorded rainfall of the district. Where the strata are fairly horizontal at the outlet of a spring, the outflow is liable to be spread over a long length, and, consequently, inconvenient for collection; and, under such conditions, it is expedient to concentrate the discharge at a suitable point, either by lowering and enlarging the outlet at this place, so as to draw away the flow from other parts, or by laying pipes or driving a gallery at right angles to the course of the spring, so as to intercept the widespread flow and direct it to the selected outlet.

When the discharge from springs is abundant and fairly regular, it can be utilized for the supply of towns, especially by the aid of works for collecting and concentrating the flow. Thus, a portion of the water-supply of London, provided by the New River Company, is drawn from large springs in the chalk at Amwell and Chadwell, in Hertfordshire, the water from this source having been first brought into London in 1613, by a conduit 40 miles in length. Paris derives a considerable portion of its water-supply from springs in the chalk forming the sources of the Vanne, the Dhuis, and the Avre, tributaries of the Seine; and Michigan obtains its water from springs in drift. The neighbourhood of mountains sometimes enables towns to be supplied by springs issuing along their slopes. Thus Malvern, situated at the base of the Malvern

Hills, is amply provided with a supply of the purest water from springs furnished by these hills; whilst Vienna derives its main water-supply from large springs issuing from the base of a dolomitic limestone formation overlying a stratum of slate, on the slopes of mountains 60 miles distant from the city. Bath is supplied by springs from the Lias rising rapidly on each side of the valley in which the city is situated; and Lancaster obtains its water from springs in the high moorlands of Wyresdale, about 10 miles distant.

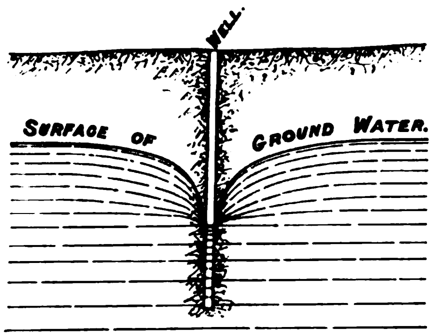
Springs, accordingly, often provide a very valuable source of water-supply, owing to their general freedom from organic impurities and their equable temperature; and even where their flow is too limited to furnish an adequate volume of water for towns, they offer a very serviceable supplemental supply, frequently of more reliable character than other available supplies. The value, moreover, of spring-water would be much enhanced if, when river-water is the only other source of supply, the spring-water could be supplied separately for drinking and cooking purposes, which is a plan so obviously beneficial that it has been often suggested, but has hitherto been generally considered impracticable on account of the cost involved in a double supply.

Wells.—Underground waters which are yielded naturally in the form of springs at definite points, depending upon the arrangement of the strata and the configuration of the ground, may be obtained artificially at a variety of places, by sinking wells into water-bearing strata, and raising the water from the wells to the surface by pumping. These wells possess the advantages of enabling the water to be delivered nearer the places to be supplied, and at a higher elevation, than generally provided by springs; but, on the other hand, the cost of sinking wells is usually considerably more than works for the collection of the water of springs, the pumping involves a constant outlay, and the volume of water obtainable from a well is subject to more uncertainty than the flow of a spring, which can be readily measured. Wells, however, are much more universally applicable for the purposes of water-supply, as well as more convenient in most cases; and though wells sometimes intercept the underground flow on its way to a spring, and thus merely divert a portion of the water to serve another locality, they are often sunk to a considerable depth, so as to reach underground waters which could not otherwise be utilized, having their outlets leading into the lower part of rivers, or through cliffs into the sea, or direct into the sea through orifices in the sea-bottom.

The water derived from wells is similar in its origin and constitution to spring-water; but it is somewhat more liable to pollution, either by the infiltration of the water near the surface into the well, which has to be guarded against by providing a watertight lining through the superficial, permeable strata in sinking the well, or, when the well is sunk in a thick porous stratum reaching to the surface, by the more rapid influx of the superficial waters into the well, resulting from the drawing down of the plane of saturation by pumping, into a form which has been termed a cone of depression, Fig. 15. In true artesian wells, where the hydrostatic pressure, due to the elevation of the water-bearing strata at the sides above the surface of the ground at the mouth of the well,

EFFECT OF PUMPING FROM WELL ON SUBSOIL WATER-LEVEL.

Fig. 15.



causes the water to rise above the top of the well, an artificial outlet is formed in boring the well through impermeable strata, from which the water issues practically as a spring, in the same manner as it finds a natural vent through a fissure in the case illustrated by Fig. 13 (p. 28). Wells, when sunk into depressions in permeable strata, are able to draw for a time, during dry weather, on natural underground reservoirs devoid of an outlet, which are the first to be replenished on the advent of rain, of which a slight indication is furnished in Fig. 14 (p. 29).

Mountain Streams.—Very pure supplies of water may be obtained from streams draining uninhabited and uncultivated mountainous districts, which receive the rainfall flowing rapidly down from sloping, impermeable strata, with little loss from evaporation, and very much in the condition in which it has fallen from the clouds, with occasionally a slight discoloration from peat often met with in such districts. Though, however, the rainfall is greater in the upper parts of river-basins, where these streams are found, than over the rest of the basin, the gathering ground is far too limited to provide a large flow; and, moreover, the absence of trees, the sparseness of the vegetation, and the impermeability of the strata in these regions, combined with the declivity of the land and the rapid fall of the valley through which the stream flows, renders the discharge very irregular, forming a rushing torrent in wet, winter weather, and falling almost to nothing in the fine summer months. Accordingly, this source of supply, so valuable on account of its excellent quality, is almost useless in its natural condition, owing to its great fluctuations; and it is only by storing up the flood discharge of these mountain streams in a reservoir, formed by constructing a dam across a narrow part of the valley, that their waters can be rendered available for the supply of towns situated in a lower part of the basin, or sometimes in an adjacent watershed.

Rivers.—Next to springs, the earliest, most obvious, and very common source of water-supply is furnished by rivers, which in the lower portion of their course, owing to their draining extensive areas subject to differences in their meteorological conditions, have generally a large and fairly regular flow, in comparison with their discharge in the upper part of their basins. The selection of sites alongside rivers for towns in olden times, has been commonly attributed to the desire of their founders to secure an ample supply of water; though possibly, in a minor degree, facilities of communication by water, in the absence of roads, may have also guided their choice. In early days, when countries were inhabited by a scanty population, and by communities in towns long distances apart, springs and shallow wells in rural districts, and the rivers flowing past the towns, furnished an abundant supply of water for the wants of the inhabitants, of an adequate standard of purity. With the growth, however, of population, and the multiplication of towns and villages along the banks of the rivers, together, in some cases, with the establishment of manufactories affording special sources of pollution in the discharge of their refuse, some rivers have had to be abandoned as sources of supply; the intakes from others have had to be shifted to sites higher up less subject to contamination, as in the case of the supply of London from the Thames in the middle of the last century; stringent regulations have had to be enforced against the pollution of rivers by sewage, manufacturing refuse, or other preventible causes; and most river-waters flowing through inhabited and cultivated districts, require to be purified by filtration, or other methods, before they can be regarded as suitable for domestic supply.

Rivers in their lower course, in temperate regions, generally furnish an ample volume of water for the requirements of towns situated on their banks. London, indeed, absorbs the whole summer flow of the Lea, and so large a proportion of the summer flow of the Thames, that the Water Companies have found it expedient to store some of the winter flow of the river in reservoirs formed by embankments on its banks; but this is an exceptional instance of the largest city in the world, mainly supplied by a river with quite a small basin in comparison with the large rivers of the Continent, and still more of Asia and America, over which there is only quite a moderate rainfall. In tropical countries, some rivers flowing in a sandy bed become apparently dried up in the dry season; but these rivers usually have an underground flow which can be reached by sinking wells into the porous stratum.

The liability of rivers flowing through populous and agricultural districts to pollution, and the tendency of the contamination to increase with the growth of population, have led in some instances to underground supplies being substituted for river-waters. The River Seine, in the neighbourhood of Paris, has had to be abandoned as a source of supply, except for municipal purposes; and recourse has had to be made to the sources of its purest tributaries, rising from springs in the chalk, for the domestic water-supply of the city. In view, also, of the gradually increasing pollution of rivers in the United States, many cities which have been conveniently supplied with river-water in the past, have found it expedient to seek for safer sources in underground waters. Many towns, however, are so situated that the river which flows past them affords the only adequate, reliable source of supply at any reasonable cost, of which Calcutta, on the banks of the River Húgli, in the delta of the Ganges, is a typical instance. London, also, under existing conditions, could not dispense with its supply from the Thames; though, in this case, proposals have been made from time to time to obtain a much purer supply off gathering grounds in hilly districts, collected in impounding reservoirs; and some of the sources formerly proposed have been now appropriated by other cities. Where rivers constitute the only available source of supply, very efficient methods of purification need to be adopted to render the waters unobjectionable for domestic use. Rivers also, when in flood, are generally very turbid, and polluted with organic matters washed down off the land; but objections to the use of their waters under those circumstances, can usually be obviated by avoiding the drawing off of the water when in its most turbid state, and by providing settling tanks and an additional area of filter-beds when the flood-waters have to be utilized, in the absence of a sufficient reservoir capacity to tide over the period of high flood.

In spite of its exposure to pollution, river-water affords in many cases an indispensable source of water-supply for large towns unfavourably situated for supplies from other sources; and considering that filtration of water drawn from a river is regarded as essential in Great Britain, it is satisfactory to note that in the United States, as reckoned by the typhoid death-rate, filtered water has been placed second on the list for purity next after mountain springs, and in front of underground waters, impounded waters, and waters from the large inland lakes.[1] It

[1] "Water and Public Health," J. H. Fuertes.

must, however, be borne in mind that the filtering of water-supplies is somewhat exceptional in the United Sates, and that, therefore, filtered river-water occupies a higher position than it would do if the purification of other sources of supply was more general.

Lakes.—Natural reservoirs are provided by lakes, formed generally by a depression in a mountain valley through which a river flows, in which the water is retained by a ridge of rock across the valley at its lower end, and over which it has to rise before the river flowing in at the upper end can continue its course down the valley below. The lake, in regulating the flow, stores up to some extent over its large area the flood discharge of the river above; and it also acts as an immense settling basin, in which all the sediment brought down by the river is gradually deposited as the current is checked on entering the lake. A notable example of this result is furnished by the River Rhone, which enters the Lake of Geneva as a very muddy, glacier-fed river, and emerges at Geneva as pure and blue as the waters of the lake. The value of lakes as storage reservoirs depends upon the discharge of the river flowing into them, together with the flow off their own gathering ground, and the freedom of the drainage-area and the shores of the lake from sources of pollution. When the lakes are situated in mountainous districts, like Loch Katrine and Thirlmere, they provide excellent reservoirs of the purest water, secured by their extent and depth from any prospect of being silted up within any measurable period; and these lakes have been utilized with great advantage for the supply of Glasgow and Manchester respectively. Moreover, lakes so situated are generally at a sufficient elevation above the towns to be supplied, that the water can be conveyed to them by the action of gravity; and in the event of any pollution, which is rare in such districts, exposure to the air and the influence of aquatic plants and fishes tend to purify the waters of a lake. New Westminster, in British Columbia, drawing its waters from Lake Coquitlam, furnishes an example of a similar source of supply in North America. This lake, surrounded by mountains and fed from snowfields, situated 436 feet above high water in the sea, and 13½ miles distant from the service reservoir along the line of the pipes, is capable of providing an unlimited supply of the purest water to this growing town established on the banks of the Fraser River about 15 miles from its mouth.[1]

Lakes, besides providing generally ample space for any silt which the streams flowing into them may bring down, possess the great advantage that, owing to the extent of their water-surface, a moderate raising of their water-level by a low dam across their outlet, furnishes

[1] "New Westminster Waterworks," A. E. B. Hill, *Proc. Inst. C.E.*, vol. cxxii. p. 193.

a large storage of water in cases where it may be undesirable to lower materially the water-level of the lake for supplying a town with water.

The Great Lakes of North America, which really constitute inland seas of fresh water, are subject to different conditions from the lakes in mountainous regions referred to above. They, indeed, supply an ample volume of fresh water for the large towns situated on their shores; but these towns, at the same time, furnish sources of pollution by discharging their sewage and refuse into the lakes from which they draw their water-supply. For example, Chicago so polluted Lake Michigan in its vicinity, that the intake for the water had to be carried two miles into the lake by a tunnel under the bed of the lake, connected with the lake by a shaft at its outer end; and Cleveland had to resort to a similar expedient for obtaining pure water from Lake Erie. As the towns established on these lakes are above the water-level of the source of supply, pumping has to be resorted to for the distribution of the water, instead of the gravitation system by which the water can be conveyed from lakes in the mountains.

Storage Reservoirs.—Pure mountain streams may be utilized as a source of water-supply for towns, if their surplus discharge in flood-time is stored up for affording a constant supply through the dry season, by forming an artificial lake, or storage reservoir, by erecting a dam across a narrow part of the valley of the stream where there is a widening out of the valley higher up. This arrangement, moreover, does not only store up the water which would otherwise run to waste, but it also has the advantages of regulating to some extent the floods of those streams by receiving them into the reservoir in filling it, and of enabling the dry-weather flow to be augmented, if desired, by discharging some of the stored-up water in the reservoir retained by the dam, into the stream below during its period of minimum flow. These impounding reservoirs, being situated in the mountain valleys at the higher part of a river-basin, are generally at a considerable elevation above the town to be supplied by them, so that the water can be readily conveyed by gravitation from the reservoir to its destination.

Different Sources of Water-Supply compared.—The direct collection of the rainfall in tanks or cisterns at the place where the water is required, furnishes the simplest, and in some cases the only available, method of supply. The water, however, is insipid, great caution has to be exercised to avoid contamination, and the area of supply is very limited; and, therefore, this source of supply should only be resorted to when no other means of supply can be obtained at any reasonable cost, or as supplementing a more reliable supply for special purposes. Nevertheless, it sometimes proves a valuable source of water for detached houses, in the absence of other sources.

Another means of supply imposed by necessity, and occupying a sort of intermediate position between the direct storage of rainwater and springs, consists in the collection of rainwater falling on sandy dunes and downs, by deep subsoil drainage. This is the system which has been adopted for the supply of some towns in the flat, low-lying districts of Holland, where the common means of supply are not available, such, for instance, as Amsterdam and The Hague. The water-supply of Amsterdam is obtained by collecting the rain percolating into the sandhills near Haarlem by underground drains; whilst The Hague derives its water from the rainfall over the sandy downs along the sea-coast, which is led away by deep subsoil drains. This method of collecting the rainfall avoids the contamination generally involved in gathering the flow off roofs; whilst its percolation through the sand secures it from evaporation, and frees it from surface impurities; and a larger and purer area is obtained as a gathering ground.

Springs issuing from hilly slopes furnish generally excellent water, the only possible drawbacks to which are that their existence is dependent on definite geological conditions, that their flow is often moderate and irregular, and that they almost always contain some inorganic salts in solution. The water is usually very palatable and uniform in temperature, and is far superior to rainwater for drinking; and its filtration through the porous stratum from which it issues, frees it from organic impurities.

Deep wells, drawing their water from the same underground sources as springs, furnish the same good quality of supply; whilst being less subject to special local conditions, they are able to provide water nearer the locality to be supplied than springs, and in a much greater number of places. Moreover, they sometimes utilize underground waters which otherwise would be wasted by discharging direct into the sea. Accordingly, wells provide a better supply of water than rainwater, and a larger and more convenient supply, in most cases, than springs.

Mountain streams without storage afford a good, but irregular supply of water, inadequate for towns; whereas large rivers furnish an abundant and fairly regular source of water, but subject more or less to pollution, and necessitating careful filtration to ensure immunity from danger.

Lakes furnish large natural reservoirs very suitable in their original condition for supplying water on a large scale; but very few towns are so situated as to have these exceptional natural facilities afforded them. Moreover, unless draining hilly, uninhabited, and uncultivated districts, they are exposed to the same sources of pollution as rivers, especially when large towns are established on their shores.

Storage reservoirs in the primitive valleys of mountain streams, provide excellent sources of water-supply for cities and large towns situated within reach of mountainous districts. They afford a much more commonly available means of storing up water than lakes, though necessarily less readily in large volumes, and at a greater cost; and they require a much more careful inspection of the site, to make certain of its impermeability, and the solidity of the foundations for a dam. The water thus stored is of the same unexceptionable quality as that of lakes in hilly regions, and can generally be more easily preserved from pollution. In fact, of all the sources of supply, water stored in a lake surrounded by hills, or in a mountain valley, is the most satisfactory, both on account of the certainty with which the volume capable of storage can be ascertained, and, consequently, the amount available for distribution in the most unfavourable years, and the exceptional purity of all the water from such a source.

The choice of source for water-supply must depend upon the physical and geological conditions of the country surrounding the place to be supplied, and the volume required. Small communities must rest satisfied with the sources of supply within easy reach; whereas where large and rapidly increasing cities have to make provision for the future, and especially where rivers are not available, it becomes expedient to obtain, if possible, an ample supply by forming impounding reservoirs in mountain valleys, often at a considerable distance from the districts to be supplied. Sometimes large towns, whilst drawing their main supply from a particular source, supplement their supply from other available sources which may be more convenient for special districts. Thus London supplements its river supply from springs and wells in the Chalk; Liverpool, though now mainly supplied by impounding reservoirs, still draws a considerable amount of water from wells in the New Red Sandstone; and Birmingham obtains part of its water-supply from similar sources. Paris also, which draws its domestic supply from springs in the Chalk, obtains water for municipal purposes from the Seine and the Marne, and a canal from the river Ourcq, and gets some water from artesian wells;[1] Vienna, which derives its chief supply from springs at a distance, conveyed by gravitation, draws also some water from wells in the Limestone; and Greater New York, though mainly provided with water from storage reservoirs, derives some of its supply from underground waters reached by wells sunk in Long Island and Staten Island, which partly supply Brooklyn, and furnish the entire water-supply of Queens and Richmond.[2]

[1] "Distributions d'Eau: Assainissement," G. Bechmann, Paris, p. 502.

[2] "The Water-Supply of the City of New York," The Merchants' Association of New York, 1900, p. 56

CHAPTER III.

WELLS.

Object of wells, and conditions affecting the supply from them—Water-bearing Strata, varieties, drift and alluvium, chalk, sandstones, lime stones, conditions affecting influx of rainfall—Classification of Wells—**Shallow Wells**: Construction, methods described; Supply of Water, source of supply, conditions affecting supply, in river valleys; Examples of Large Wells, early use in London, instances in United States, instances in river valleys; Precautions necessary, objections to shallow wells, protection from pollution; Remarks on Wells in Surface Strata, conditions affecting them, their scope—**Driven Tube-Wells**: Description, construction, methods of driving, dimensions, use of water-jet; Provisions necessary in Driving, clearance of tube, exclusion of fine sand; Supply of Water, pumping, depth attained, conjunction of several tube-wells; Uses of Tube-Wells, for rapid supply of water, testing water-bearing strata, for obtaining water in ordinary wells from greater depths—Concluding Remarks: defects and advantages of shallow wells, limits and value of tube-wells.

WELLS provide a means of tapping the underground waters contained in, or flowing along, porous strata overlying impermeable strata, by being sunk at suitable spots into the permeable strata. The volume of water that can be thus obtained depends primarily upon the rainfall, and the extent of the outcrop, together with that of any land draining to it, and also in some measure on the thickness, configuration, and nature of the permeable stratum. A thick porous stratum, with a correspondingly wide outcrop, affords a considerable storage capacity for the rainfall percolating into it; and the available storage is increased when the stratum dips down in the central portion so as to form a sort of basin, Fig. 14 (p. 29), and the underground waters are thereby to some extent prevented from escaping at the lowest part of the downstream outcrop. When a stratum is very loose and porous, such as sand and gravel, its interstices provide a considerable space for holding water, which it also readily yields up; whereas more compact strata, as for instance the Chalk and the Oolites, though absorbing large volumes of water in their pores and interstices, do not part with it readily, and the water is chiefly obtained from the flow through their fissures.

Water-bearing Strata.—Permeable strata are found at very different and variable depths; for in some places the surface layer consists of recent deposits, and at other parts, owing to denudation or geological disturbances, older strata, and even primitive rocks, appear at the surface. Igneous rocks and fissured unstratified strata do not afford facilities for the storage of water; but in the Magnesian Limestone and Lower Red Sandstone, constituting the upper portion of the primary series of rocks, large quantities of water are often found. In the secondary and tertiary formations, the permeable strata are interspersed with impermeable strata, which occasion the retention of the water percolating through the outcrop into the permeable strata overlying them.

Drift, consisting of the *débris* of rocks carried down and deposited by flowing water in valleys and depressions in the ground, and sometimes on the lower slopes of hills, having been washed down by rain from the higher ground, is very irregular in thickness, and often discontinuous. The porosity of the drift depends on the nature of the materials of which it is composed, which are usually gravel and sand, but sometimes consist of less permeable materials brought down from the adjacent hills. Alluvial deposits are very similar in their origin to drift, but they are more regular and extensive; they are usually composed of materials brought from a greater distance, often filling up ancient lakes and river-beds; and they consist mainly of sand, gravel, and stones, together with clays and marls. Sometimes these permeable strata form the surface layer, and receive their supply of water by the direct percolation of the rainfall; but they are often partially overlaid by an impervious stratum, under which the ground-water flows for considerable distances. Sand furnishes the most porous stratum, being capable of absorbing from one-third to nearly one-half its volume of water; whilst gravel and sand can contain from one-quarter to three-tenths their volume of water. Numerous wells have been sunk into these upper permeable strata, for supplying water to large towns in the United States.[1]

The Chalk is the principal water-bearing stratum for a considerable part of the southern portion of England, with its good thickness and large outcrop, absorbing almost 30 per cent. of its volume of water on the average; whilst the Greensands furnish large volumes of water, more uniformly distributed throughout them than in the Chalk; and both these formations yield good supplies to wells sunk into them. The New Red Sandstone, or Trias, though less extensive in area in England than the two above-mentioned strata, traverses the more rainy western districts, stretching from the Channel on the south coast of

[1] "Water-Supply Engineering," A. Prescott Folwell, New York, 1900, p. 147.

Devonshire to the Solway Firth, and therefore may be regarded as quite as suitable for wells.[1] Moreover, although wells have to be sunk to a considerable depth in the New Red Sandstone to reach water, the volume is abundant when found, and is less hard than water from the Chalk. This stratum, known as Trias abroad, extends over considerable areas in Europe, and also for long distances in North America. Other sandstones yield large quantities of water, proportionate to their extent, outcrop, available rainfall on them, and porosity, which ranges from at least 28 to 7 per cent. in volume in the sandstones of the United States,[2] according to their compactness, the porous Potsdam and St. Peter sandstones having been largely resorted to for deriving water-supplies from wells. Water is also drawn from wells sunk in the Oolitic, Lias, and Magnesian Limestones, both in England and North America, but not with the same certainty and facility as from sandstones, since limestones only yield water when extensively fissured, and the underground flow is liable to be obstructed by faults.

The absorption of rainfall by stratified, water-bearing strata at their outcrop, is largely affected by their dip, their freedom from a surface covering of an impermeable nature, and the flatness or depression of the ground. A considerable dip facilitates the descent of the water into the stratum along the interstices between the successive layers, but if continued for some distance, causes the stratum to descend to too great a depth below the surface. The inflow of the rain is dependent on the permeable outcrop being free from obstruction at the surface by an impermeable layer of overlying drift; and the rain is adequately retained for percolating into the porous stratum when falling on fairly flat ground, and still more on a valley or depression; whereas it would be liable to flow away down a steep slope, and be to a great extent lost to the permeable stratum.

Classification of Wells.—There are three types of wells employed for water-supply, namely (1) Shallow Wells, sunk into permeable strata a short distance below the ground; (2) Tube-Wells, driven short or long distances into the ground, according to circumstances; and (3) Deep Wells, often carried down by boring to considerable depths into underlying, permeable strata. Shallow wells obtain their water from superficial strata, so that the supply from them is limited in quantity. Tube-wells derive their supply from superficial, or underlying strata, according to the geological conditions of the locality; and they are specially valuable for obtaining water rapidly in unexplored or arid

[1] "Water-Supply: The Present Practice of Sinking and Boring Wells," Ernest Spon, 2nd edition, p. 39.

[2] "Public Water-Supplies," F. E. Turneaure and H. L. Russell, New York, 1901, p. 87.

regions. Deep wells enable supplies of water to be drawn from strata lying at a considerable depth below the surface, and extending often over considerable areas. When a deep well sunk in a valley pierces a permeable stratum underlying an impermeable stratum, and this permeable stratum rises at the sides considerably above the level of the top of the well, the water rises in the well, and overflows, owing to the hydrostatic pressure, and it is then termed an Artesian well; whereas, under ordinary conditions, the water has to be lifted out of the well, often from a considerable depth, by pumping.

SHALLOW WELLS.

Construction of Shallow Wells.—In sinking wells to moderate depths in soft soil, the wells are generally made from about 4 feet upwards in diameter, and are lined with brickwork or masonry, as wood is liable to decay and pollute the water. A well can be sunk down by building up the brick or masonry lining on an annular curb of wood or iron, laid on the ground and corresponding exactly to the section of the lining, occasionally surmounted round its outer side by a cylindrical sheathing, or drum, to protect the lower length of the lining, and facilitate its maintenance in a vertical position during its descent. The excavation for the well is carried down as far as the sides will stand vertically, before the curb is laid on the bottom, or the drum introduced with a cutting edge underneath round the outside; and the sinking is effected by weighting the curb with the lining, and continuing the excavation below; the building up of the lining being made to correspond with the sinking, which has to be conducted with great uniformity to prevent the lining getting out of plumb, and, consequently, jamming. Wells are also often constructed by underpinning, after the first length of the lining has been built up on a curb placed at the bottom of the excavation. In this case, the excavation is carried down a further distance, depending upon the nature of the ground, and formed so as to correspond with the internal section of the well; and by this arrangement the lining above is supported on a bench of earthwork, which is then partially removed in narrow sections, and timber props inserted, or portions of the lining built up in the recesses thus formed, so as to support the upper length of lining during the completion of the lining below. Sometimes the curb on which the upper part of the lining has been built up, instead of being propped up from below, is supported by iron rods fastened above to timber balks stretching across the mouth of the well. When the lining is sunk down gradually by its own weight as the excavation underneath it proceeds, the increasing friction of the prolongated lining against the earth at the sides, tends eventually to counteract the weight of the lining, and to arrest the descent of the well; and the earth-bound well, if injecting water round the outside does not release it, has to be prolonged to the requisite depth, either by sinking inside it a second well of reduced diameter from the bottom of the original well, or by resorting to the process of underpinning. Sometimes large shallow wells are constructed by excavating a circular timbered trench, in which the lining is built up; and the central core is subsequently removed.

The excavation for wells of moderate depth, deriving generally their supply of water from superficial, permeable strata, is usually effected without difficulty; but when hard rock has to be traversed, holes have to be drilled, in which charges of explosive substances, such as blasting powder, guncotton, tonite, or rackarock, are inserted, which, when fired, shatter the rock. Water, however, flows into the excavations in porous strata directly the level of saturation of the stratum has been passed, impeding the deepening of the well; and pumping has to be employed; or by lining the well with iron cylinders, the excavation in soft soil can be continued under water by a grab, or by a mizer consisting of an iron cylinder closed at the bottom but having an opening at the side, with a cutting lip, into which the material is forced on turning the cylinder round, Fig. 45 (p. 76). Occasionally, excavation for a well through a layer of quicksand, has been accomplished by congealing the soil by sinking pipes round the site, in which a very cold, unfreezable liquid is kept in circulation, or liquid ammonia is introduced, which in evaporating produces an intense cold. By this process, the influx of sand with the water into the excavations, and the resulting subsidence of the adjacent land, are obviated; but the system is necessarily so costly as only to be applicable to exceptional cases.

Brickwork is generally used for the lining of shallow wells, as being better suited for the circular form commonly adopted than masonry. Through stiff soil, a lining only half a brick thick is often employed for small wells, and a whole-brick lining for larger wells, laid sometimes dry, but preferably in cement, where greater strength is requisite, or infiltration has to be provided against. A well can be further secured from the influx of impure surface water, by backing the brick lining with clay puddle, or with concrete, which adds strength to the lining and is more impervious than clay. When, however, a well is exposed to the influx of impure water under pressure, it is advisable to line it with iron cylinders, in order to render it perfectly watertight at the sides, as infiltration is liable to take place through a good thickness of brickwork when there is a considerable head of water at the back. Moreover, iron cylinders, besides possessing the advantages of impermeability, strength, and durability, can be readily sunk through soft, water-bearing strata, without having to resort to pumping in cases where it is expedient to dispense with it.

Supply of Water to Shallow Wells.—The water drawn from shallow wells is derived from the rainfall percolating through superficial, permeable strata, and arrested by underlying impervious beds. Consequently, the determination of the general slope of the subjacent impermeable stratum, and of the extent of the porous surface stratum, and an estimate of the available rainfall on the porous stratum, enable a fair idea to be formed of the most suitable sites for sinking wells into the upper stratum, and their probable yield. The wells have to be sunk below the level of the plane of saturation at the driest period, to ensure a supply; and the water flows into the well at the bottom, and also through the lower part of the lining where the bricks have been laid dry with this object. A large volume of water may be obtained from permeable strata extending over a considerable area, and especially near the lowest depression of the underlying, impervious stratum, to which the water flows from the higher levels. On the contrary, as the supply depends on the amount of rain percolating through the superficial stratum, the water in wells sunk into such strata of very limited extent, is liable to fail in times of drought, as in the case of springs, and particularly near the higher parts of the impermeable stratum receiving only the rain descending into the surface layer directly over them. Additional wells also, sunk into a porous stratum of restricted area, are liable to diminish the yield of existing wells, owing to the strictly limited volume of the supply, and to lower the general level of the plane of saturation, which may necessitate the deepening of the shallower wells. Large wells only slightly increase the flow of water into them when the water-level is lowered by pumping, as compared with small wells, in consequence of their larger extent; but they are serviceable in increasing the amount of storage, and in facilitating the establishment of the pumps at a low level inside them when requisite.

Wells have often been sunk in valleys in the neighbourhood of the river draining the valley, when the river-water itself is liable to pollution, or otherwise unsuited for domestic supply, and in some cases, in tropical countries, when the river entirely fails in the dry season. These wells do not generally draw their supply from water percolating into them from the river; but they intercept the underground flow, which is often considerable through porous soil in these low districts, and which is either on its way to feed the river in the form of springs along the bed and banks, or flows parallel to the river as an independent, hidden stream.[1] The absence of connection between the water from these

[1] "Underground Water-Supply," B. Salbach, *Trans. Am. Soc. C.E.*, vol. xxx p. 294.

wells and the river, has been proved, in some cases, by the difference between the well-water and the river-water exhibited by chemical analysis; but it is important not to draw down the water very low in wells in near proximity to a river, when it is expedient to avoid any infiltration of water from the river. Sometimes for augmenting the supply, instead of sinking additional wells, galleries are extended from the bottom of the well, across the line of underground flow, to intercept a larger volume of water, as in the case of springs.

Examples of Large Wells.—In former times, shallow wells were extensively used in London for obtaining water from the gravel beds overlying the London Clay; and until a general distribution of water was established, the extension of the unsupplied districts of the metropolis was restricted to those parts where beds of gravel exist, owing to the impracticability of procuring water readily where the clay rises to the surface. Shallow wells are employed in many country districts of Great Britain for villages where no general water-supply has been provided, and for isolated houses where no other means of obtaining water are available except the direct collection of rainfall.

Several large towns in the United States are supplied with water from large wells, sunk often into drift, but sometimes into other formations. Thus, for instance, Des Moines is supplied by a large well, and Lincoln in Nebraska by large wells in drift; whilst Springfield, Illinois, and Columbus, Ohio, derive their supply from wells and filtration galleries in drift, and Beloit, Wisconsin, from a large well in Trenton rock.[1]

Many towns on the Continent, situated on rivers, draw their water-supply from wells sunk in the river valley, intercepting the underground flow through the alluvium, of which Halle, Leipzig, Dresden, Cologne, Düsseldorf, and Bonn furnish notable examples.[2] A large well also, sunk in a deposit of gravel which occupies the ancient bed of the Illinois River, supplies Peoria, Illinois, with water. The resort, moreover, to the underflow of rivers for the supply of towns is exemplified by Nashville, Tennessee, which by means of a filtration gallery draws its supply of water from the underground flow of the Cumberland River, and by Winona, Minnesota, which taps the underflow of the Mississippi through two large wells. An example of a water-supply derived from the perennial flow under the bed of a tropical river, which becomes dry in the dry season, is furnished by Trichinopoly, where several wells, sunk 25 feet below the bed of the river Cauveri, reach a stratum of sand, through which a flow of water is always found, though the river is practically dry for five months of the year.[3]

[1] "Water-Supply Engineering," A. Prescott Folwell, p. 147.

[2] "Underground Water-Supply," B. Salbach, *Trans. Am. Soc. C.E.*, vol. xxx. pp. 295, 301, 304, and 309.

[3] *Proc. Inst. C.E.*, vol. cxxxvii. p. 17, plate 1, figs. 2 and 3

Precautions necessary with Shallow Wells.—The great objection to drawing water for domestic supplies from shallow wells, is the danger to which the water is often exposed of pollution from surface impurities, or the proximity of cesspools. Surface water should generally be excluded by raising the well above the ground-level, and by making the lining watertight for some depth below the surface. Cesspools must be absolutely prohibited within the neighbourhood of a shallow well; and no existing shallow well should be made use of for domestic purposes, into which there is the slightest chance of infiltration from cesspools, or of impurities from the surface. Wells, however, sunk into superficial permeable strata, which are duly guarded from sources of pollution, especially in sparsely populated districts, serve the useful purpose of furnishing water which has been filtered naturally. In towns and populous places, such wells should be abandoned, owing to the impossibility of securing them satisfactorily from contamination; and they should only be used for supplying towns when they are situated at some distance off, and draw their water from pure sources quite away from the town, and capable of being preserved from injury. Wells should be covered over so as to exclude the light, and thus check vegetable growth, and also to guard against chance contamination.

Remarks on Wells sunk in Surface Strata.—Undoubtedly, wells drawing their supply from water percolating through superficial strata, must be resorted to with great caution; but at the same time, under favourable conditions and with due care, they are capable of affording satisfactory supplies. Their yield necessarily depends upon the extent of the stratum into which they are sunk, and the local conditions; whilst the purity of the water obtained is determined by the depth or distance through which it has passed before reaching the well, the nature of the stratum traversed, and its general freedom from contaminating influences. Water drawn from some depth in a thick, permeable stratum furnishing a good filtering medium, and water flowing for a considerable distance underground through an alluvial bed, are subjected to valuable purifying influences from organic pollution. Moreover, water flowing underground through an alluvial deposit, in the depression of a river valley, or below the river-bed, often provides a plentiful supply of water to wells, which having been filtered in its flow, and being independent of the river and protected from evaporation, may be decidedly superior in quality to the water of the river, and also continue to flow even in cases where the river fails in the dry season.

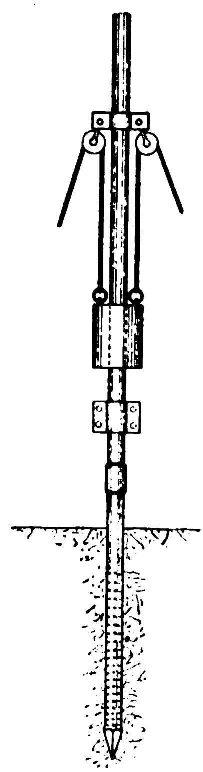
Driven Tube-Wells.

Small supplies of water may often be rapidly obtained, in soft soils, by driving iron tubes, provided with a pointed end, down to water-bearing strata at a moderate depth below the surface.

Description of Tube-Wells.—A stout, wrought-iron pipe, with a wrought-iron or steel point at the bottom, and perforated for the first 2 or 3 feet above the point, constitutes the bottom length of a tube-well. This pipe is put down into a vertical hole carefully formed at the desired spot by a crowbar; and a cap having been fitted on at the top, the pipe, in the case of small tube-wells, is driven down by the blows of a hammer. As soon as the top of the tube has been driven down to within a few inches of the ground, the cap is removed, and another length of tube screwed on; and the cap is then placed on the top of this tube, and the driving recommenced, the same process being repeated until the water-bearing stratum is reached. Great care must be taken to drive the first 2 or 3 feet of the lowest pointed length of pipe perfectly vertically, to ensure the well being sunk plumb.

DRIVING TUBE-WELL WITH CLAMPS.

Fig. 16.



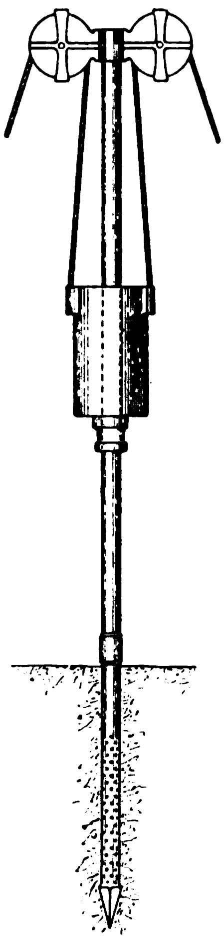
The larger tubes have to be driven down by a falling weight, or ram, as in pile-driving. A simple system, very commonly adopted, consists in making the upper part of the tubes serve as a guide for the falling weight, by constructing the weight so as to encircle the tube, and causing this weight to drive down the tubes by falling on a clamp fastened round the lower part of the tube. The weight is raised again, after delivering a blow, by ropes running round pulleys supported by an upper clamp, Fig. 16. In this arrangement, however, the clamps are liable to make indentations in the tubes, sufficient sometimes to impede the screwing on of additional lengths. This source of injury has been obviated by attaching a vertical rod on the top of a driving cap, to serve as a guide for the ram, instead of an additional length of tube;[1] and, moreover, this contrivance enables the ram to deal more direct blows on the head of the tube, through the intervention of the cap, Fig. 17.

[1] "Well-Boring for Water, Brine, and Oil," C. Isler, p. 29.

The strong, wrought-iron tubes used for these wells are from 1 inch up to 8 inches internal diameter, made in lengths of from about 3 to 10 feet. As the difficulty in driving these tube-wells increases with their circumference, it has sometimes been considered advisable not to use tubes exceeding about 2½ inches in diameter; and the choice of size must be guided by the appliances available for driving, the nature of the strata to be traversed, and the rapidity with which it is expedient to obtain the supply. Though the size of the tube does not proportionately affect the yield, a certain area of perforated surface is necessary to prevent a velocity of influx liable to bring sand into the well, so that the diameter selected for a tube-well should depend somewhat on the thickness of the water-bearing stratum. The main advantage of a large tube consists in the reduction of friction on the stream of water drawn up.

DRIVING TUBE-WELL WITH CAP AND ROD.

Fig. 17.



Sometimes the bottom length of a tube-well is formed of an open tube with a steel shoe at its base; and its driving is assisted by forcing water down a small pipe inside it, which loosens, and partially removes the material round the base of the tube—a process similar in principle to the use of the water-jet in pile-driving.

Provisions necessary in driving Tube-Wells.—Some of the finer soil finds its way into the tube-well through the perforations in the process of driving, and, if allowed to accumulate, would prevent the influx of water into the well on reaching a water-bearing stratum. This material must, therefore, be periodically removed from the well during its descent. The clearance is generally effected by lowering a small pipe down the well, keeping its orifice slightly above the surface of the deposit to avoid its becoming choked, water being at the same time poured down the well; and then, after fixing a pump at the top of the small pipe, the deposit is drawn up in suspension in the water by pumping, and thus withdrawn from the well. When very fine sand is traversed, it becomes necessary to cover the perforations of the tube-well with a brass strainer having holes proportioned to the fineness of the sand to be dealt with.

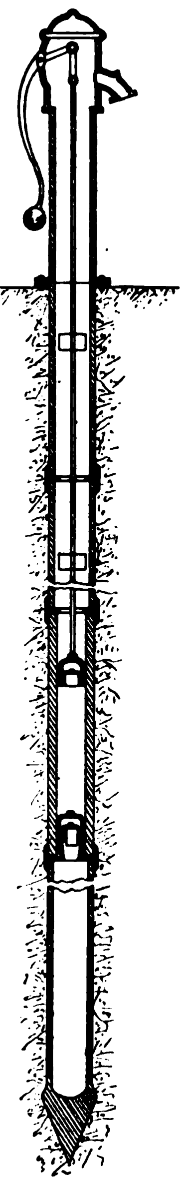
Occasionally, when quicksand is encountered, the water pumped up from a tube-well remains turbid with the finest sand in suspension. In such cases, it has proved necessary to carry down an open tube into the quicksand, and to fill rapidly with gravel the hole formed by raising or pumping up the sand; and then, on driving a small tube-well into this gravel bed, clear water is obtained.

The removal of the material which finds its way into the tube, when driven some depth into the ground, is accomplished by fastening a cylindrical shell, with a valve, to the bottom of the small clearance pipe, which, when moved up and down in the deposit in the tube, forces the material into the pipe, and thus enables it to be removed from the tube.

Supply of Water from Tube-Wells.—As soon as a tube-well has been driven a few feet into a water-bearing stratum, a pump is attached at the top, provided the depth of the water-level below the surface does not exceed about 28 feet; and then, by working the pump vigorously for some time, the small particles in the neighbourhood of the perforated tube are withdrawn, and the water becomes clear. When the underground water-level is at a greater depth, the barrel of the pump has to be placed low enough down in the tube to be within the limit of suction from the anticipated water-level after allowance for lowering by pumping,[1] Fig. 18. Though these tube-wells are commonly employed for raising water from quite moderate depths, they are frequently carried down to a depth of over 60 feet; and occasionally, where the soil is favourable, water has been obtained by their means at a depth of over 100 feet below the surface. In deep tube-wells, it is possible to avoid putting the pump in the small tube, by sinking an ordinary well down to within the limit of suction, enabling a larger pump to

TUBE-WELL WITH PUMP.

Fig. 18.



[1] "Well-Boring for Water, Brine, and Oil," C. Isler, p. 37.

be put on the top of the tube-well at the bottom of the sunk well; but this really constitutes a mixed system, in which the tube-well is only used for the lowest portion.

For providing considerable supplies of water, several tube-wells may be driven in fairly close proximity to one another, the distance apart, to avoid interference with the yield, varying with the character of the water-bearing stratum, but often not exceeding 15 to 20 feet. Several small tube-wells properly distributed, often furnish better results, and are more economical, than a smaller number of tube-wells of larger size.

Uses of Driven Tube-Wells.—The main advantage of driven tube-wells consists in their enabling small supplies of water to be obtained rapidly in an economical manner. These wells were introduced in the United States in the middle of the nineteenth century for the purposes of water-supply; and they were resorted to with remarkable success for supplying the British troops with water during the Abyssinian campaign in 1867–68. The system with small tubes is, indeed, specially valuable for armies on the march, when water needs to be obtained quickly, and only for a brief period in different places successively.

The driving of tube-wells is also useful for ascertaining the position, extent, and yield of water-bearing strata at moderate depths, previously to carrying out more permanent works for obtaining a supply. Moreover, though the supply obtained from a single tube-well is comparatively small, by driving several of these wells, and connecting them by a single suction pipe, it is possible to provide fairly large supplies for manufactories and towns. Tube-wells may also be advantageously employed for tapping water-bearing strata below the bottom of existing sunk wells, so as to increase the yield of water from the well, or to secure a purer supply where the water flowing into the sunk well has become subject to pollution since the construction of the well. These tube-wells possess the merit of excluding, by their method of construction, the infiltration of surface impurities; whilst the depth to which they are occasionally carried secures for them the more thorough natural filtration of the water they reach, which is regarded as the special advantage of deep wells.

Concluding Remarks.—Though shallow wells, at the present day, are generally regarded as objectionable sources of water-supply by communities which are very careful as to the purity of their supply, on account of the difficulty of protecting them from pollution, nevertheless, they have proved very serviceable in country districts as a means of easily procuring water at a small cost; and they are still very extensively made use of in the United States, not only in rural districts, but also for villages and towns, especially in valleys where alluvial deposits, or a layer of drift, render a good supply of water readily obtainable at a small depth below the surface. The safety of such supplies for domestic use depends upon the position of the wells in relation to dwellings, the extent of any impervious layer which may prevent surface impurities from percolating to the intake of the well, and the amount of natural filtration the water may undergo in passing through the permeable, water-bearing stratum before reaching the well.

In certain special circumstances, it may be necessary to rely on shallow wells for supplies of water; but it is very important, where practicable, especially in localities with an increasing population, to seek supplies by boring down to lower water-bearing strata, where the water is fairly secured from pollution by its depth below the surface.

Driven tube-wells are only available in soft strata; and they are chiefly suitable for small supplies in unfrequented regions. They are, however, very valuable in prospecting for water at moderate depths, and for procuring temporary supplies with rapidity and economy.

CHAPTER IV.

DEEP WELLS.

Value and mode of construction of deep wells—Conditions affecting Selection of Site, geological features, importance for deep wells, influence of faults —**Artesian Wells**: Origin of name, conditions producing them, requirements for successful borings, geological section from Cotswold to Chiltern Hills indicating required conditions; Sub-Artesian Wells: difference from artesian wells, causes of artesian wells being brought down to them; Deep Artesian Wells in Paris Basin, oldest in Europe, description of earliest in Paris at Grenelle, and of second at Passy, reduction in flow, depth and temperature of water of third well; Artesian Wells in London Basin, favourable conditions, instances in Chalk, search for water from them in Lower Greensand; Artesian Wells in Lincolnshire, peculiar conditions, examples in Oolites and Chalk; Various Artesian Wells, at Tours, Venice, and Sahara desert; Artesian Wells in Australia, and Cape Colony; statistics in Queensland, New South Wales, South Australia, and others; peculiar conditions and results in Cape Colony; Artesian Wells in United States, in Illinois, Dakota, Ohio, and Texas; Remarks on Artesian Wells, objections considered, value, limitations— **Ordinary Deep Wells**: Common use; Construction, general methods, supply increased by adits from bottom of well, examples of Brighton and Southampton; Position of Deep Wells, considerations affecting it, supply not proportionate to depth, illustrated by Southampton Waterworks, caution needed in region of faults—**Boring for Wells**: two systems, pounding by chisels, central core left with drill; Ordinary Boring, by rope, chisel, and shell, auger in clay, rope replaced by rods, forms of chisels, lining bore-hole and tools employed; Kind-Chaudron System for Large Bore-holes, as employed at Passy, cylindrical shells for removing *débris*, large trepans used for shafts to mines; Dru System of Free-falling Trepans, objects; release by shock, mechanical disengagement for large trepans; Boring Large Wells in Chalk, description of method at Otterbourne near Southampton, removal of *débris* by mizer; Modern Methods of Boring by Aid of a Rope, delays with boring rods, improvement in rope system worked by steam, Mather and Platt's system with set of chisels and special shell, American system with rope, chisel, rimer, and shell; Boring with Diamond Drill—Concluding Remarks on Deep Wells.

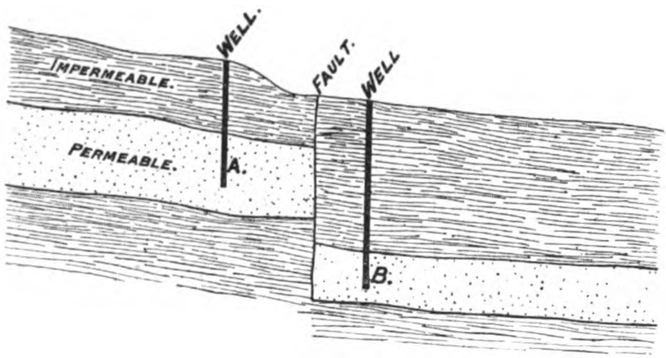
THE dangers of pollution to which shallow wells through very permeable, or considerably fissured strata, are exposed, become less in proportion to the depth attained and the compactness of the strata; whilst overlying impermeable strata form a barrier to the infiltration of surface impurities. Driven tube-wells, however, cannot be carried through rock, or even stiff clay; and the depth to which they can penetrate, even under favourable conditions, is quite limited. Accordingly, deep wells, which have to pass partially or wholly through stiff soil, chalk, or rock, are occasionally dug or blasted, and sunk in the ordinary way; generally they are partly sunk, and then taken down to the requisite depth by a boring of small diameter; and sometimes they are bored throughout, and occasionally borings of large diameter are resorted to, the system adopted depending on the nature of the strata to be traversed and the depth to be reached. Usually the upper portion of deep wells is made of fairly large diameter, from the bottom of which a small bore-hole is carried down to the water-bearing stratum from which the supply is proposed to be drawn.

Conditions affecting the Selection of a Site.—In choosing a situation for a deep well, great attention has to be devoted to the geological features of the locality, such as the depth below the surface of the stratum from which it is proposed to obtain water, the probable thickness of this stratum, its dip, its general character, the possible extent of its outcrop and freedom from pollution, and the prospects of the existence of faults. These preliminary investigations of the indications afforded by a careful inspection of the district, and the deductions drawn from geological data, are specially necessary where the well has to be taken down to a considerable depth, as the actual condition of the strata far below the surface is somewhat uncertain; whilst the cost of the undertaking increases with the depth. When tube-wells, driven to moderate depths, fail to reach water, they can be drawn out of the ground and tried at another spot; whereas an unsuccessful attempt to obtain water by boring a deep well involves a considerable waste of money, as well as delay in procuring a supply.

Faults are liable to introduce serious complications in the search for water, by forming an impermeable barrier across a water-bearing stratum, which, when occurring with a dipping stratum, renders the conditions on the two sides of the fault absolutely different. Thus, where an underlying, permeable stratum dips uniformly, though dislocated by a fault, so that its dip is towards the fault on the upper side, and away from the fault on the lower side, as indicated in Fig. 19, the permeable stratum at A will be fully charged with water, owing to the flow along it being arrested by the fault; whereas, the permeable stratum on the other side of the fault will be devoid of water, on account of the dip falling away from the fault, and the flow from the stratum on the upper side being stopped by the fault. Under these conditions, a well sunk into the permeable stratum at **A**, above the fault, will receive an abundant supply of water, by tapping the stratum

WELLS AFFECTED BY FAULT.

Fig. 19.



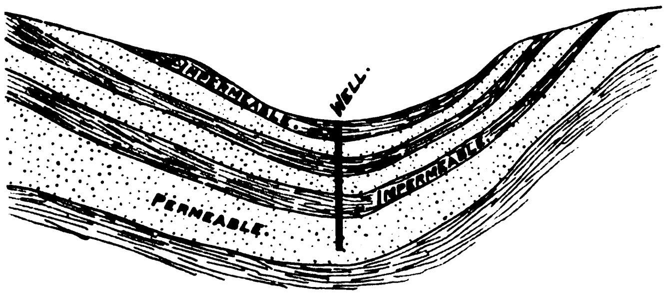
near its lowest point where it is closed at the end by the fault; whereas, a well sunk into the original continuation of the same stratum at **B**, below the fault, will fail to reach water.

ARTESIAN WELLS.

Definition of Artesian Wells.—The term "artesian," given to this particular class of well, has originated from the first borings in Europe for these wells having been carried out in the province of Artois in France, though traces of much more ancient borings for wells have been discovered in China, Persia, Egypt, and the Sahara Desert. This epithet has been often indiscriminately applied to all deep wells formed by borings; but it is more correctly confined to those wells in which the water rises up with some force to the surface and overflows, in consequence of the hydrostatic pressure of the water contained in the higher parts of a permeable stratum, being transmitted to the water in a dip of the same stratum, which the well has pierced. Wells, accordingly, to be really artesian, must be located in a valley, or on low-lying ground in relation to the surrounding district, so that the permeable stratum, enclosed between two impervious beds, into which the boring is carried, may rise sufficiently on one or both sides above the surface level at the site of the well, before reaching its outcrop, for the hydrostatic pressure in the stratum to overcome the frictional resistance to the flow of the water through the stratum and up the well, as shown in the case of a valley by the section of ground in Fig. 20. An artesian outflow may sometimes be obtained from a well in a low part of a plain, even at some distance from high ground, if the underlying permeable

ARTESIAN WELL IN VALLEY.

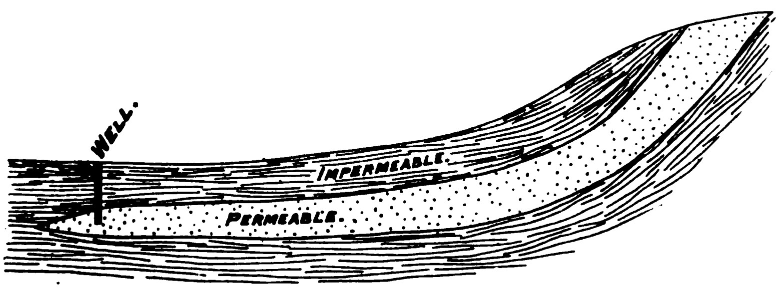
Fig. 20.



stratum pierced by the well is continuous from the high lands, is enclosed continuously between impermeable beds, and has a favourably situated outcrop, especially where the escape of the water seawards is arrested by the dying out or dislocation of the water-bearing stratum beyond the well, as indicated by the diagram in Fig. 21.

ARTESIAN WELL IN PLAIN.

Fig. 21.

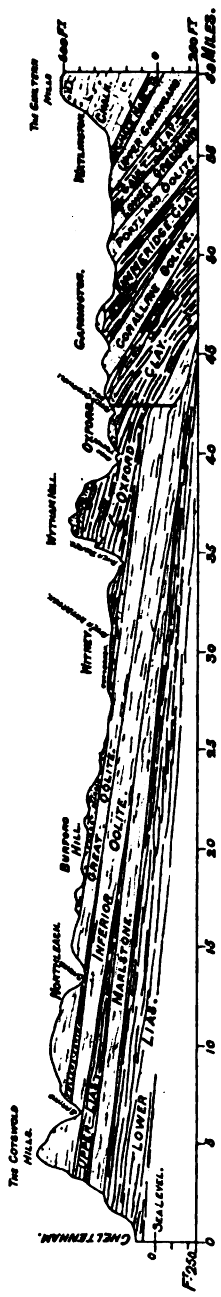


The success of an artesian boring evidently depends upon the complete enclosure of the water-bearing stratum between impermeable beds underground, the height of the outcrop of the water-bearing stratum in relation to the level of the ground at the site of the well, and the perfect continuity of the stratum between its outcrop and the well.

The succession of most of the principal strata between the Chalk

GEOLOGICAL SECTION FROM THE COTSWOLD TO THE CHILTERN HILLS.

Fig. 22.



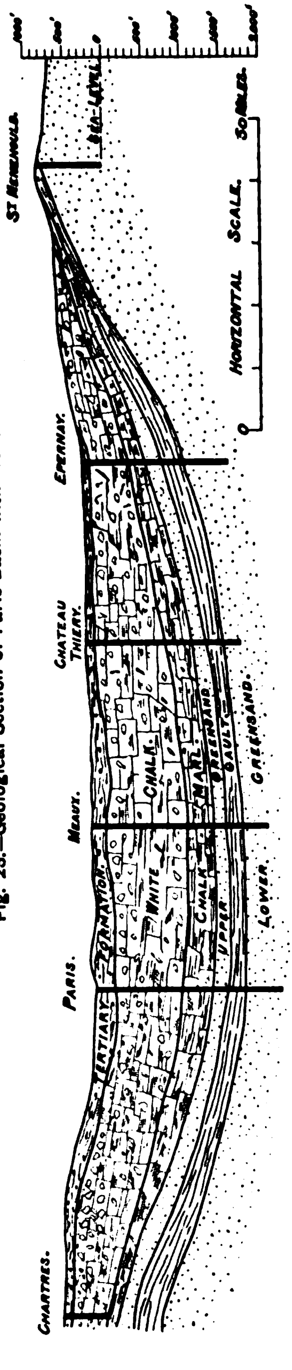
and the Lower Lias, is well illustrated by the geological section extending from the Cotswold Hills near Cheltenham, through Oxford, to the Chiltern Hills near Watlington, where permeable and impermeable strata are interspersed,[1] Fig. 22. Springs are naturally found where the permeable Oolites, forming the capping of hills, slope down to the underlying impermeable stratum; and a boring made at Oxford in 1832, through the Oxford Clay, into the Great Oolite, to a depth of 420 feet below the surface, furnished an artesian supply rising 3 to 4 feet above the ground, as might be expected from the downward dip of the stratum towards Oxford, as indicated by the section. The increased dip of the strata between Oxford and Watlington, render the permeable strata in that part less favourable for artesian supplies, except near their outcrop.

Sub-Artesian Wells.—Often the elevation of the outcrop of the water-bearing stratum, though insufficient to make the water overflow from a well situated in low-lying ground, is, nevertheless, adequate to raise the water in the well considerably above the level at which the water was first met with in piercing the water-bearing stratum in a dip. Such a well is somewhat artesian in character, or "sub-artesian" as it has been termed; and the pumping required to raise the water to the surface is proportionately diminished. The discharge from artesian wells is frequently reduced, and sometimes ceases altogether to overflow, when other borings are sunk into the same stratum, bringing the wells down to the sub-artesian type of well just described.

[1] "On the Geological Conditions affecting the Water-Supply to Houses and Towns, with special reference to Oxford," 1876, Joseph Prestwich, plate; and "On the Mineral Water of the Artesian Well at St. Clement's Oxford," 1876, J. Prestwich.

This result is due to the lowering of the general plane of saturation in the water-bearing stratum, and the consequent decrease in the hydrostatic pressure, by the increased draught on the water; and it is most apparent when the new borings are carried lower down into the water-bearing stratum than the old ones, and when the extent of the outcrop of the stratum is relatively small.

Deep Artesian Wells in the Paris Basin.—The oldest known artesian well in Europe was bored at Lilliers, in Artois, in 1126, and still furnishes a supply of water; whilst at an old well at Aire, in the same province, the water has continued to discharge 11 feet above the surface of the ground for a long period. The earliest artesian well carried down to a great depth, is the one which, on the advice of Arago, was bored in 1834–41 at Grenelle, in the outskirts of Paris, where, on reaching the Lower Greensand at a depth of 1798 feet, the water rose up the well and overflowed to a considerable height, and afforded for some time a regular supply of about 600 gallons per minute, the water having a temperature of 82° owing to the depth in the earth from which it issues, Fig. 23. The bore-hole was given a diameter of 10 inches in the upper part, through sands and clays down into the Chalk; whilst the remainder was made 6 inches in diameter,[1] thereby reducing the time and cost of construction, and facilitating the descent of the lining tubes.



ARTESIAN WELLS.

Fig. 23.—Geological Section of Paris Basin with Wells.

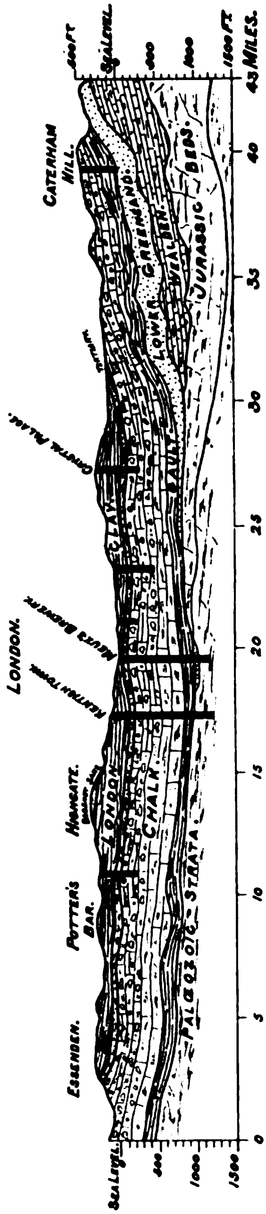
[1] *Proc. Inst. C.E.*, vol. xxiii. plate 10: and "Le Puits Artésien de Grenelle," M. Ray, Paris, 1844.

Notwithstanding the accidents and delays which attended the construction of the Grenelle well, its ultimate success, resulting from the exact verification of the predictions of geologists as to the depth and condition of the strata, led to the boring of a larger well down to the same stratum at Passy, nearly 2 miles away from the Grenelle well. This well, carried out in 1856–61, commencing with a diameter of $3\frac{1}{4}$ feet, and terminating with a diameter of $2\frac{1}{3}$ feet, reached the Lower Greensand at a depth of 1924 feet, and at first yielded a continuous flow of about $5\frac{1}{2}$ million gallons of water per day to a height of 54 feet above the ground.[1]

The yield of the Grenelle well was greatly reduced in 1852 by the crushing in of the 6-inch lining tube from the pressure of the encircling clay; and though the flow was improved somewhat by the insertion of a 4-inch lining, so as to deliver about 200,000 gallons a day, its yield was diminished again by the opening of the Passy well with its large bore, and eventually fell to a discharge not exceeding 66,000 gallons a day. The flow also from the Passy well has been reduced to $1\frac{1}{3}$ million gallons a day, by obstructions of sand in it. A third well was begun at Place Hébert, in the northern part of Paris, in 1866, and was only completed in 1888, having been greatly delayed by accidents to the boring machinery, the crushing of the lining, and stoppages. It had to be carried down to a depth of 2359 feet to reach the main water-bearing stratum of fine quartz sand in the Lower Greensand; and the temperature of the issuing water is 94°,

GEOLOGICAL SECTION OF LONDON BASIN, WITH WELLS.

Fig. 24.



[1] *Proc. Inst. C.E.*, vol. xxiii. p. 461, and plate 10.

owing to the greater depth below the surface from which it rises than the water at the Grenelle well.

Artesian Wells in the London Basin.—The hollow basin in which London is situated furnishes a favourable area for artesian wells, since the ground rises at the sides; and beds of sand and the Chalk are confined between the upper stratum of London Clay and Gault underneath,[1] Fig. 24, constituting a condition corresponding to that shown in Fig. 20 (p. 55).

The first artesian wells in London were only taken down to the bed of sand forming part of the Plastic Clay stratum underlying the London Clay, and which was saturated with water. This source of supply, however, was soon absorbed when wells became multiplied, as its outcrop is small, and influx into it is somewhat impeded by admixture with clay. Subsequent wells, accordingly, have been carried down into the Chalk, which is much thicker than the overlying bed of sand, and which, though yielding water mainly through fissures and flint beds, possesses an extensive outcrop on high ground. The well carried down 393 feet in 1844, so as to reach the Upper Chalk, for supplying the fountains of Trafalgar Square, is a noted instance of an artesian well in the London basin; whilst the artesian well at Sion House on the Thames near Richmond, tapping the Chalk, was sunk 620 feet. The numerous wells, however, which have been bored down into the Chalk in the London basin,[2] have considerably lowered the water-level in the stratum, so that in several wells which originally overflowed, the water stands at a depth of about 100 feet below the level of the Thames.

It was formerly supposed that a large supply might be obtained for London by boring wells through the Chalk, and the underlying Gault, into the Lower Greensand, as accomplished in Paris. In sinking, however, a well with this object at Kentish Town, constructed with a shaft to a depth of 539 feet, and a boring from thence to the full depth of 1302 feet, it was found, after traversing the Gault, that sandstones and clays of either the Trias or Devonian formation took the place of the Greensand, which was wanting, showing that this stratum is discontinuous under the London basin, Fig. 24, and is therefore not reliable as a source of supply. Nevertheless, in carrying down a bored well for Meux's brewery in Tottenham Court Road, to a depth of 1144 feet,

[1] "Geology, Chemical, Physical, and Stratigraphical," J. Prestwich, p. 161, and plate at p. 166, sec. 4; and *Memoirs of the Geological Survey of England and Wales*, "The Geology of London," W. Whitaker, vol. i. pp. 49 and 50.

[2] A list of many of the wells sunk in the London basin, with particulars as to diameter, depth, strata traversed, and supply obtained, is given in "Water-Supply: The Present Practice of Sinking and Boring Wells," E. Spon, 2nd edition, pp. 238 to 250.

water was obtained on penetrating 21 feet into what was supposed to be the Lower Greensand at a depth of 1022 feet, this stratum having a thickness there of 66 feet; but the yield from it was only 36,000 gallons a day.

Artesian Wells in Lincolnshire.—Though considerable portions of Lincolnshire are very flat, supplies have been furnished by artesian wells in these districts; for rain, falling on the outcrop of a permeable stratum on high ground, sometimes flows underground along the stratum for long distances across the plains; and wells sunk through the superficial impervious stratum on the flat land, tap this flow of water, which often rises in the well, and in some cases overflows. Thus several wells in South Lincolnshire, with bore-holes of from 1½ to 4 inches in diameter, carried from 70 to 170 feet below the surface into the Oolite, have furnished good supplies of water, which overflow, and in some instances attain a height of 40 feet above the ground.[1] An early example of one of these wells is the artesian well, 4 inches in diameter, bored some distance into the Oolitic stratum at Bourn in 1856, to a depth of 94 feet from the surface, where, on reaching this depth, the water rose 39¾ feet above the surface, furnishing a constant supply of 567,000 gallons per day.[2]

In the north-eastern part of Lincolnshire, the rain, falling on the outcrop of the Chalk on the Wolds, passes along the dipping Chalk stratum towards the sea; and this flow has been tapped by wells carried down into the Chalk, which in some cases, when sunk in low ground, overflow, notwithstanding the continuity of the Chalk seawards.

Various Artesian Wells.—Between 1830 and 1837, eleven artesian wells were sunk at Tours, in the combined valleys of the Loire and the Cher, into the Chalk and down to the Greensand to depths of from 390 to 560 feet,[3] and twenty wells in Venice into sand, in 1847–56, to depths of from 187 to 193 feet, with discharges varying from 7000 to 77,000 gallons a day;[4] but the multiplication of wells in both these localities produced a lowering of the water-level; and seven of the wells at Venice ceased overflowing soon after their construction.

One of the most valuable applications of the system of artesian wells has been carried out by the French, since 1856, in the Tunisian and Algerian portions of the Lower Sahara, and more particularly in the province of Constantine of the Algerian Sahara. Numerous wells have

[1] *Proc. Inst. C.E.*, vol. lxxiv. p. 160.

[2] *Ibid.*, vol. lxxv. p. 245.

[3] *Mémoires de la Société géologique de France*, Paris, 1837, vol. ii. part 2, plate 21, Geological section.

[4] "Guide du Sondeur," Degousée and Laurent, 2nd edition, Paris, 1861, vol. ii. p. 498, and Atlas, plate 52.

been bored to depths of from 150 to 800 feet, where, reaching the underground flow, which is believed to originate in the rainfall on the Atlas Mountains to the north, many of these wells overflow, forming oases where cultivation and settlement become practicable in that vast extent of desert.[1]

Artesian Wells in Australia and Cape Colony.—Artesian wells are specially serviceable in thinly populated, dry districts, where springs do not exist, and where the surface waters fail in the dry season, or are subject to pollution. In Australia, borings for wells have been more extensively resorted to in Queensland than elsewhere. The artesian area in Queensland has been estimated at 88,300 square miles, and in New South Wales at 20,000 square miles, where, next to Queensland, most progress in obtaining artesian supplies has been made. In 1901, the number of bore-holes recorded in Queensland was 891, of which 546 furnished artesian supplies rising above the surface; and the total discharge from these overflowing wells has been estimated at 351 million gallons a day, thirty-four of them yielding over 1½ million gallons a day, sixty-two of them from 750,000 to 1½ million gallons a day, and two hundred and fifty of them giving flows of 10,000 to 750,000 gallons a day.[2] These bore-holes range in depth from 50 feet up to a maximum of 5045 feet for a well completed at Bimerah Run, Whitewood, in 1899; and their average depth was estimated at 1197 feet in 1901. The strata from which these wells mostly derive their supplies are sands and sandstones of the Lower Cretaceous formation; and the great depth from which the water issues in many cases renders its temperature high, exceeding 100° generally where the depth is over 100 feet, and in one instance attaining 160° at a depth of 1751 feet; whilst some of the deepest wells deliver water at temperatures ranging from 160° to 196°. Several of these wells have experienced a diminution in their flow in course of time, partly owing, in some cases, to obstructions or defects in the lining; but it probably also results from the multiplication of wells within a limited area. Moreover, it is believed that the outcrop of this water-bearing stratum in Queensland is only of moderate extent, and that therefore, with a small rainfall over it, and considering the amount of water drawn from the stratum over an extensive area, which is continually being increased by new borings, the limit of the regular supply may be nearly reached. Stringent regulations have, accordingly, been proposed for economizing the supply,

[1] "De l'Utilisation des Eaux Artésiennes du Bas Sahara algérien," G. Rolland, Congrès International de l'Utilisation des Eaux fluviales, Paris, 1889, p. 97; and "De l'Alimentation du Basin artésien dans le Desert (Bas Sahara algérien)," G. Rolland, *Bulletin de la Société géologique de France*, 1894, p. 506, and plate 15.

[2] "Queensland Water-Supply; Report of the Hydraulic Engineer, 1901."

by stopping the waste of water from the useless, unchecked flow from the artesian wells.[1]

Both public and private artesian borings have been made in New South Wales; and at the end of 1896, one hundred and forty artesian wells had been bored, yielding about 60½ million gallons a day. These artesian wells range in depth from 114 to 3092 feet; the maximum daily flow amounts to 5 million gallons at one of these wells; and the maximum temperature of the water is 139° at two wells which are under 2000 feet in depth.[2]

South Australia comes next to New South Wales in the progress made with artesian borings, which have been carried to a maximum depth of 3000 feet; but rather a large proportion of the overflowing wells which have been hitherto bored, yield salt or brackish water. A start has been made by Western Australia in seeking supplies of water from artesian sources; and the results of the initial borings have proved very satisfactory as regards the quality and quantity of water obtained. In Victoria, the search for artesian supplies of water has not been successful, owing apparently to unfavourable geological conditions.

The conditions of Cape Colony in respect of supplies of water from artesian sources are quite different from those of Australia, the Sahara, and several other parts, owing to the absence of the Cretaceous and later formations, from which artesian supplies are so commonly derived. The strata rising to the surface in this colony appear to correspond to the Carboniferous, Devonian, Silurian, and Laurentian formations; and rock is generally met with only a few feet below the ground, and is often exposed. Borings, however, with diamond drills, carried out since 1890 with the assistance of the Government in furnishing the necessary plant and borers, have proved that supplies of good water are reached over extensive areas, in porous sandstone and shale beds, at depths of between 50 and 500 feet below the surface, varying in volume from 1000 gallons up to 100,000 gallons a day.[3] About three-fourths of the numerous bore-holes, of 2 to 6 inches in diameter, have been successful in reaching water, which generally rises in the well considerably above the level at which the water appeared; whilst in about a third of those wells the water overflows.

Artesian Wells in the United States.—Large tracts of land in the United States are available for artesian supplies; several areas have

[1] "Queensland Water-Supply; Report of the Hydraulic Engineer, 1893;" and "Artesian Water in New South Wales and Queensland," T. W. E. David, *Royal Society of N.S. Wales*, October 3, 1893.

[2] *Ibid.*, 1897, p. 11.

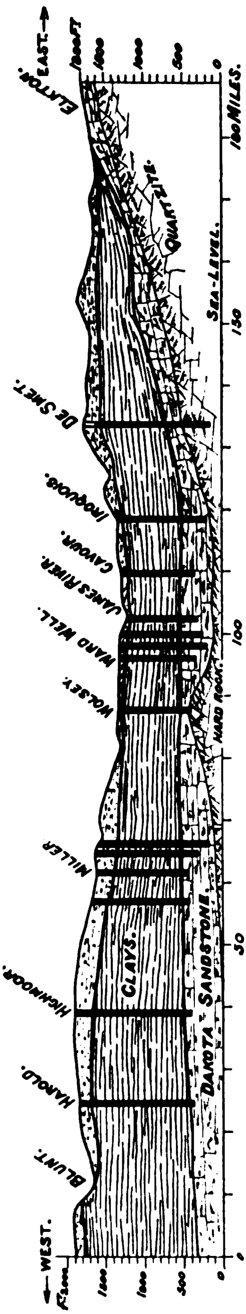
[3] "Boring for Water in the Cape Colony," R. W. Ritso, *Proc. Inst. C.E.*, vol. cxlvii. p. 312 to 314.

already been extensively utilized for water-supplies to towns and villages, and for irrigation; and others are in course of development.

In Illinois, water is obtained from a large number of artesian wells, varying in depth from 40 to 3115 feet, and having an estimated average depth of 1377 feet. The shallow wells obtain their supply from drift; whilst the deeper wells are carried down occasionally into Trenton rock or Galena, and more commonly into St. Peter sandstone, all belonging to the Lower Silurian formation, or into Potsdam sandstone of the Cambrian formation, which furnish good supplies of water.[1]

Extensive regions of the eastern part of South Dakota, and also of North Dakota, have been mapped out as suitable for artesian wells; and three hundred and fifty of the wells already bored in these parts yield an overflowing volume of water, attaining altogether 150 million gallons a day. These artesian wells draw their supplies of water mainly from the Dakota sandstone, at depths of from 500 to 1375 feet, Fig. 25; but some of them obtain water from the Chalk, and also from drift.[2]

There are numerous artesian wells in the north-west and western parts of Ohio, carried down from 50 to 250 feet below the surface, into a stratum of drift from 100 to 300 feet in thickness; and many of these wells overflow with a considerable pressure.[3]



ARTESIAN BASIN OF EASTERN DAKOTA.

Fig. 25.—Geological Section from Blunt to Elkton, with Wells.

[1] *United States Geological Survey*, Report for 1895-96, Part II. pp. 785 and 810.

[2] *Ibid.*, Report for 1895-96, Part II. pp. 609 to 694, and plates 69 and 71; and Report for 1899-1900, Part IV. p. 563.

[3] *Ibid.*, Report for 1897-98, Part IV. pp. 637 and 697.

In Texas, there is a very extensive artesian area, stretching from near the Red River in Denton county to the Rio Grande at Del Rio, and reaching down to the Gulf of Mexico, constituting a tract of land 448 miles long with an average width of 300 miles. Many towns within this area derive their supply of water from artesian wells, as for instance Galveston, Houston, Dallas, and San Antonio; and the water issues from several water-bearing strata of moderate thickness.[1] Galveston is supplied by thirty-three artesian wells on the mainland, bored to depths of from 700 to 850 feet; and there are also twenty-five artesian wells in the city itself. Houston is supplied by nearly a hundred artesian wells, from 140 to 500 feet in depth. Altogether in this district, water is obtained from numerous artesian wells, ranging from 100 to 3330 feet in depth, and varying in yield from a feeble overflow of about 15,000 gallons a day, up to a discharge, at high pressure, of one million gallons daily.

These instances indicate sufficiently the important place artesian wells occupy in the United States in relation to water-supply and irrigation; and the efforts of the geological surveyors dealing with this question, are directed to gaining precise information about the strata from the borings already carried out, and to defining the boundaries of the artesian areas, so as to enable persons to select suitable localities in their search for water, and to avoid boring at sites where there is no reasonable prospect of obtaining a supply. In several instances, water-bearing strata have been met with in boring for oil; and the knowledge thus accidentally gained has been sometimes subsequently utilized.

Remarks on Artesian Wells.—Objections have been raised against the use of artesian wells for water-supply, on account of the liability of the artesian flow to be reduced in course of time, the notable lowering of the water-level when several wells are sunk in a limited area, and the high temperature of the water issuing from a considerable depth underground. The first objection can in great measure be guarded against by inserting a strong lining in the bore-hole where it traverses upper permeable strata, through which the rising water might escape, and also through the softer strata in the lower parts, which would be liable to be forced by the pressure on them into an unlined bore-hole. The two other objections apply more or less to all deep wells, though the additional lift required for the water in ordinary wells on the lowering of the underground water-level, appears less onerous than the loss of pressure in artesian wells, involving a resort to pumping; whilst the water drawn from considerable depths in ordinary wells has

[1] *United States Geological Survey*, Report for 1899–1900, Part VII. pp. 387 to 637.

more time to cool in the large upper portion of the well, than the water spouting out of the bore-hole of an artesian well.

Artesian wells, however, furnishing a supply of water above ground without requiring any mechanical aids, are most valuable in arid, unfrequented regions, where the wind provides the only readily available, and somewhat intermittent, power for pumping. Moreover, the regular pumping up of water from wells might involve too prohibitive a cost to be utilized for irrigation.

Unless the geological conditions of a district are unmistakably indicated, the first borings for artesian supplies must be somewhat uncertain in their results, especially in regions not fully explored; but as soon as borings in different parts of a district have established clearly the position, continuity, extent, and yield of the water-bearing stratum, other borings may be made at suitable sites within the area defined with excellent prospects of obtaining a supply. The extent, however, to which these artesian wells can be multiplied, without unduly reducing the pressure and diminishing the yield of the earlier wells, depends entirely upon the area of the exposed outcrop and the available rainfall on it.

ORDINARY DEEP WELLS.

Wells sunk into a water-bearing stratum at a considerable depth below the surface, with the object of securing a supply free from pollution, from which the water has to be raised by pumping, are far more generally applicable for procuring water than artesian wells. Very special conditions, as explained above, are necessary for the success of artesian wells, and even to a minor extent for the rising of the water in sub-artesian wells; and these conditions are only found in certain localities, and within definite boundaries. On the contrary, ordinary deep wells can be carried down to any water-bearing stratum which, from its nature, extent, and area of outcrop, offers a good prospect of yielding a continuous supply of potable water. Sub-artesian wells, which occupy an intermediate position between ordinary wells and artesian wells, only differ from ordinary deep wells in reducing the lift required in proportion as the water rises in the well.

Construction of Deep Wells.—Generally, the upper portion of deep wells is constructed of large diameter, in the same manner as shallow wells previously described; and from the bottom of this large well, a boring is taken down to the water-bearing stratum from which the supply is to be drawn. The large upper portion enables the pump to be placed in it at a lower level than with a bore-hole; and it also provides a sort of reservoir for the water pumped up from the bore-hole, thereby equalizing the efflux from the well. These wells are always lined in the upper part, to provide against surface pollution, and where they traverse soft soil; and a lining has also to be introduced where the wells pass through strata which, from their geological composition, are liable to yield water unsuitable for domestic use.

In strata where the underground flow takes place mainly through fissures, as for instance in the Chalk, it is advisable, where the depth to the plane of saturation is not too great, to carry down a well of large diameter, or an enlarged boring, to the full depth, so as to enable horizontal galleries, or headings, to be driven from the bottom of the well, in order to tap water-bearing fissures which the well has not pierced, and thereby increase the yield. Thus, for instance, the wells in the Chalk supplying Brighton with water, have had their yield greatly augmented by carrying headings from them parallel to the cliffs, and, consequently, at right angles to the fissures running towards the coast; and this arrangement has enabled water to be utilized, which had previously discharged in large volumes wastefully into the sea. An example of borings of large diameter carried down into the Chalk, so that horizontal adits might be driven from them to increase the supply, is furnished by the two wells, 6 feet in diameter, bored to a depth of 100 feet in the Chalk close to its outcrop at Otterbourne, which provide the water-supply for Southampton.[1] From the bottom of these wells, about 1500 feet of horizontal headings, or adits, have been driven, enabling about four million gallons of water a day to be pumped up for supplying the population and trade of Southampton, situated about 8 miles distant.

Position of Deep Wells.—The sites for ordinary deep wells do not require to be selected with the same care as those for artesian wells; but it is advisable to choose, if possible, a position where the water-bearing stratum is not very far below the surface; and it is preferable to adopt a low site rather than an elevated one, owing to the smaller depth needed to be traversed by the well, even if the water has to be eventually discharged into a reservoir at a higher elevation than the well. Strata with a large incumbent weight over them are liable to be more compact than when nearer the surface; and sometimes the lower part of a stratum, such as the Chalk, is closer than the upper portion. It therefore often happens that, after a suitable water-bearing stratum has been reached, no advantage is gained by carrying the well down deeper; and, occasionally, when a well has been taken down through a stratum underlying a water-bearing stratum, in search for a further supply, the water originally obtained has been actually lost by the

[1] "The Wells and Borings of the Southampton Waterworks," W. Matthews, *Proc. Inst. C.E.*, vol. xc. p. 34, and plate 1, fig. 11.

bore-hole affording a vent for the water into a lower, unsaturated stratum.

The history of the Southampton waterworks affords a striking illustration of the fact that the depth to which a well is carried in the same stratum furnishes no measure of the supply to be obtained. A well was sunk on the Common near Southampton in 1838–51, with a diameter reduced from 13 feet down to 7 feet, in a depth of 563 feet; and from thence a 7½-inch bore-hole was carried down to a total depth of 1317 feet, the last 852 feet having traversed the Chalk;[1] and this well was eventually abandoned, having yielded only 130,000 gallons of water a day. The site of this old well was only slightly higher than that of the Otterbourne wells; but the Chalk there was overlaid by London Clay and other beds, 465 feet in thickness, and the flow through the Chalk there has been impeded by upheavals; whereas the Otterbourne wells, only 100 feet deep, reached the Chalk only 2½ feet below the surface, and their large yield of 4 million gallons daily has been greatly aided by adits.

In localities where the strata are dislocated and intersected by faults, great caution requires to be exercised in selecting the position for a well; and under such conditions, and also where the depth to the water-bearing stratum is not great, it is advisable to test the probable yield by trial borings before embarking upon regular works for a well.

Boring for Wells.

There are two distinct principles upon which borings are conducted; for, in the one case, the strata traversed are pounded into small fragments and dust by the boring tool in forming the bore-hole, before the materials are removed from the hole as sand or mud, which is the ordinary method of carrying out borings; whilst in the second method, a rotating diamond drill cuts out a circular ring, having its outer diameter of the requisite size of the bore-hole, and leaving a central, undisturbed, solid core, which, when taken out, indicates the exact nature and condition of the strata traversed, which cannot be ascertained with precision from the *débris* obtained from the bore-hole by the ordinary method. The first system is very convenient where a small bore-hole has to be formed through hard, loose soil, such as gravel and stones, and rock of only moderate hardness, as, though comparatively wasteful of labour in pounding up the beds traversed, it is effected by the fall of a simple tool; and it has long ago been extended to borings of large diameter by means of special appliances. When, however, a boring has to be

[1] *Proc. Inst. C.E.*, vol. xc. p. 39, and plate 1, fig. 13.

made through hard rock, and especially when the bore-hole has to be taken down to a considerable depth, and formed of large diameter, the rotary diamond drill, though costly and having complicated machinery, is decidedly the most advantageous, quite irrespectively of the valuable information as to the condition of the underlying strata afforded by the solid core brought up.

Ordinary Boring.—The earliest method of boring employed in Europe, borrowed from the Chinese, consisted of a chisel suspended from a rope, guided in a tube, and raised and dropped by means of a lever. The twisting of the rope causes the chisel to vary the position of its blow; and the shattered rock is removed at intervals by a cylindrical shell with a valve opening upwards at the bottom, through which the *débris* enters and is retained on lowering the shell into it, thereby clearing the hole on raising the shell, Fig. 26. In stiff strata such as clays, the hole is formed by an auger, which is pressed into the soil and turned so as to force the material into it; and the auger is then raised, and its load discharged, Fig. 27.

By substituting rods screwed together for the rope, greater control was obtained over the boring tool, the rods being turned by projecting arms from the top; and the chisel, weighted with the boring rods, delivered a harder blow when dropped on the bottom of the bore-hole. This modification enabled holes of small diameter to be bored to great depths in search of water, of which the Grenelle well at Paris, described on page 57, is a notable example. The jar, however, of the rigid, heavy, iron rods when the chisel struck hard rock, sometimes resulted in a fracture, the detached portion blocking up the lower part of the bore-hole, which led to great delays at Grenelle, and has occasionally involved the abandonment of the work. This very serious defect was remedied by giving the chisel a sliding motion, causing it always to fall a definite, suitable distance, and thereby obviating the jar on the boring rods and the consequent fractures.

Various forms of chisels, made of the best wrought-iron or mild steel, are employed for pounding up the rock in forming a bore-hole. Flat chisels with straight or pointed ends, Fig. 28, require to have their position constantly shifted to make a round hole; but when the flat chisels are provided with a projecting segmental T-piece at the side, corresponding to the circumference of the hole, Fig. 29, the circular form is more readily obtained with fewer shifts of the tool.[1] A duplex chisel, in the form of a cross, is another shape employed; whilst a

[1] "Well-Boring for Water, Brine, and Oil," C. Isler, pp. 44 to 46; "Guide du Sondeur," Degoussée and Laurent, 2nd edition, Atlas, plates 9, 20, 22, 23, and 30; and "Handbuch der Tiefbohrkunde," T. Tecklenburg, vol. i., 1886, vol. iv., 1890, vol. v., 1893, and vol. vi., 1896, plates.

cylindrical chisel, Fig. 30, enables a bore-hole to be trimmed to a perfectly circular and vertical form. A kind of worm screw is used for loosening soft soil; and a corkscrew form of screw serves for recovering broken-off tools, for which purpose various other kinds of apparatus are also resorted to.

In passing through soft soil, or through unsatisfactory permeable strata, the bore-hole has to be lined with tubes, to prevent the falling in of the sides in the former case, and the influx of unsuitable water in the

SHELL WITH VALVE FOR CLEARING BORE-HOLE.

Fig. 26.

AUGER FOR BORING IN CLAY.

Fig. 27.

BORING CHISELS.

Fig. 28. Flat Chisel.

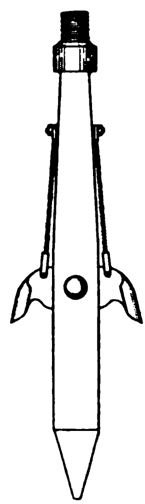
Fig. 29. Chisel with T-piece.

Fig. 30. Cylindrical Chisel.

BORING TOOLS.

Fig. 31. Spring Rimer.

Fig. 32. Expanding Cutters.



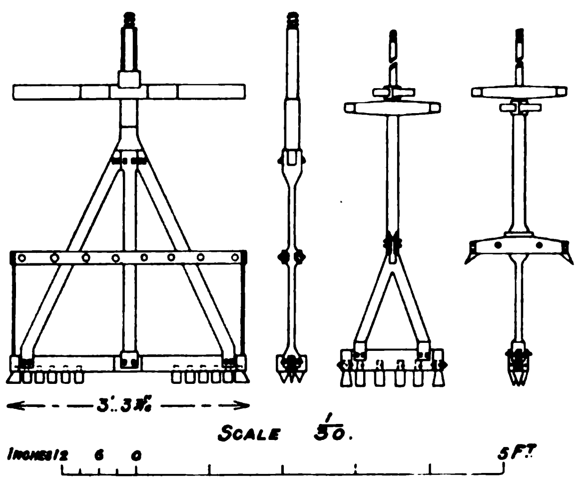
latter. The first tube, or pipe, should be furnished with a steel shoe, having a cutting edge at the bottom, so as to shear off any roughness or projections at the sides of the bore-hole, and thus facilitate the descent of the pipes. In traversing soft soil, when the tubing has been forced down with the aid of a turning movement, or driven down, if necessary, to the bottom of the hole, the boring is recommenced; and the hole is enlarged below the pipes by means of a spring rimer when the tubing does not descend readily, which is turned round in the hole on expanding after leaving the lining,[1] Fig. 31. Another tool, with expanding cutters, is also used for trimming the bore-hole below the tubing, Fig. 32. The tubes may, with advantage, be made of steel with socket joints, and a special flange screwed on at the top of the tubing when driving has to be resorted to.[2]

Trestles, shear-legs, or staging are erected over the bore-holes, and provided with a windlass and tackle for lowering and raising the boring rods and tools, and for handling the tubes; whilst occasionally, for an important work, steam power is employed.

Kind-Chaudron System for making Large Bore-holes.—The provision of large bore-holes increases notably the yield of artesian wells; and, moreover, large boring tools can be secured more effectually against injury from torsion, and from knocking against the sides of the bore-hole, which is a frequent cause of accidents.

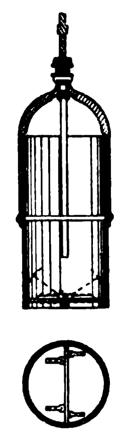
KIND SYSTEM OF BORING TREPANS AT PASSY WELL.

Fig. 33.—Large Trepan. Fig. 34.—Small Trepan.



SHELL FOR REMOVING DÉBRIS FROM BORE-HOLE.

Fig. 35.—Flap Valves.



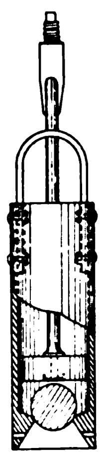
The second artesian well carried down, through the Chalk, into the Lower Greensand of the Paris basin at Paris (p. 58), consisted of a bore-hole, 1 metre (3 feet $3\frac{3}{8}$ inches) in diameter, bored by the Kind system with a large trepan, or cutter, having a flat iron frame of the

[1] "Well-Boring for Water, Brine, and Oil," C. Isler, pp. 46 to 48.

[2] *Ibid.*, pp. 63 to 65.

width of the hole, fitted at the bottom with a row of steel teeth, which is given a uniform fall.[1] A smaller cutter first breaks up the rock in the centre of the hole, and is soon after followed up by the large cutter, which, shattering a concentric ring of rock round the central hole, enlarges the bore-hole to its full size. The large trepan at Passy, 1 metre in width, weighed $1\frac{4}{5}$ tons, Fig. 33, and was given a fall of 2 feet. The fall of the trepan was, in this case, effected by the reaction of a column of water admitted into the hole, on a disc encircling the boring rod. The upward pressure of the water on this disc, on being rapidly lowered down into it, pushed open a clutch, whereby the trepan, which had been gripped by this clutch, was released; whilst the downward pressure of the water on the disc when being raised again, caused the clutch to catch hold of the trepan.[2] The boring rods carrying the trepan, being made of oak, were partially counterbalanced by the water in which they moved; and an up-and-down motion was imparted to them by a balanced beam, worked by a steam-engine, one end of which stretched over the hole, and the other end carried a weight in order to counterbalance the tools. At Passy, the large trepan dealt fifteen to twenty blows per minute on the rock. The smaller, pioneer trepan, which bored the central hole at Passy, about $1\frac{1}{3}$ feet in diameter, is shown in Fig. 34. A wrought-iron, cylindrical shell, closed at the bottom by two hinged semicircular flaps opening upwards, was employed at Passy for removing the shattered rock, by pushing the cylinder at the end of the boring rods down into the *débris* in the hole; and the *débris*, having been thus forced into the cylinder through the openings in the bottom, is retained by the closing of the flaps, and raised from the well, Fig. 35.

SHELL WITH BALL VALVE.
Fig. 36.



A cylindrical shell with a conical aperture at the bottom, closed by a ball on the inside, has been much used by borers for the removal of the *débris* from bore-holes, Fig. 36, being simpler and less liable to get out of order than the cylinder closed by flaps.

Trepans, $13\frac{3}{4}$ feet in width and weighing 4 to 5 tons, had been introduced a few years before the commencement of the Passy well, for sinking shafts through rock down to mines. The teeth in these trepans were arranged in a row all along the frame, so as to bore the shaft to the full

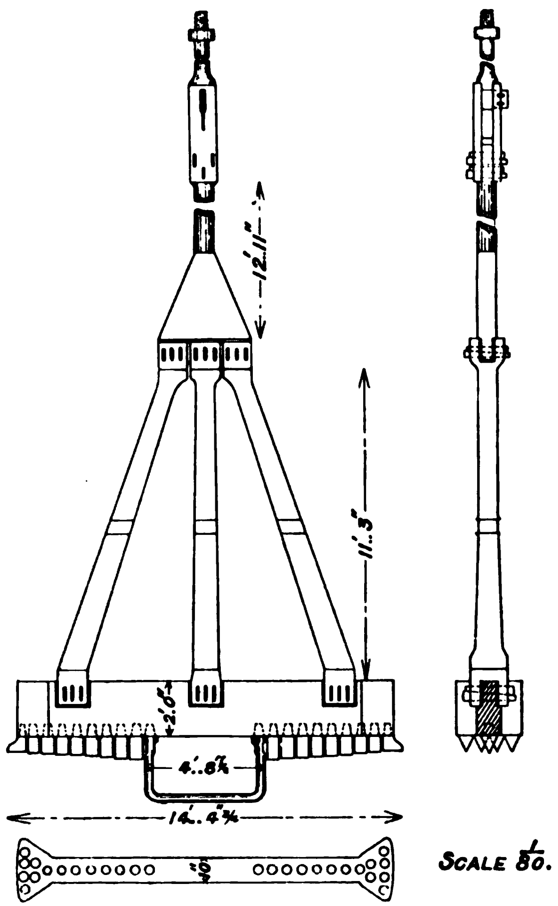
[1] "Machinery employed in sinking Artesian Wells on the Continent," G. R. Burnell, *Proc. Inst. C.E.*, vol. xxiii. p. 460, and plate 11.

[2] "Guide du Sondeur," Degoussée and Laurent, vol. ii. p. 240, and Atlas, plate 35.

width in a single operation.[1] Still heavier trepans have since been adopted, reaching a weight of 16½ tons, Fig. 37, with smaller trepans, weighing 8 tons, for the initial, central boring, as at Passy; and

KIND-CHAUDRON SYSTEM.

Fig. 37.—Large Trepan for Boring Shafts.



some improvements in the system, together with more perfectly water-tight tubbing through permeable strata, having been introduced in sinking shafts for coal-mines in Belgium, the system became known by

[1] "Guide du Sondeur," vol. ii. p. 430, and Atlas, plate 35, and plate 36, figs. 8, 9, and 10.

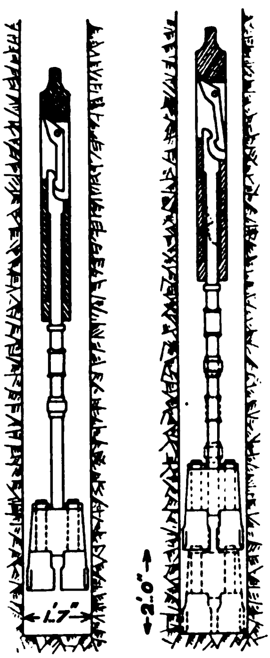
the joint name of its two designers.[1] The smaller trepan bores the central hole between 20 and 90 feet in advance of the full-sized shaft formed by the large trepan; and in this latter trepan, the teeth incline towards the centre, so as to direct the *débris* into the central hole for removal.

Dru System of Large Free-falling Trepan.—Water is not always available for effecting the release of the trepan, as required in the Kind-Chaudron arrangement; and, moreover, the form of the clutch in this system is subject to very rapid wear. The Dru system was, accordingly, specially contrived to obviate the necessity of using a column of water for causing the fall of the tool, and to reduce the wear by altering the form of the clutch.[2] The trepan in this system is furnished with one, two, or more chisels, arranged in a single line, according to the size of the bore-hole required.

DRU SYSTEM OF FALLING TREPAN RELEASED BY SHOCK.

Fig. 38. Raising Trepan.

Fig. 39. Release of Trepan.



When the hole to be bored is of moderate size, the clutch which engages the hook at the top of the rod forming the upper part of the trepan, is arranged as shown in Fig. 38, when the boring rods are lifting the tool. Directly the end of the beam carrying the boring rods reaches the top of its stroke, the tail-end of the beam knocks against a wooden buffer-block,[3] thereby producing a shock, which is transmitted by the rods to the clutch, which, resting on each side on a pivot placed in a hole somewhat oval vertically, jumps slightly, and, at the same time, owing to its inclined back face at the top, is pushed sideways, and, consequently, releases the hook forming the upper end of the trepan, which accordingly falls, Fig. 39.

Though the release by a shock, as described above, answered quite

[1] "Kind-Chaudron System of sinking Shafts through Water-bearing Strata," E. Bainbridge, *Proc. Inst. C.E.*, vol. xxxiv. p. 50, and plate 13; and "Handbuch der Tiefbohrkunde," T. Tecklenburg, vol. vi. plates 2 to 4, and 11 to 14.

[2] "Notice sur les Appareils et Outils de Sondage," Léon Dru, Paris, 1878.

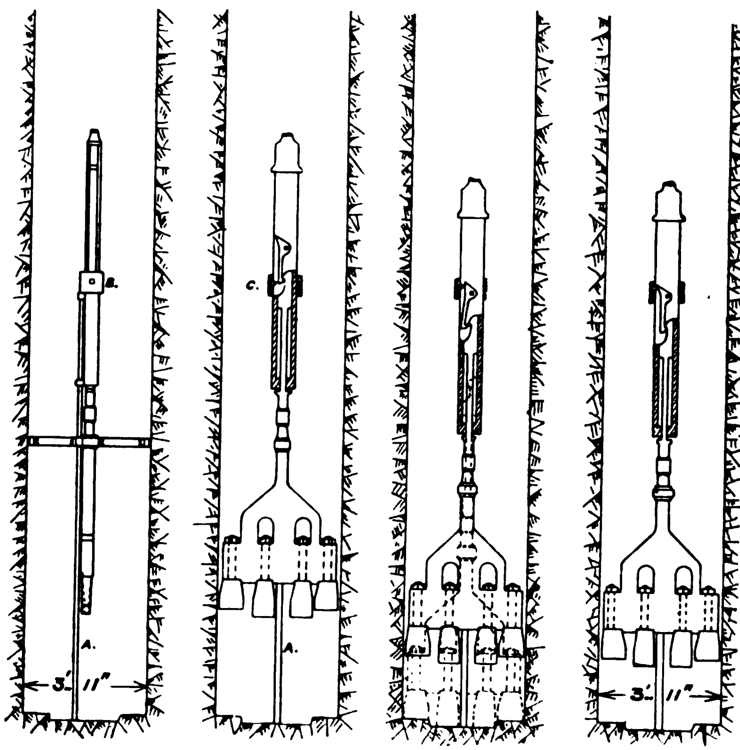
[3] "The Present Practice of Sinking and Boring Wells," E. Spon, pp. 132 and 134.

well for a trepan weighing $\frac{3}{4}$ ton, used in boring a well 19 inches in diameter, the shock required for releasing a trepan weighing $3\frac{1}{2}$ tons, needed for boring a well about 4 feet in diameter, had to be made so great as to endanger the boring rods and other machinery; and, therefore, another arrangement had to be devised for the release of large trepans.

DRU SYSTEM CF LARGE TREPANS.

Disengaging Rod and Trepan.

Fig. 40. Side. Fig. 41. Front. Fig. 42. Falling Trepan. Fig. 43. Raising Trepan.



In the contrivance designed for large trepans, the disengagement is effected by the help of a vertical rod, **A**, which, when lowered sufficiently to reach the bottom of the bore-hole, prevents the further descent of an encircling ring, **B**, attached to the top of the rod, and carrying a pawl, **C**,[1] Figs. 40 and 41. During the lowering of the trepan into the

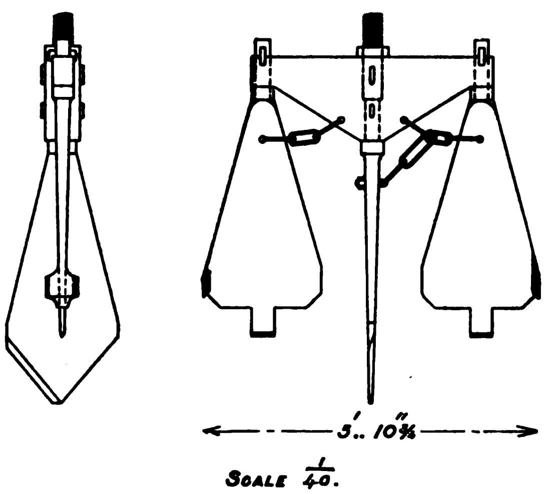
[1] "The Present Practice of Sinking and Boring Wells," E. Spon, p. 136; and "Handbuch der Tiefbohrkunde," T. Tecklenburg, vol. i. pp. 37 and 38, and plate 10, figs. 1 and 3.

hole, the clutch is kept firmly engaged in the hook of the trepan by the projecting pawl, Fig. 41. When, however, the lowering is continued beyond the point shown in the figure, the arrest of the descent of the ring by the rod causes the descending clutch to pass along the point of the projecting pawl, till at last its inclined back face is pushed forward in passing the pawl, disengaging it from the trepan, which consequently falls, Fig. 42. On lowering the boring rods after the fallen trepan, the clutch catches hold again of the hook of the trepan, which is then raised for another blow; and the clutch, on reaching the pawl, pushes it out of the way with its point, as shown in Fig. 43, till, on coming to the end of the stroke, its inclined back face passes the pawl, which then falls again to a horizontal position, as in Fig. 41, and is ready to push over the clutch again on the downward stroke.

Boring Large Wells in Chalk.—The two wells at Otterbourne near Southampton, already referred to, 6 feet in diameter, were bored through the Chalk in one operation, to a depth of 100 feet, by means of a specially designed tool carrying three chisels, the largest of which was

BORING CUTTER FOR CHALK WELLS.

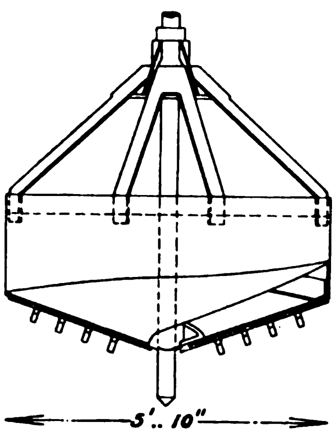
Fig. 44.



placed at right angles to the other two, Fig. 44. This cutter, weighing 1¼ tons, was attached at the end of a line of boring rods, 3 inches square, screwed together in lengths of 10 feet; and they were raised and dropped by being connected at the top with a cable actuated by a steam winch, a rotary motion being at the same time imparted to the rods and cutter so as to vary the position of the blows.[1] The chalk and flints were in this way rapidly broken up; and the *débris* was removed from time to time by a mizer lowered to the bottom, into which, on being rotated, the loose material was forced, and thus removed from the well, Fig. 45. The mizer weighed $1\frac{1}{4}$ tons, and was eventually used for rimering out the wells by means of cutters bolted on outside for the purpose. By these contrivances, under normal conditions, the boring of the second well to its full dimensions progressed at a rate of over 5 feet a day.

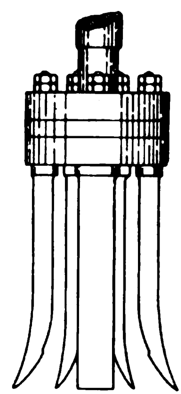
MIZER FOR REMOVING DÉBRIS.

Fig. 45.



CUTTER USED WITH ROPE FOR BORING WELLS.

Fig. 46.



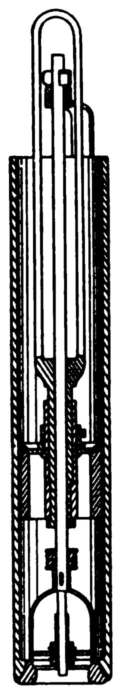
Modern Methods of Boring by Aid of a Rope.—Though the use of boring rods enables a more effective, and better directed blow to be delivered than the old Chinese method with a rope, worked up and down by a balanced pole or lever, the lifting of heavy rods from a considerable depth each time that a cutter has to be replaced by the cylindrical shell for removing the *débris* from the hole, and *vice versâ*, occupies a large amount of time. Accordingly, the employment of a rope has in some cases been reverted to for saving time, which being worked by a steam winch, rapidly raises and lowers the tools; whilst by a special ratchet arrangement, the cutter is turned slightly after each blow, the blows being delivered in rapid succession by the direct vertical action of a piston worked by steam in a cylinder.

[1] "The Wells and Borings of the Southampton Waterworks," W. Matthews, *Proc. Inst. C.E.*, vol. xc. p. 36, and plate 1, figs. 8 to 10.

In the Mather and Platt system, several chisels are fixed in a circular head for a large boring, Fig. 46; and a slight change in position is effected automatically after each blow.[1] This cutter can be lowered into the bore-hole at the rate of 500 feet a minute; and delivering about twenty-four blows a minute, it breaks up a sufficient quantity of rock in a short time for the introduction of the shell-pump; and the cutter is raised from the hole at the rate of 300 feet per minute. A special form of shell-pump has been devised, having a bucket working up and down in it, as in an ordinary pump, and closed at the bottom by a circular plate merely resting upon an annular seat, and capable of being raised bodily about 6 inches above its seat, so as to admit of large fragments of rock being drawn into the cast-iron cylinder, constituting the shell, when the bucket is raised, Fig. 47. The shell-pump is lowered and raised at the same rates as the cutter; and it is rapidly filled, when resting upon the *débris*, by working the bucket up and down two or three times.[2]

SHELL-PUMP FOR REMOVING DEBRIS.

Fig. 47.



BORING TOOLS USED WITH ROPE.

Fig. 48. Enlarging Shell.

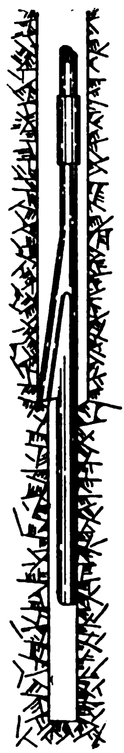


Fig. 49. Pointed Shell.

The boring of small holes has been largely effected in the United States by aid of a rope, especially for the purpose of procuring petroleum. The chisel, suspended from a rope, is raised and dropped by the action up and down of the working beam; and the chisel is turned slightly, and its depth adjusted, by the man in charge as it rises and falls. The hole formed by the chisel is followed up at intervals by a rimer for enlarging the hole to the requisite size,[3] Fig. 48; and the *débris* produced by the chisel and rimer respectively, is removed

[1] "The Present Practice of Sinking and Boring Wells," E. Spon, p. 150.

[2] *Ibid.*, p. 152.

[3] "Well-Boring for Water, Brine, and Oil," C. Isler, p. 144.

by cylindrical shells corresponding to the sizes of the holes, of the form shown in Fig. 49.

Boring with Diamond Drills.—All the systems of boring hitherto described depend upon the rock in the bore-hole being sufficiently broken up to be removed in some form of cylindrical shell, shell-pump, or mizer; whereas the rotary, circular diamond drill merely grinds out a circular ring, leaving a central, solid core of rock, which, when taken out, reveals the exact condition of the rock traversed as it exists *in situ*. Diamond drills, however, are not suited for boring in loose or soft strata, or in strata of variable character; but they are very valuable for boring in hard rock, especially where the depth to be traversed is considerable; and their economy varies in proportion to the size of the bore-hole and the hardness of the rock.

The diamonds used for this drilling are an amorphous black variety of the gem, of great hardness, found in the province of Bahia in Brazil, and some inferior qualities of Kimberley diamonds incapable of being cut into jewels.[1] These stones, though of no value for ornamental purposes, have greatly increased in price of recent years, owing to the difficulty of obtaining them. The diamonds are set at intervals in steel crowns fastened to hollow rods, which are revolved very rapidly; and when set for rimering out a hole, they have to be placed close enough together to attack the whole annular surface of the rock to be removed. The central core is removed at intervals by being gripped, broken off, and lifted by an annular steel spring attached to the crown, when the crown is raised by the boring rods. The ground-up rock is removed by forcing a stream of water down the hole, which also keeps the drill cool.

Concluding Remarks on Deep Wells.—Most cities and towns have at some period obtained subterranean supplies of water by means of wells. Very rapidly increasing centres of population have been obliged generally, in course of time, to seek other sources of supply, as the water that can be drawn from wells within the limited area usually available, is liable to prove insufficient. Nevertheless, cities which have been forced to seek more plentiful sources, such as rivers or impounding reservoirs, often retain deep wells for supplementing their supplies; whilst wells often suffice for the requirements of towns of moderate size. Wells, however, require to be carefully guarded from any chance of pollution, and, as far as practicable, from encroachments by neighbouring communities on the subterranean reservoirs from which the wells draw their water.

Artesian wells are necessarily far more limited in their extent than ordinary wells; but within their areas, which are very extensive in

[1] *Proc. Inst. C.E.*, vol. cxlvii. p. 317.

certain regions, they afford a very efficient and cheap means of water-supply.[1] These wells, however, still more than ordinary wells, cannot be multiplied indefinitely without seriously affecting the yield of previous wells; and all subterranean supplies are restricted to the available rainfall over the area of the outcrop of the permeable stratum from which they are drawn.

[1] "Artesian Wells as a Means of Water-Supply," W. G. Cox, Brisbane, 1895.

CHAPTER V.

LAKES AND STORAGE RESERVOIRS.

Purest water obtained from Lakes and impounded Streams in mountainous districts—Lakes and Storage Reservoirs compared, advantages of lakes, greater opportunities for storage reservoirs—**Lakes for Water-Supply**: Schemes proposed for London and Paris; Loch Katrine for supplying Glasgow, site of loch, storage and supply provided, extension; Thirlmere for supplying Manchester, storage and supply secured; Supplies from Great Lakes of North America, construction of intakes, provisions against pollution, silt, and ice, examples of pipe intakes, and of tunnel intakes; Remarks on Supplies from Lakes—**Storage Reservoirs**: object, method of forming, mitigate floods; Sites, in mountain valleys, requirements, proper position for dam, instance in Bear Valley, California, compared with Furens reservoir, objections to reservoirs in lower valleys, peculiar conditions at Assuan; Flow off a Drainage-Area, formula for, determination from gaugings and rainfall; Methods of Gauging Flow of Streams, through a rectangular opening, formula giving discharge, by measuring velocity of current; Compensation Water, object, different proportions imposed in relation to supply, volumes stipulated for rivers Etherow, Elan and Claerwen, Vyrnwy, and St. John's Beck from Thirlmere, benefits afforded; Amount of Storage required, dependent on longest drought, varying inversely with drainage-area and rainfall, number of days' supplies provided at Loch Katrine, Vyrnwy, Elan and Claerwen reservoirs, conditions and periods of minimum and maximum; Deposit of Silt in Reservoirs, instance of rapid accumulation, prospects in Assuan reservoir, in Spain and India, provisions against; separating weirs; Clearing Site of Reservoir, from vegetation, at Vyrnwy reservoir; Increase of Storage Capacity in Valleys, by chain of reservoirs, by high dam, example of first in Longdendale Valley, and of second in Croton Valley described, comparison of the two systems; Reservoirs for Storing Flood-Waters of Rivers, adoption of system for London at Staines, description of Staines reservoirs, proposed extension of system, merits and disadvantages of system, storage reservoirs in upper valleys of Thames tributaries preferable, prospect of London being deprived of suitable distant sites for storage if their acquisition is long delayed.

THE purest supplies of water are obtained from lakes in hilly districts, and from impounding reservoirs formed by dams enclosing the valleys of mountain streams, especially where the lands draining into them are devoid of habitations and culture. Moreover, the rainfall in mountainous districts is, under ordinary conditions, considerably greater than on lower ground; and as the hills are commonly formed of impermeable strata, and the slopes of their sides are steep, a large proportion of the rainfall flows down from them into the valley below. Accordingly, with a large available rainfall out of a considerable total fall, the flow of a given drainage-area is much greater in such regions than elsewhere; whilst the loss from evaporation, both over the land and the reservoir, is reduced by the comparative coldness of high altitudes. The catchment-basins of mountain streams are, indeed, necessarily very much smaller than those of rivers in the lower portion of their course; but lakes converted into reservoirs for water-supply, and artificial, impounding reservoirs, possess the very important advantage of storing up the surplus flow in flood-time for use during dry weather. These reservoirs of water, moreover, when situated in high, mountainous country, enjoy the further merits of being free from sources of pollution, and of being at a sufficient elevation above the district to be supplied, for the water to be conveyed by gravitation to the service reservoirs.

Lakes and Storage Reservoirs compared.—Lakes, by their very existence, prove that the strata forming their basin are thoroughly watertight, which is an essential condition for a reservoir. They also, in proportion to their superficial area, store up a large volume of water for every foot that their water-level is raised by a dam at their outlet; whereas the lower portion of a dam across a steep, narrow valley, stores up comparatively little water; for the contents of such a reservoir are quite small near the bottom, and increase notably with every foot of height near the top of the dam, finishing, in fact, with a similar increment of volume per foot raised at the top, as a lake, having originally a similar surface area as the full reservoir, commences with. This difference is forcibly illustrated by a comparison of the capabilities for storage of two sites in the Nile Valley, namely, at Assuan, where a reservoir for irrigation has been formed by a dam across the valley, about 1¼ miles in length, and at Lake Dembea, or Tsana, near the sources of the Blue Nile, on high land in Abyssinia. The water stored up by the Assuan dam as constructed, holding up a maximum head of water of 65 feet, amounts to 35,000 million cubic feet; whereas a low, short dam at the outlet of the lake, raising its water-level only 14 inches over an area of 1100 square miles, would store up the same volume of water. The reduction, moreover, of the height of the Assuan dam by a little less than one-fourth of that originally proposed, has halved the storage capacity of the reservoir, indicating the great value of a few feet additional height of dam in increasing the volume of water retained in an extensive reservoir.

Another advantage possessed by lakes for conversion into reservoirs, is the existence of a rocky barrier across their outlet, which is a cause of their existence; for the water discharged from them would have worn away any soft obstruction. In the case of storage reservoirs, rock is not always found near the surface at the site of a proposed dam; whereas a rocky foundation, convenient for a dam, is provided by nature at the outlet of lakes. When a storage reservoir is filled by a stream bringing down detritus in its rapid flow in flood-time, deposit occurs in the still waters of the reservoir, and gradually reduces its capacity; whereas the basin of a lake provides a large space for the accumulation of sediment, which, accordingly, does not interfere with the capacity of the impounded lake above its normal water-level.

On the other hand, very few cities or large towns are so situated as to have a lake within reach, available for supplying them with water; whereas numberless sites exist in hilly valleys, remote from habitations, where reservoirs can be created for storing up water. Lakes, moreover, situated in lower, accessible localities, are liable to be subjected to the limitations of vested interests, and to be exposed to sources of pollution.

LAKES FOR WATER-SUPPLY.

Schemes have occasionally been suggested for supplying large cities with water from lakes at a considerable distance. Thus it was proposed many years ago to supply London from the Cumberland Lakes, 280 miles distant;[1] whilst more recently the Lake of Neuchâtel was suggested as a source of water-supply for Paris, which would have necessitated a conduit 300 miles in length.[2] The great length, however, of the conduits needed, the long tunnels which would be required for carrying these conduits across the water-partings of the river-basins to be traversed, and the opposition that would be naturally raised against depriving large centres of population of sources of water-supply, to which, owing to their relative nearness, they appear to possess a prior claim, have hitherto prevented schemes of such a grandeur being carried out. Lakes, nevertheless, have been utilized with great advantage for supplying large towns at a less distance, though situated in a different river-basin; and the requisite storage has been provided by raising the water-level of the lake, though this raising of the water may be quite small in the case of a large lake.

Loch Katrine for supplying Glasgow.—Though Glasgow is

[1] "On the Water-Supply of the Metropolis," E. Frankland, *Proc. of the Royal Institution of Great Britain*, 1867, vol. v. p. 111.

[2] *Annales des Travaux Publics*, Paris, 1888, p. 1987.

situated on the tidal Clyde, close to the sea-level, there is very hilly country in its immediate neighbourhood; and as this hilly region is not far from the west coast, and open to the Atlantic, its yearly average rainfall is between 70 and 90 inches. In the midst of this wild, rugged district, a natural reservoir of very pure water is provided by Loch Katrine, 3000 acres in extent, and only 35 miles from Glasgow. By merely raising the water-level of the lake 4 feet above its original summer level, and arranging for lowering it a maximum of 3 feet below the same level at the close of a long period of drought, making a variation of level of only 7 feet altogether, a storage was provided of $5687\frac{1}{2}$ million gallons of water, allowing of a supply to Glasgow of 50 million gallons a day.[1] The lake, at its lowest level, being 360 feet above sea-level, enabled a fall of 10 inches per mile to be given to the conduit, $25\frac{3}{4}$ miles long, conveying the water from the lake into the service reservoir at Mugdock for distribution to Glasgow.

The area of the land draining into Loch Katrine is 22,800 acres; and out of a yearly rainfall averaging about 80 inches, the available rainfall in this damp, mountainous district, flowing off the above drainage-area into the lake, amounts to considerably more than the volume originally drawn from the lake. Provision, accordingly, has been recently made for drawing a further supply from Loch Katrine, by raising its top water-level 5 feet higher, affording a possible storage capacity of 9849 million gallons.[2] The additional volume of water thus secured is to be supplemented by 10 million gallons a day discharged through a tunnel into Loch Katrine from Loch Arklet, in which a storage capacity of 2059 million gallons can be obtained by raising its water-level 25 feet by a dam across its outlet, whose natural drainage is into Loch Lomond; and these supplies, together with a large volume to be drawn from the River Leny, provides a further supply of 50 million gallons a day, conveyed through a second conduit, to meet the rapidly increasing requirements of Glasgow.

Thirlmere for supplying Manchester.—The lake district of Cumberland and Westmorland is the rainiest in England, with its high hills near the west coast, exposed to the moist westerly winds coming off the Irish Sea, and almost direct from the Atlantic Ocean. Thirlmere, one of the lakes in this district, was selected for an additional source of water-supply for Manchester, when its large increase of population indicated that the original supply, from the chain of reservoirs in

[1] "Aqueduct," "Encyclopædia Britannica," 9th edition, vol. ii. p. 225.

[2] *The Engineering Magazine*, Sept., 1899, "The Water-Supply of the City of Glasgow," B. Taylor; and *The Scientific American Supplement*, vol. l., New York, August 18, 1900.

the Longdendale Valley, would before long prove insufficient for the wants of the city.

As Thirlmere is a comparatively small lake, with an area of $328\frac{1}{4}$ acres, provision has had to be made for raising its level a maximum of 50 feet, thereby affording a storage capacity of $8130\frac{2}{3}$ million gallons above its original normal level, which is not permitted to be lowered for purposes of supply.[1] This volume of storage is equivalent to $32\frac{2}{3}$ inches of available rainfall from the drainage-area of 11,000 acres. Since the rainfall on the Thirlmere watershed ranges between 60 and 100 inches in dry years, and between 80 and 137 inches in wet years, and averages about 71 inches in three consecutive very dry years, and the surface is steep, rocky, and devoid of peat, the flow off is unusually large; and, therefore, the available rainfall would be in excess of the depth mentioned above, required for filling the space provided. The storage is adequate for ensuring a supply of 50 million gallons a day, in addition to the volume required for compensation. The area of the lake, when filled to its top level, will be increased to 793 acres. The lake at its normal water-level is 533 feet above sea-level, and 180 feet above the service reservoir at Prestwich, into which it delivers its supply; and, accordingly, as the length of the conduit from Thirlmere to Prestwich is $95\frac{4}{5}$ miles, there is an average fall of 1 foot $10\frac{1}{2}$ inches per mile available for the flow.

The works at Thirlmere form in reality a storage reservoir on the top of the original lake, which, unlike Loch Katrine, does not furnish any portion of the supply, the whole being drawn from the volume stored above the natural water-level of the lake. As the loss from evaporation over the surface of a lake is proportionate to its area, and diminishes with the altitude, Thirlmere is favourably situated in this respect; for its area is small in proportion to its increased depth, and it is at a good elevation, in addition to being in a very rainy district.

Supplies from the Great Lakes of North America.—The chain of large lakes stretching for a long distance across the eastern portion of North America, constituting the upper portion of the basin of the St. Lawrence, affords a practically inexhaustible supply of fresh water. The favourable nature of the country, however, bordering these lakes, and the great opportunities they offer for navigation, have led to numerous towns being established on their shores, which, by discharging their sewage into the lakes, pollute the waters from which, at the same time, they draw their supplies. Large towns have, accordingly, been obliged to extend their intakes considerable distances into the lakes, in order to procure an uncontaminated supply.

[1] "The Thirlmere Works for the Water-Supply of Manchester," G. H. Hill, *Proc. Inst. C.E.*, vol. cxxvi. p. 3.

The intakes for the water-supply of these lake cities are carried into the lake, either by laying iron or steel pipes along the bed of the lake, or by driving a tunnel through the strata underlying the lake, to the requisite distance from the shore, and then sinking a shaft in the lake down to the outer end of the tunnel, protected by cribwork, through which water is drawn from the lake.[1] The sand and silt on the flat, shallow bed of the lakes near the shore, are stirred up by the waves during storms in these freshwater seas; and floating ice accumulates in these places in the winter. The pipes have, consequently, to be turned up at the inlet, and carried into moderately deep water, to avoid the introduction of sediment, and the blocking up of the inlet by ice. The wooden cribs protecting the inlet are often submerged to obviate the blocking up of the inlet by ice; but sometimes the cribwork round the shaft leading to a tunnel, is carried up above the water-level, to obtain greater control of the supply; and then the accumulation of ice round the cribs, tending to close the inlet, has to be guarded against.

Toronto draws its supply from Lake Ontario, through a steel intake with a submerged timber crib at the inlet; Syracuse obtains its supply from Lake Skaneateles, one of the smaller lakes connected with Lake Ontario, through a steel tube, $4\frac{1}{2}$ feet in diameter, with a submerged crib at the inlet, 16 feet square and 12 feet high, situated in 38 feet depth of water and 6410 feet from the shore; and Rochester takes its supply from Lake Hemlock in the same manner, the steel tube in this case being 5 feet in diameter and 1550 feet long, and the timber crib 10 feet high, with its top submerged 26 feet below the surface of the lake. Erie is supplied from Lake Erie by a cast-iron intake, 5 feet in diameter, extending 3500 feet into the lake, and terminated by a submerged crib 40 feet square and 9 feet high.

Tunnels, on the contrary, have been driven under the bed of the lakes for the intakes of Milwaukee and Chicago from Lake Michigan, and of Cleveland from Lake Erie. For Milwaukee, the tunnel is $7\frac{1}{2}$ feet in diameter, and extends 3200 feet from the shore, where a crib has been erected; and from thence two 5-feet pipes have been carried into a depth of 60 feet, and terminate with submerged cribs. Four tunnels, from 6 to 8 feet in diameter, furnish the intakes for the water-supply of Chicago, with shafts surrounded by cribs emerging from the lakes at their termination. The first two tunnels were carried two miles into the lake; but the later ones have been taken out 4 miles from the shore, into a depth of 46 feet, to ensure a pure supply. Moreover, Chicago has now diverted its sewage from Lake Michigan into the Mississippi Valley, by the construction of the Drainage Canal from

[1] "The Intakes of the Great Lakes," T. W. Barrally, *The Railway and Engineering Review*, Chicago, 1898, vol. xxxvi. p. 428.

Chicago into the Desplaines River. The longest lake intake hitherto constructed, is the tunnel, 8 feet in diameter, carried under the bed of Lake Erie to a distance of 26,000 feet, or very nearly 5 miles out from the shore, for supplying Cleveland with water from the lake.

Remarks on Supplies from Lakes.—The purity of the water draining into lakes in remote, mountainous districts, devoid of mines, is generally fully assured; and it can readily be preserved from any chance of pollution in the future, by acquiring the control over the drainage-area, consisting of lands of little value. Ample storage is easily provided by a moderate raising of the water-level of the lake, by a dam of moderate height constructed under favourable conditions; the natural fall is generally ample for the conveyance of the supply by gravity; and the lake itself, below its original water-level, provides a large space for the reception of any sediment discharged into the lake, which is rapidly deposited on entering the still waters of the lake, as in a vast settling pond, so well illustrated by the muddy Rhone on entering the Lake of Geneva.

The problems presented in drawing water-supplies from the Great Lakes, are totally different from those pertaining to lakes in hilly districts serving for storage reservoirs. In the Great Lakes, no question of storage is involved; but pure water has to be sought at a distance from the shore, where it has escaped pollution; and the inlet of the intake has to be placed at such a depth as not to be exposed to the introduction of sand and silt stirred up from the bottom of the lake by waves, or liable to be blocked up by ice from above.

STORAGE RESERVOIRS.

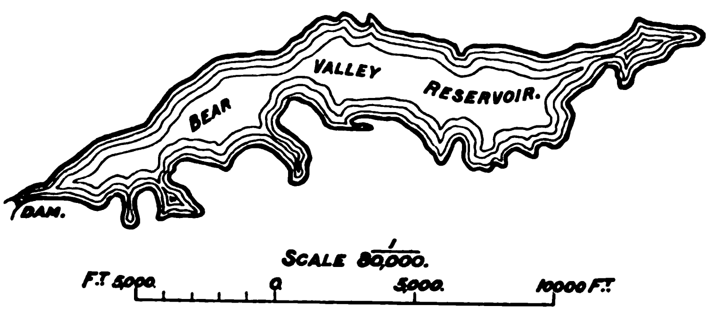
The superabundant flow of mountain streams in flood-time may be very advantageously utilized for the supply of water to towns, by being stored in an artificial reservoir formed by a dam erected at a suitable site across the valley of the stream. The water thus collected into an artificial lake, possesses the same purity as the water of natural lakes similarly situated; but a much higher dam is generally required to obtain the same amount of storage, owing to the small capacity of the reservoir near the bottom of the valley as compared with a lake; and any sediment brought down by the stream, and deposited in the still waters of the reservoir, reduces the space available for storage. A reservoir, by impounding a considerable portion of the flood-waters, mitigates the floods in the valley below, a benefit which is always naturally conferred on any river passing through a lake in its course, by the regulating influence of the lake on its flow.

Sites for Storage Reservoirs.—The valleys of mountain streams draining uninhabited and uncultivated districts, afford the most favourable sites for impounding reservoirs, owing to their freedom from pollution, and because, from their situation, they are exposed to a heavy rainfall, a large proportion of which, falling on very sloping, impermeable strata, finds its way into the watercourse draining the valley. The area to be covered by the reservoir must be free from fissures, through which water might escape; and the surface strata should be devoid of any sharp dip, and adequately impermeable and continuous throughout to be perfectly watertight, or capable of being readily made impervious in small defective places by layers of clay puddle. A narrow part of the valley should, if possible, be selected for the dam, so as to reduce its length, and a site where the valley widens out considerably above the gorge for some distance, so as to provide an extensive area for the reservoir.

The Bear Valley reservoir, in California, furnishes a typical example of the selection of a very favourable site of this kind; for by con-

BEAR VALLEY RESERVOIR, CALIFORNIA.

Fig. 50.



structing a masonry dam in a narrow rocky cañon, with a length along the top of only 300 feet, and a maximum height of 64 feet, a reservoir has been formed with an area, when full, of about 1500 acres, and containing about 10,855 million gallons, or $1742\frac{1}{2}$ million cubic feet of water, used for irrigation,[1] Fig. 50. As a contrast to this reservoir, a high dam in a steep, narrow valley near the upper extremity of a river-basin, may only impound a comparatively small volume of water, as exemplified by the lowest of the two masonry dams across the River Furens, a tributary of the Loire, retaining a maximum head of water

[1] "Reservoirs for Irrigation, Water-Power, and Domestic Water-Supply," J. D. Schuyler, New York, 1901, pp. 164, 166, and 175.

of 164 feet, which has formed a reservoir having a capacity of only 363 million gallons.

It would be possible, under certain conditions, to form large reservoirs in lower parts of valleys by long, and somewhat low embankments, on the principle of the irrigation tanks in India;[1] but such reservoirs flood a considerable area of land, and, being shallow, are exposed to a large proportionate loss from evaporation; whilst it would be difficult, in such a position, to secure the country draining into the reservoir from sources of pollution. An extensive reservoir, has, indeed, been created at Assuan, by constructing a dam right across the course of a large river in the middle portion of its basin, where the very moderate fall of the river for a long distance above the dam, enables a large volume to be impounded by a dam of moderate height above the river-bed. The land, however, flooded in this instance was of little or no value; whilst it has been necessary to build a very long dam, and to provide adequate sluice openings through it to discharge the whole flow of the Nile in flood-time.

Flow off a Drainage-Area.—Before constructing a storage reservoir, it is necessary to ascertain the probable flow off the ground draining into the proposed site, or, in fact, the available rainfall of the catchment-area of the reservoir contemplated. This volume may be estimated by deducting the assumed loss, due to evaporation and percolation, appertaining to the special district and its geological formation, from the mean rainfall reduced by one-fifth to correspond with the average rainfall of the three driest consecutive years. From these considerations, the daily supply in gallons, G, that can be drawn from a reservoir filled from a gathering ground whose area in acres is A, and over which the mean rainfall in inches is R, and the loss from evaporation, absorption, and percolation, is E, is given by the formula,[2] $G = 62{\cdot}15\ A\ (\frac{4}{5}R - E)$.

A still more reliable method is to gauge the discharge of the stream which it is proposed to impound, as this gives the actual flow off the drainage-area; but unless these gaugings can be conducted for a considerable period, so as to include a time of long drought, which is rarely feasible, the results of the gaugings must be compared with the corresponding rainfalls, so that the discharge during three consecutive years of minimum rainfall may be deduced from them.

Methods of Gauging the Flow of Streams.—The ordinary method of gauging the flow of small streams is by damming up the

[1] "Rivers and Canals," L. F. Vernon-Harcourt, 2nd edition, 1896, vol. ii. p. 419.

[2] "Water-Supply," W. Pole, "The Theory and Practice of Hydro-Mechanics," *Inst. C.E.*, 1885, p. 44.

stream, by placing a series of boards horizontally across it, at a suitable place, so as to form a pool above them, and allowing the discharge to take place through a rectangular opening, or notch, cut out in the dam, with knife-edged sides, or formed in a metal plate fastened to the weir, Figs. 51 and 52. The height, H, of the water above the bottom of the opening, is measured for convenience from the water-level in the

RECTANGULAR NOTCH FOR GAUGING STREAMS.

Fig. 51.—Sketch of Stream and Notch.

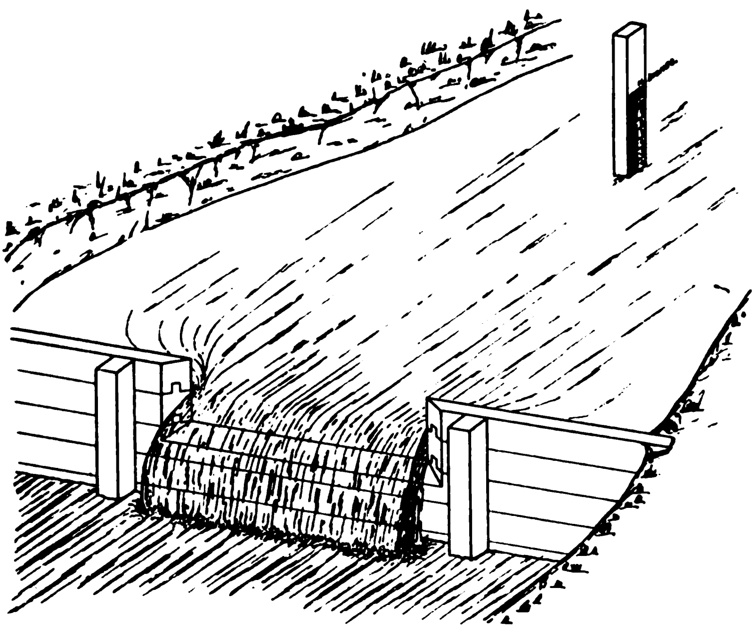
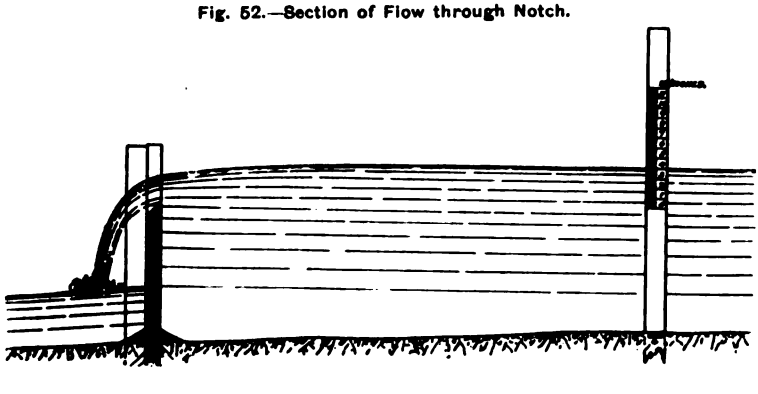


Fig. 52.—Section of Flow through Notch.



pool above the place where it begins to drop, Fig. 52; and the contraction in width of the water, in passing through the opening, from the breadth, B, of the opening has been found to be $B - 0{\cdot}2H$. Then, if c is the coefficient of the reduction in depth of the stream passing over the weir, the height of this stream is cH, and its mean velocity is $\frac{2}{3}\sqrt{2gH}$; and the discharge, D, is given by the formula, $D = \frac{2}{3}\sqrt{2gH}cH(B - \frac{1}{5}H)$. Inserting the value of g, and $0{\cdot}622$ for c, as being the mean value found by experiment, the above equation, giving the discharge in cubic feet per second, becomes $D = 3{\cdot}33\sqrt{H^3}(B - \frac{1}{5}H)$.

The discharge of larger streams is obtained by finding the depths of the stream at regular intervals across it, at a suitable place, and thence calculating its sectional area, and then measuring the velocity of the stream by a series of double floats, tube-floats, or weighted velocity-rods, along a straight, uniform reach, or by taking the velocities at various points along the measured cross section with a current-meter or a gauge-tube, so as to obtain the mean velocity.[1] This mean velocity multiplied by the cross section, gives the discharge of the stream.

Compensation Water.—If no restrictions were imposed on the impounding of a stream, the whole of its flow would be collected in dry weather; and no water would pass down the channel below the dam. On the contrary, however, in return for the privilege of impounding the flood-waters, the obligation is imposed of increasing the flow of the stream in times of drought below the dam, by discharging a sufficient volume of the stored-up water into the stream to ensure that its flow shall never fall below a fixed amount, depending on the extent of the drainage-area supplying the reservoir, and the requirements of the riparian proprietors on the stream below. When important manufactories or fisheries are established lower down the river, compensation water may have to be supplied to the stream from the reservoir during the whole period that there is no water flowing over the waste weir, to an extent equivalent to one-third of the daily volume drawn off for water-supply; and, occasionally, when mills exist on the river impounded, a still larger compensation was formerly granted. In other cases, a smaller proportion of compensation water is stipulated for, according to circumstances, a minimum of about one-ninth of the daily water-supply drawn from the reservoir being allotted where the riparian rights are of little importance.

As mills existed on the lower part of the River Etherow when the higher part of this river was impounded many years ago for the water-supply of Manchester, out of a total daily supply of 40 million gallons a day secured by storage in a series of reservoirs, 13 million gallons,

[1] "Rivers and Canals," L. F. Vernon-Harcourt, 2nd edition, 1896, vol. i. pp. 23 to 39.

or nearly a third of the whole supply, had to be discharged into the river in dry weather as compensation water. The reservoirs in course of construction in the valleys of the rivers Elan and Claerwen, in Wales, tributaries of the River Wye, for providing Birmingham eventually with a water-supply of about 75 million gallons a day, will have to furnish 27 million gallons a day for compensation water to these rivers during dry weather, or a little over one-third of the proposed supply, for the sake of the fisheries.[1] The Vyrnwy reservoir, capable of supplying Liverpool with 40 million gallons of water a day, has to provide, on the average, 13½ million gallons a day of compensation water when required, equivalent to one-third of the supply secured. The raising of the water-level of Thirlmere 50 feet, provides for the discharge of 5½ million gallons a day into St. John's Beck, a tributary of the River Derwent, in addition to the 50 million gallons a day to be eventually supplied to Manchester, the compensation water in this instance being only one-ninth of the water-supply obtained.

The creation of a reservoir in the valley of a torrential stream, in conjunction with a supply of compensation water, confers a double benefit on the riparian interests below; for the reservoir, in impounding the flood-waters, and diverting a considerable volume of the discharge of the stream during floods for water-supply, mitigates the injurious effects of floods in the lower valley; whilst the compensation water, which is obliged to be discharged throughout a drought, ensures that the flow of the stream will never fall below the stipulated amount during the longest period of dry weather. In fact, the discharge of the stream is reduced at periods when it is liable to be inconveniently large, and increased and secured when it might otherwise fail altogether.

Amount of Storage required in Reservoirs.—In order that a storage reservoir may ensure the provision of a definite daily supply, its capacity must be sufficient for it to be able to continue supplying the requisite daily volume throughout the longest period during which there may not be enough rain over the drainage-area for the reservoir to be appreciably replenished. The daily volume required to be furnished must, in addition to the water-supply, include the compensation water and loss from evaporation; and this volume, multiplied by the number of consecutive days during which practically no flow off the gathering ground may reach the reservoir, gives the required capacity of the reservoir for storage. This period of drought varies with the locality, being much shorter in very rainy districts, and over extensive drainage-areas, than in dry places with limited gathering grounds, where the variations from the mean rainfall are greater; and the longest droughts occur in those tropical regions where the periodical rains are very

[1] *Proc. Inst. C.E.*, vol. cxxvi. p. 71.

irregular, failing sometimes to a great extent during the rainy season, and occasionally in some parts during two rainy seasons in succession.

In the very wet district of Loch Katrine, the storage originally provided for the water-supply of Glasgow amounted to 113 days' supply, neglecting in this case evaporation over the surface of the lake, which source of loss was always in operation, and has not practically been modified by the small raising of the lake; whilst the compensation water was amply provided for, by raising the levels of Loch Venachar and Loch Drunkie in the same neighbourhood. Thirlmere is also situated in a very rainy district; but as its drainage-area is only about half of that of Loch Katrine, and its rainfall slightly less high, and, consequently, the fluctuations in the rainfall over its gathering ground are liable to be greater, a storage equivalent to about 145 days' discharge of water-supply and compensation water, has been provided. At the Vyrnwy reservoir, with a capacity of 12,131 million gallons, an area of 1121 acres when full, and an average rainfall of 68 inches over the drainage-area of 23,200 acres,[1] the storage, making allowance for loss from evaporation, amounts to about 224 days' supply, with an average daily drain upon the reservoir of $53\frac{1}{2}$ million gallons for water-supply and compensation water. The gathering ground for the new water-supply of Birmingham, has an average rainfall of about 70 inches, slightly larger than that on the Vyrnwy drainage-area; whilst its area of 45,560 acres is nearly double that of Vyrnwy; and, therefore, it has been possible to rely on a smaller storage in proportion to the supply.[2] In the first instance, when the first instalment of 27 million gallons a day is furnished for the supply of Birmingham, together with the same volume of compensation water, the storage provided will be adequate for a daily discharge of 54 million gallons for about 200 days; but when the eventual supply of 75 million gallons a day has to be collected off the whole drainage-area, in addition to the 27 million gallons of compensation water, or 102 million gallons daily, the storage of 18,000 million gallons in six reservoirs in the Elan and Claerwen valleys, would only suffice for 176 days of the above discharge, without allowing for loss from evaporation.

In very wet districts, and where the gathering ground is large, a storage in an impounding reservoir adequate to provide for the full daily discharge for about 110 consecutive days, may be regarded as a minimum; and in a fairly dry district, a storage for 250 to 300 days may be required in a climate like that of Great Britain. In tropical countries, where the yearly rainfall is irregular and uncertain, as in

[1] "The Vyrnwy Works for the Water-Supply of Liverpool," G. F. Deacon, *Proc. Inst. C.E.*, vol. cxxvi. pp. 27 and 121.

[2] *Ibid.*, pp. 71 to 73.

certain parts of India, it may be necessary to make provision for a total failure of the rains in one rainy season, and, consequently, to make the storage sufficient for two dry seasons;[1] whilst in some parts the rainfall is so capricious that a storage adequate for three years' supply has to be provided for, as effected in the case of the reservoirs for the supply of San Francisco.[2]

Deposit of Silt in Reservoirs.—Rain flowing off bare, steep, hilly slopes is often remarkably clear; but sometimes streams bringing down considerable quantities of detritus, have to be impounded in a reservoir, which results in the accumulation of deposit, and a reduction in the capacity of the reservoir. Under unfavourable conditions, reservoirs have, indeed, occasionally been filled up with silt. Thus, for instance, the reservoir formed by the Austin dam across the Colorado River in Texas, having a capacity of 2330 million cubic feet, and from which all the flood discharge of the river passes over the dam, was reduced 41½ per cent. in capacity, by the deposit in it of 968 million cubic feet of silt from the river, in four years from its first filling.[3] This rapid rate of deposit, however, is attributed to the small size of the reservoir in proportion to the discharge of the river, which is estimated to have amounted in those four years to about 160 times the capacity of the reservoir. Experience alone will prove how far the filling of the Assuan reservoir during the winter months, when the Nile is much less heavily charged with silt than at the height of the flood, and the flow of the river in flood-time through the 180 sluices in the dam, will prevent silt accumulating at the sides and higher parts of the reservoir. Doubtless the flood discharge will clear out the channel of the river, and the adjacent portions of the reservoir; but there seems to be considerable probability that deposit will occur in the still water at the sides and upper parts of the very long reservoir, beyond the reach of the scouring current during the annual flood.

Many reservoirs in Spain and tanks in India, storing up water for irrigation, have been encumbered by deposit; and in some instances, notwithstanding the stirring up the mud from the bottom, so as to be removed in suspension by the current in flood-time, and scouring out the reservoirs by the discharge through low sluiceways, some reservoirs and tanks have in time become so filled up with sediment as to be useless for storing up water.

Reservoirs for water-supply to towns generally draw their supplies

[1] "Water-Supply; Rainfall, Reservoirs, Conduits, and Distribution," A. R. Binnie, 2nd edition, p. 36.

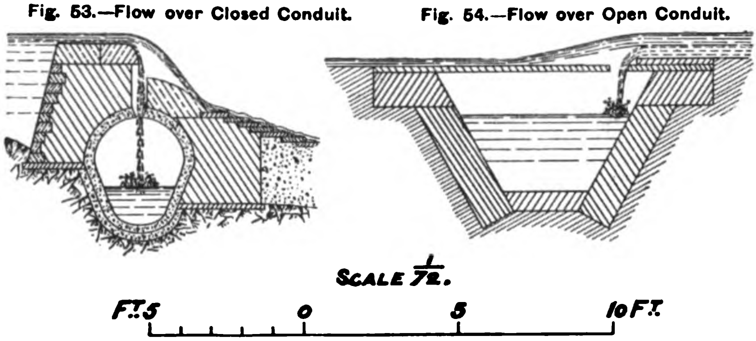
[2] "Water-Supply Engineering," A. Prescott Folwell, New York, 1900, p. 110.

[3] "Reservoirs for Irrigation, Water-Power, and Domestic Water-Supply," J. D. Schuyler, New York, 1901, pp. 245–246.

from streams which, flowing off the steep slopes of primitive rocks in hilly districts, are tolerably free from sediment; but some of these streams even are turbid in flood-time. To avoid the deposit in the reservoir which the inflow of this sediment-bearing stream would occasion, it is sometimes practicable to form a channel at the side of the reservoir leading into the stream below, into which the stream may be diverted when in flood. When the construction of a channel at the side of the reservoir for the discharge of the flood-waters would be too costly, a small reservoir, or settling pond, can be formed above the upper end of the main reservoir, into which the stream is led during floods, and, depositing the greater portion of its sediment, especially the heavier portions which are carried along the bed of the stream, overflows in a comparatively clear condition into the large reservoir. The deposit

SEPARATING WEIRS FOR STREAMS.

Fig. 53.—Flow over Closed Conduit. **Fig. 54.—Flow over Open Conduit.**



thus intercepted is readily removed periodically from this small, shallow, settling reservoir; and the main reservoir is thereby preserved from accumulations of deposit.

Small streams which become turbid, or discoloured by peat in very rainy weather, but which may be valuable in feeding a reservoir, or in augmenting the flow in a conduit on its way from the reservoir to the town to be supplied, can have their clear flow intercepted for increasing the supply, by means of a separating weir; whilst the turbid or discoloured flow is discarded, and allowed to run to waste, or into a lower compensation reservoir. Different forms of separating weirs have been adopted, two of which are shown in Figs. 53 and 54; but they all depend for the separation, on the more rapid flow of a stream in flood than under ordinary conditions. Thus the stream when flowing gently, falls through an aperture into the closed conduit, Fig. 53,[1] or open channel,

[1] "Lectures on Water-Supply at the School of Military Engineering, Chatham, in 1877," A. R. Binnie, p. 33, and plate 27.

Fig. 54;[1] but when it becomes turbid, or coloured by peat, and has a rapid discharge, it leaps over the opening and runs to waste, or is diverted into the waste-weir channel below the dam.

Clearing the Site of a Reservoir.—Before filling a storage reservoir with water, it is advisable to clear off all vegetation from the site, which, if fairly dry, may be most easily effected by burning, with the object of preventing the water being polluted by decaying vegetation during the first year or two. A small village was originally situated on the site of the Vyrnwy reservoir; and the houses and church had to be pulled down and rebuilt out of reach of the reservoir, and the contents of the churchyard removed.

Increase of Storage Capacity in Valleys.—There are two ways in which the store of water in hilly valleys may be increased, provided the flow off the drainage-area is sufficient, namely, by constructing a series of reservoirs in steps up the valley, as adopted for the original water-supply of Manchester in the Longdendale Valley, or by increasing the height of the dam forming the reservoir, and, if necessary, building it lower down the valley, so as largely to increase the capacity of the reservoir, as accomplished by the New Croton dam for augmenting the water-supply of New York. An increase in the height of a reservoir dam naturally augments the capacity of a reservoir out of all proportion to the additional height, as noted on p. 81, and is well illustrated by the Gileppe masonry dam, retaining a head of water of 147¾ feet, which was built near Verviers in Belgium, in preference to four dams 95 feet high, for providing the same storage, as at first proposed. A high dam, however, has, for perfect security, to be constructed of masonry, which involves a foundation on sound rock, not always obtainable; and, therefore, the construction of a very capacious reservoir is not always practicable, even when the conformation of the valley is suitable for it.

The original storage for the water-supply of Manchester consists of a chain of seven reservoirs, impounding the flow of the River Etherow and its tributaries by embankments, from 70 to 100 feet high, across the Longdendale Valley,[2] Figs. 55 and 56. These reservoirs, when full, have a combined area of 500 acres, and a total capacity of 4233 million gallons; and they are supplied by the flow off a drainage-area of 19,300 acres, consisting of the lower portion of the Coal-Measures. Some settling ponds were formed by weirs across the river and some streams, for intercepting any sediment brought down by floods; and a channel at one side of the valley discharges the surplus flood-waters, Fig. 55. The two bottom reservoirs, which are at too low an elevation to discharge

[1] "History and Description of the Manchester Waterworks," J. F. Bateman, p. 128, and plate of Separating Weir at Crowden, Brook.

[2] *Ibid.*, J. F. Bateman, p. 109, and plans.

STORAGE RESERVOIRS IN LONGDENDALE VALLEY FOR MANCHESTER.

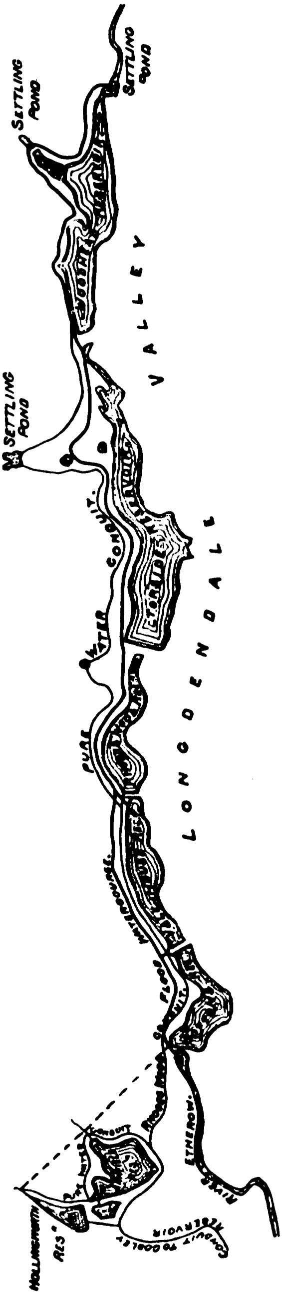


Fig. 55.—Plan of Reservoirs.

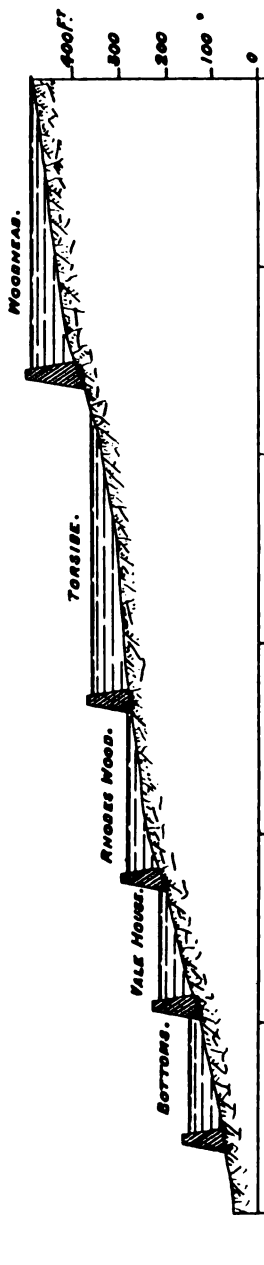


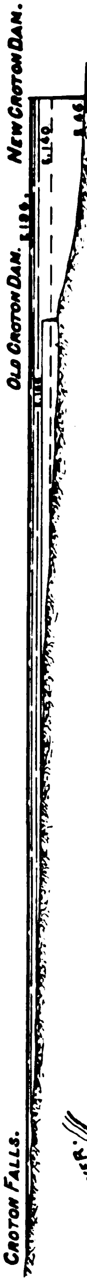
Fig. 56.—Longitudinal Section of Reservoirs and Dams.

into the conduit supplying Manchester, store up the discoloured water passing over the separating weirs, and furnish the compensation water which has to be discharged into the river below the reservoirs for the use of mills.

The original Croton dam, across the Croton River, formed a reservoir storing up 2000 million gallons for the supply of New York; whereas the New Croton dam, erected $3\frac{1}{10}$ miles lower down the valley, raised to a maximum height of 163 feet above the bed of the river, provides a reservoir which, at its highest water-level, submerges the old reservoir to the extent of about 34 feet, and stretching both considerably further up the valley and lower down, has a capacity of 32,000 million gallons, with an area of 3360 acres,[1] Figs. 57 and 58. The drainage-area of the Croton River above the dam is $360\frac{2}{8}$ square miles, or 230,656 acres, over which the average annual rainfall is $48\frac{1}{2}$ inches, half of which is estimated to reach the river. On the completion of all the reservoirs in the Croton watershed, the total available storage, including the lakes and ponds, as shown in Fig. 58, will amount to 72,633 million gallons, and ensure a daily supply of 280 million gallons to New York.

Whilst Manchester and Liverpool have, in each case, secured a great increase in their water-supplies by the formation of a single large storage reservoir, and New York is mainly relying on the vast increase in the Croton reservoir for the necessary addition to its supply, Birmingham has obtained control of a large watershed in the upper part of the basin of the River Wye, in the valleys of two of whose tributaries six reservoirs are to be successively constructed, as the demand for water increases, by means of concrete dams from 98 to 128 feet in height. This system of constructing a series of reservoirs gradually as the necessity for them arises, on the principle originally adopted for Manchester in the Longdendale Valley, with works extending over a period of nearly thirty years, though involving a larger amount of work for the requisite dams, and a greater loss from evaporation over a larger surface of water, than for a single high dam with a deep reservoir, possesses the advantage, even in valleys where a single reservoir of large capacity might be constructed, of proportioning the extent of the works to the immediate wants of the community to be supplied. With a very capacious reservoir, on the contrary, designed to be adequate for the wants of a rapidly increasing population many years hence, the reservoir with its dam and accessory works, and also the tunnels and some other works for the conduit, have to be completed at the outset; and only the laying of some of the lines of pipes constituting the conduit, can be deferred till they are required. The system, however, to be chosen must depend on the

[1] "Water-Supply of the City of New York," E. Wegmann, pp. 203-207, and plate 54.



NEW CROTON RESERVOIR.

Fig. 57.—Longitudinal Section of Old and New Reservoir.

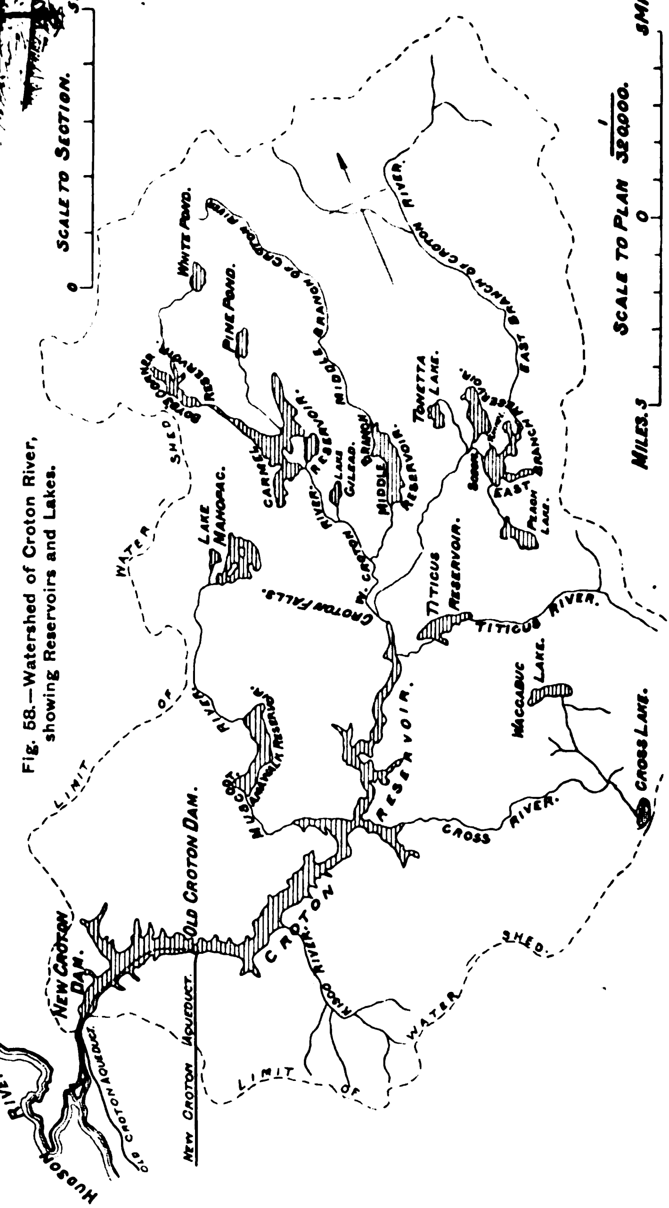


Fig. 58.—Watershed of Croton River, showing Reservoirs and Lakes.

special circumstances of each particular case, and the conditions of the locality.

Reservoirs for storing the Flood-Waters of Rivers.— London, with its rapidly increasing population, has been gradually drawing larger daily supplies from the River Thames; and if this practice had been continued unchecked, an undue proportion of the summer flow of the river would have been abstracted in dry weather in the near future. The London water-companies have the right to draw 130 million gallons of water daily from the Thames, however small the flow of the river may be; and they further are permitted to take an additional volume of 55½ million gallons a day from the river (of which 35 million gallons are drawn through the Staines conduit from above Bell Weir lock) so long as the flow of the river exceeds 200 million gallons a day over Teddington Weir. With the view, however, of enabling a still further volume to be taken from the Thames without detriment to its maintenance, it has become necessary to resort to the storage of its flood-waters, in reservoirs formed in its valley by being surrounded by embankments.

The storage reservoirs proposed for the supply of London, of which the two reservoirs at Staines constitute the first instalment, are not impounding reservoirs intercepting the flow of the river by a dam across its valley, but resemble large tanks enclosed by embankments, into which the water brought from the river by an open conduit is pumped, and from which it can be drawn as required, especially during droughts when the water drawn from the river is restricted to 130 million gallons a day.

The two adjacent reservoirs constructed at Staines are surrounded by earthen embankments exceeding 4 miles in length, raised from 21 to 39 feet above the ground, with a width of 24 feet at the top, and a maximum width of 214 feet at the base, Fig. 59. These embankments have been formed by the earth excavated from the site of the reservoirs, and rendered watertight by a central, puddle wall, 6 feet wide at the top, increasing in width to 7 feet at the ground-level, and carried down through the superficial bed of gravel, from 14 to 35 feet thick, into the London Clay.[1] Four pumping engines, with a fifth in reserve, can deliver into the reservoir the maximum volume of 65 million gallons a day, raised from the conduit drawing the water from the Thames; and two water-towers, one in each reservoir, furnished with valves, regulate the supply and discharge. The two reservoirs, only separated by a central embankment, have a total area of 421 acres, and a capacity of 3,300 million gallons; and their cost has been about £800,000.

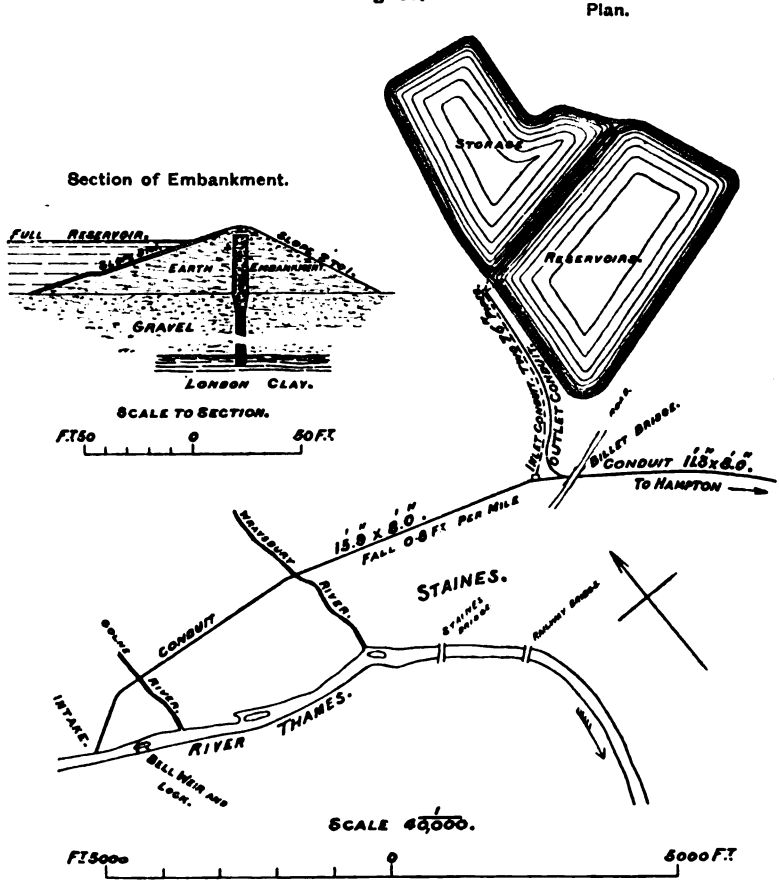
In order to increase the water-supply for London from the Thames to 300 million gallons a day, involving a supply from storage reservoirs

[1] "London Water-Supply," Walter Hunter, *Journal of the Society of Arts*, vol. xlvii. 1898–99, pp. 486–487.

of 114½ million gallons daily, which necessitates a storage of 17,500 million gallons, so as not to encroach on a minimum flow of the Thames of 200 million gallons a day over Teddington Weir, would require several

STAINES FLOOD-STORAGE RESERVOIRS.

Fig. 59.



reservoirs along the Thames Valley, similar to those at Staines, covering an area of 3½ square miles, at an estimated cost of between £9,000,000 and £10,000,000.[1] If, owing to the increase of the population of

[1] "Final Report of H.M. Commissioners on the Water-Supply within the Limits of the Metropolitan Water-Companies, 1899," p. 43, and accompanying maps, plans, and diagrams, 1900, and plate 12. The figure of 10 square miles given in the Report, is corrected by Mr. Middleton to 3½ square miles.

London, it should at some future time be proposed to augment the volume derived from the Thames to 400 million gallons a day, it has been estimated that the necessary storage of 45,000 million gallons could be obtained by reservoirs occupying an area of $7\frac{1}{2}$ square miles.[1] An advantage of this scheme of storing up the flood-waters of the Thames, which has been preferred to the grander design of obtaining a purer supply from impounding reservoirs in the valleys of some streams in South Wales, consists in the facility with which the successive moderate instalments of these numerous storage reservoirs can be provided in accordance with the actual requirements, depending upon the somewhat uncertain rate of growth of the population to be supplied. The system, however, has the disadvantages that all the water stored in these reservoirs has to be pumped up into them, and that they occupy valuable tracts of land; whereas impounding reservoirs are filled by gravity, and generally deliver their supplies into the service reservoir by gravity alone, and also submerge smaller areas of land in much less valuable situations. The valley of the Thames is not adapted for impounding reservoirs; but if suitable sites for such reservoirs could be found in the valleys of some of its tributaries, for instance in the basin of the River Kennet, they would be preferable to the multiplication of storage tanks like those of Staines. It may be expedient, as a general principle, to use up the resources within easy reach before going to a distance for new sources of water-supply for London; but it must not be overlooked that many towns draw their supplies from the Thames, and are more wholly dependent on the river for water than London is. Moreover, if London is content to wait till its population has nearly outrun the resources available in the Thames basin, before seeking external sources of supply, it may find that the suitable gathering grounds at present available, will have been taken possession of by other communities, just as Liverpool has obtained control of the head-waters of the Severn, and Birmingham of the Wye, and Manchester has monopolized a very valuable region of the lake district.

[1] "The Future of the London Water-Supply," R. E. Middleton, *Transactions of the Surveyors' Institution*, 1900–1901, vol. xxxiii. part iii. p. 87.

CHAPTER VI.

EARTHEN AND RUBBLE RESERVOIR-DAMS.

Types of Dams constructed for forming Reservoirs—**Earthen Reservoir-Dams:** Form and Construction, form of embankment, puddle trench and wall, distribution of materials, provisions against failure and slips, selected materials in place of clay; Conditions essential for Stability of Earthen Dams, causes of failure, cause of, and protection against infiltration, protection against overtopping of dam by the water in reservoir, influence of earthquakes; Limiting Height of Earthen Dams, causes, dams in steps down a valley; Masonry Central Core, instance of concrete used, example at Titicus Dam described, design and works at New Croton Dam abandoned; Hydraulic Method of constructing Earthen Dams, by jets and current of water, adopted for railway embankments, examples for forming reservoirs in Texas and California described, economy of system, sorting of materials by current; Protection of Earthen Dams against Slips, ordinary methods, counterforts in French earthen dams—**Mixed Forms of Dams:** rubble stone largely employed in remote parts of United States, advantages, rubble mound with various kinds of watertight barriers; Rubble and Earthen Dams, advantages of system, example across Pecos River, New Mexico, described, reservoir retained, cost; Rubble-Mound and Dry-Wall Dams with Facing of Planks, reduction of materials, cheapness and short life of timber, description of Escondido dam, cost, and leakage; Rubble-Mound Dam with Concrete Facing, design of Morena dam in California, partly constructed, size of reservoir formed; Rubble-Mound Dam with Masonry Face-Wall, description of Castlewood dam in Colorado, water retained, flow over and through dam; Rubble-Mound Dam with Steel-Plate Core, Lower Otay dam described, not tested with full head, size of reservoir formed—**Waste Weirs and other Provisions for the Security of Earthen Dams:** discharge of flood-waters by bye-channel and waste weir; Position of Waste Weir, at side of valley, on crest of masonry portion, at New Croton and Southborough, Mass., reservoirs; Level and Width of Waste Weirs, controlling water-level of reservoir, formula for discharge over weir; Raising of Water of Reservoir by Wind, formulæ of height of waves, observed height; Heights of Earthen Dams above the Highest Water-level, minimum, examples—**Outlet Conduits:** Under Earthen Dam, defects of old system; Valve Tower in Reservoir, and Outlet Culvert, advantages, defects, conditions essential for safety, with instances;

Tunnel Outlets from Reservoirs, superiority of system; Waste-Water Channel, velocity of flow checked by steps or curved batter below masonry waste weir, steps separated by pools in channel to reduce velocity—Concluding Remarks on Earthen and Rubble Dams.

IMPOUNDING reservoirs are formed by means of dams constructed across valleys at suitable sites, and so intimately connected with an impervious stratum at the bottom and sides, as to constitute watertight barriers against the streams draining the valleys. The flood-waters flowing down are thereby retained and stored up, in each case, so as to form an artificial lake, from which the water is drawn off as required for irrigation or water-supply during periods of drought.

Two types of dams are commonly employed for enclosing these reservoirs, namely, earthen dams, or embankments, with a central watertight portion consisting of a trench and wall of puddled clay, or, in the absence of clay, a core of specially selected impervious materials, or occasionally a concrete or masonry wall; and, secondly, masonry dams. The first type is adopted where a solid foundation of rock is not available, and for dams of moderate height; and the second type is resorted to for dams which can be founded on solid rock, and especially for those of considerable height.

Some modifications of the first type, consisting mainly in the substitution of rubble stone for the soft, consolidated materials usually employed for forming the embankment, and some slender forms of the second type, arched against the water-pressure, have been erected in the United States.

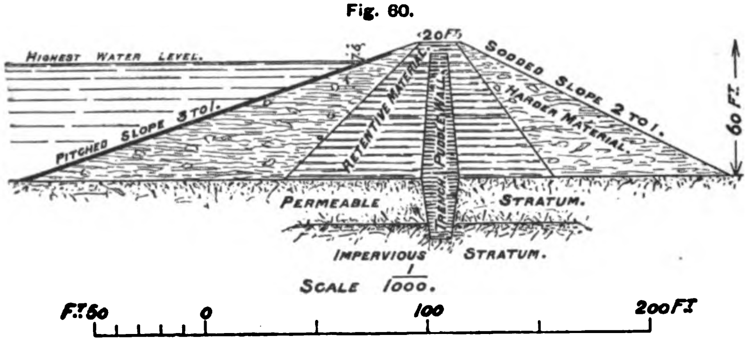
In addition to the construction of the dams, works have to be executed to provide for the harmless escape of floods after the reservoir has been filled, and for drawing off the water from the reservoir, as required, for conveyance through a conduit into the service reservoir, and for compensation to raise the discharge of the impounded stream, below the reservoir, to the stipulated minimum flow in times of drought.

EARTHEN RESERVOIR-DAMS.

Form and Construction of Earthen Dams.—Dams of moderate height are usually composed of an embankment of earthwork, brought up in carefully consolidated layers, formed with flat slopes on each side, and made watertight by a central wall of puddled clay carried up simultaneously with the rest of the embankment, Fig. 60. Before commencing the construction of the embankment, all loose soil must be removed from its site; the puddled clay has next to be carried down in a trench into an impermeable stratum, so as to form a perfectly watertight connection, sometimes to a great depth, as, for instance, 175 feet below the surface at the Yeo reservoir, Blagdon, Somerset; and the embankment is then carried up in thin, horizontal, well-punned or rolled layers, together with the wall of puddled clay in continuation of the trench below the ground. The materials for forming the embankment, which are generally excavated from the site of the reservoir, and, consequently, serve to deepen it to some extent, are usually sorted, the more retentive materials being placed in the central portion surrounding the puddle wall, and the more durable and less compact materials being placed on the outer part of the embankment. The retentive materials towards the centre of the dam, help to keep the clay puddle moist, thereby preventing its cracking; whilst the harder and looser materials on the outside, render the side slopes less liable to slip than slopes composed of more clayey materials, which would crack in dry weather,

EARTHEN RESERVOIR DAM, WITH PUDDLE WALL AND TRENCH.

Fig. 60.



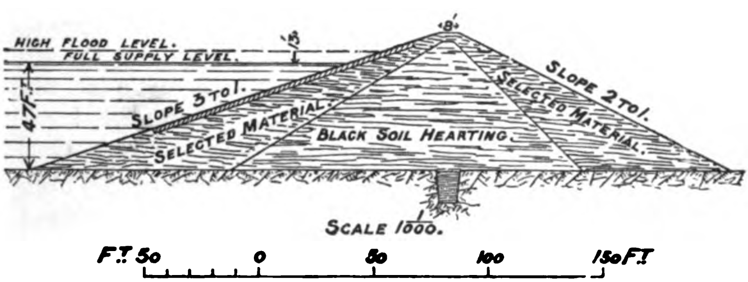
and be disintegrated by rain. In fact, even if an abundance of clay was readily procurable, it would be very unwise to form the dam entirely of clay, on account of its liability to be disintegrated and to slip when exposed to the weather. The central puddle wall has to be carried into the impervious strata at the sides of the valley, as well as at the base, so as to form a watertight barrier throughout.

The top of the embankment should have a width of not less than 8 to 12 feet, and should be raised some feet above the highest possible level of the water in the reservoir. The inner slope facing the reservoir is generally made 3 to 1, and never less than 2 to 1, and has to be protected against waves by pitching or a layer of concrete, and the outer slope is made from 2 to 3 to 1 according to the nature of the material; and in the case of high dams, or with unsatisfactory materials, the slopes should be made flatter towards the base, or berms introduced, to provide against slips, these berms also serving to carry off the rain running down the slopes above them.

Where suitable clay for the puddle wall is scarce or unobtainable, as in parts of India, the puddled clay is sometimes restricted to the trench below the ground; and the central portion of the embankment is formed with the most retentive materials available,[1] Fig. 61; or occasionally reliance is wholly placed on a wide, central bank of the best procurable materials, flanked by the looser and more permeable materials on each side, and well drained on the outer side. In such cases, the defective character of the materials is compensated for by special care in construction, which is readily practicable in India, owing to the cheapness of native labour, and the custom of the natives of carrying the earthwork in small baskets, so that the embankment is necessarily brought up in many thin layers, which are watered in dry

INDIAN EARTHEN DAM, WITH SELECTED MATERIALS AND PUDDLE TRENCH.

Fig. 61.



weather, forming a compact bank, which is further consolidated by the trampling down of the feet of the numerous workers in depositing the earth from their baskets.

Conditions essential for the Stability of Earthen Dams.— The mass of earthwork required for constructing earthen dams, with their flat slopes, is always ample, when consolidated, to withstand the pressure of the water retained by it in a reservoir: and failures of earthen dams have been due to defective construction, or the neglect of proper precautions. Infiltration of water through the dam, settlement of the dam and its consequent overtopping by the water in the reservoir, and the outflow of water from the reservoir over the dam, on account of the rise of the water-level produced by excessive floods, are the causes of the destruction of earthen dams which have to be guarded against.

[1] "Reservoirs with High Earthen Dams in Western India," W. L. Strange, *Proc. Inst. C.E.*, vol. cxxxii. p. 144, and plate 3.

Infiltration is due to insufficiency in watertightness of the central core, and of its connections with the impermeable strata under the base and at the ends of the dam, or to leakage of water, under pressure, out of an outlet culvert buried under the bottom of the dam, and damaged by unequal settlement, when not controlled by a valve at its inlet in the reservoir, for shutting off its connection with the impounded water. Accordingly, the thorough watertightness of the central core, and of its connections with the impermeable stratum, and full control of any culvert under the dam, or still better, its location along the side of the valley beyond the dam, are essential for the security of the dam.

Settlement may be caused by a yielding foundation, a want of compactness and imperfect consolidation of the bank, or the disturbance of the side slopes. Consequently, a perfectly solid, stable foundation, most careful construction of the embankment in thin layers, and the protection of the inner slope from waves, which are proportionate in size to the extent, depth, and exposure of the reservoir, and of the outer slope from slips by a suitable inclination, sodding the face of the slope, and adequate drainage, are necessary for the stability of the dam.

The overtopping of an earthen dam by an exceptional rise of the water in the reservoir, resulting from the influx of excessive floods, aided sometimes by a gale blowing down the reservoir, is produced by an inadequate provision for the discharge of floods, and an insufficient elevation of the dam to provide against such contingencies. Safety against the rise of the reservoir higher than the top of the dam, with its inevitably disastrous results, must be placed beyond doubt, by providing an ample side channel or conduit for the discharge of the maximum possible flow off the drainage-area, which might occur when the reservoir is full, and by raising the top of the embankment some feet higher than the level to which the water in the reservoir could possibly rise under the most unfavourable combination of conditions.

In regions where earthquakes are liable to occur, it would be unsafe to construct high reservoir dams, on account of the danger of a sufficient disturbance of the dam, by a severe earthquake, to afford a vent for the escape of the impounded waters. An earthen dam of moderate height, with an elastic puddle wall and trench, would be less subject to fracture from a shock than a rigid masonry dam; but, on the other hand, an earthen dam sufficiently shaken for the water to commence running through, would be more inevitably destroyed than a cracked masonry dam. The worst form of construction, however, under such conditions, would be an earthen embankment with a masonry or concrete core; for it would combine the rigidity, and consequent liability to fracture of a masonry dam when exposed to a shock, with the certainty of destruction of a permeable earthen embankment subjected to water-pressure.

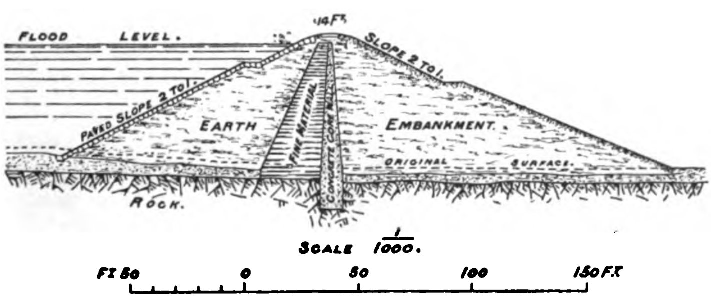
An earthquake not severe enough to dislocate a dam, is liable, by the movement it imparts to the ground, extending sometimes to long distances, to produce high waves in a full reservoir, which, if sufficiently raised to overtop an earthen dam, might produce considerable injury.

Limiting Heights of Earthen Dams.—The amount of material required for the construction of an earthen dam, and, consequently, the cost, increases rapidly with the height of the dam, owing to the long side slopes, which have to be made still flatter in the case of high dams; whilst greater care has to be taken in the construction of high embankments to secure them against settlement, slips, and damage from the higher waves formed in large, deep reservoirs. There is, therefore, a limit to the height which it is advisable to give to earthen dams, ranging between about 100 and 125 feet above the ground, according to the quality of the materials available and the local conditions, this limiting height being independent of the puddle trench or masonry core, which has sometimes to be carried down to a considerable depth below the surface to reach a solid, impermeable stratum. Consequently, when a sound, compact, rock foundation, suitable for a masonry dam, cannot be found at a reasonable depth below the ground-level, a large storage of water must be effected by a series of earthen dams of moderate height, arranged in steps down a mountain valley, according to the system adopted in impounding the River Etherow in the Longdendale Valley, Figs. 55 and 56 (p. 96).

Masonry Central Core for Earthen Dams.—Sometimes the trench excavated down to an impermeable stratum below the base of

EARTHEN RESERVOIR-DAM WITH CONCRETE WALL, SOUTHBOROUGH, MASS., RESERVOIR.

Fig. 62.

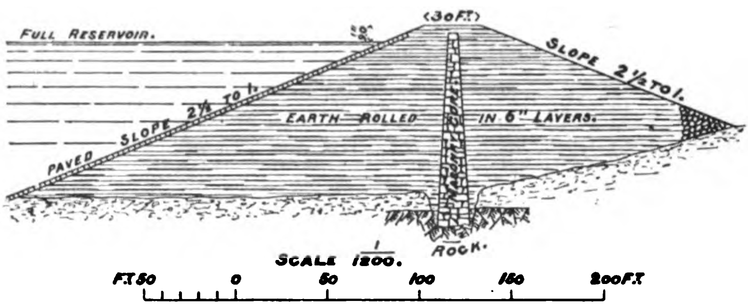


an earthen dam, is filled up with concrete in preference to puddled clay, on account of its filling more completely the irregularities in a deep, narrow trench. The connection, however, between the concrete trench and the puddle wall raised on top of it, has to be most carefully effected by toothing the two materials into one another; and occasionally, when suitable clay for puddle is difficult to obtain, a concrete core is substituted for the puddle wall, a fairly recent instance of which is afforded by the Southborough earthen reservoir-dam, erected across the Stony Brook branch of the Sudbury River,[1] for providing a daily supply of 15 million gallons of water to the city of Boston, U.S., Fig. 62.

The most notable example of a core-wall of masonry being adopted for forming the watertight barrier in the centre of an earthen dam is furnished by the embankment constituting the northern portion of the reservoir-dam constructed across the Titicus River, a tributary of the Croton River, in 1890-94, Fig. 63. This dam is composite in type; for its central portion consists of a rubble-masonry dam founded upon rock, 334 feet in length, stretching across the deepest part of the valley,

EARTHEN DAM WITH MASONRY CORE-WALL, TITICUS RESERVOIR.

Fig. 63.



continued at its southern end by a rubble-masonry waste weir, stepped on its downstream face and 200 feet long, also founded on rock; whilst from the two ends of this masonry portion, earthen dams have been carried across to the side slopes of the valley, 732 feet in length on the northern side, and 253 feet on the southern side,[2] Fig. 64. These earthen dams, formed by successive horizontal layers not exceeding 6 inches in thickness, well watered and consolidated by a heavy, grooved roller, have been carried up 9 feet above the crest of the waste weir, and given a top width of 30 feet, with side slopes of $2\frac{1}{2}$ to 1, the upstream slope being faced with rubble-stone pitching, 18 inches thick, resting on a 1-foot layer of broken stone, from the bottom up to 4 feet

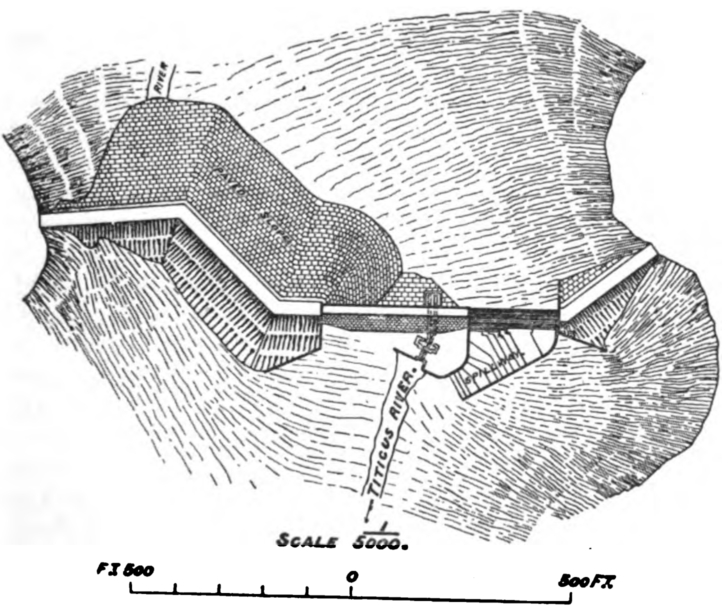
[1] *Engineering News*, Chicago, 1895, vol. xxxiii. p. 230, and plate opposite p. 224.

[2] "Water-Supply of the City of New York," E. Wegmann, pp. 197 to 199, and plates 117 and 118.

below the top, and the rest of the surface of the embankment being covered with soil and sodded; and the northern earthen dam reaches a maximum height of 102 feet above the surface of the ground. The central core-wall built of rubble masonry, with a maximum height of 124 feet, has a bottom width at the deepest part of about 20 feet, reduced by a batter on both faces to 5 feet at the top, 5 feet above the crest of the waste weir, and 4 feet below the top of the bank, Fig. 63. This wall was founded, partly on rock, but mainly on compact soft soil, owing to the depth to which the rock dips towards the sides of the

TITICUS RESERVOIR-DAM AND OVERFLOW WEIR.

Fig. 64.



valley; and as some settlement was anticipated where the wall leaves the rock, a well, 5 feet wide by 8 feet long, was built in the core-wall at this point, so as to localize the settlement at this well as being the weakest place in the wall, and to provide means of access for repairing it; which settlement manifested itself in the well as a slight crack when the core-wall had been carried up 50 feet; and the well was eventually filled up with gravel. The core-walls of the two embankments, together with the masonry dam and waste weir, form a continuous wall of masonry along the dam, 1519 feet long; and the core-walls in this instance have proved impervious to water.

In the design of the New Croton dam for the supply of New York, an earthen dam, 564 feet long, was originally proposed for the continuation of the dam from the high, central, masonry dam, 625 feet long, across the bottom of the valley, to the southern slope, owing to the depth of the rock below the surface, and the consequent saving effected in excavation and masonry by the adoption of an earthen embankment, with only a slender, masonry core-wall carried down to the rock, across the southern slope of the valley.[1] The experience, however, gained in excavating the foundations, and from other high earthen dams constructed in the Croton Valley, led to the extension of the masonry dam 110 feet further south, reducing the maximum height of the embankment above the surface from 120 feet to 70 feet, and that of the core-wall from 230 feet to 185 feet.[2] During the construction of this reduced earthen dam, a Board of Engineers, towards the end of 1901, advised the abandonment of the earthen dam as far as the gatehouse over the old aqueduct, a length of 290 feet, and the prolongation of the masonry dam to this extent, thus leaving only a low earthen dam, about 164 feet long, between the gate-house and the southern slope.[3] It was considered that with the high core-wall and sloped excavation for founding it, leakage might occur through cracks, either at the junction of the core-wall with the masonry dam, or in the core-wall itself from being subjected to deformation, or from water flowing along the masonry face at the junction, and finding a passage through seams in the rock underneath the core-wall; and this percolation of water past the core-wall, under a head of 130 feet, would have raised the plane of saturation on the downstream side of the core-wall higher than the base of the bank, which, being necessarily composed of the materials on the spot, of unreliable quality when exposed to water, would have its stability endangered. The recommendation was approved in April, 1902; and the portion of the core-wall already built along this length of the dam, has been removed; whilst the masonry dam has been given a total length of 1025 feet, extending from the masonry waste weir on the northern side, to the gate-house up the southern slope of the valley.

Hydraulic Method of constructing Earthen Dams.—The power of a large jet of water, under pressure, to disintegrate strata composed of fairly loose materials, such as soil, clay, sand, gravel,

[1] "Water-Supply of the City of New York," E. Wegmann, p. 206, and plates 127, 128, and 132; and "Civil Engineering as applied in Construction," L. F. Vernon-Harcourt, p. 522, and fig. 341.

[2] *Transactions Am. Soc. C.E.*, 1900, vol. xliii. pp. 475, 476, and 492-493.

[3] "Report of the Board of Engineers appointed June 21, 1901, to consider the Plans for the Earth Embankments of the New Croton Dam and Jerome Park Reservoir," dated November 18, 1901, kindly sent me by Mr. Edward P. North, C.E, of New York.

stones, small boulders, and loose pieces of rock, and thus enabling a rapid current of water to transport them to a lower position, and the action of the flowing water in sorting and consolidating them into an embankment at a suitable site, has occasionally been utilized in the United States for the economical construction of earthen dams, in places where the cost of the ordinary methods would have precluded the formation of reservoirs. The principle is similar to that of the method adopted on the Canadian Pacific Railway, in British Columbia, in substituting embankments for the temporary timber trestles used in the first instance for carrying the line across valleys or gorges, by sluicing down materials from the mountain slopes at the side, to form a bank along the site of the trestle viaduct. In some of the earlier dams in California, the puddle wall and central portion of the embankment were formed in the ordinary way; whilst the materials for the side slopes were conveyed and consolidated by the hydraulic process.

In 1894, a further advance of the hydraulic principle was inaugurated, by the construction of the whole of the earthen dam for forming an impounding reservoir for the water-supply of Tyler in Texas, by dislodging the materials from a neighbouring hill by a powerful jet of water, and conveying them by sluicing to the site. This dam, founded on rock, has a length of 575 feet, a maximum height of 32 feet, an inner slope of 3 to 1, with a 4-feet berm 10 feet from the top, and an outer slope of 2 to 1. The reservoir has a maximum depth of 26 feet, an area when full of 177 acres, and a capacity of $576\frac{4}{5}$ million gallons; and the dam, which contains 24,000 cubic yards of materials, cost, with all its accessories, only $2\frac{3}{8}$*d.* per cubic yard.[1]

The following year, a dam with a maximum height of 66 feet, was formed at La Mesa, California, by the aid of a current of water, across a rocky gorge of the San Diego River, for impounding its flood-waters in a reservoir of 62 acres, with a capacity of 324 million gallons, and supplying the town of San Diego, about 8 miles distant. As water could not be obtained under pressure, the surface layer of earth, sand, gravel, and stones, over an area of $11\frac{1}{2}$ acres, had to be disturbed by sluicing alone, the water for this purpose, and for conveying the eroded materials to the site of the dam, being brought in pipes, flumes, and ditches; and some of the gravel and cobblestones below the surface were so cemented together, that scrapers and horses had to be employed for loosening them and conveying them to the sluiceways, thereby largely increasing the cost.[2] The stream of water, with its burden of materials collected by the sluicing, was conveyed to the site in 24-inch

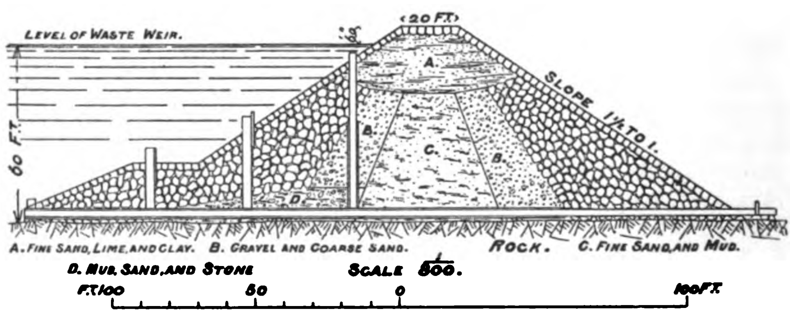
[1] "Reservoirs for Irrigation, Water-Power, and Domestic Water-Supply," J. D. Schuyler, New York, 1901, pp. 78 to 84.

[2] *Ibid.*, pp. 84 to 98.

pipes, supported and directed for deposit by trestles or wire ropes; and by keeping the discharge near the lines of the slopes, the materials are sorted, the finer portions, consisting of sand and mud, being carried on towards the centre of the bank by the current, and the largest stones being dropped on the outside, close to the outlet of the pipe, as shown in the cross section of the dam, Fig. 65. The dam contains 38,000 cubic yards of materials, conveyed a maximum distance of 2200 feet; and this mode of construction ensures their thorough consolidation; but, nevertheless, there is a small amount of leakage through this dam, owing probably to the large proportion of permeable materials of which it is composed. In spite of the additional expense necessitated for loosening the compact materials, the total cost of this dam, with the

HYDRAULIC-FILL EARTHEN DAM.

Fig. 65.—La Mesa Dam, California.



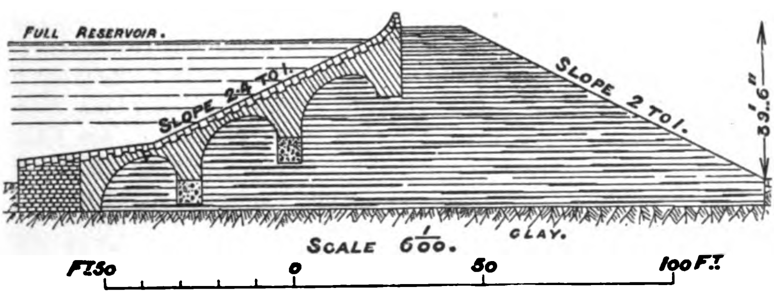
various supplementary works, was only about £3550, indicating how very economical the system would be under more favourable conditions, in respect of the location and looseness of the soil, and supply of water under pressure.

Another still larger earthen reservoir-dam has been constructed quite recently, in a similar manner, across a cañon on the North Fork of the San Joaquin River, near Fresno in California. This work, known as the Lake Christine dam, has a maximum height of 100 feet, a length of 30 feet across the bottom of the rocky cañon, and 720 feet at the top; and slopes of 2 to 1 on the upper side, and 1½ to 1 on the down-stream side. The water was conveyed in flumes and ditches, a distance of five miles to the site of the dam; and a volume of 15 cubic feet per second was delivered at the top of a hill rising 200 feet above the base of the dam, for eroding the loose layers covering the hilly slope, the materials detached ranging from fragments of granite down to sand and clay; and these materials, after being transported by the current a distance of from 600 to 2000 feet, were deposited along each slope to form the dam, with the largest pieces settling towards the outside, and the finest portions being carried towards the centre of the bank, the whole mass being consolidated by the action of the water. The water-tightness of the dam, however, has been secured in this instance by a continuous sheeting of timber along the centre of the dam, formed by a double thickness of planks; and the lower edge of this sheeting has been connected with the rock at the base and sides, by being embedded in concrete laid on the rock. Reliance is placed on the dampness and compactness of the fine particles of sand and clay enveloping the sheeting, to protect it from decay. The estimated cost of the dam, including the necessary accessory works, was only £5200. The reservoir formed by this dam, has a length of 3 miles, and an average width of half a mile; and its capacity amounts to 2250 million gallons.

Protection of Earthen Dams against Slips.—The flattening of the slopes, the formation of berms at intervals, the drainage of the

EARTHEN DAM WITH MASONRY COUNTERFORT, CERCEY RESERVOIR, FRANCE.

Fig. 66.



downstream portion of the dam and the soiling and sodding of its slope, and the pitching of the inner slope on a foundation layer of gravel or other hard material, are the ordinary methods, already mentioned, for preventing slips. When, however, clay is the only material available for forming a dam, further provision has proved sometimes necessary to prevent slips. Thus, at two earthen dams constructed wholly of clay, for providing reservoirs for supplying the Burgundy Canal with water, masonry partition walls connected by arches, so as to form buttresses against slips, were built along the inner portion of the dam, as shown in the section of the Cercey dam,[1] 47 feet high, Fig. 66.

[1] "Les Réservoirs d'Alimentation, Canal du Centre et Canal de Bourgogne," A. J. B. Fontaine, "Congrès International de Navigation intérieure, Paris, 1892," pp. 13-14.

The other dam, for the Panthier reservoir, about the same height, was similarly supported in being reconstructed, though with only two arches; but the piers of these arches were carried down to the solid ground, thereby considerably strengthening the counterfort.

MIXED FORMS OF DAMS.

Several peculiar forms of reservoir-dams have been constructed in regions specially difficult of access in the far west of the United States, owing to the necessity of employing the materials most readily available in the several localities, in which loose rock has been largely resorted to, in combination with various kinds of watertight barriers, in consequence of which they are classed in America under the general title of rock-fill dams. These dams are constructed in hilly districts remote from railways, where there is an abundance of stone, and where a good rock foundation is, in general, easily obtained; but where the great cost of carriage of cement prohibits the adoption of masonry dams. A mound of rubble stone can be erected in running water; it is stable, provided the stream does not overtop it; and it stands with much steeper side slopes than an earthen dam; but till it becomes filled with deposit, it allows of considerable leakage through it. Accordingly, various devices have been used for rendering these rock-fill dams watertight, such as an inner bank of earth in the Pecos Valley dam, New Mexico; plank facing on the inner slope of the Escondido dam, California; a facing of concrete for the Morena dam, California; a facing of rubble masonry for the Castlewood dam, Colorado; and a central steel sheeting in the Lower Otay dam, California.[1]

Rubble and Earthen Dams.—By using rubble stone for the downstream half of the embankment, where stone is abundant, the materials required for an earthen dam are considerably reduced, and the stability of the dam is increased; whilst the earthen, upstream portion retains the water in the reservoir. Examples of this type are furnished by the two reservoir-dams in the Pecos Valley, the upper one of which, across the Pecos River, and the highest dam of the two, is shown in Fig. 67. This dam, forming a reservoir known as Lake McMillan, constructed in 1893, is 1683 feet long on the top, and has a maximum height of 52 feet;[2] and it has an upper slope of $3\frac{1}{2}$ to 1, faced with stone pitching 1 foot thick; whilst the rubble bank has a $\frac{1}{2}$ to 1 slope abutting on the earthen bank, and a lower slope of $1\frac{1}{2}$ to 1. These dams have both proved thoroughly watertight; but Lake

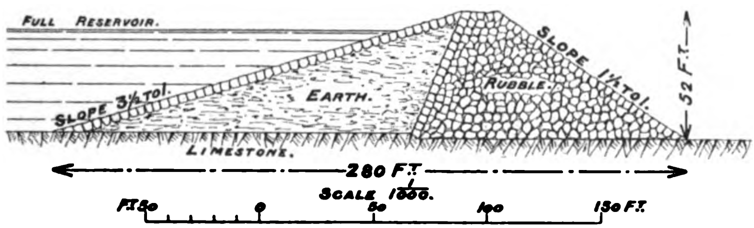
[1] *United States Geological Survey*, Report for 1896–97, Part IV. pp. 625 to 740.

[2] "Reservoirs for Irrigation, Water-Power, and Domestic Water-Supply," J. D. Schuyler, pp. 49 to 57.

McMillan has not hitherto been raised higher than 17 feet below the top of the dam, as the present waste weir has to be blocked up, and a new one cut through the rock on the right bank, before the water-level

RUBBLE AND EARTHEN DAM, LAKE McMILLAN RESERVOIR, PECOS VALLEY, U.S.

Fig. 67.



can be raised to 10 feet below the top, as designed, when the reservoir will store 24,150 million gallons, and cover an area of 8331 acres when full. The cost of this dam was about £42,000.

Rubble-Mound and Dry-Wall Dams with Facing of Planks.—The materials for a bank to form a reservoir-dam, are considerably reduced by dispensing altogether with earthwork, and relying entirely upon a sheeting of planks on the upper face for making the dam watertight. The cheapness of timber in the localities where such dams are constructed, minimizes the natural objection of rapid decay to which planks alternately in water and out of water, in a hot, dry climate, are necessarily exposed. Dams of this type have been constructed in the mining districts of northern California, with a sheeting of pinewood planks; but the Escondido dam, in the south-western extremity of California, has been built with special care, and faced with more durable redwood planking.

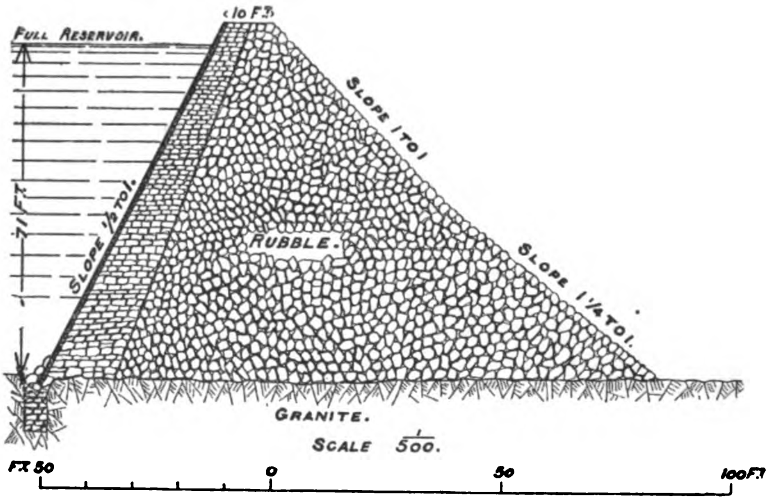
The Escondido dam has been mainly formed with a high bank of good granite rubble of all sizes up to 4 tons in weight, partly put in position by derricks, having an outer slope of 1¼ to 1 from the base up to half its height, and from thence a slope of 1 to 1 to the top.[1] The rubble bank is faced on the steep inside slope by a dry wall of rubble, 15 feet wide at the base, and decreasing to a width of 5 feet at the top, Fig. 68, redwood balks being inserted vertically in the wall at intervals of 5⅓ feet, to which a double sheeting of redwood planks is fastened at a slope of ½ to 1, with concrete rammed in at the back, the planks being 3 inches thick along the bottom third, 2 inches thick on the middle third,

[1] "Reservoirs for Irrigation, Water-Power, and Domestic Water-Supply," J. D. Schuyler, pp. 2 to 14.

and 1⅛ inches on the upper third of the height. The bottom edge of the planking is embedded in the rubble masonry filling a trench carried down into the rock foundation, in front of the upstream toe of the dam. The rubble embankment has been deposited upon the bared rock, consisting of somewhat disintegrated granite, with boulders embedded in it, so that it is not impervious to water, springs rising up in it and producing an upward pressure at the base, thereby rendering it an unsuitable foundation for a high masonry dam. The Escondido dam has a length of 100 feet at the bottom and 380 feet at the top, a height of 76 feet,

RUBBLE AND DRY-WALL DAM WITH PLANK FACING, ESCONDIDO DAM, CALIFORNIA.

Fig. 68.



and a bottom width of 140 feet at the lowest point of the gorge, and a top width throughout of 10 feet. It was finished in 1895, and cost about £18,100; and it provides a reservoir having a capacity of 953½ million gallons. There is a leakage of about 130,000 gallons a day from the reservoir when it is full; but it has not been ascertained whether the outflow takes place through the joints in the planking of the dam, or through cracks in the disintegrated granite underneath the dam.

Rubble-Mound Dam with Concrete Facing.—The water-tightness of a rubble dam can be better secured by a facing of concrete, than by planks with their numerous joints and liability to decay, provided the mound is so solidly formed on a stable foundation as not to be subject to settlement. This system has been adopted in the construction of the Morena dam, near the southern boundary of California, commenced in 1896 across a deep, narrow, rocky cañon cut through the granite barrier. This dam, as designed, is to have a maximum height of 160 feet above the bed of the stream; but it has had to be carried down in a narrow crevasse to the solid rock at a depth of $112\frac{1}{2}$ feet below the bed; and the bank of rubble stone has been formed with an upper slope of 1 to 1 and a lower slope of $1\frac{1}{2}$ to 1, and has a maximum width at the base of over 800 feet.[1] The granite rock at this site is very sound and free from fissures, and would, therefore, be a very suitable foundation for a masonry dam; but a rubble dam was preferred, on account of the cost of conveying cement to the locality; whereas the rubble is readily obtained in abundance on the spot by blasting it from the overhanging cliffs of granite on each side. The work was suspended in 1897, when it had been carried up 80 feet, owing to legal difficulties in respect of the contract; but the dam, when completed, will have a length along the top of 470 feet; and it will retain a reservoir of water with a maximum depth of 150 feet, having an area, when filled, of 1370 acres, and a capacity of 12,686 million gallons.

Rubble-Mound Dam with Masonry Face-Wall.—A vertical face-wall of masonry on the upper side of a bank of rubble stone, and supported against the water-pressure by this mound, possesses the advantage of being somewhat independent of any settlement of the bank. This system was, indeed, adopted for the Castlewood dam, erected in 1890 across a stream about 35 miles above Denver in Colorado; and though the rubble bank settled a few inches, producing a crack along the centre of the dam and moving the coping-stones out of line, the masonry face-wall was not disturbed.[2] The Castlewood dam has a maximum height of 70 feet above the ground; its masonry face-wall is 12 feet thick at the bottom, carried down to a solid foundation, and reduced to 6 feet at the top water-level by a batter of 1 in 10 on the upper face; whilst the lower face, against which the rubble bank abuts, is vertical, Fig. 69. The lower face of the rubble embankment forming the central mass of the dam, is also lined with blocks of masonry laid in cement in steps to a slope of 1 to 1, down the deepest portion of which the waste water descends on passing over an overflow weir formed along the top, 100 feet in width. The upper face-wall has been further supported on its inner face, by an earthen embankment in the reservoir, raised against the wall to about 30 feet from the top at its highest part, and protected along its exposed slope by stone pitching 1 foot thick.

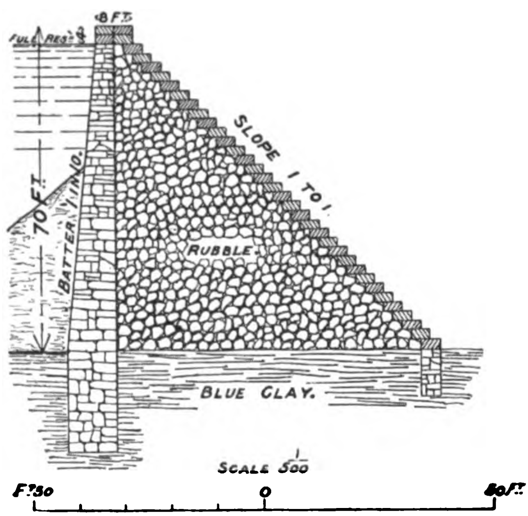
[1] "Reservoirs for Irrigation, Water-Power, and Domestic Water-Supply," J. D. Schuyler, pp. 35 to 41.

[2] *Engineering Record*, Dec. 24, 1898.

The Castlewood dam has a length of 568 feet along the top; and it retains a reservoir of water extending over 200 acres at its highest water-level, and having a capacity of 4000 million gallons. The expediency of allowing the waste water to flow over and down a dam largely

RUBBLE DAM WITH MASONRY FACE-WALL, CASTLEWOOD DAM, COLORADO.

Fig. 69.



composed of rubble stone subject to some settlement, appears doubtful; but it seems that though water has passed through the dam, it has not disturbed the rubble filling.[1]

Rubble-Mound Dam with Central Steel-Plate Core.—The flow of water through the bank of rubble stone is in this case prevented by a sheet of steel, supported on each side by the rubble, resembling in principle the Lake Christine dam with its central timber sheeting described on page 112, but constructed in a more stable and permanent manner. An interesting example of this type of dam is furnished by the Lower Otay dam across the Otay Creek, in the south-eastern corner of California near the Mexican border, which was completed in 1897.[2] This dam, the lowest one of four dams proposed for the water-supply of San Diego, Coronado, and other towns and villages within reach, and also for irrigation, consists of a bank of rubble stone deposited

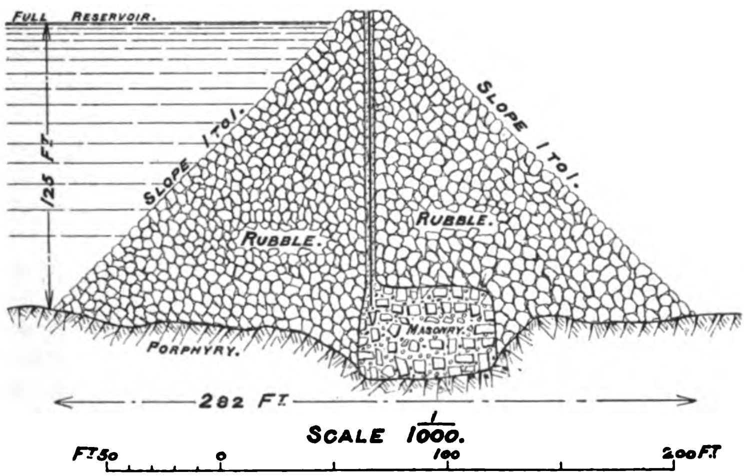
[1] *Engineering Record*, May 20, 1900.

[2] "Reservoirs for Irrigation, Water-Power, and Domestic Water-Supply," J D. Schuyler, pp. 19 to 32.

at random by the aid of transporting cable-ways, with a slope on the upstream side of about $1\frac{1}{2}$ to 1, formed with the finer materials from the quarry, and a downstream slope of 1 to 1, formed with larger blocks, and a maximum height of about 130 feet. The central core has been formed of steel plates riveted together, diminishing in thickness from $\frac{1}{3}$ inch at the bottom to $\frac{1}{4}$ inch at the top, protected from rust by a coating of asphalt, and encased in cement concrete, 1 foot thick, on each side; and the bottom of the steel plating has been built into a masonry foundation carried down to the solid rock, commenced apparently with a view to the construction of a masonry dam on account of the soundness of the porphyry foundation, and the facilities in this instance for the conveyance of cement to the site, Fig. 70.

RUBBLE DAM WITH STEEL-PLATE CORE, LOWER OTAY, CALIFORNIA.

Fig. 70.



The Lower Otay dam has a length along the top of 565 feet. It appears that the reservoir formed by this dam has not hitherto been nearly filled, so that it has not been possible to test the watertightness of the system; but, owing to expansion, the steel plates assumed a very irregular line, though they must be now protected, to a great extent, from large variations in temperature, by the concrete in which they have been encased. When the reservoir can be filled, which will necessitate works for bringing additional water to the site, it will have an area at the surface of the water of 910 acres, and contain about 10,000 million gallons of water.

WASTE WEIRS AND OTHER PROVISIONS FOR THE SECURITY OF EARTHEN DAMS.

The efficient discharge of the flood-waters of a stream impounded by an earthen dam, after the reservoir has been filled, is of the utmost importance for ensuring the security of the dam, for nothing is more certain to destroy an earthen dam than the flow of water over its crest, which commences by scouring the outer slope in its fall down it, soon resulting in the formation of a breach through the embankment. The opening thus formed, and rapidly enlarged by the current through it, gives a vent for the waters of the reservoir, which are precipitated with irresistible force down the valley, spreading destruction in their course. The requisite means for the discharge of floods is sometimes provided by a bye-channel at the side of the valley beyond the reservoir, which is primarily intended to divert the turbid or discoloured waters from the reservoir during floods, and has, consequently, to be made large enough to carry off the greatest floods. These bye-channels, to which reference has been made on pages 94 and 95, unless formed in a rocky stratum, have to be lined with pitching, masonry, or concrete, to prevent their being eroded by the current in flood-time. Usually, however, as an additional precaution, even when a bye-channel has been provided, and especially when the reservoir is fed from other streams than the one for which the bye-channel has been constructed, a means of escape for the surplus flood-waters from the reservoir is afforded by a waste weir.

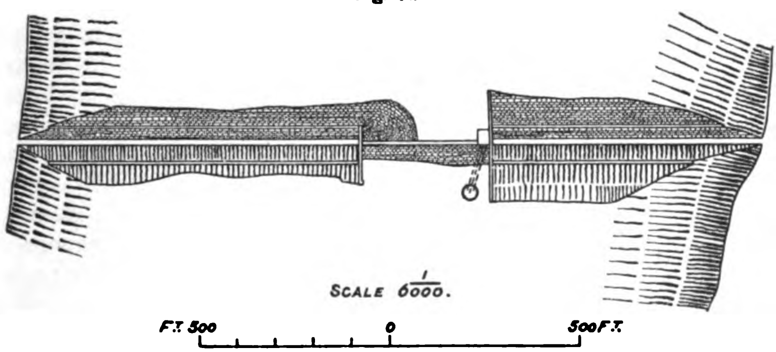
Position of Waste Weir.—The stepped masonry facing on the outer slope of the rubble Castlewood dam, enabled a portion of the dam, with a lowered crest, to be utilized for an overflow, or waste weir, to discharge the surplus water from the reservoir (p. 117); but such an arrangement, which appears to be unsatisfactory even in this case, could not be permitted at an ordinary rubble dam, and would be absolutely inadmissible for an earthen dam, even with paved top and slopes along the site of the weir. The only suitable positions for a waste weir are, at one side of the valley beyond the end of the earthen dam, or, in the case of a mixed type of dam, along the lowered crest of part of the masonry portion of the dam, out of reach of the earthen portion.

The waste weir, in the first case, must be formed in cutting in the lowest available part of the slope of the valley, beyond one extremity of the dam; and its length may be augmented, without unduly cutting into the side of the valley, by curving it, or by placing it at an angle to the line of the dam, or by extending it along the side of the reservoir parallel to the side of the valley, according to the principle adopted at the New Croton dam, Fig. 111 (p. 173). Where the valley is steep at the sides and narrow, tunnelling may have to be resorted to for the waste-weir channel, thus separating it very completely from the dam.

With the mixed type of dam, where the masonry portion extends over a considerable length of the dam, and an earthen dam is only adopted at one side, as originally designed for the New Croton dam, the conditions are in reality those of a masonry dam; and the waste weir can be located on the other side of the valley, quite away from the earthen dam. In some instances, however, in the United States, a length of masonry dam has been introduced in the centre of an earthen dam, with the express object of providing a waste weir; and the side slopes of the earthen dam, on each side, have to be retained and protected by wing

MASONRY WASTE WEIR IN CENTRE OF EARTHEN DAM, SOUTHBOROUGH, MASS., RESERVOIR.

Fig. 71.



walls, extending out at right angles to the dam both above and below it, in order to prevent the end faces of the earthen dam, abutting against the ends of the masonry waste weir, from being scoured by the outflowing water from the full reservoir converging to the waste weir, and by the rush of water down the waste-water channel, after falling over the waste weir. This arrangement has been adopted for the regulation of the reservoir of 1280 acres recently constructed at Southborough, Massachusetts, where a masonry dam, 335 feet long, provides a waste weir along its crest, 300 feet in width, over the deepest part of the valley,[1] Fig. 71, in the centre of the earthen dam, Fig. 62 (p. 107), 1615 feet in length, which is thereby divided into two parts extending from the waste weir, on each side, to the slopes of the valley. The earthen dam is protected from the currents resulting from the

[1] *Engineering News*, Chicago, vol. xxxiii., plate opposite p. 224.

discharge over the waste weir, by wing walls at its junctions with the central masonry dam, and also, in this instance, is separated from the waste weir, on the side adjoining the deepest part of the valley, by the valve house, 35 feet wide, erected at the deep end of the masonry dam for controlling the supply drawn from the reservoir.

Level and Width of Waste Weirs.—The level of the crest, or sill, of the waste weir fixes the water-level of the full reservoir, and therefore the capacity of the storage provided; for though the water rises in the reservoir above the crest of the weir during the influx of floods after the reservoir has been filled, the discharge over the waste weir rapidly lowers the water-level to the crest of the weir on the abatement of the flood. The height to which the top of an earthen embankment should be raised, is regulated by the extent of the calculated rise of the water above the crest of the waste weir, in the event of the maximum flow off the drainage-area being discharged into a full reservoir, together with the additional rise produced by waves raised by a gale blowing down the reservoir. A margin of 3 to 5 feet or more has to be added, according to circumstances, to this calculated raising of the water-level against the dam, to provide against unforeseen contingencies, and secure the dam from any possibility of being overtopped.

The rise of the water-level in the reservoir above the crest of the weir, can be reduced by increasing the width of the waste weir, which enables the height of a proposed earthen dam to be correspondingly diminished; and this, in a high embankment, effects an important reduction in the cost. On the other hand, where ice is liable to be drawn by the current towards the waste weir and heaped up, or large floating *débris*, such as trunks or roots of trees, may be brought by the stream into the reservoir, it is necessary to provide a sufficient depth of water flowing over the waste weir to prevent these obstructions from lodging on the crest of the weir, and, consequently, raising the water-level of the reservoir by impeding the discharge. The height to which the water must rise in the reservoir above the crest of the waste weir, in order to discharge the maximum estimated flow off the area of land draining into the reservoir, is given by the formula, $h = \left(\frac{D}{3{\cdot}08w}\right)^{\frac{2}{3}}$, where h is the height of the water above the crest in feet, w is the width of the waste weir in feet, and D is the maximum discharge in cubic feet per second.

Raising of Water in Reservoirs by Wind.—The height of the waves that might be generated by a gale blowing down the reservoir, depends upon the length of the reservoir, and the exposure of the site in the line of the reservoir. A formula equivalent to $H = 3\sqrt{L}$, was proposed many years ago for the calculation of the height of waves in reservoirs;[1] and another formula, $H = 1{\cdot}5\sqrt{L} + (2{\cdot}5 - \sqrt[4]{L})$, has been given as fairly representing the results of observations of the height of waves in the sea,[2] where, in both cases, H is the height of the waves in feet, and L is the length in miles of the exposed surface of water in the direction of the gale. The first formula may be regarded as applicable to reservoirs of moderate size, and the second to very long reservoirs, such as the New Croton reservoir, over 15 miles in length, and large lakes. Observations indicate that, whereas the minimum limit of the height of waves in ordinary reservoirs should be taken at 2 feet, they may attain a height of from $6\frac{1}{2}$ to 10 feet in very large, exposed reservoirs and big lakes. As the height of waves is measured from trough to crest, the actual rise of the top of the waves above the mean water-level, is only half the height stated; but as a gale blowing down a long reservoir, tends to heap up the water against the dam, as observed on a lee shore during storms, somewhat in proportion to the force of the gale and the exposure of the site, it is safer to assume a possible raising of the water against the dam to the full height of the waves.

Heights of Earthen Dams above the Highest Water-level.—The top of an earthen dam should not be less than about 5 feet above the highest level of the water in the reservoir; and, occasionally, additional protection is afforded by a low rubble wall on the reservoir side of the bank. The maximum height of the water in the Southborough reservoir has been reckoned at $2\frac{1}{4}$ feet above the crest of the masonry waste weir, 300 feet in width; and the top of the earthen dam is $4\frac{3}{4}$ feet above this level Fig. 62 (p. 107). It has been estimated that the highest water-level, or flow line, of the New Croton reservoir, may reach a height of 4 feet above the crest of the masonry waste weir, 1000 feet in length; and the top of the earthen portion of the dam was designed to be raised 20 feet above this level, under the exceptional conditions of its position. A height of from 5 to 10 feet should generally be adequate for the top of an earthen dam above the highest water-level, in most reservoirs, according to their length and exposure.

OUTLET CONDUITS.

Outlet Conduit under Earthen Dam.—As it is important to utilize the water stored up at considerable expense in a reservoir, to the fullest extent practicable, it was very natural that attempts should

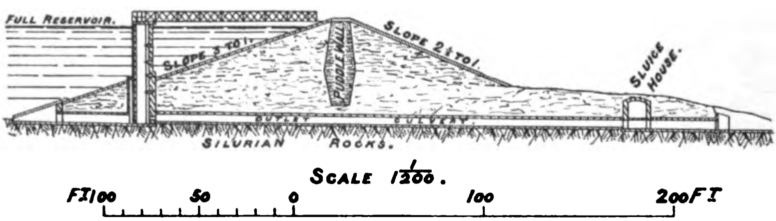
[1] *Proc. Inst. C.E.*, vol. xx. p. 361.
[2] "The Design and Construction of Harbours," T. Stevenson, 3rd edition, p. 29.

be made to provide for the complete emptying of a reservoir, by laying the outlet pipes through the earthen dam at the lowest point of the valley, in the bed of the stream draining the valley, and, consequently, under the highest part of the embankment retaining the reservoir. Unless the pipes were laid in a trench in very solid ground or rock, they were exposed to very different pressures under the centre of the embankment and towards the toe of the slopes, and therefore very liable to be subjected to unequal settlement, and consequent fracture. Moreover, the simplest position of the valve for controlling the discharge from the reservoir, was at the outer end of the outlet pipes at the bottom of the lower slope of the embankment. This arrangement, however, left the pipes always under the full pressure of the head of water in the reservoir, and rendered the stoppage of any leak, or the repair of any fracture impossible, unless the inlet could be blocked up by a diver; so that frequently water would escape unchecked into the heart of the embankment, leading gradually to disintegration, fissures, and settlement of the mass.

Valve-Tower in Reservoir, and Outlet Culvert.—A great improvement has been effected in many cases by controlling the discharge

VALVE-TOWER AND CULVERT IN ROCK UNDER EARTHEN DAM, LOGANLEA RESERVOIR, FOR EDINBURGH.

Fig. 72.



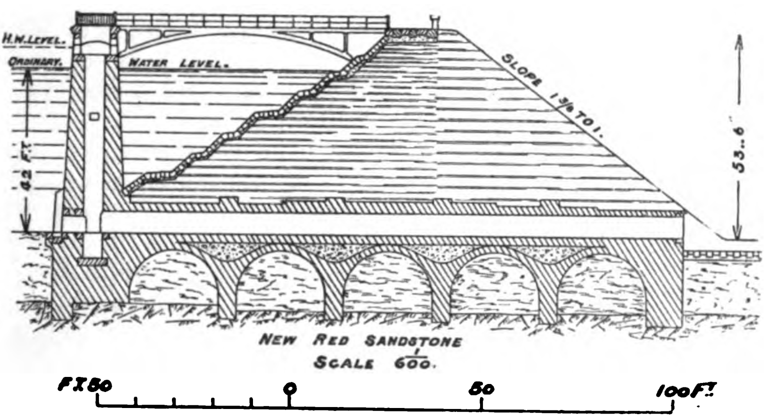
in a tower standing in the reservoir, with inlets at different levels, closed by valves, so that with a full reservoir, the water can be drawn off at different depths, according to the condition of the water;[1] and the water can at any time be shut off from the whole of the outlet conduit passing under the embankment, for inspection or repairs, Fig. 72. Moreover, by placing the outlet pipes in a culvert of brickwork, masonry, or concrete, founded very carefully in the solid ground, the pipes are more effectually secured from injury due to irregular settlement, and are made accessible for repairs or renewals. It is, however, essential to construct projections from the culvert into the bank at intervals, in order to prevent a

[1] "The Waterworks of Edinburgh," A. Leslie, *Proc. Inst. C.E.*, vol. lxxiv. p. 97, and plate 3.

leakage of water from the reservoir being established along the outside of the culvert. Though these modifications render the location of the outlet conduit under an earthen dam much less objectionable than the first system, nevertheless, it is difficult to secure the culvert from fracture where it passes through the puddle trench, which naturally tends to settle more under the weight of the culvert and the embankment, than the adjacent solid ground; whilst in the case of a concrete trench under the culvert, the settlement may be less over the concrete than on the ground. The only conditions under which this source of failure can be obviated, are where a rocky stratum crops up at the surface along the deepest part of the valley at the site of the earthen dam, in which the

OUTLET CULVERT ON ARCHES UNDER EARTHEN DAM, TORCY-NEUF RESERVOIR, FRANCE.

Fig. 73.



outlet culvert can be founded throughout its whole length, or where rock is met with so near the surface that the culvert can be carried on a masonry structure founded on the rock. The culvert was carried through the solid rock at the Loganlea reservoir of the Edinburgh waterworks, Fig. 72 (p. 124); and the latter plan was adopted at the Torcy-Neuf reservoir, near Creusot, completed in 1887, for supplying the Canal du Centre in France, where the culvert under the earthen embankment is carried on masonry arches with their piers resting on the New Red Sandstone, down to which also the foundations of the valve-tower in the reservoir, at the inner end of the culvert, have been taken down, Fig. 73. The cross section of this culvert, moreover, shows the projections made to stop infiltrations along its outer face.[1]

[1] "Les Réservoirs d'Alimentation, Canal du Centre et Canal de Bourgogne,"

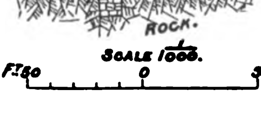
Tunnel Outlets from Reservoirs.—Owing to the great difficulty of securing a culvert from injury when buried under a high embankment, it is generally far preferable to keep the outlet conduit completely separate from the earthen dam, and to carry it in tunnel from the reservoir along one side of the valley, beyond the end of the dam, to join the conduit for conveying the supply to its destination. In the cases where it is necessary to carry the outlet tunnel under any portion of the dam, it must be driven, if possible, through rock, and at a sufficient depth to ensure the foundations of the dam from any chance of settlement.

A tunnel outlet possesses the great advantage that it can be utilized for the diversion of the stream during the construction of the dam; and, consequently, it enables the construction of a bye-channel to be dispensed with, unless required permanently for the diversion of turbid flood-waters. The provision, accordingly, of a tunnel for drawing the supply of water from a reservoir retained by an earthen dam, is in every respect the most satisfactory manner of dealing with the problem; for it both greatly facilitates the execution of the work, and also ensures the complete separation of the outlet conduit from the dam.

Waste-Water Channel.—The fall of the water discharged over the waste weir, and also of the bye-channel, if one has been constructed, is concentrated near the dam; and this fall, and the current produced by it, have to be regulated, to prevent the creation of an excessive scour in the waste-water channel leading the water into the stream below the dam. When the waste weir is on the top of a masonry portion of the dam, which must necessarily be founded upon rock, the fall is broken by a series of steps on the outer face of this masonry portion; or a curved batter, approaching the horizontal at the base, directs the falling water tangentially to the rocky bed of the stream below, as effected at the waste weir of the Southborough, Mass., earthen dam, Fig. 71 (p. 121), shown by a section of this waste weir on Fig. 74, enabling an apron of masonry, extended for a short distance from the outer foot of the dam, to prevent erosion.

MASONRY PORTION FORMING WASTE WEIR FOR EARTHEN DAM, SOUTHBOROUGH, MASS., RESERVOIR.

Fig. 74.



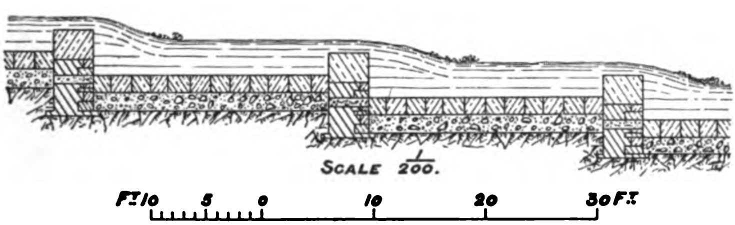
In the ordinary case, where the waste weir is formed at one side of the valley beyond the end of the earthen dam, a series of steps at

A. J. B. Fontaine, Congrès International de Navigation intérieure, Paris, 1892, pp. 2 and 3.

intervals in the waste-water channel, lead the water gradually down to the stream below. If the channel is excavated in rock, the steps can be cut in the rock, if sound and hard enough; otherwise the channel should be lined with masonry. The velocity, however, of the current increases at each fall, so that a serious scouring force may be generated towards the lower end of the waste-water channel, tending to scour soft rock, or displace the masonry. The most effective way of rendering the velocity fairly uniform, is so to construct the channel as to form a succession of pools between the steps, into which the water falls on passing over each step, with the result that the fall of the water is deadened, and its velocity checked, in coming in contact with the mass of water in each pool, Fig. 75. It has been found by experience that the

WASTE-WATER CHANNEL, WITH STEPS AND POOLS.

Fig. 75.



best results are obtained when, with the maximum discharge passing over the waste weir, the fall from the top of each step to the water-level of the pool, and the depth of the pool, are equal to the depth of the water flowing over the step, and the length of each pool is twenty times this depth.[1]

Concluding Remarks on Earthen and Rubble Dams.—Earthen dams are very generally employed for the formation of reservoirs for water-supply of moderate depth; and their stability can be fully secured by careful construction with suitable materials, so as not to be subject to infiltration or slips, with their central, watertight core carried down into an impervious stratum; and if the bank is carried up to an adequate height, and a long enough waste weir provided at its side, to render the dam safe from being overtopped by the water in the reservoir. The outlet conduit, also, should be quite separate from the dam, so as not to be liable to unequal settlement under an embankment of varying height, or to endanger the dam by internal leakage.

Dams formed largely with rubble stone deposited at random in a

[1] "Water-Supply: Rainfall, Reservoirs, Conduits, and Distribution," A. R. Binnie, 2nd edition, p. 57, and plate 26, figs. 5 and 6.

mound, exhibit a greater variety in construction than earthen dams; but these dams are only built in remote districts, where cost of carriage prohibits the construction of masonry dams, and where the materials and care needed for an earthen dam are not available. The prevention of leakage in such dams is not of vital importance, though they are exposed to failure if overtopped; and they are generally only constructed when they afford the sole means at hand for providing reservoirs for irrigation, in arid regions devoid of the ordinary facilities for communication.

CHAPTER VII.

MASONRY DAMS.

Rock foundation indispensable—Conditions of Stability compared with earthen dams, two forms, arched and straight, slight dams dependent for stability on curvature, straight dams dependent on weight and form of section—**Arched Masonry Dams of Slight Section:** narrow gorge essential, value of curvature, instances cited dependent on curvature; Bear Valley Dam, California, curvature, length, and height, head of water retained, description of dam, cost, storage provided; Zola Canal Dam, site, storage, curvature, description, unsuitable section; Sweetwater Dam, California, site, irregular supply, storage, yearly supply, curvature, description, cost, stability; Remarks on Arched Form—**Direction and Distribution of Pressures in Masonry Dams:** two sets of pressures; Line of Resultant Pressures with the Reservoirs Empty, graphical method of finding line of resultant pressures; Line of Resultant Pressures with the Reservoir Full, compounded of two pressures, points of application, graphical method of finding resultant pressure and line of resultant pressures; Distribution of Pressures with the Reservoir Empty, graphical method of illustrating distribution; Distribution of Pressures with the Reservoir Full, graphical method of illustrating distribution; Conditions of Stability with regard to Pressures, graphical indication of tension in dam, conditions reducing the stability of dams and provisions against their effects—**Forms and Construction of Masonry Dams:** massive character and defective forms of early masonry dams, introduction of correct form; Correct Form of Cross Section, indication of theoretical form, instances of approximate compliance; Modifications introduced in Form of Cross Section, width and height of top of dam, large width at base, form of section of waste weir; Materials employed in Construction, concrete used as well as masonry, instances of dams of rubble concrete, and of concrete with masonry face, protection of inner face, selection of sand—**Failures of Masonry Dams:** causes, Failure of Foundations, description of Puentes dam, reconstruction; Sliding forward on Foundation, description of Austin dam, cause of failure; Failure from Faulty Design with Tension in Masonry, form of Bouzey dam, sliding forward, cause of failure, additions to dam, tension on inner face, breach in dam, explanation of failure, bad form and foundation of Gros-Bois dam, motion forward arrested by counterforts, undue pressure on outer face; Failure from Faulty Construction, materials used in Habra dam, description of dam, failure of waste weir, reconstruction, discharge of flood over dam, leakage through dam, breach during flood; Remarks on Failures, precautionary measures.

MASONRY dams can only be erected where a sound, compact stratum of rock can be reached at a reasonable depth below the surface, on which to found the dam; for any settlement of the foundations on which a rigid mass of masonry rests must result in the dislocation of the masonry, allowing of leakage through the cracks, impairing the strength of the dam, and leading sooner or later to its failure. When a high dam is required, or where for any other reason a masonry dam is expedient, the foundations of a masonry dam have been carried down to solid, impervious rock lying at a considerable depth below the surface, notwithstanding the large addition to the mass of masonry involved in deep foundations, without any resulting increase in the depth of the reservoir thus formed, of which the New Croton, Vyrnwy, and Thirlmere dams furnish notable examples.

Conditions of Stability of Masonry Dams.—Whereas earthen and rubble dams, with their large mass and flat slopes, possess ample stability to resist the pressure of the water in the reservoirs they retain, and only fail from faultiness in construction, or neglect of precautions against their being overtopped, masonry dams depend for their stability on the correctness of their design, and the immobility of their foundations.

Two forms of masonry dams have been adopted for resisting the pressure of the water they impound. In a few instances, where the closure of a very narrow, rocky gorge serves to impound a large volume of water above it, dams curved horizontally and abutting against the rock on each side, with their convex side facing the reservoir and increasing in thickness towards the base, have been constructed; and these dams support the horizontal pressure of the water in the reservoir like an arch. Masonry dams in general, however, are built straight across the valley, and withstand the water-pressure to which they are subjected entirely by their weight, and by their rapidly increasing width towards their base; and these dams have, therefore, been sometimes termed "gravity dams" in America. Occasionally, indeed, these latter dams have been given an arched form in narrow valleys; but the arch in these cases has been only regarded as an additional element of stability, as for instance in the celebrated high Furens dam.

True arched dams are made slighter in section than gravity dams, and depend for their stability, on the amount of curvature adopted, on the cross section of the arch, at the varying depths, being proportioned to the strength of the masonry to resist compression, and the area of bearing surface at the springings being suited to the supporting power of the rocky abutments on each side. This type of dam possesses the special merit, that the span of the arch is reduced by the slopes of the valley on both sides, as the water-pressure increases with the depth.

Ordinary masonry dams, depending for their stability on their weight and the configuration of their section, have to be given such a cross section, with an increasing thickness in proportion to the depth, that, with the reservoir empty and full, the greatest pressures on the foundations, and on the masonry, may be well within the limits that the rocky stratum and stonework respectively can safely bear; that the lines of resultant pressures for the empty and full reservoir may fall within the middle third of the cross section, to avoid tensional strains on the masonry; and that the structure, when subjected to the maximum water-pressure, may be in no danger of sliding forward on its foundation, or on any horizontal joints of the masonry. Provided these conditions are duly fulfilled, any further addition to the mass of the dam, except at the base, merely imposes a greater pressure on the masonry and the foundations, and increases the cost of the work, without augmenting its stability.

ARCHED MASONRY DAMS OF SLIGHT SECTION.

Advantage can only be taken of the arched form of dam for resisting the water-pressure in a reservoir, allowing of a reduction in cross section, where a narrow, rocky cañon, with a widening-out valley above, furnishes a suitable site for forming an impounding reservoir by a short masonry dam across the cañon. The maximum proportion of thickness of dam to radius of curvature, compatible with the condition of the dam acting as an arch, has not been definitely ascertained, though it has been estimated at from one-half to one-third; and it would be difficult to determine with any accuracy the relative pressures which are borne by the curvature of a dam, and its weight, respectively. It is, however, quite certain that a considerable curvature adds greatly to the stability of a dam; for a few arched dams of too slight a cross section to comply with the conditions of stability of gravity dams, have borne the water-pressure of their reservoirs without injury for many years.

The Bear Valley dam in California, with its thin upper wall, affords the most remarkable instance of reliance being placed on the arched form for stability, and is a marvel of boldness in design; whilst the much higher Zola dam in France, built in 1843, though much thicker, is so uniform in width, that the line of resultant pressures, with the reservoir full, falls outside its base, so that it too is dependent on its arched form for its stability. The Sweetwater dam in California also, though much more suitably designed than the two other dams for sustaining the water-pressure by its weight, with its narrow top and wide base, nevertheless does not fulfil the condition of its line of resultant pressures, with a full reservoir, falling within the middle third at the base, so that its arched form is required to ensure stability. In the first two instances, indeed, the arching is necessary to secure the dams from being overturned; whereas the compressive strains resulting from the curvature of the Sweetwater dam, relieve it from tension at the upper face with a full reservoir.

Bear Valley Dam, California.—The rubble masonry dam which retains the Bear Valley reservoir, referred to and a plan of it given (Fig. 50) on page 87, has been built on a curve with a radius at the top of 335 feet, Fig. 76. It has a length along the top of a little over 300 feet, and diminishes rapidly in length towards the base, owing to the converging slopes of the narrow, rocky cañon in which it is situated,

BEAR VALLEY RESERVOIR-DAM, CALIFORNIA.

Fig. 76.—Plan and Elevation.

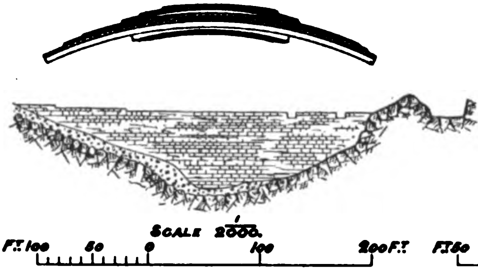
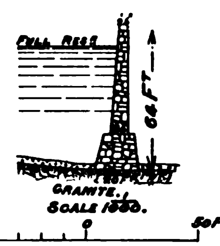


Fig. 77.—Cross Section.



as shown on the elevation of the dam below the plan. The height of the dam along the central portion, is about 64 feet from the granite rock foundation, and it has retained a maximum head of water of about 60 feet;[1] for though the waste weir at one side, cut through the solid rock, has its crest $8\frac{1}{2}$ feet below the top of the dam, its width is only 20 feet, so that the water has risen occasionally to within a few inches of the top of the dam; and an exceptional flood coming down when the reservoir is full, might raise the water 2 or 3 feet above the dam and prove too great a strain on the very slight section, Fig. 77.

The dam was commenced with a bottom width of 20 feet, which was reduced in the first 16 feet to about 13 feet, by a batter on each side, and constituted the first season's work in 1883. Owing, however, to the cost of the original plan proving too great for the very limited

[1] *Trans. Am. Soc. C.E.*, vol. xix. p. 221, and plates 32 and 33; and "Reservoirs for Irrigation, Water-Power, and Domestic Water-Supply," J. D. Schuyler, pp. 163–173.

resources available, mainly due to the great expense of the carriage of cement to a place so difficult of access, on recommencing work in 1884, the width of the wall was reduced abruptly to 8½ feet, gradually tapering by a batter on the inner face, in the remaining 48 feet, to a top width of 2½ to 3 feet, Fig. 77. The dam containing about 3400 cubic yards of ashlar and rubble granite masonry, cost about £15,600; and the maximum storage has amounted to 10,855 million gallons. The line of resultant pressures with the reservoir full, falls about 17 feet outside the base of the dam, so that it is only by virtue of its arched form that it remains standing; whilst the pressure imposed on the masonry is decidedly excessive.

Zola Canal Dam.—An arched dam was erected in France in 1843, several years before the correct form for masonry dams, exposed

ZOLA CANAL DAM, FRANCE.

Fig. 78.—Elevation and Plan.

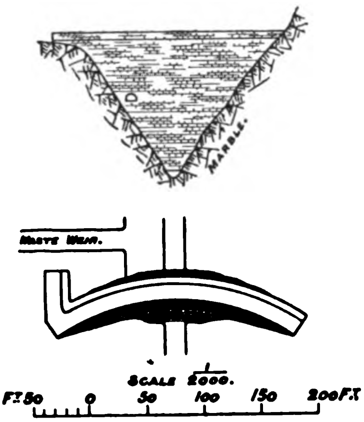
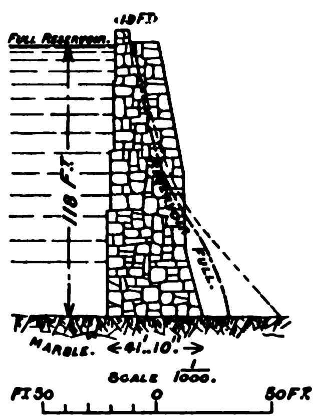


Fig. 79.—Cross Section.



to a considerable head of water, had been mathematically determined; and it is, therefore, not a matter for surprise that the materials, though not adequate, or designed, to sustain the water-pressure independently of the curvature, were not so distributed in the cross section as to render the thickness better proportioned to the strains to be borne, increasing as the square of the depth below the water-level. The dam was constructed across a narrow gorge of the Tholonet Valley, shown in section in Fig. 78, for collecting the rainfall flowing off the Jurassic strata of the valley, from a drainage-area of 9884 acres, and diverting it into a canal, named after the originator of the scheme, for the water-supply of Aix, a town founded by the Romans, about 19 miles north of Marseilles. The reservoir, $3\frac{3}{4}$ miles distant from Aix, contains 550 million gallons; but owing to the level of the intake of the canal, a quarter of this volume cannot be drawn off from the reservoir.[1]

The dam was constructed on a curve having a radius of 158 feet, and abuts against a stratum of marble on each side of the gap, with a length of 240 feet along the top, and 23 feet at the bottom, as indicated on the plan in Fig. 78. It was built of rubble masonry, pick-dressed on the face; and it was given a width of 42 feet at the foundations across the lowest part of the gap, reduced by three steps on the inner face, and a varying batter on the outer face, to 19 feet at the top, $119\frac{3}{4}$ feet above the foundations, where it is surmounted by a parapet on the inner side, 4 feet high, Fig. 79. As the line of resultant pressures for this section of dam, with the full head of water of 118 feet against it, taking only into account the resistance offered by its weight, would fall $11\frac{1}{2}$ feet beyond the outer toe of the wall at the base, it is evident that this dam also owes its stability to its curvature. The dam, however, would have been better adapted for aiding the horizontal arch, by its weight and its form, in resisting the water-pressure, if the large excess of masonry in the upper portion of the dam with its wide top, had been transferred to the lower portion, giving a curve to the outer face so as to increase the width rapidly towards the base, as indicated by a dotted line on the cross section, Fig. 79.

Sweetwater Dam, California.—Another arched dam was constructed in 1887–88, twelve miles from San Diego, across the torrential Sweetwater River flowing into the Pacific Ocean, whose drainage-area above the site of the dam, 7 miles from its mouth, is 186 square miles, the flow off which is so variable that it has ranged from practically hardly anything during three consecutive years, up to a maximum of 3200 million cubic feet in a single year. The reservoir formed by the dam has a capacity of 6144 million gallons; and it is intended that only one-half of the water in a full reservoir shall be drawn off for irrigation and water-supply during a year, so that storage for two consecutive dry years may be secured.[2] This, indeed, experience appears to show is an insufficient period to avoid occasional scarcity, especially as in two years out of three the flow off has been quite inadequate to fill the reservoir, a period of drought has already extended over three years since the reservoir was completed, and the loss from evaporation amounts to $4\frac{1}{2}$ feet in depth annually over the surface of the reservoir.

[1] *Annales des Ponts et Chaussées*, 1872 (1), p. 456, and plate 11, figs. 12 to 14.

[2] "The Construction of the Sweetwater Dam," J. D. Schuyler, *Trans. Am. Soc. C.E.*, vol. xix. pp. 201 to 218, and plates 23 to 30; and "Reservoirs for Irrigation, Water-Power, and Domestic Water-Supply," J. D. Schuyler, pp. 126 to 152.

Accordingly, the available yearly supply from the reservoir does not exceed 2180 million gallons.

The Sweetwater dam, as originally designed and commenced, resembled the Bear Valley dam in section, with a height of 50 feet, and a width of 10 feet at the base diminishing to 3 feet at the top; but during its progress the section was enlarged, and the height increased, first to 60 feet, and eventually to 90 feet, as this latter addition augmented the capacity of the reservoir nearly fivefold. The dam has been built with a curvature of 222 feet radius, having a length along the top of 380 feet, and at the bottom of 150 feet, Fig. 80. It has a maximum height of 99 feet to the top of the parapet, and can retain a head of water of about 92 feet; and it has a bottom width of 46 feet, reduced

SWEETWATER DAM, CALIFORNIA.

Fig. 80.—Plan.

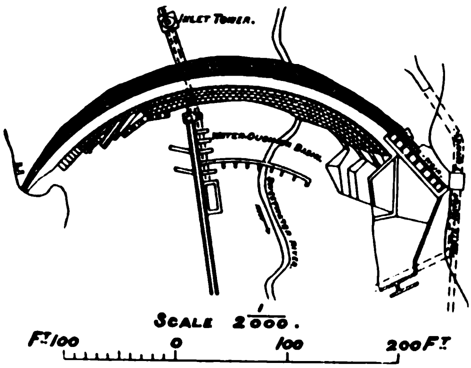
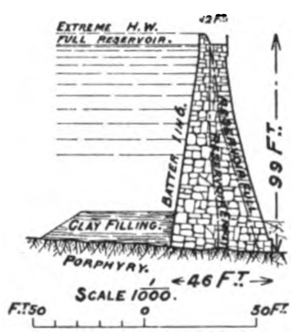


Fig. 81.—Cross Section.



to a width at the top of 12 feet by a batter on both faces, Fig. 81. A waste weir with seven openings, each 4 feet wide, was provided at one end of the dam, with its crest $5\frac{7}{10}$ feet below the top of the dam, closed by movable boards for the full storage. This waste weir, however, in conjunction with the outlet conduits, proved quite inadequate for the passage of a very exceptional flood in January, 1895, resulting from a rainfall of over 6 inches in 24 hours, which, rising $5\frac{1}{2}$ feet higher than the anticipated maximum, submerged the dam during 40 hours, flowing over the parapet with a maximum depth of 22 inches. It was then decided, besides doing the necessary repairs, to enlarge the capacity of the reservoir by raising the parapet 2 feet, and allowing for the raising of the maximum water-level in the reservoir $5\frac{1}{2}$ feet, up to the crest of the parapet, by keeping down 200 feet in length of the parapet, in the centre of the dam, to its original height for an additional waste weir, closed, when the reservoir is being filled, by movable boards resting against cast-iron frames. The dam was constructed of rubble masonry; but the repairs and additions were made with concrete, for facility and economy in execution. The original cost of the Sweetwater dam amounted to £48,765; whilst the cost of the repairs, improvements, and additions in 1895, was £6,250, making the total expenditure on the dam about £55,000.

As the line of resultant pressures, with a full reservoir, falls outside the middle third of the section, if considered irrespectively of the arched form, the dam does not fulfil the conditions of stability of a gravity dam; and the masonry would be in a tension at the upper face of the dam, with the reservoir full, if compressive strains were not introduced by the water-pressure on the arch. The curvature, however, has not only given stability to a section too slight for a straight dam, and whose form does not distribute the materials in the most advantageous manner; but it has also enabled it to withstand an unexpected additional head of water of $5\frac{1}{2}$ feet, together with the dynamical strains produced by the flow of water over the dam. An improvement in the form by a more vertical upper face, together with a flatter batter towards the base of the lower face, would doubtless have been introduced, if the upstream batter had not been fixed by the masonry of the original design already executed.

Remarks on the Arched Form for Masonry Dams.—The sites where a dam sufficiently short to be given a considerable curvature, enables a reservoir of adequate capacity to be formed above it, and where the rock on each side of the valley is firm enough to provide a suitable abutment for an arched dam, are necessarily restricted. Where, however, these conditions exist, a considerable economy of materials can be effected by curving the dam horizontally, with its convex face upstream. Such a slight section as the Bear Valley dam, Fig. 77 (p. 132), can only be justified on the ground of necessity, in the absence of funds for a more reliable structure, and the urgent need of water for irrigation; but even these pleas should not be allowed to prevail, if loss of life might be involved in the failure of such a dam; whilst the inadequacy of the waterway of the waste weir to carry off exceptional floods ought to be remedied without delay, as an essential precaution against the failure of the dam. This example possesses the special interest of demonstrating how greatly the curvature of a slight dam increases its strength, and to what an extent the reduction in section can be carried under these circumstances.

The Zola and Sweetwater dams are more stable instances of arched dams, in which the reduction of section has not been carried to an excess as in the Bear Valley dam; but both these dams, and especially the Zola Canal dam, might have been given a stronger section by modifying their form, so as to produce a more suitable distribution of the materials, without increasing their amount. All these examples of arched dams prove that, by giving a short dam across a rocky gorge a good curvature, a considerable reduction can be effected in the cross section of the dam, as compared with a dam which has to depend solely on its weight for its stability.

DIRECTION AND DISTRIBUTION OF PRESSURES IN MASONRY DAMS.

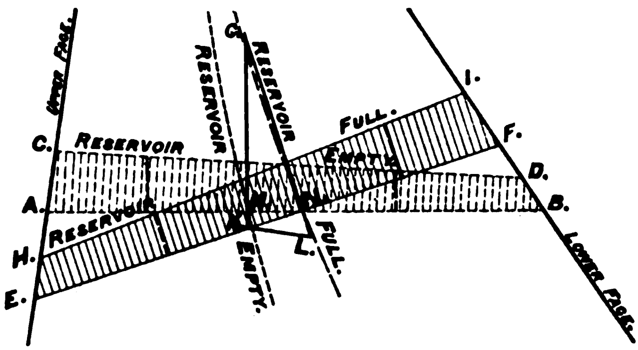
Masonry dams are subjected to two distinct sets of pressures, namely, vertical pressures due to their weight, which, when assumed to be concentrated into a single resultant pressure, may be regarded as acting vertically at the centre of gravity of the cross section of the dam; and secondly, horizontal pressures due to the water in the reservoir pressing against their inner face, and increasing with the depth below the water-level, whose resultant pressure may similarly be considered as acting at the centre of pressure, two-thirds of the depth below the surface of the water.

Line of Resultant Pressures with the Reservoir Empty.— The mean positions between all the pressures to which a dam is subjected across the whole of its width, on each side of which the pressures are practically equal throughout the whole depth of the cross section, with the reservoir empty, and also with the reservoir full, are known as the lines of resultant pressures. To find these lines graphically, the cross section of the dam is divided into a series of sections by horizontal lines at regular intervals, forming base-lines to the several sections, every one of them comprising the whole of the cross section of the dam above them.

When the reservoir is empty, the only pressures in the dam are those due to its own weight, all acting in a vertical direction. In the diagram, Fig. 82, representing a portion of the cross section of a masonry dam, **AB** is one of the horizontal lines by which the cross section has been divided into sections, and constitutes the base-line of the whole of the cross section above it, whose centre of gravity is at **G**. Then in order to find the point on the line of resultant pressures corresponding to this section, draw the vertical line **GK** from **G**, cutting the base-line **AB** at **M**; and this point of intersection of **GK** and **AB** thus obtained, is the point on the line of resultant pressures, with the reservoir empty, at the base of this particular section. After finding the centres of gravity of the other sections of the cross section of the dam, extending from the top of the dam to the several horizontal lines forming the base-lines of the respective sections, the intersections of vertical lines through the centres of gravity with the corresponding base-lines give the respective points on the line of resultant pressures, with an empty reservoir, which is obtained by drawing a line on the cross section passing through all these points.

DIAGRAM OF PRESSURES IN MASONRY DAM.

Fig. 82.



Line of Resultant Pressures with the Reservoir Full.—When the reservoir is full, the pressures in the dam are compounded of the vertical pressures due to the weight of the masonry, and the horizontal pressures due to the water. Then adopting a definite small scale to represent tons, the length of the line **GK** is made equal in length to the number of tons which the whole cross section of the dam above the base-line **AB** in Fig. 82, one foot in thickness, weighs, so that **GK** represents the sum of the vertical pressures acting at the centre of gravity **G.**

The total water-pressure on a vertical band of the inner face of the dam, 1 foot wide, extending from the top of the dam to the level of the base-line **AB**, is equal to the weight of a right-angled isosceles prism of water 1 foot thick, each of whose two equal sides adjoining the right angle is equal in length to the depth of the base-line below the surface of the water, or, in other words, is equal to the weight of a volume of water whose cubic contents amount to half the square of the depth of the base-line below the water-level, multiplied by 1 foot. This water-pressure, which is equivalent to the sum of the water-pressures on the dam from the water level to the base-line, may be considered as acting at right angles to the inner face of the dam, at the centre of pressure, which is two-thirds of the depth below the water-level. For the sake of simplicity, this centre of pressure may be assumed to coincide in level with the centre of gravity, which would be the case if the dam was triangular in cross section, and the highest water-level reached its apex. Owing to a certain width being given in practice to the top of the dam, and its being usually raised above the highest water-level, the centres of gravity of the upper sections of the dam are, indeed, rather higher than the corresponding centres of pressure; but on account of the rapid widening of a high masonry dam towards its base, the centre of gravity is proportionately lowered in the deeper sections, so that in the total cross section of the dam, it approximates to the level of the centre of pressure.

Accordingly, assuming the resultants of both sets of pressures to act at **G**, by the parallelogram of forces, if **KL** in Fig. 82, drawn from **K** at right angles to the inner face, is made equal in length, by the adopted scale, to the number of tons of the water-pressure above the base-line calculated as above, the line **GL**, joining **G** and **L**, give the resultant pressure, with the full reservoir, in tons by its length measured by the given scale, and also the direction of this pressure, so that the intersection of **GL** with the base line **AB** at **N**, is the point on the line of resultant pressures, with the full reservoir, corresponding to the portion of the cross section of the dam above **AB**. The other points on the same line relating to the other sections formed by the series of horizontal lines can be similarly determined, by obtaining graphically the intersections of the several lines corresponding to **GL** in these sections, with their respective base-lines; and the line drawn on the cross section of the dam, joining these several points of intersection, is the line of resultant pressures with the reservoir full.

Distribution of Pressures with the Reservoir Empty.— The general distribution of pressures on any base-line in the cross section of a masonry dam, can be indicated graphically in the following manner. The pressures on any base-line **AB** in Fig. 82, when the reservoir is empty, consist of the weight of the masonry of the portion of the dam above the base-line, which, for a thickness of one foot, is given in tons by the length of **GK** to the scale selected, as explained above. Now since **GK** is the resultant of these pressures distributed over **AB**, and cuts **AB** in **M**, it is approximately correct to assume that the pressures on each side of **GK** are equal, and, therefore, that **AM** and **BM** each bear half of the total weight of the dam above them. Let W represent the total weight in tons of the portion of the cross section of the dam above **AB**, 1 foot thick. Then $\frac{W}{2} \div AM$, where AM is the length in feet, gives the mean pressure in tons per square foot on a horizontal strip of the dam, of length AM, and 1 foot wide, and

$\frac{W}{2} \div$ BM gives the similar mean pressure on a strip of length BM, 1 foot wide, Fig. 82. Let x represent the mean pressure on **AM** in tons obtained in the first case, and y the mean pressure on **BM** in tons in the second case. Erecting a perpendicular equal to x in tons to a suitably large scale, at the central point of **AM**, and another perpendicular equal to the number of tons represented by y on the same scale at the middle of **BM**, and joining the upper extremities of these two verticals by the line **CD**, the band **ABDC**, shaded by dotted lines, is obtained, representing approximately the distribution of the pressures along **AB** by the varying height of the band, corresponding to tons on the square foot measured by the chosen scale at right angles to **AB**.

Distribution of Pressures with the Reservoir Full.—The portion of the dam which may be regarded as having to sustain the distributed resultant pressures with a full reservoir, due to the cross section of the dam above the base line **AB**, 1 foot in thickness, is along a strip **EF** in Fig. 82, a foot thick, determined by drawing a line across the dam at right angles to the direction of the resultant pressure **GL**, and passing through its intersection with the base-line **AB** at **N**. Then, assuming as before that half the total pressure P in tons, represented by the length of the resultant GL to the first selected scale, is borne by each of the strips EN and FN respectively, 1 foot thick, the mean pressures in tons per square foot on either side of N are obtained from $\frac{P}{2} \div$ EN and $\frac{P}{2} \div$ FN respectively. Next, erecting perpendiculars on **EF**, as before, midway between **E** and **N** and **F** and **N**, corresponding in length, by the larger scale previously adopted for the same purpose, to the values in tons of the mean pressures, the line **HI** drawn through the extremities of these verticals, forms another band with **EF**, shaded by full lines, representing approximately the distribution of pressures along **EF**, with the reservoir full, in tons per square foot, according to the varying width of the band measured by the chosen scale at right angles to **EF**.

Conditions of Stability with regard to Pressures in Masonry Dams.—If on finding the bands of pressures graphically, as indicated above, in any design for a masonry dam, the two lines defining these bands across the cross section of the dam intersect within the section, either with the reservoir empty or full, the production of tension between the point of intersection and the adjoining face of the dam is foreshadowed, showing that the proposed cross section fails to comply with the condition that the lines of resultant pressures should fall within the middle third, and that, consequently, a modification of the form of the dam is required. It is, moreover, essential that the maximum weight on the masonry near the inner face with the reservoir empty, and the maximum pressure near the outer face with the reservoir full, and the pressures at the inner and outer parts respectively, of the base of the dam on the foundations, under the two conditions, should not exceed the limits which they are fully able to bear without any injury. On this account, and also in order to increase the proportion which the vertical pressures in the dam bear to the constant water-pressures for a given section, and thus enable the width required in the lower portion of a high dam to be reduced, it is advisable to employ the hardest and densest masonry available in constructing a dam.

On the other hand, when a dam is built on fissured rock, water is liable to ascend from these fissures under pressure, thereby counteracting to some extent the weight of the dam, and, consequently, reducing its stability. It is, therefore, advisable to avoid adopting such foundations if possible; but if no other better site is available, the fissures should, if practicable, be closed, or the issuing water should be led by drains beyond the outer face of the dam, or the upward pressure must be compensated for by enlarging the cross section of the dam, and thus increasing its weight, which has been one of the reasons assigned for the excessive section given to the Gileppe dam near Verviers in Belgium.[1]

The tendency of the water-pressure to push forward the dam bodily on its foundations, is prevented by founding the masonry on the solid rock at somewhat irregular levels, aided often by the depth below the original surface to which the foundations have to be carried down, as exemplified by the Vyrnwy, Assuan, Thirlmere, and New Croton dams. Any sliding forward of the masonry or its joints, is provided against by the tenacity of the cement mortar employed, and by the avoidance of regular, horizontal joints across the dam in its construction, which latter provision of rough joints is also valuable in preventing leakage through the dam, which has sometimes proved considerable towards the base of high dams, as for instance at the Gileppe dam.

FORMS AND CONSTRUCTION OF MASONRY DAMS.

Some dams for forming reservoirs for irrigation in Spain, constructed in the sixteenth and eighteenth centuries, and the earlier half of the nineteenth century, consist of large masses of masonry, in which the magnitude and general thickness of the dams throughout appear to have been considered of more importance for withstanding the water-pressure, than their form, or in some cases the soundness of their foundations,

[1] A section of the Gileppe dam is given in "The Encyclopædia Britannica," 9th edition, vol. xxiv. p. 407, fig. 6; and *Proc. Inst. C.E.*, vol. xcvi., plate 6, fig. 3.

Fig. 83. The masonry dam also of Gros-Bois, in France, built in 1830–38, with a top width of $21\frac{1}{3}$ feet, stepped forward into the reservoir on the inner face to the extent of $21\frac{1}{3}$ feet, in a total height of $71\frac{1}{2}$ feet, and with a batter of only 1 in 20 on the outer face, besides being too massive in its upper portion, would have been much better adapted for resisting the water-pressure if it had had its section turned completely round, with the outer face on the inner side, and the stepped face downstream,[1] Fig. 86. It was only, indeed, in designing the Furens masonry dam in France, constructed in 1861–66, that the form in cross section was adjusted to the varying pressures, increasing in the case of the water-pressure in proportion to the square of the depth below the water-level, Fig. 92, and so as to fulfil the conditions of stability of the lines of pressures being kept within the middle third of the cross section, and the pressures per square foot on the masonry and foundations not anywhere exceeding the prescribed limit.[2]

Correct Form of Cross Section for Masonry Dams.—It is evident that for the cross section of a dam to be made proportionate in thickness downwards to the increasing vertical pressure due to its weight, and to the smaller, but still rapidly increasing pressure due to the water, the width at the top might be nil, increasing gradually at first from a vertical inner face by means of a curved batter on the outer face, flattening gradually from a nearly vertical face at the top, to an angle approximating to 45° to the horizontal towards the base in high dams. By this means, aided sometimes by a little batter towards the bottom on the inner face, the width of the cross section eventually rapidly increases towards the foundations, in order that the supporting area of the masonry may be augmented suitably to bear the rapidly increasing pressures with the depth, and that the pressures imposed upon the foundations by a high dam may not become excessive. With the exception of the width at the top, given to all masonry dams, the Chartrain, Villar, Ban, Periyar, and New Croton dams approximate to the theoretically correct form of cross section (see cross sections, pages 165 and 172).

Modifications introduced in the Form of Cross Section.—The top of a masonry dam is always given a certain width, to secure it from injury by blows of waves in the reservoir, or shocks of floating ice, and also sometimes to provide a roadway across the valley. It is, moreover, usually raised some feet above the highest water-level in the reservoir, so as to prevent the dam being overtopped by the water,

[1] "Les Réservoirs d'Alimentation, Canal du Centre et Canal de Bourgogne," A. J. B. Fontaine, Congrès International de Navigation intérieure, Paris, 1892, pp. 14 and 15.

[2] *Annales des Ponts et Chaussées*, 1866 (2), pp. 196 to 211, and plate 127.

except where provided for by the lower crest of the waste weir; for though the flow of water over the top of a masonry dam, does not at all necessarily entail injury, as in the case of an earthen dam, nevertheless, it is liable to produce erosion by its fall in front of the outer toe of the dam, unless special provision has been made against this action, as is always done at the base of a waste weir and along the waste-water channel. The width at the top ensures compliance with the condition of keeping the lines of the resultant pressures within the middle third in the upper portion of the cross section; and the necessity of a wide base in high dams for keeping the pressures per square foot on the masonry and foundations within due limits, makes the lower part of the cross section exceed the width necessary for keeping the lines of pressures within the middle third. Indeed, in most high dams, it is only in the central portion of the cross section, where the batter of the outer face is maintained somewhat steep towards the middle part, as in the Chartrain and Ban dams, Figs. 104 and 106 (p. 165), that the fulfilment of the condition of the middle third has to be attended to in the design.

Where a masonry waste weir is provided along a portion of the dam, as at the New Croton dam, or where a portion, or the whole of the dam is made to serve as a waste weir, of which the Vyrnwy and the La Grange dams are examples, the top is made thick, and is rounded off on the downstream side; and the outer face is given a considerable batter, formed in a series of steps to break the fall of the water, or curving out rapidly towards the base so as to make the overflowing water assume a nearly horizontal direction, and provided with a massive outer toe to sustain the shock of the descending water when altering its direction, Figs. 101, 108, and 110 (pp. 165 and 172).

Materials employed in the Construction of Masonry Dams.—Though all dams constructed of stone and mortar are commonly termed masonry dams, they are not all strictly masonry structures, which imply ashlar or rubble stone laid on a flat face, with each stone bedded in mortar. Occasionally some form of concrete has been resorted to, either with the object of dispensing with the skilled labour of masons where they are difficult to procure, or with a view to economy, or with the object by avoiding all kinds of joints, except in the facework, to render the dam more watertight. Thus, for instance, the Vyrnwy, Thirlmere, and Burrator dams have been built of a sort of rubble concrete, in which large blocks of stone are embedded in a mass of concrete enveloping them, and only the facework is formed of regular masonry; whilst the bulk of the Periyar dam has been constructed with concrete in the absence of an adequate supply of masons, the concrete being encased on both sides with rubble masonry. Even in dams built of masonry, straight joints are now avoided as far as possible to check the leakage of water through the dam. Efforts have also been made to prevent the infiltration of water under pressure into the interior of the dam by making the upstream face watertight, either by a rendering of cement mortar all over it, or, as in the case of the Remscheid dam in Germany, by a coating of asphalt combined with cement mortar protected by a facing of brickwork.

As it is important to give these masonry or concrete dams a high specific gravity, the sand used for mortar, and still more the larger quantities required for concrete should be obtained from crushed quartz, or by crushing the hardest rocks available. By this means the mass of the dam is enabled to sustain higher compressive strains than with ordinary sand; and the stability of the dam, so far as it depends on its weight, is increased. Every care, moreover, has to be exercised in the construction to render the masonry and the concrete as compact as possible, so as to form a thoroughly solid, impervious mass.

FAILURES OF MASONRY DAMS.

The failure of a masonry dam may be caused by yielding or fissured foundations; by insufficient bonding of the base of the dam into the solid rock; by an inadequate or ill-designed cross section; or by the use of faulty materials, or want of care in construction. These sources of weakness, moreover, are often aggravated by the inadequacy of the waterway of the waste weir to discharge an exceptional flood, which, flowing into the reservoir, raises its water-level several feet higher than originally contemplated, and imposes an additional water-pressure, which the dam has not been given a sufficient margin of stability to withstand.

Failure of Foundations.—The original Puentes dam, constructed of rubble masonry in 1785–91 across the River Guadalentin, about 6 miles above the town of Lorca in Spain, was 788 feet long, formed of three straight portions, the two shorter side portions projecting from the slopes of the valley transversely upstream, and joining at an angle the longer central portion directly facing the stream.[1] This dam, 164 feet high above the bed of the river, was built with a vertical inner face, and a stepped outer face by which it was gradually reduced in thickness, as it was carried up, from a bottom width of 151 feet to a top width of $35\frac{3}{4}$ feet; and, accordingly, the general outline of its faces was better designed than that of the other early, high, Spanish masonry dams,[2] though its thickness was excessive at the top, and continued too great in a diminishing proportion down to near its base, where it finally

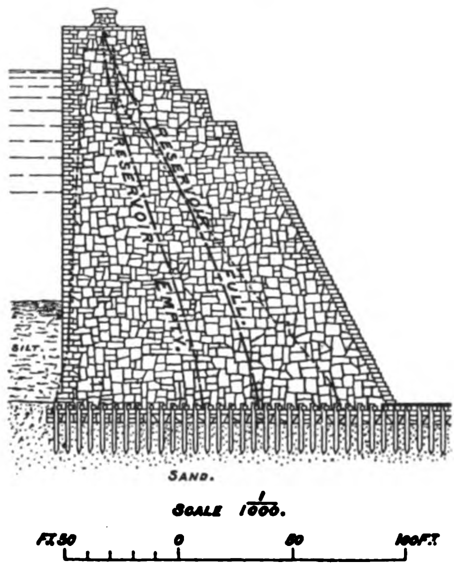
[1] "Irrigations du Midi de l'Espagne," Maurice Aymard, Paris, 1864, pp. 254–259, and plate 13.

[2] *Annales des Ponts et Chaussées*, 1866 (2), plate 127, figs. 13, 14, 15, 17, and 20.

approximated to the width of the Furens dam near its bottom, having a very similar height, Fig. 83. Notwithstanding the large excess of masonry in the upper portion of this dam, the maximum pressure at the toe of the outer face only attained 7½ tons per square foot; but apparently the dam was not very carefully built. The foundations across the bed of the river, however, constituted the weak part of the structure; for the central 70 feet of the dam, instead of being carried down to the solid rock, were built upon a series of bearing piles driven down 22 feet into the superficial, alluvial stratum covering the bottom of the river valley. For the first eleven years after the completion of the dam, the reservoir was never filled up sufficiently to attain more than half its full depth; but when a very heavy flood, in 1802, for the first time raised the head of water against the dam to 134½ feet across the bed of the river, a passage was forced by the water through the alluvial stratum underneath the dam. The foundations having been thus undermined, a breach was formed through the dam, 56 feet in width with a height of 108 feet; whilst the whole contents of the reservoir were discharged down the valley in the course of an hour.

OLD PUENTES DAM, SPAIN.

Fig. 83.



A new Puentes, or Lorca, dam was erected in 1885 at about the same place, in which the modern form of cross section was adopted, and it was carried down to a foundation of rock 79 feet below the bed of the river;[1] and an approximate section of this new dam is indicated in dotted lines on the cross section of the old dam in Fig. 83. A comparison of the superposed new section with the old cross section, shows very clearly how largely the correct form economizes masonry, and, consequently, reduces the pressure on the foundations by diminishing the weight of the dam. The new dam, 535 feet in length, has, moreover,

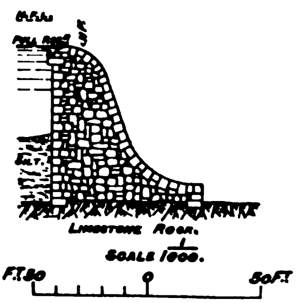
[1] "Réservoirs établis en Espagne," A. de Llaurado, Congrès International de Navigation intérieure, Paris, 1892, pp. 5 to 9.

been given a curved form in place of the rampart sort of shape of the old dam.

Sliding Forward of Masonry Dam on Foundations.—The Austin dam, 1091 feet long and 68 feet high, was built in 1891–93 straight across a gorge of the Colorado River, $2\frac{1}{4}$ miles above Austin in Texas, with a cross section designed for allowing the discharge of the river, after filling the reservoir, to flow over its crest; and its foundations were laid on limestone rock of rather variable character, with only a small amount of bonding into the rock at its upper and lower faces, Fig. 84. The reservoir formed by this dam extended over 20 miles up the river, with an average width of only 800 feet; and the capacity of this reservoir originally amounted to $14{,}562\frac{2}{3}$ million gallons, but was nearly half silted up within seven years of the completion of the dam, owing to the large volume of turbid water passing through it.[1] A flood coming down the river in June, 1899, raised the water in the reservoir $9\frac{4}{5}$ feet over the crest of the dam; but this flood passed over the dam without any apparent injury. In April, 1900, however, an exceptionally heavy flood, resulting from a downfall of rain reaching 5 inches in 15 hours on previously sodden ground, raised the reservoir 11 feet above the crest of the dam, and breaking off 500 feet of the central portion of the dam from the remainder on each side, drove it bodily downstream a distance of about 60 feet in an upright position.[2]

AUSTIN DAM, TEXAS.

Fig. 84.



The failure of this dam must be attributed to the inadequate bonding of its foundations into the rock, to resist the increased water-pressure of the raised reservoir, aided probably by a reduction in its weight resulting from an upward pressure of the water under its base, coming through unsound places in the limestone, and possibly also by some scouring away of the rock at its outer toe by the flood discharge over the dam.

Failure of Dam from Faulty Design, with Tension in Masonry.—The Bouzey masonry dam was erected across the valley of the little River Avière, near Epinal in France, in 1878–80, on somewhat fissured sandstone rock, in a straight length of 1706 feet. The dam was given a maximum height of $75\frac{1}{2}$ feet, a width of 13 feet at the top and for $14\frac{3}{4}$ feet lower down, from which point it was widened out by a curved batter on the outer face to a width of $40\frac{1}{2}$ feet at the base, above

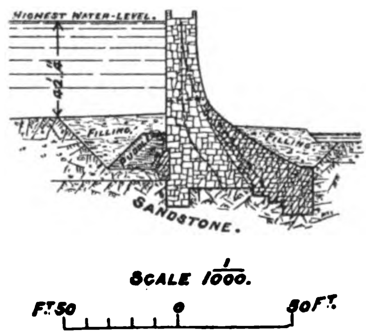
[1] *Engineering News*, Chicago, 1900, vol. xliii. pp. 135–136.

[2] *Ibid.*, vol. xliii. pp. 244–245 and 250 to 254.

the deeper foundations carried down by trenches into the rock with a thickness of $6\frac{1}{2}$ feet under both faces, Fig. 85. In March, 1884, when the reservoir had been filled for the first time to a height of $42\frac{2}{3}$ feet above the ground-level, about 430 feet of the dam were pushed forward slightly, assuming a curved form with a maximum displacement of about a foot downstream, producing vertical cracks in the masonry at the centre and each extremity of the curved portion, and fissures at the foundations, which gave vent to a leakage of about $6\frac{3}{4}$ million gallons per day.[1] This movement was clearly due to the reduction in weight of the dam, resulting from the upward pressure of water finding access to the base of the dam through the fissured rock, aided by inadequate bonding of the foundations into the rock, and the slightness of the cross section.

BOUZEY DAM, FRANCE.

Fig. 85.



The tendency of the dam to slide bodily downstream on its foundations, was prevented by the addition made to the thickness of the dam on the downstream side, in the form of a counterfort for a considerable length, as shown in the cross section by shading, Fig. 85; and the percolation of water under pressure beneath the base was hindered by a capping of masonry along the inner face near the base, as also shown by shading in Fig. 85, encircled by a mound of puddled clay. These works, carried out in 1888–89, effectually prevented any further movement of the dam on its foundations, when the reservoir was filled to its normal height; whilst the leakage removed by drains under the downstream part of the dam, was reduced to a maximum discharge of $1\frac{3}{4}$ million gallons a day of perfectly clear water with a full reservoir.[2] The capacity of the reservoir formed by this dam was 1560 million gallons, designed to supply the summit-level of the Canal de l'Est with water.

Though the lower part of the cross section of the dam appears to have been adequately widened by the additions, increasing the width at its base from $40\frac{1}{2}$ feet up to $68\frac{3}{4}$ feet, the width was so slight just above the addition, at a depth of 35 feet below the water-level of the full reservoir, that the line of resultant pressures with the reservoir full came

[1] "Alimentation des Canaux dans l'Est de la France," R. Denys, Congrès International de Navigation intérieure, Paris, 1892, pp. 10–11.

[2] *Ibid.*, pp. 11–12, and plate 1.

within $3\frac{1}{2}$ feet of the outer face of the dam, or over $2\frac{1}{4}$ feet beyond the middle third of the cross section, Fig. 85, causing tension in the masonry at the inner face at this level, which has been estimated at $1\frac{1}{4}$ tons per square foot.[1] In April, 1895, the upper portion of the dam, for a length of about 560 feet in the central part, was broken away at about the level already indicated as unduly weak and subject to a tensional strain.[2] An initial crack was no doubt formed somewhat horizontally for some distance along the inner face, at the level where rupture eventually occurred, as soon as the reservoir was filled to its full height, by tension on the masonry at the junction of the work of two successive seasons, though the masonry was good, and special care had been taken in connecting the new work with the old. The failure of the dam, some years after it had borne the pressure of the full reservoir, has been explained by the consideration that the initial crack would admit water under pressure, which by its upward thrust would relieve the weight of the portion of the dam above it, and convert the compressive strain at the inner extremity of the crack into a tensional strain, thereby leading to an extension of the crack further into the dam, and at the same time increasing the compressive strain on the outer face, till at length the crack and the strain would be carried so far as to suffice, not merely to cause the overturning of the central, upper portion of the dam if unconnected at the end, but even to sever its vertical connection with the uninjured side portions.[3]

The dam, it appears, was not designed originally to bear the water-pressure of the reservoir filled higher than $6\frac{1}{2}$ feet below the water-level of the full reservoir, to which, however, it was eventually raised. The cross section of the dam, even with the considerable additions introduced, was only adequate in width for about two-fifths of its height; and in its central portion, it differed very materially from the correct form at that depth, as established and proved by experience in the Furens dam about fifteen years earlier, which was built at a much more favourable site, with a much shorter length and an arched form.

The Gros-Bois dam, erected across the River Brenne during the construction of the Burgundy Canal in France in 1830–38, for storing up water for the canal, and founded upon a very hard, laminated stratum of clay, furnishes a much earlier instance of a dam which was unstable, owing to a very defective form, and an argillaceous foundation, but has not actually failed.[4] This rubble masonry dam, 1805 feet long, $95\frac{1}{3}$ feet

[1] *Proc. Inst. C.E.*, vol. cxxvi. pp. 91 to 97.

[2] "Rupture de la Digue de Bouzey: Rapport d'une Commission spéciale, et Délibérations du Conseil général des Ponts et Chaussées," Paris, 1895.

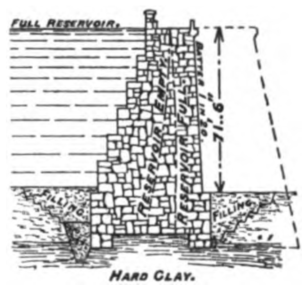
[3] *Ibid.*; and *Proc. Inst. C.E.*, vol. cxxvi. pp. 93 to 95.

[4] "Cours de Construction: La Navigation des Rivières et des Canaux,"

maximum height, with a width at the bottom of $52\frac{1}{2}$ feet, and at the top of $21\frac{1}{3}$ feet, possesses, like most of the early dams, an ample cross-section with the materials badly distributed, Fig. 86. The special fault, however, of this dam is that its cross section faces the wrong way in regard to its form; for if the dam had had its position exactly reversed, so as to have its nearly vertical face on the upstream side, and its stepped face downstream, as previously alluded to, the dam would have been quite stable on a sound foundation, though its mass is somewhat excessive, Fig. 86. Owing to the slippery stratum on which it rests, the dam showed a tendency to move forward when subjected to a head of 73 feet of water, notwithstanding its large mass; but its motion, after reaching a maximum of about 2 inches, and producing cracks in the lower part of the dam near each end of the deepest part, was arrested by the erection of nine counterforts against the downstream face, 18 feet thick at the top, and increased by a batter of 1 in 10 on each side to 37 feet at the base, and extending about 26 feet downstream at the top, and 38 feet at the bottom, as shown by dotted lines in Fig. 86.[1] The unsuitability, however, of the form of the cross section for resisting water-pressure, has caused the line of resultant pressures, with a full reservoir, to pass 4 feet outside the middle third at the base; and a pressure estimated at $9\frac{1}{2}$ tons per square foot, has been imposed upon the outer toe of the dam, which, in spite of the support afforded by the counterforts, has led to the necessity of yearly repairs to the downstream face between the counterforts. Probably the thickness of the cross section, and the soundness of the masonry, have, in this case, prevented the development of a horizontal fissure along the inner face near the base of the dam.

GROS-BOIS DAM, FRANCE.

Fig. 86.



Failure of Masonry Dam from Faulty Construction.—Though inadequate theoretical knowledge in early days, and errors of judgment in more recent times, have occasionally led to the erection of unstable masonry dams, whilst exceptional floods, or unforeseen defects in the foundations have involved their failure, it might have been anticipated that any use of unreliable materials would have been most

M. Minard, p. 330, and Atlas, plate 28, fig. 576, and plate 29, figs. 596 and 597; and *Annales des Ponts et Chaussées*, 1853 (2), p. 208, and plate 53, fig. 12.

[1] "Réservoirs d'Alimentation," A. J. B. Fontaine, Congrès International de Navigation intérieure, Paris, 1892, p. 15.

carefully avoided, in view of the great destruction of life and property which is generally entailed by their failure, from the letting loose of a great, rapid flood down the valley in the sudden emptying of a large reservoir. The failure, however, of the Habra dam in Algeria, in December, 1881, has been attributed to the poor quality and great fineness of the sand employed,[1] the presence of quicklime in the hydraulic lime used for the mortar, and the porosity of the stone forming the masonry, so that the dam was unable to withstand the large additional strain to which it was subjected by an exceptional flood.

The Habra dam was erected in 1865–71, on a solid sandstone foundation, across the valley of the River Habra, for a length of 1066 feet along the top, continued across a shallow part of the valley on one side by a waste weir 410 feet long, whose crest was $5\frac{1}{4}$ feet below the top of the dam, which it was assumed would suffice for the discharge of the greatest floods of the river, estimated at about 15,500 cubic feet per second, without raising the water-level in the reservoir the $5\frac{1}{4}$ feet required to reach the level of the top of the dam, on the outer side of which a parapet was built, about 8 feet high, to prevent waves in a full reservoir washing over the dam.[2] The surface of the full reservoir, when level with the crest of the waste weir, was $111\frac{1}{2}$ feet above the bottom of the valley; and at this level it contained 6600 million gallons of water, which were stored in the winter for the irrigation during the summer of the valley below. The cross section of the dam was made in accordance with the approved modern form, the maximum pressure being only $4\frac{1}{5}$ tons on the square foot at the inner toe with the reservoir empty, and $5\frac{1}{3}$ tons at the outer toe with the reservoir full; whilst the curves of resultant pressures were kept within the middle third of the section, Fig. 87.

The waste weir of the Habra dam was at first built in a somewhat light form, as shown by Fig. 88, with only a narrow trench of concrete carried down to the solid rock under the inner face, with the object of preventing the infiltration of water, and consequent undermining of the structure founded upon an upper layer of tufa resting upon a bed of gravel overlying a stratum of sand above the rock; and the concrete trench proved so difficult to put in, that in some places its thickness was reduced at the base to only 2 feet, imposing a pressure reaching a maximum of 32 tons on the square foot. In March, 1872, an unusually heavy flood, following on a rainfall of $7\frac{5}{8}$ inches in 3 days, raised the water-level of the reservoir $6\frac{1}{2}$ feet above the crest of the waste weir,

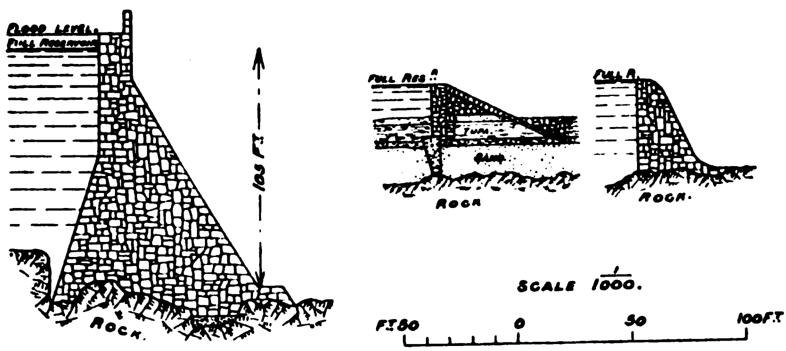
[1] The red earth, consisting of 1 part of clay to 2 parts of sand, specified to be used for cheapness in the central part of the dam, was soon abandoned, as the cost of procuring the red earth proved to be much the same as that of sand.

[2] "La Mise en Valeur de la Plaine de l'Habra," L. Pochet, *Annales des Ponts et Chaussées*, 1875 (1), pp. 275 to 334, and plates 9 and 10.

giving, with the flow through the outlet culverts, a discharge of 24,700 cubic feet per second; and a length of about 165 feet of the waste weir was carried away by the flood, which in escaping through the gap thus formed, scoured out the bottom right down to the rock, washing away about 262,000 cubic yards of materials, which were deposited in the bed of the river below. After this failure, the waste weir was rebuilt as a solid wall of masonry founded upon the sandstone rock, with a vertical

HABRA DAM, ALGERIA.

Fig. 87. Main Dam. Fig. 88. Original Waste Weir. Fig. 89. Reconstructed Waste Weir.



upstream face, a width across the rounded crest of $10\frac{2}{3}$ feet, and a batter on the downstream face of 1 in $2\frac{1}{3}$, to fit the calculated slope of the water when pouring over with a depth of $6\frac{1}{2}$ feet, and terminating in a curve to direct the falling water into a horizontal course, Fig. 89.

In December, 1881, a still more exceptional rainfall of $6\frac{1}{4}$ inches in a very short period, produced a flood with an estimated discharge of 176,600 cubic feet per second, over ten times the discharge for which provision was originally made; and it is stated that the water flowed over the top of the parapet of the main dam with a depth of about $3\frac{1}{4}$ feet, which is equivalent to a raising of the water-level of the reservoir $11\frac{1}{6}$ feet above the intended maximum water-level, and about 10 feet above the level reached during the flood of 1872 when the waste weir gave way. On the first filling of the reservoir, water began to ooze through the dam with a pressure of only 33 feet of water; and as the water-level was raised, the dam exhibited the appearance of a huge filter, accompanied with the washing out of lime, owing to the porosity of the sandstone employed, and the inferiority of the mortar.[1] The leakage appears to have diminished considerably in course of time; but the want of compactness of the masonry thus indicated remained;

[1] *Annales des Ponts et Chaussées*, 1875 (1), pp. 329 to 332.

and at the height of this remarkable flood, a breach was formed through the dam at the deepest part of the valley, along a length of about 330 feet from top to bottom.[1] The great raising of the water-level in the reservoir by the flood, which was aggravated by the high parapet, did, indeed, impose a quite unexpected strain on the dam; but it is probable that the dam, with its satisfactory cross section, if well built with thoroughly sound materials, would have been fully able to bear the additional strain, even if, as suggested, it attained twice the calculated stress. Moreover, the stability of the dam would have been increased if its inner face had been covered with a watertight coating, and especially if, after the warning given by the flood in 1872, the capacity of discharge of the waste weir had been enlarged, and even if the high parapet on the main dam had been lowered. Allowance, however, must be made for the difficulty of procuring suitable materials in a somewhat inaccessible desert district, and the quite unprecedented flood which occasioned the catastrophe.

Remarks on Failures of Masonry Dams.—The above examples of the failures of masonry dams suffice to show that, notwithstanding the great advance that has been achieved in the theoretical design of masonry dams within the last forty years, certain practical considerations must not be overlooked. In the first place, it is of the utmost importance that all the materials should be of excellent quality, and the masonry built in the most impervious manner possible; and if thoroughly reliable materials cannot be procured at a reasonable cost, the erection of a high masonry dam should be abandoned. The provision, moreover, of a watertight coating on the inner face of the dam is very advantageous in preventing infiltration into the masonry, and the consequent weakening of the structure. The necessity of so designing a dam as to secure it from any chance of tension on the inner face, more especially when the masonry is not very sound, or its cohesion has been impaired at the junction between two seasons' work, is well illustrated by the failure of the Bouzey dam, particularly on the assumption of the correctness of the theory of a progressive rupture from an initial horizontal crack resulting from tension on the masonry.

The importance of a solid, compact, rock foundation for a masonry dam is so obvious that it is hardly necessary to insist upon it; but the danger of upward pressure from springs or fissures under the dam, reducing the stability due to the weight of the dam, and modifying the pressures with a full reservoir, must be carefully guarded against. The tendency also of a dam to slide forward when exposed to a considerable head of water, especially if resting upon somewhat loosely stratified

[1] "La Rottura della Traversa dell' Habra nella Provincia d'Oran (Algeria)." G. Crugnola, *L'Ingegneria civile e le Arti Industriali*, vol. viii. 1882, pp. 49 to 51.

rock, unless firmly anchored down by the depth of its foundations, must be borne in mind; whilst any dip downstream of the strata increases the insecurity of the dam.

Lastly, the estimates of the maximum discharge of rivers in certain dry districts appears so very unreliable, owing to the very uncertain character of the rainfall, as exemplified by exceptional floods on the Colorado and Habra rivers, that very ample provision for such emergencies should be made in the construction of the waste weir, and other facilities for the discharge of water from the reservoirs exposed to such conditions. Moreover, the dam in such cases should be built with special care, and with a large factor of safety, so that it may be capable of bearing the increased strain which may be unexpectedly thrown upon it under a combination of unfavourable meteorological conditions; whilst the waste weir should be constructed with the same care as the main dam, and somewhat thicker in proportion to its height to allow for the additional stress due to water flowing over it, and with special precautions against sliding forward and the undermining of its outer toe by the falling water.

The results of some experiments, on a small scale, with two wooden models of a portion of a masonry dam, cut into a series of horizontal and vertical slices respectively, have led two mathematicians recently to the conclusion that "the current idea that the critical sections of a dam are the horizontal sections is entirely erroneous. A dam collapses first by the tension on the vertical sections of the tail.[1]" Engineers, indeed, have always realized that the vertical stresses due to the weight of the dam are greater than the horizontal stresses due to the water-pressure on the inner face; but they have also at the same time recognized as obvious, that the vertical stresses are sustained by the solid rock foundation, whereas the horizontal stresses have to be borne by the mass of the dam. In order to produce a tensile strain on the vertical sections near the outer toe, it would be necessary for the water-pressure to tip up bodily very nearly the whole mass of the dam, which, unless it could penetrate under the foundation, it has no tendency to do. Moreover, this theory is entirely at variance with practical experience. The dams which have proved unstable have either moved bodily forward horizontally, like the Gros-Bois and Austin dams, or have been first dislocated horizontally, at an unduly narrow part, by tension at the inner face, and then pushed forward, as exemplified by the Bouzey dam. There is no recorded instance of a masonry dam having collapsed by the formation of a vertical crack near the outer toe, and the subsequent tipping over of the mass of the dam.

[1] "On some Disregarded Points in the Stability of Masonry Dams," K. Pearson and L. W. Atcherley, London, 1904.

CHAPTER VIII.

TYPICAL MASONRY DAMS.

Classification of notable Dams for description—**Curved Masonry Dams:** advantages of curvature; Furens Dam, objects, first correctly designed masonry dam, greatest pressures, description, cost, arrangements for discharge and floods, capacity of reservoir formed, second dam, increased capacity obtained; Remscheid Dam, Germany, object, description, watertight inner facing, provisions for discharge from reservoir, waste weir, cost; Hemet Dam, California, construction, site, present and proposed height and capacity of reservoir, form, materials, discharge pipes through dam, waste weir, uncertainty of supply; San Mateo Concrete Dam, California, object, incomplete, construction, form, present and future capacity of reservoir, arrangements for discharge; New Puentes Dam, Spain, site, description, capacity of reservoir, object; Chartrain Dam, France, site, objects, form, capacity of reservoir, available storage, cost, description, materials, pressures, waste weir, discharge pipes through dam; Villar Dam, Spain, object, description, maximum pressure, capacity of reservoir, discharge culverts through dam, provision for floods, cost; Ban Dam, France, object, compared with Furens dam, pressures, description, waste weir, provisions for discharge, cost, capacity of reservoir, supply; Remarks, advantages of curvature, necessity for watertight inner face—**Straight Masonry Dams:** instances of straight dams, and reasons for straight form; Thirlmere Rubble-Concrete Dam, object, description, depth of foundations, construction, waste weir, intake; Periyar Concrete Dam, India, site, object, storage, description, waste weir, channel for discharging water for irrigation, section of dam compared with other dams; New Croton Masonry Dam, extension of dam, great height, cause of height, section compared with others, maximum water-level of reservoir, construction, waste weir, waste-water channel, conduits and intakes, course of conduits, discharge pipes through dam—**Masonry Dams serving as Waste Weirs:** instances of Vyrnwy and La Grange dams compared; Vyrnwy Rubble-Concrete Dam, North Wales, description, section compared with other dams, maximum pressure on masonry, mode of construction, culverts through dam, mode of discharging supply; La Grange Masonry Dam, California, description, compared with New Croton waste weir, construction, cost, object; Remarks on Masonry Dams acting as Waste Weirs, similarity of forms, contrast of Vyrnwy and La Grange sections—**Masonry Dam with**

Flood Sluiceways: object of, across the Nile, pierced with sluiceways to avoid silting up and undue scour at toe ; Assuan Masonry Dam, length, number and sizes of sluiceways, period of filling reservoir, period and object of discharge, construction of dam, description, pressures on masonry, linings, levels, and gates of sluiceways, cost of dam ; Remarks on the Assuan Dam, peculiarity and object, section compared with Vyrnwy dam, possible raising of water-level of reservoir, capability of raising dam for increased storage.

EXAMPLES of a few of the notable masonry dams will now be described, which will serve to illustrate the general modern practice in regard to these important structures; whilst special reference will be made to any particular features which they may respectively exhibit. The dams selected all comply with the condition of stability of the lines of resultant pressures being kept within the middle third of their cross section; and they have all been so designed as to be subjected to a definite limited pressure on the square foot, which is varied to some extent in different instances, according to the strength of the masonry, the nature of the foundations, or differences of opinion as to the proper limit of the compressive stresses consistent with absolute safety.

Some of these dams have been built on a curve, especially where the valley is narrow, and others have been built in a straight line; whilst two of those chosen as examples serve also as waste weirs; and in one instance, namely, the Assuan dam, provision has been made for the discharge of the whole of the flood-waters of the Nile through numerous sluiceways in the dam. For the sake of clearness and convenience of reference, these dams will be divided for description under four heads—namely, Curved Masonry Dams; Straight Masonry Dams; Masonry Dams serving as Waste Weirs; and Masonry Dam with Flood Sluiceways.

CURVED MASONRY DAMS.

Building a masonry dam on a curve, though increasing its length in proportion to the curvature, adds unquestionably to its stability. It has, indeed, been sometimes urged that, owing to the cracks which are liable to be formed in a long dam by changes in temperature and variations in pressure, all masonry dams should be given a curved form, as then the pressure imposed by a full reservoir closes any cracks that may have been formed, especially on the downstream face, which is more exposed than the inner face to variations in temperature. Rocky gorges or narrow valleys are specially suitable for curved dams; but, occasionally, fairly long dams are built presenting a convex face towards the reservoir, with a view to increased security against failure, though necessarily with a somewhat large radius, as exemplified by the Gileppe dam in Belgium, which, with a length along the top of 771 feet, was curved to a radius of 1640 feet, as well as being made exceptionally thick, to secure it as fully as possible from the great disaster which its failure would entail in the industrial valley below it.

Furens Dam, France.—The dam erected across the narrow valley of the River Furens in 1861–66, for regulating the floods of this torrential river, and for supplying St. Étienne with water for domestic and manufacturing purposes, with a maximum height of $183\frac{3}{4}$ feet, and retaining a maximum head of water of 164 feet, is celebrated as being the first high reservoir-dam designed on correct theoretical principles, for supporting the pressure of water varying with the depth, Figs. 90, 91, and 92. The lines of resultant pressures, with the reservoir empty and full, have been kept well within the middle third of the cross section;[1] and owing to the rapidly curving batter given to the outer face, and the widening out of the base by steps, the maximum pressure on the inner face at the top of the first step, is only $6\frac{1}{10}$ tons on the square foot, and on the outer face, for about 26 feet above the front step, slightly under 6 tons, Fig. 92.

The width of the rocky valley at the level of the top of the dam, is only 328 feet; and the dam has, accordingly, been built on a curve to a radius of $828\frac{2}{5}$ feet, having a length along the top of 338 feet, Fig. 91, and a length at the base across the river-bed of about 33 feet, Fig. 90. The dam, built of random, rubble, granite masonry, rough-dressed on the face, laid in mortar composed of Theil lime and granitic sand, and founded upon solid mica-schist, possesses very ample stability, with its arched form, even if, as has been stated,[2] more exact methods of calculation indicate actual maxima pressures on the masonry of $8\frac{3}{5}$ tons on the square foot. In fact, it was considered by the designers of the Furens dam, that when the strongest possible masonry is adopted, a maximum pressure of $12\frac{4}{5}$ tons on the square foot might be allowed with safety, which would admit of a notable reduction in the cross section of a dam.

A thickness of $18\frac{2}{3}$ feet was given to the top of the dam at the height of the highest water-level in the reservoir, to provide against the shocks of floating ice and waves in the reservoir; and a protecting wall was built on its crest, $6\frac{1}{2}$ feet high, carrying a roadway along the top, $9\frac{1}{3}$ feet wide, with a parapet on each side, forming a means of communication across the valley. The cost of the dam alone, amounted to about

[1] "Construction du Réservoir du Furens," M. de Montgolfier, *Annales des Ponts et Chaussées*, 1875 (1), p. 155, plate 5, figs. 12, 13, and 14, and plate 6, fig. 1.

[2] "Les Réservoirs dans le Midi de la France," Marius Bouvier, pp. 3–9, Congrès International de Navigation intérieure, Paris, 1892.

£563,000. In order to avoid any possible weakening of the dam, the discharge culvert, consisting of two cast-iron pipes $1\frac{1}{3}$ feet in diameter, was carried in a tunnel through a spur at one side of the valley, quite away from the dam; and as the reservoir was also formed to receive the surplus waters of the high floods of the River Furens, threatening St. Étienne with inundation when exceeding 3285 cubic feet per second, a second tunnel was constructed through the same ridge at a higher level, for drawing off the water in the reservoir after every high flood, down to 18 feet below the maximum water-level, so as always to provide

FURENS DAM, FRANCE.

Figs. 90 and 91.—Elevation and Plan of Dam.

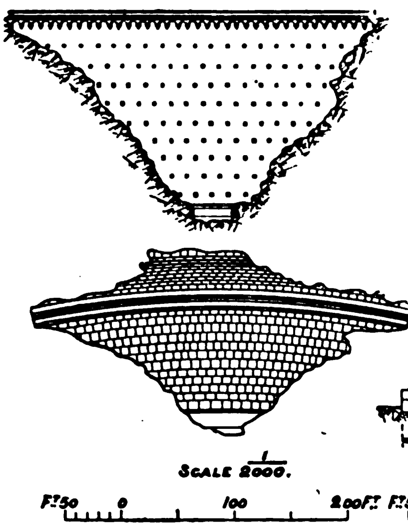
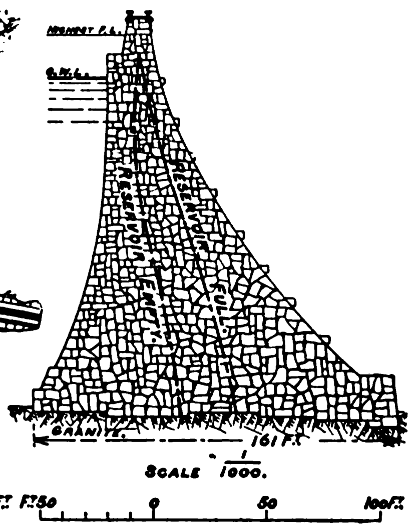


Fig. 92.—Cross Section of Dam.



a storage capacity of 14 million cubic feet for the flood-waters, thereby reducing sufficiently the rise of the floods in the river below the dam to preserve the town from inundation. A bye-channel also, skirting the left bank of the reservoir, provides a means, when desired, of conveying the discharge of the river above the reservoir, down a series of falls, direct into the river below the dam, to the safe extent of 3180 feet per second. A waste weir alongside this channel, $65\frac{2}{3}$ feet wide, discharges the surplus water from the reservoir into it.

The drainage-area of the Furens above the dam is about 6180 acres, which, with an average annual rainfall of $33\frac{1}{2}$ inches, gives a total volume of about 751 million cubic feet of water in the year, of which not more than 65 per cent. reaches the river; but the available volume would have just sufficed in an average year for the domestic and industrial needs of St. Étienne, amounting to a little under 16 cubic feet per second at the time the reservoir was constructed, if the capacity of the reservoir had enabled all the discharge of the river exceeding this supply to be stored. The narrowness, however, of the valley above the dam, together with the drawing down of the reservoir 18 feet below its full water-level for the reception of floods, gave an available storage capacity of only 265 million gallons, in spite of the great height of the dam. Accordingly, in 1872–78, another reservoir was formed by a second masonry dam $1\frac{1}{3}$ miles higher up the valley, having a total capacity of about 297 million gallons, some of which is used for the storage of floods, thereby increasing the available capacity of the older reservoir.[1] The second dam, similar in form to the first, and, like it, presenting a convex face upstream, is only $113\frac{1}{5}$ feet in height. The available capacity of the two reservoirs together for supplying St. Étienne, amounts to $90\frac{2}{3}$ million cubic feet, with a further capacity of $13\frac{2}{5}$ million cubic feet reserved for floods. The protection of St. Étienne from inundations has been quite satisfactorily accomplished by these reservoirs; but the storage, though supplemented by collecting the water from springs in the upper parts of the river-basin, was insufficient in dry years for the growing needs of the town as far back as 1892.

Remscheid Masonry Dam, Germany.—This dam was erected across the Eschbach Valley in 1889–91, for supplying water to the town of Remscheid in Rhenish Prussia, and to manufactories along the valley, by forming a reservoir capable of storing up 220 million gallons of water.[2] The dam has been built on a curve of 410 feet radius, with the special object of providing for changes in form due to the variations in pressure, and changes in length resulting from alterations in temperature, Fig. 93; and this curvature has proved perfectly successful in preventing the formation of cracks, the inevitable changes in the mass of masonry from the varying pressures and temperatures, having been confined to a forward and backward movement of about $1\frac{1}{16}$ inches.[3]

The dam is 524 feet in length along the top, and 59 feet at the foundations; it has a height at the bottom of the valley of 82 feet; and with a width of $13\frac{1}{8}$ feet at the top and $49\frac{1}{5}$ feet at the base, and with its lines of resultant pressures not passing outside the middle third of the cross section, it possesses adequate stability irrespectively of the horizontal curvature given to the dam, Fig. 94. The masonry consists

[1] "Ville de Saint-Étienne (Loire). Étude sur les Eaux d'Alimentation," Dr. Fleury and A. Clermont, Saint-Étienne, 1892, p. 4, and General Plan.

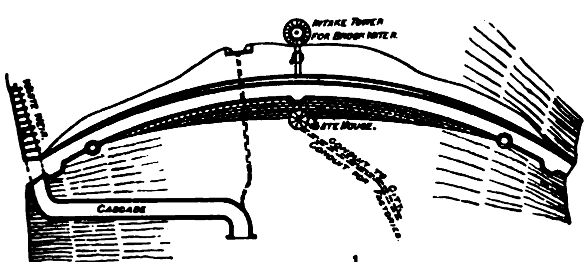
[2] "Die Erweiterung des Wasserwerkes der Stadt Remscheid," O. Intze, *Zeitschrift des Vereines Deutscher Ingenieure*, vol. xxxix. 1895, pp. 639 to 650, and plate 19.

[3] *Proc. Inst. C.E.*, vol. cxv. p. 156.

of slate rock having a specific gravity of 2·7, laid in mortar composed of 1 part of lime, 1½ parts of finely ground trass, and 1 part of Rhine sand, with curved joints in cross section sloping up towards the downstream face, so as to be approximately at right angles to the resultant pressures. The trass was selected in preference to cement, on account of its setting slowly and eventually forming stronger mortar. To secure watertightness near the inner face of the dam, a layer of asphalt on a plastering of cement was inserted between the masonry and the brickwork facing of the inner side of the dam, laid in cement mortar, alternately 1½ and 2½ bricks thick so as to form a toothing; and this asphalt coating extends from 20 inches along the base up to the top of the dam. This provision, together with the curved form, has rendered the dam perfectly watertight, which, even in the case of some curved French dams con-

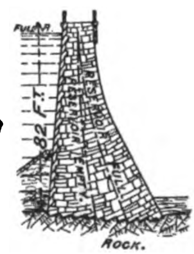
REMSCHEID DAM, GERMANY.

Fig. 93.—Plan of Dam.



Scale $\frac{1}{2000}$

Fig. 94.—Cross Section.



Scale $\frac{1}{1000}$

structed with great care, has not been attained, as for instance the Furens, Chartrain, and Ban dams.

The discharge pipe is carried in a culvert through the centre of the Remscheid dam, controlled by valves in a tower in the reservoir; and a second inlet to this culvert consists of a telescopic pipe between the valve-tower and the dam, whose floating inlet rises and falls with the water in the reservoir, so as to draw off the water from near the surface at the different levels of the water. A waste weir has been constructed alongside the left flank of the valley, extending upstream at right angles to the dam at one end, 65 feet long, providing for the discharge of 44 million gallons per hour of surplus water from the reservoir; and a sort of gridiron above the crest of the weir receives the floating ice, and enables the discharge to take place over the crest below, unimpeded by the ice. From thence the surplus water passes along the waste-water channel, left purposely with rough steps to check the current, and carried round below the dam to lead the water into the central channel,

Fig. 93. The dam with its accessory works, but exclusive of land, cost £19,150.

Hemet Masonry Dam, California.—An arched dam of granite rubble masonry, laid in Portland cement concrete, was built in 1890–95, at the lower end of the Hemet Valley, across the San Jacinto River in the San Jacinto Mountains in California, to a maximum height of $135\frac{1}{2}$ feet. The dam has not been hitherto carried out to its full proposed dimensions, as shown by dotted lines, having a length along its present top of 260 feet, and 40 feet at its base, formed to a radius of $225\frac{2}{5}$ feet, Fig. 95; whilst at its present height, $37\frac{1}{2}$ feet below the ultimate height

HEMET DAM, CALIFORNIA.

Fig. 95.—Plan of Dam.

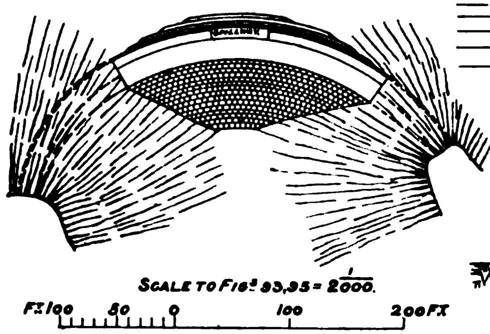
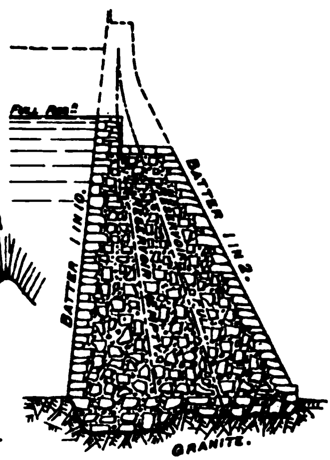


Fig. 96.—Cross Section.



Scale $\frac{1}{1000}$

proposed, it provides a reservoir for irrigation having a capacity of about 2850 million gallons, which will be increased $2\frac{1}{2}$ times when the dam is raised to its full height, as indicated by dotted lines in Fig. 96.[1] The dam, when completed, will possess the requisite stability, as shown by the lines of resultant pressures, quite independently of its arched form; and it has been built upon the solid granite rock, with a bottom width of 100 feet, and a batter of 1 in 2 on the upstream face, and 1 in 10 on the downstream face. Only the face stones were laid in mortar; whilst the rubble hearting was embedded in concrete, composed of 1 part of cement, 3 parts of sand, and 6 of small broken stone.

The water is drawn from the reservoir through three steel pipes,

[1] "Reservoirs for Irrigation, Water-Power, and Domestic Water-Supply," J. D. Schuyler, pp. 152 to 163.

carried through the centre of the dam at different levels, and controlled by valves on the inside, worked from the top, the lowest pipe having a diameter of 13 inches, and the two others 22 inches. There is also a waste weir provided in the centre of the top wall of the present dam, 50 feet in length, but with its crest only 1 foot below the top of the wall, so that in high floods the dam is liable to be overtopped some feet by the water in the reservoir, which actually occurred in January, 1893, during its construction, when it had been raised to a height of 107 feet above the valley; whereas for a few years after its completion, the flow off the drainage-area of 69½ square miles of the San Jacinto River above the dam, did not suffice to fill the existing reservoir, indicating a considerable uncertainty and fluctuation in the rainfall of this region.

San Mateo Concrete Dam, California.—The dam erected across San Mateo Creek in 1887–88, to form a reservoir for the water-

SAN MATEO DAM, CALIFORNIA.

Fig. 97.—Plan of Dam.

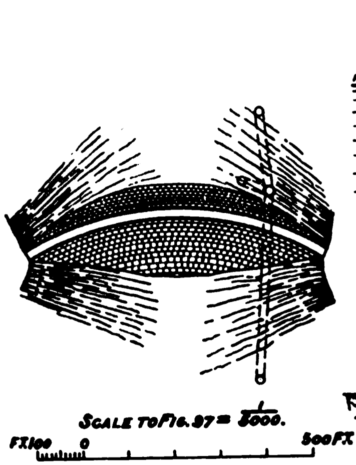
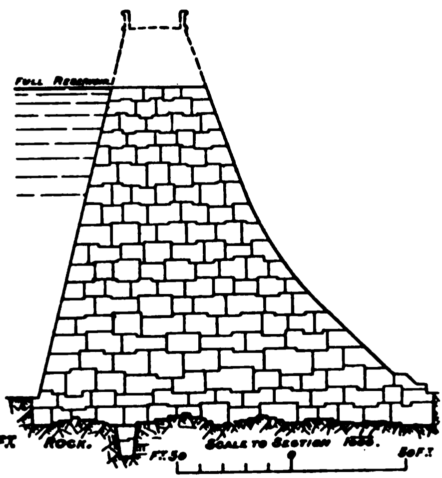


Fig. 98.—Cross Section.



supply of San Francisco, has been curved to a radius of 637 feet, and carried up to a height of 146 feet, or 24 feet below its contemplated height of 170 feet, at which level its completed length along the top will be 680 feet, Fig. 97. As stone suitable for rubble masonry was difficult to obtain near the site, the dam has been built of concrete, composed of 1 part of Portland cement, 2 parts of sand, 6½ of broken stone, and ⅔ part of water, moulded on the dam into blocks of 200 to 300 cubic yards, dovetailed into one another as shown on the cross section, Fig. 98, which were roughened on their faces with picks, and carefully cleaned and grouted with neat cement, before commencing the construction of the adjacent blocks.[1]

The dam has a width of 176 feet at the base, reduced by a batter of 1 in 4 on the inner face, and a batter on the outer face commencing at 1 to 1, and changing by a curve of 258 feet radius to a batter of 1 in $2\frac{1}{3}$ about halfway up, so that its width at the top, when raised to its full height, will be 25 feet, and at its present height is 41 feet. The capacity of the reservoir formed by the existing dam is 20,000 million gallons, which will be increased to 29,000 million gallons when the dam is raised the proposed additional 24 feet. No cracks are visible in the dam; and the dam is very nearly quite watertight, only some spots of moisture appearing on the lower face. The outlet culvert is carried in a tunnel through the northern slope of the valley, at some depth below the side portion of the dam, and draws its supply through three inlets down the slope, controlled by a valve-tower in the reservoir near the dam, continued by a shaft below the ground down to the tunnel, Fig. 97.

New Puentes Masonry Dam, Spain.—The dam which was completed in 1885 at the confluence of the small rivers Velez and Luchena, near the town of Lorca, in place of the old Puentes dam which failed in 1802, as described on page 145, has been made convex upstream to a radius of 656 feet, with a length along the top of 535 feet; and it has been given the modern type of cross section, with foundations carried down to the rock through 80 feet of alluvium below the bed of the river,[2] Figs. 99 and 100. The dam has a maximum height of 222 feet from the foundations below its crest to the top, and a width at the base, in the deepest part, of 236 feet up to within $21\frac{1}{3}$ feet below the bed of the river, where it begins to be reduced by steps on the outer side; whilst at the level of the river-bed, the width is reduced to 125 feet, where the regular form of the dam commences. From this point on the top of the foundations, $157\frac{1}{2}$ feet below the crest, the dam is diminished in thickness, to a small extent by a steep batter on the inner face, and rapidly by a batter of 1 in 1·4 on the outer face, to a width of 13 feet at the top, Fig. 99. The cross section of the dam between the water-level of the full reservoir and the old river-bed, very nearly coincides with the cross sections of the Villar and Periyar dams from the full water-level down to a similar depth, compare Fig. 99 with Figs. 105 and 107 (pp. 165 and 172), and is wider than the corresponding cross sections of the Chartrain, Ban, and New Croton dams, Figs. 104,

[1] "Reservoirs for Irrigation, Water-Power, and Domestic Water-Supply," J. D. Schuyler, pp. 189 to 203.

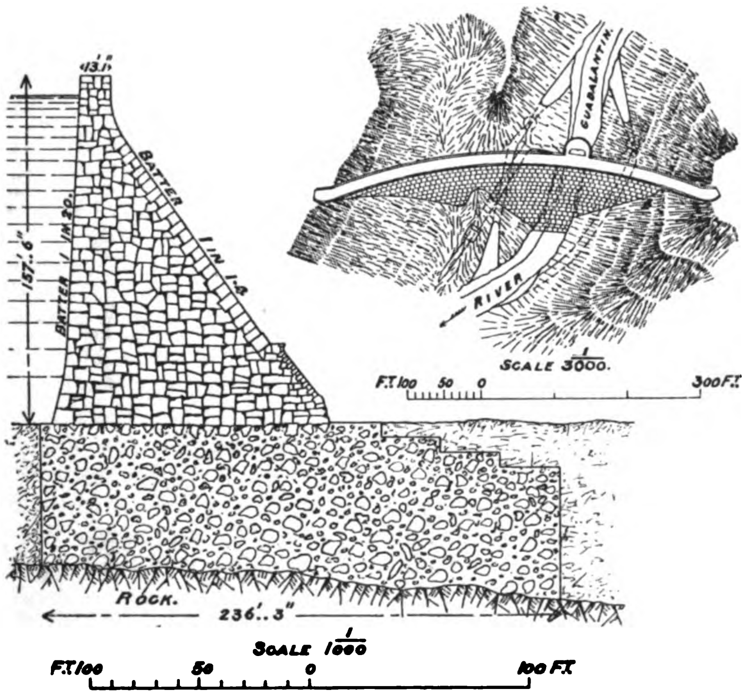
[2] A plan and cross section of the New Puentes Dam was very kindly obtained for me by Don Evaristo de Churruca, the Engineer in Chief of the Port of Bilbao.

106, and 109 (pp. 165 and 172); whilst a comparison of the new section with that of the old Puentes dam, Fig. 83 (p. 145), exhibits very clearly the contrast between the ancient and modern types of masonry dams.

The reservoir formed by the new dam has a capacity of 6943 million gallons; and the water stored in it is used for irrigating the arid plains below, and has also the advantage of reducing the flood discharge in

NEW PUENTES DAM, SPAIN.

Fig. 99.—Cross Section. Fig. 100.—Plan.



the river below the dam. Unfortunately, the reservoir is exposed to a considerable deposit of silt in spite of cleansing sluiceways near the bottom of the dam, so that the silt had attained a thickness of 61 feet near the dam is 1892.[1]

Chartrain Masonry Dam, France.—One of the most recent masonry dams in France, designed in accordance with the most approved type, was erected in 1888–92 across the Tâche rivulet, a

[1] "Réservoirs établis en Espagne," A. de Llaurado, pp. 5, 6, 8, and 9; "Congrès International de Navigation intérieure," Paris, 1892.

tributary of the River Renaison flowing into the River Loire at Roanne, to provide a reservoir for the supply of that town, and also for the mitigation of floods; and this Chartrain dam, 720 feet in length along the top, has been built convex upstream to a radius of 1312 feet.[1] The reservoir is supplied from a drainage-area of 3460 acres, and has a capacity of 990 million gallons, with a surface area, when full, of $58\frac{4}{5}$ acres; but as the water in the reservoir has to be drawn down $6\frac{3}{5}$ feet to provide for the reception of $17\frac{2}{3}$ million cubic feet of flood discharge, the actual storage available for the water-supply of Roanne is 880 million gallons, which has been obtained at a cost of £84,000.

The dam, which is solidly bedded on porphyry rock, has a maximum height of 177 feet, and at the level of the crest of the waste weir retains a head of water, above the bottom of the valley, of 151 feet, which it is estimated may be increased by a high flood of the rivulet to a maximum of $151\frac{3}{4}$ feet, with the water flowing nearly 10 inches over the waste weir; and the top of the dam is $2\frac{1}{2}$ feet above this level, Fig. 104. The dam is $155\frac{5}{6}$ feet wide at the bottom, and is reduced to $135\frac{1}{2}$ feet on the top of the footings of the foundations; and in going up, this width is mainly reduced by the diminishing batters of the downstream face, connected by curves, and commencing above the footings with a batter of slightly over 1 to 1, so that the width of the dam is $6\frac{2}{5}$ feet under the arches near the top, which partly carry the roadway along the crest of the dam on the downstream side, 13 feet wide between the parapets. The reduction in width on the upstream face is only $9\frac{5}{6}$ feet, effected by two straight batters in the first $65\frac{3}{5}$ feet above the footings, as this face is vertical for the remaining height above that level. The granitic rubble and ashlar masonry used in building the dam, was laid in Theil lime mortar, the sand being obtained by crushing the granite and porphyry obtained at the site. The maximum pressure imposed on the masonry at the toe of the inner face, above the foundations, with the reservoir empty, is 9 tons, and has been estimated at the toe of the outer face, with the reservoir full, at $7\frac{4}{5}$ tons, according to the system of calculation used for the Furens dam, and $9\frac{2}{5}$ tons by the corrected method more recently adopted, which latter pressure is well within the safe limits for the excellent masonry forming the dam. The inner face of the dam was pointed with cement mortar, and was rendered over with slow-setting 1 to 1 cement mortar, for a thickness of $1\frac{1}{5}$ inches, to within 33 feet from the top; but these precautions did not succeed in preventing infiltrations through the dam when the reservoir was first filled, though they gradually diminished.

The waste weir of the Chartrain reservoir forms the side wall of

[1] "Les Réservoirs dans le Midi de la France," Marius Bouvier, pp. 18 to 21; Congrès International de Navigation intérieure, Paris, 1892.

TYPICAL MASONRY DAMS. CROSS SECTIONS.

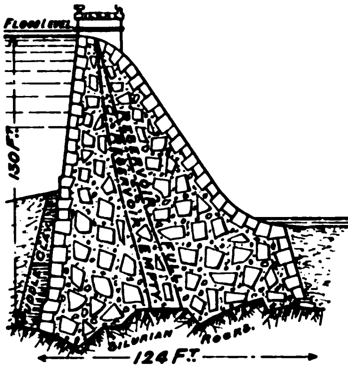


Fig. 101.—Vyrnwy Dam, Wales.

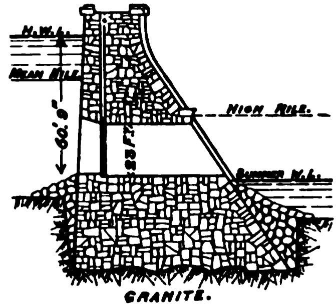


Fig. 102.—Assuan Dam, Egypt.

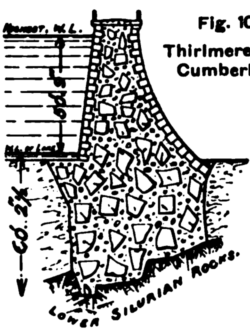


Fig. 103. Thirlmere Dam, Cumberland.

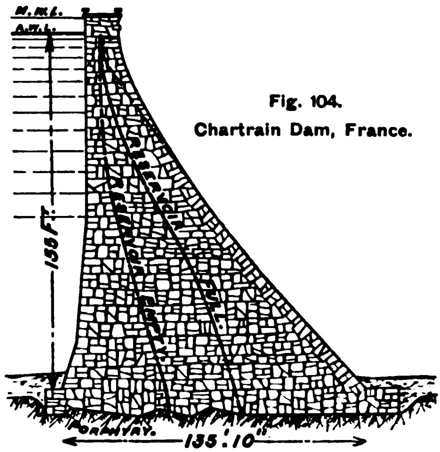


Fig. 104. Chartrain Dam, France.

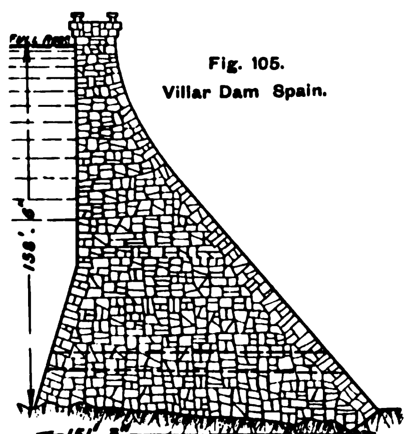


Fig. 105. Villar Dam Spain.

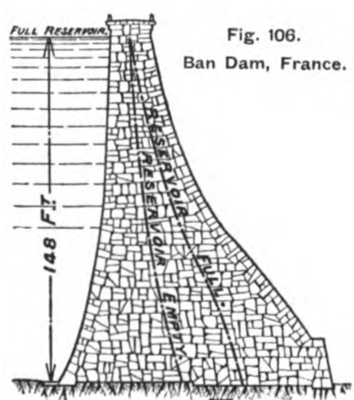
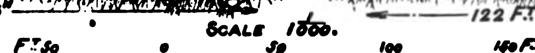


Fig. 106. Ban Dam, France.



a waste-water channel, $16\frac{1}{2}$ feet wide and 13 feet deep, and is provided with movable shutters having their sill $12\frac{1}{8}$ feet below their tops, which form the normal crest of the weir; and these shutters are removed for drawing down the water in the reservoir the $6\frac{3}{8}$ feet below the crest of the weir required for storing the flood discharge. The water-supply is drawn off from the reservoir through three cast-iron pipes built into the masonry of the dam, two of them being 18 inches in diameter and 137 feet below the crest of the waste weir, and the third 12 inches in diameter and 2 feet lower.

Villar Masonry Dam, Spain.—The dam erected in 1870–78 across the River Lozoya, at Villar, for increasing the water-supply of Madrid, has been built on a curve with a radius of 440 feet. The dam has a length along the top of 546 feet, and a maximum height of about 170 feet; but for 197 feet from the right bank, the dam has been kept down $8\frac{1}{4}$ feet below the roadway on the top for a waste weir, over which an iron bridge, with twelve spans of 16 feet, carries the roadway provided along the top of the dam, which has been given a width of $14\frac{3}{4}$ feet above the level of the sill of the waste weir.[1] The dam, which has been constructed of rubble masonry, has a width at its base, on a rocky foundation, of $151\frac{1}{4}$ feet, from which it is reduced in width by a batter on the inner face of about 1 in $3\frac{3}{8}$ up to a height of about 60 feet from the bottom, from whence the face is vertical to the top, and by a batter of 1 in $1\frac{1}{8}$ on the outer face, continued up to a height of about 105 feet from the foundations, from which level the face curves round to the vertical near the top of the dam,[2] Fig. 105 (p. 165). The cross section of this dam, differing somewhat in outline from that of Furens of very similar height, has a slightly larger area than the Furens cross section, but is a little narrower at the base; and the maximum pressure on the masonry has been estimated at $6\frac{1}{2}$ tons on the square foot.

The reservoir formed by the dam has a capacity of 4400 million gallons; and the water is drawn off from it, as required, by two outlet culverts carried through the dam, having each a sectional area of 19 square feet, and closed at their inlets by sluice-gates worked by hydraulic power from a central tower erected up the inner face of the dam to the level of the roadway. These culverts are situated at a depth of 143 feet below the crest of the waste weir; and additional outlets from the reservoir have been provided for the escape of floods, by four tunnels driven through the rock at the sides of the valley, two at the

[1] "Villar Reservoir on the River Lozoya," E. J. T. Manby, *Proc. Inst. C.E.*, vol. lxxi. p. 379.

[2] *Revista de Obras Publicas*, 1875-76, vol. xxiii. pp. 2 to 5, 25 to 27, and 49 to 53, and plates 20 to 23.

level of the outlet culverts, one at a depth of 93 feet below the water-level of the full reservoir, and the fourth 66 feet below. This storage has been obtained at a cost of £80,556.

Ban Masonry Dam, France.—The dam erected across the River Ban, a tributary of the River Gier in the Lower Rhone Valley, in 1866–70, for the domestic and industrial supply of St. Chamond, possesses the interest of having been designed on the successful completion of the Furens dam, on very similar lines, but somewhat more slender than its prototype, on account of a greater maximum pressure having been allowed on the masonry, in view of the experience of the stability of such structures gained from the construction of the first dam.[1] Thus the maximum pressure on the outer face, as originally calculated, was raised from slightly under 6 tons per square foot in the Furens dam, up to nearly 7 tons about 46 feet above the foundations in the Ban dam, or according to the corrected method of calculation, from $8\frac{3}{8}$ tons up to 10 tons per square foot. Also, as at Furens, the Ban dam, 541 feet in length along the top, has been built convex upstream, though to the considerably larger radius of $1325\frac{1}{2}$ feet in a wider valley; the outlet culvert has been carried in tunnel through the slope of the valley, away from the dam; and the waste weir has been constructed at one side of the reservoir, distinct from the dam.

The dam retains a maximum head of water of 148 feet; and its top, which provides a carriage-way, is raised $6\frac{1}{4}$ feet above the water-level of the full reservoir, Fig. 106 (p. 165). It has a width at the base of 127 feet, and at the top of 16 feet; and it was built of rubble schistous masonry founded upon the solid granitic stratum of the valley. There has, however, been a considerable leakage through the dam with the full reservoir, as at first occurred at the Chartrain dam, reaching in this case a flow of $2\frac{1}{2}$ gallons a second. The waste weir, 98 feet in length, has been placed on the right side of the valley just above the dam, with its sill $6\frac{1}{4}$ feet below the top of the dam; and it discharges the surplus water from the reservoir during floods, into a waste-water channel joining the channel of the River Ban below the dam. The supply is led out of the reservoir by two cast-iron pipes, $1\frac{1}{3}$ feet in diameter, laid in a tunnel, 197 feet long, driven through a spur on the left side of the valley, having the sill at its inlet placed $29\frac{1}{8}$ feet above the bottom of the reservoir; and the pipes on the reservoir side are encased in a very thick stopping of masonry, to shut off the rest of the tunnel from the reservoir, and are controlled by valves at their inlet. One of the pipes, in direct communication with the reservoir;

[1] *Annales des Ponts et Chaussées*, 1875 (1), plate 5, fig. 15; and "Les Réservoirs dans le Midi de la France," Marius Bouvier, pp. 15 to 17, Congrès International de Navigation, intérieure, Paris, 1892.

provides a discharge of $6\frac{3}{5}$ gallons of water per second into the river for the supply of the manufactories; and the other pipe, serving for the supply of St. Chamond having a consumption of about $2\frac{2}{3}$ million gallons a day, receives its supply from the reservoir through a conduit, $4\frac{3}{5}$ feet wide and $4\frac{1}{4}$ feet high, pierced at the top with holes which are covered over with a $6\frac{1}{2}$-feet layer of stones and gravel, with the object of filtering the water before entering the supply pipe. The cost of the works amounted to £38,000.

The reservoir thus formed, having a capacity of 407 million gallons, and a surface area, when full, of $44\frac{1}{2}$ acres, is fed from a drainage-area of 4450 acres, over which there is an average annual rainfall of $31\frac{1}{2}$ inches, only about 65 per cent. of which, amounting to about $330\frac{1}{2}$ million cubic feet, reach the river in the year. The reservoir is filled twice in the course of twelve months, namely, by the autumnal rains, and again on the melting of the snow; and the water thus collected, flowing off granitic strata, is remarkably pure.

Remarks on Curved Masonry Dams.—Though a convex curvature upstream tends to secure a masonry dam from cracks resulting from variations in temperature and pressure, and, moreover, adds to the stability of the dam, it does not prevent the infiltration of water under pressure through high dams. To render a high dam fairly watertight towards its base, in addition to the utmost care in construction to make the masonry perfectly solid and compact, and the avoidance of horizontal joints, it appears generally necessary to introduce a watertight coating for covering the inner face, for which asphalt seems to be the most efficient material. Some such coating is still more essential on the inner face of straight dams, on account of their greater liability to cracks, more especially in the case of long dams. This coating can only be dispensed with when very special care is devoted in construction to rendering the upstream portion of the dam impervious, as effected at the Vyrnwy dam by mixing the concrete with the minimum quantity of water, and ramming it tightly into the spaces between the blocks of rubble stone.

STRAIGHT MASONRY DAMS.

Sometimes masonry dams of moderate height, or of considerable length, are built in a straight line; and the curvature of a dam loses its advantage in cases where the ends of the dam cannot be made to abut against solid rock at the sides of the valley. Only one of the dams of which cross sections are given, and which remain to be described, has been built on a curve, namely, the La Grange dam in California, only 310 feet in length along the top across a rocky cañon, and over which the flood discharge of the Tuolumne River passes. All the others, comprising the Vyrnwy, Assuan, Thirlmere, Periyar, and New Croton dams, have been built perfectly straight. The length of the Vyrnwy dam along the top, amounting to 1172 feet, would not have precluded its being given a curved form; but at one end the rock does not rise rapidly enough up the slope of the valley for the dam to be able to abut against it. The Assuan dam, as constructed, with its great length of about $1\frac{1}{4}$ miles, could not have been given a sufficient curvature to be of any practical advantage; but, according to the original design, in following the lines of the soundest rock across the river, it was to have been built in the form of three curved dams, abutting at their ends against massive buttresses erected between them, on the rocky bed of the cataract, at shallow places.[1] The Thirlmere dam rises only 60 feet above the bed of the outlet of the lake, and is of ample section; but the existence of a large mass of rock in the line selected for the dam, near the centre of the outlet, and rising nearly up to the level of the top of the dam, would appear to have been particularly favourable for the construction of two curved dams convex towards the lake, provided equally suitable foundations could have been obtained along the curved lines, and the central rock proved solid enough for supporting the ends of the curved dams. The construction of the Periyar dam in a straight line was probably due, partly to the length of the dam, amounting to about 1300 feet at the top, and partly to the greater simplicity a straight form afforded in construction. The design of the New Croton dam, with its central masonry portion, joining at one end the waste weir running up into the reservoir, and at the other end a length of earthen dam, prevented the possibility of giving the masonry dam a curved form with any advantage, Fig. 111 (p. 173).

Thirlmere Rubble-Concrete Dam, Cumberland.—The dam which has been constructed across the outlet of Thirlmere, for raising the water-level of the lake a maximum height of 50 feet to store up water for the supply of Manchester, has a total length along the top of about 830 feet, divided by the central ridge of rock into two portions, which, including the shallow portions over the ridge, are respectively 310 feet long, with a maximum depth to the solid rock of 110 feet across the bed of St. John's Beck, and 520 feet long, with a maximum depth of 60 feet across the wider, but shallower, depression.[2] A section of the dam at the deepest part of the foundations is given in Fig. 103 (p. 165), illustrating very clearly to what a great extent the foundations

[1] "The Nile Reservoir Dam at Assuan and after," W. Willcocks, 1901, pp. 5-7, and plate 2.

[2] "The Thirlmere Works for the Water-Supply of Manchester," G. H. Hill, *Proc. Inst. C.E.*, vol. cxxvi. p. 3, and plates 1 and 2.

may add to the actual height of a dam when sound rock is only reached at a considerable depth below the surface. At Thirlmere, the height of the dam was practically doubled at the deepest point of the foundations; but fortunately only 40 feet in length of the dam had to be carried down to this depth. The rock was met with at a much higher level along the other portion of the dam, on the opposite side of the central ridge of rock, where the maximum height of the dam does not exceed 60 feet; but the dam has had to be built to about this height for a length of 200 feet, though its top is only from 43 to 22 feet above the original surface along this part. The dam was constructed by embedding large blocks of rock in concrete, composed of five parts of broken stone and crushed rock-sand to one of Portland cement, and covering this rubble concrete with masonry on both faces; and a roadway, 16 feet wide, is provided on the top of the dam. The existence of the ridge of rock in the line of the dam has enabled a tunnel to be formed through it for the discharge of the compensation and surplus waters, in a convenient central position without interfering in any way with the dam, which are conveyed in a $1\frac{1}{2}$-feet cast-iron pipe, and in two 3-feet cast-iron pipes respectively, passing through a brickwork stopping in the tunnel in the line of the dam; and this tunnel served for the discharge from the lake during the construction of the dam.

The waste weir was constructed in solid ground at the side of the lake, about a furlong away from the dam; and the water flowing over the ashlar sill of the weir, 100 feet long, discharges through a shaft, 20 feet in depth, into a short tunnel, from which it passes into the waste-water channel, which leads it eventually into the beck some distance below the dam. As the southern, upper end of Thirlmere is nearest to Manchester, the intake to the aqueduct for conveying the water has been placed at that end, the opposite end of the lake to the outlet at the northern end closed by the dam. Accordingly, all the provisions for the discharge of water from the lake have been placed quite away from the dam.

Periyar Concrete Dam, India.—The River Periyar draining part of the rainy district of Travancore at the south-western extremity of India, flows for some distance within a few miles of the dividing ridge of the Ghats separating its basin from that of the River Vaigai, traversing the very dry district of Madura, on the eastern side of the Ghats, which chain of mountains intercepts most of the rainfall coming from the Arabian Sea. A dam, accordingly, was erected across the valley of the Periyar, opposite a low gap in the ridge of the watershed, so as to divert a portion of the flow of the river along a channel formed up the valley of a tributary, and through the ridge in tunnel, into a tributary of the Vaigai, so as to provide water for the irrigation of the arid Madura district;[1] and the dam has been raised high enough to provide a storage of about 6815 million cubic feet above the sill of the channel, to ensure a permanent supply reaching a total of about 30,000 million cubic feet in a year. As the sill at the inlet of the discharge channel is 133 feet above the rock bottom at the inner toe of the dam, or 115 feet above the mean level of the bed of the River Periyar, the water below this level, amounting to 6484 million cubic feet, is permanently retained like a lake, and cannot be utilized for irrigation; and only the volume of water contained in the reservoir formed by the dam, between this level and the sill of the waste weir, 31 feet higher, is available.

The dam, constructed of concrete faced with masonry, has a height above the rock on the inner face of 173 feet, which is also the calculated maximum flood-level in the reservoir, with a parapet on each side of the 8-feet roadway along the top, raised 5 feet above the crest of the actual dam. The width of the dam at its base, founded upon rock, is 143 feet, which is gradually reduced by some steps and a slight batter on the inner face, and by steps near the base and a slope of 1 to 1 up to about half the height on the outer face, and then changing to a steeper slope, so that the width becomes 12 feet about 10 feet below the top, from which level the dam has been carried up vertically, Fig. 107 (p. 172). The water-level of the reservoir, when filled up to the level of the crest of the waste weir, is 162 feet above the rocky bottom constituting the original bed of the River Periyar; and the area of the reservoir at this level is 6405 acres, which, at the assumed maximum flood-level, 11 feet higher, would be increased to 7454 acres.

The waste weir, 420 feet in length, was formed by cutting into the rock down to 11 feet below the crest of the dam, in a depression at the side of the main valley, and separated from it by a ridge rising 109 feet above the sill of the weir. The discharge of the water for irrigation from the reservoir, is effected through an open cutting leading to the tunnel piercing the dividing ridge; and this channel, having a bottom width of 21 feet, starts from the reservoir with its bed 29 feet below the water-level of the full reservoir; and after running level for 3000 feet, it descends with a gradient of 1 in 320 for 2342 feet, to the entrance to the tunnel, where the sill is 37 feet below the water-level of the full reservoir, near which place the flow is controlled by sluice-gates. The tunnel connecting the two valleys, 5835 feet long, has been given a sectional area of 90 square feet, and a fall of 1 in 75.

The Periyar dam has the same height as the Furens dam, and is liable to have to sustain a few feet greater head of water; and it is

[1] "The Diversion of the Periyar," Col. J. Pennycuick, *Proc. Inst. C.E.*, vol. cxxviii. p 140.

TYPICAL MASONRY DAMS. CROSS SECTIONS.

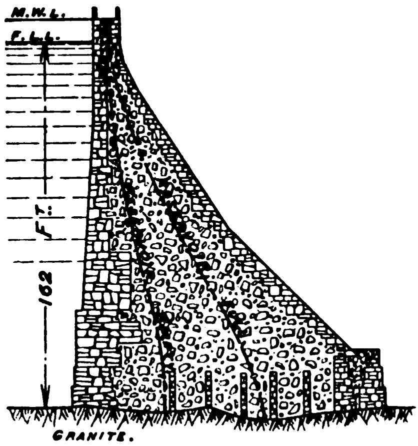


Fig. 107.—Periyar Dam, India.

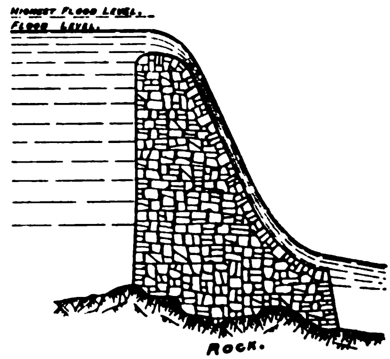


Fig. 108.—La Grange Dam, California.

NEW CROTON DAM, N.Y.

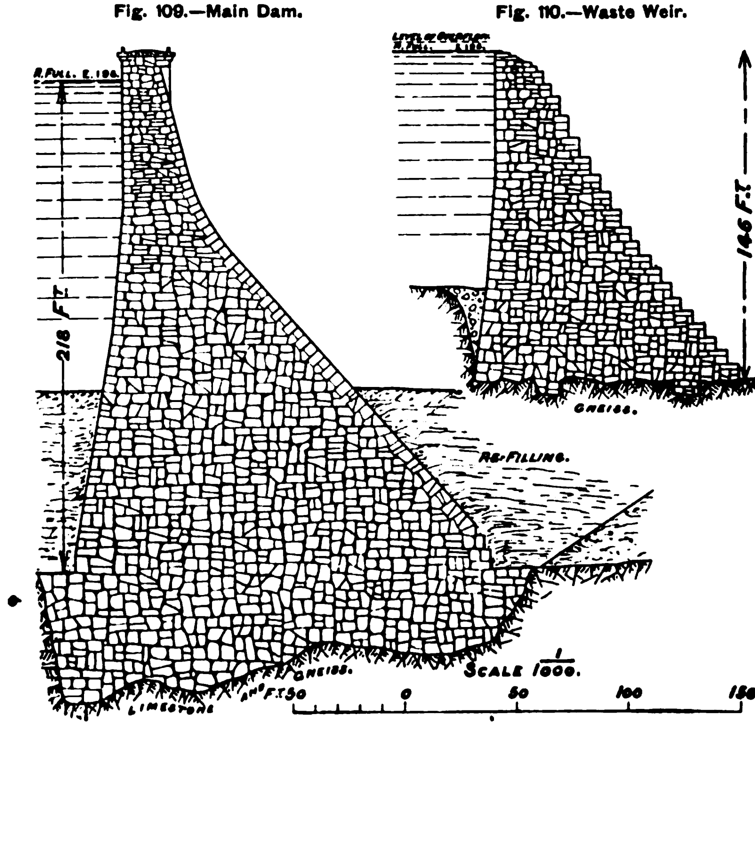


Fig. 109.—Main Dam.

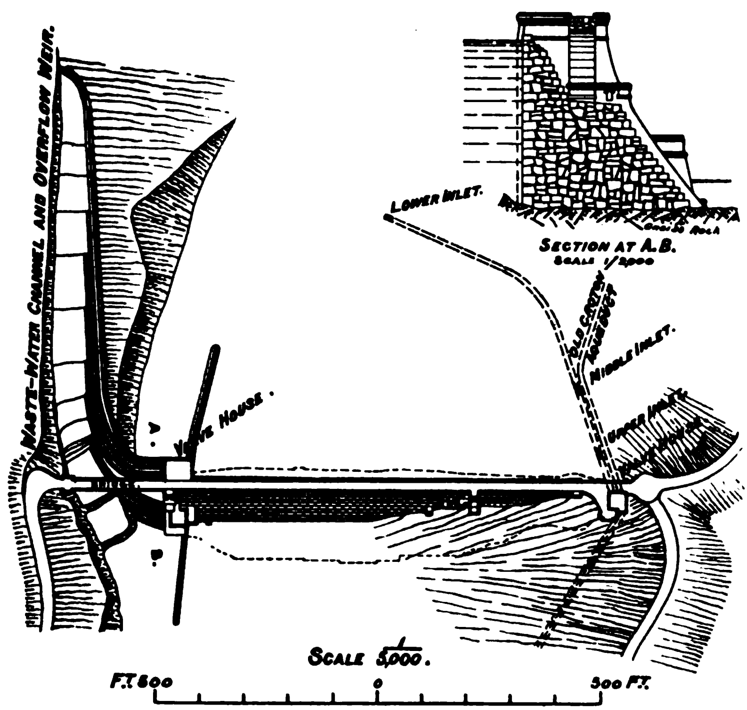
Fig. 110.—Waste Weir.

somewhat wider than the Furens dam near the middle of its height, but narrower towards the top and near the base. Its section is somewhat slighter than that of the Villar dam, and rather larger than that of the Ban dam in proportion to its height; and it approximates very closely to the Chartrain dam, but has a rather narrower base.

New Croton Masonry Dam, New York.—In designing a large storage reservoir in the Croton Valley for increasing the water-

NEW CROTON RESERVOIR-DAM AND OVERFLOW WEIR.

Fig. 111.



supply of New York, a high masonry dam constituted only the central portion of the dam required to impound the flow of the Croton River, across the deepest part of the valley, with an earthen dam across the slope of the valley on one side, and a masonry waste weir on the other side; though in construction, the earthen dam was first considerably restricted in length, and, consequently, in maximum height, by prolonging the masonry dam, which it was, indeed, eventually decided to substitute altogether for the earthen dam. The modified design is shown on the plan of this new dam for forming the greatly enlarged reservoir, Fig. 111.

The New Croton masonry dam is remarkable for its very exceptional height above the deepest part of its foundations, where its base reaches a depth of 275 feet, in the lowest part of the valley, below the water-level of the full reservoir; and as the crest of the dam is 14 feet above this level, which corresponds with the sill of the waste weir, the maximum height of the dam amounts to about 289 feet, Fig. 109 (p. 172). This great height, however, as shown by the cross section, is not due to any unusual depth of water retained by the dam, which reaches only a maximum of 140 feet at most; which depth, as indicated by the cross sections of masonry dams, pages 165 and 172, is exceeded at the Chartrain, Villar, Ban, and Periyar reservoirs. The height, indeed, of the New Croton masonry dam has been necessitated in the first instance, as foreseen, by the thickness of 80 feet of soft stratum overlying the rock at the site, and, secondly, by the unforeseen depth below the surface of the rock, to which the excavations for the foundations have had to be carried to reach sufficiently sound rock, attaining in places a maximum of about 57 feet.[1] Doubtless it was owing to the great depth of the rocky stratum below the surface, involving a great increase in the mass of masonry required, and in the excavations for a masonry dam, that led to the adoption, in the first instance, of an earthen dam across one slope of the valley, since by resorting to merely a core-wall down to the rock below the surface, a great reduction could be made, both in the masonry and in the excavations for the foundations; though the earthen dam, after being commenced, was eventually abandoned in 1902, as not affording adequate security against leakage at its junction with the masonry dam and elsewhere, as explained on page 110.

With the exception of the width given to the top of the dam, for providing a roadway, 18 feet wide, along its crest, the cross section of the New Croton dam follows very closely the sections of the Chartrain and Periyar dams, with a corresponding increasing width down to the surface of the rock to allow for the greater head of water to be retained. In the portion of the dam, however, carried down into the rocky stratum, it has been assumed that the rock abutting against each face of the dam, though not sound enough in the upper layers to bear the great weight of the high dam, together with the pressures imposed by the very considerable head of water, is firm enough to obviate any necessity for continuing the increase in width in the portion of the dam encased in the rock, Fig. 109 (p. 172). It has been estimated that during a high flood of the Croton River, the water-level of the reservoir might be

[1] *Transactions Am. Soc. C. E.*, 1900, vol. xliii. plate 35.

raised 4 feet above the water-level shown on the section, thus flowing in a stream 4 feet deep over the crest of the waste weir, at which maximum level it would be 10 feet below the top of the dam, not taking the parapets into account. The dam is built of rubble masonry, faced above the natural surface of the valley with ashlar masonry having a minimum thickness of $2\frac{1}{3}$ feet.

The New Croton waste weir, 1000 feet long, in continuation of the masonry dam along the northern slope of the valley, and built, like the main dam, of rubble masonry faced with ashlar, curves round from the line of the main dam to a direction at right angles to this line, running upstream with its lower side facing the adjacent northern slope of the valley, and decreasing gradually in height from a maximum of 150 feet at its junction with the main dam, to a minimum of 13 feet at its upper end, where it curves round to join the side of the valley, Fig. 111 (p. 173). The inner face of the waste weir, like that of the main masonry dam, is vertical at its upper part for about 53 feet from the top, and then is battered slightly to its base; whilst its outer face, which is stepped in order to check the force of the fall of the overflowing water, follows a somewhat curved batter for increasing the width more rapidly towards the bottom,[1] Fig. 110 (p. 172). It is founded upon the rock constituting the northern slope of the valley; and its cross section has been made slightly thicker than that of the main dam in proportion to its height, to allow for the increased pressure on a waste weir resulting from the impact of the overflowing water. The water falling down the steps of the waste weir, passes into a waste-water channel formed mainly in the rock, between the waste weir and the northern hillside, by excavating the toe of the slope to obtain the requisite width, and filling up hollows, owing to the dip of the rocky slope, with masonry; and steps have been formed across the channel at intervals, to check the velocity of the stream on its way to join the river channel below the dam, Fig. 111. A bridge, in line with the main dam, spans the gap formed by the curving stepped face of the waste weir and the waste-water channel, between the main dam and the northern hillside, and completes the roadway over the dam connecting the two sides of the valley.

The increased supply furnished by the greatly enlarged reservoir, has necessitated the construction of a second conduit for the conveyance of the water to New York. The original Croton conduit which formerly conveyed the water from the old reservoir, drawing it through an intake just above the old dam, at a minimum level of 157·7 feet above mean-tide level at the mouth of the Croton River, and passing down the Croton valley, has now been utilized for drawing off the water at the lower end of the new reservoir, through three inlets at

[1] "The Water-Supply of the City of New York," E. Wegmann, plate 131.

different levels near the New Croton dam, where the level of the invert of the old conduit, in its course of about 4 miles from its original intake, has been lowered to 152·7 feet above datum.[1] The discharge through this conduit is controlled by a valve-tower, or gate-house as it is termed in America, built on the lower side of the dam, over the slightly diverted line of conduit, carrying it further into the southern slope of the valley, where it crosses the line of the dam, Fig. 111 (p. 173).

The New Croton conduit, providing for the conveyance of the additional supply furnished by the new reservoir, has its intakes very near the site of the original intake, and can draw its supply from the reservoir both above and below the old dam;[2] and it is placed at a sufficiently low level to draw down the reservoir to the level of 140 feet above datum, 56 feet below the water-level of the full reservoir, and nearly 13 feet lower than it can be drawn down by the old conduit at the new dam, but 94 feet above the lowest part of the bottom of the reservoir at the new dam, Fig. 57 (p. 98). The new conduit takes a much more direct and inland course than the old conduit, till the latter, after keeping near the Hudson River from the mouth of the Croton River for a long distance on its way to New York, turns abruptly inland at Yonkers, and subsequently follows nearly the same course as adopted for the new conduit, Fig. 58 (p. 98).

A second valve-tower, or gate-house, on the inner side of the masonry dam close to the waste weir, where the rock crops up at the surface, and at a much higher level than at the bottom of the valley, controls the discharge from three 48-inch cast-iron pipes carried through the dam close to its base, which are designed for drawing off the water from the reservoir, when required,[3] Fig. 111.[4]

MASONRY DAMS SERVING AS WASTE WEIRS.

The two instances of masonry dams selected as illustrating structures which both serve as dams for forming a reservoir or raising the water-level, and at the same time provide waste weirs for the escape of the flood-waters over their crest, namely, the Vyrnwy and La Grange dams, though presenting some similarity in the general outline of their sections and their height, nevertheless differ in some important respects. Thus the Vyrnwy dam is perfectly straight; whereas the La Grange dam has been built on a curve convex upstream, which has enabled this dam to

[1] "The Water-Supply of the City of New York," E. Wegmann, pp. 51 and 206–207, and plates 54, 127, and 128.

[2] *Ibid.*, plates 54 and 65.

[3] *Ibid.*, plates 128 and 130.

[4] The plan of the New Croton dam is taken from a drawing given me by the engineers at the dam in 1904.

be given a considerably slighter cross section towards its base in proportion to its somewhat smaller height, than the Vyrnwy dam (compare Fig. 101, p. 165, with Fig. 108, p. 172). On the other hand, the Vyrnwy dam, forming a reservoir by impounding a small mountain stream, and with a crest, under the thirty-one arches carrying the roadway, 744 feet long altogether, is only liable to have a stream of water of 2 feet in depth passing over it and falling down its outer face; whereas the La Grange dam, stretching across the narrow gorge of the Tuolumne River, for raising the level of the river for irrigation, with a length of crest of only 310 feet, has already been exposed to a flood of the river passing in a stream 12 feet deep over its crest, and is liable to have a flow over it of more than double that volume.

Vyrnwy Rubble-Concrete Dam, North Wales.—The dam built across the valley of the River Vyrnwy in 1881–90, with a length of 1172 feet, and a maximum height of 144 feet from the deepest point of its foundations to its crest, has formed the reservoir known as Lake Vyrnwy, for the water-supply of Liverpool. The greatest depth of the reservoir, when full, down to the natural surface at the dam, is 84 feet; and the much greater depth of the foundations is due, like at the Thirlmere and New Croton dams, to the thickness of the alluvial stratum overlying the rock, together with a thickness of 6 to 7 feet of surface rock which had to be removed to reach the sound rock, making a maximum of 60 feet below the bottom of the reservoir.[1] As the dam serves also as a waste weir, it has been given a greater thickness at the termination of the curve of its crest than the average; and this has been increased by a flat batter on its downstream face terminating in a curve, to provide for receiving the shock and diverting the direction of the falling water, so that its section exhibits an exceptional thickness near the ground-level, which, however, is not much increased down to its base, Fig. 101 (p. 165). The greater width of the cross section in this case in proportion to the height, as compared with the Chartrain, Ban, Periyar, and even the Villar and Furens dams, may be attributed, partly to the outline of its downstream face having to be adapted to its function as a waste weir, and partly to the anxiety to secure absolute safety from failure in a valley leading to the River Severn, on whose banks some large towns are situated. The maximum pressure at the inner and outer faces of the dam, with the reservoir empty and full respectively, is approximately 7¾ tons on the square foot.

In building the dam, special care was taken to render the mass watertight, particularly near the inner face, by using the minimum amount of water adequate for mixing the Portland cement concrete in which

[1] "The Vyrnwy Works for the Water-Supply of Liverpool," G. F. Deacon *Proc. Inst. C.E.*, vol. cxxvi. pp. 27 to 29, and plate 4, fig. 5.

the large rubble blocks were embedded, and effecting the consolidation of the materials by the use of flat beaters and rammers. With the object of providing against the occurrence of any upward pressure on the base of the dam, due to springs in the rock foundation under the full head of water in the reservoir, drains were formed along the beds of rock under the dam, away from the face, from which the water is led by upward drains into a small drainage tunnel running along the centre of the dam, above the level of the backwater below the dam, so that the water collecting in the drainage tunnel can be discharged through a central cross passage on to the downstream side of the dam.

Two culverts, 15 feet in diameter, have been formed through the dam, on each side of the deeper portion of the dam, a little above the level of the backwater at the lower face, which served during construction for the discharge of the river; and pipes laid in these culverts, and carried through a stopping of brickwork to prevent leakage, enable water to be discharged direct from the reservoir into the channel of the river below the dam, the flow being controlled by valves in a tower constructed in the dam over each culvert.[1] Two pipes in the southern culvert, 18 inches and 30 inches in diameter, supply the daily and monthly compensation water respectively to the stream; whilst one pipe laid along the northern culvert, 39 inches in diameter, provides an additional means of discharging water from the reservoir. The water for the supply of Liverpool is drawn from the reservoir through a valve-tower erected near the northern bank a little distance away from the dam, on a site now submerged by the lake; and the conduit is carried from the tower to the north bank along the bed of the lake, and thence in tunnel through a ridge which separates the valley of the River Vyrnwy from another small river, the Tanat, which eventually joins the Vyrnwy near the confluence of the latter with the Severn.

La Grange Masonry Dam, California.—The La Grange dam, built in 1891–94 across the narrow cañon of the Tuolumne River at the base of mountains bordering the San Joaquin Valley, and 1½ miles above the town from which it derives its name, has been arched to a radius of 300 feet in its length of only 310 feet.[2] It has a maximum height of 125 feet on the upstream face, and 129 feet on the downstream face; and its width across its crest is 24 feet, and at its base 90 feet, having a nearly vertical upstream face, and a battered downstream face, terminating in a curve to divert the falling water into a horizontal direction, Fig. 108 (p. 172). The batter was designed to coincide with the slope of the falling water when pouring about 5 feet deep over

[1] *Proc. Inst. C.E.*, vol. cxxvi. plate 4, fig. 4.

[2] "Reservoirs for Irrigation, Water-Power, and Domestic Water-Supply," J. D. Schuyler, pp. 176 to 179, and figs. 85 to 88.

the crest of the dam; and as it is anticipated that the water in the river, which has already risen in flood-time 12 feet above the crest of the dam, may attain a maximum height of about 16 feet above the dam during an exceptional flood, and discharge a volume of 100,000 cubic feet of water per second over the dam, it was very important to provide a strong toe at the outer face for receiving the shock of the falling water. The cross section of the La Grange dam corresponds very closely with that of the waste weir of the New Croton dam for a similar height (compare Figs. 108 and 110, p. 172); but as the La Grange dam is liable to have a stream flowing over its crest of four times the depth anticipated for the maximum overflow at the New Croton dam, it is only in consideration of its arched form that the La Grange dam can be regarded as possessing an adequate cross section under such conditions.

The dam has been built of uncoursed rubble masonry laid in Portland cement concrete; and the 39,500 cubic yards of masonry contained in the dam were constructed at a cost of £114,585, the expense having been greatly increased by the contractor being paid in bonds instead of in cash.

The raising of the river a considerable height above its bed in the deep cañon, was needed to enable the Modesto and Turlock districts to be irrigated by its waters. The Modesto Canal draws its supply of 750 cubic feet per second, direct from the river alongside the dam on the right bank, which it skirts at a comparatively high level above the bottom of the cañon, for irrigating 81,500 acres; whilst the Turlock Canal obtains its supply of 1500 cubic feet per second from the reservoir above the dam through a tunnel, 560 feet long, 12 feet wide, and 11 feet high, for the irrigation of 176,210 acres.

Remarks on Masonry Dams acting as Waste Weirs.— Masonry dams which have both to retain a head of water, and perform the function of a waste weir, are given a broader crest and a more curved downstream face than ordinary masonry dams, in order to withstand the pressure of the overflowing stream at the top, and to prevent erosion at the outer toe of the dam by the falling water. Though, however, the general form of these dams, and of the masonry waste weirs forming portions only of other reservoir dams, are very similar, as indicated by a comparison of Fig. 71 (p. 121), Fig. 89 (p. 151), Fig. 101 (p. 165), and Figs. 108 and 110 (p. 172), a considerable difference in width is exhibited in the two instances selected as examples, which may be regarded as the two extremes of caution and boldness. Thus the Vyrnwy dam has been given a greater thickness throughout than the masonry dams of modern type; whereas the La Grange dam, which, if reckoned from the highest flood-level, has to support as great a head as the Vyrnwy dam, though similar in thickness of crest at the same depth below the maximum water-level, is much narrower than the Vyrnwy dam at its base, and, indeed, approximates to the slighter sections of masonry dams at the same depth below the water-level. The arched form, on the other hand, adds to the stability of the La Grange dam; but this is much more than counterbalanced by the exceptional flow of water, 16 feet in depth, which may pass over its crest during a great flood.

Masonry Dam with Flood Sluiceways.

Some of the masonry dams already described have been pierced by one or two culverts, or sluiceways, for the discharge of water from the reservoir above the dam; whilst the La Grange dam is exceptional in having to sustain the passage over its crest of the floods of a river having a maximum estimated discharge of 100,000 cubic feet a second. In erecting, however, a dam across the Nile to store up water for the summer irrigation of Egypt, it has been necessary to provide for a discharge reaching a maximum in years of very high flood of about 495,000 cubic feet per second.

Rivers bringing down large quantities of silt in flood-time are very liable to deposit the sediment in the still waters of a reservoir where the current is arrested by a dam, as previously noted in the case of the Colorado River, which rapidly silted up the reservoir formed by a dam across the river near Austin in Texas; and this result has rendered several reservoirs in Spain and Algeria comparatively useless, in spite of the provision of sluiceways through the dams, at a low level, for the periodical scouring out of the silt from the reservoirs. Accordingly, it would have been useless to attempt to form a reservoir in the Nile Valley by erecting a solid dam across the very muddy Nile, for such a course could only have resulted in a rapid reduction in the capacity of the reservoir by silting; and, moreover, the fall of so large a volume of water over the crest of a high dam during the floods of the Nile, might have endangered the stability of the outer toe of the dam. Consequently, in constructing a dam across the Nile at Assuan, it has been necessary to provide a sufficient number of sluiceways through the dam to discharge the whole flow of the Nile during the highest flood, without unduly raising the level of the river above the dam, and therefore with comparatively little checking of the current.

Assuan Masonry Dam.—A reservoir has been formed in the Nile Valley above Assuan, by a masonry dam stretching across the Nile at the first cataract, 6400 feet in length, pierced by 180 sluiceways, mostly in groups of ten, at four different levels, all 6 feet 6¾ inches wide, and 23 feet high, with the exception of the forty highest sluiceways which are only $11\frac{1}{2}$ feet high. These sluiceways, affording a combined waterway along the dam of 1181 feet in width, and providing a total area of opening of 24,145 square feet with all the sluice-gates raised, enable the Nile to flow through the dam at high flood without raising the water-level above the dam within about 30 feet of its crest, and with a head of about $15\frac{3}{4}$ feet as a maximum,[1] Fig. 102 (p. 165). Moreover, in order to prevent silting in the reservoir as far as practicable, the water will only be impounded above the dam, by the gradual closing of the sluice-gates, between November and March, after the floods have abated, and when the river carries down a much smaller, and continuously decreasing amount of sediment, in proportion to its discharge, than during high Nile. The water thus stored is discharged in April, May, and June, through the sluiceways, for supplementing the flow of the Nile during its low stage, when in years of small discharge it is quite inadequate for the irrigation of those lands prepared for summer crops. Experience alone will determine how far the precautions taken will suffice to check the accumulation of silt in the Assuan reservoir, which, under the conditions, appears almost inevitable, especially in the upper part and sides of this extensive reservoir.

The Assuan dam has been founded upon the soundest granite rock, which in one place was only reached about 45 feet below the surface of the rock; and it has been constructed of rubble granite faced with coursed rubble masonry, laid in mortar composed of 4 parts of granitic sand to 1 part of Portland cement, except for the bottom $2\frac{1}{5}$ feet, and the upstream face, which were laid in 2 to 1 mortar. It has a maximum height of 127 feet above the deepest foundations, and where pierced by the sluiceways, a bottom width of 88 feet and a top width of 23 feet, with a batter of 1 in 18 on the upper face, and of 1 in $1\frac{1}{2}$ on the lower face;[2] and the water-level of the full reservoir has been at present fixed at 10 feet below the level of the roadway along the top of the dam, giving a head of 117 feet above the lowest foundations, and of $60\frac{2}{3}$ feet above the bottom of the lowest sluiceways, Fig. 102 (p. 165) Assuming that at some future time the reservoir might be filled up level with the roadway, the maximum pressure on the masonry would be $5\frac{4}{5}$ tons per square foot on the upstream face with the reservoir empty, and 4 tons per square foot on the downstream face with the reservoir full.

[1] "The Nile Reservoir Dam at Assuan and after," W. Willcocks, p. 5, and plates 3, 4, and 5; and "Irrigation in the Nile Valley and its Future," W. Willcocks, Glasgow Engineering Congress, 1901, Section II., *Proceedings*, pp. 47-48, and plates.

[2] "The Nile Reservoir, Assuan," M. Fitzmaurice, *Proc. Inst. C.E.*, vol. clii. p. 78, and plates 1 and 2.

A thickness of $16\frac{2}{5}$ feet of masonry separates the sluices in each group; and there is a thickness of masonry of $32\frac{4}{5}$ feet interposed between each group, with a buttress projecting $3\frac{3}{4}$ feet at these places, as shown on the cross section, Fig. 102 (p. 165). In the solid portions of the dam on each side, amounting altogether to about 1800 feet in length, the bottom width of the dam, for a maximum height of about 96 feet, has been reduced to 69 feet, and the top width to $17\frac{3}{4}$ feet.

The sluiceways have been lined with granite ashlar, with the exception of thirty of the lowest sluiceways, which were lined with cast iron with the object of expediting the construction in the deepest part of the dam. All the forty shallower sluiceways, with their sills $19\frac{2}{3}$ feet and $32\frac{4}{5}$ feet below the water-level of the full reservoir, and fifty of the sluiceways having their sills 46 feet below the same level, are closed by ordinary steel sluice-gates sliding in grooves in the dam against cast-iron faces, as none of them will have to be raised against a greater head than $11\frac{1}{2}$ feet.[1] The remaining twenty-five sluiceways, with their sills 46 feet below the surface of the full reservoir, and the sixty-five deepest sluiceways having their sills $60\frac{2}{3}$ feet below the same level, are provided with steel sluice-gates sliding against free rollers, to enable them to be readily raised against a considerable head of water, thereby ensuring, together with the upper sluiceways, the perfect regulation of the variable flow through the dam at the different stages of the river, and the discharge of the water from the reservoir during the low stage for supplementing the deficiency in the flow of the river.

The cost of the dam, together with the canal and locks on the left bank, to provide for the navigation of the river past the dam and the variation in the water-level above the dam, amounted to £2,450,000. The progress of the works was much facilitated by two very low Niles during the period of construction, extending from 1898 to 1902; but, on the other hand, the foundations had to be carried down much deeper than was anticipated to reach sound rock.

Remarks on the Assuan Dam.—The exceptionally long masonry dam at Assuan furnishes a very interesting example of a reservoir-dam erected across one of the large rivers of the world, with the object of storing up a large volume of water for the summer irrigation of a country entirely dependent for its prosperity on the waters of the Nile, and through which the whole flood discharge of the turbid Nile is made to pass in order to endeavour to prevent the silting up of the reservoir; and it is a matter for great regret that its utility has been much impaired by the curtailment of the designed height of the dam by $26\frac{1}{4}$ feet, which was 13 feet lower than that originally proposed.

[1] "Sluices and Lock-Gates of the Nile Reservoir, Assuan," F. W. S. Stokes, *Proc. Inst. C.E.*, vol. clii. p. 108, and plate 3.

The additional strain imposed on the dam by having to pass the whole of the Nile floods through sluiceways, necessitating very numerous openings across the dam, has been provided for without any undue pressure on the masonry, by a dam whose cross section, even at the present water-level of the full reservoir, corresponds very closely to that of the solid Vyrnwy dam for a corresponding height, except near the bottom, where its width is somewhat less, as no provision of a curved downstream face was needed at Assuan. Moreover, it must be borne in mind that the dam has been built with the contingency in view of a possible raising of the water-level in the reservoir to the height of its crest, making an addition of 10 feet to the head, in which case, the section of the Assuan dam, for a corresponding head of water, would be slighter throughout than the Vyrnwy dam, irrespectively of the openings in the former dam. Further, it has been suggested that if at any time in the future it should be determined to increase the capacity of the Assuan reservoir, in consideration of the very minor importance to Egypt of the retention of the Temple of Philæ in its present position, as compared with a great increase in the assured supply of water for the perennial irrigation of Egypt, the dam could be raised, without in the least endangering its stability, sufficiently to impound the Nile waters to an increased height of $19\frac{2}{3}$ feet,[1] and thereby increase the storage to twice its present volume, at a cost of only about £250,000. In the event of this proposal being carried out, the section of the dam, with the increased head to be retained, though larger than the slightest modern masonry dams, would not, in consideration of its openings, be unnecessarily strong for the functions it has to perform. The scouring out of holes in the bed of the Nile below the Assuan dam by the rush of the flood-waters through the sluiceways in the dam, has not impaired its stability; but this erosion has necessitated the construction of a masonry apron in the river-bed in front of the dam to arrest the scour.

[1] "Irrigation in the Nile Valley and its Future," W. Willcocks, Glasgow Engineering Congress, 1901, Section II., *Proceedings*, pp. 47 to 48, and cross section in plate.

CHAPTER IX.

INTAKES, AND CONVEYANCE AND STORAGE OF SUPPLY.

Delivery of Supply—**Intakes**: arrangement; Intake at dam, conditions necessary for safety with earthen dams, position of valve-towers and valve-shafts with masonry dams, examples; Intakes from Reservoirs away from Dam, advisability with earthen dams, convenient course in case of Thirlmere and Vyrnwy lakes, description of Vyrnwy valve-tower, description of Thirlmere valve-shaft, New Croton reservoir intakes, arrangements of San Mateo reservoir intake—**Aqueducts**: Object, conditions affecting form; Different Types of Aqueducts, conduits contouring slopes with free flow, in siphons under pressure, relative advantages; Varieties of Aqueducts, two systems adopted, following hydraulic gradient with siphons only in deep valleys, examples, second system mainly in siphons, instances, straighter course and more siphons in modern practice; Variations in Hydraulic Gradient, objects, requirements; Aqueducts laid to Hydraulic Gradient when practicable, example of Thirlmere aqueduct described, siphons distinct with automatic valves, air valves; Aqueduct laid mainly below Hydraulic Gradient, example of Vyrnwy aqueduct described, variations in gradient, balancing reservoirs, stop-valves; Aqueduct partly following Hydraulic Gradient and partly below it, example of New Croton aqueduct described, tunnels, siphons, and pipes, lengths; Flow of Water in Aqueducts, formula of discharge, factor varying with condition of channel, formula for pipes and steep small channels, flow in siphons, coefficient for friction; Forms of Aqueducts, open channels, forms of covered conduits, metal pipes for siphons, lines of pipes proportioned to supply, wooden bored and stave pipes, advantages, protection of metal pipes, joints of cast-iron pipes, relative advantages of metal pipes; Sizes of Aqueducts, conditions affecting sizes, dimensions of Thirlmere, Elan, Loch Katrine, New Croton, and Vyrnwy aqueducts, hydraulic gradients and diameters of New Croton masonry siphons, hydraulic gradients and diameters of pipes of New Croton, Thirlmere, Loch Katrine, Elan, and Vyrnwy siphons, increased hydraulic gradients for pipes of siphons, examples—**Service Reservoirs**: Use and positions; Objects of Service Reservoirs, equalizing pressure, storage; Position of Service Reservoirs, in regard to town supplied, elevation for distribution; Construction of Service Reservoirs, methods of forming, excavated and embanked, instances, lined by retaining walls; Covered

Service Reservoirs, advantages, methods of covering, efficiency of brick arches; Water-Towers and Elevated Tanks, object, instances of masonry towers, elevated tanks in America, examples, sizes; Stand-Pipes, object, sizes; Elevated Masonry Service Reservoirs in Storeys, description of, at Paris, uses of basements.

AFTER water has been impounded and stored in a raised lake or artificial reservoir, it has to be drawn from the reservoir at a suitable point, and in the quantities required for the supply of the town served by the reservoir, and conveyed in a conduit, or aqueduct, to the neighbourhood of the town, where it is discharged into a service reservoir, placed at a sufficient elevation to command the highest parts of the town, and made of adequate capacity to continue the supply in the event of the stoppage of the flow from the reservoir owing to accidents or for repairs.

INTAKES.

For drawing water from a storage reservoir, it is important to provide inlets at different levels, so as to obtain the supply from the clearest and least discoloured layers in the reservoir, which are generally found near the surface, and also in order to work the valves for admitting the water to the aqueduct, under the least available pressure. Moreover, it is advisable to place one inlet at the lowest practicable level, so as to enable the water in the reservoir to be utilized to the fullest possible extent towards the close of a long drought.

Intake at Dam.—In considering the provisions necessary for the security of earthen dams in Chapter VI., it was pointed out that the only safe conditions under which water could be drawn off from a reservoir close to an earthen dam, and through a conduit under the embankment, were the placing of the valve-tower, at, or very near to, the inner toe of the embankment, and securing a perfectly solid foundation for the culvert under the dam. These conditions were shown to have been fulfilled in the examples cited of the old Loganlea reservoir embankment for the water-supply of Edinburgh, Fig. 72 (p. 124), and the Torcy-Neuf reservoir embankment in France, Fig. 73 (p. 125). In most cases, however, where earthen dams are resorted to, it is expedient to place the intake and discharge culvert quite away from the embankment.

With masonry dams, on a foundation of rock, the conditions are different; and it is quite permissible to place the intake near, or at the dam, leading to a culvert through the dam; though, in some instances, it has been regarded as preferable to place the intake and discharge culvert at one side, away from the dam, in order to avoid all chance of weakening it, as exemplified at the Furens and Ban reservoirs. Thus a valve-tower is placed in the deepest part of the Remschéid reservoir close to the dam, controlling and regulating the flow from two clear-water brooks, led straight in a conduit to the bottom of the tower, and thence into the discharge pipe laid in a culvert formed through the dam near its base; and a floating inlet is provided for drawing off the water from the reservoir near the surface, at the varying levels, through a telescopic pipe in communication with the discharge pipe, whose height is automatically adjusted to suit the water-level of the reservoir,[1] with an emergency inlet for use when the reservoir is very low, Fig. 112. More

VALVE-TOWER INTAKES.

Fig. 112.—Remscheid Intake and Valve-Tower.

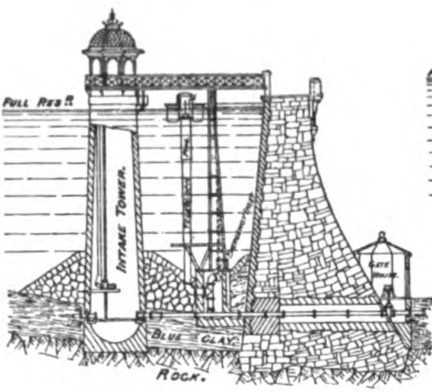
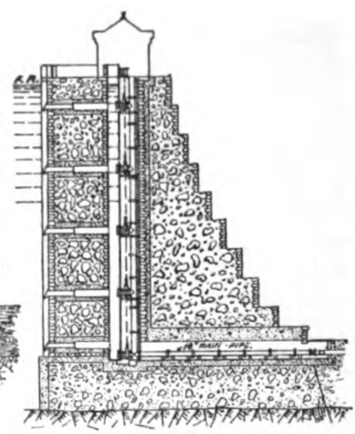


Fig. 113.—Tytam Intake and Valve-Well, Hong-Kong.



commonly, in the cases where a discharge outlet is carried through a masonry dam, the valve-tower or shaft for controlling the outflow, is constructed in a widened portion of the dam, as illustrated by the section of the Tytam concrete dam taken through the intakes placed at different levels, and the valve-well, by which water is drawn from the Tytam reservoir, and conveyed through the dam by an 18-inch pipe, laid in a culvert near the base of the dam, for supplying Hong-Kong with water,[2] Fig. 113. Valve-shafts also, formed in the New Croton and Vyrnwy masonry dams, Fig. 111 (p. 173), serve to control minor outlets through these dams, to which reference was made in the last chapter.

[1] "Die Erweiterung des Wasserwerkes der Stadt Remscheid," O. Intze, *Zeitschrift des Vereines Deutscher Ingenieure*, vol. xxxix. plate 19.

[2] "Tytam Waterworks, Hong-Kong," J. Orange, *Proc. Inst. C.E.*, vol. c. p. 255, and plate 8, fig. 4.

The valves closing the inlets of the three pipes built through the Hemet masonry dam in California, and constituting the intake from the reservoir are merely worked from the top of the dam.

Intakes from Reservoirs away from Dam.—Where a storage reservoir has been formed by an earthen dam, it is generally advisable, for the sake of the security of the dam, to provide an intake at one side of the reservoir, away from the dam, with a discharge culvert carried in tunnel along one side of the valley beyond the end of the dam. The same course, also, has been sometimes adopted as an additional precaution in the case of reservoirs formed by high masonry dams, as noted in the last chapter with regard to the Furens and Ban dams. Moreover, occasionally, the position of a reservoir formed by a masonry dam, in relation to the proper direction of the aqueduct for the supply, renders an intake at one side of the reservoir more convenient than at the dam, of which the Thirlmere intake for the supply of Manchester is a notable example.

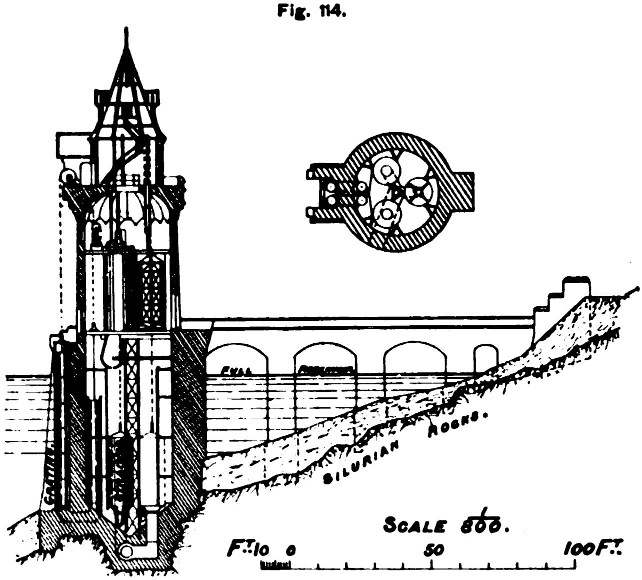
The Vyrnwy concrete valve-tower, about 170 feet in total height,[1] Fig. 114 (p. 188), furnishes an interesting example of the intake from a reservoir formed by a masonry dam, having been placed some little distance away from the dam at the eastern extremity of the lake, near the northern bank of the artificial lake, on account of this situation being more suitable for the northeasterly direction which the aqueduct had to take on its way towards Liverpool, than the southeasterly trend of the Vyrnwy Valley below the dam. On the contrary, pipes through the dam provide the most direct course for the discharge of the compensation water from the reservoir into the stream below, so as to maintain its stipulated flow. The Vyrnwy tower had to be erected on a site sufficiently far from the bank, with which it is connected by a bridge, to be submerged, on the filling of the reservoir, to a depth of about 60 feet below the water-level of the full reservoir, in order to enable the water to be drawn down to the extent of about 52 feet, Fig. 114. The two inlet valves of the Vyrnwy tower consist of two vertical columns of six steel tubes with gun-metal ends, each 9 feet long, resting one on the other, and kept in line by guides, one or more of which tubes can be raised for the admission of water in the interval left between their ends, according to the level of the water in the reservoir, from which the purest supply is drawn near the surface. The larger floating *débris* is arrested by a grating in front of the chamber containing the two inlet valves side by side; whilst, as usual in valve-towers or valve-shafts, the water is deprived of its smaller impurities in suspension by being made to pass through three cylindrical strainers of fine copper-wire gauze

[1] "The Vyrnwy Works for the Water-Supply of Liverpool," G. F. Deacon, *Proc. Inst. C.E.*, vol. cxxvi. p. 48, and plate 5, figs. 13 to 15.

stretched on wrought-iron frames, 9 feet in diameter and 25 feet high, placed in the Vyrnwy tower, before the water reaches the entrance to the aqueduct at the base of the tower. Arrangements, also, are always made for cleansing the strainers periodically from the fine deposit which accumulates on their outer surface.

VYRNWY INTAKE AND VALVE-TOWER.

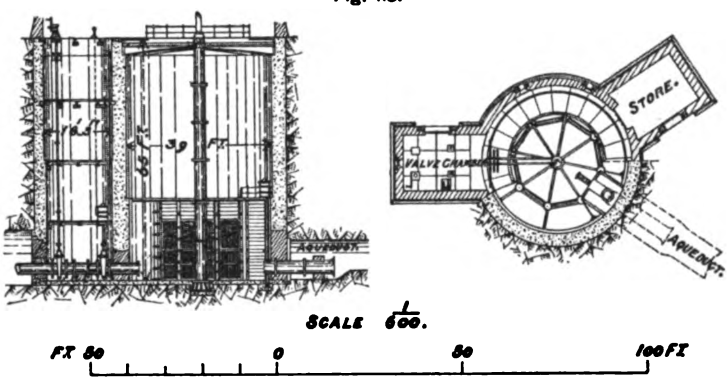
Fig. 114.



Sometimes it is more convenient to sink a shaft in the solid ground near the edge of the reservoir, for the regulation of the flow from the intake, especially where only the upper portion of the water in the reservoir can be drawn off for the supply, as in the case of the raised lake at Thirlmere. Thus, for instance, the intake from Thirlmere has been constructed near the southern end of the lake, as being nearer to Manchester than its natural outlet at its northern extremity, where its water-level has been raised by a dam; and for this purpose, a valve-shaft and straining well have been sunk about 65 feet into the rock on the line of the aqueduct conveying the water to Manchester, 100 yards inland from the edge of the lake, to which the water is conveyed from the lake through a tunnel constituting the intake, which has been given the same dimensions as the aqueduct starting from the valve-shaft. The sill of the tunnel is placed $6\frac{1}{4}$ feet below the original normal water-level of the lake, which is the lowest level to which the raised lake is allowed to be drawn down; and the tunnel terminates at the valve-shaft, which has been made rectangular, about 18⅓ feet by 15 feet, and lined with concrete,[1] Fig. 115. The water is led from the tunnel, through the valve-shaft, to the straining well in a pipe, 3½ feet in diameter, furnished with two valves worked from a chamber on the top of the valve-shaft; and the water on entering the straining well, 39 feet in diameter and lined with concrete, is made to pass through an octagonal straining screen, 22 feet in diameter and 16 feet high, lined with copper-wire gauze of 900 meshes to the square inch, before flowing out of the well through a short length of pipe, 3½ feet in diameter, into the conduit leading it to Manchester, Fig. 115. The discharge of the compensation and surplus waters through the tunnel, carried through the rocky

VALVE-SHAFT AND STRAINING WELL, THIRLMERE INTAKE.

Fig. 115.



ridge between the two portions of the dam across the outlet of the lake, into the stream below the dam, is also regulated by valves in a shaft, 10 feet in diameter, sunk in the rock down to the tunnel, a little downstream of the centre line of the dam.

The five intakes of the New Croton aqueduct near the old dam, placed at different levels and positions along the slope on the New York side of the enlarged reservoir, are controlled by timber gates or stop-planks, placed in a shaft, 90 feet by 100 feet, sunk in the rock near the reservoir to a depth of from 90 to 160 feet, containing a series of water-chambers below, and a building above ground.[2] Four of the inlets consist of culverts leading from the reservoir to the gatehouse, 14 feet in diameter; and the fifth inlet, with a diameter of 8½ feet, is in

[1] "The Thirlmere Works for the Water-Supply of Manchester," G. H. Hill, *Proc. Inst. C.E.*, vol. cxxvi. p. 7, and plate 1, fig. 5, and plate 2, figs. 6 and 7.

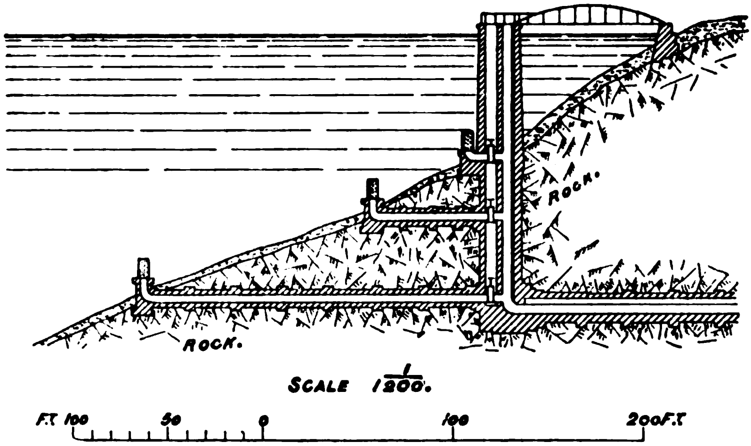
[2] "The Water-Supply of the City of New York," E. Wegmann, p. 127, and plates 65 to 71.

communication with the old aqueduct, so that it can draw water from the lower end of the reservoir, near the new dam, when the gate there is lowered. The inlets have been placed at different points and depths with the object of producing a circulation of water in the reservoir.

At the San Mateo reservoir for the water-supply of San Francisco, the discharge from the reservoir is effected through three inlets, at three different levels down the side of the valley, leading the water into a tunnel conveying the supply through the northern slope of the valley, at a good depth below the side portion of the concrete dam.[1] The flow through these inlets is controlled by a valve-tower, erected on a site near the dam now covered by the reservoir, and prolonged down-

VALVE-TOWER AND VALVE-SHAFT, SAN MATEO INTAKE, CALIFORNIA.

Fig. 116.



wards by a shaft sunk in the rock down to the tunnel carrying the supply in a $4\frac{1}{2}$ feet wrought-iron pipe, through the hillside, to the aqueduct for leading it to San Francisco, Fig. 116. Accordingly, in this instance, both a valve-tower and a valve-shaft have been employed in combination for regulating the discharge from the reservoir.

AQUEDUCTS.

The water drawn from the reservoir through the intake, under control of the valves, or gates, in the tower or shaft, after being passed through a strainer for removing suspended matters, enters the aqueduct, which conveys it to its destination for distribution throughout the

[1] "Reservoirs for Irrigation, Water-Power, and Domestic Water-Supply," J. D. Schuyler, pp. 200 to 203.

district for which it has been stored. The form of aqueduct adopted depends upon the general configuration and levels of the country, along the most suitable route between the reservoir and the town to be supplied, in relation to the hydraulic gradient, or the available fall between the lowest water-level of the storage reservoir and the level of the full service reservoir, in proportion to the length of the aqueduct between these points.

Different Types of Aqueducts.—Where the surface of the land to be traversed approximates to the level and inclination of the hydraulic gradient, it may be possible, by contouring the slopes to a moderate extent, to lay the aqueduct in shallow cuttings so as to coincide with the hydraulic gradient, either as an open or closed channel according to circumstances, interposing occasionally short bridges carrying the aqueduct across small dips; and, under these conditions, the water flows freely along the aqueduct under the action of gravity. In places where the route of the aqueduct is intersected by a ridge, causing the surface of the ground to rise considerably above the hydraulic gradient, the aqueduct has to be carried in tunnel under the ridge along the line of the hydraulic gradient, so as to enable the water to flow freely through the tunnel. Where, however, the ground dips considerably below the level of the hydraulic gradient, as in traversing a river valley or a long stretch of low-lying ground, instead of continuing the aqueduct along the line of the hydraulic gradient by raising it on a series of high arches, according to the practice of the Romans in olden times, Figs. 1 and 2 (pp. 7 and 8), the water is conveyed in pipes, generally made of cast iron, following the depression of the valley at only a little depth below the surface, through which the water flows under pressure, with a velocity depending on the difference in head between the upper and lower ends of the inverted siphon, as it is termed, where the aqueduct begins to dip below the hydraulic gradient, and where it rises up again to the hydraulic gradient.

The first type of aqueduct is the most economical, but generally necessitates a somewhat winding course, involving an increase in the length, and, consequently, a decrease in the available fall of the aqueduct; whereas tunnels and so-called siphons, though more costly to construct, can be given a more direct route, except, in the case of siphons, where interference with property, or the prospect of undue pressure, owing to the existence of a considerable depression in the shortest course, renders a deviation expedient.

Varieties of Aqueducts.—Where the aqueduct is short, it may be possible to adopt only one type, with the water flowing freely along the hydraulic gradient, or wholly under pressure, according to the configuration of the ground traversed. Generally, however, in the case of long aqueducts, the varying conditions necessitate the employment, at different places, both of aqueducts coinciding with the hydraulic gradient, and aqueducts under pressure. Nevertheless, two distinct systems have been resorted to in these long aqueducts, depending on the lie of the ground, along the route selected, in relation to the hydraulic gradient. In the one system, the hydraulic gradient is followed, wherever practicable, by contouring the slopes and tunnelling through ridges or high ground, and only crossing the valleys of rivers, or abrupt dips, by pipes laid along the depressions, as adopted for the Loch Katrine aqueduct to Glasgow, the Thirlmere aqueduct to Manchester, and part of the Elan aqueduct to Birmingham. In the other system, the aqueduct only follows the hydraulic gradient in tunnels under high ground; and elsewhere the water is carried under pressure in pipes, along low ground, for long distances below the hydraulic gradient, which is only reached at places suitable for the introduction of balancing reservoirs, arranged at intervals for the reduction of the water-pressure, which would be liable to become excessive at the lowest point in a very long, unbroken line of pipes. This was the system resorted to for the aqueduct from the Longdendale reservoirs to Manchester; and a notable recent example is the Vyrnwy aqueduct to Liverpool. The first system is adopted where the chosen course for the aqueduct mainly traverses high ground, and flat land approximating to the hydraulic gradient; and the second system is resorted to where the route passes across long stretches of low-lying land. The New Croton aqueduct conforms to the first system for nearly three-quarters of its length, in traversing high ground in tunnel for the most part along the first 24 miles from the New Croton reservoir; and it has only been constructed on the second system, underground and under pressure, along the last 9 miles, in order to avoid interference with very valuable property, as in continuing to follow the hydraulic gradient, it would have come close to, or above, the surface along this lower ground forming a suburb of New York.[1]

Owing to the improved methods devised for driving tunnels in the latter part of the nineteenth century, the aqueducts recently constructed in hilly country have been given a more direct course, with a greater length of tunnelling than the much older aqueducts in the same districts. Thus the new Loch Katrine aqueduct, for increasing the water-supply of Glasgow, is $1\frac{3}{4}$ miles shorter than the original aqueduct completed in 1860, and has nearly 7 miles greater length of tunnels;[2] and the old

[1] "The Water-Supply of the City of New York," E. Wegmann, pp. 116 and 117, and plate 54.

[2] "The Water-Supply of the City of Glasgow," Benjamin Taylor, *The Engineering Magazine*, Sept. 1899, vol. xvii. p. 936.

Croton aqueduct, constructed in 1837–42, was carried in a devious course along the lower part of the valleys of the Croton and Hudson rivers, with rather less than $1\frac{1}{3}$ miles of tunnels altogether; whereas the New Croton aqueduct, constructed in 1885–91, follows a much more direct route to New York, and passes in tunnel, following the hydraulic gradient, under high ground for over 23 miles. Moreover, several elevated conduits, consisting of iron pipes supported on piers or bridges, which were adopted for crossing ravines in the original Loch Katrine aqueduct,[1] have been dispensed with in the new aqueduct; whilst the crossing of the Harlem River, which in the old Croton aqueduct consisted of a masonry bridge, having fifteen arches and a length of 1450 feet, carrying two lines of pipes 3 feet in diameter, to which a third line was added in 1860 with a diameter of $90\frac{1}{2}$ inches, has been effected in the New Croton aqueduct by a tunnel under the river.

Variations in the Hydraulic Gradient for Aqueducts.—The average available hydraulic gradient, which is the difference in level at the two ends of the aqueduct divided by the length, and is generally expressed in feet or inches per mile, depends upon the relative positions of the storage reservoir and the locality to be served, and can only be affected to a moderate extent by adopting a somewhat circuitous, or a fairly direct route respectively, for the aqueduct. Though, however, the same hydraulic gradient is usually followed for considerable distances where the aqueduct is made to coincide with it, and, in the case of siphons, depends wholly on the difference in level at the two ends in relation to the length, however long a siphon may be, and cannot be modified at any intermediate point except by altering the size of the conduit, there is no absolute necessity to keep to the average hydraulic gradient throughout. Sometimes, for the sake of economy in construction at awkward places or special positions, it is desirable to reduce the size of the aqueduct for a certain distance, which necessitates a corresponding increase in the hydraulic gradient to maintain the same capacity of flow; or it may be expedient to vary the gradient at certain parts, in order that the aqueduct may follow more closely irregularities of the surface. These modifications can be readily effected, provided that the size of the aqueduct is adjusted to suit the gradient for the requisite flow, and that the hydraulic gradient is correspondingly reduced or increased at other convenient places for the requisite length, to compensate for divergencies from the average hydraulic gradient.

Aqueduct laid to the Hydraulic Gradient where practicable.—The aqueduct from Thirlmere to the Prestwich service reservoir for Manchester is $95\frac{7}{8}$ miles long, of which $36\frac{3}{4}$ miles of covered concrete channels and $14\frac{1}{8}$ miles of tunnels, or $50\frac{7}{8}$ miles altogether, have

[1] "Encyclopædia Britannica," 9th edition, vol. ii. pp. 225–226.

been constructed to the hydraulic gradient of 20 inches in a mile; and the remaining 45 miles consist of inverted siphons crossing the valleys of the River Lune, the River Ribble, and several other smaller rivers,[1] Fig. 117. A uniform hydraulic gradient of 20 inches in a mile is maintained for the siphons, as well as for the covered channels and tunnels, for $83\frac{1}{8}$ miles from Thirlmere; but for the last $12\frac{3}{4}$ miles, constituting the longest siphon on the aqueduct, the available hydraulic gradient is about 35 inches in a mile, which has enabled the cast-iron pipes forming this siphon to be made only 36 inches in diameter, in place of pipes 40 inches in diameter used for the other siphons. The most important siphon in respect of combined length and depth, is that crossing the valley of the Ribble, $9\frac{1}{2}$ miles long, with a dip of nearly 400 feet below the hydraulic gradient; but the siphon across the Lune Valley, though less than a mile long, has the maximum dip of 427 feet, imposing the pressure due to this head of water on the pipes. The siphons are, for the most part, carried on bridges over the rivers at the bottom of the valleys, the two longest, over the Lune and the Ribble, being cast-iron arched bridges with three spans of 70 feet; and the covered channels are carried across streams on masonry bridges. This arrangement reduces the amount of dip, and renders the pipes readily accessible for repairs.

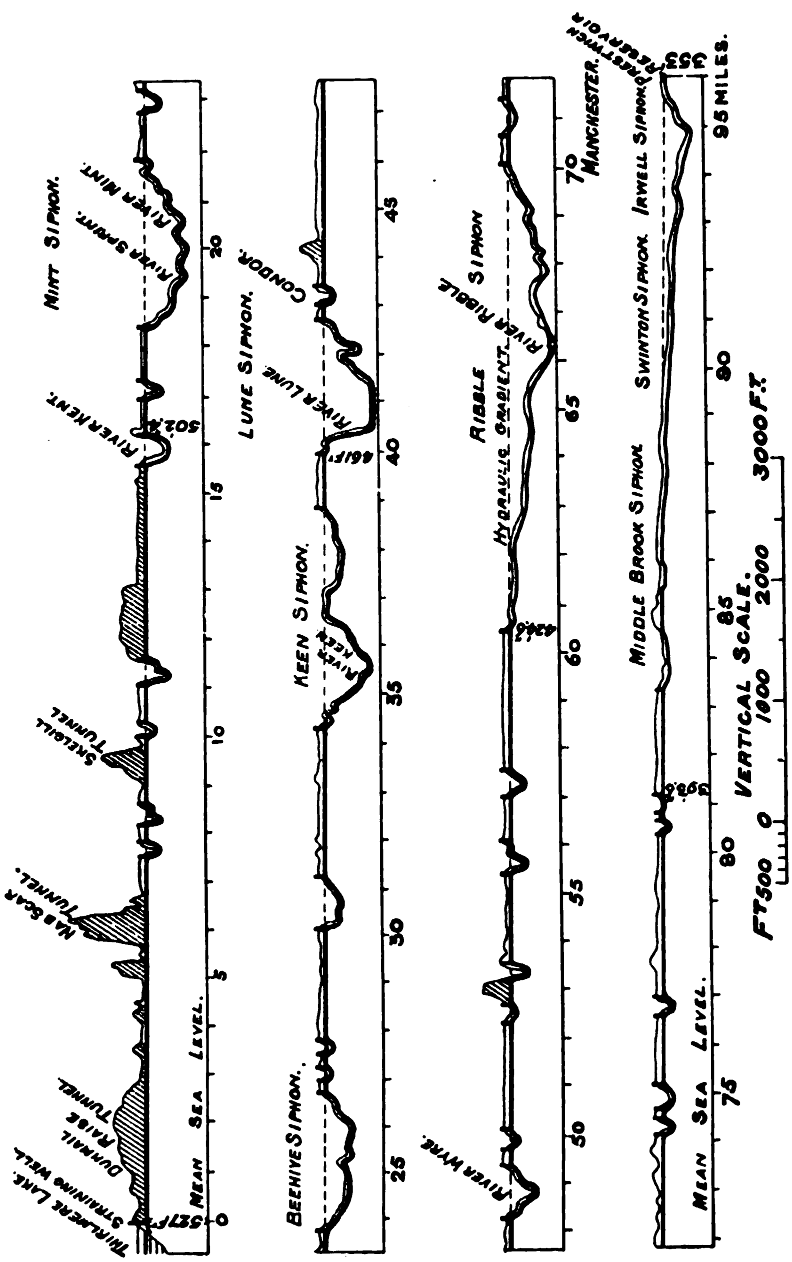
In this system, the siphons are quite distinct from the other portions of the aqueduct; and, therefore, the pipes forming them are only exposed to the pressure due to the dip of each particular siphon. Each siphon on the Thirlmere aqueduct terminates in a square well at each end, which connects it with the adjacent portions of the aqueduct; and self-acting valves are placed in the upper wells of the longer siphons, which, on the bursting of a pipe below them, close automatically and shut off the water from the aqueduct above, owing to the fall of a float in the well, resulting from the lowering of the water-level in the well by the efflux of water through the fracture; and waste of water, and flooding of adjacent lands are thereby prevented.

Air valves have to be provided at all points on a siphon where the line of pipes rises above the level of the adjacent portions on each side, to facilitate the filling of the siphons, and to allow of the escape of the air, which is liable to accumulate in these places, and would obstruct the flow if not afforded a vent; and the water in rising on drivingout the air, closes the opening by lifting a float.

Aqueduct laid mainly below the Hydraulic Gradient.—The general level of the land traversed by the Vyrnwy aqueduct, in its course of $68\frac{3}{8}$ miles between Lake Vyrnwy and the Prescot service reservoirs for Liverpool, is so low, that, with the exception of a tunnel,

[1] "The Thirlmere Works for the Water-Supply of Manchester," G. H. Hill, *Proc. Inst. C.E.*, vol. cxxvi. p. 7, and plate 1, fig. 4.

THIRLMERE AQUEDUCT.
Fig. 117.

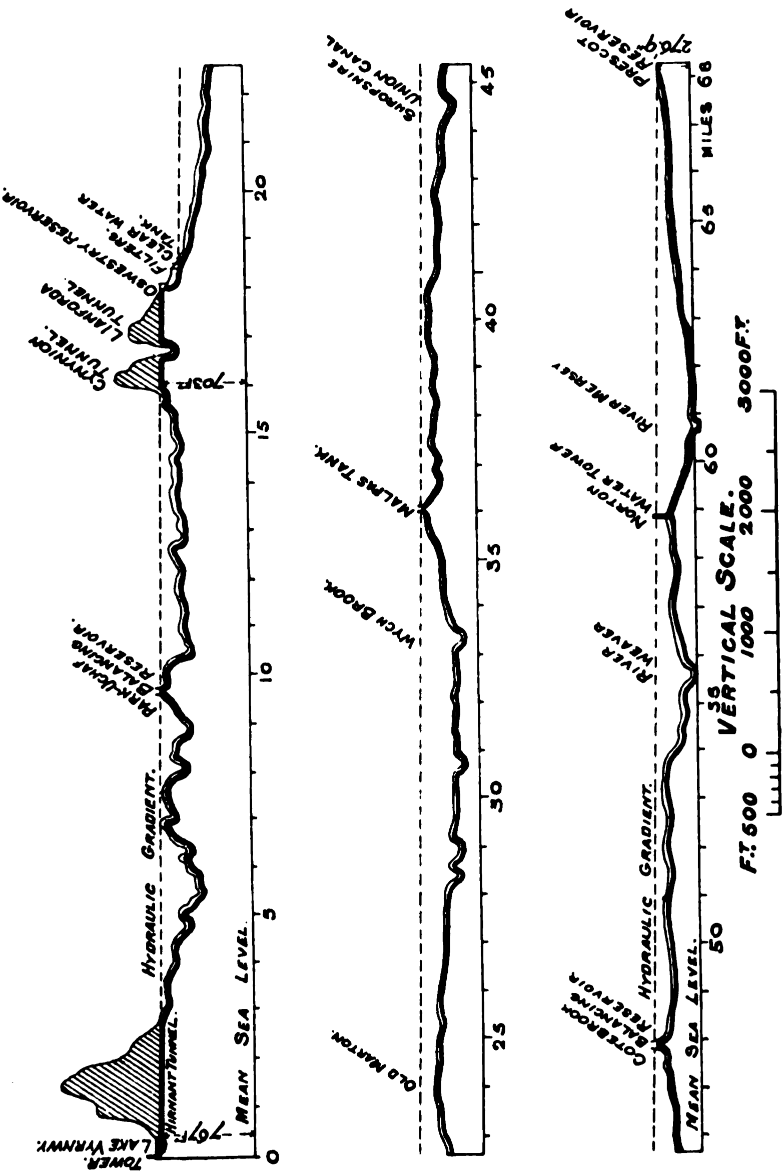


2¼ miles long, on leaving the lake and passing into another valley, and two tunnels, each under a mile in length, through two high ridges on approaching Oswestry, the aqueduct only rises to the hydraulic gradient at four places,[1] Fig. 118. The difference in level between the lowest available water-level of Lake Vyrnwy and the water-level of the full Prescot reservoirs, is 496 feet, which gives the large average hydraulic gradient of 7¼ feet per mile; but by adopting a gradient of 2 feet per mile in the tunnels, and a hydraulic gradient of 4⅓ feet per mile for the first two siphons, 7 miles and 6¼ miles long, thereby keeping the aqueduct between Lake Vyrnwy and Oswestry at a higher level, the length of tunnelling necessary has been reduced; and the fall thus saved has been utilized more advantageously in the aqueduct beyond Oswestry. From the reservoir formed at Oswestry just beyond the lower outlet of the Llanforda tunnel, the aqueduct follows the rapid slope of the land to some filter-beds, and falls 112 feet in three-quarters of a mile, which, though causing a considerable loss of available fall, diminishes proportionately the pressure on the siphon to Malpas, 17⅝ miles long, with a hydraulic gradient of 6·87 feet per mile, which, nevertheless, is exposed to the maximum head on the aqueduct of 480 feet, where it passes in steel tubes under the Wych Brook. The two next siphons, 11⅝ miles and 11 miles long, have been given hydraulic gradients of 4·6 feet and 4·8 feet per mile, respectively; whilst the last siphon, between Norton and Prescot, 9¼ miles long, has been given an average hydraulic gradient of about 6 feet per mile; but as the pipes under the Manchester Ship-Canal and the River Mersey, made of steel, have been reduced from the 3½-feet diameter cast-iron pipes of the rest of the siphon, to 3 feet and 2⅔ feet in diameter, respectively, a greater head is required at these crossings, reducing the actual hydraulic gradient elsewhere. Slight modifications of the hydraulic gradient have also been occasioned at places where valves have been introduced along the siphons, owing to these valves having been reduced to an area of about one-third that of the pipes, for the sake of economy and ease of working, necessitating a slightly increased head at these contractions to maintain the flow. In fact, in designing the Vyrnwy aqueduct, reductions in size have been readily introduced, with a view to economy, in positions where the work was costly or difficult in execution; and the loss of fall involved has been compensated for by enlargements in convenient, and readily accessible situations.

In order to reduce the pressure on the siphons, which, if they had been continuous throughout the whole distance, would have produced a pressure, on stopping the flow, of a head of water of 496 feet at Prescot,

[1] "The Vyrnwy Works for the Water-Supply of Liverpool," G. F. Deacon, *Proc. Inst. C.E.*, vol. cxxvi. p. 47, and plate 4, fig. 2, and plate 6.

VYRNWY AQUEDUCT.
Fig. 118.



and reaching a head of 848 feet in the portion under the Mersey, the siphons have been divided into six sections by balancing reservoirs at suitable points, and open wells at the outlets of the tunnels. Three balancing reservoirs have been constructed where the land along the line of the aqueduct just attains to the hydraulic gradient; and a fourth reservoir has been formed in a natural basin on the surface at Oswestry, which passes the water to filter-beds, and thence into a clear-water reservoir, from which the third long siphon starts, Fig. 118. As, however, the ground does not rise anywhere to the level of the hydraulic gradient between the Cotebrook balancing reservoir and the Prescot reservoirs, a distance of about $20\frac{3}{5}$ miles, a balancing reservoir has been constructed on the top of a large tower, where the hydraulic gradient is 110 feet above the surface, built on the summit of Norton Hill between the valleys of the Weaver and the Mersey, the highest suitable intervening site on the route. The reservoir consists of a cylindrical, steel-plate tank, 80 feet in diameter, supported by the masonry tower, with a basin-shaped bottom, containing a depth of water, at the overflow-level, of 20 feet round the side, increasing to 31 feet in the centre.[1]

Stop valves are introduced at places along all the siphons, to stop the flow and isolate any section between two of them, as desired; and some of these valves are closed by hand, whilst others close automatically on the occurrence of a fracture, owing to the increased pressure of the water on a disc, which always faces the flow, produced by the augmented velocity of the current resulting from the escaping water. The disc is kept in position against the ordinary current by a counterbalance weight; but a considerable increase in the velocity of flow pushes forward the disc, which by means of a lever-arm releases a trigger in moving, thereby setting free a weight which closes the valve.[2] To provide for the flushing out and emptying of the siphons, sluicing pipes, 12 to 18 inches in diameter, furnished with stop valves, lead from the bottom of all the downward bends of the pipes into the nearest watercourses.

Aqueduct partly following the Hydraulic Gradient, and partly below it.—The New Croton aqueduct, $33\frac{1}{10}$ miles long, is a peculiar instance of an aqueduct carried almost entirely in tunnel, where the first portion follows the hydraulic gradient under high ground, except for a short distance in crossing a swamp in a low depression, and the remainder is lowered below the hydraulic gradient under low ground, with the object of avoiding injury to valuable property.[3] The

[1] *Proc. Inst. C.E.*, vol. cxxvi. pp. 60 and 61, and plate 5, fig. 25.

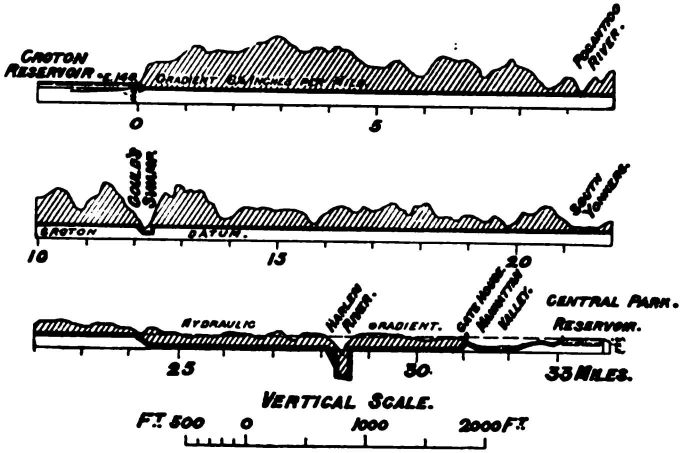
[2] "Progress of the Vyrnwy Waterworks: Annual Report of the Engineer," November, 1883, Liverpool, p. 24.

[3] "The Water-Supply of the City of New York," E. Wegmann, pp. 115 to 140, and plate 54.

brick and masonry aqueduct starting from the gate-house near the old Croton dam, proceeds in tunnel under high ground, formed to a gradient of 8½ inches in a mile, for a length of nearly 24 miles, in which the water flows freely, except where an inverted siphon, 1135 feet long, passes under Gould's Swamp, Fig. 119. On approaching lower ground after a course of nearly 24 miles from the reservoir, the aqueduct is lowered 115⅔ feet by an incline of 1 in 10, and is then carried on in tunnel as a siphon, with the water flowing under pressure, laid to the same gradient of 8½ inches per mile as before up to the Harlem River. On approaching the Harlem River, the aqueduct again descends by an

NEW CROTON AQUEDUCT.

Fig. 119.



incline of 1 in 6⅔ to the extent of 127½ feet, at which level it was originally intended to carry the siphon in tunnel through the rock under the river-bed; but as an experimental drift in the line of the tunnel from the south shaft, and borings, proved that the stratum was soft and exposed to a great pressure of water, the south shaft was carried 172 feet lower down, for the construction of the tunnel in sound, dry rock, at a depth of about 300 feet below high water in the river.[1] The tunnel has been built with a descending gradient of 1 in 100 southwards; and its invert, at its lowest southern end, is 447½ feet below the lowest level of the New Croton Lake, and 507½ feet below the highest overflow water-level. This brick and masonry tunnel, conveying the water under pressure, terminates at a gate-house in New York after a

[1] "The Water-Supply of the City of New York," E. Wegmann, pp. 159-161, and plate 80.

course of $6\frac{5}{8}$ miles; and from thence the siphon is continued with eight, and eventually, after distribution, with four lines of 4-feet cast-iron pipes, to the Central Park service reservoir, a distance of $2\frac{1}{3}$ miles.

The New Croton aqueduct consists altogether of about $29\frac{5}{8}$ miles of tunnels, $1\frac{1}{8}$ miles of open cuts in the first portion of the aqueduct, and $2\frac{1}{3}$ miles of lines of pipes in the siphon portion of the aqueduct.

Flow of Water in Aqueducts.—Provision has to be made for the discharge of a definite daily volume of water from the storage reservoir into the service reservoir; and the aqueduct connecting these two reservoirs has to be made of adequate size, form, and condition of internal surface throughout, in relation to the available fall, to convey this volume with regularity to its destination. The calculation of the discharge in a conduit of suitable form, and with a smooth channel laid to a definite gradient, having a stream of water of a certain depth flowing freely along it under the action of gravity, can be effected with considerably greater accuracy than in the case of watercourses with less regular channels, and flowing under less clearly defined conditions. The generally accepted formula for the mean velocity of flow in open or free, covered channels is $V = \dfrac{\dfrac{1{\cdot}811}{n} + 41{\cdot}6 + \dfrac{0{\cdot}00281}{S}}{1 + \left(41{\cdot}6 + \dfrac{0{\cdot}00281}{S}\right)\dfrac{n}{\sqrt{R}}}\sqrt{RS}$, where V is the mean velocity in feet per second, n is the coefficient of roughness, R is the hydraulic mean depth, or the hydraulic radius, in feet, and S is the slope or gradient, being the fall divided by the distance.[1] The value of n depends upon the condition of the wetted surface of the channel, and varies in artificial channels from a minimum of 0·010 for a glazed, plastered, or neat cement surface, or smooth timber, up to a maximum of 0·020 for a rubble-masonry channel in a bad condition, the most common values assigned to n being 0·013 for cast-iron pipes, or well-built ashlar masonry or brickwork channels, 0·015 for masonry or brickwork channels in somewhat less good order, and 0·017 for well-built rubble-masonry channels. The hydraulic radius, R, is the numerical value in feet of the cross section of the stream in square feet, divided by the length in feet of the perimeter of the cross section of the channel wetted by the stream. Having calculated the mean velocity V, the discharge D, in cubic feet per second, is readily obtained by multiplying the area A, in square feet, of the cross section of the stream by V, or D = AV. Though the coefficient which the general expression $\sqrt{RS}$, appearing in almost all formulæ of discharge, has to be multiplied by in this case to obtain the velocity, is somewhat complicated in appearance, it may be practically taken as constant for somewhat similar

[1] "Rivers and Canals," L. F. Vernon-Harcourt, 2nd edition, 1896, pp. 44 to 48.

channels, involving only moderate variations in R and S, so that a single calculation of its value suffices in such instances; and, moreover, various sets of tables have been published, calculated from this formula, giving the values of V and D corresponding to the most commonly used forms of channels and culverts, with various gradients, and the different values of n adopted in practice.[1]

For the flow in pipes, and in small channels with steep slopes, the influence of a variation in the slope on the coefficient may be neglected; and, therefore, the factor $\frac{0 \cdot 00281}{S}$ may be omitted from the formula, which then assumes the simpler form, $V = \frac{\frac{1 \cdot 811}{n} + 41 \cdot 6}{1 + \frac{41 \cdot 6n}{\sqrt{R}}}\sqrt{RS}$. Where the pipes become so-called inverted siphons, and the water flows under pressure, the difference in level at the two ends divided by the length, or the hydraulic gradient, takes the place of the variable inclination of the conduit in following the dips of the valleys. In fact, the flow of the water in a siphon under a certain head, is practically the same as the free flow would be in a pipe of the same size following the hydraulic gradient with the same fall, except for the increase in the length of the pipe following the sinuosities of the valley, which is generally neglected as too slight to signify, and any bends sharp enough to involve a loss of head, which would require to be allowed for. The flow, accordingly, through siphons may be calculated as the free flow of the same pipes running just full with the slope of the hydraulic gradient, or directly from the formula, $v = 8 \cdot 025\sqrt{\frac{dh}{4fL}}$, where v is the velocity in feet per second, $h \div L$ is the hydraulic gradient, or the head divided by the length of the siphon, d the diameter of the pipe in feet, and f the coefficient for friction, depending upon the condition of the internal surface of the pipe, which is about twice as great for pipes incrusted with rust as when they are new, and diminishes with an increase in the diameter, varying for new pipes from 0·00517 for 30-inch pipes, to 0·00510 for 48-inch pipes.[2] The friction, moreover, in new cast-iron pipes is reduced by one-third on coating them with pitch; and it has been found by measurement that the formula $v = 69\sqrt{d\frac{h}{L}}$ gives correct results for

[1] "Canal and Culvert Tables," L. D'A. Jackson; and "New Tables for the Solution of Ganguillet and Kutter's Formula," Colonel E. C. S. Moore, R.E., 1901.

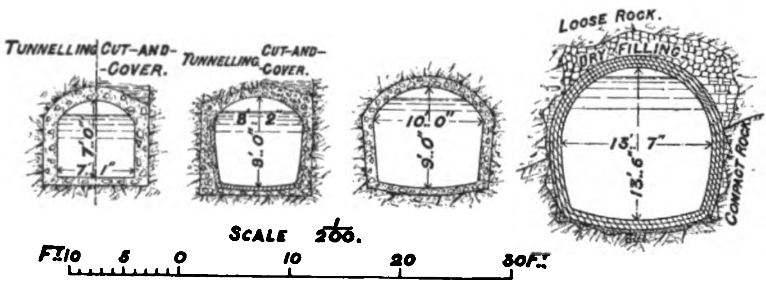
[2] "Hydromechanics," "Encyclopædia Britannica," 9th edition, vol. xii. pp. 484–485.

the flow of water in 48-inch cast-iron pipes, coated with pitch, for various hydraulic gradients.[1]

Forms of Aqueducts.—Though open channels have been sometimes employed in places where the aqueduct with a free flow runs along near the surface, it is usually advisable to cover over the aqueduct in such cases, both in order to avoid the chance of pollution, and also to secure the water from freezing in countries liable to severe winters. For this latter purpose, covered conduits and siphons are laid below the surface of the ground, so as to be provided with a covering of 2 to 4 feet or more in thickness, according to the severity of the winter climate, as indicated by the longitudinal sections of aqueducts, Figs. 117, 118, and 119 (pp. 195, 197, and 199). The form of the aqueduct is generally made the same both in tunnel, and cut-and-cover, as it is called when

AQUEDUCTS.

Fig. 120.—Thirlmere Aqueduct. Fig. 121.—Elan Aqueduct. Fig. 122.—Lock Katrine New Aqueduct. Fig. 123.—New Croton Aqueduct. Tunnel in loose and compact Rock, under high Ground.



constructed in open cutting and covered over, with only such modifications in the masonry, brickwork, or concrete lining, forming the arch and side walls, as the nature of the ground may necessitate, as illustrated by the cross sections of the Thirlmere[2] and Elan[3] aqueducts, Figs. 120 and 121. The bottom of the Thirlmere aqueduct is flat in tunnel and cut-and-cover; but usually an aqueduct of this kind is formed with a slight invert, as in the above instance of the Elan aqueduct, and also in the new Loch Katrine[4] and New Croton aqueducts,[5] Figs. 122, 123,

[1] "Flow of Water in the New Aqueduct from Loch Katrine," A. F. Bruce, *Proc. Inst. C.E.*, vol. cxxiii. pp. 413 and 414.

[2] *Proc. Inst. C.E.*, vol. cxxvi. p. 9.

[3] "The Construction of the Elan Aqueduct," H. Lapworth, *Proc. Inst. C.E.*, vol. cxl. pp. 242 and 247.

[4] *Proc. Inst. C.E.*, vol. cxxiii. p. 410.

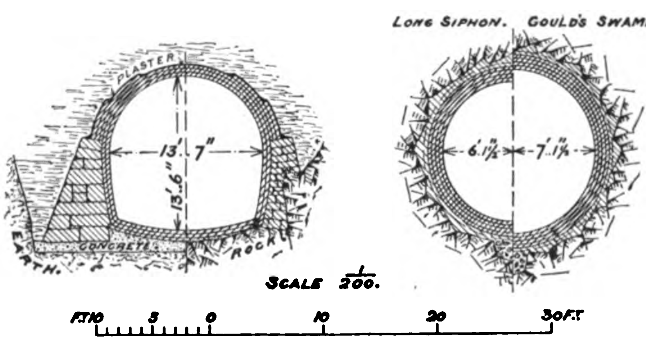
[5] "The Water-Supply of the City of New York," E. Wegmann, p. 125, and plates 60 and 61.

and 124. The portion of the New Croton aqueduct in tunnel under high ground with free flow, was given the section shown in Fig. 123, as affording more room for the passage of trucks than a circular section, and enabling the invert to be put in last, so as to avoid injuring it during construction; but the portion in tunnel under low ground, forming a long siphon with the water flowing under pressure, and the siphon under Gould's Swamp, have been given a circular section,[1] Fig. 125. The tunnels also in the Vyrnwy aqueduct, through which the water flows freely, have been made circular.

Siphons are generally constructed of lines of cast-iron, wrought-iron, or steel pipes, most commonly between 3 and 4 feet in diameter; and in crossing rivers and streams, they are often carried on bridges over

NEW CROTON AQUEDUCT.

Fig. 124.—Cut-and-Cover. Fig. 125.—Tunnel under Pressure.



the river in the absence of navigation, or laid in a tunnel formed under the river, so as to be accessible for repairs. These pipes possess the advantage over other forms of conduits, of being capable of adjustment to the supply required, one or two lines only being laid down at the outset, which are successively added to as the demands increase, instead of involving works and expenditure at the commencement for the conveyance of the whole of the eventual supply, probably not required for many years to come, as in the construction of tunnels and covered concrete or masonry conduits. Thus only one line of pipes, out of five, has been laid for the siphons of the Thirlmere aqueduct, one out of three for the siphons which, with the exception of 4 miles of tunnels, constitute the whole of the Vyrnwy aqueduct, and two lines out of six have been laid for the siphons of the Elan aqueduct; but, on the contrary, the whole of the eight lines of pipes forming the latter part of

[1] "The Water-Supply of the City of New York," E. Wegmann, p. 126, and plate 63.

the New Croton aqueduct, were laid together, probably to avoid the inconvenience of subsequent disturbance of the ground in a populous locality.

Wooden pipes, bored out of logs, were largely used before the introduction of cast-iron pipes about the commencement of the nineteenth century; and at the present time, in remote localities where carriage is very costly and timber abundant, as in some of the western parts of the United States, wooden pipes are again being extensively used for the conveyance of water, under low pressures, in the form of pipes bored out of logs, and pipes constructed with wooden staves. Bored pipes made from solid logs, 8 feet long and from 2 to 17 inches in diameter, are used for water-mains, being strengthened by spiral iron bands in proportion to the pressure to be borne, and coated over on the outside with pitch;[1] whilst pipes formed with several long staves, with radial edges and breaking joint, bound tightly together with iron or steel bands, placed closer together for higher pressures, are made from 1 foot up to about 9 feet in diameter, and being laid in a trench, are employed for aqueducts where the pressure does not exceed about 100 pounds on the square inch.[2] For pressures above this limit, the increasing weight of bands required renders these stave pipes in general more costly than steel. The commonly used sizes of staves range between 1 inch thick by 4 inches broad, and 2½ inches by 8 inches. Wooden pipes possess the advantages of a smooth inner surface, not subject to corrosion, and of being bad conductors, and, consequently, protecting the water from changes in temperature.

Metal pipes require to be most carefully protected from corrosion; for besides diminishing the strength of the metal, rust very materially reduces the discharge through the pipes by the roughening of the inner surface. Cast iron is generally coated with some preparation of coal-tar or pitch; whilst some form of asphalt has proved better for preserving wrought iron and steel, which, owing to their thinness in comparison with cast iron, need still greater protection from corrosion. The chemical ingredients contained in the water have a considerable influence on the durability of pipes, free carbonic acid, for instance, being specially injurious, as it combines with the iron to form ferrous carbonate, which is subsequently converted by the free oxygen in the water into ferric hydrate, thereby releasing the carbonic acid again for further attacks on the iron. The joints of cast-iron pipes are made, either with encircling collars, or with sockets, run with

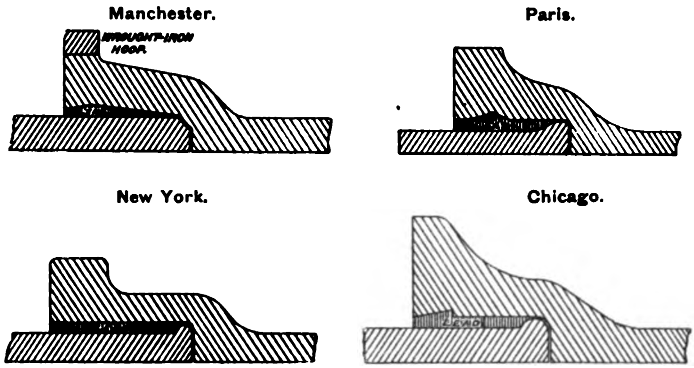
[1] "Public Water-Supplies," F. E. Turneaure and H. L. Russell, New York, 1901, p. 517.

[2] "Stave Pipe: Its Economic Design, and the Economy of its Use," A. L. Adams, *Trans. Am. Soc. C.E.*, 1899, vol. xli. p. 27.

lead,[1] some typical examples of which are given in Fig. 126. Wrought-iron and steel pipes possess the advantage, in localities where carriage is

SOCKET-PIPE JOINTS FOR AQUEDUCT.

Fig. 126.



very expensive, of being lighter, as well as stronger than cast-iron pipes; but owing to the projection of the rivets by which they are fastened together, they have to be given a slightly larger diameter than cast-iron pipes for the same flow. Steel pipes are specially suitable for positions where the pressures are high; and they have to a great extent superseded wrought-iron pipes in recent years.

Sizes of Aqueducts.—The dimensions given to an aqueduct are regulated by the available hydraulic gradient, and the required supply; and, as previously pointed out, the size may be varied at special places, with a view to economy or convenience of construction, by modifying correspondingly the hydraulic gradient. Moreover, as the flow is impeded by a rough surface, unlined tunnels through rock have to be given a larger cross section than lined tunnels to discharge the same volume; and as alterations in the form of a conduit, as for instance from tunnel or covered conduit to pipes, and changes also in direction, check the velocity of flow, these modifications, besides being made as gradual as possible, have to be allowed for by an enlargement of the section, or by the provision of an increased fall at these places, to ensure the maintenance of the required discharge.

The greater portion of the Thirlmere aqueduct, carried in covered conduit and tunnel having a concrete floor throughout, has a width of 7 feet 1 inch in the conduits and lined tunnels, and $8\frac{1}{2}$ feet in the unlined tunnels, and a height of 7 feet in the centre of the arch,

[1] *Proc. Inst. C.E.*, vol. cxxvi. p. 14; "Distributions d'Eau," G. Bechmann, p. 327; and "Public Water-Supplies," p. 504.

Fig. 120 (p. 202); and being laid to a gradient of 20 inches in a mile, the aqueduct makes provision for an ultimate discharge of 50 million gallons a day. The larger covered conduits contouring the hillsides, and constituting the main part of the Elan aqueduct, together with some short tunnels, have been given a maximum width, at the springing of the arch, of $8\frac{1}{6}$ feet, and a central height of 8 feet, Fig. 121 (p. 202), which, laid to a gradient of $15\frac{5}{6}$ inches in a mile, are designed to convey an ultimate volume of 75 million gallons a day. The new aqueduct from Loch Katrine, formed mainly in tunnel with a concrete invert, has a maximum width, at the springing of the arch, of 10 feet in the lined portions, and 12 feet in the unlined lengths, and a height in the centre of 9 feet, Fig. 122 (p. 202); and these tunnels, formed to a gradient of $11\frac{1}{2}$ inches in a mile, provide for an eventual discharge of about 70 million gallons a day. The original aqueduct, with 12 miles of tunnels and 10 miles of covered conduit arched over,[1] and, except in watertight rock, furnished with an invert and curved side walls, having a maximum width and height of 8 feet, and laid to a gradient of 10 inches in a mile, was designed to convey a supply of 50 million gallons daily; but its actual maximum discharge has not exceeded 42 million gallons a day, owing to the retardation of the flow by friction proving greater than had been anticipated.

The New Croton aqueduct, which along the first portion from the Croton lake, carried in tunnel under high ground, affords a free flow for the water at a gradient of $8\frac{1}{2}$ inches in a mile, has been given a horseshoe shape for facility of construction, with a maximum width of 13 feet 7 inches, and a height of $13\frac{1}{2}$ feet, Fig. 123 (p. 202), having a discharging capacity equivalent to that of a circular section 14 feet in diameter, amounting to 250 million gallons a day. The short lengths of cut-and-cover in the portion of the aqueduct affording a free flow, have been given the same dimensions as the adjoining tunnels under high ground, Fig. 124 (p. 203); but the siphon in tunnel, lined with brickwork under Gould's Swamp, has been made circular in section, with a diameter of $14\frac{1}{4}$ feet, Fig. 125 (p. 203), 3 inches larger than the circular section equivalent to the horseshoe section in discharging capacity, so as to compensate for the checking of the flow resulting from the dip of the siphon under the swamp, Fig. 119 (p. 199).

The tunnels on the Vyrnwy aqueduct, constituting the only portions along which the water flows freely, have been formed to a circular section with a diameter of 7 feet along the lined lengths; and where the tunnels have been left unlined, the rough surface of the rock has been blasted back 2 feet beyond this diameter, except the invert at the bottom, where the rock has been dressed to a fairly smooth surface, or

[1] "Encyclopædia Britannica," 9th edition, vol. ii. p. 226, figs. 6 to 11.

its irregularities overlaid with a lining of concrete. These tunnels, carried out with a gradient of 2 feet per mile, provide for an ultimate flow through them of 40 million gallons a day.

The total available fall of the New Croton aqueduct is 33·7 feet, of which 16·81 feet have been utilized for the gradient of 0·7 foot, or about 8½ inches per mile, of the first portion of the aqueduct with free flow, and a head of 1·61 feet lost in bends and modifications in section, leaving 15·28 feet of fall available for the portion under pressure about 9·22 miles in length, which, if distributed uniformly, would have provided a hydraulic gradient of nearly 1 foot 8 inches in a mile. In order, however, to reduce the number and dimensions of the pipes forming the siphon along the last 2·35 miles crossing the Manhattan Valley, 9·17 feet of the fall have been expended in the flow through these pipes, leaving 6·11 feet of fall available for the circular tunnel lined with brickwork constituting the first 6·86 miles of the siphon, out of which 0·46 foot is devoted to the portion under the Harlem River, 1219 feet long.[1] This portion of the aqueduct, however, has only to convey a daily supply of 208 million gallons, as the remainder of the volume of water drawn from Croton Lake is discharged into the Jerome Park Reservoir, for distribution to the suburbs, before reaching the siphon; and, accordingly, with the hydraulic gradient of 9·9 inches in a mile allotted to this portion, a circular section, 12¼ feet in diameter, suffices to discharge the supply, Fig. 125 (p. 203). The part of the tunnel dipping down and passing under the Harlem River at a considerable depth, so as to traverse sound rock, Fig. 119 (p. 199), has been reduced to a diameter of 10½ feet, in order that by increasing the velocity of flow by the augmented head required to pass the same discharge through the contracted section, the deposit of silt in this depressed part of the aqueduct may be prevented; and the head needed for this portion to ensure this maintenance of the requisite discharge through the reduced waterway, estimated at 0·46 feet, gives a hydraulic gradient of 2 feet in a mile.

The pipes forming the last 2⅓ miles of the New Croton aqueduct have been allotted a hydraulic gradient of 3·9 feet in a mile, to enable eight lines of pipes, 4 feet in diameter, to convey the required supply of 208 million gallons a day. Five lines of pipes, 40 inches in diameter, with a hydraulic gradient of 20 inches in a mile, are designed to constitute the greater portion of the siphons of the Thirlmere aqueduct, for conveying the ultimate supply of 50 million gallons a day; but in the last siphon in the aqueduct, where the available hydraulic gradient is increased to 35 inches in a mile, the diameter of the pipes has been reduced to 36 inches. Four lines of pipes, 4 feet in diameter, forming

[1] "The Water-Supply of the City of New York," E. Wegmann, pp. 184 to 187.

the siphons of the new Loch Katrine aqueduct, with a hydraulic gradient of 5 feet $3\frac{1}{2}$ inches in a mile, will convey the eventual additional supply of 70 million gallons a day. The siphons on the Elan aqueduct, which will eventually consist of six lines of pipes, 42·12 inches in diameter, with a hydraulic gradient of 3 feet in a mile, are designed to discharge a volume of water amounting to 75 million gallons a day. The Vyrnwy aqueduct, formed mainly of a series of siphons separated by tunnels and balancing reservoirs, will finally consist of three rows of pipes, $3\frac{1}{2}$ feet in diameter for the most part, with average hydraulic gradients of 4·5, 4·6, and 4·8 feet in a mile, according to the loss of head involved at certain places, and in the last siphon reaching about 6 feet in a mile, owing to the loss of head at various crossings, and especially the reduction of the steel pipes under the Manchester Ship-Canal and the River Mersey to 36 inches and 32 inches respectively. The pipes for one siphon on the Vyrnwy aqueduct have been reduced in diameter to $3\frac{1}{4}$ feet, owing to the available hydraulic gradient in this length of $17\frac{5}{8}$ miles reaching 6·87 feet.

As a general principle, it is advisable to reserve a greater proportion of the available fall for the pipes of siphons, with the object of reducing their number and keeping their diameter within the limit of 4 feet, which has proved the most convenient maximum, and to compensate for the reduced gradient remaining available for the masonry portions of an aqueduct, by increasing their section proportionately, so that the pipes with a smaller waterway, but with an increased hydraulic gradient, may discharge the same volume of water as the masonry conduits with an enlarged waterway and a reduced gradient. Thus the hydraulic gradient of the New Croton pipe lines is 3·9 feet in a mile, as compared with 8·4 inches, and 9·9 inches for the masonry conduits with free flow, and under pressure, respectively; of the Loch Katrine siphons, 5 feet $3\frac{1}{2}$ inches in a mile, in place of $11\frac{1}{2}$ inches per mile for the tunnels on the aqueduct; of the siphons on the Elan aqueduct, 3 feet in a mile, instead of the 1 foot $3\frac{5}{6}$ inches per mile for the covered conduits and tunnels; and on the siphons of the Vyrnwy aqueduct, hydraulic gradients ranging from 4·5 feet to 6·87 feet, as compared with a gradient of 2 feet for the tunnels.

SERVICE RESERVOIRS.

The supply of water stored in an impounding reservoir, or pumped up from a river or from wells, or collected from the flow of springs, is conveyed by an aqueduct to a service reservoir, situated at an elevation near the town to be supplied, into which the water is discharged by gravitation if the source of supply is at a considerable height, or is lifted into the reservoir by pumping where necessary.

Objects of Service Reservoirs.—The collection of water from the source of supply in a service reservoir facilitates its distribution, by equalizing the pressure, and by storing the water it provides for the variable demands at different periods of the day. Moreover, the capacity of the reservoir should be adequate to meet an occasional exceptional demand, caused by a large fire or the bursting of a water-main, and also to maintain the supply during repairs to the aqueduct, or a breakdown of the pumping machinery, or other accident involving a temporary stoppage of the supply. The storage provided in these reservoirs should evidently not be less than two days' ordinary consumption, and must largely depend upon the conditions of the supply. Thus where the supply for a town is derived from a variety of sources, a smaller storage is sufficient than when the supply is dependent on a single source, especially when conveyed by an aqueduct of considerable length, rendering the supply liable to be entirely cut off by a stoppage or accident till the necessary repairs can be effected, in which latter case it might be expedient to store a week's supply, or more, in the service reservoir.

Position of Service Reservoir.—A service reservoir should be constructed in a convenient position in relation to the aqueduct conveying the supply to it, on the route between the source of supply and the town to be served; sufficiently near the town for the distribution by mains branching off from it to be readily effected, and yet not close enough, if possible, for the water to be in danger of contamination by smoke, fumes from manufactories, or other sources of pollution emanating from the town. The reservoir should also be at a sufficient elevation to deliver the water by gravitation to the top of the houses in the highest part of the district; and where the land on which the town is built varies considerably in level, it is generally expedient to construct high-level and low-level reservoirs for serving the districts situated at different elevations, especially when the water has to be pumped up into the reservoirs, so as to avoid an unnecessary raising of the water, and a useless pressure on the mains and pipes in the lower parts of the town.

Construction of Service Reservoirs.—These distributing reservoirs are generally formed by excavating them in the ground, or by enclosing them with watertight embankments or retaining walls; and the most economical method consists in partially excavating them, and employing the excavated materials to form embankments for enclosing the upper portion of the reservoir, above the surface of the ground, round the excavated lower part. Where the ground in which they are constructed is not impervious, they have to be lined with puddle, concrete, brickwork coated with cement, or a layer of asphalt.

The embankments surrounding these distributing reservoirs are similar in construction, on a small scale, to the earthen dams of impounding reservoirs, with a central puddle or masonry wall, and pitched

SERVICE RESERVOIRS.

Fig. 127.—Jerome Park, N.Y.

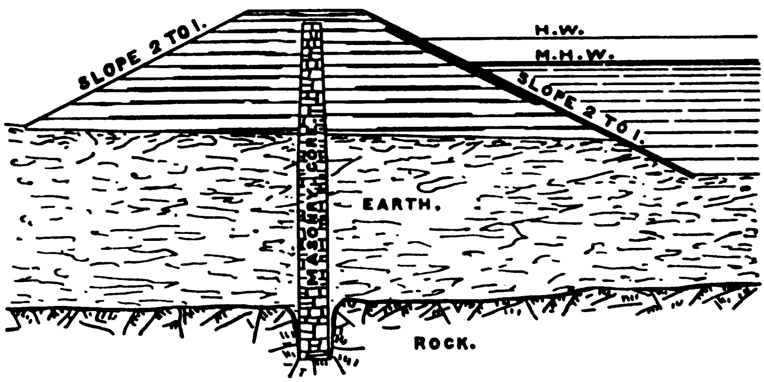
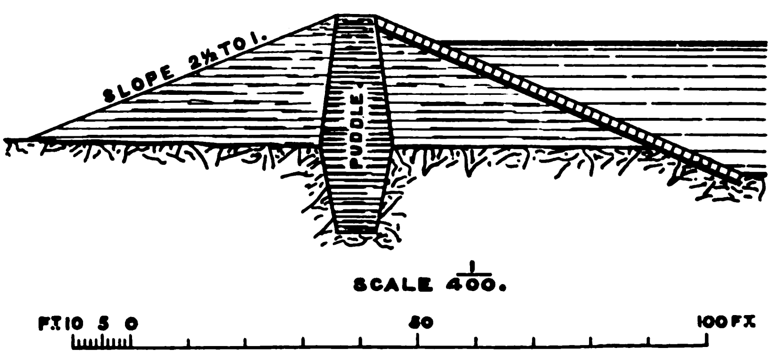


Fig. 128.



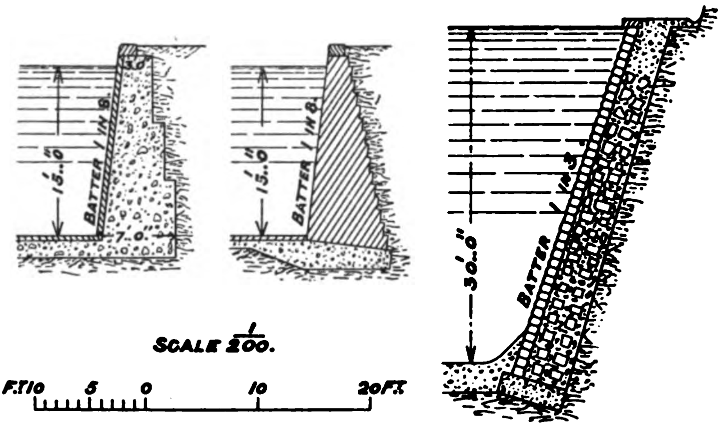
inner slope; and an example of a distributing reservoir partly excavated and partly embanked, is furnished by the Jerome Park reservoir, receiving one-sixth of the supply from the Croton Lake, for distribution in the northern district annexed to New York. This reservoir has a water-area of 228 acres, and a capacity of nearly 1600 million gallons; and it is rendered watertight by means of a masonry core-wall in the centre of the embankment, carried down into the underlying rock, and by a layer of concrete all over the bottom,[1] Fig. 127. In many cases the surrounding embankments, formed with the materials excavated from the enclosure for deepening the reservoir, are provided with a central puddle wall, in continuation of a puddle trench founded in an impervious stratum, Fig. 128, resembling the method of construction adopted for the Staines reservoirs storing the flood-waters of the Thames, Fig. 59 (p. 100).

Where the space is limited, or the available materials are unsuitable for forming a watertight embankment, or the reservoir is mainly ex-

SERVICE-RESERVOIR WALLS.

Vyrnwy Balancing Reservoir. Fig. 129. Fig. 130.

Service Reservoir, Hong-Kong. Fig. 131.



cavated below the surface, retaining walls are adopted for lining the reservoir, thereby utilizing the space which would have been occupied by the inner slope of an embankment or excavation,[2] Figs. 129, 130, and 131. It is important to place the inlet and outlet at opposite ends of these reservoirs, so as to ensure the circulation of the water, which assists in preserving its purity.

Covered Service Reservoirs.—The covering over of these service reservoirs is very advantageous, especially when the water stored in them has been collected from underground waters, or rivers fed by springs, as such waters are very liable to contain vegetable germs, whose growth is checked by the exclusion of light, and also after the

[1] "The Water-Supply of the City of New York," E. Wegmann, p. 209, and plate 134.

[2] *Proc. Inst. C.E.*, vol. cxxvi. p. 56; and vol. c. p. 270, and plate 9, fig. 14.

COVERED SERVICE RESERVOIRS.

Fig. 132.—Timber Covering.

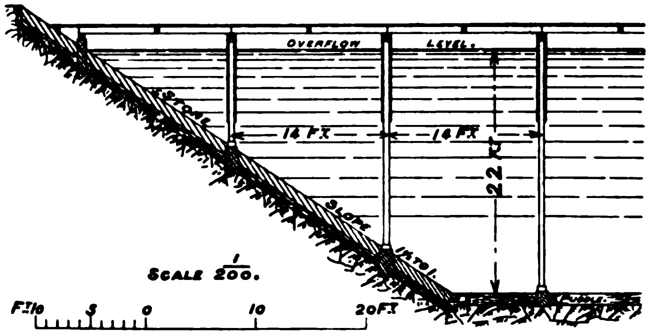


Fig. 133.—Brick Arches on Masonry Piers.

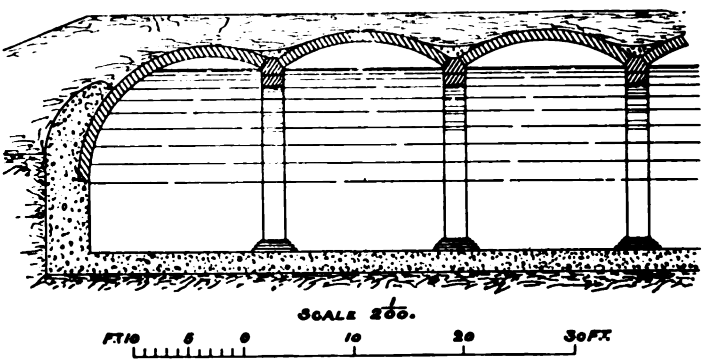
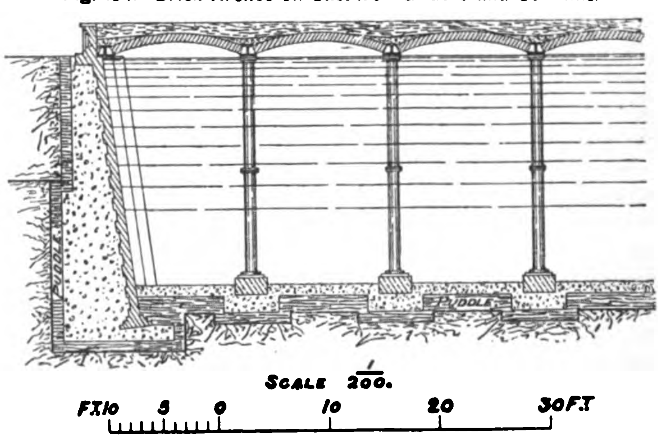


Fig. 134.—Brick Arches on Cast-iron Girders and Columns.



water has been filtered, when it is specially liable to be contaminated by any impurities to which it may be exposed. Moreover, the water in reservoirs near towns requires to be guarded from dust, smoke, fumes, and other sources of pollution; and the water in a covered reservoir is maintained at a more equable temperature than in an open reservoir. Coverings of wood or metal, and slated or tiled roofs, have sometimes been used,[1] Fig. 132; but the most durable and efficient protection consists of brick or concrete groined arches, supported by a series of brick or iron pillars standing in the reservoir, covered over on the top by 2 or 3 feet of earth,[2] as shown in Figs. 133 and 134. This form of protection, though somewhat costly, has the special value of maintaining the water at a lower temperature in summer, and a higher temperature in winter, than the open air, and also of preserving it from considerable fluctuations. Some connection with the outer air has to be furnished at the top, to provide for variations in the water-level, as well as for ventilation; and all these reservoirs must be provided with an automatic valve at the inlet for shutting off the supply when full, a pipe for emptying the reservoir when required, and an overflow for the escape of any surplus water. An arched roof of armoured concrete, with a span of $54\frac{1}{3}$ feet, was adopted for covering a reservoir at Rockford, Illinois, in 1894.

Water-Towers and Elevated Tanks.—Occasionally no site can be obtained for a service reservoir at a sufficient elevation to provide the requisite pressure for the efficient delivery of the supply; and in such cases it becomes necessary to raise the reservoir, or tank, by placing it on the top of a structure built to an adequate elevation on the highest available ground, or to erect a high steel stand-pipe, in which the water can be raised by pumping to an adequate height to furnish the required pressure, together with a certain amount of storage. Under these circumstances, however, the cost of the reservoir is considerably increased; and it is, therefore, expedient to limit the storage to the minimum adequate volume. In Europe, these elevated reservoirs are often supported on masonry towers, an instance of which for a balancing reservoir on the Vyrnwy aqueduct, at Norton Hill, has been already referred to on page 198; and a masonry water-tower at Calbe, on the River Saale, in Germany, is shown in Fig. 135.[3] In America, however, the usual arrangement consists of a cylindrical metal tank raised to the requisite height on open, light, metal trestlework, composed

[1] *Engineering News*, 1898, vol. xxxix. p. 374.

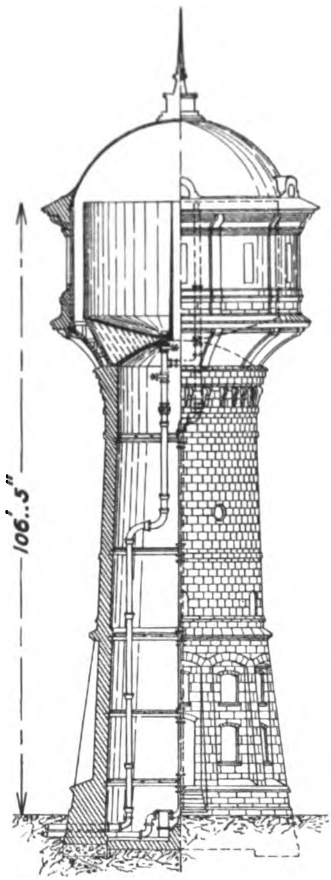
[2] "Covered Service Reservoirs," W. Morris, *Proc. Inst. C.E.*, vol. lxxiii. pp. 19 to 21, and 31 to 33, and plate 2, figs. 21 and 22, and plate 3, figs. 32 and 33.

[3] *Zeitschrift des Vereines Deutscher Ingenieure*, Berlin, 1897, vol. xli. pp. 301 to 305.

of a series of standards braced together, as illustrated by Fig. 136, with a central supply pipe extending down from the conical or hemispherical base of the tank.[1] These elevated tanks vary greatly in size according to the storage required, one of the largest in the United States having been erected at Greenwich, Connecticut, in 1889, 80 feet in diameter and 35 feet high, made of wrought iron, and having a capacity of $1\frac{1}{10}$ million gallons. The highest of these tanks has been recently erected at Camden, New Jersey, where the steel tank, 26 feet in diameter and $33\frac{1}{2}$ feet high, together with a conical top and hemispherical bottom, and containing 150,000 gallons, has its base raised 200 feet above the ground; and the structure, with the flag-staff on the top, has a total height of 300 feet.[2] At Shanghai, where, owing to the liability of the town to conflagrations, it is very important to maintain a good pressure of water in the mains, and no high site for a service reservoir was available, a plate-iron tank, containing 150,000 gallons of water, has been erected on iron columns to an elevation of $103\frac{1}{2}$ feet, having a diameter of 50 feet and a depth of $12\frac{1}{4}$ feet, and roofed over; and the central plate-iron conduit is 6 feet in diameter,[3] Fig. 137.

MASONRY WATER-TOWER AT CALBE, ON THE RIVER SAALE, GERMANY.

Fig. 136.



SCALE $\frac{1}{400}$.

F.10 0 50 FT.

[1] *Engineering News*, 1894, vol. xxxi. pp. 206-207.

[2] *The Engineering Record*, New York, June 20, 1903, p. 657.

[3] "Shanghai Waterworks," J. W. Hart, *Proc. Inst. C.E.*, vol. c. pp. 220 to 229, and plate 7, fig. 1.

Steel tanks raised upon steel trestles have been extensively adopted in the United States since 1890, as a cheaper and more efficient way of storing up water at the requisite height for the desired pressure than

ELEVATED TANKS.

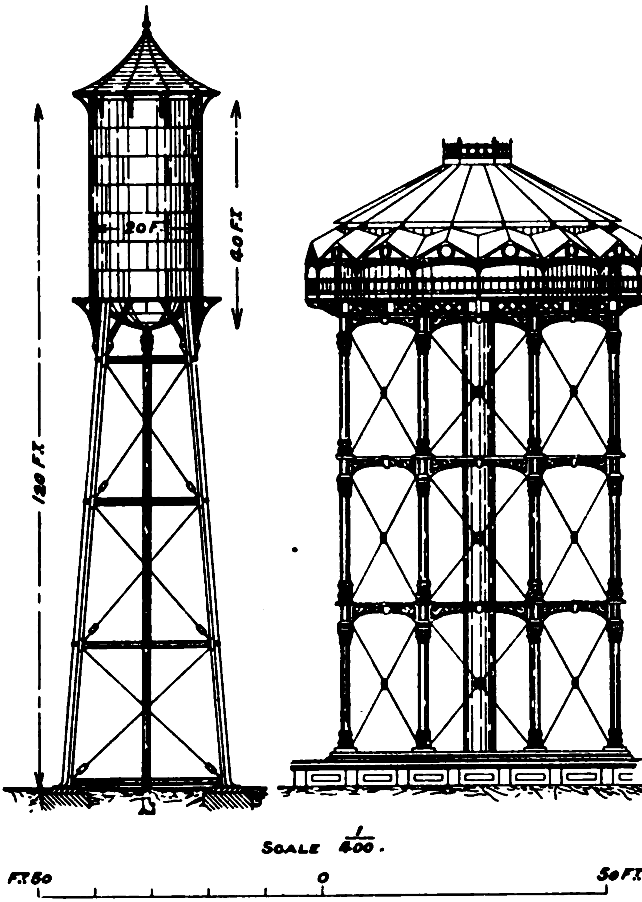


Fig. 136.—On Braced Steel Standards at Laredo, Texas.

Fig. 137.—On Iron Columns at Shanghai.

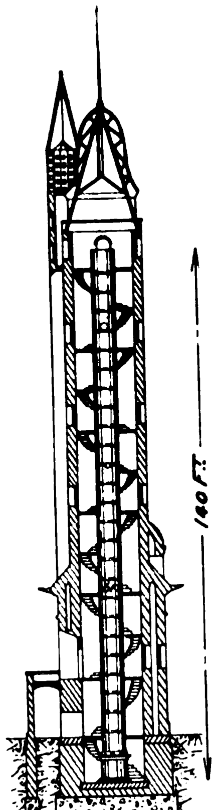
stand-pipes, which formerly were almost wholly resorted to for the purpose. On the average, these elevated tanks are about 37 feet high, 21 feet in diameter, and raised about 63 feet above the ground, and have a capacity of about 100,000 gallons.[1]

[1] "Towers and Tanks for Waterworks," J. N. Hazlehurst, New York, 1901.

Stand-Pipes.—Vertical stand-pipes, rising from the ground, serve, like elevated tanks, to regulate the flow delivered by pumps, and to provide the necessary pressure in the mains; but though somewhat simpler in construction than elevated tanks, they only store up the water effectively in the portion of the pipe above the height of the column of water needed for providing the required pressure. Accordingly, where the local conditions necessitate the provision of a considerable head of water, it becomes more economical to construct an elevated tank, which is quite as efficient as a stand-pipe in storing up water under the full pressure for emergencies. Though stand-pipes over 200 feet in height have been erected in the United States, and have been given a maximum capacity of about $1\frac{1}{4}$ million gallons, they generally range between 50 and 120 feet in height, and from 11 to 39 feet in diameter, and, on the average, have a capacity of 150,000 gallons, and afford a pressure of 62 lbs. on the square inch, being placed on the highest available site to reduce their required height. These stand-pipes are often enclosed by a tower of masonry to protect them from frost and wind-pressure, and in some cases to give them a more sightly appearance, as exemplified by the tower on Compton Hill, enclosing the stand-pipe of the St. Louis waterworks, 140 feet in height and 6 feet in diameter,[1] Fig. 138.

STAND-PIPE ENCLOSED IN MASONRY AT ST. LOUIS, U.S.

Fig. 138.



SCALE $\frac{1}{600}$.

FT10 0 50 FT.

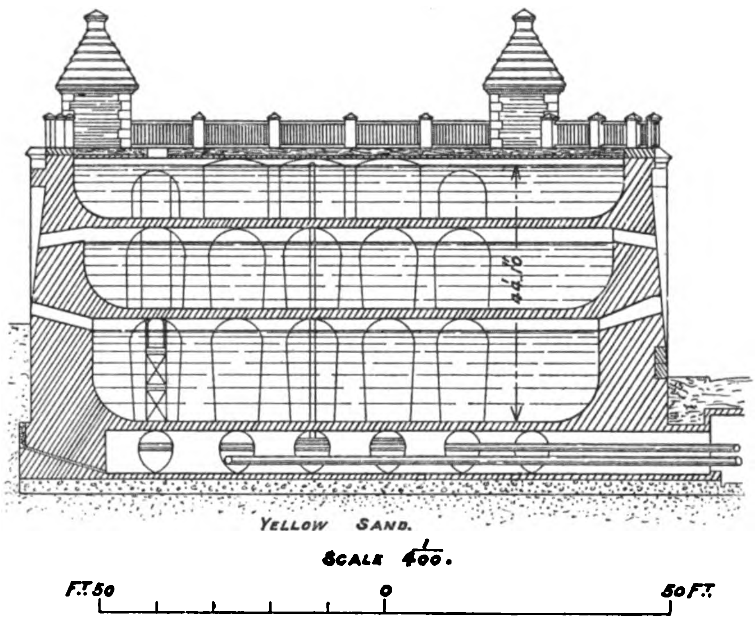
Elevated Masonry Service Reservoirs in Storeys.—Water-towers or elevated tanks have been only adopted in France under exceptional conditions; but raised service reservoirs have been sometimes constructed there, as exemplified by some of the reservoirs in the outskirts of Paris. Two of the most notable of these raised reservoirs have been erected side by side at Montmartre, for supplying the highest portions of Paris, and were completed about 1890, at

[1] *Engineering News*, 1898, vol. xxxix. pp. 206–207.

a cost of about £40,000.[1] These two adjoining reservoirs cover an area of 2750 square yards, and have a combined capacity of 2,420,000 gallons, contained on three storeys in one reservoir, and two storeys in the other, of which 1,364,000 gallons consist of spring-water, and 1,056,000 gallons of river-water, Fig. 139. The basement is reserved for the inlet and outlet pipes; the first floors in both reservoirs store up river-water, which is pumped up into them to a depth of $16\frac{3}{8}$ feet, with a total volume of 1,056,000 gallons at an elevation of 420 feet; the second

STOREYED MASONRY RESERVOIR, MONTMARTRE, PARIS.

Fig. 139.



floors in both reservoirs store up spring-water to a depth of $11\frac{1}{2}$ feet, with a total volume of 924,000 gallons at an elevation of 433 feet; and the third floor in the larger reservoir, stores up spring-water to a depth of $8\frac{1}{8}$ feet, at an elevation of 446 feet, which supplies the highest parts of Montmartre. Each floor is supported and covered over by a series of arches, supported by several central piers and the side walls; and the reservoirs in each building, and each storey, can be filled and emptied quite independently. In other reservoirs raised on arches to increase the pressure, in the outskirts of Paris and elsewhere in France, the

[1] *Nouvelles Annales de la Construction*, Paris, 1890, 4th series, vol. vii. cols. 17 to 22, and plates 6 to 8.

basement is sometimes used for shops, as at Rheims and Orleans; at Montrouge in Paris, it receives the overflow from the reservoir of spring-water above, and supplies it in case of need at a lower pressure; and in other cases, as at Belleville and Passy, it stores the inferior river-water for public uses and manufactories.[1]

[1] "Distributions d'Eau," G. Bechmann, Paris, pp. 285–286.

CHAPTER X.

PURIFICATION OF WATER-SUPPLIES.

Value of pure Source—Impurities contained in Water, visible in suspension, matters found in solution—Nature of Waters from Different Sources, peat in mountain streams, inorganic salts in deep wells and springs dependent on strata, varieties, contamination of river-waters; Indications furnished by Chemical Analysis, and Bacterial Examination, substances indicating contamination, importance of nature of bacteria, dangerous forms—Strainers, object, instances—**Sedimentation**: Settling Ponds at Impounding Reservoirs, object, advantage; Settling Basins, for turbid river-waters, instance of Calcutta supply; Methods of working Settling Basins, intermittent and continuous, merits and defects of systems; Provision for Circulation of Water in Settling Basins, arrangements in hot and cold weather—**Filtration**: Natural, essential for river-waters, instances of use for water from reservoirs; Methods of Filtration, slow and rapid; Slow Filtration through Sand, arrangement and composition of filter-beds; Standard of Size, and Uniformity of Sand for Filtration, effect of size on results, effective size, uniformity coefficient, examples; Periods for Settlement, and Thickness of Sand, instances of periods, of thickness of bed of sand; Rate of Slow Filtration, importance, reduction by clogging of sand, scraping off top layer, variation with head and temperature, examples of rate; Reserve Area and Size of Filter-Beds, object of reserve area, instances of sizes; Materials and Thicknesses of Under-Layers of Filter-Beds, variations owing to local conditions, table of various filter-beds, with maximum depth of water and rate of filtration, mode of commencing filtration; Covered Filter-Beds, filtration and cleaning impeded by frost, mode of scraping under ice, advantages of covering, increased cost of roof, covered and open filter-beds compared; Cost of Filtration, instances for construction of open and covered filter-beds per acre, and of working; Regulation of Rate of Filtration, modification of head, automatic regulators, at Warsaw, Berlin, Hamburg, and Tokio described; Covered Clear-Water Reservoir, object; Efficiency of Slow Sand Filtration, reduction of bacteria, instances, limits; Rapid Filtration, description, method of working; Action of Coagulant for Rapid Filtration, substances used, reaction produced, effects; Rate of Filtration through Mechanical Filters, amount discharged per day, expediency of subsidence; Cost of System of Rapid Filtration, instances; Efficiency of Rapid Filtration, removal of bacteria; Remarks on Filtration, importance for river-waters, the systems compared—**Various Methods of purifying Water**: Aeration, for removal of organic matter and iron, methods of effecting, use of ozone; Treatment of Water with Chemicals and Filtration, for removal of iron, substances used for precipitating iron, subsidence and filtration; Purification of Turbid Waters with Iron, by contact in revolving purifiers, results; Methods of rendering Suspected Waters safe for Drinking, boiling, chemicals used; Softening Water, defects of hard water, salts producing hard waters, removal of temporary and permanent hardness, subsidence of precipitate; Separation of Precipitate from Softened Water, simple subsidence, more rapid methods, by filter presses or disks; Cost of Softening Water, and Reduction in Hardness, examples; Concluding Remarks, value of subsidence and filtration, desirability of softening, death-rates from filtered and unfiltered waters contrasted.

IMPURITIES found in water-supplies are almost wholly due to the sources from which the water has been obtained; for though contamination of the water between its source and reception into the service reservoir occasionally occurs, it can generally be guarded against to a great extent by proper precautions. Most waters require to be subjected to some process of purification, before they can be regarded as suitable for domestic supply; but waters from a pure source possess the inestimable advantage of securing immunity from certain forms of organic pollution, which cannot be always with absolute certainty removed by purification; whilst they also reduce the amount of purification necessary to provide a perfectly satisfactory supply.

Impurities contained in Water.—Absolutely pure water is hardly ever met with; for even rain-water, the purest natural form of water, begins to collect impurities in its descent from the clouds; and the only way of artificially obtaining pure water is by its distillation, a process too costly to be resorted to for water-supplies, except where no other means are available, as on board ship, and at some rainless places by the sea, like Aden.

Substances are contained in water both in suspension and solution; but whereas suspended particles are readily observed, and can for the most part be removed by settlement in still water, and subsequent filtration, colourless matters in solution can only be detected by taste, smell, and chemical and bacterial analyses; whilst sometimes sparkling and clear water, such as has occasionally been drawn from shallow wells, or from springs passing through graveyards, though apparently perfectly pure, may contain deadly germs of disease. Moreover, only certain of the substances found in solution in some waters are capable of removal by any available process. The nature also of the substances in solution very greatly affects the wholesomeness of the water; for whereas most of the inorganic matters commonly found in solution in waters drawn from ordinary sources of supply, are innocuous in respect of health, the presence of more than a very small amount of organic matter in the water exposes it to suspicion; and most organic substances are liable to give rise to dangerous pollution. The inorganic substances commonly found in solution in river-, spring-, or well-waters, though in very variable quantities, are chiefly calcium and magnesium bicarbonates and sulphates, sodium chloride near the sea, or when exposed to tidal influences, small quantities sometimes of magnesium and sodium carbonates and potassium salts, some silica and traces of iron, and oxygen, nitrogen, and carbonic acid gas; and these substances are quite harmless in moderate quantities. Sulphuretted hydrogen and sulphurous acid, however, and even iron in appreciable quantities, which are met with in some springs, render the water unpalatable; the presence of ammonia in water generally indicates pollution; and the appearance of the nitrates of certain metals in the drinking water supplied, especially those of lead, copper, and zinc, resulting usually from artificial causes, demand instant investigation, in order to put a stop to further contamination. The organic matters found in solution consist almost wholly of peat in several waters impounded in uninhabited and uncultivated mountainous districts; but river-waters may contain in solution numerous vegetable substances, various animal impurities, the products of putrification in the form of alkaloids and amido-acids, phenol and its derivatives, and sundry waste products.[1]

Nature of Waters from Different Sources.—The waters stored in a mountain valléy, when flowing off moorlands, or derived from mountain streams passing through heather, have often a peaty discoloration. Nevertheless, provided none of the water comes from marshy swamps, which are specially subject to pollution in remaining stagnant, the impurity, though unsightly, is not in itself injurious to health; but it should be removed as much as practicable, for the sake of appearances, on account of its favouring the growth of organisms, and owing to its tendency to act upon lead pipes. The waters issuing from springs, or flowing into deep wells, are generally remarkably free from organic impurities, and devoid of matters in suspension, having undergone natural filtration in their subterranean course; but they arrive more or less charged with the soluble mineral salts contained in the strata traversed. The condition of the supply obtained in such cases depends, indeed, on the nature and state of the strata, and the distance passed through; and occasionally springs are so impregnated with iron, sulphur, and saline, magnesian, or other salts, as to be only serviceable for medicinal purposes. Usually the nature of the spring- or well-water may be inferred from the stratum supplying it. Thus the waters from

[1] "Water and its Purification," S. Rideal, 1902, p. 11.

the Bagshot Sands and the Thanet Sands are soft, but not very reliable; the Upper and Lower Chalk yield very hard water, owing to calcium salts, but organically pure; whereas water from the Grey Chalk contains organic matter; and the waters from the Upper and Lower Greensands tend to be ferruginous and turbid. The Oolite strata furnish numerous springs, giving water which is generally hard from calcium salts, but of very varied quality, sometimes clear and good, and at other places turbid or ferruginous, and occasionally saline from salts of sodium. The Upper New Red Sandstone, or Trias, in its upper beds, gives unsatisfactory water, owing to the presence of organic matter and calcium phosphate; in some beds it provides very good water; while in other parts, where deposits of gypsum, rock salt, and ferric oxide exist, the water is unfit for drinking. The Magnesian Limestone yields a considerable quantity of water, which is hard with calcium and magnesium salts, and occasionally saline and unwholesome; and the Lower New Red Sandstone provides large quantities of water, which, though organically pure, are liable to be saline and ferruginous. In the carboniferous strata, the Coal-Measures furnish a small amount of water tending to be medicinal in quality; the Millstone Grit yields excellent water; and the Mountain Limestone furnishes a plentiful supply, which in places is good though somewhat hard, but in some parts is rendered unpalatable by organic matter, and is occasionally contaminated by veins of lead and zinc. The purest supplies in the lowest primary, and igneous and metamorphic rocks, are obtained from the Lower Silurian, and Granite, Gneiss, and Quartz strata; and the Laurentian and Crystalline Limestone formations yield water similar in character, but rendered hard by calcium bicarbonate in solution; whilst the waters from trap and basalt, though also free from organic matter, are liable to be somewhat impregnated with decomposing minerals contained in these rocks.[1] Rivers, which are generally fed to some extent by springs, besides carrying down some of the substances found in spring-water in solution, are exposed to various sorts of contamination in flowing through inhabited and agricultural districts, especially in flood-time, when, in addition to the impurities discharged into the river or its tributaries and streams in the ordinary course of drainage, and the refuse thrown in, all kinds of vegetable and animal matter are washed off the land into the river, so that large quantities of organic impurities, both in suspension and solution, are brought down by the floods, rendering the water absolutely unfit for human consumption without very efficient means of purification.

Indications furnished by Chemical Analysis, and Bacterial Examination.—Certain results of chemical analysis indicate contamination of the water. Thus a higher percentage of chlorine than

[1] "Water and its Purification," S. Rideal, pp. 326 to 332.

can be accounted for by the calcium and magnesium chlorides present, and occasionally sodium chloride in the vicinity of salt deposits or the sea, point to sewage contamination; whilst the amount of combined nitrogen in the water in the forms of ammonia, nitrites, and nitrates to some extent, furnishes valuable information in the same direction.[1] A large amount of free ammonia in water generally indicates recent sewage pollution, being produced by bacteria from urea; and albuminoid ammonia, or the ammonia remaining after the removal of the free ammonia, furnishes a measure of the unoxidized organic matter present in the water. Analysis is also useful in determining the composition and relative amounts of the mineral salts in solution, whereby the suitability of a water for domestic or manufacturing purposes is further ascertained; and a determination of the amount of dissolved oxygen is valuable, in providing by its increase a measure of the purification of a river in its flow, and the effect of filtration.

Bacterial examination of the water is also essential, owing to the varied forms of the organisms, even though chemical analysis indicates their presence by changes in chemical composition, and by the discovery of nitrogenous compounds, which are necessary to the existence of bacteria, who depend upon them for their food. The bacteria are identified by examination under the microscope, and by various experiments and cultures under different conditions; and whereas much the larger numbers of bacteria, in spite of being objectionable in drinking water, perform very valuable purifying functions, there are some which indicate serious pollution, such as the bacillus *coli communis* and its congeners, whose presence denotes with certainty contamination by sewage, in which they flourish in large numbers, and which are pathogenic to the lower animals. Moreover, there are at least three forms of bacteria, already clearly identified, which are most dangerous to life, namely, the typhoid bacillus, the cholera bacillus, and the anthrax, or splenic fever, bacillus, which are all conveyed by water into the human system, though the last appears in the first instance in wool, hair, and fur, but is readily transferred to water in the process of cleaning these materials; whereas the two first are direct products of polluted water.[2] No waters are wholly free from bacteria; but the presence of bacteria in large numbers furnishes evidence of organic contamination; and the efficiency of any system of purification depends as much on the extent to which it can reduce the bacteria in the water, as upon the removal of other impurities.

Strainers.—In drawing water from storage reservoirs or rivers, besides selecting a position for the intake where the water is clearest, the

[1] "Water and its Purification," S. Rideal, p. 266.
[2] *Ibid.*, pp. 55, 56, and 300 to 308.

larger substances in suspension, such as floating leaves, sticks, vegetable refuse, and other *débris*, and also fish, are prevented from entering the conduit by interposing a strainer with fine meshes, as already described in the case of the Vyrnwy tower and the Thirlmere well, pages 187–188 and 189, and Figs. 114 and 115. These strainers have to be washed periodically to remove the slimy matters which gradually accumulate on their outer face, and by degrees choke the meshes; and this cleansing is best effected by means of jets of water.

SEDIMENTATION.

Settling Ponds at Impounding Reservoirs.—Sometimes a settling pond is formed at the upper end of a storage reservoir to receive the turbid waters of the impounded stream, so that the main portion of the sediment in suspension may settle in the pond before the water passes into the reservoir; and the accumulated deposit is periodically removed. Even, however, without this provision, a large storage reservoir acts as a settling basin for its waters; for the sediment is very effectually deposited from the still waters of the artificial lake, but, in this case, has the disadvantage of gradually reducing the capacity of the reservoir.

Settling Basins.—Water drawn from a river is often turbid with matters in suspension, especially in flood-time; and it is essential for the water to be rendered clear before being supplied to a town. The larger and heavier matters in suspension only need to be in still water to be made to deposit; and this is accomplished by passing the water from the intake, straight into settling tanks or basins, in which the water is cleared from a considerable portion of its sediment, leaving only the lighter and finer particles to be removed by filtration, which both renders this latter operation more rapid and easier, and also relieves the filter from being very quickly clogged by the large amount of sediment in suspension in the water as drawn from the river. Thus the extremely muddy waters of the River Húgli, which have the appearance of being quite unsuitable for water-supply, but furnish practically the only available source of supply for Calcutta, and also for its suburb Howrah on the opposite bank of the river, when drawn by pumps from the river at Palta, about 14 miles above Calcutta, for supplying the city, are discharged into the nearest extremity of a series of long settling tanks, from the farther end of which the cleared water passes through a main to the filter-beds,[1] Fig. 140.

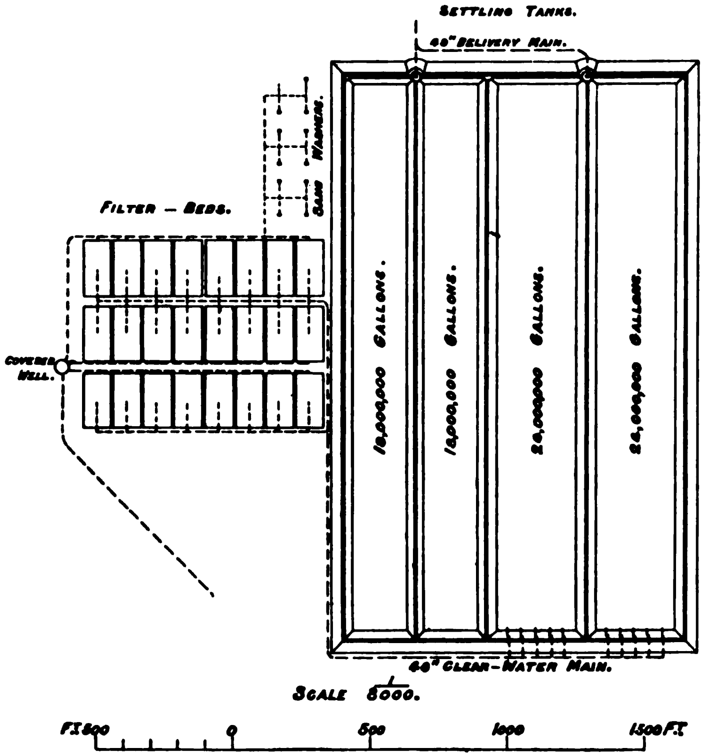
Methods of Working Settling Basins.—The deposit of

[1] "The Calcutta Waterworks," A. Pierce, *Proc. Inst. C.E.*, vol. cxlviii. p. 329, and plate 10, fig. 2.

sediment from river-water in settling tanks may be accomplished in two ways: either by the intermittent system of filling the tank, leaving the water perfectly at rest for a certain time according to the nature of the sediment, and then emptying the tank, and repeating the process; or by the continuous system of admitting the water slowly, with a constant flow, at one extremity of the settling tank, and discharging it at the

SETTLING TANKS AND FILTER-BEDS, PALTA, CALCUTTA WATERWORKS.

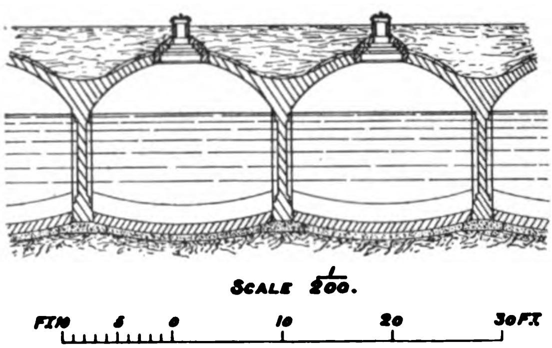
Fig. 140.



same rate by overflow into a trumpet-shaped, horizontal orifice, about flush with the water-level, at the opposite end, forming the inlet to the main leading the cleared water to the filter-beds. The intermittent method of settlement secures the perfect repose of the turbid water for a definite period; but the periods of filling and emptying are practically so much time lost in respect of the settling of the sediment; the water after having been admitted occupies a little time in coming to rest, and towards the end of the emptying is liable to carry away some of the lightest deposit at the bottom; whilst the bottom of the settling basin must be at a higher level than the filter-beds for the bottom layers of water to flow to them, involving an increased lift for the pumps. The continuous flow, on the other hand, whilst insuring an almost equal extent of stillness in a long settling basin, enables a larger quantity of water to be cleared in a given time, and, consequently, does not need the same extent of basins as the intermittent system, and only requires the overflowing top water-level to be a little higher than the filter-beds to provide the flow to them by gravitation. In the continuous system, however, it is essential that the settling basins should be made in a series of long, narrow sections, Fig. 140; for a central flow is liable to be set up by the entering water in a wide basin, resulting in an absence

SETTLING TANKS DIVIDED BY WALLS AND ROOFED OVER.

Fig. 141.



of change of the water at the sides; and, moreover, this subdivision of the settling basins facilitates their being roofed over where expedient,[1] Fig. 141.

Provision for Circulation of Water in Settling Basins.—The course of the water discharged into a settling basin is affected by any difference between its temperature and that of the water standing in the basin; so that when, as in summer, the entering water is warmer, and therefore lighter than the water in the basin, it flows over the top of the other straight to the outlet. This result can be obviated, and the due circulation of the water in the basin, and the consequent more efficient deposit of sediment secured, by placing a vertically sliding gate across the basin near its outlet, as indicated by the diagrams,[2] Figs. 142

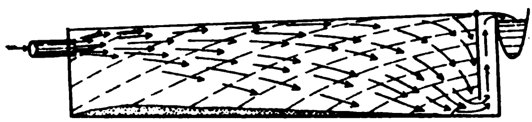
[1] "L'Usage des Eaux de Rivière," W. H. Lindley, Congrès International de l'Utilisation des Eaux fluviales, Paris, 1881, pp. 47-48.

[2] "L'Usage des Eaux de Rivière," W. H. Lindley, Paris, 1889, pp. 47 to 50.

and 143; for by lifting the gate in warm weather, so as to leave an aperture at the bottom of the tank for the escape of the water, and sufficiently at the top to arrest all surface flow, the warm incoming water is prevented from reaching the outlet till it has been adequately cooled to descend to the bottom, and thus deposit its suspended matters in its circuitous course, Fig. 142. On the contrary, by keeping the gate

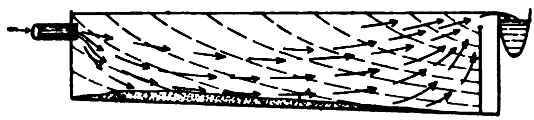
FLOW OF WATER IN SUMMER THROUGH SETTLING TANK. GATE RAISED.

Fig. 142.



FLOW OF WATER IN WINTER THROUGH SETTLING TANK. GATE LOWERED.

Fig. 143.



shut down on the bottom in cold weather, the colder water entering the tank, and falling to the bottom, requires to be warmed up to the temperature of the water in the tank, before it can rise and flow over the top of the gate to the outlet, Fig. 143. The deposit of sediment in these settling tanks has been found not only to render the water clearer, but also to reduce the number of micro-organisms, and thus make the water more organically pure.

FILTRATION.

Water which has traversed long distances underground before issuing as a spring or flowing into a deep well, has undergone a process of natural filtration in its passage through the strata, by which it is generally freed from any organic impurities it may have collected at, or near the surface. River-waters, however, more particularly require to be filtered after issuing from settling tanks, both in order to remove the finer particles of matter still remaining in suspension, and also to free them from organic matters, and especially from the dangerous forms of bacteria, such as the typhoid bacillus, and the other noxious bacilli already alluded to, with which rivers and other surface waters are liable to be contaminated. In the case of waters impounded in reservoirs, derived from hilly, uninhabited, and uncultivated watersheds, filtration is occasionally dispensed with, as for instance the water-supplies for Manchester from the Longdendale Valley and Thirlmere, and the Loch Katrine water for Glasgow, reliance being placed, both on the original purity of the water, and also on the purifying effects of the settlement of sediment in the still waters of the reservoirs and lakes. A possibility, however, always exists of accidental contamination; and therefore many of the purest impounded supplies are filtered before being delivered into the service reservoirs for distribution, as for instance the Croton reservoir supply for New York, and the supplies from the Rivington and Vyrnwy reservoirs for Liverpool. Moreover, it has proved important, under certain circumstances, for the filtration to be effected as near to the storage reservoir as practicable; for in the case of the water from the Vyrnwy Lake, which is filtered at a convenient site near Oswestry, about 18½ miles from the reservoir, where there is an ample available fall of the land, a ferruginous, gelatinous slime has been found to form in the pipes, with a growth of organic ferruginous filaments, sufficiently to impede materially the flow of the water in its passage from the Vyrnwy Lake to the filter-beds, which does not reappear after filtration, Fig. 118 (p. 197). This growth of slime has been traced to acidity in the water, and the presence of iron from springs, together with a little manganese.[1]

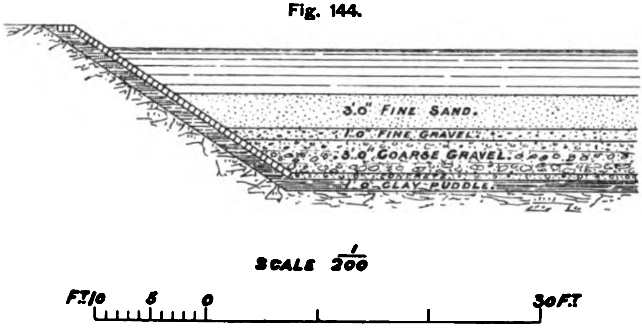
Methods of Filtration.—Two systems of filtration have been adopted, namely, slow filtration through a layer of fine sand; and rapid filtration through coarse sand, in which a chemical solution is previously added to the water, in order to cause the settlement of the fine particles in suspension, and thus render the filtration efficient. The first system is regularly employed in the United Kingdom, and very generally resorted to in Europe and the British colonies and settlements, more particularly for the purification of river-waters; whilst in the United States, though several of the principal cities use the method of slow filtration through sand, many of the towns have adopted the system of rapid filtration.

Slow Filtration through Sand.—The sand filter-beds are placed in moderate-sized, shallow reservoirs or tanks, having a watertight bottom of impermeable soil, puddled clay, concrete, or brickwork, and inclosed by watertight embankments or retaining walls, and generally left open to the air at the top,[2] Figs. 144 and 145. The filter-beds consist of a bed of sand at the top, resting upon a layer of coarse sand or fine gravel, supported by a layer of coarse gravel or small stones, which is underlaid by rubble stone, or two or more courses of bricks laid dry with intervals between them. The water is gently introduced

[1] "Liverpool Public Health Congress, 1903, Handbook," pp. 212–213.
[2] *Proc. Inst. C.E.*, vol. cxlviii. pp. 329 and 337, and plate 10, fig. 3.

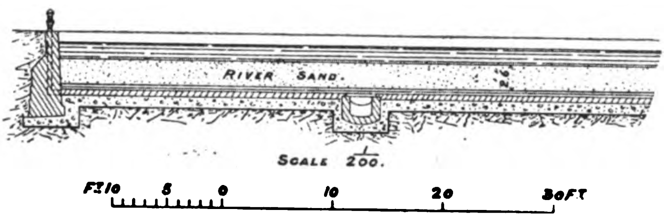
FILTER-BED WITH SLOPING SIDES.

Fig. 144.



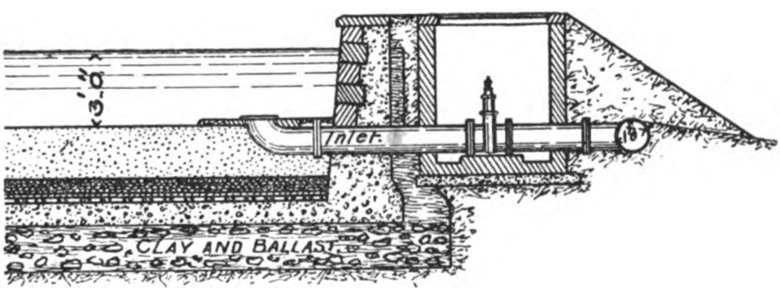
FILTER-BED WITH SIDE WALLS, CALCUTTA WATERWORKS.

Fig. 145.



FILTER-BED SHOWING INLET, YOKOHAMA WATERWORKS.

Fig. 146.



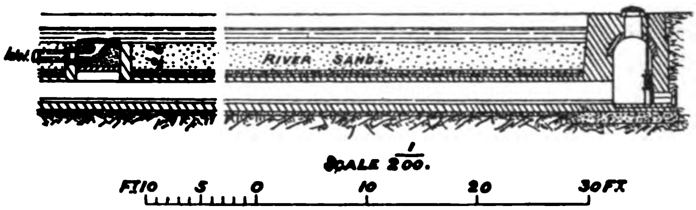
above the sand,[1] Fig. 146, and, passing slowly through the filter-bed with its gradually enlarging interstices downwards, flows down the sloping floor to the main drain below the lowest point, along which it

[1] "The Yokohama Waterworks," J. H. T. Turner, *Proc. Inst. C.E.*, vol. c. plate 10, figs. 5 and 6.

is led to the outlet, Figs. 145, 147, and 148. The top bed of sand, which has been given a thickness of from under 2 feet up to about 5 feet, constitutes the filtering medium, and is merely supported by

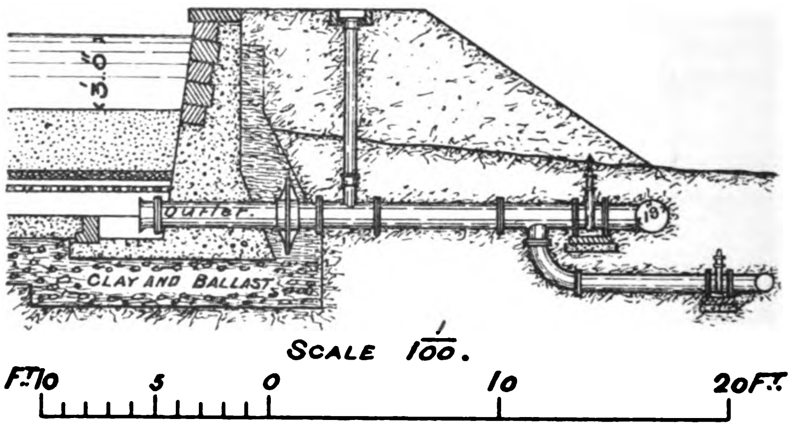
FILTER-BED SHOWING CENTRAL DRAIN, AND INLET, CALCUTTA WATERWORKS.

Fig. 147.



FILTER-BED SHOWING OUTLET, YOKOHAMA WATERWORKS.

Fig. 148.



the lower layers, of increasing coarseness for facilitating the efflux of the filtered water, which prevent the sand being washed into the drains.

Standard of Size, and Uniformity of Sand for Filtration.—It has been found that the top layer of the bed of sand, or in many cases a gelatinous film which quickly covers the surface, in reality mainly effects the filtration, and that the thickness of the bed of sand, within certain limits, is comparatively immaterial. The size, however, of the particles of sand, and the rate of filtration, exercise a great influence on the efficiency of the filtration; for a fine sand with a slow percolation, depending on the head of water over the filter-bed, provides a proportionately purer supply of filtered water.[1] Fine sand, on the other hand, becomes rapidly clogged with any impurities in suspension in the water, or by the gelatinous film which is the product of bacterial agency, and, consequently, necessitates an increased area of filter-beds, or a greater head of water, which is very liable to result in the escape of imperfectly filtered water. The sand used for filtering should evidently be clean, and as uniform in size as practicable; whilst its effective size depends mainly on the size of its finer particles. Experiments have indicated that the *effective size* of any sand is best represented by such a sized grain that 10 per cent. of the sample by weight is finer than this, and 90 per cent. coarser,[2] the size being reckoned by its diameter in millimetres, a millimetre being equivalent to 0·039 inch. The uniformity in size of the sand, or its *uniformity coefficient*, is measured by the ratio which the size of grain larger than 60 per cent. of the sample bears to the size of grain larger than 10 per cent., or the effective size, so that the sand is more uniform in proportion to the smallness of this ratio. The effective size of sand commonly employed for filter-beds, ranges from the fine dune sand ordinarily used in Holland, of 0·17 mm. at Amsterdam and 0·19 mm. at the Hague, up to 0·43 mm. at Middlesborough and the Rivington filter-beds for Liverpool, and 0·44 mm. at the East London Waterworks for London; but it is usually comprised between 0·30 mm. and 0·40 mm. The uniformity coefficient has been found to vary, in a number of samples of sand used for filtration, from 1·5 up to 4·7, but in most instances is comprised between 1·6 and 2·6.

Periods allowed for Settlement; and Thickness of Bed of Sand.—In order that the sand, especially when fine, may not be too rapidly clogged, it is essential for turbid water to be left to settle for a period depending upon the fineness, lightness, and amount of the materials in suspension; and the water of a river in flood would need much longer for settlement than water drawn off at the low stage of the river. The time, however, allowed varies considerably at different waterworks, even where the water is drawn from the same source; for in the case of the London waterworks deriving their supplies from the Thames, the Grand Junction allows 3⅓ days for subsidence, Lambeth 6 days, and Chelsea 12 days; whilst the East London Waterworks, drawing their supply from the more turbid River Lea, allow 15 days for settlement. In some places shorter periods are adopted, as, for instance, 1 to 1½ days at Hamburg, and only 1 day at Berlin and Rotterdam.

[1] "The Bacterial Purification of Water," Percy Frankland, *Proc. Inst. C.E.*, vol. cxxvii. p. 103.

[2] *Twenty-fourth Annual Report of the Board of Health of the State of Massachusetts*, Boston, 1893, pp. 541 to 556.

The thickness of the bed of sand ranges between 2¼ and 4½ feet at the London waterworks, which is reduced by scraping off the top layer, as it becomes clogged, to 1½ and 3½ feet respectively, before being renewed; and a thickness of 2 feet is adopted at Berlin. No filter-bed should be reduced by scraping to a less thickness of sand than 1 foot, as below this limit the filtered water rapidly deteriorates; and it has been sometimes considered that a thickness of at least 3 to 4 feet of sand secures the filtered water more effectually from loss of purity, in the event of the layer of slime, or top layer of sand, becoming disturbed or disintegrated in places, so as to allow of the passage of imperfectly purified water.

Rate of Slow Filtration.—The rate of filtration necessarily exercises a most important influence on the purification of the water, and is regulated by the depth of water admitted over the filter-bed; and with a given head, it depends upon the fineness of the sand, and the amount of clogging of the top layer from the deposit of sediment, and the coating of slime produced by bacteria. With a new, or recently scraped filter-bed, the rate of flow must be kept very slow; and it is important to let the water first passed through the filter run to waste, till sediment has spread over the upper layer of sand, and the gelatinous film has to some extent formed over the surface, otherwise imperfectly filtered water would be discharged. The rate may then be increased gradually, in proportion as the bed of sand in clogging offers an increasing resistance to percolation, depending upon the sediment in the water, and the time the filter-bed has been in continuous operation, till the head of water approaches the limit beyond which it might be liable to rupture the film. Eventually, when the head of water over the filter-bed has been raised to its specified limit, which, in Europe, has been generally fixed at between 2 and 4⅓ feet, the rate of percolation decreases by degrees, till at length the sand becomes so clogged that the top layer of about half an inch, up sometimes to an inch or slightly over, has to be scraped off, and washed for future use. This process of periodical scraping, at intervals depending on the amount of impurities in the water and the fineness of the sand, and frequently extending under favourable conditions up to from 30 to 50 days, is continued until the bed of sand has been reduced to its minimum allowable thickness; and then the layer has to be renewed with washed or fresh sand to its original height, the old bottom layer occasionally, where special precautions are taken, being removed and replaced again on the top of the layer of clean sand, thereby causing the whole bed of sand to be washed in rotation. Formerly the sand was renewed after each scraping, so as to maintain the depth of the bed intact; but this practice, involving considerably more labour, was discontinued directly it was discovered that a moderate reduction in the thickness of the bed of sand does not materially affect the purification; for the old system had the disadvantage that the sediment tended to pass through the fresh, open layer of sand, and settle on the top of the more compact old bed, so that it clogged the sand below the surface, where it was awkward to remove.

Though the depth of water over filter-beds has ordinarily been limited to about 4 feet, heads of about 6 feet have been adopted in the United States with satisfactory results; whilst still greater heads have been occasionally resorted to under exceptional conditions. The rate of filtration varies not only with the head and state of the sand, but also with the temperature, being double at 77° what it is near 32°; and it is usually stated in inches of vertical subsidence of the water through the filter-beds per hour, or in gallons discharged, either per square foot of filter-bed per hour, or per acre in a day. The rate of vertical subsidence through the filter-beds of the London waterworks ranges from $2\frac{1}{2}$ to 4 inches per hour; whilst at Altona it is 4 inches, at Liverpool it averages $4\frac{1}{4}$ inches, at Berlin 4 to 5 inches, and has attained $11\frac{1}{2}$ to $16\frac{2}{5}$ inches at Zurich, where the lake-water, even before filtration, is very clear and pure. The discharge, accordingly, from the London filter-beds is usually comprised between 2,180,000 gallons and 1,362,000 gallons a day per acre of filter-beds, with a maximum limit of about $2\frac{3}{5}$ million gallons per acre per day, equivalent to $2\frac{1}{2}$ gallons per square foot per hour, which is also considered the maximum standard rate in Germany, and 3 to 4 million gallons per acre per day in the United States; whilst at the Zurich filters, the outflow attains a maximum of 13,552,000 gallons per acre in a day, or 13 gallons per square foot of filter-bed per hour.

Reserve Area and Size of Filter-Beds.—The area of filter-beds to be provided depends upon the supply needed, and the rate of filtration; and besides those in actual use, an additional area is necessary to allow for a certain proportion of the filter-beds lying idle during their cleansing, according to the frequency of the scraping, and the time occupied in the renewal of the bed of sand. Filter-beds are always made of moderate size, to keep down the area thrown out of service during scraping and renewal to a minimum, and, consequently, the additional area required as a reserve. Thus, the filter-beds of Zurich are only between $\frac{1}{5}$ and $\frac{1}{6}$ of an acre each, those of Rotterdam average $\frac{1}{2}$ an acre in area, the later ones of Berlin slightly more, and of Calcutta slightly less than $\frac{1}{2}$ an acre, at Oswestry for Liverpool $\frac{4}{5}$ of an acre, of Albany $\frac{7}{10}$ of an acre, of London 1 acre, and of Hamburg nearly 2 acres. Where the total area of filter-beds is large, each bed may be given a larger area than with a small filtering area; and the cost of construction of large beds is less than that of a series of smaller beds with the same effective area.

Materials and Thicknesses of Under-layers of Filter-Beds.—The composition and thicknesses of the layers of filter-beds underneath the top layer of sand are different in various places, owing generally to local conditions and the materials available. The table opposite gives the nature and thickness of the several layers used for the filter-beds of large cities in various countries, together with the maximum depth of water admitted over the filter-beds, and the greatest rate permitted for the filtration.

A new or cleaned filter-bed is put into operation by filling the bed slowly with filtered water from below, so as to drive out the air from between the grains of sand without disturbing them, and thus avoid passages being formed through the bed of sand, allowing of the escape of the water without due filtration. As soon as the filtered water has risen slightly above the surface of the sand, the influx is stopped; and unfiltered water is admitted from above till the requisite head has been reached; and then the necessary coating of the sand with silt, and the formation of the gelatinous film, are effected, either by allowing the unfiltered water to rest on the filter-bed for about 24 hours, or by letting the water flow very slowly through the filter and run to waste. Provision is made for the escape of any air which may collect in the filter-bed during filtration, by a shaft formed in the side wall, Fig. 145 (p. 229), to prevent interference with the percolation of the water.

Covered Filter-Beds.—In countries exposed to cold winters in which the average temperature is liable to fall below 32° for some time, the formation of ice impedes filtration, and more particularly the cleaning of the filter-beds. The ice has to be broken up to enable the ordinary process of scraping to be carried out; and when the water is lowered in the filter-bed for cleaning, the wet sand is liable to be frozen, preventing scraping, and disarranging the sand. At Hamburg, where the filter-beds are open, ice is liable to form to a considerable thickness over the beds in severe winters of long duration; and the scraping, under these circumstances, was at first performed under water by a sort of bag and spoon from a barge, after breaking up the ice, and subsequently by a scraper hung down from a float moving along the underside of the ice when dragged across the surface of the sand by a rope, the sand removed by the scraper being discharged into a bag attached at the back,[1] Fig. 149.

A more efficient and simpler method of providing for the cleansing of filter-beds throughout long, severe winters, especially in cold climates,

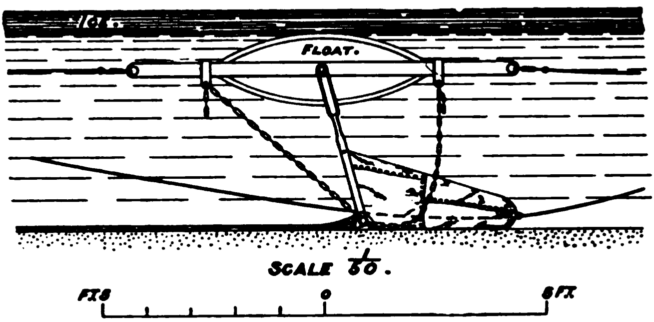
[1] "Die Reinigung der Hamburger offenen Sandfilter in der Frostzeit," *Gesundheits-Ingenieur*, 1897, p. 157.

Filtering materials.	Chelsea.	East London.	Grand Junction.	New River.	Southwark and Vauxhall.	Liverpool.	Calcutta.	Paris.	Berlin.	Hamburg.	Amsterdam.	Zurich.	Warsaw.	Tokio.	Hudson.	Poughkeepsie.	Cincinnati.
	ins.	ins.	ins.	ins.	ins.	ins.	ins.	ins.	ins.	ins.	ins.	ins.	ins.	ins.	ins.	ins.	ins.
Fine sand	54	24	36	27	36	30	30	24	24	40	30	32	24	48	6	31	30
Coarse sand	3	6	—	—	12	6	4	—	—	—	—	6	2	—	18	—	15
Small gravel	—	—	—	—	9	6	—	—	—	—	—	—	3	—	6	—	6
	39	—	15	40	—	—	—	6	12	24	12	4	—	—	—	24	—
Coarse gravel	—	12	—	—	9	—	—	—	—	—	—	—	6	—	6	—	15
Broken stone or shingle	—	—	—	—	—	—	2	—	—	—	—	—	6	—	12	—	—
Large broken stone	—	—	—	—	—	—	—	—	—	—	—	—	—	—	24	—	—
Boulders	—	—	—	—	—	—	—	6	12	—	12	—	11	—	—	—	—
Bricks or drains	—	—	6	—	—	4	6	—	—	—	—	6	—	—	—	—	—
Total thickness	96	42	57	67	66	46	42	36	48	64	54	48	52	48	72	55	66
Depth of water on filter (maximum)	60	60	48	60	48	24	24	36	48	43	39	39	47	36	72	—	48
Rate of filtration per square foot per hour (maximum)	galls. 1·75	galls. 1·33	galls. 1·63	galls. 2·25	galls. 1·50	galls. 2·20 avge.	galls. 1·66	galls. 3·59	galls. 2·04	galls. 1·30 avge.	galls. 2·56	galls. 13	galls. 2·05	galls. 3·12	galls. 17	—	galls. 2·40

consists in protecting the filters from frost by covering them over, of which the Berlin[1] and Albany,[2] N.Y., filter-beds furnish typical examples,

FLOATING SCRAPER UNDER ICE, HAMBURG WATERWORKS.

Fig. 149.



Figs. 150 and 151. The roofing over of filter-beds adds considerably to their cost; but, on the other hand, the protection of the water over the filter-beds from sun and light, whilst maintaining ventilation, prevents the formation of algæ in the water, which, settling on the sand, impede filtration and necessitate more frequent scraping; and also by fulfilling its special purpose in excluding frost, it obviates the difficulties involved by the formation of ice, or the freezing of the filter-bed, in effecting the necessary cleansing, and the danger incurred of inadequate filtration. As some of the filter-beds originally constructed at the Zurich waterworks were open, and others covered, an opportunity was afforded of comparing the two systems, which were operated simultaneously. The purification of the lake-water was accomplished equally well by the two sets of filters, and the covering of the filter-beds involved an extra cost of 27½ per cent.; but the covered filter-beds discharged 14 per cent. more water than the open ones, the amount of cleansing required for the covered filters was only two-thirds of that needed for the open ones, and the cost of filtration by means of the covered filters was 10 per cent. less than by the open ones.[3] Accordingly, the greater cost of construction of the covered filter-beds for Zurich, has been more than compensated for by their greater yield of

[1] "The Filtration of the Müggel Lake Water-Supply, Berlin," H. Gill, *Proc. Inst. C.E.*, vol. cxix. p. 247, plate 7; and "A Report on the Water-Supply System of Berlin," A. E. Silk, Calcutta, 1894, p. 3, and plate 2, figs. 1 and 2.

[2] "The Albany Water Filtration Plant," A. Hazen, *Trans. Am. Soc. C.E.*, 1900, vol. xliii. pp. 244 and 267.

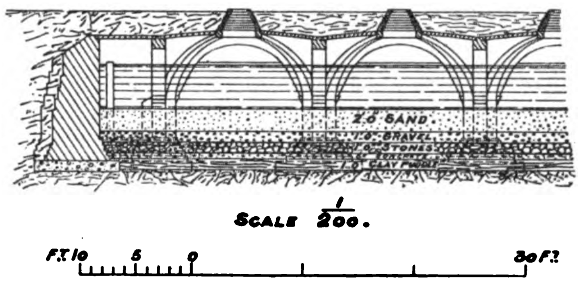
[3] "The Zurich Water-Supply," C. Preller, *Proc. Inst. C.E.*, vol. cxi. pp. 282–283 and 291.

filtered water, and their smaller cost of maintenance, than the open filter-beds; and, consequently, the open filter-beds have been roofed over, and additional filtering area has been provided by the construction of covered filter-beds.

The choice between open and covered filter-beds must be governed by the average winter temperature of the locality; and the relative cost of maintenance of the two systems is affected by the source from which

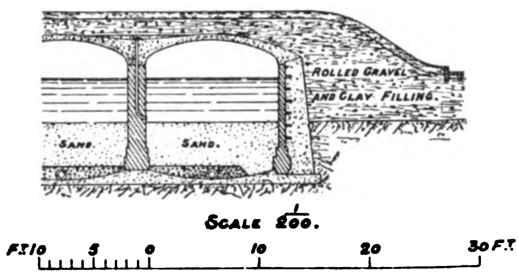
COVERED FILTER-BED, BERLIN WATERWORKS.

Fig. 150.



COVERED FILTER-BED, ALBANY WATERWORKS.

Fig. 151.



the water is derived, upon which depends the prospect of the growth of algæ in open filter-beds, and the consequent clogging of the bed of sand.

Cost of Filtration.—The cost of construction of filter-beds varies between wide limits; and though the covering over of filter-beds involves a considerable additional cost, some covered filter-beds have cost less than certain open filter-beds in a different locality;[1] whilst the conditions of the site, and the size given to the beds, materially affect the cost. The small filter-beds of the Zurich waterworks cost £13,800 per acre for the open filters, and £19,020 per acre for the covered filters, including every item except land, intake, and mains; whilst

[1] *Trans. Canadian Soc. C.E.*, 1899, vol. xiii. part 1, pp. 65 to 67.

the cost of working the filters, per million gallons of water filtered, was 4*s.* 8½*d.* for the open filters, and only 3*s.* 0½*d.* in the case of the covered filters. The London open filter-beds, including all accessory works, have cost from £8100 to £9720 per acre of filtering surface; the Hamburg open filter-beds cost £8100 per acre; the earliest Berlin covered filter-beds cost £12,950, and the most recent, £15,000 per acre; whilst the Warsaw covered filter-beds cost £16,200 per acre of filtering surface. The average cost of filter-beds in Europe has been estimated at £9070 for open filters, and £13,600 for covered filters, per acre of effective sand layer; and the cost of working the covered filters was reckoned at from 16*s.* to £1 4*s.* 4*d.* per million gallons filtered.[1] Allowing, however, for interest and sinking fund for cost of construction, and also for sedimentation before filtration, which is necessary in the case of river-waters, the total cost of supplying filtered water was calculated at from £2 5*s.* to £3 6*s.* per million gallons; whereas at the Zurich waterworks, as the relatively pure lake-water requires no settling basin, and can be passed rapidly through the filters, the total cost of filtering the water is only £1 12*s.* 8*d.* with the covered filter-beds.

In the United States, the most recent Poughkeepsie open filter-bed cost £8540 per acre of filtering area; the earlier Hudson open filter-beds cost £15,200 per acre; the Albany covered filter-beds cost £12,000 per acre, including the settling basin and clear-water tank;[2] and the Ashland, Wis., small covered filter-beds, after making a reduction for special local difficulties, have been estimated as being capable of being constructed, in the form of half-acre filter-beds, for about £14,600 per acre of filtering area. The working of the Albany filter-beds, in their first year, cost 11*s.* 2*d.* per million gallons of water filtered;[3] and the working expenses of a filtration plant has been estimated at between 12*s.* 6*d.* and 17*s.* 6*d.* per million gallons, the cost at Poughkeepsie having averaged nearly 15*s.* in a period of twenty years.[4] The total expense of filtration, including interest and redemption of the capital cost, has been estimated at from £1 15*s.* to £2 5*s.* under normal conditions in the United States.

Regulation of the Rate of Filtration.—As the efficient purification of a water-supply depends upon the water not being allowed to pass unduly rapidly through the filter-beds, it is essential for the head of water to be kept under perfect control, so that the rate of filtration may be kept within the standard limit, notwithstanding the ease with

[1] "De l'Usage des Eaux de Rivière pour les Distributions d'Eau," W. H. Lindley, Congrès International de l'Utilisation des Eaux fluviales, Paris, 1889, pp. 61–62.

[2] *Trans. Am. Soc. C.E.*, 1900, vol. xliii. p. 293.

[3] *Engineering News*, New York, 1900, vol. xliv. p. 88.

[4] "Public Water-Supplies," F. E. Turneaure and H. L. Russell, New York, 1901, p. 463.

which the water passes through a new, or recently scraped filter-bed, in comparison with its slow passage through the filter-bed after it has been in use for some time. Sometimes the apparatus merely indicates when a change in the head of water is required, which can be effected by modifying the influx of water into the filter-bed, or its efflux from it into the clear-water tank; but in other instances the regulation of the flow is accomplished automatically. The head, upon which the rate of filtration depends, is the difference in height between the water-level in the filter-bed and the water-level in the clear-water tank, or special regulating chamber, into which the filtered water is discharged; and it has to be gradually increased in proportion as the filter-bed, in being clogged by sediment and slime, offers an increased resistance to the flow of water through it, and has to be reduced quite low on starting a new or cleaned filter-bed.

One form of automatic regulator of the discharge of water into the clear-water tank, consists of a vertical, telescopic pipe attached to a float, and sliding over the vertical end of the pipe leading into the clear-water tank, placed in a chamber interposed between the filter-beds and the tank.[1] Two rectangular openings near the top of the telescopic pipe, being kept by the float at a constant depth below the water-level in the chamber, admit a constant flow of filtered water into the pipe leading to the tank from the chamber which receives its water from the filter-beds, and has a varying water-level according to the head or the difference between its water-level and that of the filter-beds, required to overcome the resistance offered by the bed of sand to the passage of the flow of water provided for by the regulator, Fig. 152. This head is gradually increased by the falling of the water-level in the chamber, so as to maintain a constant flow through the filter-beds, in spite of the decrease in the permeability of the bed of sand, till it reaches the prescribed limit, when the filter-bed has to be put out of operation for cleaning. On starting the filter, the water in the regulating chamber is level with the water over the filter-beds; and the rate of filtration can be reduced, when required, by diminishing the size of the openings in the telescopic pipe by means of a small annular valve. At Warsaw, the telescopic pipe has a diameter of 2 feet; and the horizontal, rectangular apertures or slits in the telescopic pipe, are 1 foot $3\frac{3}{4}$ inches long and $3\frac{1}{7}$ inches high.

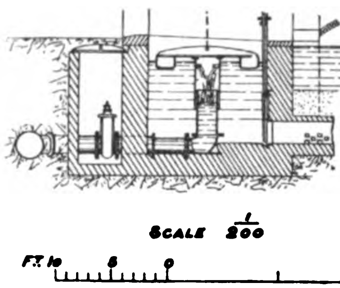
At the filter-beds of the Müggel Lake water-supply for Berlin, the passage of the water through the beds is regulated by a sluice-gate controlling the flow through a sluice connecting the sluice-chamber, into which the filtered water is discharged, with the weir-chamber across

[1] "De l'Usage des Eaux de Rivière pour les Distributions d'Eau," W. H. Lindley, Paris, 1889, pp. 55-56.

the partition wall, the bottom of the sluice being at the level of the floor of the chambers, Fig. 153. In an opening in the opposite wall of the weir-chamber, a thin metal plate is inserted, having a rectangular, submerged orifice, 2·72 feet long by 0·29 feet high, which, with a given head, affords a constant discharge into a third small chamber from which the water is led through a pipe into the clear-water reservoir.[1] When the sluice is closed, the water is at the same level in the sluice-chamber as over the filter-beds; and the filtration is started by slightly raising the sluice-gate till the standard head of water in the weir-chamber over the orifice is reached, so as to provide the requisite constant discharge. The difference in level of the water over the filter-beds and in the sluice-chamber gives the filtering head, for which, on starting the

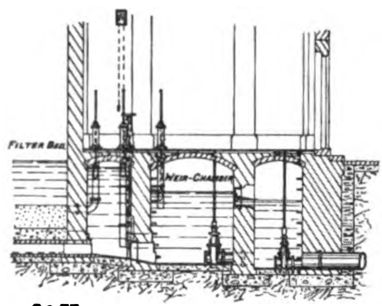
AUTOMATIC REGULATOR OF RATE OF FILTRATION, WARSAW WATERWORKS.

Fig. 152.



REGULATING ARRANGEMENTS OF FILTRATION, MÜGGEL LAKE, BERLIN WATERWORKS.

Fig. 153.



filtration, 3·9 inches suffice to afford the requisite flow; but as the bed of sand becomes clogged, the head is gradually increased by raising the sluice-gate, at first slightly, and then more each day, and thus lowering the water proportionately in the sluice-chamber, whilst maintaining a constant head over the orifice through which the water is discharged from the weir-chamber. The water-levels over the filter-beds, in the sluice-chamber, and in the weir-chamber, are indicated by means of floats in a room above the regulating chambers, which enable the attendant to determine readily each day the amount the sluice-gate requires to be raised.

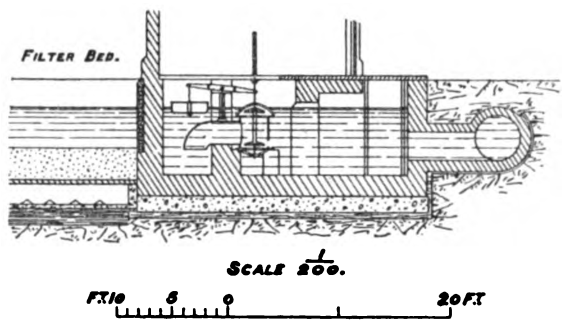
The influx of the water on to the filter-beds of the Hamburg waterworks, is regulated automatically by a float in a chamber adjoining the

[1] *Proc. Inst. C. E.*, vol. cxix., pp. 249 to 251, and plate 7, fig. 5; and "A Report on the Water-Supply System of Berlin," A. E. Silk, plate 2, figs. 3 and 4.

filter-beds and in communication with them, which is attached to one end of a rocking beam, and in rising with the water closes the inlet by lowering a double-flap valve, fastened to the other end of the beam in a second chamber, as soon as the water has reached the requisite height over the filter-beds,[1] Fig. 154.

AUTOMATIC REGULATOR OF INFLUX TO FILTER-BEDS, HAMBURG WATERWORKS.

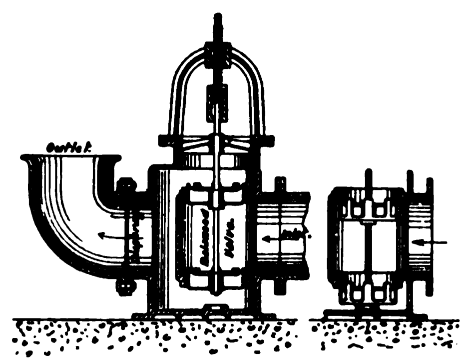
Fig. 154.



A balanced valve for maintaining a definite constant discharge from filter-beds, designed originally for the Tokio waterworks, is shown in Fig. 155. The water discharged from the filter-beds enters the valve by the inlet, and passes into the filtering well through the outlet, which communicates with the clear-water reservoir; and on its way from the valve to the outlet, on issuing from the valve-chamber, it flows through a circular orifice in a movable diaphragm, and preserves a constant discharge, provided the difference in pressure of the water in the valve-chamber and in the clear-water reservoir is kept uniform.[2] The

AUTOMATIC BALANCED VALVE FOR REGULATING EFFLUX FROM FILTER-BEDS, TOKIO WATERWORKS.

Fig. 155.



[1] "Das Wasserwerk der Freien und Hansestadt Hamburg, Filtrationsanlage," F. A. Meyer, Hamburg, 1894, p. 18.

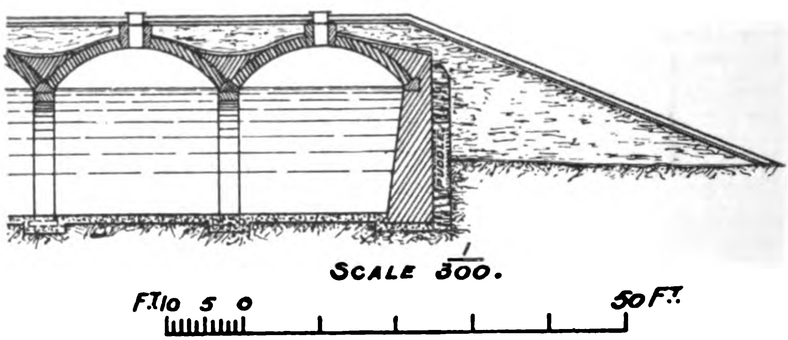
[2] "Regulating the Rate of Filtration through Sand," W. K. Burton, *Proc.*

maintenance of this constant difference in pressure is secured by the action of the water-pressure on the upper and under sides of the piston which works the balanced valve, the upper side being subjected to the pressure of the water in the clear-water reservoir with which the upper part of the valve-chamber is connected, and the under side to the pressure of the water in the valve-chamber. When the discharge from the filter-beds through the orifice increases beyond the regulation amount, the pressure in the valve-chamber is increased, causing the raising of the piston, and the closing of the balanced valve sufficiently to reduce the discharge to the proper quantity. On the contrary, when the discharge from the filter-beds falls below the standard volume, the pressure on the under side of the piston is reduced, causing the fall of the valve and piston by gravity, and the consequent opening of the valve and an increase in the discharge.

Covered Clear-Water Reservoir.—The filtered water is led into a small clear-water reservoir, or tank, of sufficient capacity to provide for variations in the consumption, without the necessity of increasing the rate of filtration beyond that considered consistent with safety. This reservoir should be covered, to protect the water from

CLEAR-WATER TANK.

Fig. 156.



pollution; for filtered water is much more liable to contamination, if brought accidentally in contact with impurities, than the same water is when in its natural condition, Fig. 156. These clear-water reservoirs are similar in construction to the covered service reservoirs which they supply, of which examples are given in Figs. 132, 133, and 134 (p. 212), though considerably smaller in extent.

Efficiency of Slow Sand Filtration in removing Bacteria.—

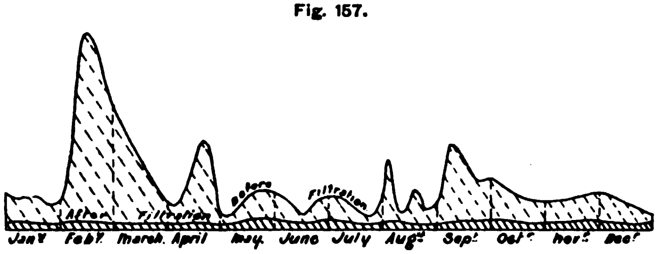
Inst. C.E., vol. cxii. pp. 322–323; and "The Water-Supply of Towns," W. K. Burton, pp. 108 to 110, and plate 29.

Though turbid river-waters experience a very large reduction in the number of bacteria contained in them, amounting in some cases to from 50 to over 90 per cent., by the subsidence of the particles in suspension during rest in settling tanks, this preliminary process of purification is mainly undertaken with the object of facilitating the subsequent process of filtration, by removing the silt which would rapidly clog the layer of sand; and reliance is chiefly placed upon slow filtration through sand for providing an adequately pure water for domestic purposes. The London water companies have succeeded, by means of subsidence and slow filtration, in reducing the considerable number of bacteria found in Thames water by between $98\frac{2}{3}$ per cent. and $99\frac{6}{7}$ per cent., thereby generally bringing the number remaining in a cubic centimetre (0·061 cubic inch) within the prescribed limit of 100 after filtration, which is also the standard of purity aimed at in Germany; but at times this degree of purity is not attained, owing more to a too rapid, or very variable rate of filtration, or defects in the filter-beds, than to a greater number of bacteria in the unfiltered water, which are found during the months when rivers are subject to floods.

Though the percentage of decrease in bacteria by filtration measures the efficiency of the action of a filter-bed, the real test of the hygienic purity of a potable water is the small number of bacteria remaining in the filtered water, and also their nature. Thus the reduction in the number of bacteria in the Zurich Lake water by filtration, in the course

DIAGRAM SHOWING REDUCTION OF BACTERIA BY ZURICH FILTER-BEDS IN 1888.

Fig. 157.



of four years, averaged 90 per cent., notably less than in the case of several river-waters; but this smaller efficiency was due to the small number of bacteria originally present in the lake-water, averaging only 195 per cubic centimetre, and the rapid rate of filtration which could, consequently, be adopted with safety, by which, nevertheless, the bacteria were reduced to an average of only 20 per cubic centimetre in the filtered water.[1] The reduction effected in the number of bacteria

[1] *Proc. Inst. C.E.*, vol. cxi. pp. 287 and 293.

in the lake-water by the Zurich filter-beds, throughout the year 1888, is very clearly indicated graphically by Fig. 157. Filtration, however, though greatly reducing the liability of the presence of pathogenic bacteria in the filtered water, cannot secure their entire removal when present in the water previously; and, accordingly, the filtered river-water supplied to Calcutta, though seldom containing more than 50 bacteria per cubic centimetre, is not free from the objectionable bacillus coli communis, which commonly exists in most waters flowing off the surface of the ground in India.[1] This circumstance proves the great importance, notwithstanding the great purification accomplished by filtration, of obtaining, if possible, a source of supply free from the pollution indicated by the presence of pathogenic bacteria.

Rapid Filtration.—Another method of filtration, largely used in America, especially for turbid river-waters, consists in the addition of a coagulant to the water previously to filtration, for aggregating the particles in suspension, followed by the rapid passage of the water through a layer of coarse sand, or crushed quartz, from 2 to 4 feet thick, contained in a tank of wood, iron, or steel, and supported on

MECHANICAL FILTER.

Fig. 158.—Sectional Elevation.

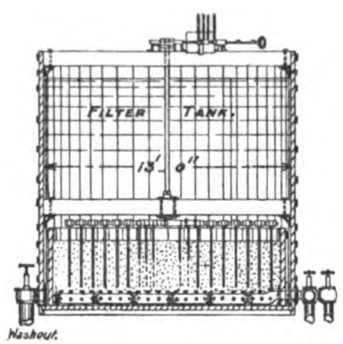
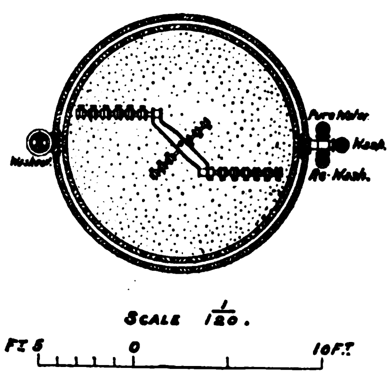


Fig. 159.—Plan showing Scrapers.



a system of strainers, which allow of the discharge of the filtered water without the washing out of the sand.[2] Owing to its introduction in the United States, it has been termed the American system, in contra-distinction to the English or European system of slow filtration; and these filters have been called mechanical filters, merely on account of the revolving mechanical contrivances resorted to in them for raking and agitating the sand, Figs. 158 and 159, in the frequent process of

[1] *Proc. Inst. C.E.*, vol. cxlviii. pp. 330 and 336.

[2] *Engineering News*, New York, vol. xxxv. p. 330.

cleaning necessitated by the deposit of the flocculent precipitate formed by the coagulant, and for the regular supply of the coagulant to the water. In open filters, the requisite head is obtained by the depth of water introduced over the bed of sand, or by producing a vacuum in the effluent pipes at the bottom; and in closed filters, the rapid filtration is effected by water under pressure. Heads of from 10 to 12 feet are often employed to avoid very frequent washing of the sand; and this washing, which occupies between 15 and 20 minutes, and has to be repeated at intervals not exceeding 24 hours, is effected by forcing up water through the layer of sand from below during its agitation by the revolving rakes or by compressed air.

Action of Coagulant used for Rapid Filtration.—The coagulant generally employed as the most suitable is aluminium sulphate; but ordinary alum, a double sulphate of potassium and aluminium, is often used, though costing quite as much, and having to be used in larger quantities than aluminium sulphate. Generally, half a grain to two grains of alum per gallon, or a somewhat smaller amount of aluminium sulphate, is first added to the water to effect the aggregation and precipitation of the suspended matters; but the proportion of coagulant should be carefully adjusted by an experienced chemist to the varying composition of most river-waters, for too small a quantity reduces the efficiency of the purification, and too large an amount impregnates the filtered water with alum, which is liable to be detrimental to health, and is objectionable for washing and some industrial purposes, and during periods of low alkalinity in flood-time leaves free sulphuric acid in the water, which acts on the iron pipes. With turbid, coloured waters, a larger quantity of coagulant, amounting to 5 or 6 grains per gallon, is required, together with the addition of lime, to effect their clarification and decoloration. Aluminium sulphate, when introduced into a river-water, is decomposed, the sulphuric acid combining with the calcium, magnesium, and sodium, salts in the water, and the aluminium hydrate, which is set free, forming flocculent masses with the fine, suspended matter in the water, which partially deposit when a settling tank is provided; and the lighter portions remaining in suspension after admission to the filter, eventually form a coating on the surface of the sand, which serves, like the gelatinous film in the slow sand filter, to arrest the finer particles in the water, and also a large proportion of the bacteria. The water, after treatment with the coagulant, is sometimes led into an independent settling basin; whilst in certain filters, there is a settling tank at the bottom of the filter, into which the water is first led, and then passes up through a central pipe to the top of the filter, and thence down through the layer of sand to the collecting pipes placed between the filter-bed and the settling tank.

Rate of Filtration through Mechanical Filters.—The rate of discharge through these filters ranges between 50 million and 250 million gallons of water per acre per day; but it is generally comprised between 100 million and 125 million gallons per acre in a day. This very rapid flow of water and precipitate on to the filter, if not preceded by sedimentation, not merely clogs the filter quickly, but also carries the flocculent precipitate into the sand for some depth below the surface, necessitating a very thorough cleansing of the sand at short intervals; and this inconvenience can best be partially provided against by reducing the amount of suspended matter in the water, before filtration, by a prolonged period of subsidence, involving a larger area of settling tanks than is generally furnished.

Cost of System of Rapid Filtration.—The cost of filtration with mechanical filters necessarily varies with the condition of the water operated upon. A comparative estimate of the cost of working mechanical filters and ordinary sand filter-beds in the United States, gives £1 2*s.* 7*d.* per million gallons filtered for the former, and £1 1*s.* 11*d.* per million gallons for the latter;[1] whilst the working cost of filtering the waters of the Schuylkill River and Delaware River at Philadelphia, has been reckoned at £1 4*s.* and £1, respectively, per million gallons with mechanical filters, and 18*s.* and 15*s.*, respectively, per million gallons with ordinary sand filter-beds.[2] The total cost per million gallons filtered by mechanical filters, including interest and depreciation, has been estimated at from about £1 17*s.* 9*d.* up to £2 13*s.* 11*d.* at different places in the United States.

Efficiency of Rapid Filtration in the Removal of Bacteria.—With due care, and an exact proportioning of the coagulant to the varying condition of the river-water, it has been found possible to obtain a large reduction in the number of bacteria by rapid filtration, ranging in some instances between about 86 and 99 per cent.[3] Though, however, the efficiency of these mechanical filters has sometimes approximated to that of slow sand filtration, the system has not as yet borne the same long test of practical experience for over half a century, and requires much more careful watching and manipulation to approach as satisfactory a result.

Remarks on Filtration.—All river-waters used for supplying towns should be subjected to filtration; and a high death-rate has been

[1] "The Purification of Public Water-Supplies," J. W. Hill, New York, 1898, pp. 192–193.

[2] "Public Water-Supplies," F. E. Turneaure and H. L. Russell, New York, 1901, p. 477.

[3] "The Purification of Public Water-Supplies," J. W. Hill, pp. 195, 199, and 202; and "Public Water-Supplies," F. E. Turneaure and H. L. Russell, pp. 471, 472, and 473.

found to accompany the neglect of this essential precaution. Generally, even in the case of the much purer waters collected from uninhabited gathering grounds, filtration serves as a valuable precautionary measure against chance pollution; and in certain cases, as with the Vyrnwy water-supply, it also acts as a preventive to the obstruction of the flow by organic growths in the pipes conveying the supply.

With reference to the relative values of the two systems of filtration, much importance must not be attached to the advantage possessed by mechanical filters of taking up much less space, for no large city in the United States, in the neighbourhood of which land would be specially valuable, has adopted rapid filtration; but where an adequate area is not obtainable, in a suitable site, for slow sand filtration, the small space required for a mechanical filter would favour its adoption. The use of a coagulant with the mechanical filter renders the system specially adapted for the purification of very turbid and discoloured waters, and also for the removal of iron and the fine precipitate formed in the process of softening water; and the system possesses advantages where the waterworks are on a small scale, and, consequently, would necessitate the construction of very small filter-beds for slow filtration at an enhanced cost. Slow filtration, however, is a more reliable, and more easily regulated system of purification; and it can be rendered more applicable to very turbid waters by the preliminary use of a coagulant. Moreover, for the proper treatment of very turbid waters, both systems need an ample area of settling basins.

VARIOUS METHODS OF PURIFYING WATER.

Though subsidence and filtration are the chief means relied upon for the purification of water for domestic purposes, various methods have been resorted to in special cases, in conjunction with filtration, for the removal of particular impurities contained largely in certain waters, which filtration alone is incapable of adequately effecting.

Aeration of Water.—Where water is deficient in oxygen, the introduction of oxygen by aeration is valuable in assisting the purification of the water from organic pollution. Moreover, when water contains iron in solution as ferrous carbonate, from having passed through gravel and other strata containing iron which is acted upon by the carbonic acid contained in the water, the addition of oxygen to the water causes the precipitation of the iron as ferric oxide, forming a flocculent sediment which is readily removed by filtration. This aeration is effected by exposing the water in open channels, by causing it to fall down a series of steps, by discharging it as a fountain into an open reservoir, by dispersing it in a series of small jets, or by making it percolate between large pieces of coke or blocks of stone.

The allotropic form of oxygen, known as ozone, which is commonly produced by electrical discharges, possesses great powers of oxidation, especially in the nascent state; and it has, accordingly, been occasionally employed for the purification of water, which it effects by oxidizing the organic impurities and destroying bacteria. Its action is the same as that of aeration, but much more powerful, owing to its being pure oxygen in a concentrated form; but as suspended matters impair its efficiency, turbid water should be cleared to some extent by filtration before the ozone is applied.[1] The employment of ozone for the purification of the water-supply of Wiesbaden, has been accomplished at a total cost of 1*d.* per thousand gallons, with very satisfactory results.

Treatment of Water with Chemicals, and Filtration, for Removal of Iron.—Iron is a frequent ingredient of underground waters drawn from springs or deep wells, tapping the flow in certain strata; and as its amount in water intended for domestic use should not exceed one-tenth of a grain per gallon, the removal of an excess of iron is often necessary before waters from such sources become suitable for supply. The removal of iron by aeration and filtration has been referred to above; but where the proportion of iron in the water is somewhat large, it is expedient to use chemicals, as well as aeration, for precipitating the iron. Thus by the addition of lime-water to water containing iron in solution as carbonate or sulphate, ferrous oxide is formed, which is rapidly converted into ferric oxide by admitting air under pressure into the water, which is then passed into a settling tank where a considerable portion of the ferric oxide subsides; and in the United States, the finer precipitate is removed by passing the water through a mechanical filter, from which the deposit of ferric oxide is readily washed out of the layer of sand at short intervals by aid of the revolving agitators,[2] Figs. 160 and 161. In some cases, in addition to the lime, ferric chloride or ferric sulphate, and in other instances aluminium sulphate has been used for facilitating the precipitation of the ferric oxide, and for aggregating its particles so as to render its subsidence and separation from the water by filtration easier. By these processes, the reduction of the iron in the water has reached from 90 per cent. up to entire removal in some instances.[3]

Purification of Turbid Waters with Iron.—Turbid river-waters are deprived of colour and clarified by contact with iron. This result was at first effected by passing the water through spongy iron

[1] *Engineering News*, New York, vol. xlii. p. 250.
[2] *Ibid.*, vol. xxxv. p. 366.
[3] "The Purification of Public Water-Supplies," J. W. Hill, pp. 205 to 207.

REMOVAL OF IRON BY SOFTENING, AND MECHANICAL FILTRATION.

Fig. 160.—Sectional Elevation.

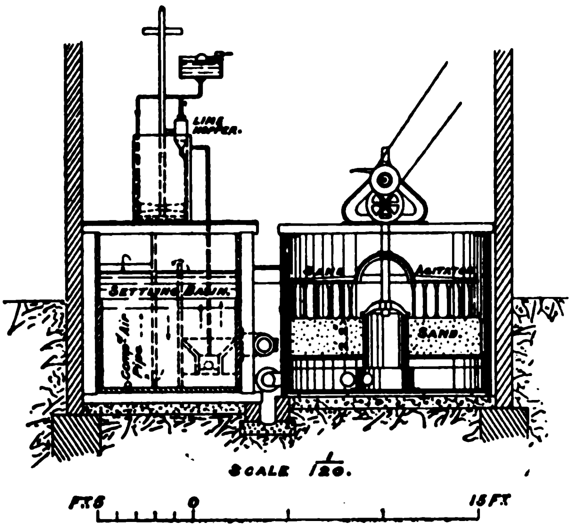
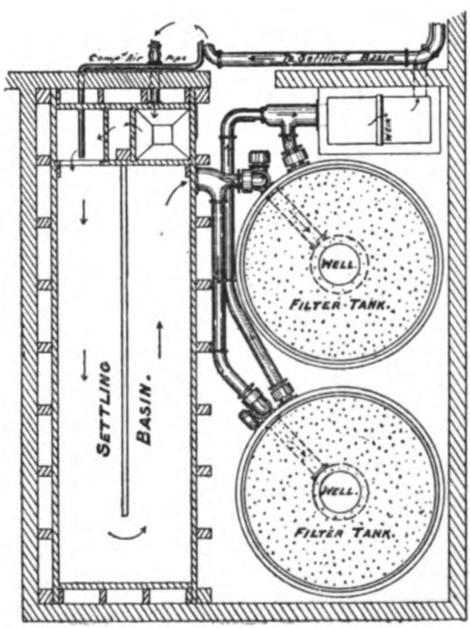


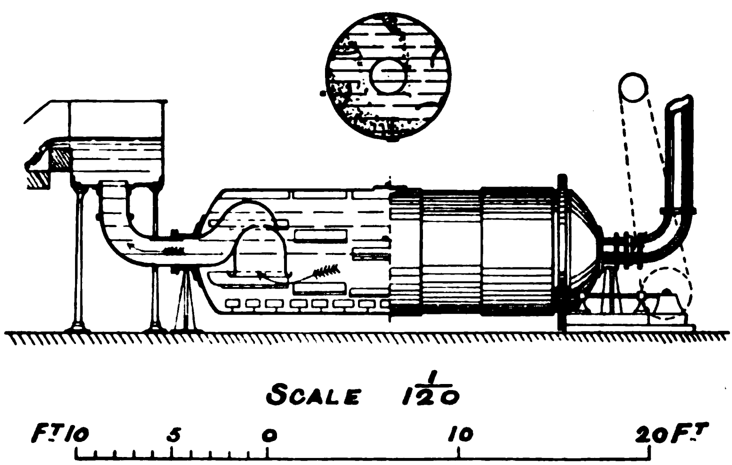
Fig. 161.—Plan.



filters; but these filters clogged rapidly, and expended a considerable amount of iron in being cleaned. To reduce the quantity of iron to a minimum, revolving purifiers have been adopted, in which cast-iron borings are placed; and by means of a set of ledges or shelves attached along the circumference of an iron cylindrical vessel laid horizontally and rotated, the particles of iron are successively raised and dropped through the water in its passage through the cylinder, so that all the water in turn comes in contact with the iron for an average period of 3½ minutes, which is increased to 8 or 10 minutes for very impure waters,[1] Fig. 162. A minute amount of the iron, of one-tenth to one-

TREATMENT WITH IRON IN REVOLVING PURIFIERS.

Fig. 162.



fifth of a grain per gallon, is dissolved by the water as ferrous carbonate, which is almost at once converted into ferrous oxide, and then, after a short period of exposure to the air, becomes ferric oxide, which is readily removed by subsidence and filtration.

By this treatment, in which the ferric oxide is practically used as a coagulant, not only is the colour and turbidity of the water removed, but, in common with all processes of purification by precipitation, the bacteria are to a very great extent carried down with the precipitate.

Methods of rendering Suspected Waters safe for Drinking.—The safest way of rendering water, which may have been subjected

[1] "The Purification of Water by Means of Iron," W. Anderson, *Proc. Inst. C.E.*, vol. lxxxi. p. 280, and plate 12; and "The Purification of Water by Metallic Iron," G. H. Ogston, *Ibid.*, pp. 287–288.

to pollution, innocuous for drinking, is by boiling it. When it is not practicable to boil the water, it can be rapidly sterilized by $4\frac{1}{5}$ grains of free bromine per gallon of water, dissolved in a solution of potassium bromide; and then the excess of bromine is removed by a corresponding amount of ammonia, so as to render the water palatable.[1] Chlorine in a sodium chloride solution, with the addition of a sodium sulphite solution to remove the excess of chlorine, has been proposed as a more efficient method of sterilizing impure water.[2] The danger of contracting enteric fever from having to drink polluted water, may be avoided by adding 15 grains of sodium bisulphate to a pint of water, which is stated to destroy the typhoid bacillus.[3] Calcium chloride and hydrogen peroxide have also proved capable of sterilizing water.

Softening Water.—Water is called *hard* when it contains a considerable quantity of calcium or magnesium salts in solution, mostly as bicarbonates and sulphates, collected in its passage through certain strata. Hard water is not injurious to health; but it is not suitable for general domestic and manufacturing purposes, owing to its curdling soap, and encrusting kettles, boilers, and pipes with a deposit on being boiled. The hardness of water is usually stated as so many degrees, one degree corresponding to one grain of calcium carbonate per gallon; and it is commonly measured by the amount of a definite solution of soap in proof spirit, required to be added to a given volume of the water under examination to produce a lather. When this hardness consists of calcium and magnesium bicarbonates, which can be precipitated as carbonates by boiling (the latter only partially so, owing to the relative solubility of magnesium carbonate), it is called *temporary* hardness; and so far as it consists of calcium and magnesium salts remaining in solution after boiling, mainly sulphates, and also possibly chlorides and nitrates, it is called *permanent* hardness. The relative amounts of temporary and permanent hardness in any water can be readily found, after having ascertained the total hardness, by boiling the water, and then determining the remaining permanent hardness by means of the soap solution. Some waters, such as those from deep wells in the Chalk or Limestone formations, have as much as about 24° of hardness; and waters are called hard when the hardness exceeds 5°.

Temporary hardness can be removed by boiling, the excess of carbonic acid being driven off, and calcium carbonate, and to some extent magnesium carbonate, being deposited, the reaction being

Calcium bicarbonate. Carbonic acid. Calcium carbonate.

$CaO_2(CO_2) = CO_2 + CaOCO_2$; but as a process for softening water, boiling is too expensive. The Clark process consists in adding

[1] *Zeitschrift für Hygiene*, vol. xxxiii. p. 53. [2] *Ibid.*, vol. xxxix. p. 379.
[3] "Water and its Purification," S. Rideal, 1902, p. 167.

lime-water for converting the soluble calcium bicarbonate into the insoluble calcium carbonate, this reaction being represented by $CaO2(CO_2)$ $+ CaOH_2O = 2CaOCO_2 + H_2O$ (water); and by the same means, the more soluble magnesium carbonate is to some extent precipitated with the calcium carbonate, as indicated by the equation, $MgO2(CO_2) + CaOH_2O = MgCO_2 + CaCO_2 + H_2O$, when originally present in the water as magnesium bicarbonate. The temporary hardness, however, due to magnesium bicarbonate, is more effectively removed by converting the somewhat soluble magnesium carbonate, obtained as above, into the fairly insoluble magnesium hydrate, which is thrown down by an excess of lime-water, as represented by $MgCO_3 + CaOH_2O = MgOH_2O$ $+ CaCO_3$, together with a further quantity of calcium carbonate.

Permanent hardness is not affected by the above softening processes; and in order to remove calcium and magnesium sulphates from the water, sodium hydrate has to be added, which effects the transposition of the sodium and the calcium or magnesium, with the formation of their hydrates and sodium sulphate, which does not increase the hardness of the water. The calcium hydrate thus formed enters at once into combination with the calcium bicarbonate in solution in the water to form calcium carbonate, which is precipitated, the two reactions, taking place in rapid succession, being represented by $CaOSO_3 + 2NaOH = CaOH_2O + Na_2SO_4$, and $CaOH_2O + CaO2(CO_2) = 2CaCO_3 + H_2O$; whereas the magnesium hydrate is precipitated directly without change, the reaction being $MgOSO_3 + 2NaOH = MgOH_2O + Na_2SO_4$. When great permanent hardness of the water is due to the presence of a considerable quantity of calcium sulphate, sodium carbonate should be used instead of sodium hydrate, resulting directly in the transposition of the two bases, so that calcium carbonate and sodium sulphate are formed, as expressed by the equation, $CaSO_4 + Na_2CO_3 = CaCO_3 + Na_2SO_4$.

The precipitate of calcium carbonate and magnesium hydrate formed by the above softening processes, is granular and very fine, and in settling greatly increases the organic and bacterial purity of the water; but its subsidence is much slower, and its separation from the water more difficult, than in the case of the flocculent precipitate formed by coagulants. In the Maignen softening process, which hitherto has only been applied to private water-supplies, the precipitation is accelerated by an automatic arrangement for thoroughly mixing the precipitant with the hard water; and the subsidence of the precipitate is expedited by the addition of alum to the lime and sodium hydrate or carbonate, employed as a very fine powder for the softening, and by arresting the precipitate by a series of filtering frames and shelves. The sodium aluminate formed on the dissolving of the alum in the water, hastens the precipitation of the calcium salts, and the removal of the precipitate from the water by settlement.

Separation of the Precipitate from the Softened Water.— In the original method of softening water by the addition of lime-water, the resulting precipitate was left to deposit in large settling tanks, till the softened water became clear enough to be drawn off, a system of separation requiring a considerable space for storing the water during subsidence, and rather a long time. Different systems have, accordingly, been devised for reducing the extent of the settling tanks, and for effecting the separation more rapidly.

A simple method of accelerating the subsidence of the fine precipitate formed in the process of softening, and, consequently, reducing the area of land required, consists in stirring up some of the precipitate deposited during a previous softening at the bottom of the settling tank, so as to put it in suspension in a fresh amount of hard water to which lime-water has just been added; for the old precipitate having coalesced by subsidence into larger particles, deposits a second time much more rapidly than at first, and carries down with it the freshly formed fine precipitate.[1]

Another plan by which area and time are economized, consists in making the hard water, after treatment with lime-water and soda also if necessary, flow upwards from the bottom of two or more high tanks successively, in a zigzag course, past a series of projecting baffle plates and shelves, which ensure the complete mixing of the precipitant with the water, and upon which the precipitate falls through the ascending water as it is formed, and from which it is readily removed from time to time.[2] By this arrangement, the water is gradually freed from the precipitate in its passage through the tanks, and finally flows out as a clear, softened water.

A very efficient method of separating the precipitate from the softened water consists in passing the turbid water through filter presses, or by a later improvement through circular filter disks, across which sheets of stout, twilled, cotton cloth are stretched, forming the filtering medium. The advantage of the latter arrangement, which has been adopted on a large scale for softening the water-supply of Southampton, derived from deep wells in the chalk, is that the deposit of calcium carbonate which accumulates on the cloth in the process of filtering, can be removed periodically by jets of water in the form of spray, instead of the filter presses having to be disengaged, and the frames taken out by hand, to

[1] "Water Softening and Purification," L. Archbutt, *Proc. Inst. Mech. E.*, 1898, p. 414.

[2] "Water Softening and Filtering Apparatus at Penarth," W. F. Pullen, *Proc. Inst. C.E.*, vol. xcvii. pp. 362–363, and plate 8, fig. 7.

be cleansed,[1] Figs. 163, 164, and 165. The merit of the system is that, provided the filtering cloth is removed as soon as necessary, the

SEPARATION OF PRECIPITATE FROM SOFTENED WATER, SOUTHAMPTON WATERWORKS.

Fig. 163.—Filter Disk: Section. **Fig. 164.—Filter Disks: Elevation.**

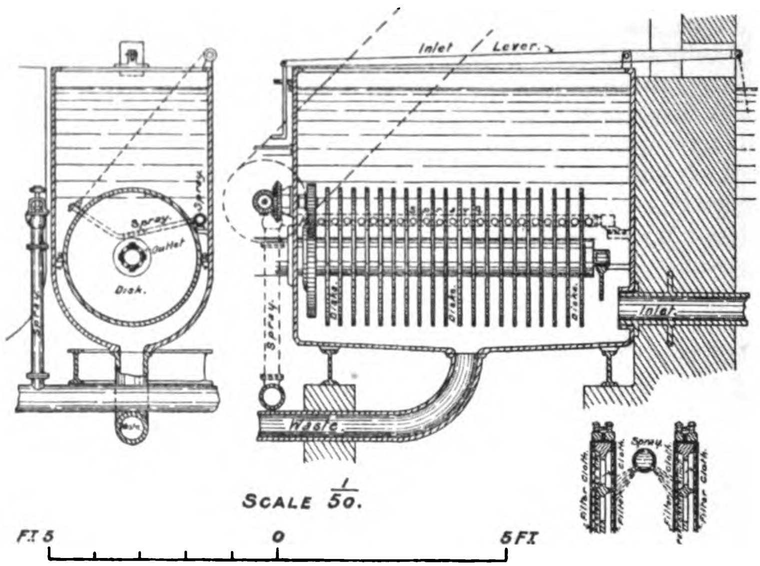


Fig. 165.—Section of Disks with Spray for Cleaning.

rate of flow is limited to the filtering capacity of the medium, and the clarification of the softened water maintained to the degree of the efficiency of this medium; whereas in processes involving subsidence, the rate of flow is liable to be varied with the demand; whilst the period of subsidence should depend on the variable condition of the water.

Cost of Softening Water, and Reduction in Hardness.—The cost of softening the water-supply of Southampton, averaging $2\frac{1}{4}$ million gallons of water per day, is about £850 a year, or slightly under one farthing per 1000 gallons; and the water is reduced in hardness from 18° to 6°. The same system of softening in some other places has cost from $\frac{1}{3}d.$ to $\frac{2}{3}d.$ per 1000 gallons, with a reduction of between 12° and $16\frac{1}{2}$° of hardness; whilst in the very exceptional case of

[1] "The Southampton Waterworks and Softening Plant," W. Matthews, *Proc. Inst. C.E.*, vol. cviii. pp. 287–288 and 290 to 296, and plate 9, figs. 2 to 7.

Wellingborough, the hardness has been reduced from 37° to 13°, at a cost of $\frac{4}{5}d$. per 1000 gallons.[1] The cost of the earlier process, in which subsidence is relied upon for separating the precipitate, has been in a few cases about $\frac{3}{10}d$. to $\frac{1}{3}d$. per 1000 gallons, with a reduction of between 10° and $14\frac{1}{4}^\circ$ of hardness; whereas at Caterham, the hardness has been reduced from 24° down to $3\frac{1}{2}^\circ$, at a stated cost of 1*d*. per 1000 gallons.

Concluding Remarks on Water Purification.—The supply of a wholesome water for domestic uses is of such vital importance for the health and welfare of the community at large, that every effort should be made, in the first place to obtain as pure a source of supply as may be available, and, secondly, to give the water supplied the highest degree of purity reasonably attainable. Subsidence and slow filtration through sand have been proved by long experience to provide a very reliable method of purification, even for river-waters, if conducted with care, and proper precautions are taken; and no water taken from rivers flowing through inhabited districts, or exposed in lakes, springs, or wells to sources of pollution, should be delivered for consumption without being subjected to thorough filtration.

Hard water, though not injurious to health, is unsuited for various domestic purposes; and it seems reasonable that consumers should be furnished with water fitted for cooking and washing, and not liable to encrust kettles, boilers, and hot-water pipes, especially as, not only can softening be applied to hard waters at a moderate expense, but the process at the same time may materially improve the bacterial purity of the water.

Filtration has, indeed, been very generally resorted to in Great Britain and the Continent for the purification of waters liable to contamination; but river-waters supplying some important cities in the United States have not been thus purified; so that whereas Berlin, being supplied by filtered water derived from the rivers Havel and Spree, has an annual typhoid death-rate of only 7 per 100,000 of the population, Washington, Louisville, and Pittsburg, supplied with unfiltered water from the rivers Potomac, Ohio, and Allegheny, respectively, are stated to have typhoid death-rates of 71, 74, and 84 per 100,000 of the population per annum,[2] clearly demonstrating the great importance of filtering river-waters subject to pollution. On the other hand, in comparatively few instances has any attempt been made to soften the hard waters which so often form the sources of supply, before distributing the water to the population of large towns, thereby entailing a considerable

[1] "The Southampton Waterworks and Softening Plant," W. Matthews, *Proc. Inst. C.E.*, vol. cviii. pp. 300–301.

[2] "Water Filtration Works," J. H. Fuertes, New York, 1901, p. 2.

amount of avoidable inconvenience to the communities thus supplied with hard water. It is to be hoped that the time may not be far distant, when water companies generally will manifest the same sense of their duty to the public as has been evinced by Southampton and a few other places, and will deem it essential, not merely to supply water free from pathogenic bacteria, but also water of adequate softness to meet the reasonable requirements of domestic supply.

CHAPTER XI.

DISTRIBUTION OF WATER-SUPPLY.

Uses of Water—Consumption of Water per Head per Day, causes of variation —Consumption of Water in various Towns in the United Kingdom, differences—Consumption of Water in Continental Towns, notable differences, normal requirements—Consumption of Water in the United States, great waste, largely increased demands, reasonable consumption—Fluctuations in the Demand for Water, in the day, in the year—Distributing Mains and Pipes, arrangements, methods of distribution, network of pipes, advantages, precautions for service-pipes—Provisions necessary for Mains and Pipes, depth below ground, protection from rust, incrustations and their removal by brushes and scrapers—Accessories requisite for Water-Mains, air-valves, outlet valves, reflux valves—Dual Supplies of Water for different Purposes, economical advantages, objections for domestic supply, merits for municipal purposes, instances for large towns, for extinguishing fires in cities in the United States—Watering Posts and Fire-Hydrants, position of mains, situation of watering posts, position of hydrants, valves for hydrants—Connecting Distributing Mains with Principal Main under Pressure, objections to ordinary method, machine for drilling and tapping large water-main, method of forming connection under pressure—Service-Pipes connected with Distributing Mains, arrangements for connection under pressure, contrivances for drilling and tapping distributing main under pressure, means of effecting connection—Intermittent Service, arrangement, inconveniences and disadvantages of system for the public, merits and defects of system for water companies—Constant Service, advantages and objections, great sanitary value of system—House Fittings for Constant Supply, provisions against leakage, taps, flush out cisterns—Prevention of Waste of Water, importance with constant service, methods of reducing waste—Waste-Water Meter, introduction in Liverpool to stop waste, description of working of meter—Detection of Waste by the Waste-Water Meter System, mode of operations, results in Liverpool—The Venturi Meter, principle, description of meter, mode of recording flow through it, form regulated for flow—Water-Meters, for measuring house supplies, action of positive, and of inferential meters, examples of positive and inferential meters, relative merits—Reduction in Waste by the Sale of Water by Meter, requirements for meters, equity of system, objections raised and remedy, water-rate advantageous for water companies, importance of general adoption of meter system.

THE volume of water which has to be provided, and distributed to a town or district, depends upon the population to be supplied, and the amount used per head in a day, together with the supply needed for manufactories and industrial purposes, and also for municipal requirements, such as watering the streets and roads, flushing the sewers, and in some cases for drinking fountains, public baths, and wash-houses. Water for the extinction of fires, only required at long intervals apart, forms a very small part of the consumption if reckoned in the daily supply; but it absorbs a large volume when in actual demand, and has to be allowed for when determining the requisite storage to be provided.

Consumption of Water per Head per Day.—Though the water supplied to towns is used for various other purposes than domestic requirements, as indicated above, it is customary to reckon the average yearly consumption in gallons per head of the population in a day. This rough method of denoting the supply delivered, though convenient for comparison, involves great variations in the apparent consumption per head in different cities and towns, according to the amount of water used for trade purposes as compared with domestic consumption, the climate of the locality, and the views of the municipality as to the employment of water for public purposes and hygienic considerations. Moreover, the domestic consumption varies considerably in different places with the climate, and the general habits of the population in the use of water for baths, water-closets, stables, and the watering of gardens and lawns. Provisions also against waste, by a system for detecting leakage in mains, careful supervision of house fittings, and periodical inspection to guard against useless outflows, assist greatly in keeping the supply within moderate limits.

Consumption of Water in various Towns of the United Kingdom.—In Great Britain, there are some residential towns of moderate size where the average daily supply conforms approximately to the consumption which may be regarded as necessary for almost purely domestic purposes, such as Leamington, Bath, and Scarborough, where the supply amounts to 19, 20, and 23 gallons per head per day; whilst at Leicester and Nottingham, the daily consumption only reaches 16 and 17 gallons per head; and for the poor colliery populations of Wigan and St. Helens, it is 17 and 20 gallons respectively. At fashionable seaside resorts, with a rich population and a warm southern aspect, the consumption is fairly large, as, for instance, Torquay and Brighton, using 30 and 33 gallons per head per day; and the mainly residential capitals, London, Edinburgh, and Dublin consume 35, 38, and 36 gallons respectively. On the other hand, large commercial and manufacturing centres requiring considerable quantities of water for trade purposes, as, for example, Birmingham, Manchester, Liverpool, Belfast, and Bradford have all their various wants supplied by $24\frac{1}{2}$, $29\frac{1}{2}$, $31\frac{3}{4}$, 35, and 40 gallons, respectively, per head per day.[1] In London, 27 gallons constitute the domestic supply, and 8 gallons are used for trade and public purposes; in Birmingham, the domestic, and trade and public supplies are $15\frac{1}{4}$ and $9\frac{1}{4}$ gallons, in Manchester, $16\frac{1}{2}$ and 13 gallons, and in Dublin, 29 and 7 gallons;[2] whilst in Bradford, the domestic and trade supplies are each about 20 gallons; and in Liverpool, $18\frac{1}{2}$ gallons per head per day are employed for domestic purposes, about $3\frac{3}{5}$ gallons for municipal and other public requirements, and $9\frac{2}{3}$ gallons for trade supplies.[3] Glasgow draws the large daily supply of 54 gallons per head from Loch Katrine; but of this amount, about 20 gallons are devoted to trade and public purposes. The volume of water spent in Liverpool for extinguishing fires, though only making an addition of $\frac{1}{80}$ of a gallon to the average supply per head per day, sometimes amounts to over 16 million gallons during a large fire.

Consumption of Water in Continental Towns.—In Paris, the consumption of nearly 49 gallons per head per day consists of 18 gallons used for domestic supply, and $30\frac{4}{5}$ gallons for trade and public purposes; but several years ago the total supplies to Marseilles and Carcasonne had reached 99 and 88 gallons, respectively, per head per day, far above other towns in France,[4] and indicating great waste, for which remedial measures have been taken in Marseilles. Considerable variety is also exhibited in the supplies furnished to the large towns of Germany, as indicated by the following amounts per head per day, namely, Berlin $17\frac{1}{5}$ gallons, Hamburg $46\frac{1}{5}$ gallons, Breslau $16\frac{3}{4}$ gallons, Munich $20\frac{1}{2}$ gallons, Dresden $17\frac{3}{5}$ gallons, Cologne $34\frac{4}{5}$ gallons, Leipzig 23 gallons, and Frankfort on the Main 27 gallons.[5] The average daily supply is only 15 gallons per head to Vienna, consisting of 9 gallons for domestic supply and 6 gallons for other purposes; whereas it attains 100 gallons in Rome. At Melbourne, the total supply delivered is 61 gallons per head per day; whilst at Calcutta, the domestic supply of filtered water is 24 gallons. Even in the principal towns of Switzerland, with comparatively moderate populations, the consumption of water exhibits great differences, ranging between 28 gallons and 100 gallons per head per day, amounting, for example, to 31 gallons at Basle, 48 gallons at Berne, 50 gallons at Geneva and Lucerne, 51 gallons at Zurich, 59 gallons at Lausanne, and 100 gallons at Neuchâtel

[1] "Rate per Head per Day of Supply in Different Towns," J. Watson, Engineering Conference, 1899, *Proc. Inst. C.E.*, vol. cxxxviii. pp. 465–466.

[2] "Presidential Address," James Mansergh, *Proc. Inst. C.E.*, vol. cxliii. pp. 40 to 79.

[3] "Liverpool Public Health Congress, 1903, Handbook," p. 220.

[4] "Distributions d'Eau," G. Bechmann, Paris, p. 61.

[5] *Proc. Inst. C.E.*, vol. cvii. pp. 230 to 233.

and Vevey-Montreux. These relatively large supplies are doubtless due to the facility with which water is obtained from the lakes and mountain streams in Switzerland, and also to the industrial purposes to which water-power is applied there. Thus at Zurich, $33\frac{1}{2}$ gallons per head per day are delivered for domestic supply, $4\frac{1}{2}$ gallons are devoted to public uses, and 13 gallons are applied to industrial purposes.[1]

In northern Europe, under ordinary conditions, 15 to 20 gallons of water per head per day may be regarded as adequate for domestic wants, and 5 gallons as ample for public requirements; whilst the quantity employed for purposes of trade varies with the industrial needs of each district.

Consumption of Water in the United States.—The largest supplies of water have been provided for certain cities in the United States, where the carelessness of the consumers and their opposition to control, as well as considerable leakage from the mains, have led to large quantities of water being run to waste, which, in many instances, have been estimated at from one-third to one-half of the whole supply. At Buffalo, indeed, with a supply which has attained 210 gallons per head per day, 70 per cent. of the consumption, reaching 147 gallons, has been attributed to waste. The supply, moreover, furnished per head of population has not only often been doubled, but in some instances has had to be trebled, and even occasionally quadrupled within the last thirty or forty years, to meet the demand. Thus the water-supply to New York, which was 64 gallons per head per day in 1850, and about the same in 1880, had risen to 108 gallons in 1899;[2] the supply to Brooklyn rose from 39 gallons in 1870, to 72 gallons in 1893; to Boston, from 50 gallons in 1872, to 83 gallons in 1895; and to Louisville, from 19 gallons in 1870, to 62 gallons in 1893. Moreover, between 1860 and 1893, the consumption per head per day increased from 36 gallons to 122 gallons at Chicago, from 30 gallons to 83 gallons at Philadelphia, from 25 gallons to 103 gallons at Cincinnati, from 12 gallons (and 26 gallons in 1870) to 108 gallons at Cleveland, and from 43 gallons to 120 gallons at Detroit.[3] An increase in the demand for water per head has, indeed, been a common occurrence in the last fifty years, owing to the progress in sanitary science, the increased regard for cleanliness, and the augmented trade requirements, having, for example, risen in Liverpool from $10\frac{3}{4}$ gallons in 1851, to $28\frac{2}{3}$ gallons in 1861, and to $31\frac{3}{4}$ gallons in 1902:[4] but the peculiarity in the above examples from the

[1] *Proc. Inst. C.E.*, vol. cxii. p. 299.

[2] *Ibid.*, vol. cxliii. p. 68.

[3] "Consumption and Waste of Water," D. Brackett, *Trans. Am. Soc. C.E.*, vol. xxxiv. pp. 186–187.

[4] "Liverpool Public Health Congress, 1903, Handbook," p. 221.

United States, consists in the very large increase within comparatively recent times, and the large supplies attained per head per day.

It is considered that a reasonable average consumption per head per day in the United States, after bringing the waste down to a practicable limit, would be 25 gallons for domestic use, 4 gallons for public requirements, and 33 gallons for trade purposes, making 62 gallons altogether, and that any excess over this amount must be regarded as needless waste. At Yonkers, N.Y., where the supply is all measured by meters, only 18 gallons per head per day are used; and the trade supply in Boston is about 25 gallons.

Fluctuations in the Demand for Water.—The quantities given above represent the average daily consumption of water per head of population throughout the year; but the actual consumption exhibits considerable fluctuations, according to the hour of the day and the season of the year. Thus though the period of the fluctuations differs somewhat according to habits of the population in different places, the daily demand generally reaches a maximum in English towns between 9 and 10 a.m., and falls to a minimum from about 9 p.m. to about 6 a.m., the actual maximum between 9 and 10 a.m., amounting to about four times the minimum in the middle of the night.[1] A similar record taken every three hours in the United States, gives a maximum demand between 7 and 10 a.m., and a minimum demand between 1 and 4 a.m., the consumption being relatively small from 7 p.m. to 7 a.m.; and the maximum in this case, during three hours, was two and a half times the minimum.[2] In London, the maximum consumption of water takes place in June, July, and August, and the minimum in February; but the difference in this case between the maximum and minimum demand, is only 20 per cent. of the average demand. The conditions appear to be somewhat different in Liverpool; for in 1902, the maximum day's consumption in the year, of 34,188,000 gallons, occurred in February, as compared with the maximum day's consumption in the summer of 31,578,000 gallons, which occurred in July; and the average of 31,443,000 gallons per day during a week in February, 1902, as compared with an average of 28,168,000 gallons per day during four weeks in June and July, shows that the consumption generally in February is larger in Liverpool than during the period of greatest summer consumption. One reason why the demand is raised during the winter months in countries exposed to severe and frequent frosts, such as parts of the United States, is the practice of keeping a

[1] "The Theory and Practice of Hydro-Mechanics," Inst. C.E., 1885, "Water-Supply," W. Pole, p. 35.

[2] *Trans. Am. Soc. C.E.*, vol. xxxiv. p. 195.

continuous flow through the service-pipes during a frost, so that by maintaining the water in motion it may be prevented from freezing.

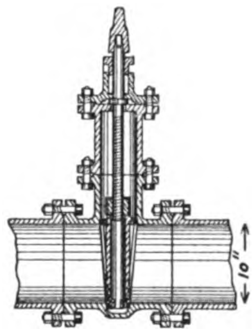
It is essential to estimate the greatest supply of water which may have to be delivered in a given period, for this is the supply for which provision has to be made in designing a system of water distribution to any district.

Distributing Mains and Pipes.—The water is led from the service reservoir to the district to be supplied, by a supply main, which, like the other mains and pipes, is almost always made of cast iron; and principal mains branch off from the supply main, or from reservoirs filled from the supply main, or from stand-pipes or high-level reservoirs into which the water is pumped, so as to supply the sub-districts into which the district is divided. These sub-districts are sometimes at very different levels, and therefore require to be supplied from reservoirs at different elevations, so that the water may be delivered at sufficient, and at the same time not excessive pressures, to the various parts of the town. Distributing mains and pipes branch off from the principal main of each sub-district along the various streets and roads of the inhabited portions, to supply all the houses with water, and also to lead the water to stand-pipes in the streets for watering them and flushing sewers, and to fire-hydrants. The supply main may be made of such a diameter that, with the available head, it will discharge an amount of water equivalent to the maximum demand, after allowing for a little diminution of the sectional area of the pipe owing to corrosion, and for the increase in population for which the supply has been provided. Theoretically, the diameters of the principal mains might be made proportionate to the demands which they have to meet, depending upon the population of the several sub-districts, and diminishing as the distributing mains branch off from them; but it is advisable to give them a slightly larger size, so as to ensure the water being delivered at an adequate pressure. Where, however, a distributing main branches off to supply a portion of the sub-district situated at a somewhat high level, and the principal main passes on to supply a low-lying part, it becomes advisable to reduce the size of the principal main below the inlet of the distributing main rather more than proportionately to the supply drawn off by this main, in order to check the tendency of the low-lying parts to draw off more than their proper share of the supply, at the expense of the higher parts.

The simplest and cheapest way of arranging the distribution is to make the mains and pipes branch off independently to the various side streets by the shortest route; but this results in a number of dead-ends at the termination of these branches, tending to produce stagnation of the water at these places, and only admitting of drawing the supply from one direction. Accordingly, the mains and pipes are generally arranged so as to be connected in a sort of network or gridiron, so that dead-ends are for the most part avoided; and in the event of a great sudden demand, such as occasioned by the breaking out of a fire, the water being drawn from two directions, flows much more freely to the required spot. This arrangement, however, necessitates the provision of a greater number of valves, Fig. 166, to shut off the water from a certain section in the event of the bursting of a main, or for the detection of waste, than the simple system of branches off the principal main.

SLUICE-VALVE ON MAIN.

Fig. 166.



It is not advisable to reduce the diameter of the pipes at the further ends of the distributing system of each sub-district below 3 inches, and even in the case of certain waters below 4 inches, on account of the incrustation which is liable to take place on the inner surface, especially with the acid waters flowing off from some hilly districts, and in order to provide adequately, after a certain reduction in area from this cause, for the supply of the hydrants on the outbreak of a serious fire. Moreover, in order to avoid the serious impediment which such incrustations would cause in the service-pipes of small diameter bringing the water from the smaller mains into every house, these pipes are generally made of lead, which is convenient for forming bends; or where a supply of very soft water is liable to act on the lead, owing to the free oxygen and carbonic acid contained in it, the pipes have to be made of galvanized iron, or iron lined inside with tin, or occasionally with cement.

Provisions necessary for Mains and Pipes.—There must be a cover of at least 2½ to 3 feet to protect the mains and pipes from fracture under heavy loads passing along the streets, and to prevent the water from freezing in winter in such a climate as that of England. In countries where the winter climate is more severe, such, for instance, as parts of the United States, the tops of the pipes have to be laid from 4 feet up to 7 feet in some places below the surface, to protect the water from frost. Moreover, the smaller and less continuous flow in the smaller pipes, renders the water passing through them more liable to get frozen than in the large mains and aqueducts. In hot countries, also, it is advisable to lay the pipes at least 3 or 4 feet below the ground, so as to prevent the water becoming unduly heated in its passage along the mains for distribution.

Oxidation of the inner surface of cast-iron pipes, producing projecting tubercles of rust in unprotected parts, which gradually spread so as to form in time a continuous coating, seriously impairs the flow of water through the pipes, both by the roughness imparted to the surface, and also by the reduction in sectional area, which is specially prejudicial in small pipes. Accordingly, all water-pipes laid underground should be protected as efficiently as possible from corrosion, by a preservative coating both inside and outside, to prevent the water flowing inside, and the dampness in the ground outside, from coming in contact with the iron. This is best accomplished by dipping the newly made pipes, before their surface has had time to become rusty, into a bath of coal-tar with sufficient mineral oil to give it fluidity, and to which resin is sometimes added, which has been heated up to between 300° and 400° F.; and the pipes are kept immersed in the bath till they have been raised to its temperature; and then after the coating has become cool, the pipes should be dipped a second time in a less hot bath, and for a shorter period, so as to avoid melting the first coat. By this process, the inner and outer surfaces of the pipes are covered all over with a sort of smooth, hard varnish; and this continuous coating, if carefully effected, generally preserves the iron from corrosion for several years. Usually, however, sooner or later imperfections in the inside covering, or small holes gradually forming in it from various causes, lead to the formation of these tubercles; whilst when the water passing through the pipes is soft and acid, and has not undergone filtration, an organic slime is apt to accumulate on the surface, and long, projecting, ferruginous organic filaments are formed, which considerably impede the flow through the pipes, an instance of which was mentioned in the last chapter, page 228, as having occurred in the aqueduct between Lake Vyrnwy and the filter-beds at Oswestry. This obstructive organic growth has been cleared away from the Vyrnwy aqueduct, by introducing a brush made to fit the pipe, and impelled by the flow of the water, which is provided with sets of wire bristles on its outer circumference, being formed in two pieces revolving in opposite directions as the brush travels along the pipe, Fig. 167; and this brush by revolving detaches the organic growth from the sides of the pipe, and carries it off in its onward course.[1] This brush was designed merely to remove the soft organic growth, and not for scraping off any rusty incrustations; for in large pipes, the growth of the incrustations appears to cease after they have attained a thickness of from 1 inch to $1\frac{1}{2}$ inches, but commences again directly this protecting covering is taken off,[2] and therefore reduces the thickness of the pipes after each operation of scraping.

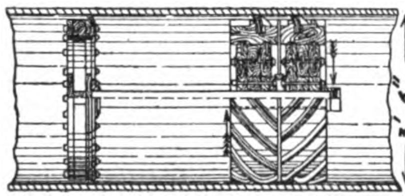
[1] *Proc. Inst. Mech. E.*, 1899, p. 507, and plate 136, fig. 11, and plate 137, figs. 13 to 16.

[2] "Deposits in Pipes and other Channels conveying Potable Water," Prof. J. C. Brown, *Proc. Inst. C.E.*, vol. clvi. p. 2.

The removal periodically of the incrustations, which may be inexpedient in the large pipes of aqueducts, is essential for the small pipes of distributing mains, which would otherwise be too much reduced in area by the rust to convey the required supply. Accordingly, scrapers have been designed for insertion in the pipes, worked, either by being drawn along the line of pipes, or more commonly by being pushed along by the flow of the water, the latter being in some instances furnished with rings at each extremity, to which a wire can be fastened for dragging the scraper along, or for recovering it if it should stick fast, as shown on the type of scraper used at Halifax,[1] Nova Scotia, Fig. 168,

DOUBLE-ACTING REVOLVING BRUSH FOR REMOVING ORGANIC GROWTHS IN PIPES, VYRNWY AQUEDUCT.

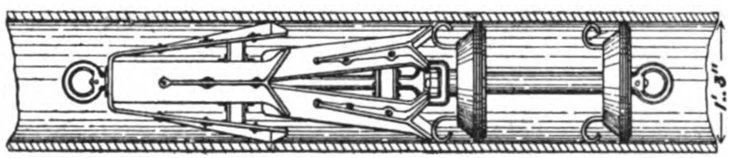
Fig. 167.



SCALE $\frac{1}{48}$.

PIPE SCRAPER, HALIFAX (NOVA SCOTIA) WATERWORKS.

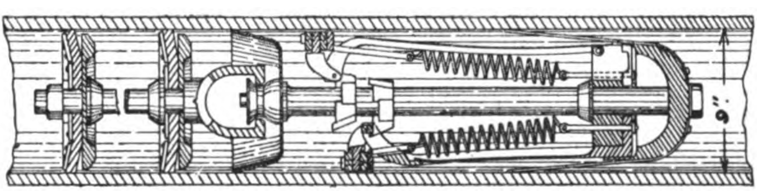
Fig. 168.



SCALE $\frac{1}{24}$.

PIPE SCRAPER, TORQUAY WATERWORKS.

Fig. 169.



SCALE $\frac{1}{12}$.

which is provided with eight scraping arms. These scrapers are also generally jointed, to enable them to go round bends in the pipes, as

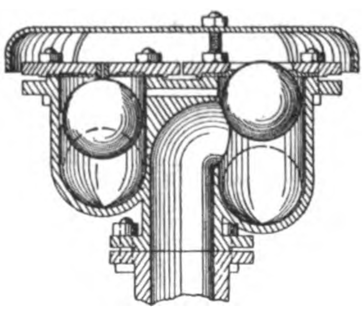
[1] "On the Removal of Incrustations in Water-Mains," E. H. Keating, *Trans. Am. Soc. C.E.*, vol. xi. pp. 127 to 135.

arranged in the scraper used at Torquay,[1] Fig. 169. The materials scraped off fall into receptacles at the bottom of the pipes, provided at intervals along the line, from which they are readily removed.

Accessories Requisite for Water-Mains.—At the highest points of the supply main and principal mains, it is necessary to make provision for the escape of the air which tends to accumulate in these places, as already mentioned in the case of aqueducts, and which if not got rid of would impede the flow of water, and also for the removal of the air from the mains when they are being filled with water. This can be effected by air-pipes furnished with cocks, which are opened by hand for letting the air escape. It is, however, preferable to fix self-acting air-valves, as used on aqueducts, at the summits of the chief mains, formed by an orifice which is closed by a floating ball as the water rises in the pipe on the escape of the air through the orifice; and when the air collects and causes a lowering of the water-level, the ball drops down into a receptacle, and thus opens the orifice for the air to escape. A small orifice suffices for the escape of the air which collects at the summits of the pipes; but a large opening is required for the discharge of the air from the pipes when being filled with water; and though these may be provided as separate, small and large air-valves, they can be conveniently combined in a single apparatus, as shown in Fig. 170. The escape of the air from the smaller distributing mains is adequately effected in the drawing off of the supplies; whilst the removal of the air in filling these mains can be readily accomplished by opening the hydrants.

DUPLEX AUTOMATIC AIR-VALVES.

Fig. 170.



Outlet valves should be provided at the lowest points of the supply main and principal water-mains, for the purpose of clearing out any deposit from the pipes, and for emptying the mains when required, the issuing water being discharged into the nearest watercourse. The hydrants serve for these objects in the case of the distributing mains.

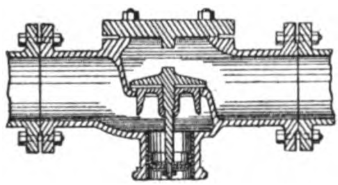
Where the variations in level of a district are considerable, it may be necessary to introduce valves for reducing the pressure in the lower portions of the district. This is accomplished in a simple manner by the pressure-reducing valve shown in Fig. 171. A cup-leather valve,

[1] "The Incrustation of Iron Pipes at Torquay Waterworks," W. Ingham, *Proc. Inst. Mech. E.*, 1899, p. 479, and plates 131 to 135.

open to the atmosphere on its under side, is fastened to the lower end of the spindle which supports at its upper end the larger valve closing the water-main; and in lifting this apparatus for the efflux of the water past the raised upper valve, a portion of the water-pressure is expended, the amount of reduction of the pressure being regulated by the relative areas given to the two valves.[1]

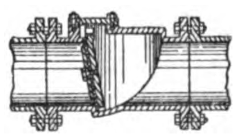
PRESSURE-REDUCING VALVE.

Fig. 171.



REFLUX VALVE.

Fig. 172.



In places where a water-main ascends for some distance in the direction of the flow, it is advisable to insert a reflux valve at the bottom of the ascending gradient, so that in the event of a fracture of a pipe on the service-reservoir side of the rising water-main, the water may be prevented from flowing backwards down the gradient towards the outlet provided by the fracture. A reflux valve consists of a flap door capable of revolving on a hinge fixed in a recess at the top of the main, which is ordinarily pushed back into the recess provided for its reception at the top, by the flow of the water from the service reservoir along the main, leaving the main open; but directly the direction of the flow in the main is reversed under the conditions mentioned above, the flap door falls down against its bearings, and, by thus at once closing the main, stops the efflux of the water down the descending gradient, Fig. 172.

Dual Supplies of Water for Different Purposes.—Generally when water for a large town is derived from different sources, provided these sources supply water suitable for domestic consumption, each source serves the district for which it is most conveniently situated, both by proximity, and by the height at which the water has to be delivered. When, however, a town is situated on a river or the sea-coast, or has any other source of supply at hand which is unfit for potable purposes, and draws its water-supply from the river considerably higher up, which has to be filtered before being delivered for consumption, or from impounding reservoirs or springs at a distance, economical considerations point to the conclusion that the pure water obtained at a large cost should be reserved for domestic consumption, and that the

[1] "Waterworks Distribution," J. A. McPherson, 1900, p. 89.

water close at hand should be used for municipal purposes, such as watering the streets, flushing the sewers, and extinguishing fires. It has, indeed, been occasionally suggested that a double supply might be furnished to every house; but this would add considerably to the cost of the internal fittings, would increase the loss from waste, and is open to the serious objection that small householders and servants would be careless about always drawing the water for drinking, cooking, and rinsing the utensils used for meals, from the pure-water taps. The only possible objection, however, to the use of any easily accessible supply of water for municipal purposes, is the necessity of providing a second line of mains if this unpurified water is to be distributed throughout a large town. As a compensation, nevertheless, for this expenditure must be set the cost of procuring and purifying a supply of water from a distance, considerably in excess of the actual domestic requirements, and the saving which may be effected by deferring for a considerable period the need of searching for an additional supply, probably further off, to meet the growing requirements of an increasing population, which can often only be obtained at an enhanced cost in comparison with the original supply.

Several large towns have already adopted to some extent a dual supply; and this system might be extended with advantage to places where a second source of supply is available in the neighbourhood, and the provision of a pure supply has to be sought a long distance away, is limited in quantity, and the conveyance or purification of which is expensive. Paris furnishes an instance of a dual supply, drawing considerable quantities of water from the Seine and the Marne, for the large service of street-watering in operation there and other public uses, and also for trade supplies, by pumps capable of raising 143 million gallons per day.[1] Calcutta possesses a very complete supply of unfiltered water pumped straight from the river, which is conveyed throughout the city in a separate set of mains, and is used for the extensive watering of roads and streets necessary in the dry season, flushing the sewers and privies, watering the gardens, and the extinction of fires, the supply for these purposes having reached 12 million gallons a day.[2] Several towns by the seaside use sea-water for watering the streets, for which purpose it is specially serviceable on account of the hygroscopic magnesium salts contained in it, which keep the surface damp by readily absorbing moisture; and sea-water is also largely used for baths. Liverpool makes use of 317,000 gallons of salt water daily, on the average. Several cities in the United States, situated on rivers, lakes, or the sea, have special large mains laid down from the water into the interior of the city, and furnished with hydrants for extinguishing fires. These mains are supplied with water, when required, from

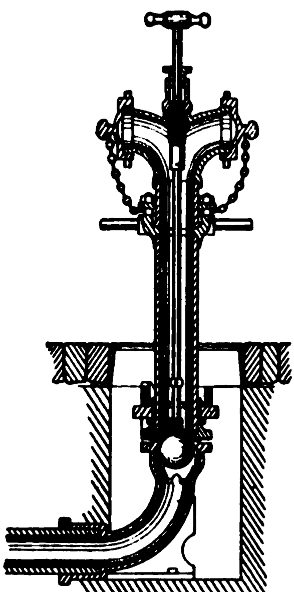
[1] *Proc. Inst. C.E.*, vol. cxliii. pp. 58-59. [2] *Ibid.*, vol. cxlviii. pp. 333-334.

boats provided with pumps which deliver the water through a hose into the mains. This system is in operation at Boston, Buffalo, Cleveland, Detroit, and Milwaukee, and has proved very efficient in combating conflagrations.[1]

Watering Posts and Fire-Hydrants.—In wide main thoroughfares, and especially where tramways run along the street, it is advisable to lay a main on each side of the roadway; but in ordinary streets or roads, the best position for the main is about three feet from the footpath on one

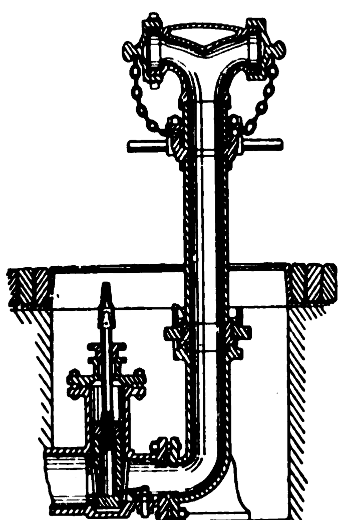
FIRE-HYDRANT WITH BALL-VALVE.

Fig. 173.



FIRE-HYDRANT WITH SLUICE-VALVE AT SIDE.

Fig. 174.



side of the road, so as to be out of the principal stream of traffic, and not in the way of the foot-passengers. In this position, the main, valves, and hydrants are more accessible for repairs or working; and the water from the hydrants when the mains are flushed or emptied, readily flows away to the gratings at the side. Where, however, the streets are paved with stone sets, wood blocks, or asphalt, on a concrete foundation, it is preferable to lay the main under the foot-pavement if practicable, on account of the expense of taking up and relaying the paving, and because

[1] "Public Water-Supplies," F. E. Turneaure and H. L. Russell, pp. 713-714.

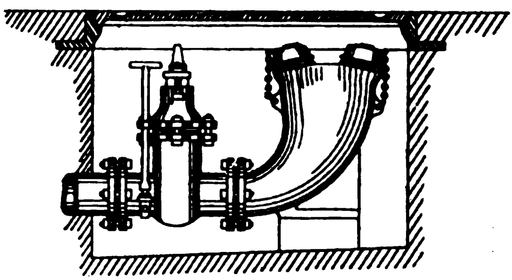
leaks from the mains may not be detected for a long time under the impervious roadway.

Watering posts are fixed at convenient intervals at the edge of the footpath in the principal streets, controlled by a valve, for filling watering carts, or for sluicing the roadway with a hose, which is also often effected from an outlet valve in the roadway.

Fire-hydrants have to be placed along the water-mains, at intervals not exceeding 100 yards, over which a special upright pipe, generally made of copper for lightness, is fixed when required to be put in action, to which the hose is attached. In some types of hydrants, a ball-valve is provided; and the hydrant is opened by turning a screw in the upright pipe by means of a tap head at the top, which, by pushing down a cap attached to its lower end, forces down the ball, and gives vent to the water,[1] Fig. 173 (p. 269). In other cases, the hydrant is opened and closed by an ordinary valve in a pipe at the side, leading to the hydrant at the base of the upright pipe, Fig. 174; and sometimes the fire-hose

FIRE-HYDRANT UNDER ROADWAY.

Fig. 175.



is attached straight on to the outlet of a special pipe laid below the roadway and controlled by a valve, Fig. 175.

Connecting Distributing Mains with Principal Mains under Pressure.—In order to form a new connection with a water-main in operation in the ordinary way, it is necessary, first to shut off the supply, and then to empty the adjacent portion of the main, sometimes for a considerable length, which, especially in the case of a constant supply, may occasion much inconvenience, and is a very serious matter in the event of an outbreak of fire. Consequently, arrangements have been devised for effecting the required connection under the full pressure of the water in the main, so as to avoid any interference with the supply.

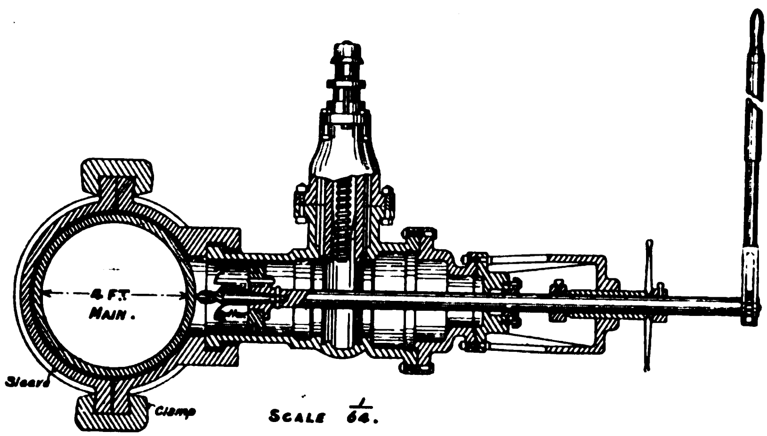
A machine for drilling and tapping a large water-main under pressure

[1] "Sluice-Cocks, Hydrants, etc.," Guest & Chrimes, Rotherham.

to connect a distributing main on to it, is shown in Fig. 176, where a special length of pipe, furnished with a sluice-valve, is attached to the main by an encircling sleeve run with lead.[1] A stuffing box is then bolted on to the end of this pipe, through the centre of which a drill and cutters have been passed. The drill is then turned to form a central hole in the main; and as soon as the drill has pierced the pipe, the cutters are brought into action, and cut out a disk of metal of

DISTRIBUTING MAIN CONNECTED WITH PRINCIPAL MAIN UNDER PRESSURE.

Fig. 176.



the required size from the main, the disk being prevented from falling into the main by two hinged little arms attached to the side of the drill, which open out inside the main directly the drill has pierced it. The drill, with the disk of metal, and the cutters are then withdrawn past the sluice-valve, which is at once closed; and on the removal of the stuffing box with the drilling apparatus, the distributing main can be connected to the special piece of pipe, and is controlled by the sluice-valve.

Service-Pipes connected with Distributing Mains.—As distributing mains are generally extended into suburban districts before most of the houses have been built, it is very commonly necessary to form new service connections after the distributing mains have come into use; and therefore screwing ferrules into these mains in the ordinary way, or tapping them for new service-pipes, is open to the objection of necessitating shutting off the supply and emptying the mains. This inconvenience, however, can be obviated, either by laying a smaller pipe parallel to the main in each street, controlled by sluice-valves at its two ends where it joins the main, to which the service-pipes

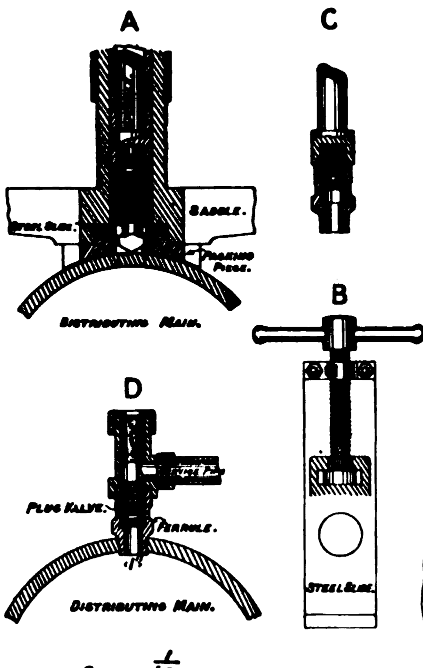
[1] "The Smith Tapping Machine," C. W. Harrison & Co., London.

can be readily connected, or by special contrivances for effecting the connections without interfering with the flow and water-pressure in the mains.

Two methods of forming the connections of the service-pipes under the full pressure in the distributing mains, are shown in Figs. 177 and 178. In the first apparatus, Fig. 177, the drill and tap, with the spindle

SERVICE-PIPE CONNECTED WITH MAIN UNDER PRESSURE.

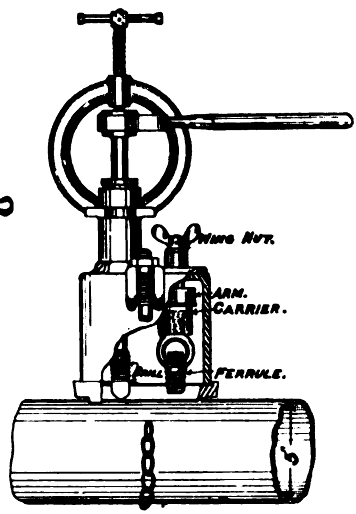
Fig. 177.



SCALE $\frac{1}{10}$.

SMALL SERVICE-PIPE CONNECTED WITH MAIN UNDER PRESSURE.

Fig. 178.



SCALE $\frac{1}{10}$.

and encircling stuffing box, **A**, are fastened to the main by a saddle and chain strap; and after the hole in the main has been drilled and tapped, the hole is closed by moving the horizontal slide **B** over it. The drill and tap are then withdrawn; and the ferrule **C**, with its plug-valve, is attached to the spindle, inserted in the stuffing-box, and, after the slide has been drawn back, is screwed into the hole in the main.[1] Lastly, the apparatus **D** is screwed on to the upper part of the ferrule, to the horizontal branch of which the service-pipe can be attached; and

[1] "Morris's Tapping Apparatus," J. Stone & Co., Deptford.

connection between the main and the service-pipe is opened by screwing up the plug-valve, by a key at the top, into the recess above the entrance to the branch.

A newer and simpler contrivance for connecting small service-pipes to mains under pressure, consists of a sort of box containing both the drill and the ferrule with its plug-valve and connections, fastened closely to the top of the main by a chain, Fig. 178; and the required hole is first drilled and tapped in the main. The drill is then raised; and the ferrule, attached at the end of an arm, is moved over the hole by half a turn of the nut on the top of the box; and, lastly, the drill is lowered into the carrier at the top of the ferrule and its adjuncts. Then the drill, when turned like a key, screws the ferrule into the main, and at the same time unscrews the upper part of the connections of the ferrule from the carrier, which can then be removed with the box and drill, leaving the ferrule and its connections in place ready for the attachment of the service-pipe.[1]

Intermittent Service.—Formerly the supply of water to the houses in a district was generally intermittent; for it was usually turned on, by opening valves in the mains, for only two or three hours in the day, and was shut off throughout the night, and during the rest of the day. This necessitated every house being provided with a cistern, which was filled when the water was turned on, or sometimes only partially filled if the pressure was deficient and the house at a high elevation; and the water thus collected had to suffice for the house throughout the period during which the supply from the main was shut off. If therefore the cistern happened to be emptied by some exceptional demand, no more water could be obtained till the supply was turned on again. In large towns, moreover, various impurities were liable to get into the cisterns and pollute the water; and frequently the cleaning out of the cisterns, which were often placed in situations which were difficult of access, was neglected or overlooked for long periods, probably not uncommonly for years. The shutting off also of the water from the mains occasioned serious delay on the outbreak of a fire, as it necessitated summoning the turn-cock before the water could be turned on. Moreover, the emptying of the mains during the long periods that the water was turned off, favoured the infiltration of liquids and gases into the mains from outside, through defective joints or cracks, producing contamination of the water.

This system was advantageous to the water company in fixing with precision the amount of the daily demand, and the period when it had to be supplied; and it also prevented the possibility of much loss from waste. On the other hand, it involved the regular service of a number of turn-cocks, and necessitated the provision of comparatively large

[1] "The Shelley Tapping Machine," C. W. Harrison & Co., London.

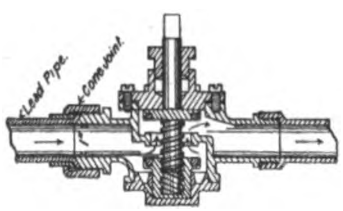
pipes to enable the full supply to be delivered in a short time to every house. Eventually, public convenience, and the progress of sanitary precautions, have gradually led to the obligation being imposed upon the water companies of abandoning by degrees intermittent services of water.

Constant Service.—By providing a constant service, water is always available to any extent in every house; the mains and pipes are kept continually full, so that there is no danger of any noxious infiltration into them; and water can be obtained at once for fire-hydrants or other public purposes. The constant pressure, however, on the pipes, increases greatly the loss of water from leaks at the joints, or through cracks in the mains, and from defective house fittings; whilst the unlimited supply of water often leads to carelessness on the part of the consumers, in leaving taps running, and disregarding waste of water in various ways.

A stop-cock placed on the service-pipe before it enters the house, enables the supply to be shut off from any house, without interfering with the general supply, for repairs, or for detecting leakage at night without entering the premises, one form of which is shown in Fig. 179; whilst others are similar to the sluice-valves on mains. A very great advantage in the constant service is that water can be drawn direct and fresh from the main; and this should be the invariable rule in the case of water used for drinking and cooking, even if, as is sometimes the case, the general supply is delivered into a cistern at the top of the house, which is always replenished as soon as it is drawn upon. The intervention of a cistern is open to the serious objection, already noted, of proving a source of contamination if the cistern is not frequently cleaned out; and the only excuse for its retention is a reduction in pressure, and therefore of injury to fittings and waste, and that it furnishes a small store of water in the event of a fracture of a main or a great consumption of water by a fire. Probably cisterns in many of the older houses are merely relics from the period of intermittent service, which are merely retained to save the trouble and expense of adapting the arrangements to a constant service. The great benefit, however, of drawing fresh water direct from the main, is liable to be lost by having a house cistern; and it would be far preferable, as a sanitary precaution, in spite of occasional minor disadvantages, to draw all domestic supplies straight from the mains.

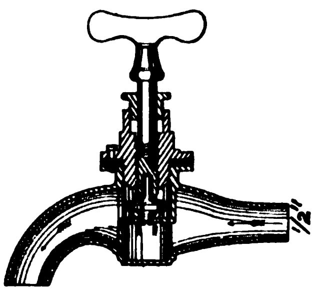
STOP-COCK ON SERVICE-PIPE.

Fig. 179.

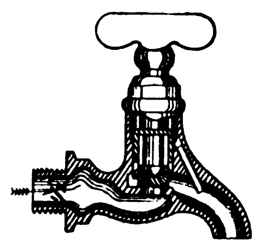


House Fittings for Constant Supply.—Owing to the increased pressure when a constant service of water is introduced, in direct communication with the mains, and the relief of the consumers from all fear of running short of water from undue waste, it is essential that all the pipes and house fittings should be thoroughly overhauled, so as to provide against leakage. All the taps should be screw-down taps, so that they may not be able to be closed very rapidly, and thus throw a sudden, and undue pressure on the pipes; and they should be strong, and of the best make approved by the water company, so as not to be liable to leak or get out of order quickly, and should be examined, repaired, and renewed from time to time. An ordinary screw-down tap is shown in Fig. 180, and an improved, very durable form, designed by Lord Kelvin, in Fig. 181.

SCREW-DOWN TAP.
Fig. 180.



KELVIN'S SCREW-DOWN TAP.
Fig. 181.



Flush-out cisterns, containing 2 to 3 gallons, have to be provided for water-closets; and like in other house cisterns, the inlet is closed, as the cistern fills, by the rising of a floating copper ball, attached at the end of a lever arm, turning the valve in the nozzle of the inlet pipe, and is opened by the falling of the ball on the emptying of the cistern by the sudden release of the flush. An outlet is provided above the regular water-level of the cistern when full, for the escape of the water when, from some defect in the ball-valve or the waterlogging of the ball, the inlet is not fully closed, and the water therefore rises too high; and an overflow pipe from the outlet leads the water outside the house to a point where its escape is sure to be noticed, and attention is thus drawn to the need of repairing the ball-valve.

Prevention of Waste of Water.—The general introduction of a constant service of water, by increasing the pressure and removing any inducement for the consumers to be careful in their use of water, renders it specially important to take precautions against waste. The large waste of water, in proportion to the population supplied, which occurs in some American cities, as previously alluded to, would soon, in a country of very limited area such as Great Britain, with its large and rapidly increasing population, render it difficult and very costly to provide the required supply of suitable water; and even the considerably smaller amount of waste which has often occurred in towns in Great Britain, has thrown a considerable strain on the resources of the water companies or municipal authorities who furnish the supply. Accordingly, efforts have been made for many years past to reduce the needless waste of water, and thereby avoid having to search unnecessarily soon for fresh sources of supply, involving an increased charge for the water, or an enhanced cost to the municipalities, results detrimental to the interests of the whole community.

There are three methods by which the waste of water can be greatly reduced. In the first place, considerable leakage inside houses can be prevented by providing pipes strong enough to withstand the pressure imposed by a constant supply; by the exclusive use of durable fittings of suitable design; and by periodical inspections, together with the immediate execution of any repairs required to stop leaks.

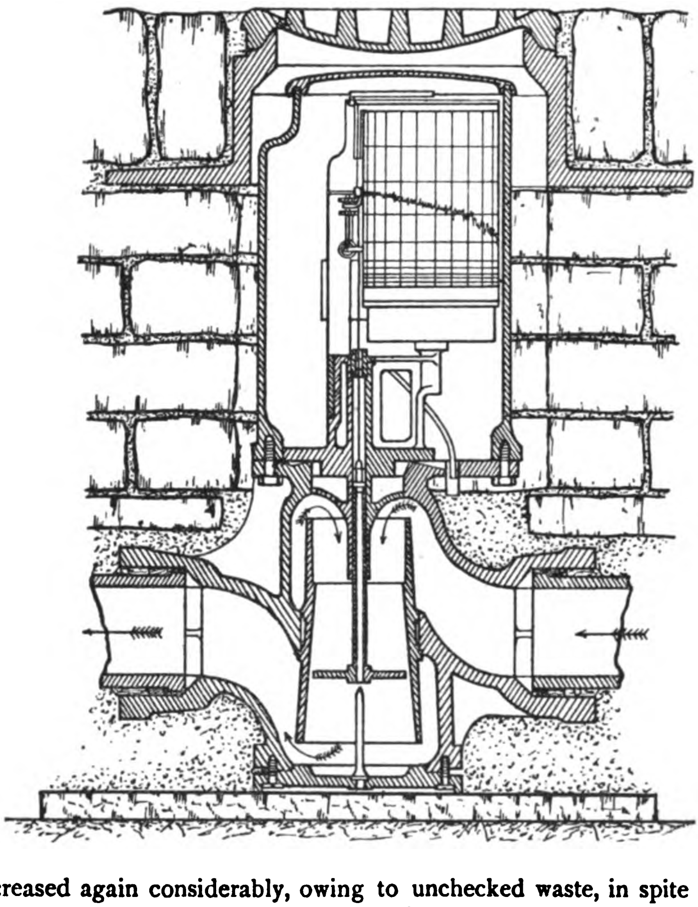
Secondly, leakage and waste in the houses can be more rapidly and surely detected, without unnecessary inspections, and leakage from the mains can be discovered, by dividing the town into a series of separate districts, which, when an investigation about water flowing to waste is made, are successively isolated at night; and the flow taking place through the distributing mains leading to each district is recorded, in the case of each distributing main, by a diagram drawn by a waste-water meter placed on the main near where it branches off from the principal main. An inspection of the series of diagrams indicates the amount of the flow through the several distributing mains in the middle of the night, which if considerable is a certain evidence of water flowing to waste.

Thirdly, unnecessary waste of water, due to the carelessness of householders, can be very materially checked by making each householder interested pecuniarily in preventing waste, by charging for the supply according to the amount actually consumed in each house, instead of by a water-rate based on the value of the house. This is readily accomplished by measuring the consumption by means of a water-meter placed on the service-pipe close to where it enters the house, so that the water used throughout the house must pass through it, on precisely the same principle that the gas burned in a house is measured by a gas-meter.

Waste-Water Meter.—A constant service was first provided in Liverpool in 1858; but the rate of consumption increased so much, that in 1865 it became necessary to return to an intermittent supply; and though this produced a considerable reduction in the rate of consumption for a couple of years, the consumption subsequently

WASTE-WATER METER.

Fig. 182.



increased again considerably, owing to unchecked waste, in spite of a regular system of house-to-house inspection. With a view of remedying this very unsatisfactory condition and reintroducing a constant supply, the system of detecting waste by means of a waste-water meter, devised by Mr. G. F. Deacon, was introduced in 1874. This waste-water meter consists of a horizontal, circular, gun-metal disk, carried by a hollow brass rod which slides in a vertical brass tube extending upwards from the conical chamber in which the disk moves up and down, and over a rod at the bottom,[1] Fig. 182. A fine wire connects the rod of the disk with a small carriage running vertically between guides, which is supported by a wire cord passing over a pulley, to the end of which a counterbalance weight is attached; and the carriage carries a pencil which traces a diagram on a sheet of paper, wrapped round a drum which makes one revolution in 24 hours, on which the vertical lines indicate the hours, and the horizontal lines a flow proceeding from zero by increments of 500 gallons per hour. The diagram, accordingly, records the motions of the disk, which are transmitted to the carriage by the wire; and when no flow is taking place, the disk is raised to its highest position in the cone by the counterbalance weight, and the pencil rises to the zero line at the top of the diagram. Directly any flow occurs, the disk is carried down the cone in proportion to its volume; and the pencil undergoing a corresponding vertical movement, records the amount of flow and the period of its occurrence on the diagram, so that the diagram indicates at a glance the variations in flow along the main throughout the 24 hours. When, owing to a fire or other exceptional cause, the demand on the main is greatly in excess of the ordinary discharge, the disk is carried down to the bottom, clear of the cone, so that there is no interference with the flow. The waste-water meter is either placed in the roadway directly on the main, or, where the traffic is considerable over the road, in a branch pipe under the footway, leading off the main and controlled at each end by a valve.

Detecting Waste by the Waste-Water Meter System.—This system which was first introduced in Liverpool for reducing excessive waste, has since been resorted to in many places. The town is first divided into suitable districts, which in Liverpool at the present time number 260, with from 200 to 1500 houses in each district; and a waste-water meter is fixed on the distributing main to each district, near its junction with the principal main; and the flow indicated on the diagram drawn by the meter as taking place between 1 a.m. and 4 a.m., in residential districts, measures the waste taking place in each district. The districts showing the greatest waste are then dealt with first; and operations are commenced towards midnight by sounding all the stop-cocks to the service-pipes in succession with the turning key or a steel bar, or occasionally with a wooden bar having an ear-piece at the end, serving as a stethoscope, and also the hydrants and valves accessible from the street or footway. If any noise of running water is heard, the

[1] "The Constant Supply and Waste of Water," G. F. Deacon, *Journal of the Society of Arts*, 1881–82, pp. 738 and 746–747; and "The Systems of Constant and Intermittent Water-Supply, and the Prevention of Waste," G. F. Deacon, *Proc. Inst. C.E.*, vol. xlii. p. 143, and plates 4, 5, and 6.

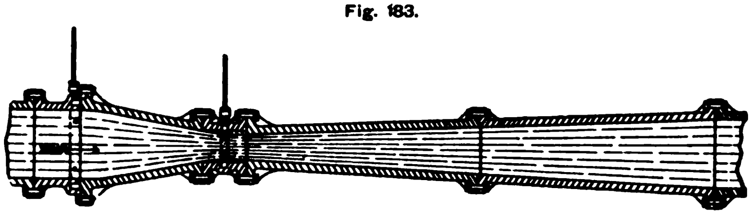
stop-cock is closed, and its number and the time noted; and if the noise still continues, leakage is occurring from the service-pipe between the stop-cock and the main, or from the main, which is further located by other soundings close by; and subsequent inspection of the diagram shows the reduction in the flow with each closing of a stop-cock.[1] The premises where waste has been thus detected, are then examined, and repairs and renewals ordered; and the effect of the repairs is indicated on a subsequent diagram of the night flow in the district.

The total number of defects discovered by this means in Liverpool in the year 1902, amounted to 40,299, mainly in private pipes, cocks, and the apparatus of water-closets;[2] and waste was found to be taking place in about 4 per cent. of the houses in the districts inspected.

The Venturi Meter.—This meter is very valuable in measuring flows of water in large pipes in a cheap manner, with only a very slight loss of head; and it has been named after the Italian philosopher Venturi who established the principle upon which it is based, as a scientific fact, in 1796, namely, that water flowing through a pipe diminishing in sectional area, loses lateral pressure as it gains in velocity. The Venturi meter consists of two cast-iron, hollow, truncated cones, joined at their smaller ends, of equal diameter, by a short piece of pipe, termed the throat, which is made of, or lined with bronze.[3] This meter forms part of the water-main, the flow through which has to be measured, being connected at its two ends, constituting the larger extremities of the cones, to the regular line of pipes, where they attain the same diameter as the main, Fig. 183. The water entering the

THE VENTURI METER.

Fig. 183.



shorter cone gains velocity and loses pressure in flowing to the throat, in proportion to the reduction in diameter between the inlet to the meter and the throat; and the difference in pressure at the inlet, and at the throat, is known as the Venturi head. In flowing from the throat along

[1] *Journal of the Society of Arts*, 1881-82, pp. 744-745.

[2] "Liverpool Public Health Congress, 1903, Handbook," pp. 223 to 225.

[3] "The 'Venturi' Meter for the Measurement of Water in Bulk," George Kent, London, 1898; and "The Venturi Meter," Clemens Herschel, *Trans. Am. Soc. C.E.*, vol. xvii. p. 228, and plates 33 and 34.

the longer cone, the water regains pressure and loses velocity, the initial pressure and velocity being practically restored on reaching the farther end of the meter; and the slight loss of pressure which occurs in the passage of the water through the meter, is termed the friction head.

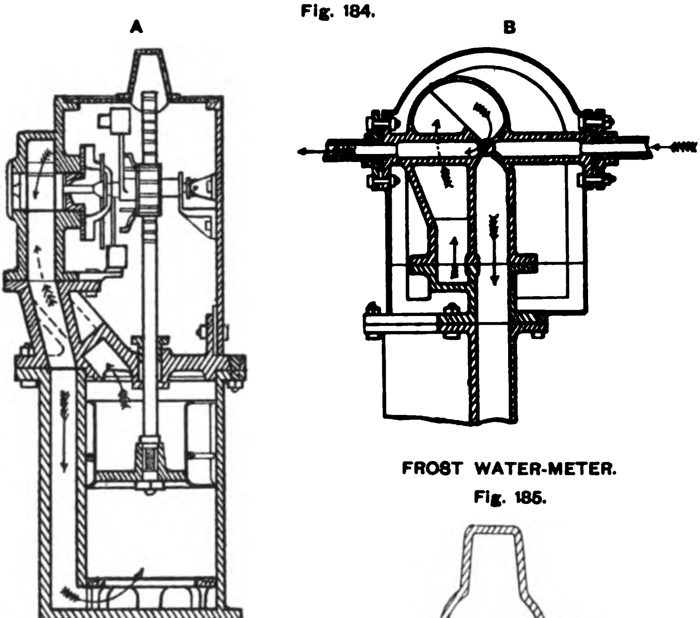
As the velocity of the flow of water through the throat, and, consequently, the discharge, is proportionate to the square root of the Venturi head, it is only necessary to obtain a record of this head and its changes, in combination with the time, to enable the discharge through the main during any period to be ascertained. The difference of the pressures at the inlet of the meter, and at the throat, is obtained by forming a communication, by some small holes, between the water in the meter and small pressure chambers encircling the inlet, and the throat of the meter respectively, from which the pressures are transmitted by small copper pipes to a U tube containing mercury, by which the difference between the two pressures is measured, Fig. 183. Floats on the mercury in connection with clockwork, combine changes in pressure with the element of time; and by means of suitable mechanism, a diagram is drawn shown the varying rates of flow throughout a given period, and a counter indicates the total amount of water which has been discharged. The relation of the sectional area of the throat to that of the main, is determined according to the volumes of flow which have to be measured, the throat being made relatively large when the maximum and minimum flows to be recorded are high, and small when these flows are low.

Water-Meters.—When water is charged for according to the volume actually supplied, as is the case in large trade supplies, instead of levying a fixed water-rate based upon the value of each house, it is necessary to place a meter for measuring the flow on the service-pipe of every house, near the spot where it enters. A great variety of water-meters have been invented for this purpose; but they may all be broadly divided into two classes, namely, positive and inferential water-meters. Positive water-meters measure the actual quantity of water passing through them, by filling and emptying chambers of known capacity by aid generally of a piston, moved up and down by the flow of water, whose strokes are recorded by a registering mechanism, each stroke representing a definite volume of water passed through, as shown on an index attached to the water-meter. Inferential water-meters, on the other hand, consist essentially of a series of vanes, or a turbine, revolving on a horizontal axis when impelled by a flow of water through the meter, and therefore serving as a measure of the velocity of the passing water, and not of its volume; but an index at the top, turned round a dial by the revolutions of the axis of the meter through the intervention of a set of toothed wheels, records the volume of water passing through as deduced from the number of revolutions of the meter.

The best types of positive piston meters record with great accuracy both large and small flows of water. The Kennedy meter is an old-established form of piston meter, which measures very accurately the amount of water passed through it, Fig. 184. The complete section **A**

KENNEDY WATER-METER.

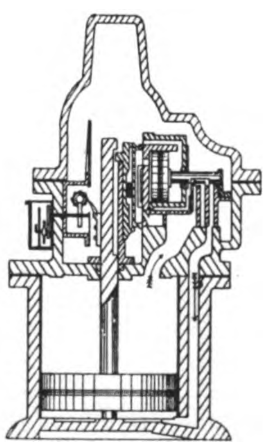
Fig. 184.



shows the piston working in the cylinder, and the passages for the entrance and exit of the water; and the partial section **B**, in Fig. 184, shows the valve near the top, which, when turned over by the sudden fall of a hammer, indicated in **A**, directly the piston reaches the end of its stroke, reverses instantaneously the direction of the flow. The Frost piston meter is another well-known positive meter, Fig. 185; and the flow in this case is reversed by a horizontal slide-valve moved backwards or forwards by two small pistons, behind each of which water is

FROST WATER-METER.

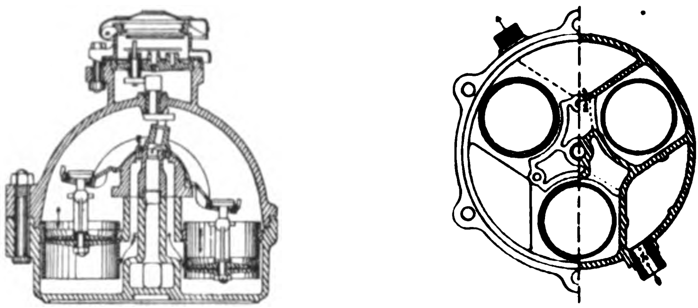
Fig. 185.



admitted alternately by an auxiliary vertical slide-valve moved by the rod of the large piston.[1] A peculiar, and more recent form of positive meter, specially designed by Mr. Schönheyder, to secure the measurement of very small flows, is shown in Fig. 186, where three cylinders

SCHÖNHEYDER WATER-METER.

Fig. 186.

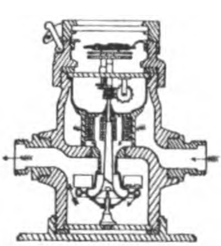


are placed symmetrically in a round, cast-iron case, in which pistons are raised and lowered in rotation.[2]

Good inferential meters record large and ordinary flows of water with very fair accuracy, if kept in proper order; but they are liable not to revolve with small flows, especially if the water is turned on very gradually. The Siemens reaction turbine meter is given in Fig. 187; but this form of inferential meter produces a considerable reduction in the pressure, owing to the small orifices of the turbine; and its accuracy is liable to be impaired from a reduction in the size of these orifices by accretion from sediment in the water. Accordingly, when the sale of water by meter began to be extended in Berlin about 1870, which eventually was made universal throughout the city in 1878, a vane meter was introduced by Siemens and Halske,[3] and gradually improved, which furnished satisfactory results, and is sh own in Fig. 18; and another form of vane meter,[4] known as the Tylor rotary meter, is illustrated in Fig. 189.

SIEMENS TURBINE WATER-METER.

Fig. 187.



Inferential meters are cheaper than positive meters—a very important

[1] *Proc. Inst. Mech. E.*, 1882, p. 41, and plate 1, fig. 4.

[2] "Imperial Positive Meter," *Proc. Inst. Mech. E*, 1900, p. 37, and plate 1.

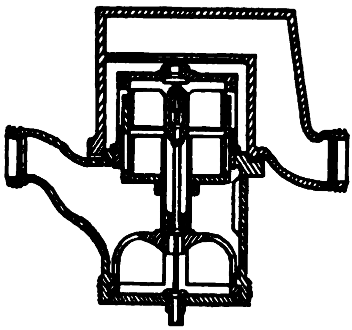
[3] *Proc. Inst. C.E.*, vol. cvii., pp. 206–210.

[4] *Proc. Inst. Mech. E.*, 1900, p. 61, and fig. 28.

consideration when the system of supplying water exclusively by meter is adopted, as in Berlin; and though distinctly less accurate than positive meters, inferential meters satisfy generally the requirements of measure-

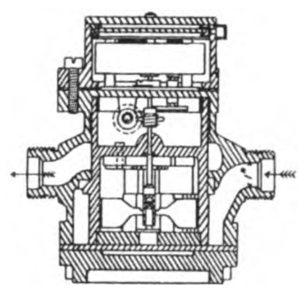
SIEMENS AND HALSKE VANE WATER-METER.

Fig. 188.



TYLOR VANE WATER-METER.

Fig. 189.



ment, except in the case of very small flows, which are liable to pass unrecorded, but for which an allowance can be made in fixing the price of the water supplied.

Reduction in Waste by the Sale of Water by Meter.—In order to be suitable for the sale of water generally by measure for domestic purposes, water-meters should be cheap, of moderate size, and easily fixed and removed, capable of running for a long time without needing repairs, producing only a slight reduction in the pressure of the water passing through them, and recording the flow at different pressures with an error not exceeding about 2·5 per cent. The sale of water by measure is such an obviously equitable and satisfactory method of reducing waste, by enlisting the interests of the householders on the side of the purveyors of the supply, that at first sight it appears remarkable that any other system has been at all generally adopted. Two objections, however, have been strongly urged against this method of charging for the water consumed, namely, that it would induce the poorer population to stint themselves in the use of water to an extent that would be prejudicial to sanitary conditions, and also that the cost of a meter and its supervision would be out of all proportion to the receipts for water in the poorest tenements. These objections, however, could be readily surmounted, by making a fixed charge for an amount of water per head sufficient to comply with sanitary requirements, in the case of houses where the cost of the water might be regarded as a matter of concern by the inhabitants, and by only charging by measure for any consumption beyond this standard quantity; and dispensing with the use of the meter system for the poorest class of houses, where the value of the water consumed is too small to bear the cost of the meter and its supervision for every house. In not arranging for the supply of the larger and richer houses by meter, the water companies are probably influenced by the circumstance that probably, especially in the wealthy districts of large cities, a charge for water according to the rateable value is more profitable than the sale of water by measure; for such houses are often not nearly full, and generally for a portion of the year are nearly empty, when the consumption of water is very low, though the rate has equally to be paid. Moreover, in selling water by measure, water companies could not charge for water which they do not supply, as actually occurred in many places in England in the severe winter of 1894–95, when, owing to the freezing of the water in the mains and their consequent fracture, the water-supply was in many instances cut off for one, or even two months, and yet the water-rate, by a legal anomaly, had to be paid.

A country with a very restricted area and a rapidly increasing population, like Great Britain, must use every possible means to prevent waste; and though waste-water meters, the adoption of reliable fittings, and efficient inspection and timely repairs, may result in a considerable reduction in waste, the sale of water by measure, as a general rule, would serve as a most powerful auxiliary in bringing the waste within the narrowest possible limits; and its universal adoption, except in the particular circumstances alluded to above, would be as advantageous as it is equitable.

PART II.

SEWAGE DISPOSAL.

CHAPTER XII.

HOUSE DRAINAGE, AND DISPOSAL OF REFUSE.

Sewage Disposal contrasted with Water-Supply—Methods of removing Sewage from Houses in the Country:—Earth-Closets, description, superiority over middens, advantages, objections; Cesspools, description, emptying, disadvantages; Septic Tank, cycle of bacterial changes, merits, upward filtration—Disposal of Solid Refuse from Houses, methods adopted—Flow from Houses, nature of, separation of rainfall in rural districts, difficulties in towns—The Pail System for Towns, advantages, objections—Sanitary Arrangements in Houses, position of conveniences, trapping of soil-pipes and waste pipes, flushing cistern separated from supply cistern—Flushing Cisterns, methods of action, merits, overflow—Water-Closets, former defective systems, modern improvements, special forms, general arrangements—Arrangement of Soil-Pipes, and their Ventilation, description of precautions adopted—Provisions for the Discharge from Baths and Sinks, description of contrivances for baths and sinks, special grease traps, various precautions—Drainage of Stables, flooring, drainage, gullies and traps—House Drains under Buildings, provisions—Drains from Houses to Sewer, in towns, in suburban districts, fall, removal of projections and obstructions, disconnecting manhole and trap near junction—Underground Conveniences—Refuse Destructors, deposit of refuse objectionable, resort to burning, utilization of clinkers, importance of high temperature with forced draught, top-fed destructors, disadvantages, destructors fed from the back, merits, special advantages of feeding in front; combination of steam boilers for power production with destructors, limits of system, value of destructors as sanitary agents.

THE disposal of sewage, so far as it is effected by means of water-carriage, is to a considerable extent the reverse process to water-supply; for the fouled water, with the matters it carries along with it, is collected from the several houses by drains and small sewers, and is conveyed to its destination in a main sewer. Moreover, whereas the supply of water has to be carefully guarded from pollution throughout its passage from the source of supply to the locality where it is distributed, the sewage has to be prevented from causing pollution during its conveyance from the district where it is collected to the site of disposal.

In fact, by means of sewers, the water used for domestic supply is enabled to complete a sort of cycle; for after having been brought in conduits from its source to the towns it supplies, it is conveyed away again by the sewers, after having accomplished its domestic and cleansing purposes, and eventually finds its way back to the sea, from which it was originally drawn by the action of the sun's heat.

Methods of removing Sewage from Houses in the Country.—As sewage decomposes rapidly, giving rise to noxious exhalations, which, especially in the case of disease, are liable to be very injurious to health, it is of the utmost importance that the sewage should be removed as expeditiously as possible from houses; but the means by which this can be effected depends upon the conditions of the locality. In the country, where the houses are at some distance apart, each house has to make provision for its own drainage; and this can be accomplished, either partially by the adoption of earth-closets, or by the construction of a cesspool, or underground tank, at a little distance from the house, for receiving the sewage, which is discharged into it through drains by the aid of water.

Earth-Closets.—These closets are provided with a hopper receptacle at the back filled with dry earth, from which, by the automatic rising of the seat, or by pulling a handle, a definite quantity of earth is discharged under the seat each time the closet is used. By placing a pail under the seat for receiving the sewage and the successive layers of earth, the contents are readily removed when the pail has been filled, and are usefully employed for manuring the adjacent garden or land. The earth deodorizes the sewage, and facilitates its deposit on the land; whilst the manurial value of the mixture depends upon the earth used, the best results being obtained with ordinary soil, loam, and peat; whereas ashes, which are sometimes employed on the score of convenience, are unsatisfactory in this respect. By drying the contents of the pails, and using them three or four times over in place of fresh earth, the manurial value of the mixture is increased, and the amount of earth required is reduced.

The earth-closet is undoubtedly a great improvement on the old midden, where the crude sewage was allowed to accumulate for some weeks, either simply on the ground under the seat, or for a longer period in a special receptacle formed in the ground, being only removed when absolutely necessary. The system possesses advantages, if carefully managed, for isolated dwellings where the water-supply is liable to be inadequate for water-closets; but it is best suited for outdoor conveniences, or where the deposit can be removed from the outside, and thus carrying it through the house is avoided. Earth-closets, however, merely dispose of the greater portion of the sewage proper, and not that collected habitually in bedrooms and during illness; and they leave the fouled waters from sinks, slop-pails, and baths, which all come properly under the general definition of sewage, entirely unprovided for.

Cesspools.—By constructing a watertight, underground, brick tank, arched over at the top, a little distance from an isolated house, into which all the drains from the house discharge, it is possible to remove all the sewage by water-carriage. This cesspool, or cesspit, should be provided with a ventilating pipe and an inlet for fresh air; and its liquid contents must be periodically pumped out and distributed over the adjacent garden. To reduce the nuisance of spreading some of the solid contents of the cesspool, stirred up by pumping, over the ground with the liquids, the overflow from the cesspool can be led by a pipe into a second tank suitably placed for distribution, from which it can be pumped as required; whilst in porous soils, the amount of liquid to be dealt with can be reduced by forming the tank without a bottom, thereby allowing the liquid sewage to percolate through the soil, provided there are no wells for water-supply in the neighbourhood.

When the cesspool has been nearly filled with the solids of the sewage, they are removed at night, after being deodorized to some extent by lime or iron sulphate, and are either spread over land at a distance, or deposited in a pit dug out in the grounds, and covered over with soil, where they are gradually decomposed, and can be eventually used for manure. The occurrence of nuisance in this process of emptying cesspools is avoided by the adoption of the pneumatic system. A large, airtight, cylindrical tank, carried on wheels, is put in connection with the cesspool by a flexible 3-inch pipe; and the air having been withdrawn from the tank by an air-pump, or by filling it with steam, which on condensing creates a vacuum, and the contents of the cesspool having been placed in communication with the tank by opening a tap at its junction with the pipe, the sewage in the cesspool is forced by atmospheric pressure through the pipe into the tank, which is then wheeled away to the place of deposit.

Cesspools are fairly unobjectionable for detached houses with good-sized grounds; but they are liable to prove a nuisance in the case of cottages built close together with very small gardens. Accordingly, in growing, inland, suburban districts, the local authorities have been substituting a system of sewers, conveying the sewage of their districts to a sewage farm for disposal by irrigation, in place of the cesspool system, which is unsuitable for a populous, increasing community restricted to a limited area.

Septic Tank, or Bacterial Action.—The most modern and effective system for the disposal of the sewage of country communities, consists in bacterial purification, which has generally been accomplished by leading the sewage gently into the bottom of a septic, or decomposing, tank, in its passage through which, being made to flow slowly in the dark, it undergoes decomposition by the action of the anaerobic bacteria contained in it, which flourish best out of contact with the air; and the process is completed by the action of aerobic bacteria, on the sewage flowing out of the septic tank, when fully exposed to the air by filtration, or by passing it along a series of perforated trays containing coke. In this manner, the solids in the sewage are liquefied; and the sewage in undergoing nitrification, is rendered innocuous, and is converted into a valuable liquid for spreading over the land, on account of its serving to restore the nitrogen and other mineral constituents to the soil, which are drawn from it in the raising of crops.

Though septic tanks, with subsequent aeration, have been adopted in many places within recent years for the treatment of sewage, both on a small and large scale, the proper cycle of changes by means of the different forms of bacteria, appears to be best accomplished by subjecting the sewage to upward filtration through layers of suitable materials in a tank, so that at first it is wholly acted upon by anaerobic bacteria; then, on rising, it is exposed to the influences of both anaerobic and aerobic bacteria; and on reaching the top of the tank, and further full exposure to the air, the final changes are wholly effected by aerobic bacteria.

Disposal of Solid Refuse from Houses.—In every household there is a daily amount of refuse which has to be got rid of, such as cinders, ashes, parings of vegetables and fruit, waste paper, and odds and ends of various kinds. In the country, this collection of refuse is placed on a dust heap, which is either burnt periodically by aid of dead leaves, branches, and other garden rubbish, or is used for filling up hollows. In suburban districts and towns, the refuse from the houses is collected regularly from the dust-bins by the dust-carts of the sanitary authority, and, after being sorted, is burnt in refuse destructors, or is employed for raising waste, low-lying land, or at towns near the sea-coast is sometimes carried out in barges to sea and deposited.

Flow from Houses.—The whole of the water supplied to each house is discharged from it again, in a more or less polluted state, through the drains from the sinks, baths, lavatories, wash-houses, and stables, and also from the closets, unless earth-closets or the pail system are in use. Moreover, in addition to this flow, the rainfall on the roof is collected in the gutters, and carried down the rainwater pipes to the ground. This supply of soft, and in country districts tolerably clean, water, is often collected in the country in underground brick tanks, to supplement a deficient water-supply, or to provide soft water for washing in places supplied by water from Chalk or Limestone strata. On introducing sewers in rural and suburban districts, the inconvenience and cost of having to provide for the large additional and variable flow of the rainfall, is generally avoided by leaving the rainfall to be collected in tanks, to find its way into the watercourses, or to sink into the ground, as before, and by merely receiving in the sewers the tolerably uniform flow of sewage from the houses, equivalent approximately to the water-supply. In towns, however, the flow of the rainwater from the roofs, yards, and streets, becomes so soiled or polluted in its course, that it becomes necessary to receive it in the sewers with the sewage, greatly increasing the volume to be discharged in times of heavy rainfall, except in cases where the remodelling of the network of sewers to remedy defects, and to provide for a large extension of population, leaves the old lines of sewers free for the discharge of the rainwater.

The Pail System for Towns.—The volume of solid matter to be dealt with is so much increased by the addition of earth to the sewage in earth-closets, as to render the system inapplicable economically for towns. Pails, accordingly, have been introduced, notably in Rochdale and other northern towns in England, for receiving the sewage proper of the closets of houses, which are collected at night and emptied at regular intervals. The pails are sometimes lined inside with an absorbent refuse, a plan adopted at Halifax; and the lining is renewed each time the pail is emptied, before being replaced in position. The advantages of the system are, the collection of the sewage in a concentrated form, and, consequently, in a more valuable form for manure; the keeping out of the sewers the main portion of one of the most serious sources of pollution; and the saving of water, and a consequent reduction in the volume of flow to be discharged by the sewers, as compared with water-closets. On the other hand, though the pail system is a decided improvement on the midden and cesspool, the cost of removal is considerable; moreover, like the earth-closet, the pail system fails to deal with a very important portion of the sewage from houses; and, besides being more unpleasant, statistics show that it is very distinctly less favourable to health than the water-closet system.

Sanitary Arrangements in Houses.—The water-closets, sinks, baths, and wash-basins in houses should be placed close to outer walls, so that the polluted or soiled water may be led outside the house as rapidly as possible, and the lengths of soil-pipes and drains inside the house reduced to a minimum. These pipes, also, should be left as open to view as practicable, so that any leakage or other defect may be detected at once, and remedied. Every soil-pipe must be trapped close to its inlet from the water-closet, efficiently ventilated beyond the trap outside the house, and the drain-pipe leading from it also trapped before discharging into the sewer or cesspool. The pipes, also, from the sinks, baths, and basins, must be trapped near their inlets, and discharge outside the house into an open gully, covered over by a grating, which is trapped at its junction with the branch drain-pipe, discharging eventually into the main drain-pipe communicating with the sewer or cesspool. The object of these various provisions is to prevent the possibility of any sewer gas from the cesspool, sewer, drain, soil-pipes, and gullies, finding its way into the house.

The flushing cisterns in the water-closets, and also those which may be provided with advantage for the kitchen and scullery sinks, must be filled independently from an open pipe, controlled by a ball-cock, leading from the water-main or supply cistern, so that the supplies to the water-closets and sinks may be entirely disconnected from the water-supply to the house, which is thereby preserved from any danger of pollution.

Flushing Cisterns.—These cisterns, providing generally a flush of 2 to 3 gallons, are made in various forms, for the most part with a central siphon arrangement which is set in action by pulling a handle, and rapidly discharges the contents of the cistern into the basin of the closet, or into a sink; and the fall of the water in the cistern causes the descent of the ball, and the consequent opening of the ball-cock for the refilling of the cistern, Figs. 190 to 193. The siphon in these cisterns is sometimes put in motion by lifting some of the water in the cistern by raising an iron plate in pulling the handle, so as to fill the upper part of the siphon with water, as shown in Fig. 190; or the water is forced up a fixed, inverted cylinder, or bell, closed at the top and open at the bottom, enclosing the discharge pipe, till the rising water flows over into the inlet of the pipe at the top, and forms a siphon, which is effected by pushing an iron plate down nearly to the bottom of the cistern, Fig. 192. In the two other siphon cisterns, illustrated in Figs. 191 and 193, the bell covering the outlet pipe is raised with the water enclosed in it, till the water in the bell rises high enough to overflow into the mouth of the discharge pipe and start the siphon.

The above siphon cisterns cannot be used again for flushing till they have been refilled, which occupies 2 or 3 minutes; and where it is desired that two or more flushes should be capable of following one another in rapid succession, a different arrangement has been resorted to, as shown in Fig. 194. A larger cistern is placed immediately above the flushing cistern, holding sufficient water to fill the lower cistern three or four times over, through an aperture in its bottom, which can be closed by a plug-valve. A similar valve also serves to close the inlet of the discharge pipe at the bottom of the lower cistern; and both valves are connected with the handle. Under normal conditions, the upper valve is kept raised, so that the two cisterns are in communication; and the lower valve, controlling the discharge, is closed, Fig. 194. On pulling down the handle, however, the lower valve is raised, causing the rapid discharge of the contents of the flushing cistern; and the

FLUSHING CISTERNS.

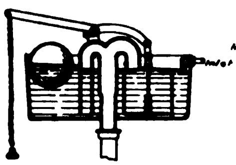


Fig. 190.—Doulton's "Paisley" Cistern.

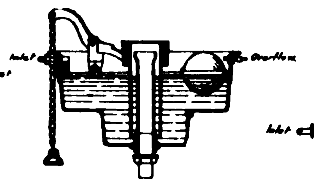


Fig. 191.—Doulton's "Bothwell" Cistern.

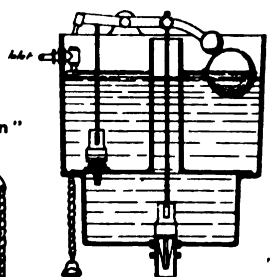


Fig. 194.—Guest and Chrimes' Double Valve Cistern.

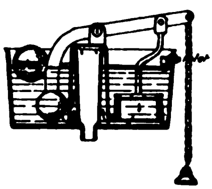


Fig. 192.—Finch's "Holborn" Cistern.

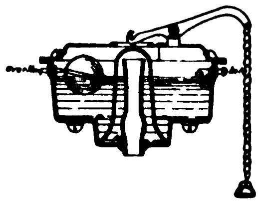


Fig. 193.—Shanks' "Levern" Reversible Cistern.

upper valve is at the same time lowered on to its seat, cutting off the communication between the two cisterns. Directly the handle is released, a counterweight raises it again to its original position, and reverses the positions of the valves, which occasions the rapid filling again of the lower cistern by the water in the upper one, in readiness for another flush.

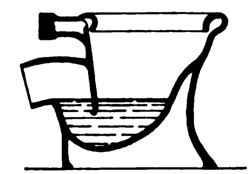
These flushing cisterns are all termed waste preventers; for, though they provide an ample flush on pulling the handle, they prevent the occurrence of leakage at other times, as frequently took place when the objectionable, old-fashioned pan or valve closets were served direct from the supply cistern, and the flow controlled by imperfect valves, which, besides frequently leaking, were not unfrequently kept open for some time by tying the handle up, with the view of flushing the drains. The only source of waste from these cisterns is by water escaping through the overflow pipe when the floating ball becomes waterlogged, and, consequently, fails to completely close the ball-cock of the supply pipe; and, accordingly, it is essential to place the outlet of the overflow pipe, outside the house, in a position where any flow from it is certain to be noticed, thereby directing attention to the defective action of the ball-cock.

Water-Closets.—Two or three generations ago little attention was paid either to the position or ventilation of water-closets, which were sometimes placed well inside a house, and devoid of any communication with the outer air, and with the soil-pipes quite hidden from view in passing through the house. Pan closets were also very generally used, with a large container underneath for the pan to turn down in, and into which it discharged its contents. The sides of the container were inaccessible for cleaning, and only a small portion of their surface was exposed to the flush on lowering the pan; and, consequently, they gradually became very foul with deposit, from which a noxious smell emanated every time the pan was opened. The valve closet, fitting tight to the aperture at the bottom of the basin, was a great improvement upon the pan closet, affording a more direct discharge into the soil-pipe, allowing of the maintenance of a larger amount of water in the basin, which was strictly limited by the depth of the pan in the former case, and, owing to the small size of the valve as compared with the pan, needing less space below for the valve to turn in, thereby leaving a comparatively small space behind the open valve for deposit to take place.

In the improved sanitary practice of the present day, all moving parts have been eliminated from the closet; the basin and the siphon water-trap, made of earthenware or porcelain, are formed in one piece; and the whole of the pipe leading from the basin to the soil-pipe, is fully exposed to the flush from the cistern every time the closet is used, Figs. 195 to 200. The wash-out water-closet was often adopted a few years ago, in which the outlet of the basin is at the side, and the siphon and water seal are out of sight; but to ensure a thorough wash-out, the water retained in the basin has to be shallow; the flush is apt to break up the solids in the basin in washing them out, and its force is somewhat expended at the far side of the outlet before reaching the siphon; whilst the portion of the pipe between the outlet and the water in the siphon, is liable to become somewhat foul without being noticed. Accordingly, this once popular type of closet has been superseded by the forms shown in the illustrations. Though the modern water-closets adopted present considerable variety in detail, they may all be classed under two definite types. In the first type, Figs. 195–197, which is most commonly used as the simplest and cheapest, there is a single pipe leading the flush from the cistern into the basin of the closet, to wash down the contents of the basin past the trap at the back into the drain; and the three closets illustrated are similar in principle, merely differing in form and slight details. Thus the closet shown in Fig. 195 presents a larger water-area than the other two, but a smaller seal; the one in Fig. 196 contains a greater depth of water, and has a deeper seal than the other two; whilst the third, illustrated in Fig. 197, besides having a more projecting-out seat, is arranged so that its junction with the soil-pipe is below the water-level of the trap, thereby preventing any escape of sewer gas into the house through this joint, and ensuring the discovery of any leakage there.

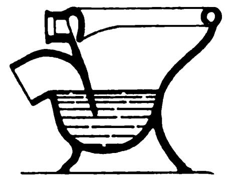
DOULTON'S LARGE AREA WASH-DOWN CLOSET.

Fig. 195.



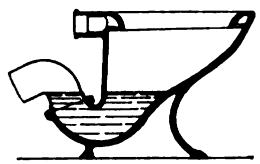
JENNINGS' DEEP-SEAL WATER-CLOSET.

Fig. 196.



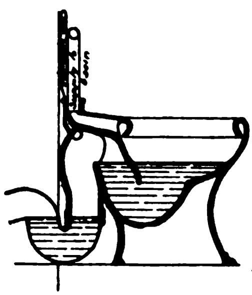
SHANKS' MODERN WASH-DOWN CLOSET.

Fig. 197.



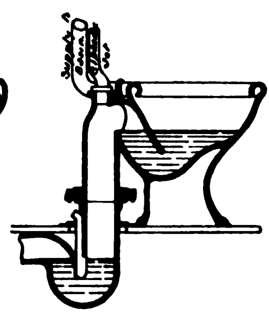
DOULTON'S SIPHONIC WATER-CLOSET.

Fig. 198.



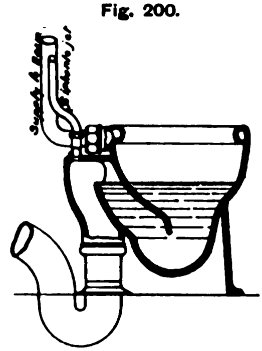
JENNINGS' SIPHONIC DISCHARGE CLOSET.

Fig. 199.



FINCH'S SIPHONIC WASH-DOWN CLOSET.

Fig. 200.



The second modern type of water-closet, Figs. 198 to 200, is furnished with two pipes leading from the flushing cistern, a large pipe serving, as in the closets previously described, for delivering the flush of water into the basin, and a small pipe through which the first flow takes place into the upper part of the siphon, starting the siphoning action, and thereby withdrawing completely the contents from the basin, which is filled again by the later portion of the flush discharged by the large pipe; and this siphon arrangement provides a second trap to the water-closet. In the first illustration of these siphonic closets, Fig. 198, the siphon is above the level of the floor, which is often convenient for the connection with the soil-pipe, and especially where there is a deficiency of fall. The two other siphonic closets shown, Figs. 199 and 200, have the siphon going down below the floor; the last closet,

Fig. 200, has a greater depth of water in its basin, and a deeper seal than the others; whilst all these three closets present an ample water-area.

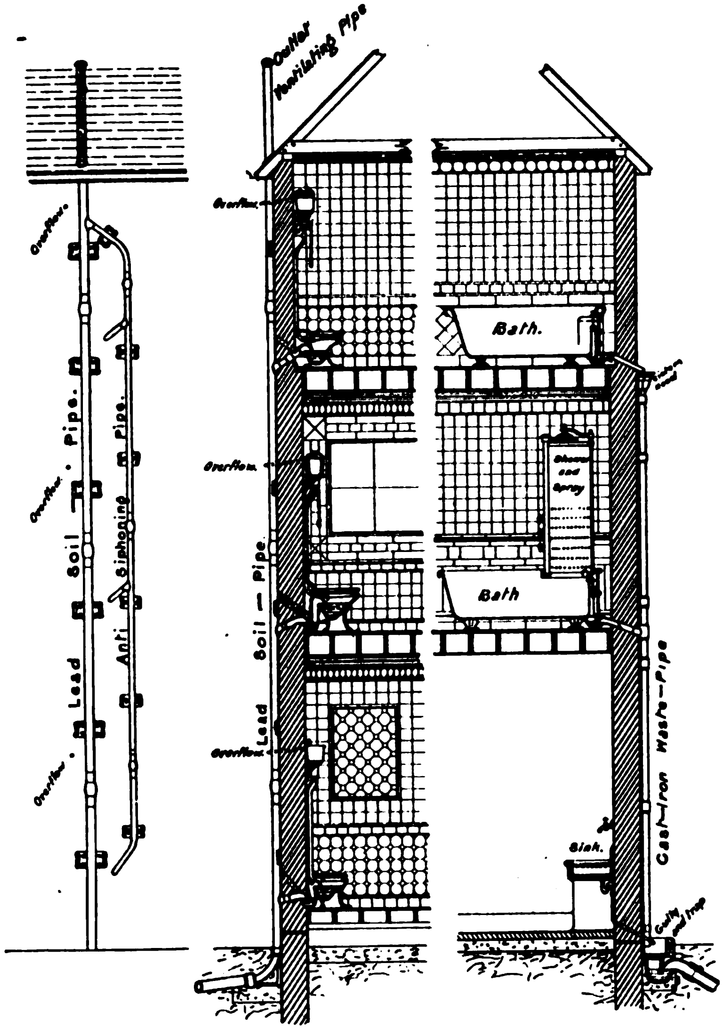
Formerly the mechanism of a water-closet was enclosed in a large wooden casing; but the present practice is to leave the closet with its pedestal completely exposed on a tiled floor, so that any accumulations of dust and dirt may be avoided, and leakage at any point may be at once detected. Moreover, by hinging the wooden seat at the back so that it can be readily raised, the closet can be used as a urinal and for emptying slops. The chamber of the water-closet should be well lighted, and ventilated by a window opening out into the external air, and reaching nearly up to the ceiling; and the introduction of air-bricks in the outer wall near the bottom and top, for ensuring the renewal of air in the chamber, is advantageous.

Arrangement of Soil-Pipes, and their Ventilation.—The soil-pipes of two or more water-closets directly above one another, on different floors of a house, are led into a vertical lead or cast-iron soil-pipe against the outside of the wall, connected with the house drain at the bottom, and continued up beyond the outlet of the highest water-closet to serve as a ventilating shaft, rising well above any window and the eaves of the roof, and away from a chimney-pot to avoid the chance of any fumes issuing from it being sucked down the chimney into the house by a down-draught, Fig. 201. The grease contained in ordinary sewage protects cast-iron pipes laid on an incline from corrosion; but vertical soil-pipes, and more especially ventilation pipes, being exposed to sewer gas, are liable to rust rapidly, if made of cast-iron, and, therefore, must be efficiently protected by a sort of enamel coating, or lead pipes must be used. The outlet of the pipe should be covered over with copper-wire netting or a perforated cap, to prevent birds building their nests in it and blocking it up. Soil-pipes should be made as small in size as consistent with efficiency, a diameter of 3 inches being very commonly adopted; but for a vertical soil-pipe into which several closets discharge, a diameter of $3\frac{1}{2}$ to 4 inches, or even more for buildings of considerable height, would be necessary, both to provide for a larger flow, and to secure more effective ventilation. The ventilating pipe, in prolongation of the vertical soil-pipe, is generally made of the same diameter as the soil-pipe; but a pipe of larger diameter is advantageous for very high buildings, to provide better ventilation.

In the case of high houses, or sets of flats, where three or more water-closets discharge into the same vertical soil-pipe, the rapid fall of the discharge from the upper closets, by creating a vacuum behind it, tends to produce a siphoning action on the traps in the lower closets, withdrawing the water, and thereby unsealing the traps. To obviate this danger, besides increasing the size of the vertical soil-pipe, which

SANITARY ARRANGEMENTS IN A HOUSE.

Fig. 201.



diminishes this action, it is essential to erect an anti-siphoning pipe near the soil-pipe, this second pipe being connected with the soil-pipe of each closet by a branch pipe starting from the soil-pipe beyond the trap, Fig. 201, so as to place the water in each trap in direct communication with the open air, thus preventing the water being sucked out of the traps. This anti-siphoning pipe, merely providing a connection between the lower side of the traps of the closets and the outer air, can be made smaller in size than the adjoining vertical soil-pipe, except in the case of very high houses when it is advisable to make this pipe also large.

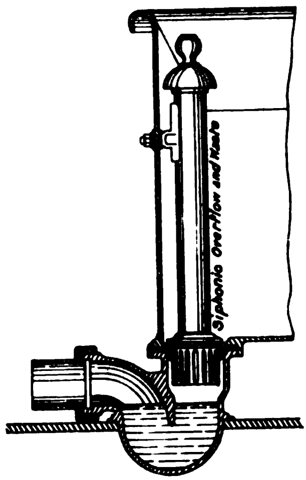
Provisions for the Discharge from Baths and Sinks.—The discharge from baths, consisting generally of a large volume of only slightly soiled water, from which soap or other solids are excluded by a grating at the inlet of the waste-pipe of the bath, is passed, on emerging from the house, into a vertical pipe with a large open inlet, through which it flows down into an open gully, with a trap intervening between the gully and the drain, Fig. 201. Where the bath water is liable to be made very dirty, it should be discharged into a closed pipe, corresponding to a soil-pipe, with a ventilating pipe rising from it above the eaves, and discharging at the bottom into a gully as in the former case. The emptying of the bath is commonly effected by raising a vertical pipe, open at both ends, enclosing the inlet to the waste-pipe when lowered on to its seat. When this pipe is placed within the bath, the top of the pipe being level with the water of a full bath, the overflow takes place directly down this pipe through its upper orifice, Figs. 202 and 203. Where, however, this arrangement for the discharge is put outside the bath, a pipe leads the water from the bottom of the bath to the discharge valve; and by providing an outer casing for the movable, vertical pipe shown in Fig. 203, the upper orifice still serves for discharging the overflow, which comes in by an outlet pipe from the bath, and flows up between the outer casing and the pipe to the orifice at the top on a level with the full bath. A trap placed in the outlet pipe just below the bath, Figs. 202 and 203, shuts off the bath outlet and the overflow inlet from the waste-pipe. When a bath is placed upon a wooden floor, the precaution is sometimes adopted of placing a lead tray, or safe, under the bath to receive the water splashed over; and where the amount of water spilt is liable to be large, an overflow pipe passing through the outside wall drains off the water. The same arrangement is also occasionally resorted to, in similar conditions, under water-closets.

The separation of small waste-pipes, such as are used for baths, sinks, and basins, from any drain or sewer, and the maintenance of the seal of water, are of special importance; for if, for instance, the use of a bath is discontinued for some time, the small amount of water in the trap is liable to evaporate and open the trap; and then if the outlet pipe from the bath is connected directly with the drain or sewer, the pipe acts as a ventilator, conveying sewer gas into the house.

The discharge from slop sinks must be guarded by the same precautions as the discharge from water-closets; but modern water-closets, with

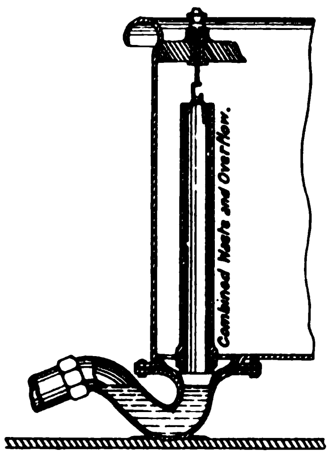
SHANKS' BATH WASTE AND OVERFLOW DISCHARGE.

Fig. 202.



JENNINGS' BATH WASTE VALVE AND OVERFLOW.

Fig. 203.

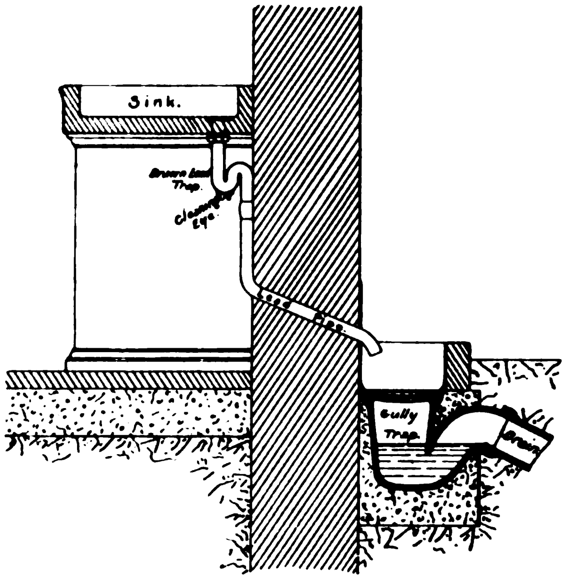


a lifting seat, generally serve conveniently for this purpose. The sinks usually provided in a well-appointed house, comprise a housemaid's sink upstairs for cleaning bedroom utensils, a pantry sink for washing glass, china, and plate, and a scullery or kitchen sink for washing up dishes, plates, and all kitchen articles. The sinks should be placed alongside an outside wall, and under a window, so that the waste-pipe may be as short as possible, and ample light afforded for the washing operations; and taps for hot and cold water are generally provided. The housemaid's and scullery sinks may be advantageously made of glazed stoneware; but the pantry sink is usually formed of wood lined with lead, owing to the less likelihood of the fragile articles washed in the pantry being broken by coming in contact with these materials, than with stoneware.

The same precautions of trapping the waste-pipe near its inlet, and disconnecting this pipe from the drain, are adopted for these sinks as for the waste from a bath; and an illustration of the arrangements provided for ordinary sinks is given in Fig. 204. In consequence, however, of the large amount of grease which usually passes down from the scullery sink in a warm, melted condition, and would be liable in becoming solid on cooling to adhere to the sides of the drain, special arrangements have been resorted to in order to intercept this grease before it reaches the drain, or to render it perfectly solid previously, so that it may be carried away floating by the flow of sewage. The

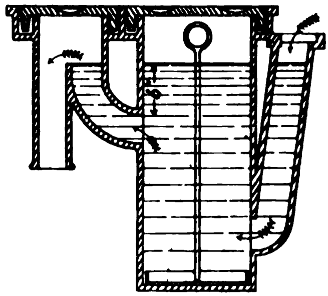
SINK WITH TRAPS AND GULLY.

Fig. 204.



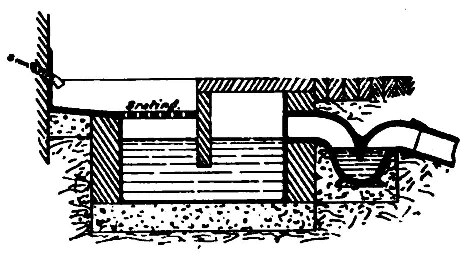
COL. MOORE'S GREASE TRAP.

Fig. 205.



DIP GREASE TRAP.

Fig. 206.



grease is solidified rapidly, on being discharged with the washings into the open gully, by bringing it in contact with a large volume of cold water; and rising to the surface, it is removed from time to time by raising an intercepting tray fitting into the sort of catch-pit containing the water. A recent, improved type of this arrangement is shown in Fig. 205,[1] in which the discharge from the sink, taking place near the bottom of the deep catch-pit, or trap chamber, is brought in contact with a considerable volume of cold water filling the chamber, so that the grease becoming cooled in passing up through the water, is solidified and floats on the surface; whilst any heavy solid matters fall to the bottom of the catch-pit. Then, by lifting the strainer from the bottom of the receptacle, the heavy solid matters and the floating grease can be periodically removed. Another form of grease trap is shown in Fig. 206, in which the melted grease from a scullery sink is solidified by passing through cold water in the trap, and rising to the surface of the water beyond the actual trap, is discharged in a solid state into the drain, and carried away by the flow; but this dip trap is more commonly used in connection with the discharge of surface water into sewers. Where the grease from scullery sinks is not removed, it is expedient to provide a flushing cistern in the scullery, so that the waste-pipe, gully, and adjacent drain may be frequently scoured out with clean water.

The water from a laundry must be dealt with in the same manner as the waste water from sinks, being led straight through the adjoining outside wall, and discharged into an open gully leading through a trap into the drain. Sometimes, as an additional precaution against the admission of sewer gas into the waste-pipe, and its escaping from thence into the house, the mouth of the gully is put a little away from the outlet of the waste-pipe, the discharge from the waste-pipe being led along a short, sloping, open, converging channel into the gully, which is termed a "self-cleansing channel gully," the short channel being kept clean by the flow.

Drainage of Stables.—The floor of a stable should be paved with some hard, non-absorbent material, not liable to become slippery, which latter forms the only objection to the use of granite sets, vitrified bricks, and asphalt; and the most suitable flooring appears to be a bed of Portland cement concrete, $2\frac{1}{2}$ to 3 inches thick, placed upon a firm foundation layer of small rubble, coarse gravel, broken bricks, or other hard, dry materials. The concrete floor of the stalls should be grooved to give a better foothold for the horses, and to facilitate drainage; and the best arrangement consists of a series of diagonal, parallel grooves on each side, leading to a shallow channel running down the centre of each stall. The drainage from the stalls may be led to the outside by underground drains, or suitably formed surface channels with a good fall. The latter system is to be preferred, both on account of the general

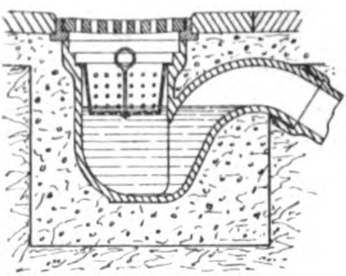
[1] "Sanitary Engineering," Col. E. C. S. Moore, R.E., p. 302, fig. 260.

inexpediency, on sanitary grounds, of laying drains inside a building which is commonly lived in, and also because these drains are particularly liable to be obstructed by straw, dung, and other dirt.

The gully into which the surface drainage of the stalls discharges before entering the drain, must be furnished with a grating at the top; and the inlet of the drain-pipe leading from it should be protected by a grating, and the end of the pipe turned down, to prevent straw and other floating refuse from entering the drain; and the catch-pit at the bottom, in which the heavier matters settle, needs frequent clearing out. Another form of stable gully, shown in Fig. 207, provides against straw and other refuse entering the drain, by means of a bucket-shaped, movable strainer which retains the solid materials. As this accumulation of refuse is particularly noxious in hot climates, a special form

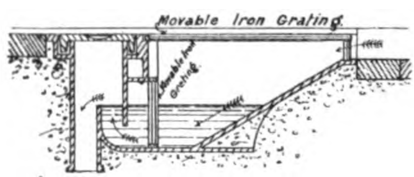
STABLE GULLY WITH MOVABLE STRAINER.

Fig. 207.



COL. MOORE'S STABLE GULLY AND TRAP.

Fig. 208.



has been designed for such conditions, as shown in Fig. 208, and adopted with success in Bermuda.[1]

House Drains under Buildings.—In the crowded parts of towns it is frequently impracticable to place all the water-closets, sinks, and other conveniences close to the outer wall adjoining the street sewer; and the carrying of a drain under the house from back to front becomes a necessity. In such cases, the drain under the building should consist of a straight line of iron pipes, laid on a solid bed of concrete and jointed with lead, with inspection chambers at suitable places,[2] Figs. 209 and 210. Stoneware pipes have been often used under these conditions; but iron pipes are stronger, and being made in longer lengths and having more reliable joints, are less liable to leakage,

[1] "Sanitary Engineering," Col. E. C. S. Moore, 2nd edition, p. 379, fig. 302.

[2] "Modern Sanitation," W. D. Scott-Moncrieff, *Professional Papers of the Corps of Royal Engineers*, 1897, vol. xxiii. p. 173, and plates.

and are more readily made to any shapes or bends required, so that in every respect they are preferable to stoneware pipes for this special purpose, though more costly.

CAST-IRON DRAIN UNDER HOUSE.

Fig. 209.—Plan.

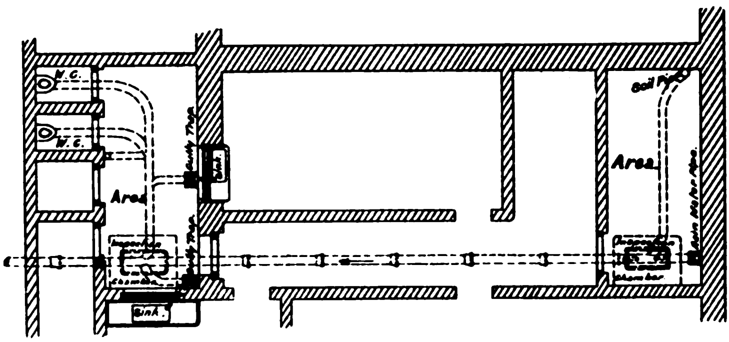
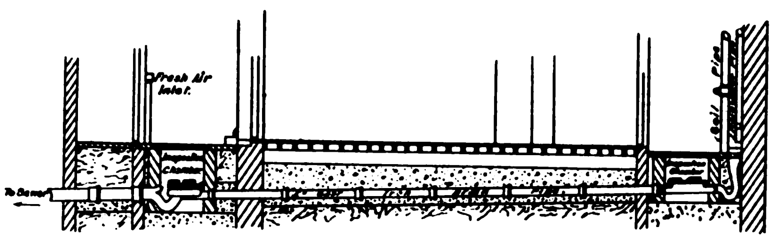


Fig. 210.—Longitudinal Section.

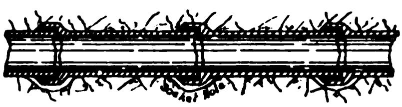


Drains from Houses to Sewer.—Where the houses in a town abut on the street, the main drain from each house is led straight to the sewer running under the centre of the roadway, after passing through a siphon trap, with an air inlet or disconnecting inspection chamber at its upper end, placed in the area or cellar adjoining the house, Fig. 210.

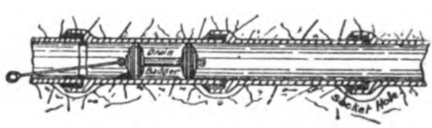
In the case of houses standing in their own gardens, the drains from the soil-pipes and gullies are led in the most suitable manner into a main drain, which conveys the whole discharge to the public sewer. These drains are generally made of stoneware pipes with cement joints, laid on a firm foundation, from 4 to 6 inches in diameter, with specially made blocks, formed of cement mortar surrounding semi-circular stoneware channels, at every bend and junction, with a brick lining carried up to the surface, and covered over with a cast-iron cover so as to be readily accessible for periodical inspection. The drains should be given a sufficient fall to impart a velocity of flow adequate to prevent the occurrence of deposit, varying inversely with the size of the pipes. The proper rate of flow has been estimated at from 3 to 4 feet per second for small pipes, necessitating a fall of 1 in 36 to 1 in 20 for 4-inch pipes, 1 in 70 to 1 in 40 for 6-inch pipes, and 1 in 130 to 1 in 75 for 9-inch pipes. For house drains, however, where the flow is generally small and intermittent, the fall has to be made greater than for public drains of similar dimensions; but a very rapid flow is liable to injure the joints. The diameter of the pipes is increased at bends and junctions to allow for the obstruction to the flow at these places. For scraping off any cement which may have oozed out from the joints into the pipes in laying them, and for clearing out any deposit or obstruction, a sweeper termed a "badger," consisting of two circular wooden disks connected by a flexible

Fig. 211.

A.—DRAIN-PIPES WITH CEMENT PROJECTIONS INSIDE.

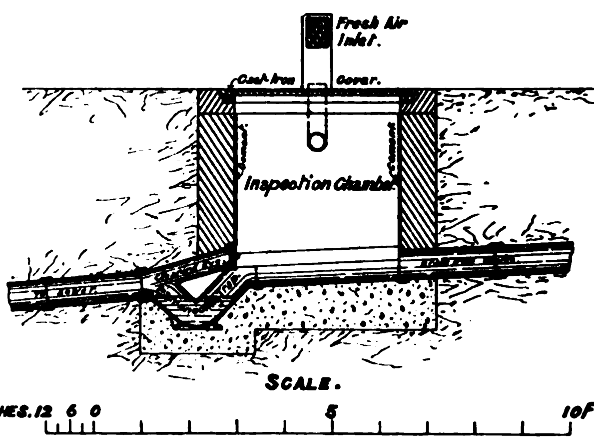


B.—DRAIN-PIPES AFTER USING BADGER.



DISCONNECTION OF HOUSE DRAIN FROM SEWER.

Fig. 212.

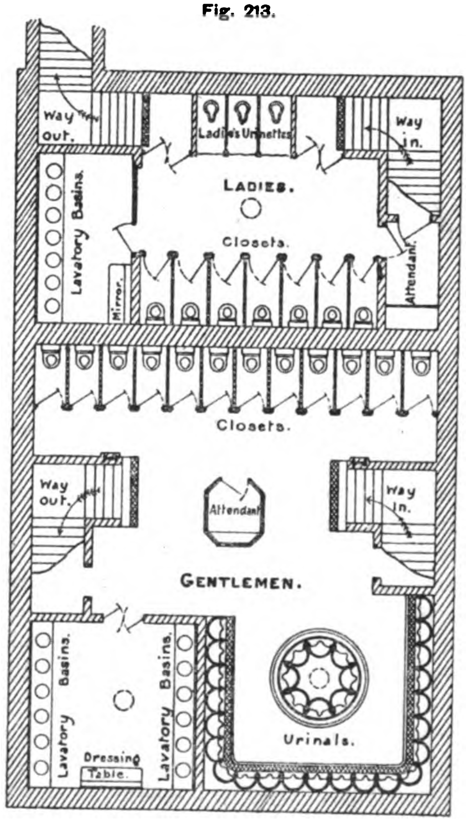


piece, and provided with indiarubber edges fitting closely to the pipes, has been devised, which is drawn through the drain, and removes any projecting cement or other obstruction; compare sections **A** and **B**,

Fig. 211. The pipes are always laid with the socket facing the direction of the flow; and, consequently, the laying is commenced at the lower end. Iron pipes are substituted for stoneware pipes in bad ground, and also where the drain is exposed to considerable pressure, either by being laid some depth below the ground, or near the surface across a roadway.

AN UNDERGROUND CONVENIENCE.

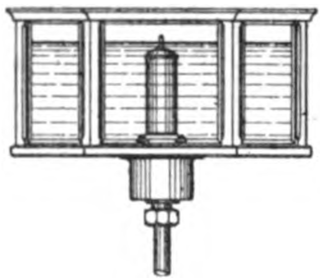
Fig. 213.



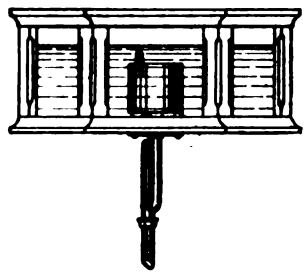
The main drain, before crossing the boundary of a suburban property, on its way to the sewer in the adjacent roadway, is effectually cut off from the sewer by a disconnecting manhole, or inspection chamber, with a siphon trap at its lower end, to which a cleaning eye above is added with advantage for removing any obstruction in the branch sewer below the trap, Fig. 212. By this arrangement, all danger of sewer gas from the public sewer entering the house is obviated.

Underground Conveniences.—Quite recently several underground conveniences, comprising urinals, water-closets, and lavatories, have been constructed at centres of traffic in London for men and women, thereby supplying a much-needed want, for which formerly the only provision consisted of a few ugly, dirty urinals erected at haphazard at a few places alongside the foot-pavements. The new constructions, besides being placed out of sight underground, except the railings guarding the entrances and exits, are commodious, clean, well-lighted places, presenting the greatest possible contrast to the former inadequate erections, and serving as models which other European capitals might imitate with advantage. A typical example of these underground conveniences is given in Fig. 213, showing the general arrangements adopted in London.[1] The urinals are furnished with automatic flushing tanks, which discharge the contents of the tank, ranging from about 3 to 8 gallons of water, at regular intervals, by siphonic action when the water flowing slowly into the tank reaches a definite level, Figs. 214 and 215.

FINCH'S AUTOMATIC FLUSH TANK.
Fig. 214.



DOULTON'S AUTOMATIC FLUSH TANK.
Fig. 215.



Refuse Destructors.—The refuse collected in towns from houses, shops, factories, and other places, though very variable in quality according to the conditions of the locality, always contains a certain amount of putrescible matter, and therefore when tipped on land is liable to become a source of disease. The deposit also of the refuse in the sea, though much less objectionable on sanitary grounds, can only be resorted to by seaside towns, or by towns conveniently connected with the sea by water-carriage; and except under favourable conditions, accompanied by special precautions, this deposit is liable to be brought on to the shore by storms and tidal currents, and to create shoals, unless carried well out to sea and into deep water. Accordingly, the burning of the refuse in furnaces to get rid of the combustible portions, and to convert the remainder into innocuous clinker, has been increasingly resorted to in recent years by towns for disposing of their refuse without creating a nuisance; and the clinker, if hard enough, is utilized for making mortar, bricks, paving-slabs, and also in place of coke for forming bacterial beds.

In the earlier refuse destructors, the combustion was effected slowly with natural draught, at a temperature which was inadequate to destroy thoroughly the offensive portions of the refuse; and noxious fumes were emitted from the chimney, occasioning a nuisance; whilst the clinker obtained was soft and unsaleable. By introducing, however, forced draught by driving air into the furnace with fans, or, where the

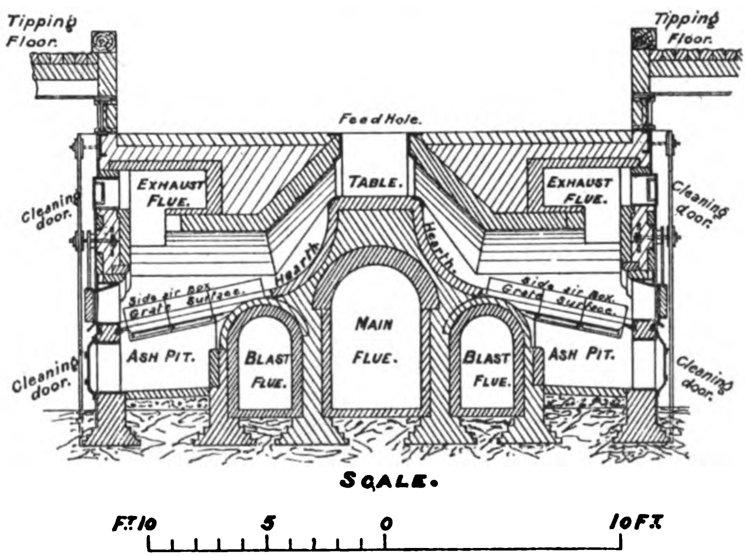
[1] "Finch's Sanitary Engineering Company. Illustrated Catalogue of Underground Conveniences," p. 12.

refuse has a sufficient proportion of combustible materials to maintain a high temperature, by jets of steam converted when exposed to great heat into water-gas by decomposition, the noxious portions of the refuse are completely consumed, the gases passing up the chimney are inoffensive, and a hard, vitrified clinker is produced, with a temperature that can be maintained at 1600° F., and may rise to over 2000° F. Town refuse has been roughly estimated to consist, on the average, of one-third of combustible matter by weight, one-third of moisture, and one-third of incombustible matter converted into clinker;[1] and though its actual composition varies considerably in different towns, there is generally a sufficient proportion of cinders in the refuse to enable its combustion to be effected at a high temperature without the addition of fuel for the purpose.

Two forms of destructors are in use, and two methods of feeding the furnace are employed. Where a large number of cells are required, they are generally placed back to back, and are fed from the top, the refuse being tipped from both sides on to a floor above the furnaces,

HORSFALL'S BACK TO BACK TOP-FED DESTRUCTOR.

Fig. 216.



and deposited through a central hole on to a table, from which it is pushed as required into the cells on each side,[2] Fig. 216. Though this

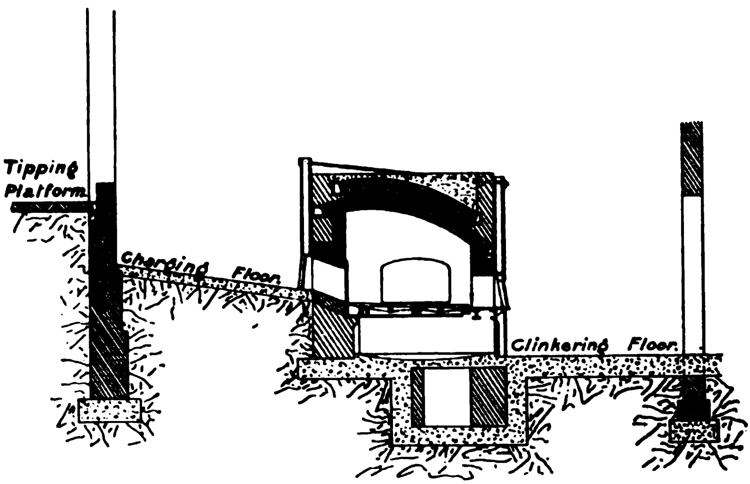
[1] "Refuse Furnaces," George Watson, *Proc. Inst. C.E.*, vol. cxxxv. pp. 300–301.
[2] *Ibid.*, plate 8, fig. 7.

is a convenient type for a large destructor in saving space, it has the disadvantages of heating the refuse spread on the floor above the furnaces, producing dust and offensive smells, and of feeding the cells with heaps of refuse not evenly spread over the grates, and therefore liable to be unequally clinkered. Moreover, on a level site, the platform on to which the refuse has to be raised is about 18 feet above the ground, and, consequently, necessitates a long approach road for the heavily laden carts to mount up to it, or a lift where space is not available for a road.

With cells in a single row, the feeding is generally effected by raising the refuse from an adjacent floor by shovels, and spreading

MELDRUM'S SINGLE ROW BACK-FED DESTRUCTOR.

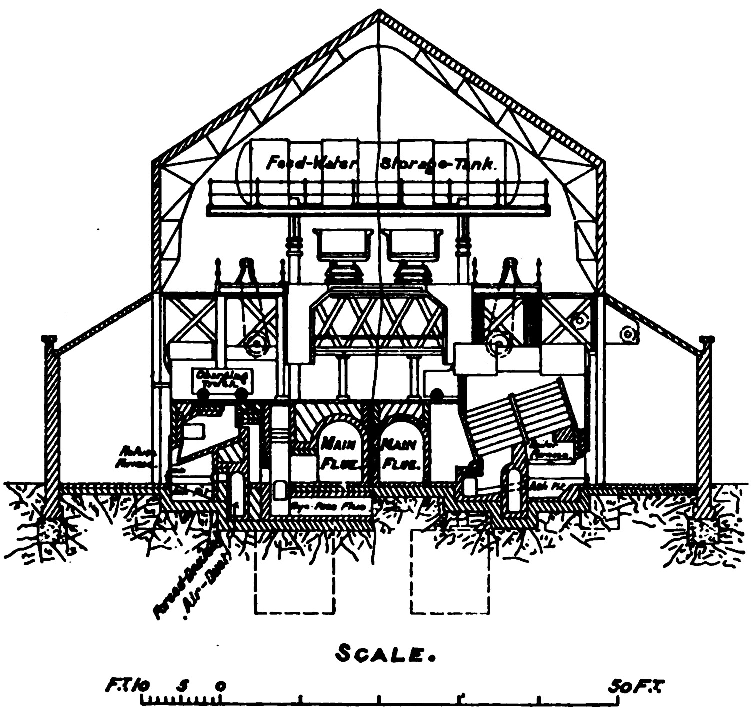
Fig. 217.



it over the cells as required. This feeding may be done from a charging floor at the back of the furnace, on to which the refuse is tipped from a platform further back, as shown in Fig. 217; or it may be accomplished from the clinkering floor, on to which the refuse is discharged from a storage hopper placed parallel to the cells a little above the floor, by shovelling the refuse direct into the cells from the front. In both these methods, the refuse is tipped on to a cool floor, avoiding the dust and fumes resulting from the central, top feeding; and the spreading of the refuse over the grates is much better effected, and under more favourable conditions, though with somewhat greater labour in raising it. Since, in the back-fed type of cells, the charging floor is a little higher than the grates, the labour of raising the refuse is reduced to a minimum, being shovelled on to the sloping drying hearth adjoining the floor,[1] Fig. 217; and though the refuse has to be dragged forward and spread uniformly over the grate by a man standing on the clinkering floor, this labour is less onerous than in the case of the top-fed type of furnace, and is much more under control and supervision. When the cells are fed from the front, the labour of lifting and distributing the refuse by shovelling is greater; but the work of raking

COMBINED REFUSE DESTRUCTOR AND BOILERS.

Fig. 218.



and spreading the refuse is dispensed with. Moreover, the refuse is at once deposited, in small quantities at a time, in the most active part of the cells, and is therefore rapidly consumed in the most advantageous manner; whilst the feeding and distribution are under perfect control.

Whilst keeping in mind the primary object of refuse destructors, namely, to consume the noxious portions of the rubbish collected in towns, it has naturally been sought to utilize the heat generated in the

[1] "Refuse Disposal and Power Production," W. F. Goodrich, pp. 42 to 44.

process as a source of power for electric lighting and other purposes. This result has been attained by combining boilers for producing steam with refuse destructors,[1] as shown in Fig. 218, the amount of power thus developed depending on the special calorific value of a given quantity of the refuse consumed, which, though varying greatly at different places, remains generally fairly constant at each locality. The utilization of the burning of refuse for the production of power for electric lighting, can only be satisfactorily effected with the ample refuse of large towns; but a moderate amount of power can be obtained from the destruction of the refuse of smaller towns, which can be applied to other purposes.

Refuse destructors unquestionably afford the most sanitary and effectual means for the disposal of house refuse; and though the heat generated by them can only be economically used for the production of power where the amount of refuse is fairly large and of good quality, they can be advantageously adopted by small towns, and even villages, for ensuring the prevention of a nuisance and a danger to health; and it is to be hoped that before long the system will constitute the universal method for dealing with refuse.

[1] "Combined Refuse Destructors and Power Plants," C. N. Russell, *Proc. Inst. C.E.*, vol. cxxxix. p. 186.

CHAPTER XIII.

SEWERAGE.

Sewers for Towns—Difficulties concerning the Disposal of Rainfall; fluctuations in flow produced by rainfall—Separation of Rainfall from Sewage; advantages, practical difficulties, adoption of system under certain conditions, value of system—Conveyance of Sewage and Rainfall in Sewers, reasons for combined system, need for storm overflows and their objections, partially combined system—Materials employed for Sewers; conditions affecting choice, joints for stoneware and iron pipes—Forms of Brickwork or Concrete Sewers; advantage of egg-shaped sewers and their construction, circular sewers, peculiar forms adopted in Paris, intermediate forms, rainwater conduit and sewer in single structure—Fall of Sewers; velocities of flow, formula for fall, formula of discharge—Cleansing of Sewers; flushing tanks, methods of flushing in Paris and falls of sewers—Provision against an Excessive Fall in Sewers; introduction of steps—Compensation for Inadequate Fall; methods adopted—Ejectors; Shone system described, advantages—Suction system; description of Lienur system—Automatic Sewage Lift; principle, working, different arrangements—Laying of Sewers; in trenches, in headings, as tunnels—Gullies; catch-pits, trap, receptacle for refuse, special provisions in steep streets—Manholes; positions, lamp-holes, arrangements for flushing sewers—Ventilation of Sewers; to remove sewer gas, through manholes, importance in combined system of drainage, difficulties, by open shafts and with lamp, mechanical, specially necessary in high places.

WHATEVER may be the method of disposal adopted for the sewage, the discharge collected from the houses of the town or district, has to be conveyed to its destination by a series of branch sewers leading into a main sewer. The sizes of these sewers depend upon the estimated maximum discharge in each case, and the available fall, which varies with the locality; and, as in the case of house drains, where the fall is ample, it must be restricted within limits not liable to impart a velocity of flow which might prove injurious to the sewer, varying inversely with the volume discharged, and, consequently, with the size of the sewer. On the other hand, in places where the fall is small, as the velocity of flow must be sufficient to prevent the occurrence of deposit, it may be necessary to resort to measures for obtaining artificially an increased fall, or to provide for a periodical flushing of the sewers.

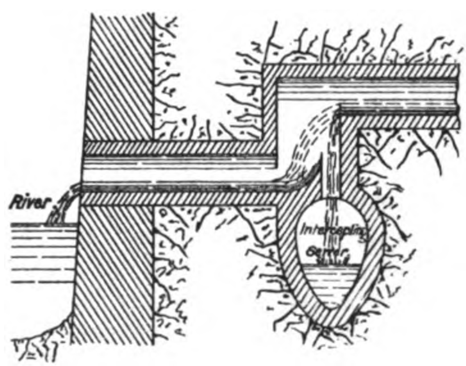
Difficulties concerning the Disposal of Rainfall.—Before preparing a design for the sewerage of any town or district, it is necessary to decide how the rainfall flowing off the area under consideration is to be dealt with. In estimating the discharge to be provided for, it may be reckoned that the sewage proper corresponds approximately to the water-supply, and therefore, though differing in volume per head according to the habits of the population, it remains fairly constant at any given locality, with merely the ordinary daily and seasonal fluctuations; and, consequently, there would be no difficulty in determining the sizes of the sewers to carry off this definite discharge of sewage. If, however, the rainfall has to be disposed of in combination with the sewage, a great fluctuation in the discharge is at once introduced, depending on the weather; and in heavy storms, the volume to be conveyed away may amount to many times the volume of the sewage alone, and during short periods may exceed the flow that, in view of the cost, the sewers can be reasonably made large enough to deal with. Endeavours have, accordingly, been made to obviate these difficulties by excluding the rainfall and subsoil waters from the sewers; but this arrangement, though apparently simple in principle, can only be rigidly carried out under favourable conditions, is only capable of being partially effected in many cases, and is often inapplicable for the sewerage of large towns.

Separation of Rainfall from Sewage.—The exclusion of the rainfall from the sewers, as far as practicable, is in every respect advantageous; for not only does this render the sewage to be discharged a comparatively moderate and tolerably constant quantity, with merely the ordinary fluctuations at different periods of the day, thereby enabling fairly small sewers to provide sufficiently for the flow, but also the volume of sewage to be dealt with in any system of sewage disposal is kept within reasonable limits, and the value of the sewage for agricultural purposes is enhanced. This separate system, as it is termed, may be carried out in suburban districts, especially where the soil is porous, by leaving the rainfall to follow its natural methods of disposal, either by sinking into the soil, or running into ditches or watercourses, or by being led into underground tanks for supplementing the water-supply. By this means, the sewers have only to fulfil the object for which they have been provided, namely, to carry off the sewage alone; whilst the subsoil water is kept out of the sewers, either by making their joints watertight, or by laying alongside the sewers, during their construction, a line of drain-pipes for conveying the subsoil water to the nearest watercourse or river, which latter arrangement, besides facilitating the laying of the sewers, affords the additional advantage, where houses are situated on damp, low-lying land, of lowering the underground water-level.

In crowded districts, however, and towns, the removal of the rainfall has to be provided for; and the rainwater flowing off the back yards and back roofs of small houses, and from the streets of towns with a considerable traffic, is liable to be sufficiently polluted to require to be treated as sewage and discharged into the sewers. As the first flow off of rainwater after a dry period is specially exposed to pollution, by its washing away the accumulated dust and dirt, it is sometimes sufficient to discharge the first washings from rainfall into the sewers, and to allow the subsequent flow of fairly pure rainwater to pass direct to the nearest watercourse. This can be accomplished by the arrangement shown in Fig. 219, where the sluggish first flow of the soiled rainfall falls straight down the vertical channel into the sewer;[1] whereas the subsequent larger and more rapid flow of tolerably clear rainwater passes over the orifice into the channel beyond, leading it to the river.

SEWER RECEIVING FOUL FIRST FLOW OF RAINWATER.

Fig. 219.



To keep the rainfall out of the sewers where, in thickly populated places, it is deprived of its natural means of dispersal, special provision has to be made for the discharge of the uncontaminated rainfall by a separate system of drains. The construction of this duplicate drainage for sewage and surface water is necessarily very costly; but in cases where a complete rearrangement of the sewerage system is found expedient, the old, abandoned, and defective network of sewers, which have generally been made unnecessarily large for conveying the sewage alone, can be utilized for the discharge of the rainwater of the district, and the advantages of the separate system secured at a reasonable cost. When this convenient and economical mode of disposing of the rainfall is not available, the choice lies between laying down two separate sets of drains, one for the sewage and the other for the rainwater, and the

[1] "Sanitary Engineering," Baldwin Latham, 2nd edition, p. 460, plate 17, fig. 9.

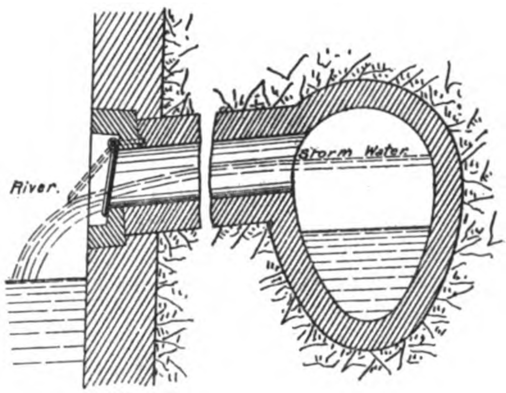
adoption of the combined method, by which the sewage and the rainfall are carried away together in a single network of sewers, of which the drainage systems of London and Paris furnish notable examples. The only other disadvantage, beside cost, which might be urged against the separate system, is that when the available fall is small, the small size of the sewers and the small flow might lead to deposit; but this can be provided against by periodical flushing; whereas the other remedy sometimes proposed, of allowing a certain proportion of the rainfall to discharge into the sewers, would fail in dry weather when it is most wanted.

The duplicate system of drains is not only the most satisfactory for the already mentioned reasons of providing a moderate and fairly regular flow of sewage, facilitating considerably, and materially reducing the cost of, its disposal, but it is also a much more sanitary method; for the storm overflows which relieve the sewers in the combined system during periods of heavy rainfall, pass crude, though considerably diluted, sewage into the river or estuary into which they discharge. Accordingly, it is very important that the separate system should be adopted in all cases where the occasional introduction of crude sewage into a river is liable to be injurious, or to cause a nuisance, provided the cost of the duplicate system of drains does not prohibit it.

Conveyance of Sewage and Rainfall in Sewers.—In many cases undoubtedly, circumstances prevent the complete separation of the rainfall from the sewage, as previously pointed out; and whereas a variable flow is introduced into the sewers, necessitating a considerable increase in their dimensions, the second network of drains can only deal with the rainwater flowing off the roofs of the houses, gardens, and open spaces. These considerations, combined with the large cost of a duplicate set of drains, and the comparative simplicity of a single set of sewers for the whole drainage, have led in many instances to the adoption of the combined system, notwithstanding the manifest advantages of the separate system where it can be fully carried out without involving an undue expense. The combined system, however, in its turn 'would often result in an excessive cost if the sewers had to be made large enough to carry off, in addition to the sewage, the rainwater flowing off the drainage-area in exceptionally heavy storms occurring at considerable intervals apart. Consequently, storm overflows have been introduced, so that when the flow in the sewer rises above a fixed height, it is discharged through openings provided at the side of the sewer, Fig. 220; and the excess of the discharge passes out of the sewer, and is led into the neighbouring river, estuary, or the sea. This discharge of foul water, if properly arranged, may be unobjectionable in the sea or a tidal estuary; but it can only be allowed in a tidal river where the sewage is not liable to deposit on the foreshores or at the tidal limit; and it is inadmissible in a river above the tidal limit, unless the dilution

STORM-WATER OVERFLOW.

Fig. 220.



is very large, and the river is not used for water-supply by towns situated on its banks a moderate distance lower down.

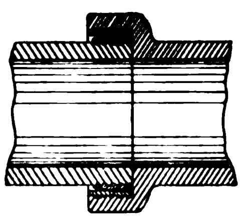
In cases where the absolutely separate system cannot be adopted, a partially combined system may prove advantageous, the fouled water from back yards and streets being discharged into the sewers, and the fairly clean rainwater conveyed separately to the nearest river or water-course, provided this portion of the rainfall constitutes a considerable part of the whole. The amount of rainfall that has to be carried off separately, or partly separately and partly with the sewage, or wholly in combination with the sewage, depends on the maximum rainfall of the district, the nature of the strata in the parts not paved or covered with houses, and the general inclination of the district, steep slopes with impermeable strata and paving bringing the rainfall more rapidly and in greater proportion into the drains and sewers.

Materials Employed for Sewers.—Stoneware circular pipes are commonly employed for small sewers, not exposed to danger of fracture from settlement in soft ground, pressure of earth, or, near the surface, from heavy vehicular traffic; and they must be encased in concrete, or iron pipes substituted for them, under any of these unfavourable conditions. The suitable limit of size of stoneware pipes is about 18 inches diameter, beyond which brickwork or concrete is usually employed for sewers; whilst iron is used under special conditions.

The joints of stoneware pipes are ordinarily made with cement, by first inserting some tarred hemp into the socket to prevent the cement oozing out into the interior of the pipe, and then putting in the cement; but asphalt joints, formed by pouring heated asphalt into the socket, after encircling the spigot with a ring of indiarubber, or canvas filled with cork shavings, to prevent the asphalt entering the pipe, have been very extensively used in Germany in preference to cement joints, as more impermeable and elastic, and being, moreover, unaffected by frost and readily loosened by heating them for separating the pipes. Special joints have also been contrived for facilitating the laying, and at the same time ensuring a perfectly watertight joint, not liable to fracture by a slight settlement or by changes of temperature, by fastening on to the inside of the socket and the outside of the spigot of the pipes, rings of a mixture of ground stoneware, sulphur, and tar, or other flexible composition. The pipes being supplied with these rings on them, it is only necessary to tar or grease their surfaces before connecting the pipes, when a slight twist after the pipes are in position completes the formation of the joint, Fig. 221. Sometimes impermeability of the joints is still further secured by inserting cement between two sets of flexible rings, Fig. 222, or by an outer filling of cement, Fig. 223.

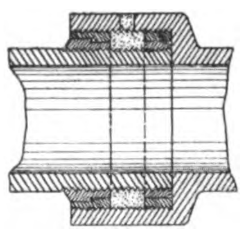
DOULTON'S SELF-ADJUSTING JOINT.

Fig. 221.



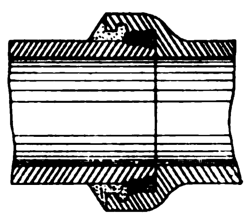
HASSALL'S SAFETY JOINT.

Fig. 222.



JENNINGS' DOUBLE SEAL JOINT.

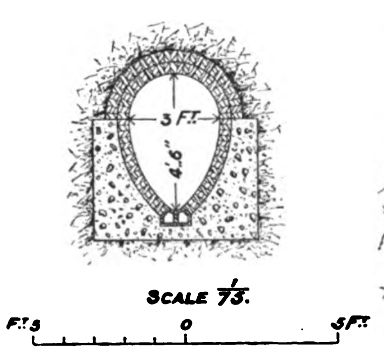
Fig. 223.



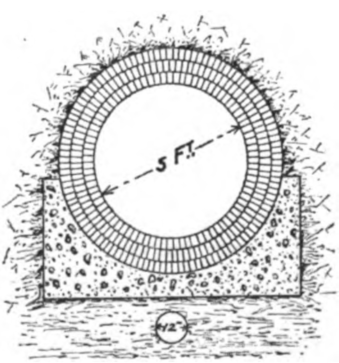
Cast-iron pipes for sewers, formed with sockets and spigots in longer lengths than stoneware pipes, are jointed with lead and caulked; but where these pipes are exposed to internal pressure, as in the case of an inverted siphon under a river, tending to draw the joints apart, flanged pipes should be adopted, connected together by bolts, the joint in this case being made by an indiarubber ring, or by the close fitting of the planed ends of the pipes when the bolts are screwed up.

Forms of Sewers made of Brickwork or Concrete.—An egg-shaped section is generally resorted to for the smaller class of branch sewers which are too large to be constructed of stoneware pipes, and is even sometimes used for large main sewers. The advantage of this form is that it affords a considerably larger hydraulic mean depth for small flows than a circular section of the same size, and, consequently, a greater velocity for the same fall, and less liability for deposit to occur; and, therefore, it is specially suitable for the discharge of very variable flows, and particularly where at times the flow is liable to become very small, Fig. 224. This modern egg-shaped form presents a remarkable contrast to the old flat-bottomed form with vertical side walls and a semi-circular arch at the top, as exemplified by the Cloaca Maxima, still visible in Rome, and many far more recent drains and sewers extending into the last century, a design which no doubt originated in the covering over of streams as towns enlarged, and eventually utilizing them for sewers. A very interesting comparison of the old system with the new one was furnished, only a few years ago, when an egg-shaped sewer was built inside the large flat-bottomed sewer running down the centre of Great George Street, Westminster, after removing the covering arch. These egg-shaped sewers are generally constructed of brickwork set in cement mortar, and rendered over inside with cement, the lower portion, up to the springing of the arch, being embedded in concrete in traversing soft or treacherous ground; whilst the construction of the base is facilitated by inserting an invert block of stoneware or blue brick at the bottom, with a flat base, on which the brickwork on each side is built up, Fig. 224.

EGG-SHAPED SEWER.
Fig. 224.



BARREL BRICK SEWER.
Fig. 225.

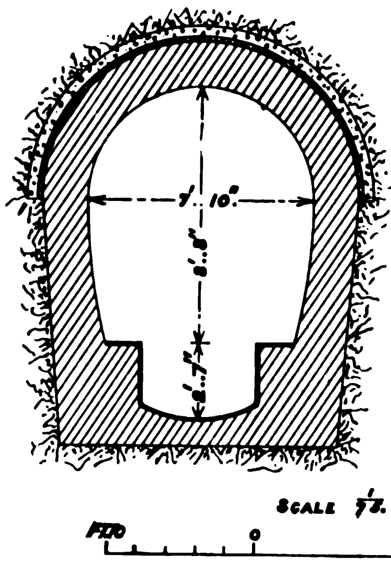


Large sewers are usually made barrel-shaped, with two or more rings of brickwork according to their size, the nature of the ground, and the pressure of earthwork to which they may be exposed; and in passing through bad ground, the lower part is embedded in concrete, as shown in Fig. 225, in which also a pipe drain for subsoil water is shown underneath the sewer, which is specially serviceable in carrying the water away, during construction, from the trench in which the sewer is being laid. The advantages of the circular section are, that it gives the maximum discharging capacity in proportion to the amount of materials used, and is more easily constructed.

Circular and egg-shaped sewers are the ordinary forms adopted; but various other shapes have been occasionally employed. Thus in Paris, the peculiar types shown in Figs. 226 and 227, are commonly

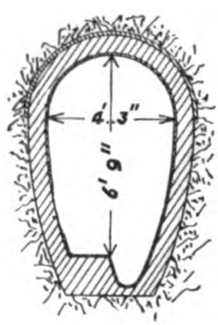
LARGE PARIS SEWER WITH BENCHES.

Fig. 226.



SMALLER PARIS SEWER WITH A SINGLE BENCH.

Fig. 227.



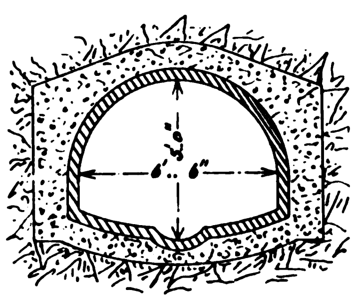
found,[1] the large sewers having a bench on each side of the central channel serving for the discharge of the sewage in dry weather, along which the workmen inspecting and cleaning out the sewer can walk; whilst the smaller sewers with their narrow side channel and single bench, serve the same purposes in the case of a smaller and more variable flow. These sewers, moreover, afford a large waterway in times of heavy rainfall when the benches are submerged; and they are sufficiently commodious to accommodate gas- and water-pipes supported on side brackets near the top, or borne, in the case of large water conduits, on slender columns standing on a side bench. The large sewers also provide an adequate waterway in the centre for boats to

[1] "Salubrité Urbaine: Distributions d'Eau; Assainissement," G. Bechmann, Paris, p. 572.

traverse them, which has proved serviceable for cleansing purposes. A somewhat similar principle, in a modified form, has been adopted in Buenos Ayres, at places where the depth was inadequate for the oval form of the larger concrete sewers generally used, a small channel having been formed in the centre of the invert of the flat tunnel-shaped sewer, for the flow of the sewage in dry weather,[1] Fig. 228; whilst the large section above has been provided for the discharge of a rainfall estimated at a maximum of $1\frac{1}{2}$ inches an hour over the area drained. Frequently where the flow is not liable to fall very low, an oval section, intermediate between the egg-shaped and circular forms, has been employed, as exemplified by the typical section adopted in Rome and Milan,[2] Fig. 229; and this form, whilst affording a larger discharging

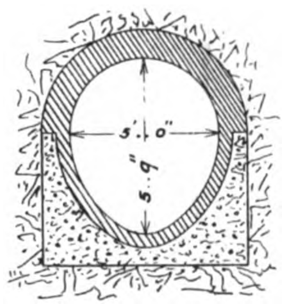
SEWER IN SHALLOW GROUND AT BUENOS AYRES.

Fig. 228.



TYPICAL OVAL SECTION OF ROME AND MILAN SEWERS.

Fig. 229.



SCALE $\frac{1}{75}$.

capacity than an egg-shaped sewer with the same amount of material, gives a better hydraulic radius than a circular section for small flows.

Occasionally, a sewer for sewage, and a separate conduit for rainwater, are combined in a single structure, as illustrated by the arrangement adopted at Buenos Ayres, comprising an oval sewer of small size below, and a large tunnel-shaped conduit above, for the considerable volume of rainfall provided for there, Fig. 230, and also by an ovoid construction in concrete with a flat base, in which a small lower channel has been formed for the sewage, resembling the bottom portion of an

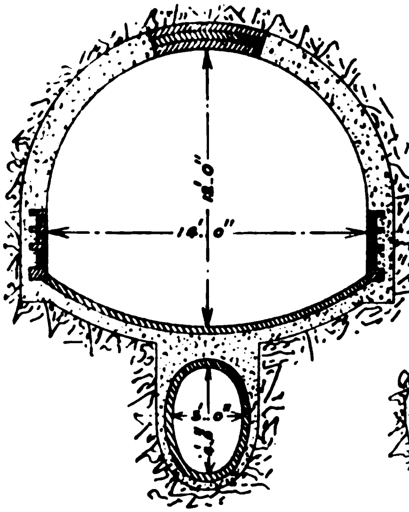
[1] "The Sanitary Works of Buenos Ayres," R. C. Parsons, *Proc. Inst. C.E.*, vol. cxxiv. plate 3, fig. 28.

[2] "Les Égouts de Rome," M. Ronna, *Bulletin de la Société d'Encouragement pour l'Industrie Nationale*, Paris, 1897, pp. 1333 and 1334.

egg-shaped sewer, and a much larger circular conduit for rainwater in the upper portion, as adopted for the separate system in Bromberg and Insterburg,[1] Fig. 231. Moreover, a peculiar structure, with an egg-

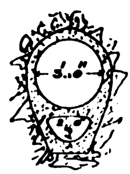
SEWER AND CONDUIT FOR RAINWATER, BUENOS AYRES.

Fig. 230.



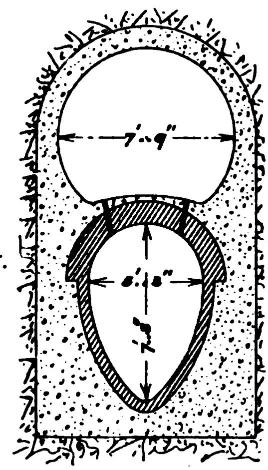
SEWER; AND RAIN-WATER CONDUIT.

Fig. 231.



SEWER WITH GALLERY ABOVE IN ROME.

Fig. 232.



SCALE 1/100.

shaped sewer at the bottom, and an upper gallery to serve as a subway for pipes, has been resorted to in portions of the sewerage of Rome, Fig. 232.

Fall of Sewers.—Like house drains, sewers have to be given falls inversely proportional to their sizes and the ordinary minimum flow; and whereas a small fall causes a sluggish flow and leads to deposit of solids from the sewage, a large fall, when available, may produce too rapid a flow tending to injure the sewer. To prevent deposit, it is advisable that the velocity of flow should not be less than $2\frac{1}{2}$ feet per second for sewers not exceeding 2 feet in diameter, and 2 feet per second for large sewers; whilst the velocity ought not to exceed $4\frac{1}{2}$ to 5 feet per second, to preserve the sewer from injury by the current.

[1] *Gesundheits-Ingenieur*, Munich and Leipzig, 1897, "Uber Trennungssysteme," Oberingenieur Metzger, pp. 246–247.

The following formula has been devised for calculating the fall required for pipes to obtain the minimum velocity of flow of $2\frac{1}{2}$ feet per second,[1] namely, $F = \frac{7 \cdot 56}{2R} = \frac{7 \cdot 56}{r}$ when running full, where F is the fall in feet per mile, R is the hydraulic radius, and r is the radius of the pipe in feet; and this gives a minimum fall of about 1 in 340 for a 1-foot pipe, and 1 in 700 for a 2-feet pipe. Naturally the best available fall consistent with the preservation of the pipes and brickwork sewers should be adopted, so as to reduce the dimensions of the sewers to a minimum for a given discharge; but local conditions often necessitate considerable variations in the fall. A larger fall should be given at bends in sewers, to compensate for the obstruction to the flow at these points; and public drains should not be made less than from about 9 to 7 inches in diameter, on account of the liability of small drains to become blocked by chance obstructions introduced carelessly.

An egg-shaped sewer can be given a smaller fall than a circular sewer of corresponding size, on account of its better hydraulic radius for small flows; and main sewers receiving the sewage from a large area, and, consequently, possessing a more regular flow than branch sewers, can be kept free from deposit with a smaller fall. Falls of 1 in 500 to 1 in 1000 are suitable for main sewers of moderate size; but intercepting and outfall sewers have frequently to be given less falls, for which their large and continuous flow renders them well adapted.

The dimensions of sewers depend upon the fall, and the volume of rainwater, in addition to the sewage, which it is estimated they may have to discharge. Tables have been compiled giving the discharging capacities of sewers, both circular and egg-shaped, for the most usual falls and the most common dimensions, the velocity of flow having been calculated by the well-known formula of Ganguillet and Kutter.[2] A simpler formula, however, has been recently prepared,[3] which has been found to give quite as accurate results, in the case of sewers formed of good brickwork, or cast-iron pipes, as the older, somewhat complicated formula, namely, $V = 124 \sqrt[3]{R^2} \sqrt{S}$, in which R is as usual the hydraulic radius, and S the surface slope; and the relative simplicity of this formula enables the velocity V, in feet per second, to be readily calculated for any form of sewer with a given fall, dispensing with the necessity of referring to tables, which do not always give the velocities for the particular cases required.

[1] "Sanitary Engineering," Col. E. C. S. Moore, R.E., p. 34.

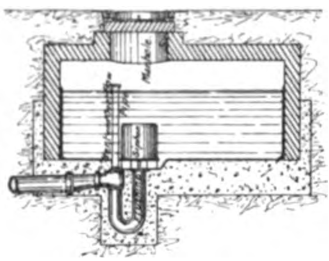
[2] *Ibid.*, 2nd edition, pp. 142 to 203; and "Canal and Culvert Tables," L. D'A. Jackson, 2nd edition, pp. 130 to 179.

[3] "A New Formula for the Flow in Sewers and Water-Mains," W. S. Crimp and C. E. Bruges, *Proc. Inst. C.E.*, vol. cxxii. p. 199.

Cleansing of Sewers.—Where the available fall is insufficient, automatic flushing tanks can be used for keeping small sewers clear by a periodical discharge of a good volume of water, through siphonic action in a tank gradually filled, Figs. 233 and 234; but when the fall

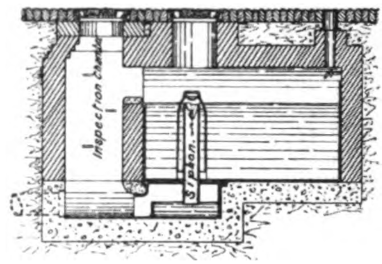
ADAMS' AUTOMATIC FLUSHING TANK.

Fig. 233.



FIELD'S AUTOMATIC FLUSHING TANK.

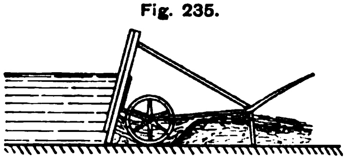
Fig. 234.



that can be obtained is very inadequate, an artificial fall has to be provided by raising the sewage at suitable points by pumps, ejectors, or lifts. Flushing tanks are also expedient at the dead-ends of sewers in streets terminating in a cul-de-sac, or at the upper ends of the sewerage of a town, to keep these extremities free from deposit in the absence of any flow. In Paris, and also in some other towns in France, the public sewers are made high enough for men to walk through them without stooping; and reliance is placed upon large gangs of workmen to keep the sewers clear. This is accomplished in dry weather by the workmen pushing the deposit forward with brooms and rakes, and directing it into special receptacles provided at intervals, from which it can be periodically removed, or by loading it into trolleys running along the sewers, with spades and shovels. When there is a sufficient flow, a temporary flush is created by damming back the sewage by lowering a suitably shaped panel of wood or iron across the lower portion of the sewer, the scouring being effected, either by the rush of the penned-back water through an interstice purposely left between the edge of the panel and the side of the sewer, or through openings provided in the panel. In the small sewers, these panels are attached to the back of a sort of wheelbarrow which is drawn along by a man, Fig. 235; but in the larger sewers, the panels are either hung down from trolleys running over the central channel,

FLUSHING PANEL DRAWN BY WHEELBARROW, PARIS SEWERS.

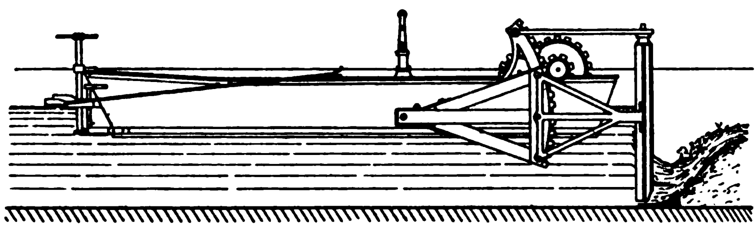
Fig. 235.



on rails formed of angle-irons fastened to the edges of the side benches, or carried at the end of a boat navigating the main collecting sewers when the depth of water is adequate,[1] Fig. 236. The necessity for this systematic clearing of the Paris sewers appears to be due, in the smaller ovoid sewers devoid of benches, to the flat invert provided for facilitating the passage of the men, to the somewhat flat bottom of the central channel of the larger sewers, Fig. 226 (p. 318), and to the dirt and detritus of the streets being swept as quickly as possible into the sewers, without adequate provision for their interception by catch-pits; for the fall of the ordinary Paris sewers, which is occasionally as much as 1 in 143, ranges for the most part from 1 in 200 to a minimum of 1 in 1000, and for the smaller main sewers from 1 in 666

BOAT WITH FLUSHING PANEL, PARIS SEWERS.

Fig. 236.



to 1 in 1000, 1 in 1000 for the larger ones, and 1 in 3333 down to 1 in 3846 in the great collecting sewers, where the velocity drops to from $1\frac{1}{2}$ feet to 1 foot, and in one part to 10 inches per second.[2] In the smaller Paris sewers, it has been found that a fall of not less than 1 in 200 is required to prevent the deposit of silt, and that sand deposits with a fall of 1 in 100, but is scoured away with a fall of 1 in 66.

The Paris sewers, besides being subject to the disadvantage of not being self-cleansing, must have been very costly to construct, on account of their height. On the other hand, they provide the conveniences of a subway under the streets; and they also afford a large discharging capacity for carrying off the rainfall, though not adequate to dispense with storm overflows into the Seine during exceptional rainfall.

Provision against an Excessive Fall in Sewers.—Where the available fall of the ground is such that, if adopted for the slope of the sewers, it would impart an undue velocity of flow, a flatter gradient

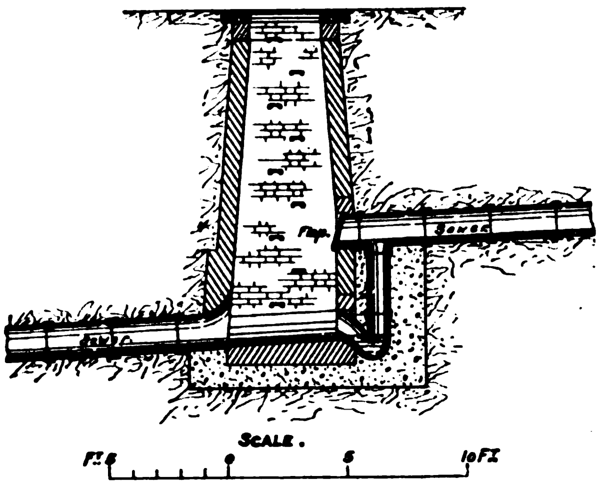
[1] "Salubrité Urbaine: Distributions d'Eau; Assainissement," G. Bechmann, pp. 575-576.

[2] "L'État actuel de l'Assainissement de Paris," MM. Bechmann and Launay, *Annales des Ponts et Chaussées*, 1895 (1), p. 265, and plate 3.

must be employed, and its deficiency in fall made up by introducing steps or vertical falls at convenient places. Manholes should be provided at these points; and where the vertical drop is large, it is advisable to retain a cushion of water at the bottom to lessen the shock of the falling water, which can be effected by the siphon arrangement shown in Fig. 237. The continuation of the sewer, past the inlet to

STEP IN SEWER TO AVOID STEEP SLOPE.

Fig. 237.



the siphon, up to the manhole, where its outlet is ordinarily closed by a flap, enables the flow in the sewer during heavy rainfall to discharge direct into the manhole as soon as the vertical pipe of the siphon has been filled, and when the bottom of the manhole is protected by an ample depth of water.

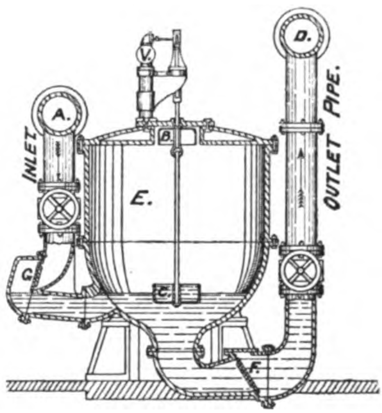
Methods of Compensating for an Inadequate Fall in Sewers.—In very flat districts, where the available fall is insufficient to produce a velocity of flow in the sewers capable of scouring away deposit, and would also involve the construction of large sewers to provide adequate discharging capacity in periods of heavy rainfall, it becomes necessary to furnish an artificial fall by raising the sewage to a higher level, so as to give the sewer into which it is discharged a better fall by placing it at a higher elevation at its upper end, corresponding to the lift given to the sewage at the termination of the low-level sewer.

The requisite lift is readily effected by the ordinary method of pumping with centrifugal or lift pumps; but with a foul, gritty liquid such as sewage, and especially in view of the advantage of raising moderate volumes of sewage at several points in a flat district, ejectors have often been resorted to; and removal by suction, and automatic lifts, have been adopted in special cases.

Ejectors for lifting Sewage.—The Shone hydro-pneumatic ejector, shown in Fig. 238, consists of a cast-iron ejector **E**, connected at the bottom on each side with an inlet and an outlet pipe, and having an arrangement at the top for the admission of compressed air from a reservoir, supplying the power for working it.[1] The sewage from the low-level sewer flows into the ejector **E**, through the inlet pipe **A**; and on filling the ejector up to the underside of the bell **B** at the top, it encloses the air contained in this bell. Then the sewage continuing to flow in, rises outside the bell, compressing the air imprisoned in the bell, till at last the pressure becomes sufficient to raise the bell and the spindle attached to it, which opens the valve **V**, and admits compressed air into the ejector. This inrush of compressed air drives the sewage out through an opening at the bottom of the ejector into the rising outlet pipe **D**, through which it passes into a sewer at a higher level. As soon as the level of the sewage in the ejector has fallen, under the pressure of the compressed air, sufficiently low to leave the full cup **C** unsupported, the cup descends by its own weight, and pulling down the bell and spindle to their original positions by its fall, closes the compressed-air valve above, and enables the compressed air in the ejector to escape. Directly the pressure of air in the ejector is relieved, the valve **F** at the bottom of the outlet pipe closes; and the valve **G** at the base of the inlet pipe is opened by the influx of a fresh charge of sewage into the empty ejector, to be in its turn forced out and raised to the upper sewer by a fresh admission of compressed air. The advantages of this system are, that the ejector is devoid of any parts liable to be injured by the sewage or grit, or to get out of order; a free passage is provided throughout for the solids in the sewage; the heavier solids, falling to the bottom of the ejector, are the first portions

SHONE'S HYDRO-PNEUMATIC EJECTOR.

Fig. 238.

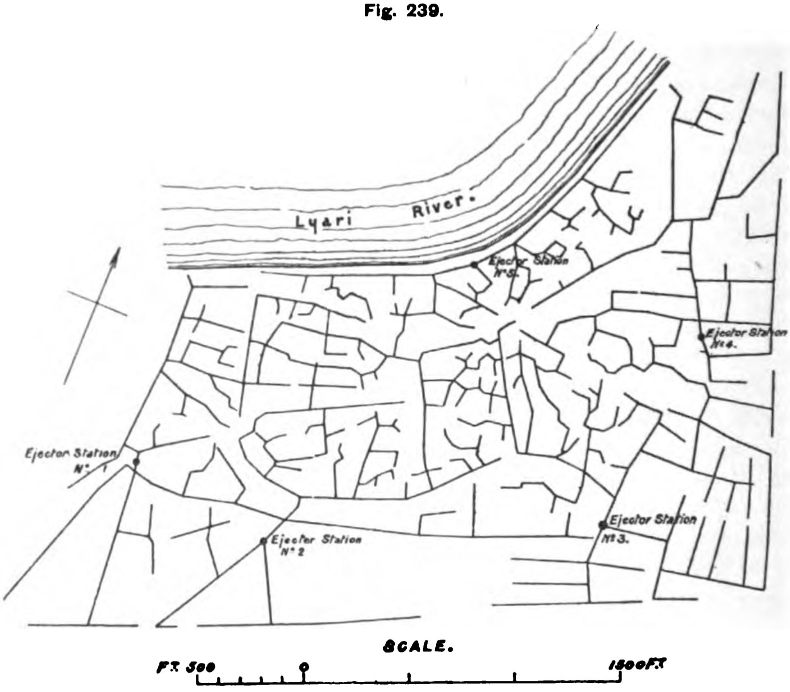


[1] "The Shone Hydro-pneumatic Ejector," Illustrated description, Chester, 1892.

forced into the outlet pipe by the compressed air; a powerful flush of liquid is discharged into the upper sewer at each emptying of the ejector, enabling smaller pipes to be used than would be otherwise advisable; and the lower and upper sewers are completely separated by the ejector. Moreover, as several ejectors can be satisfactorily worked from one central compressing-air station, this system enables a town to be readily divided into several minor, independent drainage districts; and the ejectors are made in different sizes and shapes according to the requirements, ranging in capacity from about 50 to 1000 gallons. These ejectors are particularly valuable for the sewerage of perfectly flat, low-lying towns near the sea-level; for by placing ejectors on the sewers at numerous points only moderate distances apart, it is possible to create the necessary fall for the sewers, which nature has failed to provide, Fig. 239.[1]

SEWERAGE OF KARACHI SHOWING POSITIONS OF EJECTORS.

Fig. 239.



Suction System for Sewage.—The Lienur pneumatic system, introduced into Amsterdam in 1870, and adopted at several other

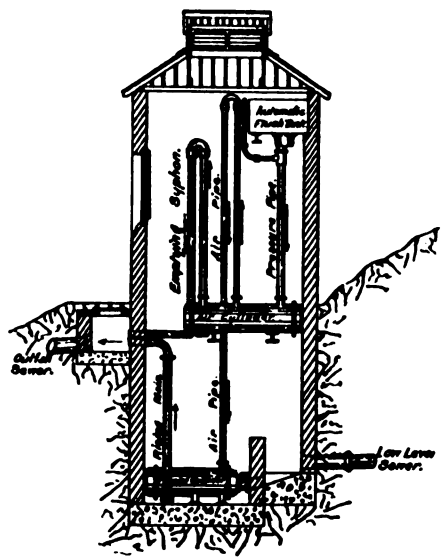
[1] "Karachi Sewerage Works," J. Strachan, *Proc. Inst. C.E.*, vol. cxxxv. p. 272, and plate 7.

low-lying towns on the Continent, furnishes another method of compensating for an inadequate natural fall in laying out sewers; but in this case, instead of lifting the sewage, it is drawn through air-tight pipes by creating a vacuum in a receiver. The sewage of each separate district of the town is drawn from the houses by suction into an air-tight iron tank by opening valves; and the sewage collected in these tanks is eventually conveyed to a central reservoir, in which a vacuum has been formed, from whence it is raised and dealt with. In this system, the amount of rainfall admitted to the sewers is reduced to a minimum, in order to avoid diluting the sewage, and to limit the amount to be treated; but this arrangement involves the provision of channels or conduits for the rainfall in a low-lying country such as Holland, if the soil is not to be waterlogged, which it has been estimated would have to be sufficient at Amsterdam to discharge a maximum rainfall of 1 inch in twenty-four hours over the drainage-area.

Automatic Sewage Lift.—Where only a small portion of a town is at a low level, a method has been devised for securing the requisite fall by lifting the sewage from the low-level sewer to a higher level, either by the descent of water from a high-level tank which is filled with water, as required, from the water-mains of the town, or by the power generated by the descent of some of the sewage intercepted from one of the high-level sewers. The Adams automatic sewage lift is based on the principle that the fall of a liquid from a high level to an intermediate level, can be utilized, by means of a suitable arrangement of pipes and cylinders, in raising a corresponding volume of sewage from a low-level sewer to a sewer at the intermediate level, provided the fall from the high level to the intermediate level is sufficiently greater than the vertical distance from the low-level sewer to the sewer at the intermediate level, to overcome the friction tending to check the

ADAMS' AUTOMATIC SEWAGE LIFT WORKED BY WATER.

Fig. 240.

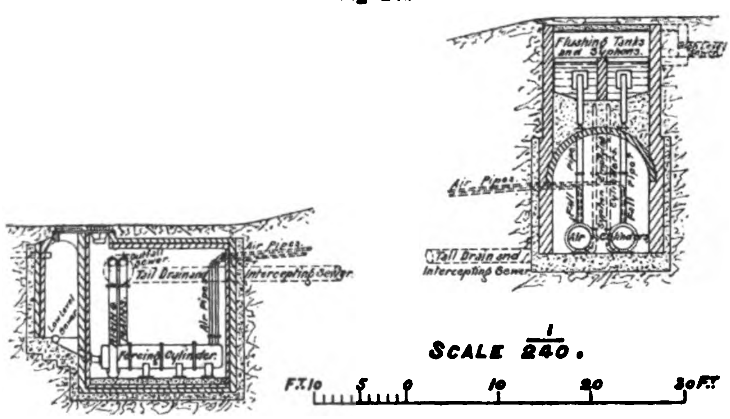


flow in the pipes and cylinders. The general arrangement of one of these lifts, when worked by water supplied from a high level, is shown in Fig. 240, where the upper part of the building containing it is raised some height above the ground.[1] The tank at the top of the building, fed from the water-supply of the town, discharges through a vertical pressure pipe into the air-pressure cylinder at the intermediate level, and driving the air out of this cylinder into a long air pipe issuing from the centre of the cylinder, and communicating with the forcing cylinder at the bottom, forces the sewage discharged from the low-level sewer into this cylinder, up the rising main to the sewer at the intermediate level. When the air-pressure cylinder has become full of water, it is emptied by the siphon pipe leading to the upper sewer; and the supply of water to the tank is so controlled by a ball-cock, that the supply is shut off when there is no more sewage at the bottom to raise, and the lift ceases working till a fresh volume of sewage flows in.

In the above arrangement, all the apparatus for lifting the sewage is contained in a single high building; but when sewage is used as the motive power, the apparatus is placed in two separate buildings, often a considerable distance apart, depending upon the position of the high-

ADAMS' AUTOMATIC SEWAGE LIFT WORKED BY SEWAGE.

Fig. 241.



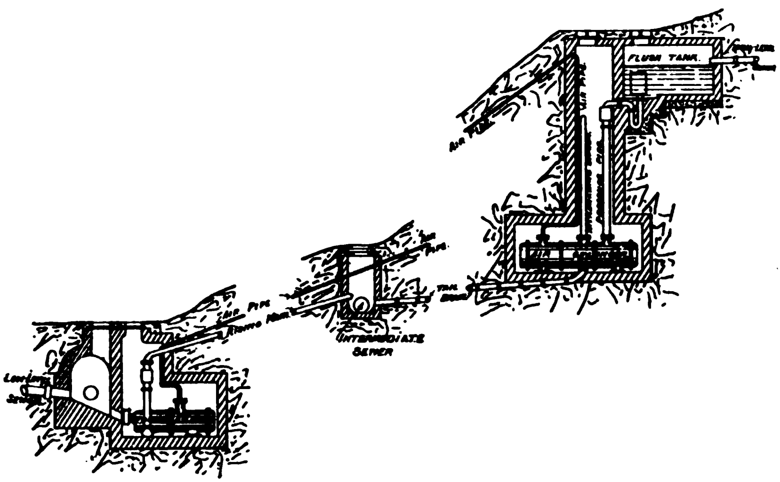
level sewer, from which the sewage for working the lift is drawn, in relation to the low-level sewer, the sewage of which has to be raised. The buildings in the latter case are generally constructed underground; and the high-level chamber, into which the sewage intercepted from the high-level sewer is led for working the lift, contains the air-pressure

[1] "On the Automatic Raising of Sewage," Adams' Patent Sewage Lift Company, York, p. 2.

cylinder, from which an air pipe transmits the pressure to the forcing cylinder in the low-level chamber, Fig. 241. The sewage of the low-level sewer, discharged into the forcing cylinder, is thus lifted, through a rising main, into an adjacent sewer at a higher level; and the sewage drawn from the air-pressure cylinder by the siphon, is conveyed away to a sewer at a high level; or the sewage is lifted through a main to a sewer at an intermediate level, somewhat lower than half the height between the high-level and low-level sewers, into which the sewage discharged from the air-pressure cylinder in working the lift, can also be made to flow down, Fig. 242. Before the sewage from the high-level sewer is

SEWAGE LIFTED TO INTERMEDIATE SEWER.

Fig. 242.

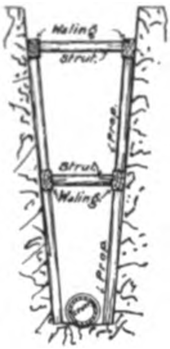


admitted into the upper chamber for compressing the air, it is passed through a screen for arresting solid refuse, and into a tank for the deposit of the sludge.

Laying of Sewers.—Generally sewers are laid at the requisite depth at the bottom of a timbered trench, with cross struts to keep up the ground on each side, the struts being proportioned in number and strength to the nature of the soil and its liability to slip in, as shown in Figs. 243 and 244 (p. 330). A sewer has to be laid very uniformly to its proper fall to ensure regularity of flow and discharge throughout; and the levels of the invert of the sewer are determined and checked by means of a boning rod, put down vertically till its top is in line with the top of the sight rails, fixed at intervals over the trench to the required fall by levelling, Fig. 245. Where sewers have to be carried at a considerable depth below the surface, timbered headings are driven underground, proportioned in strength to the instability of the stratum traversed, inside

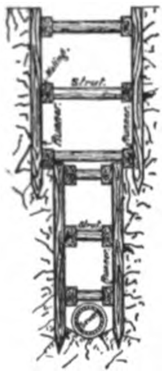
SLOPING TIMBERED TRENCH FOR LAYING SEWER.

Fig. 243.



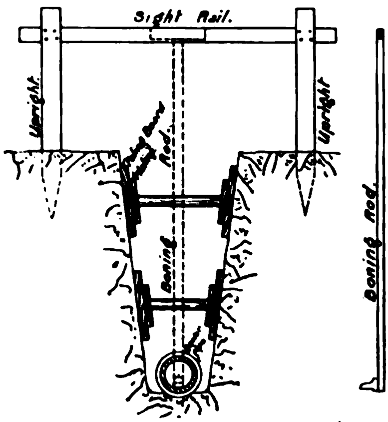
VERTICAL HEAVILY TIMBERED TRENCH IN TWO LIFTS.

Fig. 244.



ARRANGEMENT FOR ADJUSTING FALL OF SEWER.

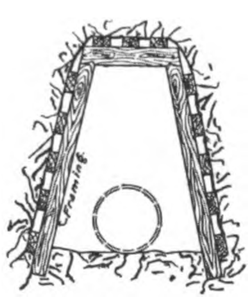
Fig. 245.



which the sewers are constructed, Figs. 246 and 247. The trenches and headings are filled in again with soil as the work proceeds; and it is

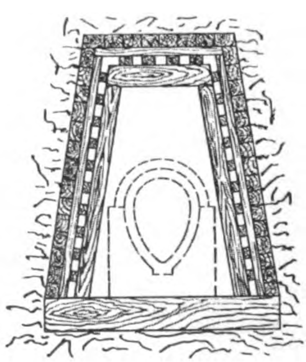
TIMBERED HEADING FOR CARRYING SEWER THROUGH HIGH GROUND.

Fig. 246.



STRONGLY TIMBERED HEADING FOR MAIN SEWER.

Fig. 247.

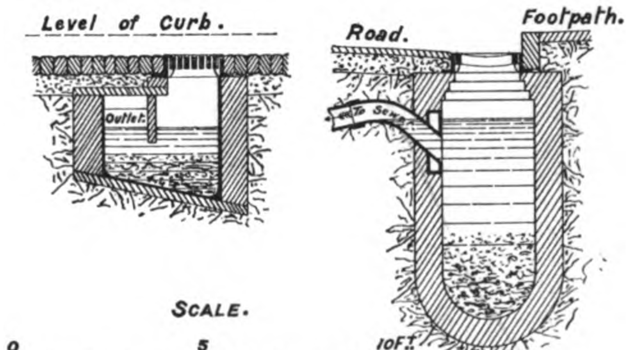


important for the sewers to be laid on firm ground, or on a widened-out concrete foundation, to secure them from unequal settlement when the trench is filled in. Sometimes, when a sewer has to be carried for some distance at a considerable depth below the surface across high or hilly land, especially in the case of a large outfall sewer, it is constructed like a railway tunnel, by the aid of shafts sunk at intervals; and in constructing the Clichy outfall sewer for the Paris sewage, both at a small depth below a boulevard and under high ground, a tube with shield was employed, to avoid disturbing the traffic in the first case, and to obviate settlement in the second instance.

Gullies.—It is very important to prevent the solid matters washed off the streets, through the gratings at the sides, from reaching the sewers, and thereby increasing the tendency for deposit to take place and obstruct the discharge. Accordingly, the gullies under the gratings, receiving the water flowing off from the roads, are provided with catch-pits at the bottom, into which the solid matters drop, and from which they are periodically removed, in the form of mud, by means of iron ladles. The gullies, moreover, are usually so constructed that the water contained in them prevents the escape of sewer gas into the road through

GULLY WITH SLOPING CATCH-PIT AND TRAP. GULLY WITH DEEP CATCH-PIT.

Fig. 248. Fig. 249.

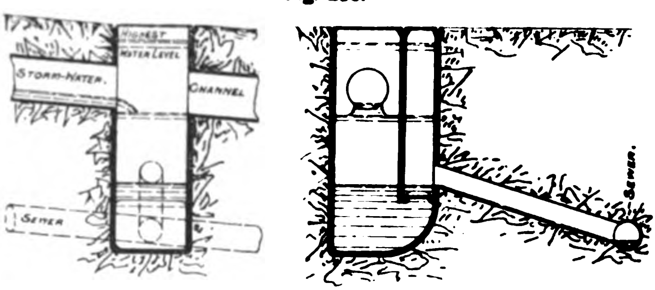


the gratings, by forming a seal to the trap; whilst the overflow passes through an aperture at the side of the gully into the sewer under the centre of the road, Figs. 248 and 249. A shallow receptacle is sometimes formed under the footway, with an aperture along its whole length in line with the face of the curb, and covered over with iron covers in place of the curb-stone and adjacent flags, into which horse-droppings and other solid refuse are swept by the street scavengers, which are readily emptied on raising the covers, and removed by carts. This arrangement possesses the advantage of enabling the dirt on the surface of the streets to be rapidly removed, particularly in dry weather, and put out of sight; but it necessitates frequent emptying of the receptacle.

An ingenious form of gully has been devised in Germany, which, after allowing a sufficient quantity of water to pass into the sewer for flushing it, on becoming filled up above a certain level during periods of heavy rainfall, discharges the surplus storm water into a large upper channel to be disposed of as rainwater.[1] This division of the flow is effected by a vertical partition in the gully, dividing the gully from the top to below the outlet to the sewer into two unequal portions, and by the relatively large size of the storm-water outlet, Fig. 250.

GULLY WITH STORM-WATER OVERFLOW.

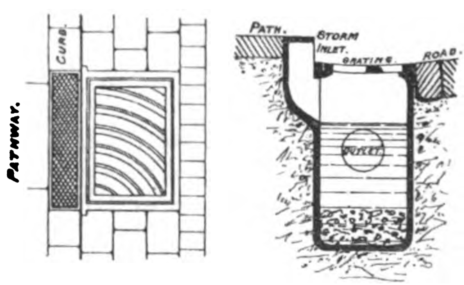
Fig. 250.



In streets having a considerable fall, the flow of water along the side channels becomes so rapid during heavy rainfalls, that it passes over to a great extent the narrow cross openings provided by the ordinary gratings over the gullies, and rushes down into the lower cross streets. To prevent this, openings at the side under the curb-stones are sometimes formed, with a drop in the side channel in front of them in order to direct the water towards the openings; but a more effectual method of catching the rapidly flowing water, is to lay down a grating with curved bars and wide apertures between, which, besides admitting more water, directs the surplus flow to the side opening, Fig. 251;

INLET TO GULLY UNDER CURB-STONE, WITH CURVED GRATING, FOR STEEP STREETS.

Fig. 251.

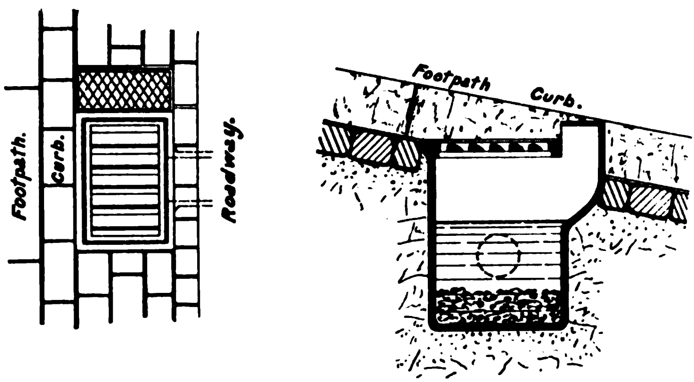


[1] *Gesundheits-Ingenieur*, 1902, p. 139.

and in very steep streets, it is expedient to provide a projecting opening facing the side channel, at the lower end of the grating, to catch the remainder of the flow, as shown in Fig. 252.

PROJECTING INLET FACING FLOW FOR VERY STEEP STREETS.

Fig. 252.

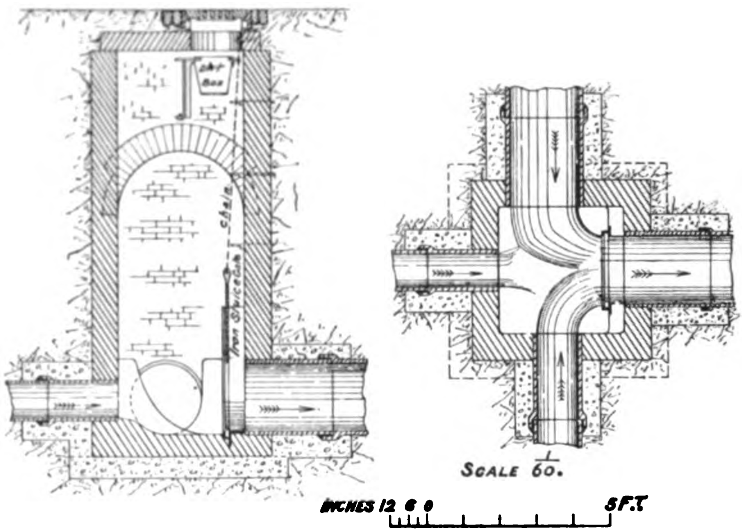


Manholes for Sewers.—Manholes or inspection chambers should, if possible, be placed at all junctions, steps, and bends of sewers, and even in long straight sewers at intervals of about 100 to 150 yards, so that the sewers may be readily inspected, and any obstructions, which are specially liable to form where two or three sewers join, or at changes of direction and of level, may be promptly detected and removed. Lamp-holes also, consisting of vertical pipes of small diameter from the top of the sewer to the street, may be advantageously inserted at intervals between the manholes, down which a lamp may be lowered to the sewer, so that any deposit between the lamp and the nearest manhole, by hiding the light, may be observed from the manhole, and thus located. A manhole at the junction of three sewers is shown in Fig. 253 (p. 334), and at a step for avoiding a too rapid slope, in Fig. 237 (p. 324). Generally the manholes are placed in the centre of the road, directly over the sewer; but sometimes a manhole for a large sewer running under a road with a very large traffic, which has to be frequently descended, is placed in the footway, so as to be more out of the way; and access is obtained to the sewer through a side passage.

By providing a sluice-gate in front of the inlet to a sewer at the bottom of a manhole, as shown in Fig. 253, it is possible to flush the sewer, by first lowering the sluice-gate, and thus penning back the sewage up to a certain level, and then raising the gate for the escape of the accumulated flow down the sewer. A more satisfactory arrangement for flushing sewers at manholes, consists in providing means of closing the outlet from a sewer into a manhole, as well as the inlet from the man-

MANHOLE AT JUNCTION OF SEWERS.

Fig. 253.



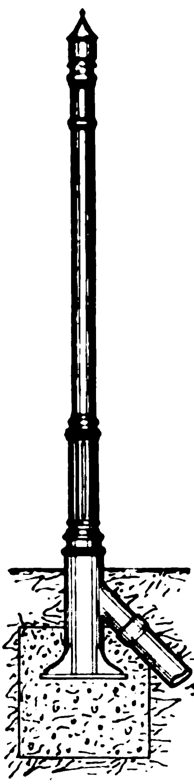
hole into the sewer; and then by filling up the manhole with clean water, and reopening the sewer, a more thorough flush is effected, without fouling the manhole with sewage water.

Ventilation of Sewers.—Sewer gas is constantly being evolved from the sewage passing along sewers; and it is most important that this noxious exhalation should not be allowed to penetrate into houses, and that it should be dispersed into the open air without causing a nuisance or injury to health. This gas is specially liable to accumulate in the highest parts of the sewers, where portions of the district are at a considerable elevation. Formerly, the manholes were provided with perforated covers at the surface of the streets, so as to allow of the escape of the gas and the admission of fresh air into the sewers; and sometimes, where it is possible to carry up the manhole above the surface of the road, with the provision of an entrance at the side, the manhole serves very advantageously as a ventilating shaft, Fig. 268 (p. 352). The objections to openings at the street level are, that the escape of the sewer gas at the surface is liable to be a public nuisance before it becomes adequately diluted with air, and that dirt from the street falls through the openings, or clogs them up in dry weather. The dropping of dirt and *débris* into the sewer is prevented by placing a sort of movable bucket below the opening for intercepting these deposits, which is periodically emptied, Fig. 253, or by placing the opening slightly on one side, Fig. 267 (p. 352); and the removal of the dirt from the apertures can be effected by the street scavengers, simultaneously with the clearing of the gratings of the side gullies. Endeavours have been made to remedy the insanitary plan of allowing undiluted sewer gas to issue straight into the streets, by causing it to pass through a layer of charcoal or other disinfecting substance interposed below the manhole cover; but this arrangement, at the same time, checks the free passage of the air from the sewer, and, consequently, the ventilation; and the charcoal which should be dry, is kept damp in such a situation. Accordingly, the tendency of recent practice has been to close the manholes with air-tight covers at the surface in much-frequented streets, and to rely on other means for the ventilation of the sewers.

Ventilation is the more necessary in sewers which have to carry off the rainfall of the district in addition to the sewage; for the rising of the water-level in the sewer during a heavy fall of rain, reduces the space in the sewer available for the sewer gas, and, therefore, tends, by the pressure it exerts, to make the sewer gas force a passage through the traps designed to exclude it, unless easier means of escape are provided at intervals. The liability of the sewer gas, under such conditions, to accumulate specially at the highest parts of the network of sewers draining districts having very different levels, is fortunately to some extent counteracted by the tendency of the air in the sewers to be drawn along by the more rapidly flowing liquid at such times; but this complicates the problem of providing efficient ventilation, by introducing variable conditions. Moreover, natural ventilation, which constitutes the basis of most of the arrangements adopted for ventilating sewers, is greatly affected by the force and direction of the wind. High, vertical, iron pipes furnish the ordinary method, at the present time, for inducing ventilation in sewers, where openings in manhole covers have been abandoned, combined with inlets for the admission of fresh air over the sewers in the streets, or in the footways, or elsewhere at the side; but as these inlets are liable to become outlets under certain conditions, they should be provided with flaps closing against an issuing current. The vertical shafts, which should not be less than 6 inches in diameter, and should be increased in proportion to the size of the sewer they ventilate, are sometimes left perfectly open at the top, and sometimes provided with a cowl and apertures at the side, which prevents their becoming blocked by anything falling down the clear opening, or by birds building nests, Fig. 254 (p. 336). As, however, sewer gas rapidly corrodes iron, which, falling to the bottom as rust, may in time impede the passage of the air, these pipes should be lined internally with lead or copper. An improvement on the simple, though somewhat uncertain system of natural ventilation by open shafts, is afforded by introducing a gas-lamp at the top of the ventilating shaft, which by its heat draws the air from the sewer up the shaft, and increases the circulation of the air in the sewer, as shown in Fig. 255, where there is a copper pipe provided for the ventilation inside the iron shaft supporting the lamp. By this arrangement, the ventilating pipes can be conveniently placed on the shelters provided for foot-passengers in the centre of crowded streets.

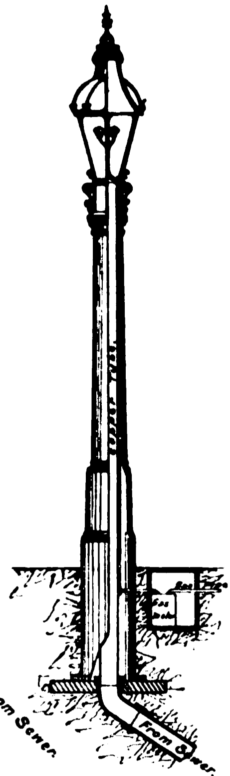
ADAMS' VENTILATING SHAFT.

Fig. 254.



WEBB'S VENTILATING SHAFT WITH GAS-LAMP.

Fig. 255.



Mechanical ventilation has recently been occasionally introduced for sewers by electrically driven fans; and undoubtedly such a system, if efficiently carried out throughout the network of sewers of a district, would ensure the more complete removal of sewer gas than natural ventilation. Unfortunately, this method of artificial ventilation would be costly, would need very careful management and supervision, and if providing a very effective out-draught of the air in the sewer, would be very liable to disarrange the traps of the house drains.

In any case, the ventilation of the sewers in the upper portions of towns presenting considerable differences in level, should be most carefully provided for; whilst, on the other hand, the flooding of the lower portions of such towns in times of exceptional rainfall, by the rapid descent of the drainage from the higher districts, needs equally to be guarded against as the incursion of sewer gas in the upper parts.

CHAPTER XIV.

OUTFALLS; AND CLARIFICATION OF SEWAGE.

Conveyance of Sewage to Outfall—Methods of Disposal of Sewage of Towns—Collecting and Intercepting Sewers, objects, forms, instances, arrangements at Edinburgh—Outfall Sewers, object, differences in length, variety of works—Construction of Outfall Sewers, form, materials, fall—Storm Overflows for Outfall Sewers—Manholes at Outfall Sewers, uses—Ventilation of Outfall Sewers, importance, by manholes and shafts, precautions against sewer gas, artificial ventilation—Siphons for Sewers, provisions for inspection, repairs, and flushing—Control of Discharge at Outfall, by flap-valves and penstocks—Choice of Situation for Outfall, precautions necessary, period of discharge, indications of floats, influence of wind—Outfall Sewers discharging into a Tidal River, London Northern and Southern outfall sewers—Outfall Sewers discharging into the Sea, Brighton outfall sewer, Torquay outfall sewer—Irrigation Outfall Sewer, Paris irrigation sewer to Achères—Outfall Sewers discharging into a Tideless River, Rome outfall sewers—Outfall Canal into adjacent River-Basin, Chicago Drainage Canal—Clarification of Sewage, by settlement and precipitation, proportion of suspended matters, chemicals used for precipitation, settling and precipitating tanks, treatment of London sewage at outfalls, clarified effluent not purified—Conveyance of Sewage Sludge out to Sea, as effected with London and Manchester sewage sludge.

Collecting, intercepting, or outfall sewers have to be provided for conveying the flow of sewage in the main sewers of a town or district to its final destination. The arrangement and length of these sewers, and the destination of the sewage they convey, depend upon the local conditions. Formerly, the sewage was discharged at the nearest points of the rivers alongside which inland towns have generally grown up, and, in the case of towns on the sea-coast, direct into the sea, without regard to the pollution of the waters of the rivers and the foreshores of the sea in front of the towns. With the growth, however, of population and the advance of sanitary science, and in view of the circumstances that towns frequently draw their water-supply lower down non-tidal rivers, that in tidal rivers the sewage discharged on the ebb is partially brought back by the flood tide, and that seaside resorts are entirely dependent for their prosperity on their healthiness and the cleanliness of their beach, those simple and economical means of disposing of the sewage of towns have had in many cases to be abandoned, especially where, as in England, the rivers above their estuaries are small. In large continents, where the rivers have a very large fresh-water flow, and where the towns are long distances apart, and particularly where the sewage is much diluted by an abundant water-supply, as in the United States, the discharge of sewage into the river below a town may be unobjectionable, as the very dilute sewage is gradually purified by the oxygen in the river-water, and in the course of its flow.

Methods of Disposal of Sewage from Towns.—Where towns are situated on the sea-coast, or on a wide tidal estuary, the sewage is still discharged into the sea or tideway; but instead of carrying the main sewers direct to their nearest discharging points on the beach, their flow is led into a collecting or intercepting sewer which conveys the sewage into an outfall sewer; and this sewer, after being carried along the coast for some distance from the town, traverses the beach, and discharges its contents into the sea beyond low-water mark.

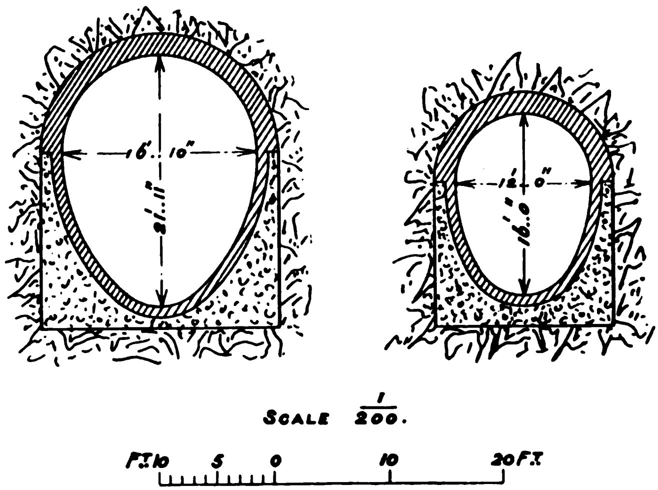
When a large town adjoins a tidal river, besides carrying its sewage in an outfall sewer for a considerable distance below the town, and discharging it during the earlier portion of the ebb tide, it may be very advisable, in order to avoid serious pollution, to collect the solid constituents of the sewage and convey them out to sea for deposit.

Towns situated upon rivers of only moderate size, above the tidal limit, and in a populous neighbourhood, cannot be allowed to discharge their crude sewage into the adjacent river; and though in certain cases, where the river is fairly large, and is not drawn upon for water-supply by towns for a long distance below, clarification by subsidence and chemical precipitation may suffice before discharging the liquid sewage into the river, in most instances it is necessary to purify the sewage by irrigation on land, or by bacterial or chemical processes, before the effluent can be suffered to pass into the river.

Collecting and Intercepting Sewers.—In laying out, reconstructing, or extending the sewerage system of a district, the discharge from the main sewers is received by a collecting or intercepting sewer for conveyance to the outfall sewer. A collecting sewer, as its name implies, collects the sewage of the various main sewers of the district on each side of it; whereas an intercepting sewer is the term properly applied to a sewer which, running along the coast or alongside a river, intercepts the flow of the main or old outfall sewers, which formerly, passing down to the sea or a river, discharged their contents at several outfalls. The object and design, however, of both collecting and intercepting sewers are practically the same; and as they receive the drainage of a much larger area than the main sewers, especially in the lower part of their course, their discharge is not only larger, but also more uniform than that of the main sewers. Accordingly, they are less liable to the

INTERCEPTING SEWERS IN TIBER EMBANKMENTS, ROME.

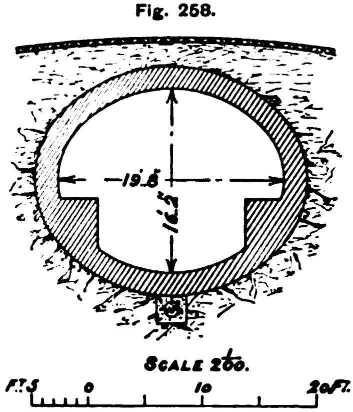
Fig. 256.—Low-level Sewer. Fig. 257.—High-level Sewer.



occurrence of deposit than branch and main sewers, and are generally made circular, or approximately circular in section; though the lower and upper intercepting sewers on each bank of the Tiber through Rome, have been given an oval form, Figs. 256 and 257. In Rome, as in London, advantage has been taken of the embanking of the river, to construct intercepting sewers in the embankments. The dimensions of these sewers are varied with the discharge and the available falls; and the sewers have to be given a sufficient discharging capacity to carry off the maximum estimated flow. The Clichy collecting sewer has been given the peculiar form of the Paris main sewers, with a central channel, and side benches for the workmen to pass along the sewer for clearing it of deposit, Fig. 258. At Brighton, tanks were constructed

CLICHY COLLECTING SEWER, PARIS.

Fig. 258.

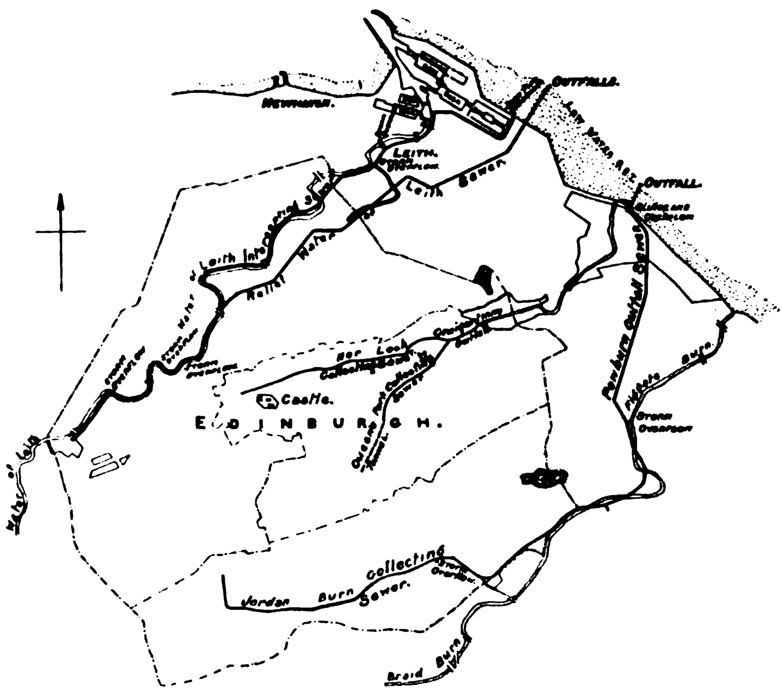


close to the outlets of the main sewers into the intercepting sewer, with their bottoms below the invert of the sewers; and the heavy sediment brought down by the main sewers from the streets drops into these pits, and is thus kept out of the intercepting sewer.

An interesting example of intercepting and collecting sewers from separate districts leading the sewage of an important town to outfall sewers, is furnished by Edinburgh, where the various drainage works have been carried out gradually.[1] The city has been divided into three districts, as shown by dotted lines on the plan, Fig. 259, each having its

EDINBURGH INTERCEPTING, COLLECTING, AND OUTFALL SEWERS.

Fig. 259.



intercepting or collecting sewers discharging into an outfall sewer. The northern district formerly discharged its sewage into the Water of Leith, a river which drains this district and flows into the Firth of Forth at Leith. Many years ago an intercepting sewer was made following the

[1] "The Main Drainage and Sewage Disposal of Edinburgh," W. Fairley, *Proc. Inst. C.E.*, vol. cxxi. p. 226, and plate 5.

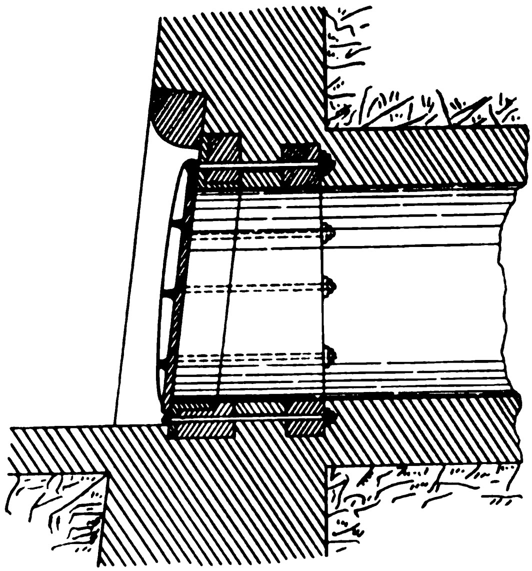
windings of the river, and at a sufficient depth to intercept the discharge of the main sewers on each side; and some years later a relief collecting sewer was constructed, diverging from the line of the old sewer at Stockbridge, and passing on to a second outfall sewer discharging at an outfall in the Firth of Forth, a little to the east of the old one, Fig. 259. The main collecting sewers of the central district discharge together into a central outfall sewer terminating in an open stream traversing meadows, whose polluted waters on reaching the foreshore, about a mile to the south-east of Leith, are intercepted and carried to an outfall beyond low water; whilst the collecting sewer of the southern district discharges into a fourth outfall sewer, which conveys the sewage to the same outfall as the central district.

Outfall Sewers.—All the flow of water and sewage collected from the main sewers of a district by collecting or intercepting sewers, has ultimately to be conveyed to its place of disposal by an outfall sewer. These outfall sewers differ from collecting and intercepting sewers in dealing with the discharge of the whole of the drainage of their districts throughout their whole length, and, consequently, remain uniform in section, except where any variation in their fall has to be introduced. They, moreover, vary greatly in their length, according to the distance the sewage has to be conveyed from the town to its outfall. Thus Liverpool, extending for about six miles along the deep channel of the Mersey between its large inner and outer estuaries, which is traversed by rapid tidal currents, discharges its sewage straight into the river through seventeen outfalls; whilst London, on the contrary, has had to carry its two outfall sewers, on either side of the Thames, $4\frac{1}{2}$ miles and 8 miles respectively, to their outfalls at Barking on the north side, and Crossness on the south side of the river. Brighton, also, discharges its sewage in the sea at Portobello, by an outfall sewer running along the coast eastwards for $5\frac{1}{3}$ miles; whereas the Paris sewage is conveyed nearly 10 miles by an outfall sewer to the lands which it irrigates, the extension of which for a further distance of 16 miles is proposed in the future, Fig. 264 (p. 352). These long outfall sewers often necessitate considerable works, such as tunnels under high ground or cliffs, siphons under rivers and streams, and bridges in other instances over rivers, as well as over roads and railways; whilst where the outfall is below high water, provision has to be made for closing the outlet against the rising tide by a hinged flap, which is raised by the issuing flow on the fall of the tide outside, Fig. 260 (p. 342), and also for the collection of the sewage, during the closure of the outlet, in a tank or an enlarged length of sewer adjoining the outfall.

As the outfall sewer has to carry off the flow of sewage from the lowest, as well as from the higher levels of a town, it is necessary in some cases to lift the discharge of a low-level collecting sewer into the outfall sewer by pumping, or, where the general level of the district drained is low,

TIDAL FLAP AT OUTLET OF OUTFALL SEWER.

Fig. 260.



to pump up the flow of a low-level outfall sewer, at its termination at the outfall, for its efficient discharge. Where a fair-sized river traverses a large town, it often forms a convenient division of the town into two independent districts, with their separate sewerage systems and outfall sewers, as exemplified by the sewerage arrangements in London and Rome, where the sewage on each bank of the river is separately provided for, and eventually discharged some distance down the river in each case, by an outfall sewer on either side.

In constructing these outfall sewers for large towns, considerable provision has to be made for future requirements; for with every extension of the town into the surrounding country, not only is the flow of sewage increased in proportion to the increase in population, but the rainfall over the area newly covered with houses and streets, flows much more rapidly off the roofs and pavements into the sewers than from unoccupied land; and therefore the variation in the discharge in the outfall sewer, as well as the amount, is proportionately increased.

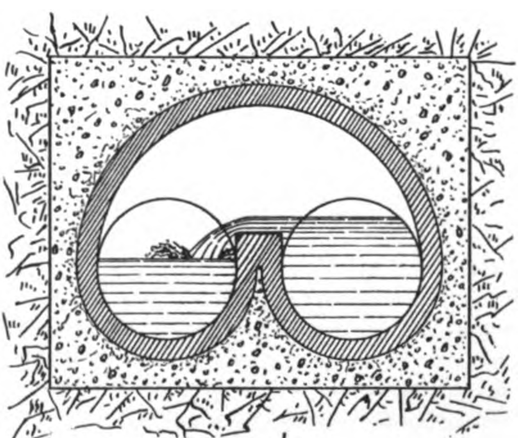
Construction of Outfall Sewers.—Large outfall sewers are almost always made circular, or nearly circular, in section, as being a suitable, and the most economical form for a large flow; and they are constructed of brickwork, masonry rendered inside with Portland cement, concrete, or ferro-concrete, except in certain special positions where iron or steel pipes are introduced. Thus where the flow takes place under pressure, as in siphons under rivers or streams, or where the sewage has to be raised to a higher level by pumping, the sewer is formed of iron or steel pipes; and metal pipes are also employed where the sewer has to be carried across a soft and shifting foreshore into the sea, being supported and kept in position by piles.

The fall available for long outfall sewers is frequently comprised between 2 and 3 feet in a mile, or even less in some parts. This small fall is occasioned by the sewer having to drain the lowest portion of a town situated on the sea-coast, often only slightly above the sea, or in the lower part of a river valley, as is often the case with large towns, where the fall of the land is small, so that the difference in level between the lowest part of the town and the outfall, consistently with obtaining a sufficiently free discharge into the sea or a tidal river during the earlier part of the ebb, or into a river above its tidal limit in flood-time, only admits of a very moderate fall for a sewer having to convey the sewage to a distant outfall.

Storm Overflows for Outfall Sewers.—In order to reduce the large dimensions of outfall sewers, having a small fall, which would be

STORM OVERFLOW FOR OUTFALL SEWER.

Fig. 261.



SCALE $\frac{1}{200}$.

necessary to discharge the whole flow during a heavy storm of rain over this drainage-area, it is customary to introduce storm overflows at suitable points, so as to relieve the sewer of the excessive influx into it at such periods, by diverting a portion of the diluted discharge into the nearest watercourse or river, or into the sea, Fig. 259 (p. 340). This can be effected by leading the outfall sewer into an enlarged culvert or chamber, with a vertical partition down the centre of it raised to a suitable height above the invert, so that the surplus water flows over this barrier into a separate conduit directly the water in the sewer has reached the desired limit, and is discharged at a convenient outlet, Fig. 261.

Manholes at Outfall Sewers.—Though outfall sewers do not generally have junctions, and follow for the most part a straight course, and being large and having always a fair flow are not liable to be obstructed like small sewers, manholes are, nevertheless, necessary for inspection, and cleansing when required; and, moreover, manholes can serve, in the suburban and country districts traversed by outfall sewers, as surface ventilators. These manholes, however, need not be placed so near together as along the smaller sewers, but may be put at intervals of about 300 to 400 yards. Where the sewer is carried through a cliff near the seashore, side adits may be driven to it from the face of the cliff for access and ventilation; and in the case of tunnelling under high ground, the shafts sunk for driving the tunnel provide afterwards means for the descent of workmen, and also serve as ventilators.

Ventilation of Outfall Sewers.—Where the discharge at the outfall is affected by the tide, and the sewage has to be stored in an enlarged, or tank sewer during the periods that the tide is too high to allow of its discharge, and the state of the tidal current precludes its satisfactory removal, it is very important that ample means of ventilation should be afforded to remove the sewer gas evolved by the volume of stagnant sewage collected in the tank, and prevent its passing up the sewer on the filling of the tank, and the consequent reduction of the free space above the water-level. Fortunately, long outfall sewers traverse uninhabited districts for the most part, and therefore enable full advantage to be taken of the outlets provided by the manholes and shafts from the sewer to the external air, which may be advantageously supplemented by special ventilating shafts, where the tidal conditions necessitate the storage of the sewage for a considerable period at each tide.

In situations like Liverpool, where the town slopes rapidly down to the river, and the sewage is discharged into the river at outfalls directly in front of the town, it is absolutely essential that the partially tide-locked outfall sewers should be most efficiently ventilated by tall shafts of ample dimensions, to preclude the possibility of the sewer gas from the accumulated sewage near the outfalls ascending the sewers to the higher parts of the town. The passage of sewer gas along a sewer from the lower to the upper portions of a district on a steep slope, may also be prevented by dividing the sewer into sections by introducing a step in the sewer at suitable intervals, and putting a valve at the outlet of the sewer just above the step, which closes against an upward current of sewer gas, and thereby restricts the sewer gas to the section where it is generated in the interval between two successive steps, for the removal of which suitable provision should be made in each section. Under very unfavourable local conditions, it may be expedient to resort to artificial ventilation, both near the outfall where the sewage has to be stored for a time, and also at the upper part of sewers draining a steeply sloping town. This may be effected by the strong upward draught of a furnace burning at the base of a high chimney; or a ventilating fan may be placed at the top of a tall shaft, for drawing the foul air out of the sewers.

Siphons for Sewers.—Sometimes when a sewer has to be carried across a river, a siphon is adopted passing under the river, as, for instance, where a bridge does not exist, or cannot be utilized, and the river is navigable. This is a very common arrangement for conduits conveying water across valleys, as has been shown in the earlier part of this book; but sewers are constructed under different conditions to conduits from reservoirs in hilly districts; and, moreover, siphons conveying sewage are much more liable to be obstructed by deposit than when used for water-supply. Accordingly, the introduction of siphons in sewers should be avoided where practicable; and when employed, these siphons require special provisions to ensure their being kept clean. It is advisable to place a manhole at each extremity of the siphon; and it is advantageous to carry the sewage in two or more metal pipes laid inside a gallery, with means of shutting off the flow through them, so that they may be readily inspected, and repaired when necessary, or renewed. Arrangements also should be made for periodically flushing these siphons; and an ingenious device for scouring away deposit, adopted first at the siphon of a sewer carried under the Seine at the Alma Bridge in Paris, consists of a floating ball slightly smaller in diameter than the siphon, which, when introduced into the pipe, leaves a small space below it as it is carried along by the flow, through which the escaping water produces a powerful scouring current along the invert of the siphon.

Control of the Discharge at the Outfall.—To regulate the discharge of the sewage, and especially at tidal outfalls to close the sewer at certain periods of the tide, and to shut out the rising tide from the sewer in the absence of a flap-valve, or in the event of its failure in closing, penstocks, or vertical gates, are provided, which are worked from a large penstock chamber, being lowered down grooves at the side, and closing tightly against a smooth flat face round the outlet of the sewer into the chamber. Penstocks are also often placed at other points of a sewer for penning back the flow, so that by its sudden release it may flush the sewer below, and more particularly at the upper end of a tank sewer, which is specially exposed to the occurrence of deposit in it during the storage of the sewage when the outfall is closed. Where the outfall sewer terminates in pipes carried across the beach to low water or beyond, the penstock chamber is placed at some convenient sheltered site above high water.

Choice of Situation for Outfall.—The utmost care has to be exercised in selecting a site for an outfall discharging into a river or the sea, in order that there may be no chance of the sewage becoming a nuisance in the neighbourhood of its outlet, or, in the case of its being discharged into tidal waters on the ebb, of its being partially brought back by the succeeding flood tide. To justify the discharge of crude sewage from a town into a fresh-water river, the river must be large enough to afford rapid and ample dilution and aeration, and there should be no town on its banks for many miles below the outfall; and in a country with small rivers, and a large and increasing population, like England, such a course is wholly inadmissible.

It is well known, from the results of experiments with floats in tidal rivers, that floating matters oscillate down and up a river for several tides before they are finally carried out to sea, the period occupied depending upon the distance to be traversed, the proportion the fresh water coming down bears to the tidal water brought in by the flood tide, and the direction and force of the wind. Theoretically, to obtain the best results, sewage, which floats on the surface, should be discharged into a tidal river directly the ebb current commences to flow down; but as the discharge at the outfall must occupy a certain period of the ebb, the sewage let out at the later time is carried back by the succeeding flood tide higher up the river than the outfall, the highest point reached by the sewage discharged last being increased by a strong wind blowing up the river, and decreased by a heavy flood coming down. Thus the sewage of London, though discharged into the Thames about 14 miles below London Bridge during the ebb tide, proved a very serious nuisance to the docks above the outfalls, in which deposits of filthy matter took place, and to other places on the banks of the river, so that the insanitary condition which, previously to the execution of the main drainage works, existed in the Thames and on its banks through London, was merely transferred lower down the river, where, however, the comparatively scattered population, and the greater volume of the tidal river, rendered it less objectionable. The sewage of London is, indeed, very exceptional in quantity; but the result illustrates the importance of most carefully ascertaining the course sewage would take when discharged during different periods of the ebb, at any point in a tidal river or estuary proposed for an outfall, under all the varying conditions of wind and fresh-water flow, by means of experiments with floats. Moreover, it must be borne in mind that as the population of a town increases, and, consequently, the volume also of its sewage, the discharge at the outfall during the earlier portion of the ebb has to be prolonged, which originally was intended to be effected at the London outfalls in the first hour of the ebb tide, but many years later was stated to be for the most part discharged during the ebb. Accordingly, in selecting an outfall on a tidal river for a large town, it must be frankly recognized that where provision is made at the opening of the drainage works for the discharge of all the sewage very early on the ebb, by collecting it during most of the tidal period in tanks, the discharge will at some future time have to be continued so much later, that pollution must be anticipated some distance above the outfall, as well as in the river below. Under these circumstances, it is generally inadvisable to discharge a large volume of sewage into a tidal river without previous clarification.

Even when it is proposed to discharge sewage into a large estuary or the open sea, it is absolutely necessary to determine by the aid of floats the set of the currents along the coast, and also the influence on them of the prevailing and strongest winds in the locality; for a powerful wind produces a very marked effect on the top layers of water in exposed situations, sufficient sometimes to reverse the direction of the current at the surface, which is running fairly strongly lower down. When the most suitable site has been chosen for the outfall, at a sufficient distance from a seaside town to prevent a possibility of its being brought back by a littoral current resulting from winds, waves, or tide, and in a position where there is every prospect of the sewage being quickly dispersed without producing pollution on the adjacent beach or banks, the discharge should take place beyond low water of spring tides, and only during the period of the ebb most favourable for the complete removal of the sewage. In fact, the interests of the persons in the neighbourhood of an outfall have to be specially safeguarded with reference to the period of the discharge of the sewage; for whereas the authorities of the town concerned protect themselves by placing the outfall at an adequate distance off, they are very liable to diminish their expenditure on their drainage works, at the expense of the inhabitants near the outfall, by making inadequate provision for limiting the period for the discharge of the sewage, and prolonging the time of discharge as their population increases.

Outfall Sewers discharging into a Tidal River.—The London outfall sewer on the northern side of the Thames, receives the whole of the drainage of the metropolitan area on that side of the river from three main sewers, a high-level, a middle-level, and a low-level sewer, together with a branch sewer from the Isle of Dogs.[1] The high-level sewer coming from the base of Hampstead Hill, and the middle-level sewer from Bayswater, converge together at Old Ford; and their flow is conveyed in two combined sewers of 9 feet by 9 feet to Abbey Mills, where they are joined by the low-level intercepting sewer, following approximately the north bank of the river from Chelsea, in continuation of the Western sewer and the Fulham branch, till it diverges at Limehouse to the north-east to reach Abbey Mills in conjunction with the Isle of Dogs branch running north. The high-level and middle-level sewers where they join at Old Ford, being of larger capacity than the two combined sewers in which their discharge is conveyed to Abbey Mills, are relieved by a large storm overflow of their excessive flow during heavy rainfall, which is conveyed into the River Lea; and these combined sewers, raised considerably above the level of the low-lying land they traverse, discharge their contents direct into the Northern outfall sewer at Abbey Mills. The low-level sewer, on the contrary, after having had the sewage which it has collected from the low lands adjoining the Thames to the west, raised 18 feet by pumps at Pimlico, in order that it might not have to be laid very low beyond, and to avoid a great lift of its sewage at its termination, has its flow, together with that of the Isle of Dogs sewer, raised 36 feet at Abbey Mills so as to discharge it into the Northern outfall sewer. This sewer consists of three culverts of identical dimensions, placed side by side in a single brickwork structure,[2] Fig. 262, raised on an embankment about 17 feet above the surface of the ground at Abbey Mills, and as much as 25 feet in some places, and laid with a fall of 2 feet per mile, in which the sewage passes by gravitation to the Northern outfall at Barking. The three culverts are connected together by overflow weirs controlled by gates, so that the flow in each of them can be equalized at pleasure, thereby providing against the overcharging of any one of them. The structure is supported for the most part on a thick foundation of concrete, carried down through the surface stratum of peat of the marshes to the underlying bed of gravel; but where the peat is very thick for about $1\frac{1}{2}$ miles at the lower end of the sewer, the structure has been carried upon brick arches resting upon concrete piers, $6\frac{1}{2}$ feet thick

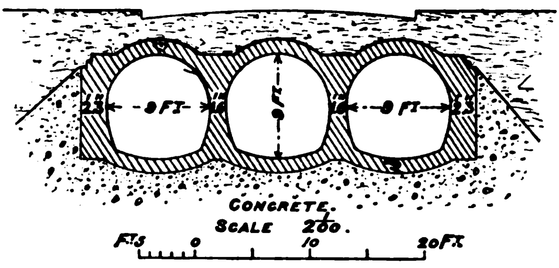
[1] "The Main Drainage of London," J. W. Bazalgette, *Proc. Inst. C.E.*, vol. xxiv. p. 295, and plate 14; and "The Main Drainage of London," J. E. Worth and W. S. Crimp, *Proc. Inst. C.E.*, vol. cxxix. p. 50, and plate 3.

[2] "Metropolitan Main Drainage and Intercepting Works, Contract Drawings," Sir J. W. Bazalgette, London, 1859–73.

and 21 feet apart, founded on the gravel bed, as being a cheaper method in this case. The concrete forming the foundations is carried

LONDON NORTHERN OUTFALL SEWER.

Fig. 262.



up against the outer sides of the culverts with a 1 to 1 slope, so as to form abutments; and the whole structure is enclosed by an embankment of earthwork, with side slopes of $1\frac{1}{2}$ to 1, and a roadway formed on the top. Near Abbey Mills, the outfall sewer is carried over a waterway with two spans of 40 feet, and over two railways and a road, in three cast-iron culverts of similar dimensions to the others, supported by wrought-iron girders; but the brick culverts cross the other roads further along on bridges. The level of the invert of the outfall sewer is 11 feet above mean sea-level; and the highest recorded tide in the Thames rose 6 feet above this level. The outlet into the river, at about low water, consists of nine 6-feet culverts, the outflow being controlled by penstocks; and the sewage from the outfall sewer was formerly stored in a large reservoir during the flood tide, to be emptied during the ebb, when the sewer also discharged direct through the culverts into the river.

The London Southern outfall sewer receives the drainage of the metropolitan districts on the south side of the river through three main low-level sewers, namely, the Effra branch from Norwood, the southern, comparatively high-level sewer from Balham, and the southern low-level sewer from Putney. The two first sewers converge to New Cross Road near the station, and are carried along that road to Deptford, where, after being relieved of surplus water by a storm overflow, they discharge by gravitation into the low-level Southern outfall, barrel, sewer, $11\frac{1}{2}$ feet in diameter, to which place the southern low-level sewer is brought, and its flow is raised 18 feet by pumping, and is discharged into the Southern outfall sewer. The combined discharge from these three main sewers is conveyed by gravitation along the low-level outfall sewer, having a fall of 2 feet per mile, to Crossness, where, being raised by pumps from 10 to 30 feet according to the water-level in the river, was formerly delivered straight into the river during the ebb tide, or stored in a reservoir during the flood tide. In addition to the clarification of the sewage before it is discharged into the Thames, considerable improvements and extensions of the pumping machinery, and the construction of relief sewers, have been carried out in recent years, to prevent the overcharging of the sewers and the frequent flooding of the low-lying districts during storms, and to provide for the delivery of the increasing flow into the river from the outlets of storm water, and from the outfalls.

Outfall Sewers discharging into the Sea.—The Brighton outfall sewer, in continuation of the intercepting sewer receiving the flow from eight outfall sewers which formerly discharged direct into the sea in front of the town, has been carried for nearly four miles from the eastern end of the town in tunnel along the chalk cliff, 7 feet in diameter, and lined with brickwork, with a fall of 3 feet in a mile, to an outfall, through a penstock chamber, across the beach into the sea.[1] The storm water escapes from the intercepting sewer by three storm overflows discharging above high water into three of the old outfall sewers, so that the outfall sewer is relieved from having to convey this water. The final portion of the outfall sewer consists of three 4-feet cast-iron pipes, discharging the sewage into the sea at low water. The outflow is controlled by a lifting gate, or penstock, near the upper end of the penstock chamber, which can be used for penning back the sewage for a short time in order to flush the outfall; and the influx of the tide is arrested by valves in the centre of the chamber, which close the sewer automatically against the flood tide; whilst a second penstock has been provided near the lower end of the chamber, for closing the sewer against the tide in the event of the valves failing to act. The outfall sewer is ventilated by twenty shafts, side entrances, and special ventilating openings, so that ventilators are provided at average intervals of about 350 yards. The eastern side of Brighton was selected for the outfall on account of the prevailing winds, greatest waves, and littoral currents giving a predominating easterly trend to the sea along the coast.

Torquay being situated on an inner recess of Torbay, is in a peculiarly unsuitable position for the discharge of its sewage into the sea in the neighbourhood of the town; but until the main drainage works were carried out some years ago, the three drainage districts into which the town and its suburbs were divided, discharged their sewage at three outfalls at low-water mark on the shore of Torbay in front of

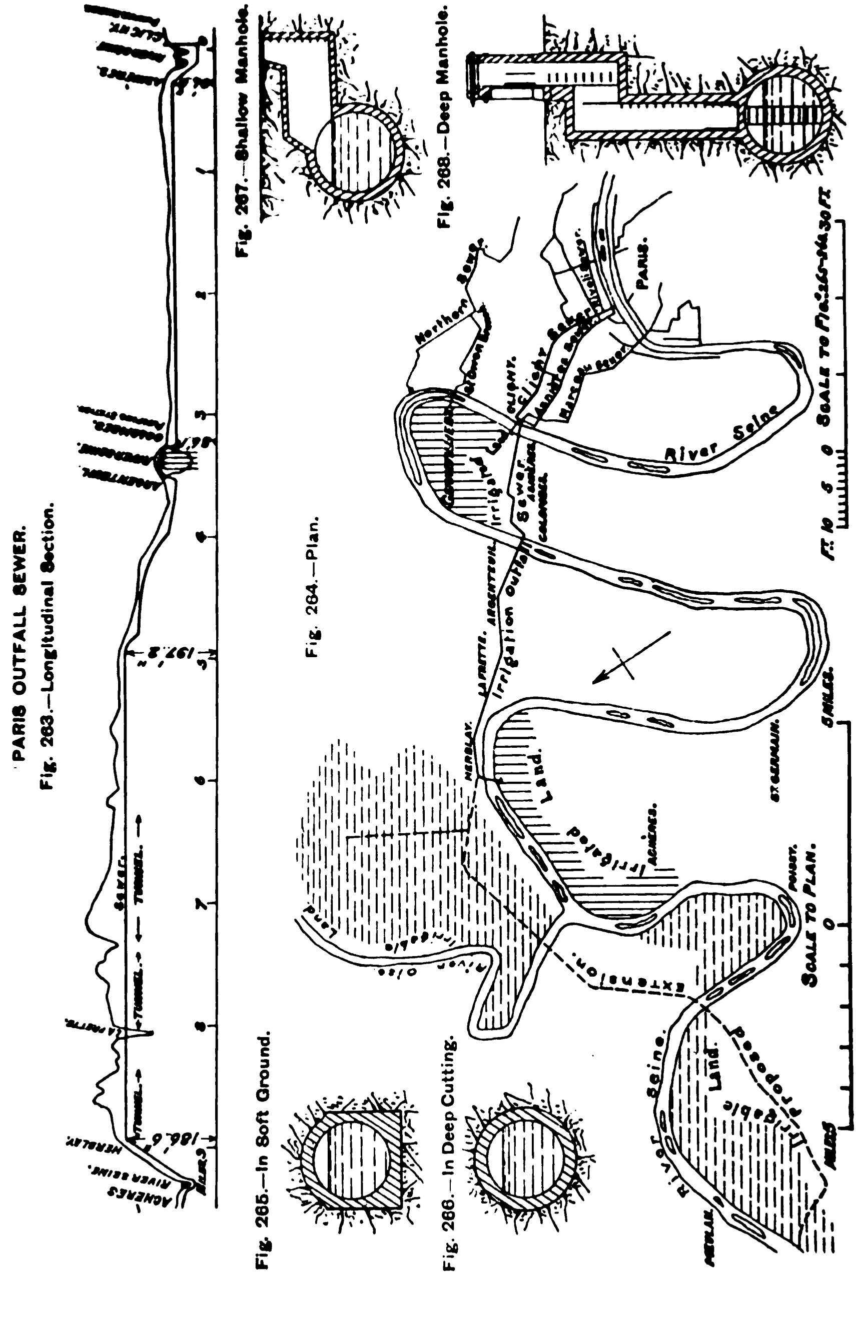
[1] "The Brighton Intercepting and Outfall Sewers," J. G. Gamble, *Proc. Inst. C.E.*, vol. xliii. p. 194, and plates 8–10.

them. It is evident that sewage should never be discharged into a partially sheltered bay or harbour; for the transporting agencies of waves, wind, and littoral currents along an open sea-coast, are more or less excluded from such localities; and the winds which alone affect the water in a bay, come from the quarter to which the bay is open, and therefore tend to drive suspended matters on to the shore. The sewage, accordingly, of Torquay was diverted by an outfall sewer to an outfall on the sea-coast outside Torbay. The sewage is intercepted and collected by two main sewers; a high-level sewer commencing near Tor Abbey, intercepting the sewage of two of the old outfall sewers, and traversing Torquay from west to east; and a low-level sewer, nearly a mile long, draining the north-east part of the town, and terminating at a pumping station adjoining the high-level sewer, where its flow is raised $10\frac{1}{6}$ feet to the upper level. From thence the outfall sewer, forming a continuation of the high-level sewer, 7 feet in diameter, carried in tunnels under the high ground, and lined with concrete through rock, was given a fall of $4\frac{1}{2}$ feet in a mile throughout its length of about $2\frac{1}{6}$ miles to its outfall at Hope's Nose, a projecting promontory slightly to the north of Torbay.[1] The outfall sewer is ventilated by seven shafts, six having been sunk for driving the tunnels and one for ventilation; but owing to the abandonment of two shafts partially sunk in high ground, the openings are at irregular intervals. The invert of the outfall is 4 feet above mean sea-level; and spring tides rise 4 feet above it.

Irrigation Outfall Sewer.—The outfall sewer conveying the sewage of Paris for the irrigation of lands lower down the Seine Valley, furnishes a very interesting example of the very varied works which may have to be carried out for the disposal of the sewage of a large inland city in this manner. The River Seine is so tortuous in its course below Paris, that it was impossible to make the outfall sewer follow one of its banks; and in taking a fairly direct course, it has been necessary to cross the river three times, and to traverse country varying considerably in elevation, so that siphons, aqueducts, and tunnels have had to be constructed, and the sewage conveyed for some distance under pressure by pumping, as well as elsewhere by gravitation,[2] Figs. 263 and 264 (p. 352). The sewage is conveyed out of Paris to the outfall sewer by three main collecting sewers, namely, the Clichy, Fig. 258 (p. 339), Asnières, and Marceau sewers, converging together at Clichy, where the combined flow is pumped up $19\frac{1}{2}$ feet into a tank on the right bank of the Seine.

[1] "The Main Drainage of Torquay," G. Chatterton, *Proc. Inst. C.E.*, vol. lxi. p. 144, and plates 4 and 5.

[2] "Travaux de l'Aqueduc et du Parc Agricole d'Achères," MM. Bechmann and Launay, *Annales des Ponts et Chaussées*, 1897 (2), p. 14, and plates 9–18.



PARIS OUTFALL SEWER.

Fig. 263.—Longitudinal Section.

Fig. 264.—Plan.

Fig. 265.—In Soft Ground.

Fig. 266.—In Deep Cutting.

Fig. 267.—Shallow Manhole.

Fig. 268.—Deep Manhole.

From this tank, the sewage passes under pressure through a cast-iron siphon, $7\frac{1}{2}$ feet in diameter and 1520 feet long, formed by a tunnel driven some depth below the bed of the Seine from a vertical shaft, $11\frac{1}{2}$ feet in diameter and $78\frac{3}{4}$ deep, by means of a shield and compressed air, and is then discharged on the opposite bank at Asnières into a circular masonry culvert, 10 feet in diameter, constituting the ordinary section of the outfall sewer, Figs. 265 and 266, through which it flows by gravitation, with a fall of 1 in 2000, or about $2\frac{2}{3}$ feet per mile, to the right bank of the Seine again at Colombes, a distance of 3 miles. At this point, the sewage is again raised by pumping, in order to cross over the Seine, and subsequently reach the higher ground beyond, the total lift amounting to 111 feet, Fig. 263. The flow is carried over the Seine in four steel pipes, $3\frac{3}{8}$ feet in diameter, laid side by side under the roadway of the Argenteuil bridge along a length of 817 feet; and from thence it is forced up the rising sewer, about $1\frac{1}{2}$ miles long, consisting of two conduits formed partly of steel pipes and partly of ferro-cement, 6 feet in diameter, laid inside an elliptical gallery, and is finally discharged at the summit-level into a masonry or ferro-cement culvert with a fall of 1 in 2000, through which it flows freely to Herblay, a distance of nearly 4 miles. This portion of the outfall sewer traversing high ground, passes through three tunnels, on two aqueducts on arches, and mostly in cutting for the rest of its length, Fig. 263. At the commencement of the branch for leading the sewage on to the lands of Achères, where the main outfall sewer at present terminates, the flow is regulated in a penstock chamber before the sewer descends the valley for crossing the Seine. This descent is effected under pressure in two cast-iron pipes, $3\frac{1}{4}$ feet in diameter, laid in a gallery along a length of 1541 feet down to the siphon crossing under the Seine. This siphon consists of two lines of wrought-iron pipes, $3\frac{1}{4}$ feet in diameter and 671 feet in length, which were connected together on the bank, closed at the ends, floated out into position over a trench dredged across the bed of the river, and sunk down into the trench by weighting them with old rails; and the siphon was then enclosed and protected by filling up the trench with cement concrete. The sewage is lastly conveyed under pressure from the siphon to the lands to be irrigated, in two conduits of ferro-cement, $3\frac{1}{4}$ feet in diameter, for a distance of 1755 feet, which constitutes the present outfall of the sewer. Manholes have been constructed along the outfall sewer at average intervals of about 300 yards, by which the sewer is reached and ventilated; and types of these manholes where the sewer is only a little below the surface, and at some depth down respectively, are shown by Figs. 267 and 268. The flow in the outfall sewer is conveyed under pressure for $2\frac{3}{4}$ miles, out of a total length of conduit of $9\frac{2}{3}$ miles. As the disposal of the Paris sewage by irrigation is to be extended to additional tracts of land further down the valley of the Seine when necessitated by the increase in population, Fig. 264 (p. 352), the outfall sewer has been given sufficient capacity to carry off a flow of 344 cubic feet per second, which amounts to about double the discharge the Paris sewers conveyed to the outfall sewer on the completion of the works. Accordingly, ample provision has been made, within reasonable limits, for future requirements; and the elevation to which the sewage is raised at the summit-level of the rising culvert, about 1½ miles beyond Argenteuil, will enable it to be delivered, without further pumping, at a sufficient pressure to be distributed over the extreme limits of the lands proposed eventually to be irrigated, reaching the vicinity of Mantes.

Outfall Sewers discharging into a Tideless River.—The intercepting sewers constructed simultaneously with the embankments along both banks of the Tiber through Rome, Figs. 256 and 257 (p. 339), deliver the sewage of the city they have collected from the district on each bank, into two outfall sewers on either side of the river, which, cutting across the bends of the river below the town, are being extended to outfalls into the river at Magliana Station on the Civita Vecchia railway on the right bank, and Mezzo Cammino on the left bank, about 6¼ miles below Rome. An ingenious arrangement in a penstock chamber on the left bank at Piazza Bocca della Verità on the outskirts

CONTROL OF STORM WATERS IN ROMAN SEWERS.

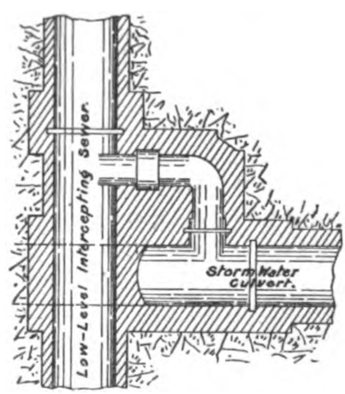


Fig. 269.—Sectional Plan.

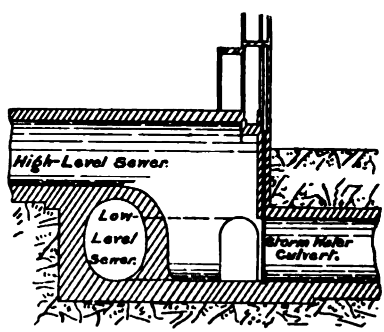
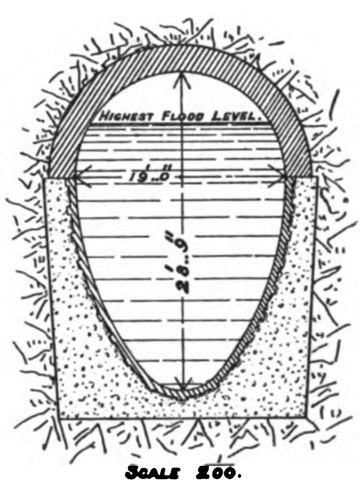


Fig. 270.—Vertical Section.

of Rome, to which some culverts at different levels converge, controls the flow by means of three penstocks, one across the culvert discharging the flow of the Mariana watercourse and the high-level sewers direct into the Tiber below the inlet of the culvert connecting it with the low-level sewer, another across the junction culvert, and the third across the low-level sewer beyond the junction. The flow of the Mariana culvert is thereby enabled to be discharged direct into the Tiber in flood-time, or into the low-level sewer in dry weather, which assists in flushing out the sewer; or, lastly, the discharge of this sewer and the flow from the higher levels can be directed straight into the Tiber during floods,[1] Figs. 269 and 270. By this means, the flow in the outfall sewer beyond is improved in dry weather, and the sewer is relieved of an excess of storm water during heavy rainfall. The two outfall sewers pass underground in culverts of ovoid section, Fig. 271, and have been given a fall of 1 in 2500 throughout.[2] Manholes have been provided along the intercepting and outfall sewers at intervals of about 186 yards; and outlets into the Tiber have been constructed at convenient places, for enabling sections of the sewers to be emptied for repairs during the low stage of the river. The penstock chamber for regulating the discharge at the outfall on the left bank, has been built about 110 yards back from the river bank, so as to avoid impeding the flow in the outfall channel by placing supports in it, the working of the penstocks being effected by cranes; and the open outfall channel joins the Tiber at Mezzo Cammino almost in a line with the river, at the termination of a concave curve of the left bank, under the protection of a long spur facing downstream, so that the discharge of the sewage takes place in the direct course of the current. A hope has been expressed that the present discharge of the sewage of Rome into the Tiber will be only a temporary expedient, and that eventually the sewage will be utilized in fertilizing by irrigation the deltaic lands bordering the river.

OUTFALL SEWER BELOW ROME.

Fig. 271.



SCALE 1/200.

Outfall Canal into Adjacent River-Basin.—Quite an exceptional form of outfall sewer has been adopted by Chicago for the disposal of its sewage, which for so long was discharged through the Chicago River straight into Lake Michigan, from which the town also

[1] "Les Égouts de Rome," M. Ronna, *Bulletin de la Société d'Encouragement pour l'Industrie Nationale*, Paris, 1897, pp. 1325 and 1326.

[2] Sections of these sewers were given me by Comm. Luiggi and Signor Fornari.

draws its water-supply, a conjunction which necessitated extending the intake a long distance into the lake to obtain a pure supply. Chicago being situated near the southern extremity of Lake Michigan, is in the basin of the St. Lawrence, not far from the water-parting between that basin and the adjacent basin of the Mississippi; and therefore it has only been necessary to cut a channel across the dividing ridge to divert the sewage of Chicago right away from the lake, and discharge it into the Mississippi Valley. This has been effected by the excavation of the Chicago Drainage Canal, twenty-eight miles in length, through the divide, which is only $11\frac{3}{4}$ feet above Lake Michigan at its lowest point, having been given a bottom width of 202 feet, slopes of 2 to 1, and a depth of 22 feet, with a gradient of 1 in 40,000, through soft soil; and 160 feet width at the bottom, nearly vertical sides, and a depth of 36 feet, with a gradient of 1 in 20,000, through rock.[1] This channel connects the Chicago River with the Desplaines River at Lockport, a tributary of the Illinois River which flows into the Mississippi thirty miles above St. Louis; and though the primary object of this work has been to serve as an outfall sewer for Chicago, it is also designed to constitute the first length of a large waterway for connecting the Great Lakes with the Mississippi. The discharge at the outfall into the Desplaines River is controlled by a series of lifting gates sliding on free rollers; and this new method of disposal of the sewage of Chicago has been in operation since 1900. Though, however, the distance from Chicago to St. Louis by this route is about 350 miles, and floating matter takes on the average about $2\frac{1}{2}$ weeks to traverse this distance, which is reduced to under ten days during the descent of floods, an action has been brought against the sanitary authorities of Chicago, alleging the pollution of the water-supply of St. Louis, drawn from the Mississippi, by the sewage discharged through the drainage canal.[2] If this charge could be substantiated before the Supreme Court of the United States, it would practically put an end to proposals to discharge the sewage of any town into the main stream or tributary of a river, from which another town derives its water-supply lower down, even if the two towns are a long distance apart. It would, in fact, prove the incorrectness of the view, very generally held, that considerable dilution, and a long course down a river, gradually oxidize sewage and convert it into innocuous constituents. A great distinction, however, must undoubtedly be made, as regards the amount of sewage admissible in proportion to

[1] "Sanitary District of Chicago. A Concise Report on its Organization, Resources, Constructive Works, Methods, and Progress," Isham Randolph, Chief Engineer, Chicago, 1904.

[2] "Sewage Disposal in America," G. W. Fuller, *Trans. Am. Soc. C.E.*, vol. liv., part E, p. 152.

the fresh-water flow, and the distance above a town situated on a river at which sewage may safely be discharged into the river, according as the town does, or does not derive its water-supply from the river; for water-supplies from rivers require to be most carefully guarded from sources of pollution, which otherwise might be quite unobjectionable.

Clarification of Sewage.—A great reduction in the pollution of a river by the discharge of sewage into it, can be effected by the previous removal of the solid constituents by settlement and precipitation; and the only objections to the invariable adoption of such a desirable course are, the expense involved, and the difficulty experienced in dealing with, and disposing of, the resulting deposit of sewage sludge without creating a nuisance. It has been estimated that ordinary sewage contains, on the average, two tons of suspended matters in every million gallons,[1] or 160,000 cubic feet, of which about half consists of mineral and half of organic substances; but the composition of sewage may be greatly modified in manufacturing towns by the introduction of trades waste. Owing to the general lightness of the suspended matters in sewage, and especially of the more objectionable portions, little separation of the solids can be accomplished by sedimentation alone, and more particularly after pumping, by which the solids are subdivided into minute particles, and commingled with bubbles of air increasing their flotation. Consequently, recourse must be had to chemicals for the proper precipitation of the solids; and lime is much the most common precipitant employed, sometimes as milk of lime, but far preferably as lime-water, as the lime in solution is the really effective chemical agent.[2] The lime-water, however, should only be added in just sufficient quantity to effect the precipitation, depending upon the composition of the sewage; for an excess of lime-water, besides unnecessarily diluting the sewage, is prejudicial to the subsequent bacterial decomposition, which so often plays a very important part in the purification of sewage, and also rather tends to increase the amount of putrefying matter in solution. The precipitating efficiency of lime is materially improved by the addition of some aluminium sulphate, or ferrous sulphate, the latter having been found much more efficient and economical for precipitating the solids from the London sewage.

For the settling and precipitation of the solids, the sewage is led into long rectangular tanks, built of brickwork or concrete lined inside with a coating of cement, and sloping up at the bottom from the inlet;

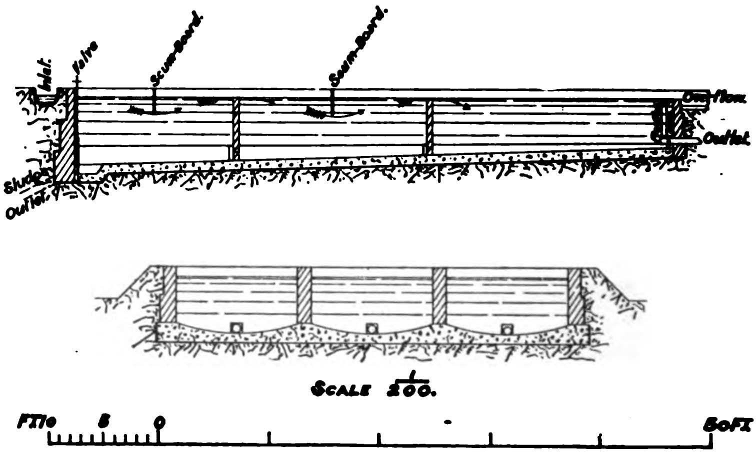
[1] "Sewage Disposal," W. Santo Crimp, "Encyclopædia Britannica," 10th edition, vol. xxxii. p. 525.

[2] "Sewage Sludge and its Disposal," W. J. Dibdin, *Proc. Inst. C.E.*, vol. lxxxviii. p. 160.

and either they are built open above the ground, their outer walls being protected and strengthened by an earthen embankment against them, Fig. 272; or they are formed underground and arched over, an arrange-

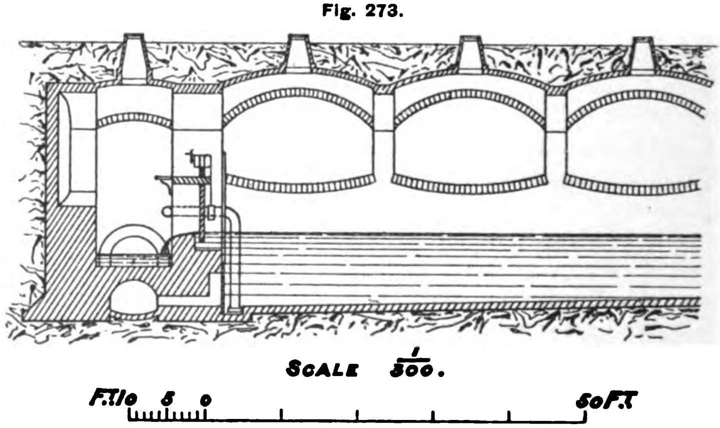
OPEN SETTLING AND PRECIPITATING TANK.

Fig. 272.



UNDERGROUND SETTLING AND PRECIPITATING TANK.

Fig. 273.



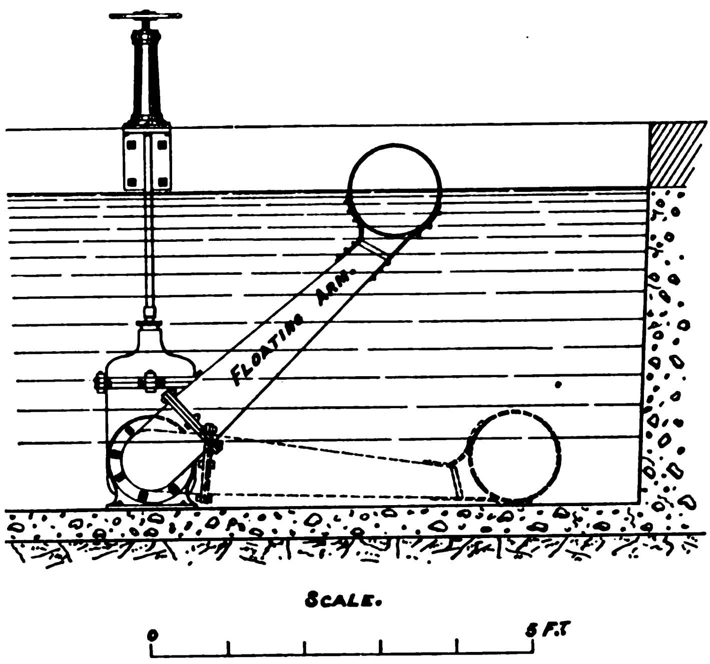
ment adopted for the Frankfort sewage alongside the River Main below the town,[1] Fig. 273. The first form of tank is more cheaply constructed, and enables the clarified sewage to be discharged into the adjacent river

[1] "Die Klarbeckenanlage fur die Sielwasser von Frankfurt am Main," W. H. Lindley, 1884.

by gravitation, but is liable to necessitate the sewage being pumped up into it, which it is desirable to avoid, if possible, previously to clarification; whereas in the second method of construction, the tank is out of sight, and protected from frost, and allows the sewage to be discharged into it by gravitation, but the admission of the clarified liquid into the river has to be accomplished by pumping. The sewage is made to flow slowly along the tank; and by means of cross walls and intermediate scum boards, it is forced to follow a devious course, keep-

HINGED PIPE WITH FLOATING INLET FOR EMPTYING TANK.

Fig. 274.



ing the liquid in due circulation throughout the tank, and in proper admixture with the chemical reagents; and during its passage the heavier portions and the precipitated solids settle down at the bottom of the tank, and the clarified liquid flows out at the far end. When a sufficient quantity of sludge has accumulated at the bottom, the liquid above it is drawn off at the outlet at the far end, through a hinged pipe, which is attached to a float at its upper end so that its orifice is always kept a little below the surface, so as to avoid the admission of the scum floating on the top when the liquid is being drawn off, Fig. 274. On the removal of the liquid, the sludge is drawn off through a pipe at the inlet end of the tank, which is facilitated by the sloping bottom. Spare tanks have to be provided for receiving the sewage during the emptying, and occasional cleaning, of some of the tanks.

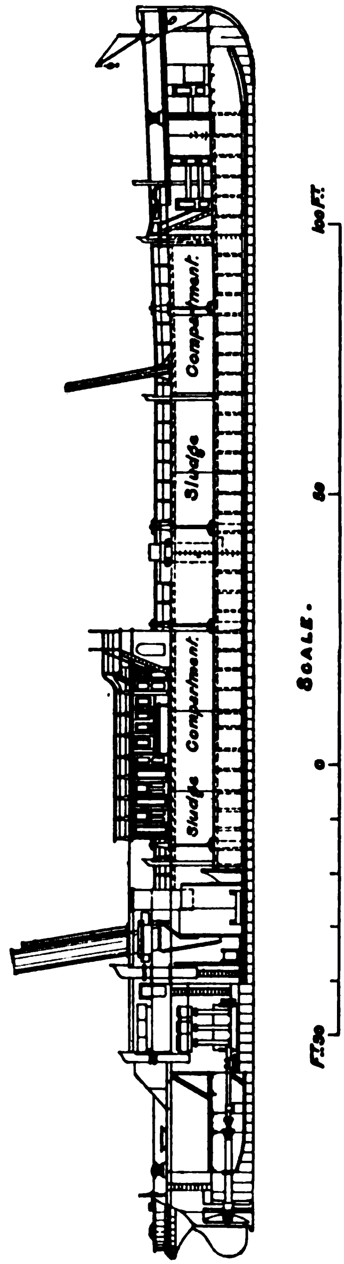
During the last few years, the sewage of London has been clarified at the outfalls at Barking and Crossness; and only the liquid effluent has been discharged into the Thames after the removal of the suspended solids, whereby the serious pollution which was taking place in the river in the neighbourhood, has been arrested, and the river restored to a fairly sanitary condition. The sewage delivered by the outfall sewers, after passing through screens to arrest coarse rubbish, is first treated with lime-water and a solution of ferrous sulphate, or copperas, and is then passed along precipitating channels, from which the clarified effluent is conveyed to the river and discharged; and the sludge is swept through culverts to a receiving chamber, from which it is pumped into the settling channels, where, after it has settled, and the liquid on the top has been drawn off by siphons and again treated, the sludge is passed into storage tanks below, ready for removal.[1] Though ferric salts are more efficient than ferrous salts for precipitating and oxidizing, copperas, besides being a very cheap reagent, possesses the advantage that, when precipitated by lime, it produces ferrous hydrate, which combining with the oxygen dissolved in the water is converted into ferric hydrate, but readily parts with it again in oxidizing the sewage, when the process is again repeated; and thus the ferrous sulphate acts as a carrier of the oxygen to the sewage.

The removal of the solids from the sewage does not appreciably modify the amount of organic matter in solution in the effluent; and its only effect, beyond greatly reducing the volume of putrescent matter discharged into a river, is that the sewage still poured in a liquid form into the river, is much more effectually diluted by the river-water, and is in a state that is more readily decomposed by natural agencies in its course down the stream. The clarified liquid must, accordingly, be regarded as an unpurified effluent, whose admission into a non-tidal river of moderate size is only allowable under favourable conditions, and in thinly populated districts, far removed from any place where the river is used for water-supply.

Conveyance of Sewage Sludge out to Sea.—Where a large town is situated within reach of the sea, the disposal of the large quantities of sewage sludge produced by precipitation in the process of clarification, by conveying it out to sea in special vessels, and depositing

1 "The Main Drainage of London," J. E. Worth and W. S. Crimp, and "The Purification of the Thames," W. J. Dibdin, *Proc. Inst. C.E.*, vol. cxxix. p. 67, and plate 4, and p. 82.

it at a distance from the coast, has occasionally proved a convenient and comparatively economical method, as evidenced by the experience of London and Manchester. The sludge collected in the storage tanks at Barking and Crossness, containing about 91 per cent. of moisture, is pumped into specially built hopper steamers, Fig. 275, which convey it outside the estuary, 10 miles seawards of the Nore, and then deposit it gradually 10 feet under water, in about an hour along 8 to 10 miles of Barrow Deep in the open sea, whilst the steamer is going ahead all the time at a speed of about 10 knots an hour so as to distribute the sludge over a considerable area, which rapidly disappears and is consumed by the animal life in the sea. In fact some traces of mineral sediment from the sludge furnish the only evidence of the large deposit that has taken place in the locality. The cost of this method of disposing of the sludge amounts to about $4\frac{1}{4}d.$ per ton.



SLUDGE HOPPER STEAMER.
Fig. 275.

The Manchester sewage after passing through a series of screens and catch-pits to retain large floating rubbish and detritus from the streets, has been treated with lime and copperas; and the precipitated sludge was forced by ejectors into two storage tanks near the ship-canal below Barton Locks, from which it was discharged by gravitation into a sludge steamer with a carrying capacity of 1000 tons, in which it was conveyed to sea and deposited beyond the Mersey bar in Liverpool Bay.[1] The clarified effluent was discharged into the ship-canal, and naturally produced considerable pollution in the water of the canal; and, consequently, a more thorough system of purification, dealing with the effluent as well as the solids, has had to be resorted to in this instance.

[1] "The Treatment of Manchester Sewage, City of Manchester, Rivers Department," J. P. Wilkinson, Manchester, 1902.

CHAPTER XV.

UTILIZATION AND PURIFICATION OF SEWAGE ON LAND.

Disposal of Sewage on Land by Small Communities—Various Methods of Purification, irrigation on land, chemical precipitation, bacterial decomposition—Irrigation of Land with Sewage, favourable conditions, considerations affecting schemes—Proportion of Land to Population, dependent on soil and method of distribution, reduced by clarification—Broad Irrigation, whole of sewage spread on land, methods of distribution, quantities of sewage applied per acre at Berlin and Paris sewage farms—Distribution of Paris Sewage at Achères, general arrangements of distribution and subsoil drainage—Intermittent Irrigation, necessary on restricted areas, methods of conducting—Crops grown on Lands irrigated with Sewage, list of suitable plants, influence of soil, climate, and position—Irrigation preceded by Precipitation of Solids, advantages, defects—Collection of Sewage Sludge, Dortmund cylindrical precipitating tank, circular tank with chambers, methods of working—Disposal of Sewage Sludge, on land, modes of drying, formation of cake in filter presses, variable value of sludge cake—Conditions affecting Financial Results of Sewage Irrigation, variety, sewage regarded as refuse—Remarks on Sewage Disposal in the Sea, and on Land, justification of waste of sewage in the sea, great merits of disposal on land.

In the methods adopted for the disposal of sewage hitherto described, the sole object is to get rid of it as a waste product, and a nuisance, in the quickest and most economical manner, consistently with the general well-being of the community. Experience, however, has proved that many products which for a long time have been considered worthless, are capable by certain processes of being turned to valuable account. Sewage especially is composed of constituents which have to a great extent been derived from the land; and the residue and products of which, after having served their purpose of sustaining human life, should in the due cycle of nature be restored to the soil, to be again used for the production of food. This has been very advantageously done in sparsely populated country districts, where by the aid of earth-closets, or other simple arrangements, the sewage of the inhabitants can be directly and continuously used for manuring the land. The sewage, however, of towns cannot be so simply utilized without causing a nuisance; though cesspools and the pail system were introduced with the object of disposal on land. Unfortunately, neither cesspools nor the pail system deal with the impure flow off the streets; whilst cesspools are quite unsuited for a densely populated town; and the flow from sculleries and sinks is not provided for by the pail system, which becomes increasingly difficult to manage satisfactorily as a town extends; and in other respects, these systems do not comply with the present-day standard of sanitary requirements. The disposal, moreover, of sewage on land is rendered more difficult by the general use of water-closets with a large flush of water, the extension of baths, and the introduction of trades waste, and especially by the great increase in volume and dilution of the sewage by the admission of rainwater into the sewers.

Various Methods adopted for the Purification of Sewage.—Irrigation on land is, on the whole, the simplest and most suitable method for disposing of the sewage of towns, as it restores as manure to the land ingredients which have been derived from it; and the filtration through the soil which takes place in the process, not only retains the solid portions of the sewage, but also by aeration and the decomposition which occurs, purifies the effluent in its passage, so that in draining out of the land it has become sufficiently pure to be discharged without risk into the nearest watercourse or river. The impediments in the way of carrying out this combined method of utilization and purification of sewage, consist in the large area of land required to provide for the large volume of liquid conveyed by the sewers, comprising sewage proper, rainfall, and sometimes trades waste, which has to be poured on to the land in wet, as well as in dry weather; the greatly reduced manurial value of the sewage, owing to its large dilution; the objections raised against the conversion of land in the neighbourhood of a town into a sewage farm; the difficulty in some places of procuring land of an adequately porous character, and of sufficient extent; and the enhanced price demanded for land for such a purpose, on account of the depreciation which is liable to ensue in the value of the adjoining property.

Frequently irrigation of land is employed as the final process for the purification of the effluent, after the suspended materials have been deposited by chemicals in the form of sewage sludge, or when the sewage has been decomposed by bacterial changes, before the effluent is allowed to be discharged into a river.

Chemical processes have been tried in considerable variety for the treatment of sewage, with the object of its purification, and at the same time procuring a valuable manure; but they have been more successful in precipitating the solids and forming sludge, than in purifying the liquid portions of the sewage. Moreover, the composition and dilution of sewage is so varied under different conditions, as to render chemical reagents, which give fairly satisfactory results in certain cases, of little value in other instances.

The decomposition of sewage into harmless constituents of considerable value to agriculture, without leaving a deposit of sludge, effected by the agency of bacteria under conditions favourable to their growth and action, in separate stages, offers the best prospects of a satisfactory treatment of sewage of general application, and is especially suitable for dealing with the sewage of places where its large volume, and the difficulty of procuring an adequate area of land of proper quality, render irrigation of land with it not economically practicable.

Irrigation of Land with Sewage.—Land is irrigated with sewage under the most favourable conditions when the sewage is delivered in a fairly concentrated form by the exclusion of the rainfall and subsoil waters from the sewers, as can be effected more or less in small towns and villages, and especially on porous soils. A small rainfall is also a favourable condition, in rendering irrigation more generally advantageous, less liable to be absolutely superfluous or harmful at times, and less likely to waterlog the ground. Thus in the arid districts of California, sewage irrigation is cordially welcomed as a very important aid to agriculture, quite irrespectively of the question of the disposal of the sewage. This method of disposal is very suitable, and has been extensively adopted, for inland villages and small towns, sewered on the separate system, which are precluded from turning their sewage, or an unpurified effluent, into the nearest stream or river; and yet where the surrounding country is sufficiently sparsely populated for there to be no difficulty in obtaining the requisite land.

In arranging a scheme for the disposal of the sewage of a district by irrigating land, it is essential to determine the area of land required for dealing with the volume of sewage discharged, depending upon the nature of the soil and the composition of the sewage, and also the manner in which the sewage is to be distributed over the land, which has to be regulated according to the amount of land available and its slope.

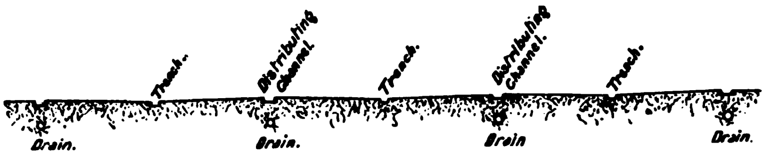
Proportion of Land to Population.—Stiff clay and peaty or marshy ground are the worst soils for irrigation; whilst sand, gravel, and loamy soil are the best. Clay soils have to be most carefully prepared by deep ploughing, mixture with ashes, and occasional subsoiling, to prevent the cracks, which tend to form in hot weather, from allowing the sewage to pass down them to the underdrains without undergoing purification. The area of land required for purifying a definite volume of sewage does not, however, depend merely upon the physical condition of the soil, but also in some cases on its chemical composition; for a certain loam at Dursley, in Gloucestershire, was found to possess a remarkable purifying efficiency, which was traced to its containing about 8 per cent. of calcium carbonate, a substance promoting the growth of nitrifying organisms. Moreover, the area needed, largely depends upon the manner in which the sewage is distributed over the land, and the condition of the sewage when it is applied. Thus in England, when broad irrigation is adopted, and the raw sewage is distributed continuously over the surface of the land, the official requirements, under normal conditions, are one acre for every 25 persons on stiff clay, and for 100 persons on loamy gravel; whilst with intermittent filtration, where dosing the land with sewage is alternated with periods of rest, from 100 to 300 persons per acre are allowed on sandy gravel, clay not being a suitable soil for such treatment. When precipitation of the solids precedes irrigation, one acre is considered sufficient for the broad irrigation of the sewage of 200 persons in the case of clay, and of 400 persons with loamy gravel; whilst with intermittent filtration on sandy gravel under such conditions, the sewage of 500 to 600 persons can be dealt with per acre. Where, in addition to precipitation, the passage of the sewage through special filters is provided for previously to irrigation, one acre of land is considered sufficient for receiving the sewage of each 2000 of the population accommodated. In England, the actual sewage, as deduced from the water-supply, averages between 30 and 40 gallons per head per day; but the volume discharged by the sewers varies greatly according as the rainfall is admitted into the sewers, or the separate system is partially or wholly adopted. The clarifying of the sewage previously to irrigation has the great advantage of reducing considerably the amount of land required, where land is difficult to obtain or the soil is not well adapted for irrigation; but it involves the production of sludge which has to be disposed of in some way.

Broad Irrigation for the Disposal of Sewage.—In places where an ample area of land can be obtained for sewage irrigation at a reasonable cost, the distribution of the raw sewage over the surface of the ground, known as broad irrigation, can be resorted to. By this arrangement, the whole of the constituents of the sewage taken originally from the land, are restored to it, and serve to manure it; and the effluent being relieved of its solids and purified, by passing through the ground in the case of porous soils, or flowing slowly along near the surface of clay soils, is eventually discharged into a watercourse.

On flat land, the ground is laid out with slightly raised parallel ridges about 40 feet apart, along the top of which the main distributing channels, formed of brickwork, concrete, or stoneware, are carried; and by arresting the flow of the sewage in them at suitable points by stop-boards or iron plates, the sewage is made to overflow on each side, and pass gradually down the flat slopes and percolate into the soil according to its porosity, or find its way into the trenches at the bottom of the slopes, along which it can flow to irrigate lower land beyond, or from which it passes into the subsoil drains, Fig. 276. Where the land has

BROAD IRRIGATION ON FLAT LAND.

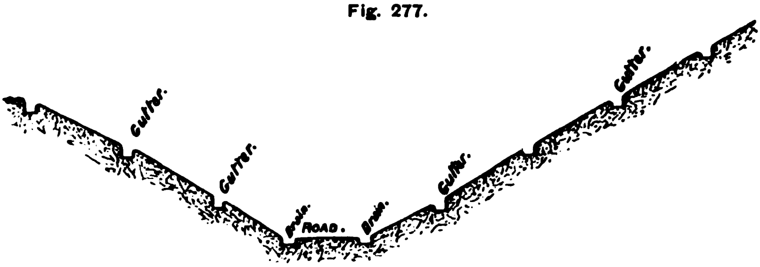
Fig. 276.



a good slope, a series of contour channels resembling catchwater drains are formed at intervals down the slope, which successively receive the sewage flowing down the land above, and in turn distribute it by overflow on to the sloping land below, Fig. 277.

BROAD IRRIGATION ON SLOPING LAND.

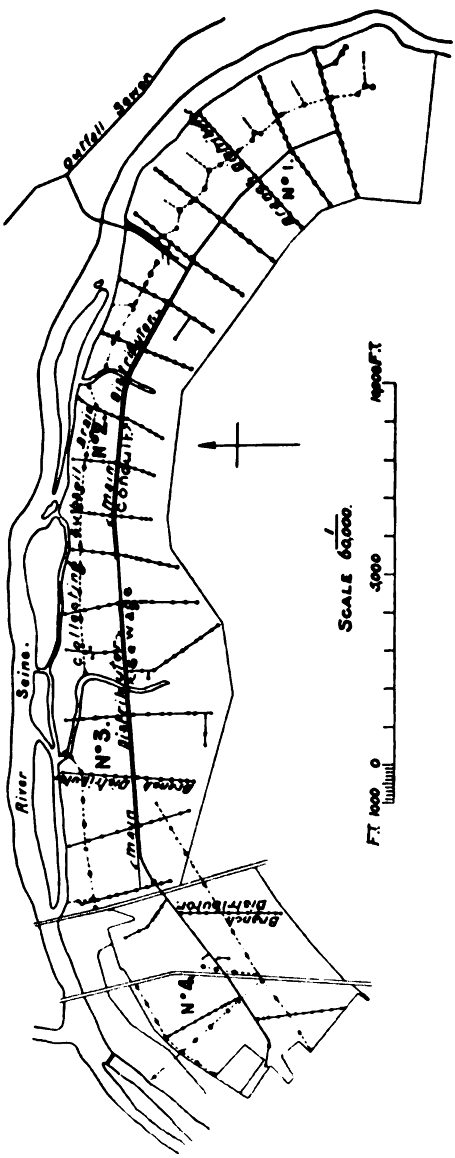
Fig. 277.



To prevent the formation of a layer of black decomposing matter on the surface of the irrigated land, preventing aeration and creating a nuisance where crude sewage is freely applied, it is necessary to allow periods of rest for the occurrence of the requisite chemical changes, or to break up and turn in the surface layer of sludge by periodical ploughing and subsoiling. In broad irrigation, as in the other systems, the amount of liquid that can be discharged on the land depends necessarily on the nature of the soil and the dilution of the sewage. Thus as regards the sewage of the two cities for the disposal of which sewage farming has received its greatest development, namely, Berlin and Paris, the average daily volume of sewage distributed over the land per acre is between 2000 and 3000 gallons in the case of Berlin, where the farms have a moderate depth of light soil, and the sewage is only about 19 gallons per head per day; whereas at the Achères farms, where the soil consists of sand and gravel, the maximum regulation daily quantity of Paris sewage that may be distributed on the farms amounts to 9800 gallons per acre, for in Paris the whole of the rainfall is discharged into the sewers. The irrigation, however, of the different plots at Achères is only carried out at intervals of from three to five days according to the crops, so that the sewage is being distributed over only one-third to one-fourth of the total area at any one time.

DISTRIBUTION OF PARIS SEWAGE, AND SUBSOIL DRAINAGE AT ACHÈRES.

Fig. 278.



Distribution of Paris Sewage at Achères.—The general method by which the Paris sewage is distributed over the Achères domain, and the subsoil drains by which the purified effluent passes into the Seine, are shown in Fig. 278. The irrigated land bordering the Seine, about $6\frac{1}{4}$ miles in length and $\frac{2}{3}$ mile in breadth, is divided transversely into four sections, which can be entirely shut off from one another by means of valves placed across the carrier of the sewage.[1] The two ferro-cement conduits, $3\frac{1}{4}$ feet in diameter, conveying the sewage under pressure from the Herblay siphon pipes, passing under the Seine on to the Achères farms, discharge into a single ferro-cement carrier, $3\frac{3}{5}$ feet in diameter, which traverses longitudinally the two central sections, and is prolonged at its two extremities across the two outer sections by distributing pipes, starting with a diameter of $2\frac{3}{5}$ feet, which is reduced further on to 2 feet. Distributing pipes of the same dimensions run alongside the carrier on its northern side throughout the central sections; and these distributing pipes, in all four sections, deliver their flow at intervals of about 1300 feet on the average, into a series of transverse distributing pipes, $1\frac{1}{3}$ feet in diameter for the most part, with outlets placed at suitable positions with regard to the slope of the ground, generally from 250 to 330 feet apart, for distributing the sewage over the land. The irrigated land is laid out in ridges and furrows, so that the roots of the plants placed on the ridges may receive a watering by infiltration from the sewage flowing along the furrows, whilst the leaves are preserved from being wetted by the sewage.

In spite of the porous character of the soil at Achères, and the general fall of the subsoil water towards the Seine into which it eventually finds an outlet, the rapid flow of the drainage waters into the river is prevented along the convex portions of the river bank by an impermeable layer of muddy alluvium; and, accordingly, irrigation soon raises the plane of saturation under these conditions, leading to the flooding of the low parts of the lands, and unduly reducing the thickness of the filtering layer of dry soil. Subsoil drains have, consequently, been laid, as shown by dotted lines on the plan, Fig. 278, to collect the drainage waters tending to accumulate along the upper side of the impermeable barrier, and convey them at convenient points through this stratum into the river, or into a watercourse joining the river; and by this means, the subsoil waters are kept down at a low enough level to avoid flooding, and provide for efficient irrigation.

Intermittent Irrigation for Sewage Disposal.—In broad irrigation, an endeavour is made to utilize the sewage as far as possible for the benefit of the crops, in combination with its purification. Where, however, an ample area of suitable land cannot be obtained, it becomes necessary to aim primarily at the purification of the sewage, treating the raising of crops as a subsidiary consideration. It has been found that the purification of sewage can be effected on a smaller area of land if the irrigation is conducted intermittently by interposing intervals of rest between the waterings, and also providing ample subsoil drainage.

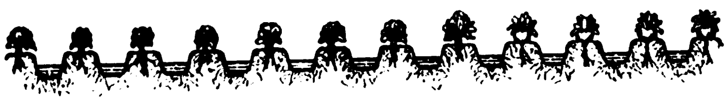
[1] "Travaux de l'Aqueduc et du Parc Agricole d'Achères," MM. Bechmann and Launay, *Annales des Ponts et Chaussées*, 1897 (2), p. 202, and plate 18.

This operation of intermittent irrigation can be carried out by two different methods. In the one case, the land is treated like a series of tanks, into which the sewage is poured in succession, and filters through the soil to the drains; and long intervals of repose are allowed for the thorough aeration of the land, aided when requisite by scraping and ploughing, and the resulting bacterial changes. This method, which in reality combines simple filtration through soil with bacterial decomposition, has the defect that pouring sewage over the leaves is injurious to most crops, so that only special crops can be grown which are not affected by this treatment, or the irrigation must be stopped as soon as the plants appear above ground.

The second method of conducting copious intermittent irrigation obviates the above serious objection with regard to the crops. It consists in laying out the land with a series of broad ridges and furrows, the plants being grown on the ridges, and the sewage poured into the furrows or trenches, Fig. 279. A main carrier constructed along the

IRRIGATION WITH RIDGE AND FURROW.

Fig. 279.



highest part of the land, at right angles to the lines of ridges and furrows, discharges the sewage into the furrows at suitable intervals; and the sewage gradually percolates sideways to the roots of the plants growing on the ridges, on its way to the drains. The periods of rest left between the filling of the furrows allow for the bacterial decomposition of the deposit of sewage in them, and also its aeration, which latter is further promoted by the drains if suitably laid.

Crops grown on Land irrigated with Sewage.—Necessarily the crops raised on sewage farms depend upon the system of irrigation adopted, the nature of the soil, and also to some extent on the position of the farm in relation to the market for its produce. Those crops are naturally the best which flourish with an abundance of moisture, such as Italian rye-grass and other grasses, prickly comfrey and other quickly growing green fodder, and osiers. Certain root crops, also, are often grown on sewage farms, such as beet-root, mangold-wurzels, carrots, potatoes, parsnips, and onions. Celery, cabbages, cauliflowers, and other market-garden produce answer well with the ridge and furrow system; but cereals are generally only irrigated before the plants come out of the ground, and tend to run to straw. Strawberries, gooseberries, currants, and fruit trees give very satisfactory results under sewage irrigation. Italian rye-grass appears to absorb the largest quantity of sewage; but after three years it has to be replaced by root crops. It has been generally found that the plants commonly grown in the locality succeed best on sewage farms, indicating the influence of soil and climate on the crops, provided of course that ample watering is not liable to affect them injuriously.

At Gennevilliers, where the first farms for the disposal of the Paris sewage were established, market-garden crops have proved the most profitable, owing to the presence of a large rural population, and its proximity to the Paris markets. On account, however, of the difficulty of obtaining adequate labour at Achères, and the distance of these sewage farms from Paris, green fodder, and especially lucerne, a favourite crop in that locality, are mostly cultivated, together with beet-root, carrots, and potatoes. Lucerne has also been very successfully grown under very different conditions at Karachi, as well as Italian rye-grass, guinea-grass, some cereals, and vegetables. The great difference in the climatic conditions of the valley of the Seine and Karachi, both in regard to temperature and rainfall, is forcibly illustrated by provision having had to be made at Achères for the discharge through subsoil drains into the Seine of one-third of the volume of the sewage distributed over the farms, it being estimated that about another third is absorbed by the crops, and the remainder evaporated; whereas all the liquid portion of the sewage distributed over the Karachi sewage farm, at intervals of eight or nine days, is evaporated and absorbed by the soil, so that no effluent passes off from the fields.[1]

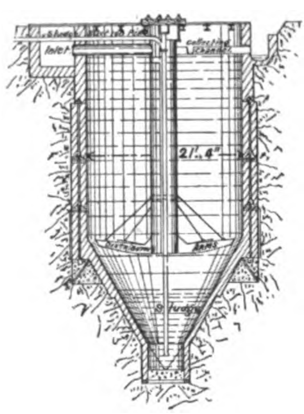
Irrigation preceded by Precipitation of the Solids from the Sewage.—Often in inland suburban districts, where land is costly or difficult to obtain in a suitable position, as for instance in the case of the towns and villages in the neighbourhood of London, the sewage is delivered into tanks, and the solids precipitated by chemicals before being distributed over a sewage farm. This preliminary treatment greatly reduces the area necessary for the purification of the clarified liquid, as compared with raw sewage; it renders the land much less liable to be choked with a coating of slime; and it renders broad irrigation practicable on heavy soils. On the other hand, however, this system removes some of the constituents of the sewage which are valuable for manuring the land, and in producing bacterial changes; it demands great care in the selection and volume of the precipitants, so as not to be injurious to the land during the subsequent irrigation with the clarified liquid; and it necessitates the drawing off, solidifying, and disposal of the precipitated sludge.

[1] "Karachi Sewerage Works," J. Strachan, *Proc. Inst. C.E.*, vol. cxxxv. pp. 278-279.

Collection of Sewage Sludge.—The rectangular tanks commonly provided for the sedimentation and precipitation of the suspended solids in sewage, have been already described in the last chapter under the heading of "Clarification of Sewage," Figs. 272 and 273 (p. 358); but occasionally, when the space available is very limited or the conditions exceptional, deep cylindrical tanks, sunk down in the ground, are employed, which are terminated by an inverted conical bottom, in which the sludge collects, and is concentrated in the narrowing tank on passing down the sloping sides towards the centre, Fig. 280. This form of tank was first adopted at Dortmund, where the sewage, owing to its receiving large quantities of fermenting refuse from numerous breweries, becomes rapidly very foul, which created a serious nuisance in the ordinary tanks by the putrefaction of the sludge, so that these tanks have since been only used as preliminary settling tanks. Each of the tanks at Dortmund has a cylindrical portion, about 30 feet in depth and $21\frac{1}{3}$ feet in diameter; and the inverted conical part at the bottom adds about 18 feet to the depth. The sewage after the addition to it of the precipitating chemicals, is passed down a central pipe to the bottom of the cylindrical portion, where it is distributed throughout the whole area of the base of the cylinder by a series of

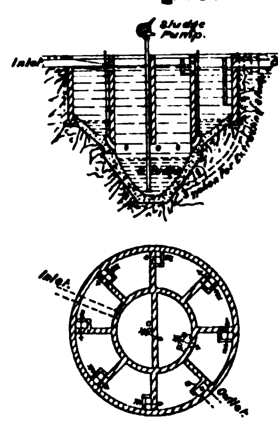
CYLINDRICAL SEWAGE PRECIPITATING TANK.

Fig. 280.



CIRCULAR PRECIPITATING TANK WITH CHAMBERS. COSHAM'S SYSTEM.

Fig. 281.



SCALE $\frac{1}{300}$.

F.T 10 5 0 — 50 F.T

radial arms, and gradually rising up in the cylinder, a thorough admixture of the sewage and chemicals is effected, resulting in the precipitation of the solids, which settle down in the conical bottom. The clarified effluent rising to the surface of the liquid in the cylinder, overflows into a channel by which it is led for purification on to land or to filter-beds; whilst the sludge, which accumulates in a concentrated form in the contracting conical base, is periodically drawn off by suction through a central pipe, dipping down nearly to the bottom of the tank, into receivers at the top, from which the air has been exhausted. An objection has been raised against this form of tank, that the sludge is liable to adhere to the sloping sides of the cone, and thus diminish the clarification of the effluent; but the conical shape undoubtedly facilitates the removal of the sludge, and appears preferable to a deep cylindrical tank with a flat bottom. Nevertheless, these deep tanks are considerably more difficult to cleanse than the shallow, rectangular tanks in ordinary use.

Another precipitating tank, similar in general form, but shallower, and with a different internal arrangement, is shown in Fig. 281. This circular tank, introduced at Nuneaton,[1] contains a series of side chambers separated by radial walls, through which the treated sewage, introduced into the central division, passes in succession, flowing in its course under one wall and over the next, and so on, thereby depositing a portion of its precipitated solids in each chamber, till at length it is discharged from the tank, through the outlet in the last chamber, as a clarified effluent. The solids settling to the bottom in each side chamber, are periodically discharged through sluiceways into the central conical receptacle, from which the concentrated sludge is eventually removed by pumping or suction.

Disposal of Sewage Sludge.—The sewage sludge, as delivered from the precipitating tanks in inland places, can only be disposed of in its existing liquid condition by being ploughed into the ground, or poured into trenches and covered over with earth; whilst in most cases this course is not practicable, and is very liable to cause a nuisance. Some drying process is, accordingly, generally necessary, depending largely upon the climate of the locality. In dry countries, by discharging the liquid sludge into wide trenches dug in a porous soil, the drying may be accomplished by absorption and evaporation; or by pouring the sludge into tanks, the water may be removed wholly by evaporation. Drying the sludge in wet countries by artificial heat is expensive, and liable to produce noxious fumes. The most satisfactory method of rendering the sludge capable of disposal and available for manure, especially in moist climates, is by converting it into solid cakes, by forcing out the liquid by pressing the sludge, enclosed within jute canvas or other filtering medium, between flat plates, enabling the

[1] "Sanitary Engineering," Col. E. C. S. Moore, R.E., 2nd edition, p. 547, and plate 48.

fluid to ooze out, and leaving the solidified sludge behind. By this treatment, about 5 tons of sludge are converted into 1 ton of hard portable cake; and it has been estimated that about 8 to $9\frac{1}{4}$ tons of pressed cake can be derived from a million gallons of ordinary sewage. The use of lime as a precipitant is important in the case of sewage where the sludge is to be solidified in a filter press, owing to its dissipating the glutinous character of ordinary sludge, which interferes with the formation of hard cake; and lime is sometimes added to deposited sludge with this object.

The value of the sludge cake as manure depends on the nature of the sewage, the precipitants used, the extent to which the water is eliminated by pressing, followed by drying in the air, and various local conditions, such, for instance, as the predilections of the farmers, the other forms of manure readily obtainable, and their cost. In some cases there is a fair sale for the cake; sometimes farmers will not take it as a gift; and occasionally it is burnt in a destructor as refuse, or as a poor class of fuel, from which the remaining moisture has to be driven off before it becomes of any calorific value.

Conditions affecting the Financial Results of Sewage Irrigation.—It is unnecessary to consider, in connection with irrigation on land, those cases where, in addition to the precipitation of the solids, the clarified effluent is passed through special filters before being turned on to the land; for under such conditions, the irrigation is only a final stage in the purification of the effluent, in which chemical processes and bacterial changes play the principal parts.

In all sewage farming, local conditions in respect of climate, rainfall, suitable soil with its position and slope, the practicability of adopting the separate system of sewerage, the cost of labour, and a market for the produce, are most important elements in the financial results of this mode of disposing of sewage. Occasionally, under a combination of favourable conditions, and with good management, a fair return is obtained on the capital expenditure and cost of working; but where, as in the case of the Paris sewage, all the rainfall, except during exceptional storms, passes down the sewers, and very expensive works have to be constructed to bring the sewage on to the land at a distance, and this has to be effected by the aid of considerable pumping, it is impossible to render the system remunerative. In most cases, sewage farms must be regarded as the cheapest means available for the disposal of the sewage under certain conditions, without creating a nuisance, and valuable from an agricultural standpoint in restoring fertilizing constituents to the land, which would be otherwise wasted; and, though formerly great expectations were formed as to the financial success of such a method of dealing with sewage, it has gradually been realized that, though benefiting the land, sewage thus utilized must be reckoned as refuse which it is advantageous to dispose of at a moderate cost.

Remarks on Sewage Disposal in the Sea and on Land.—It has sometimes been urged that depositing sewage and sludge in the sea is a waste of valuable manure; but an unanswerable reply to this accusation from a ratepayer's point of view, is that, though the authorities would gladly allow any one to take these products off their hands free of cost, and obtain whatever profit they can out of them, provided that they do not create a nuisance thereby, no offers are forthcoming. Theoretically, it is expedient that these waste products should be turned to profitable use; but so many difficulties surround the problem, that the combined efforts of chemists and engineers have been unable in most instances to find a remunerative solution. Till this result is achieved, it is impossible to blame communities for disposing of their refuse sewage by the most economical method attainable, in spite of the apparent waste that it sometimes entails. The great merit sewage irrigation possesses, as compared with discharging the sewage into a river or the sea, is that it utilizes the sewage to the utmost extent practicable, with great benefit to the land, and, under proper management, at a reasonable expense, and with efficient purification of the effluent.

CHAPTER XVI.

CHEMICAL, ELECTROLYTIC, AND BACTERIAL PURIFICATION OF SEWAGE.

Different Processes tried for the Purification of Sewage—Chemical Processes, difficulties encountered, A.B.C. process, ferrozone and polarite process, lime with copperas or aluminium sulphate, objections to chemical purification, deodorization, clarification—Electrolytic Processes, methods adopted, changes effected, oxychloride system—**Bacterial Purification of Sewage**: action of bacteria; Methods adopted for Bacterial Purification, sedimentation, irrigation, processes in use referred to; Intermittent Sand Filtration, mode of conducting, precautions needed; Contact Beds, description, cycle of operations adopted, liability to fill up, siphonic connection of beds, at Sutton, and at Hampton; Continuous Trickling Filters, object, description, different arrangements, merits and defects; Septic Tank, description of working; Upward Filtration, effective bacterial action in due sequence; Successive Changes effected in Bacterial Purification, description of changes in three stages, divergent views as to relative action of bacteria, advantages of continuous processes; Bacterial Treatment of Trades Waste, satisfactory results; Regulations of Local Government Board, principal regulations briefly defined—Examples of Complete Bacterial Purification, experimental septic tank and filters, or contact beds, at Exeter described, extension of system to the whole of the sewage, plant for bacterial treatment of Manchester sewage described, explanation of arrangements for upward filtration and subsequent nitrification—Concluding Remarks on Bacterial Purification of Sewage, advantages of system, value in combination with irrigation, successful solution of problem of sewage disposal.

VARIOUS attempts have been made for a long time past to discover processes for the purification of sewage, without having recourse to irrigation where the land is unsuitable or difficult to obtain; and chemical treatment on a large scale was naturally resorted to, as a means by which it was hoped to convert a noxious product into a valuable manure, or, at any rate, to render it perfectly innocuous. More recently electrolysis has been tried, which, though really chemical in its action, possesses the advantage of applying the purifying reagents in a nascent state, when they exercise a more energetic action than under ordinary conditions. Lastly, of late years it has gradually been realized that the decomposition of sewage, which had been found to take place under certain conditions, is the result of the action of innumerable living micro-organisms, termed bacteria, when placed in surroundings favourable to their growth; and the changes produced by these bacteria appear to offer the best solution of the difficulties connected with sewage disposal at inland districts, in cases where irrigation on land is not economically practicable, or where it is necessary or desirable to keep the area irrigated within narrow limits.

Chemical Processes for the Purification of Sewage.—One great difficulty met with at the outset in the endeavour to effect the adequate purification of sewage by chemical reagents, consists in the very variable composition and dilution of the sewage in different places. It is evidently expedient for chemical purification, especially on economical grounds, that the sewage should be as concentrated as possible, thereby allowing of the use of a smaller quantity of chemicals, rendering the admixture more easy, and expediting the requisite reactions; whilst the chemical reagents employed, and their amounts, have to be varied in accordance with the differences in the constituents of the sewage.

A process of purification which was extensively tried, and adopted at some places many years ago, is known as the A.B.C. process, so called on account of these being the first letters of the original substances used, namely, alum, blood, and clay, which appellation was doubtless further serviceable in suggesting the simplicity of the process, though charcoal was subsequently substituted for blood. In this process, the alum serves as the precipitant or clarifier, and the charcoal and clay as deodorizers. In a more recent and effective process, ferrozone, composed chiefly of ferrous sulphate, magnetic oxide of iron, and sand, together with small quantities of aluminium, calcium, and magnesium sulphates, has been employed as the precipitant; and the clarified liquid is filtered through polarite, consisting mainly of magnetic oxide of iron and sand. In this case, the ferrous sulphate precipitant is assisted by the heavy magnetic oxide in producing a rapid subsidence of the solids; and the magnetic oxide of iron, which forms the main ingredient of the filter, exercises a powerful oxidizing effect on the organic constituents of the effluent.

Many of the chemical processes which have been brought forward from time to time, consist in the treatment of sewage with lime in combination with some other substance, due probably to the efficiency of lime as a precipitant and its cheapness. These processes, however, can only be regarded as producing a more or less satisfactory clarification of the sewage, as indicated on pages 357 and 360 in the case of lime-water with copperas, or with aluminium sulphate; for no chemical treatment of sewage hitherto proposed has by itself produced a properly purified effluent; whilst an excess of the reagents, as for instance with lime, is liable to dissolve some of the suspended organic matters in the sewage, and thereby actually increase the amount of organic impurity in the effluent. The addition of chemicals necessarily increases the amount of the solids which have to be eventually disposed of as sludge; and where, as in the A.B.C. process, large quantities of insoluble precipitants are used, the volume of sludge deposited is largely augmented. Moreover, certain chemicals render the effluent unsuitable for its disposal by dilution in a river, like lime, for example, which is fatal to fish; whilst salts rendering the effluent acid are also objectionable.

It is possible by means of some chemical reagents to sterilize or deodorize sewage; but in many cases the cost would be excessive for general adoption; in other instances the changes produced would not be free from smell, of which the powerful disinfectant calcium chloride furnishes an example; and in some, the sterilization would be only temporary. Under special circumstances, such as for offensive sewage from hospitals, expensive disinfectants may be employed, as, for instance, cuprous chloride and cupric sulphate; whilst the London sewage discharged into the Thames was for a time deodorized, during the construction of the precipitation works, by an acid solution of sodium permanganate, which, when mixed with the sewage, rapidly destroys the putrescent matter by the free generation of ozone, and has been proposed to be used for the deodorization of the effluent, after precipitation of the solids, if at any time it should prove a nuisance when discharged into the river.[1]

Notwithstanding the efforts of chemists for so many years to devise processes for converting sewage into a valuable product, and the apparently considerable improvements in respect of purification which, under specially favourable conditions, have seemed for a time to be derived from some of these attempts, no process of chemical purification has been presented which is at all generally applicable, or which has materially reduced the organic impurities dissolved in the effluent. The clarification, however, of the sewage, as a preliminary operation to the purification of the effluent by irrigation, dilution, or bacterial treatment, has been successfully achieved, lime with ferrous sulphate or aluminium sulphate having given in most cases the best results.

Electrolytic Processes for purifying Sewage.—By passing an electric current through sewage, the water and chloride salts contained in it are broken up into their elements, oxygen and chlorine being

[1] "Sewage Sludge and its Disposal," W. J. Dibdin, *Proc. Inst. C.E.*, vol. lxxxviii. p. 165.

liberated at the positive pole; and these gases being evolved in a nascent state, exercise a powerful deodorizing effect on the sewage. When iron plates are employed as the electrodes between which the sewage is made to flow, iron salts are produced by the electrolysis, which precipitate the solids. Aluminium plates have also been tried, though their cost is prohibitive; and in this case, the aluminium hydrate produced serves as the precipitant. This system of purification is not well suited for large volumes of sewage; and a very serious objection to the process is the danger that the electrolytic action may be merely local, and that a considerable portion of the sewage may pass between the plates without undergoing purification.[1]

In another process, an electric current is passed through sea-water, or a solution of magnesium and sodium chlorides, causing the decomposition of the magnesium chloride into magnesium hydrate and hypochlorous acid, the former being precipitated, and the latter acting as a disinfectant. The resulting liquid is discharged into the sewage at the head of the sewer, or is used for flushing the water-closets and drains. In this manner, electrolysis is employed for producing an active and serviceable disinfectant. This process has been recently developed for use on a larger scale, by the design of a special apparatus for conducting the electrolysis, with a large electrical area, under the name of the oxychloride system of purification; and the liquid produced has been applied to raw sewage and effluents with satisfactory results.[2]

BACTERIAL PURIFICATION OF SEWAGE.

The most recent method adopted for the purification of sewage consists in promoting the decomposition of sewage by the natural action of bacteria, by placing the sewage in conditions favourable to their growth for the time requisite for the completion of the changes. It has been discovered that two distinct species of bacteria produce the changes in sewage favourable to its purification, namely, anaerobic bacteria which flourish best out of contact with air, and aerobic bacteria which require air for the performance of their functions. The anaerobic bacteria, acting in the dark and shut off from air, cause putrefaction of the sewage, liquefying the solids, but producing noxious fumes in the operation. The aerobic bacteria, with the aid of the oxygen in the air, produce the nitrification of the sewage without the emission of any offensive odours.

Methods adopted for Bacterial Purification.—An important preliminary step to any bacterial purification, is the removal of the

[1] "Sewage and the Bacterial Purification of Sewage," S. Rideal, p. 150.

[2] "Oxychloride Sewage Purification," *The Sanitary Record*, October 6, 1904.

mineral solids in suspension from the sewage; for these solids cannot be acted upon by the bacteria, and merely clog up by degrees the land or filters in which the bacterial changes take place, and, consequently, impede the purification. This is effected at the outset by passing the sewage through a grit chamber, or sedimentation tank, in which the heavier solids are deposited; and the coarser *débris* are also removed by screening. A considerable quantity of mineral detritus is found in the sewage where the combined system is in operation, especially where a proper provision of catch-pits is absent, owing to the washings of the streets coming into the sewers; whereas under an effective separate system, the mineral solids in the sewage are reduced to a minimum.

Irrigation on land, as described in the last chapter, is in reality a bacterial method of purification, whether conducted as broad irrigation, or by intermittent irrigation, which latter approximates very closely to bacterial downward filtration. The bacterial process known as intermittent sand filtration, differs, indeed, from intermittent irrigation, merely in the filter being prepared by the use of fairly coarse sand of somewhat uniform size, instead of relying solely on porous soil in its natural condition; and this process has been extensively adopted in the United States. Contact beds, in which larger materials are employed, are filled with sewage, then left full for a time, and subsequently, after emptying, are allowed to rest empty before being refilled; and this system, repeated generally two or three times to ensure better results, constitutes a very common method of bacterial purification. Continuous sprinkling filters differ chiefly from contact beds in the method in which the sewage is applied to them, though the materials used are generally somewhat coarser. Thus the sewage is spread continuously over these filters in a sort of spray or rain, from fixed or movable pipes, the object being to distribute it uniformly over the surface of the filter in small enough quantities at a time that it may only trickle down through the filter. In the septic tank, the putrefaction of the sewage and the liquefaction of the sludge are accomplished under favourable conditions; but the process is not complete in itself, necessitating other means being resorted to for the essential nitrification of the effluent to complete the purification. The filters alluded to above carry out their purification of sewage by what is sometimes termed downward filtration; but in one system, the first stage of purification, corresponding to that effected by the septic tank, is accomplished by upward filtration, the sewage being introduced at the bottom of the filter instead of on the top; and the necessary nitrification is subsequently carried out by passing the effluent over a series of trays containing coke.

In the above processes of bacterial purification which are in regular use at the present time, only the septic tank, and upward filtration, provide systematically for preliminary anaerobic action, the other processes depending in principle on aerobic action; but even in aerobic processes, some anaerobic action incidentally occurs, and is to some extent secured by the withdrawal of some of the suspended organic matters by sedimentation, and by passing the sewage through a roughing filter, before proceeding with the purification. Moreover, decomposition in a septic tank is generally regarded as a desirable preliminary operation, to prevent the organic solids in the sewage clogging the bacterial filters or contact beds.[1]

Intermittent Sand Filtration.—The advantage of this form of bacterial filter over intermittent irrigation on porous land, is that, provided coarse sand can be procured, the filter can be established at any convenient spot, without having to try to obtain suitable land within easy access, which is liable to be difficult to accomplish on reasonable terms, and also that less area suffices for a carefully prepared filter adapted to the particular conditions. It is essential that this filtration should be intermittent, as otherwise the filter would soon choke up, and a layer of slime form over the surface excluding the air from the layers below, and, consequently, arresting bacterial action; and the surface layer of sand has to be frequently raked, and occasionally removed. By applying the sewage in moderate doses intermittently, the effluent passes off through porous soil below, or into subsoil drains at the bottom of the filter, enabling air to enter the pores of the filter; and thus the filter is fully aerated between each watering, thereby securing the continuance of the purification of the sewage by aerobic bacteria. It is important to avoid having any layer of fine material in the filter, which, by retaining the liquid by capillary attraction, would impede the draining off of the effluent, and, consequently, interfere with the requisite aeration; and as the bacterial changes take place in the upper 2 or 3 feet of the filter, there is no necessity to increase the thickness of the filter beyond this limit, except over the subsoil drains, where a somewhat greater thickness is expedient to ensure the completion of the purification of the effluent before it enters the drains.

In all of these processes, a certain time is required with new installations for what may be regarded as the cultivation of the necessary bacteria, under conditions favourable to their growth, before they are numerous enough to deal efficiently with the sewage supplied them; and the proper establishment of the process of purification requires more time in the winter than in the summer.

Contact Beds.—A contact bed is formed by filling a watertight tank with pieces of coke, burnt clay, broken stone, or clinker, of sizes

[1] "The Purification of Sewage and Water," W. J. Dibdin, 3rd edition, 1903, p. 100.

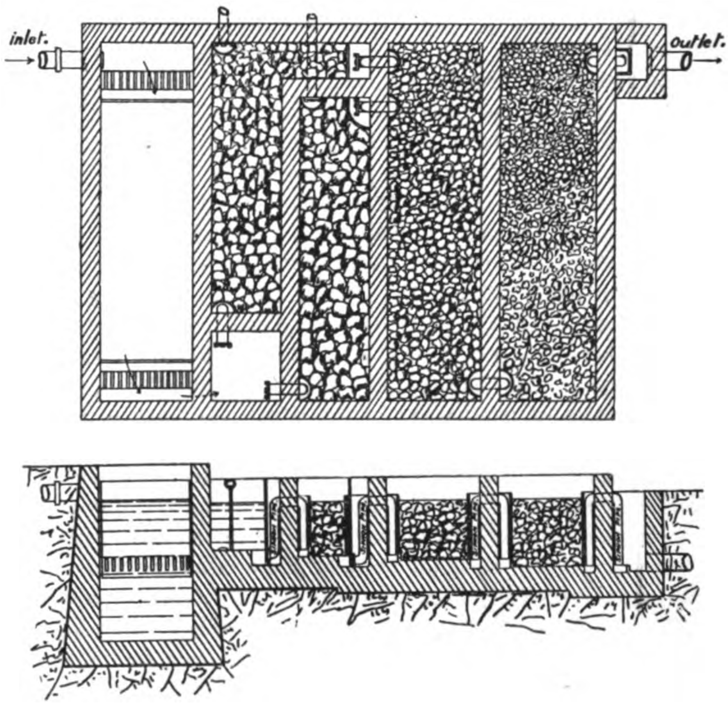
approximately that would pass through a sieve of $1\frac{1}{2}$ inches mesh, and be retained by a $\frac{1}{4}$ inch mesh. This bed, about 3 to 4 feet in depth, is filled with sewage and allowed to stand full for some time, and is then emptied and left to rest for another period, after which the process is repeated. The cycle often adopted is 1 hour filling, 3 hours full, 1 hour emptying, and 3 hours empty, allowing of three series of operations in a day, with a rest empty of one day occasionally; whilst it is proposed that the contact beds dealing with the Manchester sewage, $3\frac{1}{3}$ feet deep, shall be filled four times in the 24 hours, with a rest of 1 day in a week, enabling about half a million gallons of sewage to be treated per acre of contact bed in a day. Coarse-grained and somewhat porous materials, provided they are durable, appear to be the most suitable for forming the beds, affording probably convenient recesses for the accumulation of bacteria; and the efficiency of the process in purifying the sewage, and providing a good effluent, is increased by repeating the operation on a second bed, known as double contact; and occasionally a treble contact is resorted to. These contact beds, like filtration systems, are very liable to be gradually filled with deposit, if raw sewage, or sewage containing much organic matter in suspension, is poured into them; and their life is considerably prolonged if some preliminary treatment for the removal of suspended matter is adopted. It is difficult to prevent some diminution in the voids of these beds by degrees, resulting from deposit; but the deposit can be removed by flushing the bed with water, or by washing and replacing the cleansed material. Both filters and contact beds require a fall; but where two or three beds are used for double or treble contact, the fall required can be materially reduced by connecting the beds by siphons which automatically fill them in succession.[1] When the first bed has been filled, it discharges into the second; and the liquid falls in the first bed till it is at the same level in both beds, which then are filled simultaneously and discharge into the third bed, with a lowering of the liquid in the first two beds till it is level in all three beds; and finally, when the three beds have been filled, their contents are discharged through the siphon in the outer wall of the last bed, leaving the beds empty for a period of aeration, Fig. 282. By placing the inlet and outlet siphons of each bed as far apart as possible, in the manner shown on the plan, ample contact of the liquid with the materials of each bed is secured by the devious course it has to follow through the beds.

The first bacterial contact beds were constructed at Sutton; and in 1896, crude sewage was poured into a tank filled with coarse, burnt ballast, after merely straining the sewage through a screen to arrest

[1] "Sewerage and Sewage Disposal Apparatus," Adams—Hydraulics, York, 1904, p. 63.

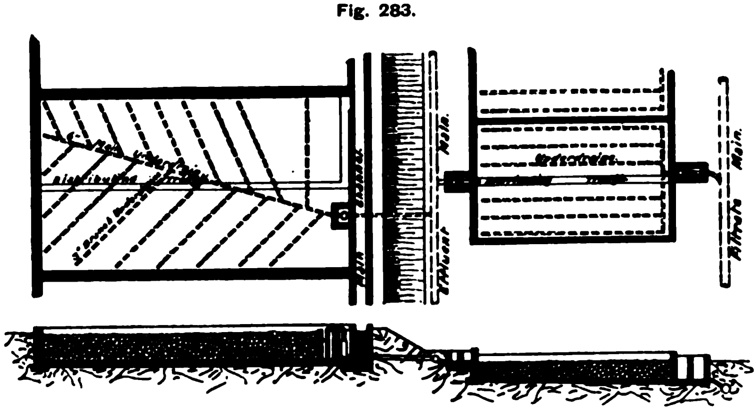
TRIPLE CONTACT BEDS CONNECTED BY SIPHONS.

Fig. 282.



SUTTON BACTERIAL CONTACT BEDS.

Fig. 283.



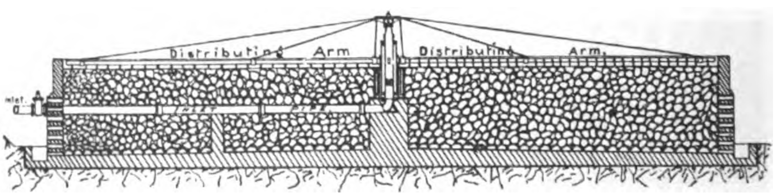
floating *débris* and the grosser particles,[1] the cycle adopted at first in this case being $\frac{3}{4}$ hour filling, 2 hours full, $1\frac{1}{4}$ hours emptying, and 2 hours empty, allowing of four treatments of sewage in 24 hours; but it appears that subsequently three treatments per day were substituted for the coarse beds. The upper layers of the ballast in the coarse beds has to be raked or turned over occasionally. The effluent from the coarse beds was then passed on to contact beds of coke breeze, which completed the purification, Fig. 283; and by these arrangements an amount of sewage equivalent to 300,000 gallons per acre per day has been treated with very satisfactory results.

At Hampton, the same principle has been adopted as at Sutton; but three contact beds arranged in terraces have been employed, coarse clinker larger than $\frac{1}{2}$ inch forming the first bed, fine clinker under $\frac{1}{2}$ inch the second, and sand being used for the third bed. In this case, the cycle is 1 hour filling, 1 hour full, 1 hour emptying, and about 5 hours standing empty; and these contact beds have afforded an excellent effluent, the rate of purification of the sewage per acre of beds being 106,000 gallons a day.

Continuous Trickling Filters.—These filters differ chiefly from contact beds in the sewage being introduced continuously in a small stream, instead of being filled intermittently; and the materials used in the filters are larger than in the beds, having dimensions of about 1 to 3 inches; and, moreover, it is unnecessary that the tank holding the filter should be made watertight. The object aimed at in these filters is to spread the liquid as uniformly as possible over the whole area in the form of rain or spray, so that it may trickle slowly down the surfaces of the materials composing the filter, without ever forming a continuous stream. This distribution is effected as a continuous fine rain or spray from a series of nozzles in fixed pipes, or from automa-

CONTINUOUS FILTER WITH REVOLVING SPRAY DISTRIBUTOR.

Fig. 284.



tically revolving pipes or troughs through a number of holes,[2] Fig. 284, the flow being carefully regulated to prevent the liquid passing

[1] "The Purification of Sewage and Water," W. J. Dibdin, 3rd edition, pp. 79–81.
[2] "Sewerage and Sewage Disposal Apparatus," Adams—Hydraulics, p. 90.

sufficiently quickly through the coarse filter to escape due purification. An arrangement to obviate the possibility of irregularities occurring in the distribution from the occasional clogging of some of the holes by solid particles in suspension in the liquid, consists of a number of narrow troughs placed at right angles to the channel supplying them with sewage, from the underside of which a series of points project vertically; and when the liquid flows over the edges of the troughs, it passes down the under surface of the troughs and falls in a number of small drops on to the filter; whilst the suspended matters settle in the troughs, and have to be swept out from time to time. The thickness of the filtering materials should not be less than 4 feet, to ensure the purification of the effluent; and where the flow into the filter is liable at times to fall very low, it is necessary to interpose a storage, or dosing tank to provide for the continuity of the revolutions of the automatic distributor. The spraying in distribution, and the slow trickling through the filter, secure the thorough aeration of the effluent; and, consequently, the rate of purification is greater than with contact beds. Whilst, however, contact beds can be worked in an emergency more rapidly than their proper rate for a little time without injurious effects, directly a continuous filter is over-worked it discharges an impure effluent. In cold countries, moreover, spraying filters are more liable to be impeded by frost than contact beds.

A continuous filter is not well adapted for dealing with raw sewage, as it is not capable of duly purifying the bulk of the organic solid matter in suspension, which it would discharge to a large extent in its effluent; and, therefore, a preliminary stage for the purification of the solids, such as is provided by a septic tank, or their removal by a series of screens, appears to be essential before using a continuous filter. After, however, the solids have been liquefied or removed, the continuous filter affords a very efficient means for the aerobic purification of a clarified effluent.

Septic Tank.—Anaerobic changes take place in the dark out of contact with the air, and effect the liquefaction of the solids of the sewage, so that the resulting effluent can be purified by aerobic action in contact beds or by filtration, without the prospect of the beds and filters being clogged up by deposit. These valuable preliminary changes are very satisfactorily accomplished by discharging the raw sewage into a septic tank, after the removal of the heavier mineral solids by deposit in a grit chamber. The septic tank, as it is called from the putrefaction of the sewage which takes place in it, is generally a brick tank sunk in the ground, which should not be less than 6 or 7 feet in depth, and large enough to contain from three-quarters to one and a quarter of the daily volume of the dry-weather flow of sewage. The flow into the tank is continuous, and is introduced some 4 or 5 feet below the surface of the liquid, so as not to disturb the bacterial action or the scum which forms at the surface; and unlike filters and contact beds, no fall is required, as the effluent flows out practically at the same level as the sewage flows in. The establishment of anaerobic action in a septic tank requires a considerable time, especially in the winter, though it can be hastened by inoculation with some of the scum of putrefying sewage from a well-established tank. A kind of fermentation takes place of the solids in the sewage in the tank with evolution of gases; a scum forms on the top from 2 inches to about 1 foot in thickness, and the solids are in a large measure liquefied; but the effluent is putrescible, and has to be passed on to filters or contact beds for final purification. Owing to the covering of scum, the bacterial changes take place equally well in an open tank as in a covered-over tank, so that discarded precipitation tanks have been often utilized, though they are liable to be too shallow; but covered tanks are preferable for preventing nuisance, and because the scum is liable to be disturbed by wind in open tanks. The time occupied by the sewage in passing through the tank has to be limited; for though some of the more intractable solids may need a considerable time for their liquefaction, the sewage is liable to undergo changes unfavourable to the subsequent purification by oxidation if retained in the tank for this purpose; and a period of 12 to 24 hours in the septic tank affords satisfactory results.

Where sedimentation alone is adopted as a preliminary to purification by contact beds or filtration, this process takes to some extent the place of the septic tank; for though the primary object is the deposit of the heavier mineral solids in the sewage, a certain quantity of the organic solids is carried down with the mineral sediment, and undergoes liquefaction by anaerobic action out of contact with the air at the bottom of the settling tank.

Upward Filtration.—Many years ago it was observed that the slow filtration of sewage through sand in an upward direction produced remarkable changes in it, where no opportunity was afforded of the occurrence of oxidation in the absence of air in the lower layers. This is now known to be due to the action of anaerobic bacteria in the lower layers of the bed of sand, or other filtering medium, through which the sewage is passed, which in the dark, and out of contact with the air, produces the liquefaction of the organic solids. In fact, upward filtration appears to furnish the most effective means of promoting the proper cycle of bacterial changes in the purification of sewage; for not only are the changes continuous, but wholly anaerobic action is assured in the bottom layers of the filter; then as the liquid ascends, some aerobic action commences, owing to partial access to the air being attained, during the continuance of anaerobic action; and, finally, in the upper layers of the filter where there is ample aeration, anaerobic action ceases entirely, and aerobic action is fully established.

Successive Changes effected in the Bacterial Purification of Sewage.—Where the sewage is subjected to anaerobic decomposition out of contact with air, before it is placed under conditions of ample aeration favourable to the action of aerobic bacteria, as in the septic tank and by upward filtration, a fairly definite series of changes takes place in three successive stages. In the first stage, in which air is excluded, anaerobic bacteria attack the albuminous matters, cellulose, and fats in the sewage, producing soluble nitrogeneous compounds, phenol derivatives, gases, and ammonia, and resulting in the liquefaction of the organic solids of the sewage.[1] In the second stage, where there is a partial admission of air, aerobic bacteria begin to appear, and, in conjunction with a continuation of anaerobic action to a diminishing extent, break up amido compounds, fatty acids, dissolved residues, and phenolic bodies into ammonia, nitrites, and gases. The third stage, in which complete aeration is provided and the essential nitrification takes place, consists in the conversion of ammonia and carbonaceous residues into carbonic acid gas, water, and nitrates by aerobic bacteria alone. This sequence of changes is fairly regularly carried out where sewage, after being decomposed by anaerobic bacteria in a septic tank, or to a less extent in a sedimentation tank, is partially aerated on being first discharged on to a contact bed or filter, and by degrees undergoes complete aeration; whilst the cycle of changes is more strictly and completely followed by upward filtration, and thorough subsequent aeration.

Bacteriologists and chemists, however, are not all in agreement as to the absolute necessity of the action of anaerobic bacteria as a preliminary stage in the bacterial purification of sewage, or as to the extent to which the liquefaction of the sludge is accomplished.[2] It appears, indeed, that in irrigation on land, and by the exclusive treatment of sewage in two or three contact beds, the initial anaerobic action is not definitely provided for; but though the sequence indicated above is not followed in such cases, opportunities for the occurrence of anaerobic decomposition present themselves by the digging of the surface sludge into the soil, and by the periods of rest when the contact beds have

[1] "Sewage and the Bacterial Purification of Sewage," S. Rideal, pp. 75 and 207.

[2] "The Purification of Sewage and Water," W. J. Dibdin, 3rd edition, pp. xiv.–xvi., 20-23, 101, and 107; "Report to the London County Council by the Chemist," April 17, 1902; and "Sewage and the Bacterial Purification of Sewage," S. Rideal p. 209.

been filled. Evidence varies as to the amount of sludge left behind after anaerobic decomposition; but it is certain that a very considerable proportion of the sludge undergoes liquefaction in septic or sedimentation tanks, and especially in the process of upward filtration, though possibly rarely the whole of it as formerly was definitely affirmed. Undoubtedly, if aerobic bacteria can by themselves efficiently purify sewage, a source of possible nuisance from the putrefaction of sewage by anaerobic bacteria would be avoided. The acknowledgment, however, that filters and contact beds are preserved from clogging up by preliminary septic treatment, and the view very generally held that anaerobic action is a valuable first step in the purification of sewage, indicates that this initial stage, though not indispensable, is in the great majority of cases advisable for the efficient and rapid carrying out of the necessary bacterial changes.

Some bacterial processes are continuous, as for instance the septic tank with subsequent trickling filtration, and upward filtration followed by efficient nitrification; whilst others are intermittent in their action, of which intermittent filtration through sand, irrigation on land, and contact beds furnish examples. It is difficult to obtain adequately reliable comparisons of the working of the different processes, owing to differences in the sewage of various places, and in the care with which the systems are carried out. It might, however, be reasonably supposed that a continuous system, in which the bacteria remain undisturbed under favourable and constant conditions for the prosecution of the requisite changes, would prove more efficient than an intermittent method, where the conditions are frequently completely altered, necessitating the alternate action of anaerobic and aerobic bacteria, or periodically placing the latter in unfavourable environments; and it appears that the continuous systems referred to above produce more thorough nitrification than the intermittent processes.

Bacterial Treatment of Trades Waste.—In manufacturing towns, complications in the purification of sewage are liable to be introduced by the discharge of trades waste into the sewers; and owing to the strongly acid or alkaline character of such refuse in many cases, it was for a long time considered that sewage containing these products would not be amenable to bacterial treatment. It has, however, been found that where several varieties of trades waste are produced in a town, these acid and alkaline products neutralize one another to a considerable extent; whilst experiments have proved that some samples of dilute trades waste can be purified by bacterial action in a trickling filter, but that generally the refuse from factories and breweries, especially those of the worst description, require to undergo anaerobic decomposition in a septic tank, inoculated with sewage sludge, before being passed on to a sprinkling filter.[1] This latter treatment, by prolonged decomposition in the septic tank and ample filtration, has enabled the worst and most intractable refuse to be satisfactorily purified.

Regulations of Local Government Board.—Certain requirements have been drawn up by the Local Government Board, compliance with which is necessary to obtain their consent to any sewage disposal scheme by means of bacterial treatment,[2] of which the following are some of the most important provisions. Sedimentation and septic tanks must be capable of receiving one day's dry-weather flow, and should not be less than 6 feet in depth. Contact beds, and continuous, or trickling filters, fed by spray, are considered equally efficacious in dealing with a given volume of sewage, and have the same volume, area, and depth assigned to them; but whereas contact beds may be worked with a minimum fall of 4 feet, and a minimum depth of bed of 2½ feet, 6 feet are regarded as the smallest fall allowable for a continuous filter, corresponding to a fall of 5½ feet in the case of a contact bed, and 4 feet as the minimum depth of filter. On the other hand, whilst a fall of 7½ feet, with a depth of bed of 6 feet, is assumed to be the maximum for contact beds, a fall of 9 feet, with a depth of filtering materials of 7 feet, is considered suitable for continuous filters. The maximum volume of sewage, after sedimentation, that may be passed through a contact bed or continuous filter in 24 hours, per cubic yard of contents of bed or of filter, is one cubic yard, equivalent to 168·7 gallons, allowing three fillings a day for contact beds worked automatically, each cycle of eight hours consisting of 1 hour filling, 2 hours full, 1 hour emptying, and 4 hours resting empty. Where, however, the contact beds are not worked automatically, only two cycles of 8 hours each are permitted in the 24 hours; and provision has to be made for the flow during the night. Though, according to the above allowance, 1,000 cubic feet of sewage, after sedimentation, can be dealt with by 1,000 cubic feet of filters or contact beds, including the filtrant, where three fillings of the contact beds per day are permissible, only three-fourths of the amount, or 750 cubic feet of unsettled sewage, can be dealt with by the same capacity of filter or contact bed. With a total fall of 5½ feet for contact beds, and 6 feet for continuous filters, and a depth of bed or filtrant of 4 feet, the maximum volume of sewage that can be treated in a day, per square yard of bed or filter, is 225 gallons; and the area of bed or filter required for dealing with the sewage, on the separate system, of 1000 persons, with a water-supply of 35 gallons a head,

[1] "The Bacterial Treatment of Trades Waste," W. Naylor, *Proc. Inst. C. E.*, vol. cxlviii. p. 170.

[2] "Sewage Disposal Schemes: Local Government Board Requirements," S. H. Adams, October, 1903, London.

calculated at twice the dry-weather flow, reaching 70,000 gallons pei day, would amount to 311 square yards.

The dry-weather flow of sewage in any district is regarded as equivalent to the water-supply; and twice the dry-weather flow in the case of the separate system of sewerage, has to be treated as sewage, and three times the dry-weather flow with the combined system. The remainder of the flow up to six times the dry-weather flow, or four times the dry-weather flow in the separate system, and three times the dry-weather flow in the combined system, has to be passed through storm-water filters, or on to land. When the flow in the sewers exceeds six times the dry-weather flow, the excess may be discharged by storm overflows direct into the nearest watercourse or river. Storm-water filters filled with clinker 3 feet in depth, can deal with 500 gallons of diluted sewage per square yard of filter in 24 hours.

Filtration through land as a final process of purification was formerly insisted upon; but in view of the fact that, owing to the efficiency of bacterial methods of purification, the effluent has been found occasionally less pure after passing through land than before, and the frequent difficulty and cost of obtaining suitable land, this regulation is sometimes dispensed with, additional filtration through sand taking its place.

Examples of Complete Bacterial Purification of Sewage.—At Exeter, the septic tank was introduced, for the first time on a large scale, in 1895, for the preliminary anaerobic treatment of a portion of the sewage, which was followed by aerobic purification on filters worked intermittently, acting as contact beds, Fig. 285. The raw sewage was admitted direct into the septic tank, 5 feet below the normal water-level, so as not to disturb the bacterial changes, and to avoid the admission of air.[1] The sewage first passed through a grit chamber, 7 feet across and 10 feet deep, at the inlet extremity of the tank, for the deposit of the heavier sediment, from which it flowed over a partition, with its top one foot below the water-level, into the main portion of the tank, which in this instance was made 57 feet long, 18 feet wide, and $7\frac{1}{2}$ feet deep, so as to be capable of containing an average day's flow.[2] The sewage, in this manner, flowed slowly through the whole length of the tank, of about 64 feet, in about 24 hours, giving time for the anaerobic changes to take place quietly before it passed out at the far end of the tank, through a long slit about a foot below the surface of the liquid; and after flowing over an aerating weir, it was led along channels on to filters of coke breeze or clinker, Fig. 285, where aerobic bacteria completed the decomposition of the sewage under favourable conditions,

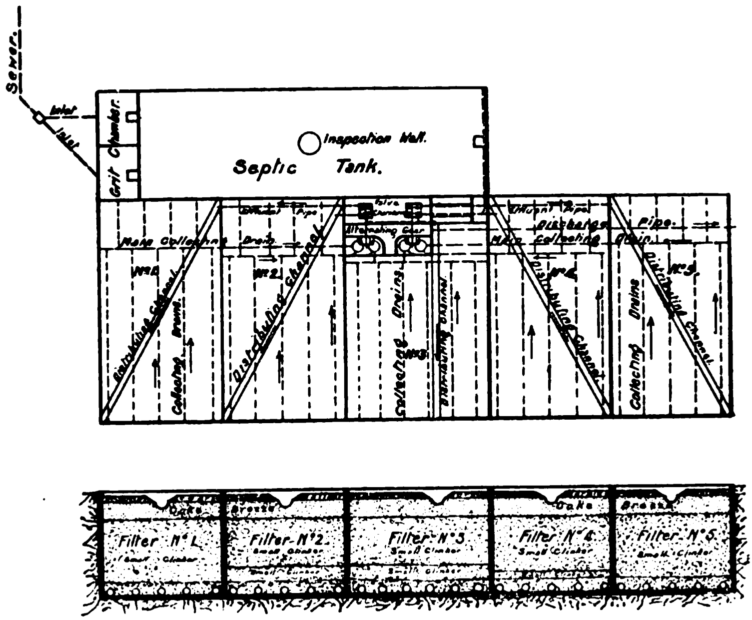
[1] "Sewage and the Bacterial Purification of Sewage," S. Rideal, pp. 213-219.

[2] Designed by Mr. Cameron, City Surveyor of Exeter.

before the effluent was discharged on to land for its final purification, the filling, resting full, emptying, and aeration of the filters being accomplished automatically. This experimental process proved so efficient a method of purification, that it has been extended to the sewage of the whole of Exeter. For this purpose, six septic tanks have been provided, 181 feet long, 35 feet wide, and 7 feet deep; and there are eight filters covering an area of $2\frac{1}{2}$ acres, each filter being composed of an upper layer of crushed clinker, $3\frac{1}{2}$ feet thick, resting upon a 6-inch layer

SEPTIC TANK AND CONTACT BEDS, EXETER.

Fig. 285.



of coarse gravel, the working of the filters being regulated automatically, and the effluent being finally passed on to land. This is the general system which has been very extensively adopted during recent years for the bacterial purification of sewage, and with such satisfactory results in several instances, that the treatment of the effluent on land actually produces deterioration of a good effluent, instead of its further purification,[1] so that the obligation of final treatment on land is being gradually relaxed, especially in cases where the available land is unsuitable or costly, and where an additional filter or contact bed is provided as a substitute.

[1] "Sewage and the Bacterial Purification of Sewage," S. Rideal, p. 119.

Continuous sprinklers, also, are being largely introduced as a more efficient means of distributing and purifying the sewage in contact beds and filters, than the intermittent method of application, even when worked automatically. The only defects of this method of sewage purification are, that the transition from anaerobic action in the septic tank to the aerobic action in the contact beds or filters, is somewhat abrupt, so that the desirable intermediate stage of anaerobic decreasing action and aerobic increasing action, is not fully developed; whilst the intermittent working of the contact beds disturbs the action of the aerobic bacteria. Nevertheless, the septic tank process with subsequent treatment in contact beds or filters, combined with automatic intermittent action or continuous sprinklers, have rendered immense services for the satisfactory purification of sewage, where land for irrigation is unsuitable or very difficult to obtain.

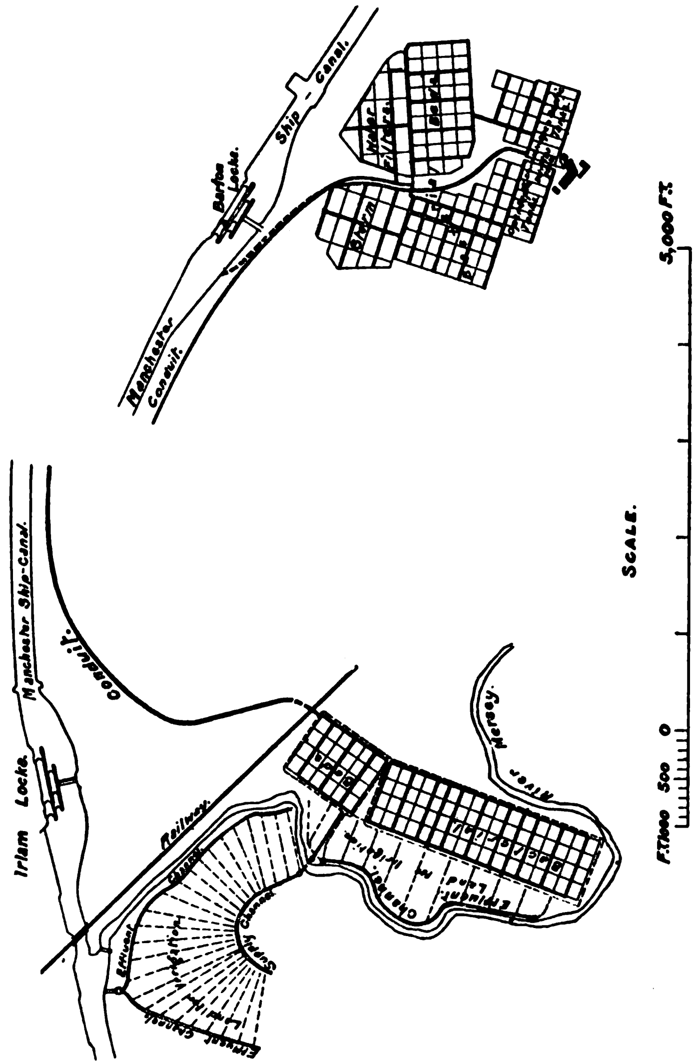
The largest example of sewage purification by means of septic tanks and contact beds, and subsequent irrigation, is being carried out for disposing of the sewage of Manchester, which for a long time very seriously polluted the rivers Mersey and Irwell; and when the waters of these rivers were intercepted for supplying the Manchester Ship-Canal, their pollution became an intolerable nuisance and a danger to health. This state of things was only partially remedied by the chemical precipitation of the solids, and their conveyance out to sea; for, as previously pointed out, the effluent is not purified in the process, and still seriously contaminated the comparatively stagnant waters of the canal by being discharged into it. After obtaining the advice of an engineer, a chemist, and a bacteriologist, who carried out a series of experiments on the application of bacteriological processes to the purification of Manchester sewage, it was determined to treat the sewage in septic tanks, after the removal of the grosser solids by screening and settlement, and then to distribute the sewage from the septic tanks over two sets of contact beds in succession, known as double contact, followed by irrigation on land before the purified effluent is discharged into the ship-canal. The works for effecting this purification are shown in Fig. 286, and are situated at Davyhulme near the Barton Locks on the Manchester Ship-Canal, where the chemical precipitation was formerly conducted, and at Flixton alongside the Mersey near its outlet into the canal a little below the Irlam Locks, the two sets of works being connected by a conduit, running for the most part alongside the canal, for conveying the sewage from the primary bacterial beds at Davyhulme to the secondary beds at Flixton.[1] The plant at Davyhulme consists of twelve open septic tanks, seven of which formerly served as precipitation

[1] "The Treatment of Manchester Sewage, City of Manchester, Rivers Department," G. J. Fowler and J. P. Wilkinson, Manchester, 1902, pp. 18–24, and diagram 8.

tanks, with four storm-water detritus tanks, as a considerable quantity of road grit finds its way into the Manchester sewers in the absence of catch-pits to the street gullies; and there are also 46 acres of bacterial

BACTERIAL DISPOSAL OF MANCHESTER SEWAGE.

Fig. 286.

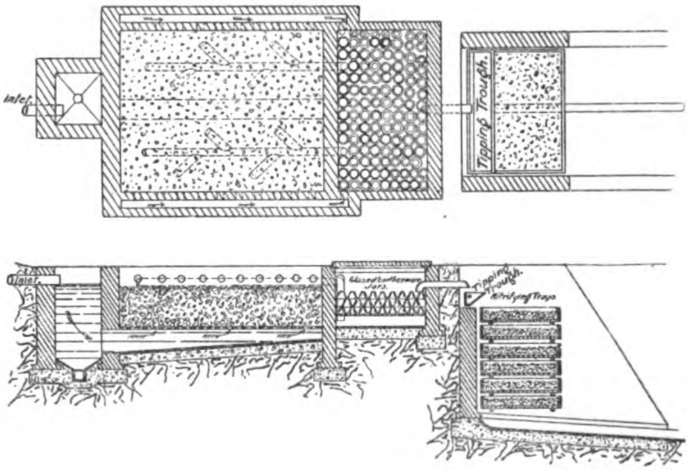


contact beds, comprising 92 beds of half an acre each, and 26 acres of storm-water filter-beds, divided into 32 beds. After treatment in the septic tanks and the adjoining contact beds, the sewage is conveyed through the conduit to Flixton. Here another 46 acres of bacterial beds, in 92 beds of half an acre, have been provided for a second contact; and two areas of land, one at Flixton between the bacterial second contact beds and the Mersey, and the other nearer the canal at Carrington, covering altogether about 100 acres, serve for the final purification of the effluent before it is discharged into the canal. These works have been designed for dealing with a maximum flow of 126 million gallons in 24 hours, the flow up to 63 million gallons being treated by double contact on the bacterial beds, and the remaining flow, up to 63 million gallons, being turned on to the storm-water filter-beds; whilst any excess during heavy rain beyond the 126 million gallons of flow in 24 hours, may be passed direct into the ship-canal at the Mode Wheel storm-water overflow. Each of the septic tanks is 300 feet long, 100 feet wide, and about 6 feet deep, and contains 1,125,000 gallons; and the total liquid capacity of the septic tanks is 15,820,500 gallons, which, with the addition of the four storm-water tanks, capable of receiving the night flow of sewage, reaches 20,320,500 gallons. The results of the experiments with Manchester sewage showed, that the effluents from open and closed septic tanks are practically identical in composition; that with a tank capacity of half the daily flow of sewage, it is possible to liquefy about 25 per cent. of the total matters in suspension in the sewage; and that the changes effected in a septic tank minimize the disturbing influences of excessive quantities of manufacturing refuse, and produce a fairly uniform effluent. The contact beds consist of a layer, about $3\frac{1}{3}$ feet deep, of coarse furnace clinkers; and the liquid conveyed in a supply channel from the septic tank, is distributed by radial channels over a pair of beds from a small reservoir on each side of the centre of the dividing partition, and after passing through the bed, converges again by similar radial drains at the bottom to an under effluent channel. The storm-water filter-beds, each about 1 acre in area, are filled with $2\frac{1}{2}$ feet of unscreened clinkers, the coarser materials being used at the bottom and for covering the under drains; and they are designed to deal with a flow not exceeding 500 gallons a day per square yard of surface.

The treatment of raw sewage by upward filtration, according to the system indicated in Fig. 287, serves the same purpose as a septic tank in producing the requisite preliminary anaerobic decomposition; but by providing surfaces in the materials composing the filter for the cultivation of the organisms, it is more effective in causing the separation and solution of the suspended matters in the sewage, and in purifying the effluent. Moreover, this system conforms most closely to the due cycle of bacterial changes; for in the lower portion of these cultivation filters, the anaerobic bacteria are placed under the conditions most favourable for their growth and activity; then as the sewage rises to the upper part, some aeration commences favouring the appearance of aerobic

UPWARD FILTRATION AND NITRIFICATION, SCOTT-MONCRIEFF SYSTEM.

Fig. 287.



bacteria, the two working simultaneously in the second stage; and, lastly, in this arrangement, the effluent from the filter is spread by tipping troughs over the top row of nitrifying trays, consisting of a series of rows of perforated trays containing coke, one below the other, through which the liquid falls successively on to the row below, and by this process becomes thoroughly nitrified under the action of aerobic bacteria placed in conditions most favourable to their growth.[1] Though this system appears to follow the cycle of conditions most suited for successive bacterial changes, and has been found to produce the most thorough nitrification of the effluent, it has not hitherto been nearly so extensively developed as the septic tank and contact beds, and has been mostly confined to the disposal of the sewage of comparatively small towns and large establishments, possibly owing to the greater simplicity of the septic tank, and also in consequence of the facility with which precipitation tanks are converted into septic tanks.

Concluding Remarks on Bacterial Purification of Sewage.—The efficiency of the action of bacteria in accomplishing the decomposition and purification of sewage, and which for a long time has

[1] "The Scott-Moncrieff Improvements in the Bacterial Purification of Sewage," North British Plumbing Company, London.

played a most important, though formerly unnoticed part in the successful disposal of sewage by irrigation on land, has now been fully established; and the application of bacterial processes to the purification of the sewage of the third largest city in the United Kingdom, demonstrates its universal value as a means of sewage disposal. Instead of largely increasing the volume of sludge by the addition of chemicals for the precipitation of the solids, which it has often been found difficult to dispose of, even after a considerable expenditure on its solidification, the anaerobic changes in the septic tank, or in the first stages of upward filtration, produce the liquefaction of a very large proportion of the sludge, together with the emission of inflammable gases which have been sometimes utilized for lighting purposes. Moreover, the nitrification which takes place in the subsequent aerobic changes, renders the effluent very valuable for manuring the land, quite irrespectively of its own further purification, which is sometimes quite unnecessary, and for restoring to the soil the nitrogen of which it has been deprived by the raising of crops, and which is lost when the sewage is discharged into a river or the sea.

Bacterial processes need not entirely supersede the irrigation of land; but the septic tank, and upward filtration, enable the bacterial changes to be effected in a proper sequence, instead of in a sort of inverted order by the irrigation of land with raw sewage, and in a much smaller space; whilst the effluent produced by these processes can be most advantageously distributed over land in the vicinity. Where irrigation on land has previously been definitely adopted, a bacterial process may be resorted to for dealing with a portion of the sewage, so as to avoid the bane of irrigation schemes, namely, waterlogging the land in wet weather with an excess of liquid, and also in place of purchasing additional land to cope with an increase of sewage, with the growth of the population of a district, beyond what was originally provided for in determining the area of the sewage farm. The bacterial purification of sewage, however, is of the greatest value where no suitable land for irrigation is procurable within a reasonable distance, or where its cost is practically prohibitive, and where the conditions render a pure effluent of special importance. When a duly nitrified effluent is produced by a properly conducted bacterial process, the provision of suitable land for irrigation is of no importance; for such an effluent may be regarded as a very valuable product for agriculture, which may be distributed with great profit over any kind of land. Bacterial purification may, accordingly, be regarded as having most effectively solved by natural means, judiciously directed and controlled, the complicated, and apparently almost insoluble problem of the innocuous and useful disposal of sewage at a reasonable cost.

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