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THE SEWERAGE OF SEA COAST TOWNS

BY

HENRY C. ADAMS

CONTENTS

PREFACE.

These notes are internal primarily for those engineers who, having a general knowledge of sewerage, are called upon to prepare a scheme for a sea coast town, or are desirous of being able to meet such a call when made. Although many details of the subject have been dealt with separately in other volumes, the writer has a very vivid recollection of the difficulties he experienced in collecting the knowledge he required when he was first called on to prepare such a scheme, particularly with regard to taking and recording current and tidal observations, and it is in the hope that it might be helpful to others in a similar difficulty to have all the information then obtained, and that subsequently gained on other schemes, brought together within a small compass that this book has written.

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CHAPTER I.

THE FORMATION OF TIDES AND CURRENTS.

It has often been stated that no two well-designed sewerage schemes are alike, and although this truism is usually applied to inland towns, it applies with far greater force to schemes for coastal towns and towns situated on the banks of our large rivers where the sewage is discharged into tidal waters. The essence of good designing is that every detail shall be carefully thought out with a view to meeting the special conditions of the case to the best advantage, and at the least possible expense, so that the maximum efficiency is combined with the minimum cost. It will therefore be desirable to consider the main conditions governing the design of schemes for sea-coast towns before describing a few typical cases of sea outfalls. Starting with the postulate that it is essential for the sewage to be effectually and permanently disposed of when it is discharged into tidal waters, we find that this result is largely dependent on the nature of the currents, which in their turn depend upon the rise and fall of the tide, caused chiefly by the attraction of the moon, but also to a less extent by the attraction of the sun. The subject of sewage disposal in tidal waters, therefore, divides itself naturally into two parts: first, the consideration of the tides and currents; and, secondly, the design of the works.

The tidal attraction is primarily due to the natural effect of gravity, whereby the attraction between two bodies is in direct proportion to the product of their respective masses and in inverse proportion to the square of their distance apart; but as the tide-producing effect of the sun and moon is a differential attraction, and not a direct one, their relative effect is inversely as the cube of their distances. The mass of the sun is about 324,000 times as great as that of the earth, and it is about 93 millions of miles away, while the mass of the moon is about 1-80th of that of the earth, but it averages only 240,000 miles away, varying between 220,000 miles when it is said to be in perigee, and 260,000 when in apogee. The resultant effect of each of these bodies is a strong "pull" of the earth towards them, that of the moon being in excess of that of the sun as 1 is to 0.445, because, although its mass is much less than that of the sun, it is considerably nearer to the earth.

About one-third of the surface of the globe is occupied by land, and the remaining two-thirds by water. The latter, being a mobile substance, is affected by this pull, which results in a banking up of the water in the form of the crest of a tidal wave. It has been asserted in recent years that this tidal action also takes place in a similar manner in the crust of the earth, though in a lesser degree, resulting in a heaving up and down amounting to one foot; but we are only concerned with the action of the sea at present. Now, although this pull is felt in all seas, it is only in the Southern Ocean that a sufficient expanse of water exists for the tidal action to be fully developed. This ocean has an average width of 1,500 miles, and completely encircles the earth on a circumferential line 13,500 miles long; in it the attraction of the sun and moon raises the water nearest to the centre of attraction into a crest which forms high water at that place. At the same time, the water is acted on by the centripetal effect of gravity, which, tending to draw it as near as possible to the centre of the earth, acts in opposition to the attraction of the sun and moon, so that at the sides of the earth 90 degrees away, where the attraction of the sun and moon is less, the centripetal force has more effect, and the water is drawn so as to form the trough of the wave, or low water, at those points. There is also the centrifugal force contained in the revolving globe, which has an equatorial diameter of about 8,000 miles and a circumference of 25,132 miles. As it takes 23 hr. 56 min 4 sec, or, say, twenty-four hours, to make a complete revolution, the surface at the equator travels at a speed of approximately 25,132/24 = 1,047 miles per hour. This centrifugal force is always constant, and tends to throw the water off from the surface of the globe in opposition to the centripetal force, which tends to retain the water in an even layer around the earth. It is asserted, however, as an explanation of the phenomenon which occurs, that the centripetal force acting at any point on the surface of the earth varies inversely as the square of the distance from that point to the moon, so that the centripetal force acting on the water at the side of the earth furthest removed from the moon is less effective than that on the side nearest to the moon, to the extent due to the length of the diameter of the earth. The result of this is that the centrifugal force overbalances the centripetal force, and the water tends to fly off, forming an anti-lunar wave crest at that point approximately equal, and opposite, to the wave crest at the point nearest to the moon. As the earth revolves, the crest of high water of the lunar tide remains opposite the centre of attraction of the sun and moon, so that a point on the surface will be carried from high water towards and past the trough of the wave, or low water, then past the crest of the anti-lunar tide, or high water again, and back to its original position under the moon. But while the earth is revolving the moon has traveled 13 degrees along the elliptical orbit in which she revolves around the earth, from west to east, once in 27 days 7 hr. 43 min, so that the earth has to make a fraction over a complete revolution before the same point is brought under the centre of attraction again This occupies on an average 52 min, so that, although we are taught that the tide regularly ebbs and flows twice in twenty-four hours, it will be seen that the tidal day averages 24 hr. 52 min, the high water of each tide in the Southern Ocean being at 12 hr. 26 min intervals. As a matter of fact, the tidal day varies from 24 hr. 35 min at new and full moon to 25 hr. 25 min at the quarters. Although the moon revolves around the earth in approximately 27-1/3 days, the earth has moved 27 degrees on its elliptical orbit around the sun, which it completes once in 365± days, so that the period which elapses before the moon again occupies the same relative position to the sun is 29 days 12 hr. 43 min, which is the time occupied by the moon in completing her phases, and is known as a lunar month or a lunation.

Considered from the point of view of a person on the earth, this primary tidal wave constantly travels round the Southern Ocean at a speed of 13,500 miles in 24 hr. 52 min, thus having a velocity of 543 miles per hour, and measuring a length of 13,500/2 = 6,750 miles from crest to crest. If a map of the world be examined it will be noticed that there are three large oceans branching off the Southern Ocean, namely, the Atlantic, Pacific, and Indian Oceans; and although there is the same tendency for the formation of tides in these oceans, they are too restricted for any very material tidal action to take place. As the crest of the primary tidal wave in its journey round the world passes these oceans, the surface of the water is raised in them, which results in secondary or derivative tidal waves being sent through each ocean to the furthermost parts of the globe; and as the trough of the primary wave passes the same points the surface of the water is lowered, and a reverse action takes place, so that the derivative waves oscillate backwards and forwards in the branch oceans, the complete cycle occupying on the average 12 hr. 26 min Every variation of the tides in the Southern Ocean is accurately reproduced in every sea connected with it.

Wave motion consists only in a vertical movement of the particles of water by which a crest and trough is formed alternately, the crest being as much above the normal horizontal line as the trough is below it; and in the tidal waves this motion extends through the whole depth of the water from the surface to the bottom, but there is no horizontal movement except of form. The late Mr. J. Scott Russell described it as the transference of motion without the transference of matter; of form without the substance; of force without the agent.

The action produced by the sun and moon jointly is practically the resultant of the effects which each would produce separately, and as the net tide-producing effect of the moon is to raise a crest of water 1.4 ft above the trough, and that of the sun is 0.6 ft (being in the proportion of I to 0.445), when the two forces are acting in conjunction a wave 1.4 + 0.6 = 2 ft high is produced in the Southern Ocean, and when acting in opposition a wave 1.4 - 0.6 = 0.8 ft high is formed. As the derivative wave, consisting of the large mass of water set in motion by the comparatively small rise and fall of the primary wave, is propagated through the branch oceans, it is affected by many circumstances, such as the continual variation in width between the opposite shores, the alterations in the depth of the channels, and the irregularity of the coast line. When obstruction occurs, as, for example, in the Bristol Channel, where there is a gradually rising bed with a converging channel, the velocity, and/or the amount of rise and fall of the derivative wave is increased to an enormous extent; in other places where the oceans widen out, the rise and/or velocity is diminished, and similarly where a narrow channel occurs between two pieces of land an increase in the velocity of the wave will take place, forming a race in that locality.

Although the laws governing the production of tides are well understood, the irregularities in the depths of the oceans and the outlines of the coast, the geographical distribution of the water over the face of the globe and the position and declivity of the shores greatly modify the movements of the tides and give rise to so many complications that no general formulae can be used to give the time or height of the tides at any place by calculation alone. The average rate of travel and the course of the flood tide of the derivative waves around the shores of Great Britain are as follows:—150 miles per hour from Land's End to Lundy Island; 90 miles per hour from Lundy to St. David's Head; 22 miles per hour from St. David's Head to Holy head; 45-1/2 miles per hour from Holyhead to Solway Firth; 194 miles per hour from the North of Ireland to the North of Scotland; 52 miles per hour from the North of Scotland to the Wash; 20 miles per hour from the Wash to Yarmouth; 10 miles per hour from Yarmouth to Harwich. Along the south coast from Land's End to Beachy Head the average velocity is 40 miles per hour, the rate reducing as the wave approaches Dover, in the vicinity of which the tidal waves from the two different directions meet, one arriving approximately twelve hours later than the other, thus forming tides which are a result of the amalgamation of the two waves. On the ebb tide the direction of the waves is reversed.

The mobility of the water around the earth causes it to be very sensitive to the varying attraction of the sun and moon, due to the alterations from time to time in the relative positions of the three bodies. Fig. [Footnote: Plate I] shows diagrammatically the condition of the water in the Southern Ocean when the sun and moon are in the positions occupied at the time of new moon. The tide at A is due to the sum of the attractions of the sun and moon less the effect due to the excess of the centripetal force over centrifugal force. The tide at C is due to the excess of the centrifugal force over the centripetal force. These tides are known as "spring" tides. Fig. 2 [Footnote: Plate I] shows the positions occupied at the time of full moon. The tide at A is due to the attraction of the sun plus the effect due to the excess of the centrifugal force over the centripetal force. The tide at C is due to the attraction of the moon less the effect due to the excess of the centripetal force over centrifugal force. These tides are also known as "spring" tides. Fig. 3 [Footnote: Plate I] shows the positions occupied when the moon is in the first quarter; the position at the third quarter being similar, except that the moon would then be on the side of the earth nearest to B, The tide at A is compounded of high water of the solar tide superimposed upon low water of the lunar tide, so that the sea is at a higher level than in the case of the low water of spring tides. The tide at D is due to the attraction of the moon less the excess of centripetal force over centrifugal force, and the tide at B is due to the excess of centrifugal force over centripetal force. These are known as "neap" tides, and, as the sun is acting in opposition to the moon, the height of high water is considerably less than at the time of spring tides. The tides are continually varying between these extremes according to the alterations in the attracting forces, but the joint high tide lies nearer to the crest of the lunar than of the solar tide. It is obvious that, if the attracting force of the sun and moon were equal, the height of spring tides would be double that due to each body separately, and that there would be no variation in the height of the sea at the time of neap tides.

It will now be of interest to consider the minor movements of the sun and moon, as they also affect the tides by reason of the alterations they cause in the attractive force. During the revolution of the earth round the sun the successive positions of the point on the earth which is nearest to the sun will form a diagonal line across the equator. At the vernal equinox (March 20) the equator is vertically under the sun, which then declines to the south until the summer solstice (June 21), when it reaches its maximum south declination. It then moves northwards, passing vertically over the equator again at the autumnal equinox (September 21), and reaches its maximum northern declination on the winter solstice (December 21). The declination varies from about 24 degrees above to 24 degrees below the equator. The sun is nearest to the Southern Ocean, where the tides are generated, when it is in its southern declination, and furthest away when in the north, but the sun is actually nearest to the earth on December 31 (perihelion) and furthest away on July I (aphelion), the difference between the maximum and minimum distance being one-thirtieth of the whole.

The moon travels in a similar diagonal direction around the earth, varying between 18-1/2 degrees and 28-1/2 degreed above and below the equator. The change from north to south declination takes place every fourteen days, but these changes do not necessarily take place at the change in the phases of the moon. When the moon is south of the equator, she is nearer to the Southern Ocean, where the tides are generated. The new moon is nearest to the sun, and crosses the meridian at midday, while the full moon crosses it at midnight.

The height of the afternoon tide varies from that of the morning tide; sometimes one is the higher and sometimes the other, according to the declination of the sun and moon. This is called the "diurnal inequality." The average difference between the night and morning tides is about 5 in on the east coast and about 8in on the west coast. When there is a considerable difference in the height of high water of two consecutive tides, the ebb which follows the higher tide is lower than that following the lower high water, and as a general rule the higher the tide rises the lower it will fall. The height of spring tides varies throughout the year, being at a maximum when the sun is over the equator at the equinoxes and at a minimum in June at the summer solstice when the sun is furthest away from the equator. In the Southern Ocean high water of spring tides occurs at mid-day on the meridian of Greenwich and at midnight on the 180° meridian, and is later on the coasts of other seas in proportion to the time taken for the derivative waves to reach them, the tide being about three- fourths of a day later at Land's End and one day and a half later at the mouth of the Thames. The spring tides around the coast of England are four inches higher on the average at the time of new moon than at full moon, the average rise being about 15 ft, while the average rise at neaps is 11 ft 6 in.

The height from high to low water of spring tides is approximately double that of neap tides, while the maximum height to which spring tides rise is about 33 per cent. more than neaps, taking mean low water of spring tides as the datum. Extraordinarily high tides may be expected when the moon is new or full, and in her position nearest to the earth at the same time as her declination is near the equator, and they will be still further augmented if a strong gale has been blowing for some time in the same direction as the flood tide in the open sea, and then changes when the tide starts to rise, so as to blow straight on to the shore. The pressure of the air also affects the height of tides in so far as an increase will tend to depress the water in one place, and a reduction of pressure will facilitate its rising elsewhere, so that if there is a steep gradient in the barometrical pressure falling in the same direction as the flood tide the tides will be higher. As exemplifying the effect of violent gales in the Atlantic on the tides of the Bristol Channel, the following extract from "The Surveyor, Engineer, and Architect" of 1840, dealing with observations taken on Mr. Bunt's self-registering tide gauge at Hotwell House, Clifton, may be of interest.

Date: Times of High Water. Difference in Jan 1840. Tide Gauge. Tide Table. Tide Table. H.M. H.M. 27th, p.m....... 0. 8 .... 0. 7 ..... 1 min earlier. 28th, a.m....... 0.47 .... 0.34 ..... 13 min earlier. 28th, p.m....... 11.41 .... 1. 7 ..... 86 min later. 29th, a.m....... 1.29 .... 1.47 ..... 18 min later. 29th, p.m....... 2.32 .... 2.30 ..... 2 min earlier.

Although the times of the tides varied so considerably, their heights were exactly as predicted in the tide-table.

The records during a storm on October 29, 1838, gave an entirely different result, as the time was retarded only ten or twelve minutes, but the height was increased by 8 ft On another occasion the tide at Liverpool was increased 7 ft by a gale. The Bristol Channel holds the record for the greatest tide experienced around the shores of Great Britain, which occurred at Chepstow in 1883, and had a rise of 48 ft 6 in The configuration of the Bristol Channel is, of course, conducive to large tides, but abnormally high tides do not generally occur on our shores more frequently than perhaps once in ten years, the last one occurring in the early part of 1904, although there may foe many extra high ones during this period of ten years from on-shore gales. Where tides approach a place from different directions there may be an interval between the times of arrival, which results in there being two periods of high and low water, as at Southampton, where the tides approach from each side of the Isle of Wight.

The hour at which high water occurs at any place on the coast at the time of new or full moon is known as the establishment of that place, and when this, together with the height to which the tide rises above low water is ascertained by actual observation, it is possible with the aid of the nautical almanack to make calculations which will foretell the time and height of the daily tides at that place for all future time. By means of a tide-predicting machine, invented by Lord Kelvin, the tides for a whole year can be calculated in from three to four hours. This machine is fully described in the Minutes of Proceedings, Inst.C.E., Vol. LXV. The age of the tide at any place is the period of time between new or full moon and the occurrence of spring tides at that place. The range of a tide is the height between high and low water of that tide, and the rise of a tide is the height between high water of that tide and the mean low water level of spring tides. It follows, therefore, that for spring tides the range and rise are synonymous terms, but at neap tides the range is the total height between high and low water, while the rise is the difference between high water of the neap tide and the mean low water level of spring tides. Neither the total time occupied by the flood and ebb tides nor the rate of the rise and fall are equal, except in the open sea, where there are fewer disturbing conditions. In restricted areas of water the ebb lasts longer than the flood.

Although the published tide-tables give much detailed information, it only applies to certain representative ports, and even then it is only correct in calm weather and with a very steady wind, so that in the majority of cases the engineer must take his own observations to obtain the necessary local information to guide him in the design of the works. It is impracticable for these observations to be continued over the lengthy period necessary to obtain the fullest and most accurate results, but, premising a general knowledge of the natural phenomena which affect the tides, as briefly described herein, he will be able to gauge the effect of the various disturbing causes, and interpret the records he obtains so as to arrive at a tolerably accurate estimate of what may be expected under any particular circumstances. Generally about 25 per cent. of the tides in a year are directly affected by the wind, etc., the majority varying from 6 in to 12 in in height and from five to fifteen minutes in time. The effect of a moderately stiff gale is approximately to raise a tide as many inches as it might be expected to rise in feet under normal conditions. The Liverpool tide-tables are based on observations spread over ten years, and even longer periods have been adopted in other places.

Much valuable information on this subject is contained in the following books, among others—and the writer is indebted to the various authors for some of the data contained in this and subsequent chapters—"The Tides," by G. H. Darwin, 1886; Baird's Manual of Tidal Observations, 1886; and "Tides and Waves," by W. H. Wheeler, 1906, together with the articles in the "Encyclopaedia Britannica" and "Chambers's Encyclopaedia."

Chapter II

Observations of the rise and fall of tides.

The first step in the practical design of the sewage works is to ascertain the level of high and low water of ordinary spring and neap tides and of equinoctial tides, as well as the rate of rise and fall of the various tides. This is done by means of a tide recording instrument similar to Fig. 4, which represents one made by Mr. J. H. Steward, of 457, West Strand, London, W.C. It consists of a drum about 5 in diameter and 10 in high, which revolves by clockwork once in twenty-four hours, the same mechanism also driving a small clock. A diagram paper divided with vertical lines into twenty-four primary spaces for the hours is fastened round the drum and a pen or pencil attached to a slide actuated by a rack or toothed wheel is free to work vertically up and down against the drum. A pinion working in this rack or wheel is connected with a pulley over which a flexible copper wire passes through the bottom of the case containing the gauge to a spherical copper float, 8 inches diameter, which rises and falls with the tide, so that every movement of the tide is reproduced moment by moment upon the chart as it revokes. The instrument is enclosed in an ebonized cabinet, having glazed doors in front and at both sides, giving convenient access to all parts. Inasmuch as the height and the time of the tide vary every day, it is practicable to read three days' tides on one chart, instead changing it every day. When the diagrams are taken of, the lines representing the water levels should be traced on to a continuous strip of tracing linen, so that the variations can be seen at a glance extra lines should be drawn, on the tracing showing the time at which the changes of the moon occur.

Fig. 5 is a reproduction to a small scale of actual records taken over a period of eighteen days, which shows true appearance of the diagrams when traced on the continuous strip.

These observations show very little difference between the spring and neap tides, and are interesting as indicating the unreliability of basing general deductions upon data obtained during a limited period only. At the time of the spring tides at the beginning of June the conditions were not favourable to big tides, as although the moon was approaching her perigee, her declination had nearly reached its northern limit and the declination of the sun was 22° IN The first quarter of the moon coincided very closely with the moon's passage over the equator, so that the neaps would be bigger than usual. At the period of the spring: tides, about the middle of June, although the time of full moon corresponded with her southernmost declination, she was approaching her apogee, and the declination of the sun was 23° 16' N., so that the tides would be lower than usual.

In order to ensure accurate observations, the position chosen for the tide gauge should be in deep water in the immediate vicinity of the locus in quo, but so that it is not affected by the waves from passing vessels. Wave motion is most felt where the float is in shallow water. A pier or quay wall will probably be most convenient, but in order to obtain records of the whole range of the tides it is of course necessary that the float should not be left dry at low water. In some instances the float is fixed in a well sunk above high water mark to such a depth that the bottom of it is below the lowest low water level, and a small pipe is then laid under the beach from the well to, and below, low water, so that the water stands continuously in the well at the same level as the sea.

The gauge should be fixed on bearers, about 3 ft 6 in from the floor, in a wooden shed, similar to a watchman's box, but provided with a door, erected on the pier or other site fixed upon for the observations. A hole must be formed in the floor and a galvanized iron or timber tube about 10 in square reaching to below low water level fixed underneath, so that when the float is suspended from the recording instrument it shall hang vertically down the centre of the tube. The shed and tube must of course be fixed securely to withstand wind and waves. The inside of the tube must be free from all projections or floating matter which would interfere with the movements of the float, the bottom should be closed, and about four lin diameter holes should be cleanly formed in the sides near to the bottom for the ingress and egress of the water. With a larger number of holes the wave action will cause the diagram to be very indistinct, and probably lead to incorrectness in determining the actual levels of the tides; and if the tube is considerably larger than the float, the latter will swing laterally and give incorrect readings.

A bench mark at some known height above ordnance datum should be set up in the hut, preferably on the top of the tube. At each visit the observer should pull the float wire down a short distance, and allow it to return slowly, thus making a vertical mark on the diagram, and should then measure the actual level of the surface of the water below the bench mark in the hut, so that the water line on the chart can be referred to ordnance datum. He should also note the correct time from his watch, so as to subsequently rectify any inaccuracy in the rate of revolution of the drum.

The most suitable period for taking these observations is from about the middle of March to near the end of June, as this will include records of the high spring equinoctial tides and the low "bird" tides of June. A chart similar to Fig. 6 should be prepared from the diagrams, showing the rise and fall of the highest spring tides, the average spring tides, the average neap tides, and the lowest neap tides, which will be found extremely useful in considering the levels of, and the discharge from, the sea outfall pipe.

The levels adopted for tide work vary in different ports. Trinity high-water mark is the datum adopted for the Port of London by the Thames Conservancy; it is the level of the lower edge of a stone fixed in the face of the river wall upon the east side of the Hermitage entrance of the London Docks, and is 12 48 ft above Ordnance datum. The Liverpool tide tables give the heights above the Old Dock Sill, which is now non-existent, but the level of it has been carefully preserved near the same position, on a stone built into the western wall of the Canning Half Tide Dock. This level is 40 ft below Ordnance datum. At Bristol the levels are referred to the Old Cumberland Basin (O.C.B.), which is an imaginary line 58 ft below Ordnance datum. It is very desirable that for sewage work all tide levels should be reduced to Ordnance datum.

A critical examination of the charts obtained from the tide- recording instruments will show that the mean level of the sea does not agree with the level of Ordnance datum. Ordnance datum is officially described as the assumed mean water level at Liverpool, which was ascertained from observations made by the Ordnance Survey Department in March, 1844, but subsequent records taken in May and June, 1859, by a self-recording gauge on St. George's Pier, showed that the true mean level of the sea at Liverpool is 0.068 ft below the assumed level. The general mean level of the sea around the coast of England, as determined by elaborate records taken at 29 places during the years 1859-60, was originally said to be, and is still, officially recognised by the Ordnance Survey Department to be 0.65 ft, or 7.8 in, above Ordnance datum, but included in these 29 stations were 8 at which the records were admitted to be imperfectly taken. If these 8 stations are omitted from the calculations, the true general mean level of the sea would be 0.623 ft, or 7.476 in, above Ordnance datum, or 0.691 ft above the true mean level of the sea at Liverpool. The local mean seal level at various stations around the coast varies from 0.982 ft below the general mean sea level at Plymouth, to 1.260 ft above it at Harwich, the places nearest to the mean being Weymouth (.089 ft below) and Hull (.038 ft above).

It may be of interest to mention that Ordnance datum for Ireland is the level of low water of spring tides in Dublin Bay, which is 21 ft below a mark on the base of Poolbeg Lighthouse, and 7.46 ft below English Ordnance datum.

The lines of "high and low water mark of ordinary tides" shown upon Ordnance maps represent mean tides; that is, tides halfway between the spring and the neap tides, and are generally surveyed at the fourth tide before new and full moon. The foreshore of tidal water below "mean high water" belongs to the Crown, except in those cases where the rights have been waived by special grants. Mean high water is, strictly speaking, the average height of all high waters, spring and neap, as ascertained over a long period. Mean low water of ordinary spring tides is the datum generally adopted for the soundings on the Admiralty Charts, although it is not universally adhered to; as, for instance, the soundings in Liverpool Bay and the river Mersey are reduced to a datum 20 ft below the old dock sill, which is 125 ft below the level of low water of ordinary spring tides. The datum of each chart varies as regards

Ordnance datum, and in the case of charts embracing a large area the datum varies along the coast.

The following table gives the fall during each half-hour of the typical tides shown in Fig, 6 (see page 15), from which it will be seen that the maximum rate occurs at about half-tide, while very little movement takes place during the half-hour before and the half-hour after the turn of the tide:—

Table I.

Rate of fall of tides.

State of Eqionoctial Ordinary Ordinary Lowest
Tide. Tides. Spring Tides. Neap Tides. Neap Tides.

High water — — — — 1/2 hour after 0.44 0.40 0.22 0.19 1 " " 0.96 0.80 0.40 0.31 1-1/2 " " 1.39 1.14 0.68 0.53 2 " " 1.85 1.56 0.72 0.59 2-1/2 " " 1.91 1.64 0.84 0.68 3 " " 1.94 1.66 0.86 0.70 3-1/2 " " 1.94 1.66 0.86 0.70 4 " " 1.91 1.64 0.84 0.68 4-1/2 " " 1.35 1.16 0.59 0.48 5 " " 1.27 1.09 0.57 0.46 5-1/2 " " 1.06 0.91 0.47 0.38 6 " " 1.04 0.89 0.46 0.37 6-1/2 " " 0.53 0.45 0.24 0.18 Totals.... 17 ft 6 in 15 ft 0 in 7 ft 9 in 6 ft 3 in

The extent to which the level of high water varies from tide to tide is shown in Fig. 7 [Footnote: Plate III.], which embraces a period of six months, and is compiled from calculated heights without taking account of possible wind disturbances.

The varying differences between the night and morning tides are shown very clearly on this diagram; in some cases the night tide is the higher one, and in others the morning tide; and while at one time each successive tide is higher than the preceding one, at another time the steps showing: the set-back of the tide are very marked. During the earlier part of the year the spring-tides at new moon were higher than those at full moon, but towards June the condition became reversed. The influence of the position of the sun and moon on the height of the tide is apparent throughout, but is particularly marked during the exceptionally low spring tides in the early part of June, when the time of new moon practically coincides with the moon in apogee and in its most northerly position furthest removed from the equator.

Inasmuch as the tidal waves themselves have no horizontal motion, it is now necessary to consider by what means the movement of water along the shores is caused. The sea is, of course, subject to the usual law governing the flow of water, whereby it is constantly trying to find its own level. In a tidal wave the height of the crest is so small compared with the length that the surface gradient from crest to trough is practically flat, and does not lead to any appreciable movement; but as the tidal wave approaches within a few miles of the shore, it runs into shallow water, where its progress is checked, but as it is being pushed on from behind it banks up and forms a crest of sufficient height to form a more or less steep gradient, and to induce a horizontal movement of the particles of water throughout the whole depth in the form of a tidal current running parallel with the shore.

The rate of this current depends upon the steepness of the gradient, and the momentum acquired will, In some Instances, cause the current to continue to run in the same direction for some time after the tide has turned, i.e., after the direction of the gradient has been reversed; so that the tide may be making—or falling—in one direction, while the current is running the opposite way. It will be readily seen, then, that the flow of the current will be slack about the time of high and low water, so that its maximum rate will be at half-ebb and half-flood. If the tide were flowing into an enclosed or semi-enclosed space, the current could not run after the tide turned, and the reversal of both would be simultaneous, unless, indeed, the current turned before the tide.

Wind waves are only movements of the surface of the water, and do not generally extend for a greater depth below the trough of the wave than the crest is above it, but as they may affect the movement of the floating particles of sewage to a considerable extent it is necessary to record the direction and strength of the wind.

The strength of the wind is sometimes indicated wind at the time of making any tidal observations. By reference to the Beaufort Scale, which is a graduated classification adopted by Admiral Beaufort about the year 1805. The following table gives the general description, velocity, and pressure of the wind corresponding to the tabular numbers on the scale:—

[Illustration: PLATE III

PERIOD OF SIX MONTHS.

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The figures indicating the pressure of the wind in the foregoing table are low compared with those given by other authorities. From Mutton's formula, the pressure against a plane surface normal to the wind would be 0.97 lb per sq. foot, with an average velocity of 15 miles per hour (22 ft per sec.), compared with o.67 lb given by Admiral Beaufort, and for a velocity of 50 miles per hour (73.3 ft per sec.) 10.75 lb, compared with 7.7lb Semitone's formula, which is frequently used, gives the pressure as 0.005V^2 (miles per hour), so that for 15 miles per hour velocity the pressure would be 1.125 lb, and for 50 miles it would be l2.5 lb It must not be forgotten, however, that, although over a period of one hour the wind may *average* this velocity or pressure, it will vary considerably from moment to moment, being far in excess at one time, and practically calm at another. The velocity of the wind is usually taken by a cup anemometer having four 9 in cups on arms 2 ft long. The factor for reducing the records varies from 2 to 3, according to the friction and lubrication, the average being 2.2.

The pressure is obtained by multiplying the Beaufort number cubed by 0.0105; and the velocity is found by multiplying the square root of the Beaufort number cubed by 1.87.

A tidal wave will traverse the open sea in a straight line, but as it passes along the coast the progress of the line nearest the shore is retarded while the centre part continues at the same velocity, so that on plan the wave assumes a convex shape and the branch waves reaching the shore form an acute angle with the coast line.

CHAPTER III.

CURRENT OBSERVATIONS.

There is considerable diversity in the design of floats employed in current observations, dependant to some extent upon whether it is desired to ascertain the direction of the surface drift or of a deep current, it does not by any means follow that they run in simultaneous directions. There is also sometimes considerable difference in the velocity of the current at different depths—the surface current being more susceptible to influence of wind. A good form of deep float is seen in Fig. 8. It consists of a rod 2 in by 2 in, or 4 sq in The lower end of which a hollow wooden box about 6 in by 6 in is fixed, into which pebbles are placed to overcome the buoyancy of the float and cause it to take and maintain an upright position in the water with a length of 9in of the rod exposed above the surface. A small hole is formed in the top of the box for the insertion the pebbles, which is stopped up with a cork when the float is adjusted. The length of the rod will vary according to the depth of water, but it will generally be found convenient to employ a float about 10 ft and to have a spare one about 6 ft deep, but otherwise it is similar in all respects, for use in shallow water. A cheap float for gauging the surface drift can be made from an empty champagne bottle weighted with stones and partly filled with water. The top 12 in of rods and the cord and neck of the bottle, as the case may be, should be painted red, as this colour renders floats more conspicuous when in the water and gives considerable assistance in locating their position, especially when they are at some distance from the observer.

A deep-sea float designed by Mr. G. P. Bidden for ascertaining the set of the currents along the base of the ocean has recently been used by the North Sea Fisheries Investigation Committee. It consists of a bottle shaped like a soda-water bottle, made of strong glass to resist the pressure of the water, and partly filled with water, so that just sufficient air is left in it to cause it to float. A length of copper wire heavy enough to cause it to sink is then attached to the bottle, which is then dropped into the sea at a defined place. When the end of the wire touches the bottom the bottle is relieved of some of its weight and travels along with the currents a short distance above the bed of the sea. About 20 per cent. of the bottles were recovered, either by being thrown up on the beach or by being fished up in trawl nets.

[Illustration: FIG. 8.—DETAIL OF WOOD TIDAL FLOAT 10 FEET DEEP.]

A double float, weighing about 10 lb complete, was used for the tidal observations for the Girdleness outfall sewer, Aberdeen. The surface portion consisted of two sheet-iron cups soldered together, making a float 9 in in diameter and 6 in deep. The lower or submerged portion was made of zinc, cylindrical in shape, 16 in diameter and 16 in long, perforated at intervals with lin diameter holes and suspended by means of a brass chain from a swivel formed on the underside of the surface float.

In gauging the currents the float is placed in the water at a defined point and allowed to drift, its course being noted and afterwards transferred to a plan. The time of starting should be recorded and observations of its exact position taken regularly at every quarter of an hour, so that the time taken in covering any particular distance is known and the length of travel during any quarter-hour period multiplied by four gives the speed of the current at that time in miles per hour.

The method to be employed in ascertaining the exact position of the float from time to time is a matter which requires careful consideration, and is dependent upon the degree of accuracy required according to the importance of the scheme and the situation of neighbouring towns, frequented shores, oyster beds, and other circumstances likely to be injuriously affected by any possible or probable pollution by sewage.

One method is to follow the float in a small boat carrying a marine compass which has the card balanced to remain in a horizontal position, irrespective of the tipping and rolling of the boat, and to observe simultaneously the bearing of two prominent landmarks, the position of which on the plan is known, at each of the quarter-hour periods at which the observations are to be taken. This method only gives very approximate results, and after checking the value of the observations made by its use, with contemporary observations taken by means of theodolites on the shore, the writer abandoned the system in favour of the theodolite method, which, however, requires a larger staff, and is therefore more expensive. In every case it is necessary to employ a boat to follow the float, not only so as to recover it at the end of each day's work, but principally to assist in approximately locating the float, which can then be found more readily when searching through the telescope of the theodolite. The boat should be kept about 10 ft to 20 ft from the float on the side further removed from the observers, except when surface floats are being used to ascertain the effect of the wind, when the boat should be kept to leeward of the float. Although obviously with a large boat the observations can be pursued through rougher weather, which is an important point, still the difficulty of maintaining a large boat propelled by mechanical power, or sail, sufficiently near the float to assist the observers, prevents its use, and the best result will be obtained by employing a substantial, seaworthy rowing boat with a broad beam. The boatmen appreciate the inclusion of a mast, sails, and plenty of ballast in the equipment to facilitate their return home when the day's work is done, which may happen eight or nine miles away, with twilight fast passing into darkness. There should be two boatmen, or a man and a strong youth.

In working with theodolites, it is as well before starting to select observation stations at intervals along the coast, drive pegs in the ground so that they can easily be found afterwards, and fix their position upon a 1/2500 ordnance map in the usual manner. It may, however, be found in practice that after leaving one station it is not possible to reach the next one before the time arrives for another sight to be taken. In this case the theodolite must be set up on magnetic north at an intermediate position, and sights taken to at least two landmarks, the positions of which are shown on the map, and the point of observation subsequently plotted as near as possible by the use of these readings. Inasmuch as the sights will be taken from points on the edge of the shore, which is, of course, shown on the map, it is possible, after setting up to magnetic north, to fix the position with approximate accuracy by a sight to one landmark only, but this should only be done in exceptional circumstances.

The method of taking the observations with two theodolites, as adopted by the writer, can best be explained by a reference to Fig. 9, which represents an indented piece of the coast. The end of the proposed sea outfall sewer, from which point the observations would naturally start, is marked 1, the numerals 2, 3, 4, etc., indicating the positions of the float as observed from time to time. Many intermediate observations would be taken, but in order to render the diagram more clear, these have not been shown. The lines of sight are marked 1A, 1B, etc. The points marked A1, A2, etc., indicate the first, second, etc., and subsequent positions of observer A; the points B1, B2, etc., referring to observer B. The dot-and-dash line shows the course taken by the float, which is ascertained after plotting the various observations recorded.

It is very desirable to have a horse and trap in waiting to move the observers and their instruments from place to place as required, and each observer should be provided with small flags about 2 ft square, one white and one blue, for signalling purposes.

The instruments are first set up at A1 and B1 respectively, and adjusted to read on to the predetermined point 1 where the float is to be put in Then as soon as the boatmen have reached the vicinity of this point, the observers can, by means of the flags, direct them which way to row so as to bring the boat to the exact position required, and when this is done the anchor is dropped until it is time to start, which is signalled by the observers holding the flags straight above their heads. This is also the signal used to indicate to the men that the day's work is finished, and they can pick up the float and start for home.

[Illustration: FIG. 9.—PLAN OF INDENTED COAST-LINE LLUSTRATING METHOD OF TAKING CURRENT OBSERVATIONS WITH TWO THEODOLITES.]

Directly the float is put in the water, and at every even quarter of an hour afterwards, each observer takes a reading of its exact position, and notes the time. As soon as the readings are taken to the float in position 2, the observer A should take up his instrument and drive to A2, where he must set up ready to take reading 3 a quarter of an hour after reading 2. It will be noticed that he might possibly have been able to take the reading 3 from the position A1, but the angle made by the lines of sight from the two instruments would have been too acute for accurate work, and very probably the float would have been hidden by the headland, so that he could not take the reading at all. In order to be on the headland A4 at the proper time, A must be working towards it by getting to position A3 by the time reading 4 is due. Although the remainder of the course of the float can be followed from B1 and A4, the instruments would be reading too much in the same line, so that B must move to B2 and then after reading 5 and 6 he should move to B3. As the float returns towards the starting point, A can remain in the position A4 while B goes to B4 and then moves back along the shore as the float progresses.

The foregoing description is sufficient to indicate the general method of working, but the details will of course vary according to the configuration of the shore and the course taken by the float. Good judgment is necessary in deciding when to move from one station to the next, and celerity in setting up, adjusting the instrument, and taking readings is essential. If the boatmen can be relied upon to keep their position near the float, very long sights can be taken with sufficient accuracy by observing the position of the boat, long after the float has ceased to be visible through the telescope.

The lines of sight from each station should be subsequently plotted on the 1/2500 ordnance map; the intersection of each two corresponding sight lines giving the position of the float at that time. Then if a continuous line is drawn passing through all the points of intersection it will indicate the course taken by the float.

It is very desirable that the observers should be able to convey information to each other by signalling with the flags according to the Morse code, as follows. The dashes represent a movement of the flag from a position in front of the left shoulder to near the ground on the right side and the dots a movement from the left shoulder to the right shoulder.

TABLE 3.

MORSE ALPHABET.

E . A .- R .-. L .-.. W .— P .—. J .—- I .. U ..- F ..-. S ... V ...- H .... T - N -. K -.- C -.-. Y -.— D -.. X -..- B -... M — G —. Q —.- Z —.. O —-

The signal to attract attention at starting and to signify the end of the message is .. .. .. continued until it is acknowledged with a similar sign by the other observer; that for a repetition is .. — .. which is signalled when any part of the message is not understood, otherwise after each word is signalled the receiver waves - to indicate he understands it. Until proficiency is attained, two copies of the alphabet should be kept by each observer for reference, one for dispatching a message arranged in alphabetical order and the other far reading a message arranged as set out above. The white flag should be used when standing against a dark background, and the blue one when on the skyline or against a light background.

The conditions in tidal rivers vary somewhat from those occurring on the coast. As the crest of the tidal wave passes the mouth of the river a branch wave is sent up the river. This wave has first to overcome the water flowing down the river, which is acting in opposition to it, and in so doing causes a banking up of the water to such a height that the inclination of the surface is reversed to an extent sufficient to cause a tidal current to run up the river. The momentum acquired by the water passing up-stream carries it to a higher level towards the head of the river than at the mouth, and, similarly, in returning, the water flowing down the river gains sufficient impetus to scoop out the water at the mouth and form a low water below that in the sea adjoining. Owing to a flow of upland water down a river the ebb lasts longer than the flood tide by a period, increasing in length as the distance from the mouth of the river increases; and, similarly to the sea, the current may continue to run down a river after the tide has turned and the level of the water is rising. The momentum of the tide running up the centre of the river is in excess of that along the banks, so that the current changes near the shore before it does in the middle, and, as the sea water is of greater specific gravity than the fresh, weighing 64 lb per cubic foot against 62-1/2 lb, it flows up the bed of the river at the commencement of the tide, while the fresh water on the surface is running in the opposite direction. After a time the salt water becomes diffused in the fresh, so that the density of the water in a river decreases as the distance from the sea increases. The disposal of sewage discharged into a river is due primarily to the mixing action which is taking place; inasmuch as the tidal current which is the transporting agent rarely flows more rapidly than from two to four miles per hour, or, say, twelve to fifteen miles per tide. The extent to which the suspended matter is carried back again up stream when the current turns depends upon the quantity of upland water which has flowed into the upper tidal part of the river during the ebb tide, as this water occupies a certain amount of space, according to the depth and width of the river, and thus prevents the sea water flowing back to the position it occupied on the previous tide, and carrying with it the matter in suspension. The permanent seaward movement of sewage discharged into the Thames at Barking when there is only a small quantity of upland water is at the rate of about one mile per day, taking thirty days to travel the thirty-one miles to the sea, while at the mouth of the river the rate does not exceed one- third of a mile per day.

CHAPTER IV.

SELECTION OF SITE FOR OUTFALL SEWER.

The selection of the site for the sea outfall sewer is a matter requiring a most careful consideration of the many factors bearing on the point, and the permanent success of any scheme of sewage disposal depends primarily upon the skill shown in this matter. The first step is to obtain a general idea of the tidal conditions, and to examine the Admiralty charts of the locality, which will show the general set of the main currents into which it is desirable the sewage should get as quickly as possible. The main currents may be at some considerable distance from the shore, especially if the town is situated in a bay, when the main current will probably be found running across the mouth of it from headland to headland. The sea outfall should not be in the vicinity of the bathing grounds, the pier, or parts of the shore where visitors mostly congregate; it should not be near oyster beds or lobster grounds. The prosperity—in fact, the very existence—of most seaside towns depends upon their capability of attracting visitors, whose susceptibilities must be studied before economic or engineering questions, and there are always sentimental objections to sewage works, however well designed and conducted they may be.

It is desirable that the sea outfall should be buried in the shore for the greater part of its length, not only on account of these sentimental feelings, but as a protection from the force of the waves, and so that it should not interfere with boating; and, further, where any part of the outfall between high and low water mark is above the shore, scouring of the beach will inevitably take place on each side of it. The extreme end of the outfall should be below low-water mark of equinoctial tides, as it is very objectionable to have sewage running across the beach from the pipe to the water, and if the foul matter is deposited at the edge of the water it will probably be brought inland by the rising tide. Several possible positions may present themselves for the sea outfall, and a few trial current observations should be made in these localities at various states of the tides and plotted on to a 1:2500 ordnance map. The results of these observations will probably reduce the choice of sites very considerably.

Levels should be taken of the existing subsidiary sewers in the town, or, if there are none, the proposed arrangement of internal sewers should be sketched out with a view to their discharging their contents at one or other of the points under consideration. It may be that the levels of the sewers are such that by the time they reach the shore they are below the level of low water, when, obviously, pumping or other methods of raising the sewage must be resorted to; if they are above low water, but below high water, the sewage could be stored during high water and run off at or near low water; or, if they are above high water, the sewage could run off continuously, or at any particular time that might be decided.

Observations of the currents should now be made from the selected points, giving special attention to those periods during which it is possible to discharge the sewage having regard to the levels of the sewers. These should be made with the greatest care and accuracy, as the final selection of the type of scheme to be adopted will depend very largely on the results obtained and the proper interpretation of them, by estimating, and mentally eliminating, any disturbing influences, such as wind, etc. Care must also be taken in noting the height of the tide and the relative positions of the sun, moon, and earth at the time of making the observations, and in estimating from such information the extent to which the tides and currents may vary at other times when those bodies are differently situated.

It is obvious that if the levels of the sewers and other circumstances are such that the sewage can safely be discharged at low water, and the works are to be constructed accordingly, it is most important to have accurate information as to the level of the highest low water which may occur in any ordinary circumstances. If the level of a single low water, given by a casual observation, is adopted without consideration of the governing conditions, it may easily be that the tide in question is a low one, that may not be repeated for several years, and the result would be that, instead of having a free outlet at low water, the pipe would generally be submerged, and its discharging capacity very greatly reduced.

The run of the currents will probably differ at each of the points under consideration, so that if one point were selected the best result would be obtained by discharging the sewage at high water and at another point at low water, whereas at a third point the results would show that to discharge there would not be satisfactory at any stage of the tide unless the sewage were first partially or even wholly purified. If these results are considered in conjunction with the levels of the sewers definite alternative schemes, each of which would work satisfactory may be evolved, and after settling them in rough outline, comparative approximate estimates should be prepared, when a final scheme may be decided upon which, while giving the most efficient result at the minimum cost, will not arouse sentimental objections to a greater extent than is inherent to all schemes of sewage disposal.

Having thus selected the exact position of the outfall, the current observations from that point should be completed, so that the engineer may be in a position to state definitely the course which would be taken by sewage if discharged under any conditions of time or tide. This information is not particularly wanted by the engineer, but the scheme will have to receive the sanction of the Local Government Board or of Parliament, and probably considerable opposition will be raised by interested parties, which must be met at all points and overcome. In addition to this, it may be possible, and necessary, when heavy rain occurs, to allow the diluted sewage to escape into the sea at any stage of the tide; and, while it is easy to contend that it will not then be more impure than storm water which is permitted to be discharged into inland streams during heavy rainfall, the aforesaid sentimentalists may conjure up many possibilities of serious results. As far as possible the records should indicate the course taken by floats starting from the outfall, at high water, and at each regular hour afterwards on the ebb tide, as well as at low water and every hour on the flood tide. It is not, however, by any means necessary that they should be taken in this or any particular order, because as the height of the tide varies each day an observation taken at high water one day is not directly comparable with one taken an hour after high water the next day, and while perhaps relatively the greatest amount of information can be gleaned from a series of observations taken at the same state of the tide, but on tides of differing heights, still, every observation tells its own story and serves a useful purpose.

Deep floats and surface floats should be used concurrently to show the effect of the wind, the direction and force of which should be noted. If it appears that with an on-shore wind floating particles would drift to the shore, screening will be necessary before the sewage is discharged. The floats should be followed as long as possible, but at least until the turn of the current—that is to say, a float put in at or near high water should be followed until the current has turned at or near low water, and one put in at low water should be followed until after high water. In all references to low water the height of the tide given is that of the preceding high water.

The time at which the current turns relative to high and low water at any place will be found to vary with the height of the tide, and all the information obtained on this point should be plotted on squared paper as shown on Fig. 10, which represents the result of observations taken near the estuary of a large river where the conditions would be somewhat different from those holding in the open sea. The vertical lines represent the time before high or low water at which the current turned, and the horizontal lines the height of the tide, but the data will, of course, vary in different localities.

[Illustration: Hours before turn of tide. FIG 10]

It will be noticed that certain of the points thus obtained can be joined up by a regular curve which can be utilised for ascertaining the probable time at which the current will turn on tides of height intermediate to those at which observations were actually taken. For instance, from the diagram given it can be seen that on a 20 ft tide the current will turn thirty minutes before the tide, or on a 15 ft tide the current will turn one hour before the tide. Some of the points lie at a considerable distance from the regular curve, showing that the currents on those occasions were affected by some disturbing influence which the observer will probably be able to explain by a reference to his notes, and therefore those particular observations must be used with caution.

The rate of travel of the currents varies in accordance with the time they have been running. Directly after the turn there is scarcely any movement, but the speed increases until it reaches a maximum about three hours later and then it decreases until the next turn, when dead water occurs again.

Those observations which were started at the turn of the current and continued through the whole tide should be plotted as shown in Fig. 11, which gives the curves relating to three different tides, but, provided a sufficiently large scale is adopted, there is no reason why curves relating to the whole range of the tides should not be plotted on one diagram. This chart shows the total distance that would be covered by a float according to the height of the tide; it also indicates the velocity of the current from time to time. It can be used in several ways, but as this necessitates the assumption that with tides of the same height the flow of the currents is absolutely identical along the coast in the vicinity of the outfall, the diagram should be checked as far as possible by any observations that may be taken at other states of tides of the same heights. Suppose we require to know how far a float will travel if started at two hours after high water on a 12 ft tide. From Fig. 10 we see that on a tide of this height the current turns two hours and a quarter before the tide; therefore two hours after high water will be four hours and a quarter after the turn of the current. If the float were started with the current, we see from Fig. 11 that it would have travelled three miles in four hours and a quarter; and subtracting this from four miles, which is its full travel on a whole tide, we see that it will only cover one mile in the two hours and a quarter remaining before the current turns to run back again.

Although sewage discharged into the sea rapidly becomes so diffused as to lose its identity, still occasionally the extraneous substances in it, such as wooden matches, banana skins, etc., may be traced for a considerable distance; so that, as the sewage continues to be discharged into the sea moving past the outfall, there is formed what may be described as a body or column of water having possibilities of sewage contamination. If the time during which sewage is discharged is limited to two hours, and starts, say, at the turn of the current on a 12 ft tide, we see from Fig. 11 that the front of this body of water will have reached a point five-eighths of a mile away when the discharge ceases; so that there will be a virtual column of water of a total length of five-eighths of a mile, in which is contained all that remains of the noxious matters, travelling through the sea along the course of the current. We see, further, that at a distance of three miles away this column would only take thirty minutes to pass a given point. The extent of this column of water will vary considerably according to the tide and the time of discharge; for instance, on a 22 ft tide, if the discharge starts one hour after the turn of the current and continues for two hours, as in the previous example, it will form a column four miles long, whereas if it started two hours after the current, and continued for the same length of time, the column would be six miles and a half long, but the percentage of sewage in the water would be infinitesimal.

[Illustration: Hours after turn of current FIG. 11]

In some cases it may be essential that the sewage should be borne past a certain point before the current turns in order to ensure that it shall not be brought back on the return tide to the shore near the starting point. In other words, the sewage travelling along the line of a branch current must reach the junction on the line of the main current by a certain time in order to catch the connection. Assuming the period of discharge will be two hours, and that the point which it is necessary to clear is situated three miles and a half from the outfall, the permissible time to discharge the sewage according to the height of the tide can be obtained from Fig. 11. Taking the 22 ft tide first, it will be seen that if the float started with the current it would travel twelve miles in the tide; three and a half from twelve leaves eight and a half miles. A vertical line dropped from the intersection of the eight miles and a half line with the curve of the current gives the time two hours and a half before the end, or four hours after the start of the current at which the discharge of the sewage must cease at the outfall in order that the rear part of the column can reach the required point before the current turns. As on this tide high water is about fifteen minutes after the current, the latest time for the two hours of discharge must be from one hour and three-quarters to three hours and three-quarters after high water. Similarly with the 12 ft tide having a total travel of four miles: three and a half from four leaves half a mile, and a vertical line from the half-mile intersection gives one hour and three-quarters after the start of the current as the time for discharge to cease. High water is two hours and a quarter after the current; therefore the latest time for the period of discharge would be from two hours and a half to half an hour before high water, but, as during the first quarter of an hour the movement of the current, though slight, would be in the opposite direction, it would be advisable to curtail the time of discharge, and say that it should be limited to between two hours and a quarter and half an hour before high water. It is obvious that if sewage is discharged about two hours after high water the current will be nearing its maximum speed, but it will only have about three hours to run before it turns; so that, although the sewage may be removed with the maximum rapidity from the vicinity of the sea outfall, it will not be carried to any very great distance, and, of course, the greater the distance it is carried the more it will be diffused. It must be remembered that the foregoing data are only applicable to the locality they relate to, although after obtaining the necessary information similar diagrams can be made and used for other places; but enough has been said to show that when it is necessary to utilise the full effect of the currents the sewage should be discharged at a varying time before high or low water, as the case may be, according to the height of the tide.

CHAPTER V.

VOLUME OF SEWAGE.

The total quantity of sewage to be dealt with per day can be ascertained by gauging the flow in those cases where the sewers are already constructed, but where the scheme is an entirely new one the quantity must be estimated. If there is a water supply system the amount of water consumed per day, after making due allowance for the quantity used for trade purposes and street watering, will be a useful guide. The average amount of water used per head per day for domestic purposes only may be taken as follows:—

DAILY WATER SUPPLY
(Gallons per head per day.)

Dietetic purposes (cooking, drinking, &c.) 1 Cleansing purposes (washing house utensils, clothes, &c.) 6

If water-closets are in general use, add 3

If baths are in general use, add 5

Total 15

It therefore follows that the quantity of domestic sewage to be expected will vary from 7 to 15 gallons per head per day, according to the extent of the sanitary conveniences installed in the town; but with the advent of an up-to-date sewage scheme, probably accompanied by a proper water supply, a very large increase in the number of water-closets and baths may confidently be anticipated, and it will rarely be advisable to provide for a less quantity of domestic sewage than 15 gallons per head per day for each of the resident inhabitants. The problem is complicated in sea coast towns by the large influx of visitors during certain short periods of the year, for whom the sewerage system must be sufficient, and yet it must not be so large compared with the requirements of the residential population that it cannot be kept in an efficient state during that part of the year when the visitors are absent. The visitors are of two types—the daily trippers and those who spend several days or weeks in the town. The daily tripper may not directly contribute much sewage to the sewers, but he does indirectly through those who cater for his wants. The resident visitor will spend most of the day out of doors, and therefore cause less than the average quantity of water to be used for house-cleansing purposes, in addition to which the bulk of the soiled linen will not be washed in the town. An allowance of 10 gallons per head per day for the resident visitor and 5 gallons per head per day for the trippers will usually be found a sufficient provision.

It is, of course, well known that the flow of sewage varies from day to day as well as from hour to hour, and while there is no necessity to consider the daily variation—calculations being based on the flow of the maximum day—the hourly variation plays a most important part where storage of the sewage for any length of time is an integral part of the scheme. There are many important factors governing this variation, and even if the most elaborate calculations are made they are liable to be upset at any time by the unexpected discharge of large quantities of trade wastes. With a small population the hourly fluctuation in the quantity of sewage flowing into the sewers is very great, but it reduces as the population increases, owing to the diversity of the occupations and habits of the inhabitants. In all cases where the residential portions of the district are straggling, and the outfall works are situated at a long distance from the centre of the town, the flow becomes steadier, and the inequalities are not so prominently marked at the outlet end of the sewer. The rate of flow increases more or less gradually to the maximum about midday, and falls off in the afternoon in the same gradual manner. The following table, based on numerous gaugings, represents approximately the hourly variations in the dry weather flow of the sewage proper from populations numbering from 1,000 to 10,000, and is prepared after deducting all water which may be present in the sewers resulting from the infiltration of subsoil water through leaky joints in the pipes, and from defective water supply fittings as ascertained from the night gaugings. Larger towns have not been included in the table because the hourly rates of flow are generally complicated by the discharge of the trade wastes previously referred to, which must be the subject of special investigation in each case.

[TABLE NO. 4.

APPROXIMATE HOURLY VARIATION IN THE FLOW OF SEWAGE.

Percentage of Total Flow Passing Off in each Hour.

Hour.	Population. 1,000	2,000	3,000	4,000	5,000	6,000	8,000	10,000
Midnight	1.0	1.0	1.2	1.3	1.5	1.5	1.8	2.0
1.0 a.m.	0.7	0.7	0.7	0.8	0.8	1.0	1.0	1.0
2.0 "	nil	nil	nil	nil	0.2	0.2	0.3	0.5
3.0 "	nil	nil	nil	nil	nil	nil	nil	0.2
4.0 "	nil	nil	nil	nil	nil	nil	nil	nil
5.0 "	nil	nil	nil	nil	nil	nil	nil	0.2
6.0 "	0.2	0.2	0.3	0.5	0.6	0.5	0.7	0.8
7.0 "	0.5	0.5	1.0	1.5	1.6	1.7	2.0	2.5
8.0 "	1.0	1.5	2.0	2.5	3.0	3.5	4.0	5.0
9.0 "	3.5	4.5	4.5	4.8	5.5	5.8	6.0	6.5
10.0 "	6.5	6.5	6.8	7.0	7.5	7.7	8.0	8.0
11.0 "	10.5	11.0	10.5	10.0	9.6	9.3	9.0	8.8
Noon	11.0	11.3	10.8	10.3	9.3	9.5	9.2	9.0
1.0 p.m.	6.0	5.5	6.0	6.7	7.0	7.2	7.3	7.5
2.0 "	7.0	7.3	7.0	7.0	6.5	6.5	6.2	6.0
3.0 "	6.8	6.5	6.5	6.5	6.5	6.3	6.3	6.0
4.0 "	7.5	7.5	7.3	7.0	6.7	6.5	6.2	6.7
5.0 "	6.5	6.5	6.5	6.3	6.0	6.0	6.0	5.8
6.0 "	4.5	4.5	4.7	4.8	5.0	5.0	5.0	5.2
7.0 "	6.5	6.2	6.0	5.8	5.5	5.5	5.5	4.7
8.0 "	6.2	6.0	5.8	5.5	5.5	5.3	5.0	4.8
9.0 "	5.0	4.8	4.7	4.5	4.5	4.5	4.5	4.0
10.0 "	4.8	4.6	4.2	4.0	3.8	3.5	3.0	3.0
11.0 "	4.3	3.5	3.5	3.2	3.2	3.0	3.0	2.8
Total	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0

ANALYSIS OF FLOW]

Percentage of total flow passing off during period named.

	Population. 1,000	2,000	3,000	4,000	5,000	6,000	8,000	10,000
7.0 a.m. to 7.0 p.m	77.3	78.8	78.6	78.7	78.5	78.8	78.7	75.2
7.0 p.m. to 7.0 a.m	22.7	21.2	21.4	21.3	21.5	21.2	21.3	21.8
Maximum 12 hrs.	84.0	83.6	82.6	81.7	81.0	80.6	79.7	78.2
" 10 "	72.8	72.8	72.1	71.4	70.0	69.8	69.2	68.5
" 9 "	66.3	66.6	66.1	65.6	64.5	64.8	64.2	63.3
" 8 "	61.8	62.1	61.4	60.8	59.5	59.0	58.2	57.5
" 6 "	48.8	49.1	43.1	47.5	46.8	46.5	46.0	45.8
" 3 "	23.0	28.8	27.11	27.3	26.8	26.5	26.2	25.8
" 2 "	21.5	22.3	21.3	20.3	19.3	18.5	18.2	17.3
" 1 "	11.0	11.3	10.8	10.3	9.8	9.5	9.2	9.0
Minimum 9 "	3.4	3.9	5.2	6.6	7.5	6.9	8.8	10.0
" 10 "	6.9	7.4	8.7	9.8	10.7	10.4	11.8	13.0

The data in the foregoing table, so far as they relate to populations of one, five, and ten thousand respectively, are reproduced graphically in Fig. 12.

This table and diagram relate only to the flow of sewage—that is, water which is intentionally fouled; but unfortunately it is almost invariably found that the flow in the sewers is greater than is thus indicated, and due allowance must be made accordingly. The greater the amount of extra liquid flowing in the sewers as a permanent constant stream, the less marked will be the hourly variations; and in one set of gaugings which came under the writer's notice the quantity of extraneous liquid in the sewers was so greatly in excess of the ordinary sewage flow that, taken as a percentage of the total daily flow, the hourly variation was almost imperceptible.

[Illustration: Fig 12 Hourly Variation in Flow of Sewage.]

Provision must be made in the scheme for the leakage from the water fittings, and for the subsoil water, which will inevitably find its way into the sewers. The quantity will vary very considerably, and is difficult of estimation. If the water is cheap, and the supply plentiful, the water authority may not seriously attempt to curtail the leakage; but in other cases it will be reduced to a minimum by frequent house to house inspection; some authorities going so far as to gratuitously fix new washers to taps when they are required. Theoretically, there should be no infiltration of subsoil water, as in nearly all modern sewerage schemes the pipes are tested and proved to be watertight before the trenches are filled in; but in practice this happy state is not obtainable. The pipes may not all be bedded as solidly as they should be, and when the pressure of the earth comes upon them settlement takes place and the joints are broken. Joints may also be broken by careless filling of trenches, or by men walking upon the pipes before they are sufficiently covered. Some engineers specify that all sewers shall be tested and proved to be absolutely water-tight before they are "passed" and covered in, but make a proviso that if, after the completion of the works, the leakage into any section exceeds 1/2 cubic foot per minute per mile of sewer, that length shall be taken up and relaid. Even if the greatest vigilance is exercised to obtain water-tight sewers, the numerous house connections are each potential sources of leakage, and when the scheme is complete there may be a large quantity of infiltration water to be dealt with. Where there are existing systems of old sewers the quantity of infiltration water can be ascertained by gauging the night flow; and if it is proved to be excessive, a careful examination of the course of the sewers should be made with a view to locating the places where the greater part of the leakage occurs, and then to take such steps as may be practicable to reduce the quantity.

CHAPTER VI.

GAUGING FLOW IN SEWERS.

A method frequently adopted to gauge the flow of the sewage is to fix a weir board with a single rectangular notch across the sewer in a convenient manhole, which will pond up the sewage; and then to ascertain the depth of water passing over the notch by measurements from the surface of the water to a peg fixed level with the bottom of the notch and at a distance of two or three feet away on the upstream side. The extreme variation in the flow of the sewage is so great, however, that if the notch is of a convenient width to take the maximum flow, the hourly variation at the time of minimum flow will affect the depth of the sewage on the notch to such a small extent that difficulty may be experienced in taking the readings with sufficient accuracy to show such variations in the flow, and there will be great probability of incorrect results being obtained by reason of solid sewage matter lodging on the notch. When the depth on a l2 in notch is about 6 in, a variation of only 1-16th inch in the vertical measurement will represent a difference in the rate of the flow of approximately 405 gallons per hour, or about 9,700 gallons per day. When the flow is about lin deep the same variation of 1-16th in will represent about 162 gallons per hour, or 3,900 gallons per day. Greater accuracy will be obtained if a properly-formed gauging pond is constructed independently of the manhole and a double rectangular notch, similar to Fig. 13, or a triangular or V- shaped notch, as shown in Fig. 14, used in lieu of the simpler form.

In calculating the discharge of weirs there are several formulæ to choose from, all of which will give different results, though comparative accuracy has been claimed for each. Taking first a single rectangular notch and reducing the formulae to the common form:

Discharge per foot in width of weir =

$$C\sqrt{H^3}$$

where H = depth from the surface of still water above the weir to the level of the bottom of the notch, the value of C will be as set out in the following table:—

TABLE No. 5.

RECTANGULAR NOTCHES.

Discharge per foot in width of notch = $C\sqrt{H^3}$

	Values of C.			
H Measured in	Feet.		Inches.	
Discharge in	Gallons per hour.	C. ft per min	Gallons per hour.	C. ft per min
Authority.				
Box	79,895	213.6	1,922	5.13
Cotterill	74,296	198.6	1,787	4.78
Francis	74,820	200.0	1,800	4.81
Mo'esworth	80,057	214.0	1,926	5.15
Santo Crimp	72,949	195.0	1,755	4.69

In the foregoing table Francis' short formula is used, which does not take into account the end contractions and therefore gives a slightly higher result than would otherwise be the case, and in Cotterill's formula the notch is taken as being half the width of the weir, or of the stream above the weir. If a cubic foot is taken as being equal to 6-1/4 gallons instead of 6.235 gallons, then, cubic feet per minute multiplied by 9,000 equals gallons per day. This table can be applied to ascertain the flow through the notch shown in Fig. 13 in the following way. Suppose it is required to find the discharge in cubic feet per minute when the depth of water measured in the middle of the notch is 4 in Using Santo Crimp's formula the result will be

$$C\sqrt{H^3} = 4.69\sqrt{4^3} = 4.69 \times 8 = 37.52$$

cubic feet per foot in width of weir, but as the weir is only 6 in wide, we must divide this figure by 2, then

$37.52/2 = 18.76$ cubic feet, which is the discharge per minute.

+———+ +———+ | | FIG. 13 | | | | | | | | | | | +———+ +———+ | | | | | | | | | | | | | | +———+ | | | | | | | | | +—————————————————+

Fig. 13.-ELEVATION OF DOUBLE RECTANGULAR NOTCHED GAUGING WEIR.

+———+ +———+ | \ FIG. 13 / | | \ / | | \ / | | \ / | | \ / | | \ / | | \ / | | \ / | | \ / | | \/ | | | | | | | | | | | + —————————————————+

FIG. 14.-ELEVATION OF TRIANGULAR NOTCHED GAUGING WEIR.

FIG. 15.-LONGITUDINAL SECTION, SHOWING WEIR, GAUGE-PEG, AND HOOK-GAUGE

If it is required to find the discharge in similar terms with a depth of water of 20 in, two sets of calculations are required. First 20 in depth on the notch 6 in wide, and then 4 in depth on the notch, 28 in minus 6 in, or 1 ft wide.

(1) $C\sqrt{H^3} = 4.69/2\sqrt{10^3} = 2.345 \times 31.62 = 74.15$

(2) $C\sqrt{H^3} = 1.0 \times 4.69\sqrt{4^3} = 1.0 \times 4.69 \times 8 = 37.52$

Total in c. ft per min = 111.67

The actual discharge would be slightly in excess of this.

In addition to the circumstances already enumerated which affect the accuracy of gaugings taken by means of a weir fixed in a sewer there is also the fact that the sewage approaches the weir with a velocity which varies considerably from time to time. In order to make allowance for this, the head calculated to produce the velocity must be added to the actual head. This can be embodied in the formula, as, for example, Santo Crimp's formula for discharge in cubic feet per minute, with H measured in feet, is written

$$195\sqrt{(11^3 + .035V - H^2}$$

instead of the usual form of

$$195\sqrt{H^3},$$ which is used

when there is no velocity to take into account. The V represents the velocity in feet per second.

Triangular or V notches are usually formed so that the angle between the two sides is 90°, when the breadth at any point will always be twice the vertical height measured at the centre. The discharge in this case varies as the square root of the fifth power of the height instead of the third power as with the rectangular notch. The reason for the alteration of the power is that *approximately* the discharge over a notch with any given head varies as the cross-sectional area of the body of water passing over it. The area of the 90° notch is half that of a circumscribing rectangular notch, so that the discharge of a V notch is approximately equal to that of a rectangular notch having a width equal to half the width of the V notch at water level, and as the total width is equal to double the depth of water passing over the notch the half width is equal to the full depth and the discharge is equal to that of a rectangular notch having a width equal to the depth of water flowing over the V notch from time to time, both being measured in the same unit, therefore $C\sqrt{H^3}$ becomes $C \times H \times \sqrt{H^3}$ which equals $C\sqrt{H^5}$.

The constant C will, however, vary from that for the rectangular notch to give an accurate result.

TABLE No. 6.

TRIANGULAR OR V NOTCHES.

Discharge = $C \times \sqrt{H^5}$.

Values of C.

H Measured in	Feet.		Inches.	
Discharge in	Gallons per hour	C. ft per min	Gallons per hour.	C. ft per min
Alexander	59,856	160	120.0	0.321
Cotterill	57,013	152.4	114.3	0.306
Molesworth	59,201	158.2	118.7	0.317
Thomson	57,166	152.8	114.6	0.306

Cotterill's formula for the discharge in cubic feet per minute is

$$16 \times C \times B\sqrt{2g}\ H^3$$

when B = breadth of notch in feet and H = height of water in feet and can be applied to any proportion of notch. When B = 2H, that is, a 90° notch, C = .595 and the formula becomes $152.4\sqrt{H^5}$,

and when B = 4H, that is, a notch containing an angle of 126° 51' 36", C = .62 and the formula is then written

$$318\sqrt{H^5}.$$

The measurements of the depth of the water above the notch should be taken by a hook-gauge, as when a rule or gauge-slate is used the velocity of the water causes the latter to rise as it comes in contact with the edge of the measuring instrument and an accurate reading is not easily obtainable, and, further, capillary attraction causes the water to rise up the rule above the actual surface, and thus to show a still greater depth. When using a hook-gauge the top of the weir, as well as the notch, should be fixed level and a peg or stake fixed as far back as possible on the upstream side of the weir, so that the top of the peg is level with the top of the weir, instead of with the notch, as is the case when a rule or gauge-slate is used. The hook-gauge consists of a square rod of, say, lin side, with a metal hook at the bottom, as shown in Fig. 15, and is so proportioned that the distance from the top of the hook to the top of the rod is equal to the difference in level of the top of the weir and the sill of the notch. In using it the rod of the hook-gauge is held against the side of the gauge-peg and lowered into the water until the point of the hook is submerged. The gauge is then gently raised until the point of the hook breaks the surface of the water, when the distance from the top of the gauge-peg to the top of the rod of the hook-gauge will correspond with the depth of the water flowing over the weir.

CHAPTER VII.

RAINFALL.

The next consideration is the amount of rain-water for which provision should be made. This depends on two factors: first, the amount of rain which may be expected to fall; and, secondly, the proportion of this rainfall which will reach the sewers. The maximum rate at which the rain-water will reach the outfall sewer will determine the size of the sewer and capacity of the pumping plant, if any, while if the sewage is to be stored during certain periods of the tide the capacity of the reservoir will depend upon the total quantity of rain-water entering it during such periods, irrespective of the rate of flow.

Some very complete and valuable investigations of the flow of rain-water in the Birmingham sewers were carried out between 1900 and 1904 by Mr. D. E. Lloyd-Davies, M.Inst.C. E., the results of which are published in Vol. CLXIV., Min Proc. Inst.C.E. He showed that the quantity reaching the sewer at any point was proportional to the time of concentration at that point and the percentage of impermeable area in the district. The time of concentration was arrived at by calculating the time which the rain-water would take to flow through the longest line of sewers from the extreme boundaries of the district to the point of observation, assuming the sewers to be flowing half full; and adding to the time so obtained the period required for the rain to get into the sewers, which varied from one minute where the roofs were connected directly with the sewers to three minutes where the rain had first to flow along the road gutters. With an average velocity of 3 ft per second the time of concentration will be thirty minutes for each mile of sewer. The total volume of rain-water passing into the sewers was found to bear the same relation to the total volume of rain falling as the maximum flow in the sewers bore to the maximum intensity of rainfall during a period equal to the time of concentration. He stated further that while the flow in the sewers was proportional to the aggregate rainfall during the time of concentration, it was also directly proportional to the impermeable area. Putting this into figures, we see that in a district where the whole area is impermeable, if a point is taken on the main sewers which is so placed that rain falling at the head of the branch sewer furthest removed takes ten minutes to reach it, then the maximum flow of storm water past that point will be approximately equal to the total quantity of rain falling over the whole drainage area during a period of ten minutes, and further, that the total quantity of rainfall reaching the sewers will approximately equal the total quantity falling. If, however, the impermeable area is 25 per cent. of the whole, then the maximum flow of storm water will be 25 per cent. of the rain falling during the time of concentration, viz., ten minutes, and the total quantity of storm water will be 25 per cent. of the total rainfall.

If the quantity of storm water is gauged throughout the year it will probably be found that, on the average, only from 70 per cent. to 80 per cent. of the rain falling on the impermeable areas will reach the sewers instead of 100 per cent., as suggested by Mr. Lloyd-Davies, the difference being accounted for by the rain which is required to wet the surfaces before any flow off can take place, in addition to the rain-water collected in tanks for domestic use, rain required to fill up gullies the water level of which has been lowered by evaporation, and rain-water absorbed in the joints of the paving.

The intensity of the rainfall decreases as the period over which the rainfall is taken is increased. For instance, a rainfall of lin may occur in a period of twenty minutes, being at the rate of 3 in per hour, but if a period of one hour is taken the fall during such lengthened time will be considerably less than 3 in In towns where automatic rain gauges are installed and records kept, the required data can be abstracted, but in other cases it is necessary to estimate the quantity of rain which may have to be dealt with.

It is impracticable to provide sewers to deal with the maximum quantity of rain which may possibly fall either in the form of waterspouts or abnormally heavy torrential rains, and the amount of risk which it is desirable to run must be settled after consideration of the details of each particular case. The following table, based principally upon observations taken at the Birmingham Observatory, shows the approximate rainfall which may be taken according to the time of concentration.

TABLE No. 7.

INTENSITY OF RAINFALL DURING LIMITED PERIODS.

Time of Concentration, period of concentration	Equivalent rate in inches per hour of aggregate rainfall during				
	A	B	C	D	E
5 minutes	1.75	2.00	3.00	—	—
10 "	1.25	1.50	2.00	—	—
15 "	1.05	1.25	1.50	—	—
20 "	0.95	1.05	1.30	1.20	3.00
25 "	0.85	0.95	1.15	—	—
30 "	0.80	0.90	1.05	1.00	2.50
35 "	0.75	0.85	0.95	—	—
40 "	0.70	0.80	0.90	—	—
45 "	0.65	0.75	0.85	—	—
1 hour	0.50	0.60	0.70	0.75	1.80
1-1/2 "	0.40	0.50	0.60	—	1.40
2 "	0.30	0.40	0.50	0.50	1.10

The figures in column A will not probably be exceeded more than once in each year, those in column B will not probably be exceeded more than once in three years, while those in column C will rarely be exceeded at all. Columns D and E refer to the records tabulated by the Meteorological Office, the rainfall given in column D being described in their publication as "falls too numerous to require insertion," and those in column E as "extreme falls rarely exceeded." It must, however, be borne in mind that the Meteorological Office figures relate to records derived from all parts of the country, and although the falls mentioned may occur at several towns in any one year it may be many years before the same towns are again visited by storms of equal magnitude.

While it is convenient to consider the quantity of rainfall for which provision is to be made in terms of the rate of fall in inches per hour, it will be useful for the practical application of the figures to know the actual rate of flow of the storm water in the sewers at the point of concentration in cubic feet per minute per acre. This information is given in the following Table No. 8, which is prepared from the figures given in Table No. 7, and is applicable in the same manner.

TABLE No. 8.

MAXIMUM FLOWS OF STORM WATER.

Time of Concentration.	Maximum storm water flow in cubic feet per min per acre of impervious area.				
	A	B	C	D	E
5 minutes	106	121	181	—	—
10 "	75	91	121	—	—
15 "	64	75	91	—	—
20 "	57	64	79	73	181
25 "	51	57	70	—	—
30 "	48	54	64	61	151
35 "	45	51	57	—	—
40 "	42	48	54	—	—
45 "	39	45	51	—	—
1 hour	30	36	42	45	109
1-1/2 "	24	30	36	—	85
2 "	18	24	30	30	67

l inch of rain = 3,630 cub. feet per acre.

The amount of rainfall for which storage has to be provided is a difficult matter to determine; it depends on the frequency and efficiency of the overflows and the length of time during which the storm water has to be held up for tidal reasons. It is found that on the average the whole of the rain on a rainy day falls within a period of 2-1/2 hours; therefore, ignoring the relief which may be afforded by overflows, if the sewers are tide-locked for a period of 2-1/2 hours or over it would appear to be necessary to provide storage for the rainfall of a whole day; but in this case again it is permissible to run a certain amount of risk, varying with the length of time the sewers are tide-locked, because, first of all, it only rains on the average on about 160 days in the year, and, secondly, when it does rain, it may not be at the time when the sewers are tide-locked, although it is frequently found that the heaviest storms occur just at the most inconvenient time, namely, about high water. Table No. 9 shows the frequency of heavy rain recorded during a period of ten years at the Birmingham Observatory, which, being in the centre of England, may be taken as an approximate average of the country.

TABLE No. 9.

FREQUENCY OF HEAVY RAIN ————————————————————————————————————-

Total Daily Rainfall. Average Frequency of Rainfall

————————————————————————————————————-

0.4 inches and over 155 times each year 0.5 " 93 " 0.6 " 68 " 0.7 " 50 " 0.8 " 33 " 0.9 " 22 " 1.0 " 17 " 1.1 " Once each year 1.2 " Once in 17 months 1.25 " " 2 years 1.3 " " 2-1/2 1.4 " " 3-1/3 1.5 " " 5 years 1.6 " " 5 years 1.7 " " 5 years 1.8 " " 10 years 1.9 " " 10 years 2.0 " " 10 years

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It will be interesting and useful to consider the records for the year 1903, which was one of the wettest years on record, and to compare those taken in Birmingham with the mean of those given in "Symons' Rainfall," taken at thirty-seven different stations distributed over the rest of the country.

TABLE No. 10.
RAINFALL FOR 1903.

	Birmingham	Mean of 37 stations in England and Wales.
Daily Rainfall of 2 in and over ……	None	1 day
Daily Rainfall of 1 in and over ……	3 days	6 days
Daily Rainfall of 1/2 in and over ….	17 days	25 days
Number of rainy days………………	177 days	211 days
Total rainfall ………………….	33.86 in	44.89 in
Amount per rainy day …………….	0.19 in	0.21 in

The year 1903 was an exceptional one, but the difference existing between the figures in the above table and the average figures in Table 9 are very marked, and serve to emphasise the necessity for close investigation in each individual case. It must be further remembered that the wettest year is not necessarily the year of the heaviest rainfalls, and it is the heavy rainfalls only which affect the design of sewerage works.

CHAPTER VIII.

STORM WATER IN SEWERS.

If the whole area of the district is not impermeable the percentage which is so must be carefully estimated, and will naturally vary in each case. The means of arriving at an estimate will also probably vary considerably according to circumstances, but the following figures, which relate to investigations recently made by the writer, may be of interest. In the town, which has a population of 10,000 and an area of 2,037 acres, the total length of roads constructed was 74,550 lineal feet, and their average width was 36 ft, including two footpaths. The average density of the population was 4.9 people per acre. Houses were erected adjoining a length of 43,784 lineal feet of roads, leaving 30,766 lineal feet, which for distinction may be called "undeveloped"—that is, the land adjoining them was not built over. Dividing the length of road occupied by houses by the total number of the inhabitants of the town, the average length of road per head was 4.37 ft, and assuming five people per house and one house on each side of the road we get ten people per two houses opposite each other. Then 10 x 4.37 = 43.7 lineal feet of road frontage to each pair of opposite houses. After a very careful inspection of the whole town, the average area of the impermeable surfaces appertaining to each house was estimated at 675 sq. ft, of which 300 sq. ft was apportioned to the front roof and garden paths and 375 sq. ft to the back roof and paved yards. Dividing these figures by 43.71 in ft of road frontage per house, we find that the effective width of the impermeable roadway is increased by 6 ft 10 in for the front portions of each house, and by a width of 8 ft 7 in, for the back portions, making a total width of 36 ft + 2(6 ft 10 in) + 2(8 ft 7 in) = 66 ft 10 in, say 67 ft On this basis the impermeable area in the town therefore equals: 43,7841 in ft x 67 ft =2,933,528; and 30,766 lin ft x 36 ft = 1,107,576.

Total, 4,041,104 sq. ft, or 92.77 acres. As the population is 10,000 the impermeable area equals 404, say, 400 sq. ft per head, or ~ (92.77 x 100) / 2037 = 4.5 per cent, of the whole area of the town.

It must be remembered that when rain continues for long periods, ground which in the ordinary way would generally be considered permeable becomes soaked and eventually becomes more or less impermeable. Mr. D. E. Lloyd-Davies, M.Inst.C.E., gives two very interesting diagrams in the paper previously referred to, which show the average percentage of effective impermeable area according to the population per acre. This information, which is applicable more to large towns, has been embodied in Fig. 16, from which it will be seen that, for storms of short duration, the proportion of impervious areas equals 5 per cent. with a population of 4.9 per acre, which is a very close approximation to the 4.5 per cent. obtained in the example just described.

Where the houses are scattered at long intervals along a road the better way to arrive at an estimate of the quantity of storm water which may be expected is to ascertain the average impervious area of, or appertaining to, each house, and divide it by five, so as to get the area per head. Then the flow off from any section of road is directly obtained from the sum of the impervious area due to the length of the road, and that due to the population distributed along it.

[Illustration: FIG. 16.—VARIATION IN AVERAGE PERCENTAGE OF
EFFECTIVE IMPERMEABLE AREA ACCORDING TO DENSITY OF POPULATION.]

In addition to being undesirable from a sanitary point of view, it is rarely economical to construct special storm water drains, but in all cases where they exist, allowance must be made for any rain that may be intercepted by them. Short branch sewers constructed for the conveyance of foul water alone are usually 9in or 12 in in diameter, not because those sizes are necessary to convey the quantity of liquid which may be expected, but because it is frequently undesirable to provide smaller public sewers, and there is generally sufficient room for the storm water without increasing the size of the sewer. If this storm water were conveyed in separate sewers the cost would be double, as two sewers would be required in the place of one. In the main sewers the difference is not so great, but generally one large sewer will be more economical than two smaller ones. Where duplicate sewers are provided and arranged, so that the storm water sewer takes the rain-water from the roads, front roofs and gardens of the houses, and the foul water sewer takes the rain-water from the back roofs and paved yards,

it was found in the case previously worked out in detail that in built-up roads a width of 36 ft + 2 (8 ft 7 in) = 53 ft 2 in, or, say, 160 sq. ft per lineal yard of road would drain to the storm water sewer, and a width of 2 (6 ft 10 in) = 13 ft 8 in, or, say, 41 sq. ft per lineal yard of road to the foul water sewer. This shows that even if the whole of the rain which falls on the impervious areas flows off, only just under 80 per cent. of it would be intercepted by the special storm water sewers. Taking an average annual rainfall of 30 in, of which 75 per cent. flows off, the quantity reaching the storm water sewer in the course of a year from each lineal

yard of road would be $\frac{30}{12} \times 160 \times \frac{75}{100} = 300$ cubic feet = 1,875 gallons.

[Illustration: FIG. 17.—SECTION OF "LEAP WEIR" OVERFLOW]

The cost of constructing a separate surface water system will vary, but may be taken at an average of, approximately, l5s. 0d. per lineal yard of road. To repay this amount in thirty years at 4 per cent, would require a sum of 10.42d., say 10-1/2d. per annum; that is to say, the cost of taking the surface water into special

sewers is $\frac{10\text{-}1/2 \text{ d.} \times 1000}{1875} = 5.6$, say 6d. per 1,000 gallons.

If the sewage has to be pumped, the extra cost of pumping by reason of the increased quantity of surface water can be looked at from two different points of view:—

1. The net cost of the gas or other fuel or electric current consumed in lifting the water.

2. The cost of the fuel consumed plus wages, stores, etc., and a proportion of the sum required to repay the capital cost of the pumping station and machinery.

The extra cost of the sewers to carry the additional quantity of storm water might also be taken into account by working out and preparing estimates for the alternative schemes.

The actual cost of the fuel may be taken at approximately 1/4 d. per 1,000 gallons. The annual works and capital charges, exclusive of fuel, should be divided by the normal quantity of sewage pumped per annum, rather than by the maximum quantity which the pumps would lift if they were able to run continuously during the whole time. For a town of about 10,000 inhabitants these charges may be taken at 1-1/4 d. per 1,000 gallons, which makes the total cost of pumping, inclusive of capital charges, 1-1/2 d. per 1,000 gallons. Even if the extra cost of enlarging the sewers is added to this sum it will still be considerably below the sum of 6 d., which represents the cost of providing a separate system for the surface water.

Unless it is permissible for the sewage to have a free outlet to the sea at all states of the tide, the provision of effective storm overflows is a matter of supreme importance. Not only is it necessary for them to be constructed in well- considered positions, but they must be effective in action. A weir constructed along one side of a manhole and parallel to the sewer is rarely efficient, as in times of storm the liquid in the sewer travels at a considerable velocity, and the greater portion of it, which should be diverted, rushes past the weir and continues to flow in the sewer; and if, as is frequently the case, it is desirable that the overflowing liquid should be screened, and vertical bars are fixed on the weir for the purpose, they block the outlet and render the overflow practically useless.

Leap weir overflows are theoretically most suitable for separating the excess flow during times of storm, but in practice they rarely prove satisfactory. This is not the fault of the system, but is, in the majority of the cases, if not all, due to defective designing. The general arrangement of a leap weir overflow is shown in Fig. 17. In normal circumstances the sewage flowing along the pipe A falls down the ramp, and thence along the sewer B; when the flow is increased during storms the sewage from A shoots out from the end of the pipe into the trough C, and thence along the storm-water sewer D. In order that it should be effective the first step is to ascertain accurately the gradient of the sewer above the proposed overflow, then, the size being known, it is easy to calculate the velocity of flow for the varying depths of sewage corresponding with minimum flow, average dry weather flow, maximum dry weather flow, and six times the dry weather flow. The natural curve which the sewage would follow in its downward path as it flowed out from the end of the sewer can then be drawn out for the various depths, taking into account the fact that the velocity at the invert and sides of the sewer is less than the average velocity of flow. The ramp should be built in accordance with the calculated curves so as to avoid splashing as far as possible, and the level of the trough C fixed so that when it is placed sufficiently far from A to allow the dry weather flow to pass down the ramp it will at the same time catch the storm water when the required dilution has taken place. Due regard must be had to the altered circumstances which will arise when the growth of population occurs, for which provision is made in the scheme, so that the overflow will remain efficient. The trough C is movable, so that the width of the leap weir may be adjusted from time to time as required. The overflow should be frequently inspected, and the accumulated rubbish removed from the trough, because sticks and similar matters brought down by the sewer will probably leap the weir instead of flowing down the ramp with the sewage. It is undesirable to fix a screen in conjunction with this overflow, but if screening is essential the operation should be carried out in a special manhole built lower down the course of the storm-water sewer. Considerable wear takes place on the ramp, which should, therefore, be constructed of blue Staffordshire or other hard bricks. The ramp should terminate in a stone block to resist the impact of the falling water, and the stones which may be brought with it, which would crack stoneware pipes if such were used.

In cases where it is not convenient to arrange a sudden drop in the invert of the sewer as is required for a leap weir overflow, the excess flow of storm-water may be diverted by an arrangement similar to that shown in Fig. 18. [Footnote: PLATE IV] In this case calculations must be made to ascertain the depth at which the sewage will flow in the pipes at the time it is diluted to the required extent; this gives the level of the lip of the diverting plate. The ordinary sewage flow will pass steadily along the invert of the sewer under the plate until it rises up to that height, when the opening becomes a submerged orifice, and its discharging capacity becomes less than when the sewage was flowing freely. This restricts the flow of the sewage, and causes it to head up on the upper side of the overflow in an endeavour to force through the orifice the same quantity as is flowing in the sewer, but as it rises the velocity carries the upper layer of the water forward up the diverting plate and thence into the storm overflow drain A deep channel is desirable, so as to govern the direction of flow at the time the overflow is in action. The diverting trough is movable, and its height above the invert can be increased easily, as may be necessary from time to time. With this arrangement the storm-water can easily be screened before it is allowed to pass out by fixing an inclined screen in the position shown in Fig. 18. [Footnote: PLATE IV] It is loose, as is the trough, and both can be lifted out when it is desired to have access to the invert of the sewer. The screen is self- cleansing, as any floating matter which may be washed against it does not stop on it and reduce its discharging capacity, but is gradually drawn down by the flow of the sewage towards the diverting plate under which it will be carried. The heavier matter in the sewage which flows along the invert will pass under the plate and be carried through to the outfall works, instead of escaping by the overflow, and perhaps creating a nuisance at that point.

CHAPTER IX.

WIND AND WINDMILLS.

In small sewerage schemes where pumping is necessary the amount expended in the wages of an attendant who must give his whole attention to the pumping station is so much in excess of the cost of power and the sum required for the repayment of the loan for the plant and buildings that it is desirable for the economical working of the scheme to curtail the wages bill as far as possible. If oil or gas engines are employed the man cannot be absent for many minutes together while the machinery is running, and when it is not running, as for instance during the night, he must be prepared to start the pumps at very short notice, should a heavy rain storm increase the flow in the sewers to such an extent that the pump well or storage tank becomes filled up. It is a simple matter to arrange floats whereby the pump may be connected to or disconnected from a running engine by means of a friction clutch, so that when the level of the sewage in the pump well reaches the highest point desired the pump may be started, and when it is lowered to a predetermined low water level the pump will stop; but it is impracticable to control the engine in the same way, so that although the floats are a useful accessory to the plant during the temporary absence of the man in charge they will not obviate his more or less constant attendance. An electric motor may be controlled by a float, but in many cases trouble is experienced with the switch gear, probably caused by its exposure to the damp air. In all cases an alarm float should be fixed, which would rise as the depth of the sewage in the pump well increased, until the top water level was reached, when the float would make an electrical contact and start a continuous ringing warning bell, which could be placed either at the pumping station or at the man's residence. On hearing the bell the man would know the pump well was full, and that he must immediately repair to the pumping-station and start the pumps, otherwise the building would be flooded. If compressed air is available a hooter could be fixed, which would be heard for a considerable distance from the station.

[Illustration: PLATE IV.

"DIVERTING PLATE" OVERFLOW.

To face page 66.]

It is apparent, therefore, that a pumping machine is wanted which will work continuously without attention, and will not waste money when there is nothing to pump. There are two sources of power in nature which might be harnessed to give this result—water and wind. The use of water on such a small scale is rarely economically practicable, as even if the water is available in the vicinity of the pumping-station, considerable work has generally to be executed at the point of supply, not only to store the water in sufficient bulk at such a level that it can be usefully employed, but also to lead it to the power-house, and then to provide for its escape after it has done its work. The power-house, with its turbines and other machinery, involves a comparatively large outlay, but if the pump can be directly driven from the turbines, so that the cost of attendance is reduced to a minimum, the system should certainly receive consideration.

Although the wind is always available in every district, it is more frequent and powerful on the coast than inland. The velocity of the wind is ever varying within wide limits, and although the records usually give the average hourly velocity, it is not constant even for one minute. Windmills of the modern type, consisting of a wheel composed of a number of short sails fixed to a steel framework upon a braced steel tower, have been used for many years for driving machinery on farms, and less frequently for pumping water for domestic use. In a very few cases it has been utilised for pumping sewage, but there is no reason why, under proper conditions, it should not be employed to a greater extent. The reliability of the wind for pumping purposes may be gauged from the figures in the following table, No. 11, which were observed in Birmingham, and comprise a period of ten years; they are arranged in order corresponding with the magnitude of the annual rainfall:—

TABLE No. 11.

MEAN HOURLY VELOCITY OF WIND

Reference Number	Rainfall for year	Number of days in year during which the mean hourly velocity of the wind was below 6 m.p.h.	10 m.p.h.	15 m.p.h.	20 m.p.h.
1...	33·86	16	88	220	314
2...	29·12	15	120	260	334
3...	28·86	39	133	263	336
4...	26·56	36	126	247	323
5...	26·51	34	149	258	330
6...	26·02	34	132	262	333
7...	25·16	33	151	276	332
8...	22·67	46	155	272	329
9...	22·30	26	130	253	337
10...	21·94	37	133	276	330
Average		31·4	131·7	250·7	330·8

It may be of interest to examine the monthly figures for the two years included in the foregoing table, which had the least and the most wind respectively, such figures being set out in the following table:

TABLE No. 12

MONTHLY ANALYSIS OF WIND

Number of days in each month during which the mean velocity of the wind was respectively below the value mentioned hereunder.

Month	Year of least wind (No. 8) 5 m.p.h.	10 m.p.h.	15 m.p.h.	20 m.p.h.	Year of most wind (No. *8*) 5 m.p.h.	10 m.p.h.	15 m.p.h.	20 m.p.h.
Jan.	5	11	23	27	3	6	15	23
Feb.	5	19	23	28	0	2	8	16
Mar.	5	10	20	23	0	1	11	18
April	6	16	23	28	1	7	16	26
May	1	14	24	30	3	11	24	31
June	1	12	22	26	1	10	21	27
July	8	18	29	31	1	12	25	29
Aug.	2	9	23	30	1	9	18	30
Sept.	1	13	25	30	1	12	24	28
Oct.	5	17	21	26	0	4	16	29
Nov.	6	11	20	26	3	7	19	28
Dec.	1	5	19	24	2	7	23	29
Total	46	155	272	329	16	88	220	314

During the year of least wind there were only eight separate occasions upon which the average hourly velocity of the wind was less than six miles per hour for two consecutive days, and on two occasions only was it less than six miles per hour on three consecutive days. It must be remembered, however, that this does not by any means imply that during such days the wind did not rise above six miles per hour, and the probability is that a mill which could be actuated by a six-mile wind would have been at work during part of the time. It will further be observed that the greatest differences between these two years occur in the figures relating to the light winds. The number of days upon which the mean hourly velocity of the wind exceeds twenty miles per hour remains fairly constant year after year.

As the greatest difficulty in connection with pumping sewage is the influx of storm water in times of rain, it will be useful to notice the rainfall at those times when the wind is at a minimum. From the following figures (Table No. 13) it will be seen that, generally speaking, when there is very little wind there is very little rain Taking the ten years enumerated in Table No. 11, we find that out of the 314 days on which the wind averaged less than six miles per hour only forty-eight of them were wet, and then the rainfall only averaged .l3 in on those days.

TABLE No. 13.

WIND LESS THAN 6 M.P.H.

Ref. No. from Table No. 11.	Total No. of days in each year.	Days on which no rain fell.	Rainy days.	Rainfall on each rainy day in inches.
1	16	14	2	.63 and .245
2	15	13	2	.02 and .02
3	39	34	5	.025, .01, .26, .02 and .03
4	36	29	7	.02, .08, .135, .10, .345, .18 and .02
5	34	28	6	.10, .43, .01, .07, .175 and .07
6	32	27	5	.10, .11, .085, .04 and .135
7	33	21	2	.415 and .70
8	46	40	6	.07, .035, .02, .06, .13 and .02
9	26	20	6	.145, .20, .33, .125, .015 & .075
10	37	30	7	.03, .23, .165, .02, .095 .045 and .02
Total	314	266	48	Average rainfall on each of the 48 days = .13 in

The greater the height of the tower which carries the mill the greater will be the amount of effective wind obtained to drive the mill, but at the same time there are practical considerations which limit the height. In America many towers are as much as 100 ft high, but ordinary workmen do not voluntarily climb to such a height, with the result that the mill is not properly oiled. About 40 ft is the usual height in this country, and 60 ft should be used as a maximum.

Mr. George Phelps, in a paper read by him in 1906 before the Association of Water Engineers, stated that it was safe to assume that on an average a fifteen miles per hour wind was available for eight hours per day, and from this he gave the following figures as representing the approximate average duty with, a lift of l00 ft, including friction:—

TABLE NO. 14 DUTY OF WINTDMILU

Diameter of Wheel.

10

12

14

16

18

20

25

30

35

The following table gives the result of tests carried out by the United States Department of Agriculture at Cheyenne, Wyo., with a l4 ft diameter windmill under differing wind velocities:—

TABLE No. 15.

POWER or l4-rx WINDMILL IN VARYING WINDS.

Velocity of Wind (miles per hour).

0—5 6-10 11-15 16-20 21-25 26-30 31-35

It will be apparent from the foregoing figures that practically the whole of the pumping for a small sewerage works may be done by means of a windmill, but it is undesirable to rely entirely upon such a system, even if two mills are erected so that the plant will be in duplicate, because there is always the possibility, although it may be remote, of a lengthened period of calm, when the sewage would accumulate; and, further, the Local Government Board would not approve the scheme unless it included an engine, driven by gas, oil, or other mechanical power, for emergencies. In the case of water supply the difficulty may be overcome by providing large storage capacity, but this cannot be done for sewage without creating an intolerable nuisance. In the latter case the storage should not be less than twelve hours dry weather flow, nor more than twenty-four. With a well-designed mill, as has already been indicated, the wind will, for the greater part of the year, be sufficient to lift the whole of the sewage and storm-water, but, if it is allowed to do so, the standby engine will deteriorate for want of use to such an extent that when urgently needed it will not be effective. It is, therefore, desirable that the attendant should run the engine at least once in every three days to keep it in working order. If it can be conveniently arranged, it is a good plan for the attendant to run the engine for a few minutes to entirely empty the pump well about six o'clock each evening. The bulk of the day's sewage will then have been delivered, and can be disposed of when it is fresh, while at the same time the whole storage capacity is available for the night flow, and any rainfall which may occur, thus reducing the chances of the man being called up during the night. About 22 per cent, of the total daily dry weather flow of sewage is delivered between 7 p.m. and 7 a.m.

The first cost of installing a small windmill is practically the same as for an equivalent gas or oil engine plant, so that the only advantage to be looked for will be in the maintenance, which in the case of a windmill is a very small matter, and the saving which may be obtained by the reduction of the amount of attendance necessary. Generally speaking, a mill 20 ft in diameter is the largest which should be used, as when this size is exceeded it will be found that the capital cost involved is incompatible with the value of the work done by the mill, as compared with that done by a modern internal combustion engine.

Mills smaller than 8 ft in diameter are rarely employed, and then only for small work, such as a 2 1/2 in pump and a 3-ft lift The efficiency of a windmill, measured by the number of square feet of annular sail area, decreases with the size of the mill, the 8 ft, 10 ft, and l2 ft mills being the most efficient sizes. When the diameter exceeds l2 ft, the efficiency rapidly falls off, because the peripheral velocity remains constant for any particular velocity or pressure of the wind, and as every foot increase in the diameter of the wheel makes an increase of over 3 ft in the length of the circumference, the greater the diameter the less the number of revolutions in any given time; and consequently the kinetic flywheel action which is so valuable in the smaller sizes is to a great extent lost in the larger mills.

Any type of pump can be used, but the greatest efficiency will be obtained by adopting a single acting pump with a short stroke, thus avoiding the liability, inherent in a long pump rod, to buckle under compression, and obviating the use of a large number of guides which absorb a large part of the power given out by the mill. Although some of the older mills in this country are of foreign origin, there are several British manufacturers turning out well-designed and strongly-built machines in large numbers. Fig. 19 represents the general appearance and Fig. 20 the details of the type of mill made by the well-known firm of Duke and Ockenden, of Ferry Wharf, Littlehampton, Sussex. This firm has erected over 400 windmills, which, after the test of time, have proved thoroughly efficient. From Fig. 20 it will be seen that the power applied by the wheel is transmitted through spur and pinion gearing of 2 1/2 ratio to a crank shaft, the gear wheel having internal annular teeth of the involute type, giving a greater number of teeth always in contact than is the case with external gears. This minimises wear, which is an important matter, as it is difficult to properly lubricate these appliances, and they are exposed to and have to work in all sorts of weather.

[Illustration: Fig. l9.—General View of Modern Windmill.]

[Illustration: Fig. 20.—Details of Windmill Manufactured by Messrs. Duke and Ockenden, Littlehampton.]

It will be seen that the strain on the crank shaft is taken by a bent crank which disposes the load centrally on the casting, and avoids an overhanging crank disc, which has been an objectionable feature in some other types. The position of the crank shaft relative to the rocker pin holes is studied to give a slow upward motion to the rocker with a more rapid downward stroke, the difference in speed being most marked in the longest stroke, where it is most required.

In order to transmit the circular internal motion a vertical connecting rod in compression is used, which permits of a simple method of changing the length of stroke by merely altering the pin in the rocking lever, the result being that the pump rod travels in a vertical line.

The governing is entirely automatic. If the pressure on the wind wheel, which it will be seen is set off the centre line of the mill and tower, exceeds that found desirable—and this can be regulated by means of a spring on the fantail—the windmill automatically turns on the turn-table and presents an ellipse to the wind instead of a circular face, thus decreasing the area exposed to the wind gradually until the wheel reaches its final position, or is hauled out of gear, when the edges only are opposed to the full force of the wind. The whole weight of the mill is taken upon a ball-bearing turn-table to facilitate instant "hunting" of the mill to the wind to enable it to take advantage of all changes of direction. The pump rod in the windmill tower is provided with a swivel coupling, enabling the mill head to turn completely round without altering the position of the rod.

CHAPTER X.

THE DESIGN OF SEE OUTFALLS.

The detail design of a sea outfall will depend upon the level of the conduit with reference to present surface of the shore, whether the beach is being eroded or made up, and, if any part of the structure is to be constructed above the level of the shore, whether it is likely to be subject to serious attack by waves in times of heavy gales. If there is probability of the direction of currents being affected by the construction of a solid structure or of any serious scour being caused, the design must be prepared accordingly.

While there are examples of outfalls constructed of glazed stoneware socketed pipes surrounded with concrete, as shown in Fig. 21, cast iron pipes are used in the majority of cases. There is considerable variation in the design of the joints for the latter class of pipes, some of which are shown in Figs. 22, 23, and 24. Spigot and socket joints (Fig. 22), with lead run in, or even with rod lead or any of the patent forms caulked in cold, are unsuitable for use below high-water mark on account of the water which will most probably be found in the trench. Pipes having plain turned and bored joints are liable to be displaced if exposed to the action of the waves, but if such joints are also flanged, as Fig. 24, or provided with lugs, as Fig. 23, great rigidity is obtained when they are bolted up; in addition to which the joints are easily made watertight. When a flange is formed all round the joint, it is necessary, in order that its thickness may be kept within reasonable limits, to provide bolts at frequent intervals. A gusset piece to stiffen the flange should be formed between each hole and the next, and the bolt holes should be arranged so that when the pipes are laid there will not be a hole at the bottom on the vertical axis of the pipe, as when the pipes are laid in a trench below water level it is not only difficult to insert the bolt, but almost impracticable to tighten up the nut afterwards. The pipes should be laid so that the two lowest bolt holes are placed equidistant on each side of the centre line, as shown in the end views of Figs. Nos. 23 and 24.

[Illustration: Fig. 2l.-Stoneware Pipe and Concrete Sea Outfall.]

With lug pipes, fewer bolts are used, and the lugs are made specially strong to withstand the strain put upon them in bolting up the pipes. These pipes are easier and quicker to joint under water than are the flanged pipes, so that their use is a distinct advantage when the hours of working are limited. In some cases gun-metal bolts are used, as they resist the action of sea water better than steel, but they add considerably to the cost of the outfall sewer, and the principal advantage appears to be that they are possibly easier to remove than iron or steel ones would be if at any time it was required to take out any pipe which may have been accidentally broken. On the other hand, there is a liability of severe corrosion of the metal taking place by reason of galvanic action between the gun-metal and the iron, set up by the sea water in which they are immersed. If the pipes are not to be covered with concrete, and are thus exposed to the action of the sea water, particular care should be taken to see that the coating by Dr. Angus Smith's process is perfectly applied to them.

[Illustration: Fig. 22.—Spigot and Socket Joint for Cast Iron Pipes.]

[Illustration: Fig. 23.—Lug Joint for Cast Iron Pipes.]

[Illustration: Fig. 24.—Turned, Board, and Flanged Joint for Cast Iron Pipes.]

Steel pipes are, on the whole, not so suitable as cast iron. They are, of course, obtainable in long lengths and are easily jointed, but their lightness compared with cast iron pipes, which is their great advantage in transport, is a disadvantage in a sea outfall, where the weight of the structure adds to its stability. The extra length of steel pipes necessitates a greater extent of trench being excavated at one time, which must be well timbered to prevent the sides falling in On the other hand, cast iron pipes are more liable to fracture by heavy stones being thrown upon them by the waves, but this is a contingency which does not frequently occur in practice. According to Trautwine, the cast iron for pipes to resist sea water should be close-grained, hard, white metal. In such metal the small quantity of contained carbon is chemically combined with the iron, but in the darker or mottled metals it is mechanically combined, and such iron soon becomes soft, like plumbago, under the influence of sea water. Hard white iron has been proved to resist sea water for forty years without deterioration, whether it is continually under water or alternately wet and dry.

Several types of sea outfalls are shown in Figs. 25 to 31.[1] In the example shown in Fig. 25 a solid rock bed occurred a short distance below the sand, which was excavated so as to allow the outfall to be constructed on the rock. Anchor bolts with clevis heads were fixed into the rock, and then, after a portion of the concrete was laid, iron bands, passing around the cast iron pipes, were fastened to the anchors. This construction would not be suitable below low-water mark. Fig. 26 represents the Aberdeen sea outfall, consisting of cast iron pipes 7 ft in diameter, which are embedded in a heavy concrete breakwater 24 ft in width, except at the extreme end, where it is 30 ft wide. The 4 in wrought iron rods are only used to the last few pipes, which were in 6 ft lengths instead of 9 ft, as were the remainder. Fig. 27 shows an inexpensive method of carrying small pipes, the slotted holes in the head of the pile allowing the pipes to be laid in a straight line, even if the pile is not driven quite true, and if the level of the latter is not correct it can be adjusted by inserting a packing piece between the cradle and the head.

Great Crosby outfall sewer into the Mersey is illustrated in Fig. 28. The piles are of greenheart, and were driven to a solid foundation. The 1 3/4 in sheeting was driven to support the sides of the excavation, and was left in when the concrete was laid. Light steel rails were laid under the sewer, in continuous lengths, on steel sleepers and to 2 ft gauge. The invert blocks were of concrete, and the pipes were made of the same material, but were reinforced with steel ribs. The Waterloo (near Liverpool) sea outfall is shown in Fig. 31.

[Footnote 1: Plate V.]

Piling may be necessary either to support the pipes or to keep them secure in their proper position, but where there is a substratum of rock the pipes may be anchored, as shown in Figs. 25 and 26. The nature of the piling to be adopted will vary according to the character of the beach. Figs. 27, 29, 30, and 31 show various types. With steel piling and bearers, as shown in Fig. 29, it is generally difficult to drive the piles with such accuracy that the bearers may be easily bolted up through the holes provided in the piles, and, if the holes are not drilled in the piles until after they are driven to their final position, considerable time is occupied, and perhaps a tide lost in the attempt to drill them below water. There is also the difficulty of tightening up the bolts when the sewer is partly below the surface of the shore, as shown. In both the types shown in Figs. 29 and 30 it is essential that the piles and the bearers should abut closely against the pipes; otherwise the shock of the waves will cause the pipes to move and hammer against the framing, and thus lead to failure of the structure.

Piles similar to Fig. 31 can only be fixed in sand, as was the case at Waterloo, because they must be absolutely true to line and level, otherwise the pipes cannot be laid in the cradles. The method of fixing these piles is described by Mr. Ben Howarth (Minutes of Proceedings of Inst.C.E., Vol. CLXXV.) as follows:—"The pile was slung vertically into position from a four-legged derrick, two legs of which were on each side of the trench; a small winch attached to one pair of the legs lifted and lowered the pile, through a block and tackle. When the pile was ready to be sunk, a 2 in iron pipe was let down the centre, and coupled to a force-pump by means of a hose; a jet of water was then forced down this pipe, driving the sand and silt away from below the pile. The pile was then rotated backwards and forwards about a quarter of a turn, by men pulling on the arms; the pile, of course, sank by its own weight, the water-jet driving the sand up through the hollow centre and into the trench, and it was always kept vertical by the sling from the derrick. As soon as the pile was down to its final level the ground was filled in round the arms, and in this running sand the pile became perfectly fast and immovable a few minutes after the sinking was completed. The whole process, from the first slinging of the pile to the final setting, did not take more than 20 or 25 minutes."

[Illustration: PLATE V.

ROCK BED. Fig. 26—ABERDEEN SEA OUTFALL. Fig. 27—SMALL GREAT CROSBY SEA OUTFALL. Fig. 29—CAST IRON PIPE ON STEEL CAST AND BEARERS. Fig. 31—WATERLOO (LIVERPOOL) SEA OUTFALL.]

(*To face page 80.*)

Screw piles may be used if the ground is suitable, but, if it is boulder clay or similar material, the best results will probably be obtained by employing rolled steel joists as piles.

CHAPTER XI.

THE ACTION OF SEA WATER ON CEMENT.

Questions are frequently raised in connection with sea-coast works as to whether any deleterious effect will result from using sea-water for mixing the concrete or from using sand and shingle off the beach; and, further, whether the concrete, after it is mixed, will withstand the action of the elements, exposed, as it will be, to air and sea-water, rain, hot sun, and frosts.

Some concrete structures have failed by decay of the material, principally between high and low water mark, and in order to ascertain the probable causes and to learn the precautions which it is necessary to take, some elaborate experiments have been carried out.

To appreciate the chemical actions which may occur, it will be as well to examine analyses of sea-water and cement. The water of the Irish Channel is composed of

Sodium chloride.................... 2.6439 per cent.
Magnesium chloride................. 0.3150 " "
Magnesium sulphate................. 0.2066 " "
Calcium sulphate................... 0.1331 " "
Potassium chloride................. 0.0746 " "
Magnesium bromide.................. 0.0070 " "
Calcium carbonate.................. 0.0047 " "
Iron carbonate..................... 0.0005 " "
Magnesium nitrate.................. 0.0002 " "
Lithium chloride................... Traces.
Ammonium chloride.................. Traces.
Silica chloride.................... Traces.
Water.............................. 96.6144
————
100.0000

An average analysis of a Thames cement may be taken to be as follows:—

Silica............................... 23.54 per cent. Insoluble residue (sand, clay, etc.)........................... 0.40 " Alumina and ferric oxide.............. 9.86 " Lime................................ 62.08 " Magnesia.............................. 1.20 " Sulphuric anhydride................... 1.08 " Carbonic anhydride and water........... 1.34 " Alkalies and loss on analysis.......... 0.50 " ——- 100.00

The following figures give the analysis of a sample of cement expressed in terms of the complex compounds that are found:—

Sodium silicate (Na_2SiO_3)........ 3.43 per cent.

Calcium sulphate ($CaSO_4$)......... 2.45 "
Dicalcium silicate (Ca_2SiO_4).... 61.89 "
Dicalcium aluminate ($Ca_2Al_2O_5$).. 12.14 "
Dicalcium ferrate ($Ca_2Fe_2O_5$)..... 4.35 "
Magnesium oxide (MgO)............ 0.97 "
Calcium oxide (CaO)............. 14.22 "
Loss on analysis, &c............. 0.55 "
——-
100.00

Dr. W. Michaelis, the German cement specialist, gave much consideration to this matter in 1906, and formed the opinion that the free lime in the Portland cement, or the lime freed in hardening, combines with the sulphuric acid of the sea-water, which causes the mortar or cement to expand, resulting in its destruction. He proposed to neutralise this action by adding to the mortar materials rich in silica, such as trass, which would combine with the lime.

Mr. J. M. O'Hara, of the Southern Pacific Laboratory, San Francisco, Cal., made a series of tests with sets of pats 4 in diameter and 1/2 in thick at the centre, tapering to a thin edge on the circumference, and also with briquettes for ascertaining the tensile strength, all of which were placed in water twenty-four hours after mixing. At first some of the pats were immersed in a "five-strength solution" of sea-water having a chemical analysis as follows:—

Sodium chloride.................... 11.5 per cent.
Magnesium chloride................. 1.4 " "
Magnesium sulphate................. 0.9 " "
Calcium sulphate................... 0.6 " "
Water.............................. 85.6 " "
100.0

This strong solution was employed in order that the probable effect of immersing the cement in sea-water might be ascertained very much quicker than could be done by observing samples actually placed in ordinary sea-water, and it is worthy of note that the various mixtures which failed in this accelerated test also subsequently failed in ordinary sea-water within a period of twelve months.

Strong solutions were next made of the individual salts contained in sea-water, and pats were immersed as before, when it was found that the magnesium sulphate present in the water acted upon the calcium hydrate in the cement, forming calcium sulphate, and leaving the magnesium hydrate free. The calcium sulphate combines with the alumina of the cement, forming calcium sulpho-aluminate, which causes swelling and cracking of the concrete, and in cements containing a high proportion of alumina, leads to total destruction of all cohesion. The magnesium hydrate has a tendency to fill the pores of the concrete so as to make it more impervious to the destructive action of the sea-water, and disintegration may be retarded or checked. A high proportion of magnesia has been found in samples of cement which have failed under the action of sea water, but the disastrous result cannot be attributed to this substance having been in excess in the original cement, as it was probably due to the deposition of the magnesia salts from the sea-water; although, if magnesia were present in the cement in large quantities, it would cause it to expand and crack, still with the small proportion in which it occurs in ordinary cements it is probably inert. The setting of cement under the action of water always frees a portion of the lime which was combined, but over twice as much is freed when the cement sets in sea-water as in fresh water. The setting qualities of cement are due to the iron and alumina combined with calcium, so that for sea-coast work it is desirable for the alumina to be replaced by iron as far as possible. The final hardening and strength of cement is due in a great degree to the tri-calcium silicate ($3CaO, SiO_2$) which is soluble by the sodium chloride found in sea-water, so that the resultant effect of the action of these two compounds is to enable the sea-water to gradually penetrate the mortar and rot the concrete. The concrete is softened, when there is an abnormal amount of sulphuric acid present, as a result of the reaction of the sulphuric acid of the salt dissolved by the water upon a part of the lime in the cement. The ferric oxide of the cement is unaffected by sea- water.

The neat cement briquette tests showed that those immersed in sea-water attained a high degree of strength at a much quicker rate than those immersed in fresh water, but the 1 to 3 cement and sand briquette tests gave an opposite result. At the end of twelve months, however, practically all the cements set in fresh water showed greater strength than those set in sea- water. When briquettes which have been immersed in fresh water and have thoroughly hardened are broken, the cores are found to be quite dry, and if briquettes immersed in sea-water show a similar dryness there need be no hesitation in using the cement; but if, on the other hand, the briquette shows that the sea-water has permeated to the interior, the cement will lose strength by rotting until it has no cohesion at all. It must be remembered that it is only necessary for the water to penetrate to a depth of 1/2 in on each side of a briquette to render it damp all through, whereas in practical work, if the water only penetrated to the same depth, very little ill-effect would be experienced, although by successive removals of a skin 1/2 in deep the structure might in time be imperilled.

The average strength in pounds per square inch of six different well-known brands of cement tested by Mr. O'Hara was as follows:—

TABLE No. 16.

EFFECT OF SEA WATER ON STRENGTH OF CEMENT.

	Neat cement set in Sea Water	Neat cement set in Fresh Water	1 cement to 3 sand set in Sea Water	1 cement to 3 sand set in Fresh Water
7 days	682	548	214	224
28 days	836	643	293	319
2 months	913	668	313	359
3 months	861	667	301	387
6 months	634	654	309	428
9 months	542	687	317	417
12 months	372	706	325	432

Some tests were also made by Messrs. Westinghouse, Church, Kerr, and Co., of New York, to ascertain the effect of sea- water on the tensile strength of cement mortar. Three sets of briquettes were made, having a minimum section of one square inch. The first were mixed with fresh water and kept in fresh water; the second were mixed with fresh water, but kept immersed in pans containing salt water; while the third were mixed with sea-water and kept in sea-water. In the experiments the proportion of cement and sand varied from 1 to 1 to 1 to 6. The results of the tests on the stronger mixtures are shown in Fig. 32.

The Scandinavian Portland cement manufacturers have in hand tests on cubes of cement mortar and cement concrete, which were started in 1896, and are to extend over a period of twenty years. A report upon the tests of the first ten years was submitted at the end of 1909 to the International Association of Testing Materials at Copenhagen, and particulars of them are published in "Cement and Sea-Water," by A. Poulsen (chairman of the committee), J. Jorsen and Co., Copenhagen, 1909, price 3s.

[Illustration: FIG. 32.—Tests of the Tensile Strength of Cement and Sand Briquettes, Showing the Effect of Sea Water.]

Cements from representative firms in different countries were obtained for use in making the blocks, which had coloured glass beads and coloured crushed glass incorporated to facilitate identification. Each block of concrete was provided with a number plate and a lifting bolt, and was kept moist for one month before being placed in position. The sand and gravel were obtained from the beach on the west coast of Jutland. The mortar blocks were mixed in the proportion of 1 to 1, 1 to 2, and 1 to 3, and were placed in various positions, some between high and low water, so as to be exposed twice in every twenty- four hours, and others below low water, so as to be always submerged. The blocks were also deposited under these conditions in various localities, the mortar ones being placed at Esbjerb at the south of Denmark, at Vardo in the Arctic Ocean, and at Degerhamm on the Baltic, where the water is only one-seventh as salt as the North Sea, while the concrete blocks were built up in the form of a breakwater or groyne at Thyboron on the west coast of Jutland. At intervals of three, six, and twelve months, and two, four, six, ten, and twenty years, some of the blocks have, or will be, taken up and subjected to chemical tests, the material being also examined to ascertain the effect of exposure upon them. The blocks tested at intervals of less than one year after being placed in position gave very variable results, and the tests were not of much value.

The mortar blocks between high and low water mark of the Arctic Ocean at Vardo suffered the worst, and only those made with the strongest mixture of cement, 1 to 1, withstood the severe frost experienced. The best results were obtained when the mortar was made compact, as such a mixture only allowed diffusion to take place so slowly that its effect was negligible; but when, on the other hand, the mortar was loose, the salts rapidly penetrated to the interior of the mass, where chemical changes took place, and caused it to disintegrate. The concrete blocks made with 1 to 3 mortar disintegrated in nearly every case, while the stronger ones remained in fairly good condition. The best results were given by concrete containing an excess of very fine sand. Mixing very finely-ground silica, or trass, with the cement proved an advantage where a weak mixture was employed, but in the other cases no benefit was observed.

The Association of German Portland Cement Manufacturers carried out a series of tests, extending over ten years, at their testing station at Gross Lichterfeld, near Berlin, the results of which were tabulated by Mr. C. Schneider and Professor Gary. In these tests the mortar blocks were made 3 in cube and the concrete blocks l2 in cube; they were deposited in two tanks, one containing fresh water and the other sea-water, so that the effect under both conditions might be noted. In addition, concrete blocks were made, allowed to remain in moist sand for three months, and were then placed in the form of a groyne in the sea between high and low-water mark. Some of the blocks were allowed to harden for twelve months in sand before being placed, and these gave better results than the others. Two brands of German Portland cement were used in these tests, one, from which the best results were obtained, containing 65.9 per cent. of lime, and the other 62.0 per cent. of lime, together with a high percentage of alumina. In this case, also, the addition of finely-ground silica, or trass, improved the resisting power of blocks made with poor mortars, but did not have any appreciable effect on the stronger mixtures.

Professor M. Möller, of Brunswick, Germany, reported to the International Association for Testing Materials, at the Copenhagen Congress previously referred to, the result of his tests on a small hollow, trapezium shape, reinforced concrete structure, which was erected in the North Sea, the interior being filled with sandy mud, which would be easily removable by flowing water. The sides were 7 cm. thick, formed of cement concrete 1:2 1/2:2, moulded elsewhere, and placed in the structure forty days after they were made, while the top and bottom were 5 cm. thick, and consisted of concrete 1:3:3, moulded *in situ* and covered by the tide within twenty-four hours of being laid. The concrete moulded *in situ* hardened a little at first, and then became soft when damp, and friable when dry, and white efflorescence appeared on the surface. In a short time the waves broke this concrete away, and exposed the reinforcement, which rusted and disappeared, with the result that in less than four years holes were made right through the concrete. The sides, which were formed of slabs allowed to harden before being placed in the structure, were unaffected except for a slight roughening of the surface after being exposed alternately to the sea and air for a period, of thirteen years. Professor Möller referred also to several cases which had come under his notice where cement mortar or concrete became soft and showed white efflorescence when it had been brought into contact with sea-water shortly after being made.

In experiments in Atlantic City samples of dry cement in powder form were put with sea-water in a vessel which was rapidly rotated for a short time, after which the cement and the sea- water were analysed, and it was found that the sea-water had taken up the lime from the cement, and the cement had absorbed the magnesia salts from the sea-water.

Some tests were carried out in 1908-9 at the Navy Yard, Charlestown, Mass., by the Aberthaw Construction Company of Boston, in conjunction with the Navy Department. The cement concrete was placed so that the lower portions of the surfaces of the specimens were always below water, the upper portions were always exposed to the air, and the middle portions were alternately exposed to each. Although the specimens were exposed to several months of winter frost as well as to the heat of the summer, no change was visible in any part of the concrete at the end of six months.

Mons. R. Feret, Chief of the Laboratory of Bridges and Roads, Boulogne-sur-Mer, France, has given expression to the following opinions:—

1. No cement or other hydraulic product has yet been found which presents absolute security against the decomposing action of sea-water.

2. The most injurious compound of sea-water is the acid of the dissolved sulphates, sulphuric acid being the principal agent in the decomposition of cement.

3. Portland cement for sea-water should be low in aluminium and as low as possible in lime.

4. Puzzolanic material is a valuable addition to cement for sea-water construction,

5. As little gypsum as possible should be added for regulating the time of setting to cements which are to be used in sea- water.

6. Sand containing a large proportion of fine grains must never be used in concrete or mortar for sea-water construction.

7. The proportions of the cement and aggregate for sea-water construction must be such as will produce a dense and impervious concrete.

On the whole, sea-water has very little chemical effect on good Portland cements, such as are now easily obtainable, and, provided the proportion of aluminates is not too high, the varying composition of the several well-known commercial cements is of little moment. For this reason tests on blocks immersed in still salt water are of very little use in determining the probable behaviour of concrete when exposed to damage by physical and mechanical means, such as occurs in practical work.

The destruction of concrete works on the sea coast is due to the alternate exposure to air and water, frost, and heat, and takes the form of cracking or scaling, the latter being the most usual when severe frosts are experienced. When concrete blocks are employed in the construction of works, they should be made as long as possible before they are required to be built in the structure, and allowed to harden in moist sand, or, if this is impracticable, the blocks should be kept in the air and thoroughly wetted each day. On placing cement or concrete blocks in sea water a white precipitate is formed on their surfaces, which shows that there is some slight chemical action, but if the mixture is dense this action is restricted to the outside, and does not harm the block.

Cement mixed with sea water takes longer to harden than if mixed with fresh water, the time varying in proportion to the amount of salinity in the water. Sand and gravel from the beach, even though dry, have their surfaces covered with saline matters, which retard the setting of the cement, even when fresh water is used, as they become mixed with such water, and thus permeate the whole mass. If sea water and aggregate from the shore are used, care must be taken to see that no decaying seaweed or other organic matter is mixed with it, as every such piece will cause a weak place in the concrete. If loam, clay, or other earthy matters from the cliffs have fallen down on to the beach, the shingle must be washed before it is used in concrete.

Exposure to damp air, such as is unavoidable on the coast, considerably retards the setting of cement, so that it is desirable that it should not be further retarded by the addition of gypsum, or calcium sulphate, especially if it is to be used with sea water or sea-washed sand and gravel. The percentage of gypsum found in cement is, however, generally considerably below the maximum allowed by the British Standard Specification, viz., 2 per cent., and is so small that, for practical purposes, it makes very little difference in sea coast work, although of course, within reasonable limits, the quicker the cement sets the better. When cement is used to joint stoneware pipe sewers near the coast, allowance must be made for this retardation of the setting, and any internal water tests which may be specified to be applied must not be made until a longer period has elapsed after the laying of the pipes than would otherwise be necessary. A high proportion of aluminates tends to cause disintegration when exposed to sea water. The most appreciable change which takes place in a good sound cement after exposure to the sea is an increase in the chlorides, while a slight increase in the magnesia and the sulphates also takes place, so that the proportion of sulphates and magnesia in the cement should be kept fairly low. Hydraulic lime exposed to the sea rapidly loses the lime and takes up magnesia and sulphates.

To summarise the information upon this point, it appears that it is better to use fresh water for all purposes, but if, for the sake of economy, saline matters are introduced into the concrete, either by using sea water for mixing or by using sand and shingle from the beach, the principal effect will be to delay the time of setting to some extent, but the ultimate strength of the concrete will probably not be seriously affected. When the concrete is placed in position the portion most liable to be destroyed is that between high and low water mark, which is alternately exposed to the action of the sea and the air, but if the concrete has a well-graded aggregate, is densely mixed, and contains not more than two parts of sand to one part of cement, no ill-effect need be anticipated.

CHAPTER XII

DIVING.

The engineer is not directly concerned with the various methods employed in constructing a sea outfall, such matters being left to the discretion of the contractor. It may, however, be briefly stated that the work frequently involves the erection of temporary steel gantries, which must be very carefully designed and solidly built if they are to escape destruction by the heavy seas. It is amazing to observe the ease with which a rough sea will twist into most fantastic shapes steel joists 10 in by 8in, or even larger in size. Any extra cost incurred in strengthening the gantries is well repaid if it avoids damage, because otherwise there is not only the expense of rebuilding the structure to be faced, but the construction of the work will be delayed possibly into another season.

In order to ensure that the works below water are constructed in a substantial manner, it is absolutely necessary that the resident engineer, at least, should be able to don a diving dress and inspect the work personally. The particular points to which attention must be given include the proper laying of the pipes, so that the spigot of one is forced home into the socket of the other, the provision and tightening up of all the bolts required to be fixed, the proper driving of the piles and fixing the bracing, the dredging of a clear space in the bed of the sea in front of the outlet pipe, and other matters dependent upon the special form of construction adopted. If a plug is inserted in the open end of the pipes as laid, the rising of the tide will press on the plugged end and be of considerable assistance in pushing the pipes home; it will therefore be necessary to re-examine the joints to see if the bolts can be tightened up any more.

Messrs. Siebe, Gorman, and Co., the well-known makers of submarine appliances, have fitted up at their works at Westminster Bridge-road, London, S.E., an experimental tank, in which engineers may make a few preliminary descents and be instructed in the art of diving; and it is distinctly more advantageous to acquire the knowledge in this way from experts than to depend solely upon the guidance of the divers engaged upon the work which the engineer desires to inspect. Only a nominal charge of one guinea for two descents is made, which sum, less out-of-pocket expenses, is remitted to the Benevolent Fund of the Institution of Civil Engineers. It is generally desirable that a complete outfit, including the air pump, should be provided for the sole use of the resident engineer, and special men should be told off to assist him in dressing and to attend to his wants while he is below water. He is then able to inspect the work while it is actually in progress, and he will not hinder or delay the divers.

It is a wise precaution to be medically examined before undertaking diving work, although, with the short time which will generally be spent below water, and the shallow depths usual in this class of work, there is practically no danger; but, generally speaking, a diver should be of good physique, not unduly stout, free from heart or lung trouble and varicose veins, and should not drink or smoke to excess. It is necessary, however, to have acquaintance with the physical principles involved, and to know what to do in emergencies. A considerable amount of useful information is given by Mr. R. H. Davis in his "Diving Manual" (Siebe, Gorman, and Co., 5s.), from which many of the following notes are taken.

A diving dress and equipment weighs about l75 lb, including a 40 lb lead weight carried by the diver on his chest, a similar weight on his back, and l6lb of lead on each boot. Upon entering the water the superfluous air in the dress is driven out through the outlet valve in the helmet by the pressure of the water on the legs and body, and by the time the top of the diver's head reaches the surface his breathing becomes laboured, because the pressure of air in his lungs equals the atmospheric pressure, while the pressure upon his chest and abdomen is greater by the weight of the water thereon.

He is thus breathing against a pressure, and if he has to breathe deeply, as during exertion, the effect becomes serious; so that the first thing he has to learn is to adjust the pressure of the spring on the outlet valve, so that the amount of air pumped in under pressure and retained in the diving dress counterbalances the pressure of the water outside, which is equal to a little under 1/2lb per square inch for every foot in depth. If the diver be 6 ft tall, and stands in an upright position, the pressure on his helmet will be about 3lb per square inch less than on his boots. The breathing is easier if the dress is kept inflated down to the abdomen, but in this case there is danger of the diver being capsized and floating feet upwards, in which position he is helpless, and the air cannot escape by the outlet valve. Air is supplied to the diver under pressure by an air pump through a flexible tube called the air pipe; and a light rope called a life line, which is used for signalling, connects the man with the surface. The descent is made by a 3 in "shot-rope," which has a heavy sinker weighing about 50 lb attached, and is previously lowered to the bottom. A 1-1/4 in rope about 15 ft long, called a "distance- line," is attached to the shot-rope about 3 ft above the sinker, and on reaching the bottom the diver takes this line with him to enable him to find his way back to the shot-rope, and thus reach the surface comfortably, instead of being hauled up by his life line. The diver must be careful in his movements that he does not fall so as suddenly to increase the depth of water in which he is immersed, because at the normal higher level the air pressure in the dress will be properly balanced against the water pressure; but if he falls, say 30 ft, the pressure of the water on his body will be increased by about 15 lb per square inch, and as the air pump cannot immediately increase the pressure in the dress to a corresponding extent, the man's body in the unresisting dress will be forced into the rigid helmet, and he will certainly be severely injured, and perhaps even killed.

When descending under water the air pressure in the dress is increased, and acts upon the outside of the drum of the ear, causing pain, until the air passing through the nose and up the Eustachian tube inside the head reaches the back of the drum and balances the pressure. This may be delayed, or prevented, if the tube is partially stopped up by reason of a cold or other cause, but the balance can generally be brought about if the diver pauses in his descent and swallows his saliva; or blocks up his nose as much as possible by pressing it against the front of the helmet, closing the mouth and then making a strong effort at expiration so as to produce temporarily an extra pressure inside the throat, and so blow open the tubes; or by yawning or going through the motions thereof. If this does not act he must come up again Provided his ears are "open," and the air pumps can keep the pressure of air equal to that of the depth of the water in which the diver may be, there is nothing to limit the rate of his descent.

Now in breathing, carbonic acid gas is exhaled, the quality varying in accordance with the amount of work done, from .014 cubic feet per minute when at rest to a maximum of about .045, and this gas must be removed by dilution with fresh air so as not to inconvenience the diver. This is not a matter of much difficulty as the proportion in fresh air is about .03 per cent., and no effect is felt until the proportion is increased to about 0.3 per cent., which causes one to breathe twice as deeply as usual; at 0.6 per cent. there is severe panting; and at a little over 1.0 per cent. unconsciousness occurs. The effect of the carbonic acid on the diver, however, increases the deeper he descends; and at a depth of 33 ft 1 per cent. of carbonic acid will have the same effect as 2 per cent. at the surface. If the diver feels bad while under water he should signal for more air, stop moving about, and rest quietly for a minute or two, when the fresh air will revive him. The volume of air required by the diver for respiration is about 1.5 cubic feet per minute, and there is a non-return valve on the air inlet, so that in the event of the air pipe being broken, or the pump failing, the air would not escape backwards, but by closing the outlet valve the diver could retain sufficient air to enable him to reach the surface.

During the time that a diver is under pressure nitrogen gas from the air is absorbed by his blood and the tissues of his body. This does not inconvenience him at the time, but when he rises the gas is given off, so that if he has been at a great depth for some considerable time, and comes up quickly, bubbles form in the blood and fill the right side of the heart with air, causing death in a few minutes. In less sudden cases the bubbles form in the brain or spinal cord, causing paralysis of the legs, which is called divers' palsy, or the only trouble which is experienced may be severe pains in the joints and muscles. It is necessary, therefore, that he shall come up by stages so as to decompress himself gradually and avoid danger. The blood can hold about twice as much gas in solution as an equal quantity of water, and when the diver is working in shallow depths, up to, say, 30 ft, the amount of nitrogen absorbed is so small that he can stop down as long as is necessary for the purposes of the work, and can come up to the surface as quickly as he likes without any danger. At greater depths approximately the first half of the upward journey may be done in one stage, and the remainder done by degrees, the longest rest being made at a few feet below the surface.

The following table shows the time limits in accordance with the latest British Admiralty practice; the time under the water being that from leaving the surface to the beginning of the ascent:—

TABLE No. l7.—DIVING DATA.

Stoppages in Total time
minutes at for ascent
Depth in feet. Time under water. different depths in minutes.

at 20 ft 10 ft

Up to 36 No limit - - 0 to 1

36 to 42 Up to 3 hours - - 1 to 1-1/2 Over 3 hours - 5 6

42 to 48 Up to 1 hour - - 1-1/2 1 to 3 hours - 5 6-1/2 Over 3 hours - 10 11-1/2

48 to 54 Up to 1/2 hour - - 2 1/2 to 1-1/2 hour - 5 7 1-1/2 to 3 hours - 10 12 Over 3 hours - 20 22

54 to 60 Up to 20 minutes - - 2 20 to 45 minutes - 5 7 3/4 to 1-1/2 hour - 10 12 1-1/2 to 3 hours 5 15 22 Over 3 hours 10 20 32

When preparing to ascend the diver must tighten the air valve in his helmet to increase his buoyancy; if the valve is closed too much to allow the excess air to escape, his ascent will at first be gradual, but the pressure of the water reduces, the air in the dress expands, making it so stiff that he cannot move his arms to reach the valve, and he is blown up, with ever-increasing velocity, to the surface. While ascending he should exercise his muscles freely during the period of waiting at each stopping place, so as to increase the circulation, and consequently the rate of deceleration.

During the progress of the works the location of the sea outfall will be clearly indicated by temporary features visible by day and lighted by night; but when completed its position must be marked in a permanent manner. The extreme end of the outfall should be indicated by a can buoy similar to that shown in Fig. 33, made by Messrs. Brown, Lenox, and Co. (Limited), Milwall, London, E., which costs about £75, including a 20 cwt. sinker and 10 fathoms of chain, and is approved for the purpose by the Board of Trade.

[Illustration: FIG 33 CAN BUOY FOR MARKING OUTFALL SEWER.]

It is not desirable to fasten the chain to any part of the outfall instead of using a sinker, because at low water the slack of the chain may become entangled, which by preventing the buoy from rising with the tide, will lead to damage; but a special pile may be driven for the purpose of securing the buoy, at such a distance from the outlet that the chain will not foul it. The buoy should be painted with alternate vertical stripes of yellow and green, and lettered "Sewer Outfall" in white letters 12 in deep.

It must be remembered that it is necessary for the plans and sections of outfall sewers and other obstructions proposed to be placed in tidal waters to be submitted to the Harbour and Fisheries Department of the Board of Trade for their approval, and no subsequent alteration in the works may be made without their consent being first obtained.

CHAPTER XIII.

THE DISCHARGE OF SEA OUTFALL SEWERS.

The head which governs the discharge of a sea outfall pipe is measured from the surface of the sewage in the tank, sewer, or reservoir at the head of the outfall to the level of the sea. As the sewage is run off the level of its surface is lowered, and at the same time the level of the sea is constantly varying as the tide rises and falls, so that the head is a variable factor, and consequently the rate of discharge varies. A curve of discharge may be plotted from calculations according to these varying conditions, but it is not necessary; and all requirements will be met if the discharges under certain stated conditions are ascertained. The most important condition, because it is the worst, is that when the level of the sea is at high water of equinoctial spring tides and the reservoir is practically empty.

Sea water has a specific gravity of 1.027, and is usually taken as weighing 64.14 lb per cubic foot, while sewage may be taken as weighing 62.45 lb per cubic foot, which is the weight of fresh water at its maximum density. Now the ratio of weight between sewage and sea water is as 1 to 1.027, so that a column of sea water l2 inches in height requires a column of fresh water 12.324, or say 12-1/3 in, to balance it; therefore, in order to ascertain the effective head producing discharge it will be necessary to add on 1/3 in for every foot in depth of the sea water over the centre of the outlet.

The sea outfall should be of such diameter that the contents of the reservoir can be emptied in the specified time—say, three hours—while the pumps are working to their greatest power in pouring sewage into the reservoir during the whole of the period; so that when the valves are closed the reservoir will be empty, and its entire capacity available for storage until the valves are again opened.

To take a concrete example, assume that the reservoir and outfall are constructed as shown in Fig. 34, and that it is required to know the diameter of outfall pipe when the reservoir holds 1,000,000 gallons and the whole of the pumps together, including any that may be laid down to cope with any increase of the population in the future, can deliver 600,000 gallons per hour. When the reservoir is full the top water level will be 43.00 O.D., but in order to have a margin for contingencies and to allow for the loss in head due to entry of sewage into the pipe, for friction in passing around bends, and for a slight reduction in discharging capacity of the pipe by reason of incrustation, it will be desirable to take the reservoir as full, but assume that the sewage is at the level 31.00. The head of water in the sea measured above the centre of the pipe will be 21 ft, so that

[*Math: $21 \times 1/3$],

or 7 in—say, 0.58 ft—must be added to the height of high water, thus reducing the effective head from 31.00 - 10.00 = 21.00 to 20.42 ft The quantity to be discharged will be

[*Math: $\frac{1,000,000 + (3 * 600,000)}{3}$]

= 933,333 gallons per hour = 15,555 gallons per minute, or, taking 6.23 gallons equal to 1 cubic foot, the quantity equals 2,497 cubic feet per min Assume the required diameter to be 30 in, then, by Hawksley's formula, the head necessary to produce velocity =

[*Math: $\frac{Gals. per min^2}{215 \times diameter in inches^4} = \frac{15,555^2}{215 * 30^4}$]

= 1.389 ft, and the head to overcome friction =

[*Math: $\frac{Gals. per min^2 \times Length in yards}{240 * diameter in inches^5} = \frac{15,555^2 * 2042}{240 * 30^5}]

= 84.719. Then 1.389 + 84.719 = 86.108—say, 86.11 ft; but the acutal head is 20.42 ft, and the flow varies approximately as the square root of the head, so that the true flow will be about

[*Math: $15,555 * \sqrt{\frac{20.42}{86.11} = 7574.8$]

[Illustration: FIG 34 DIAGRAM ILLUSTRATING CALCULATIONS FOR THE
DISCHARGE OF SEA OUTFALLS]

—say 7,575 gallons. But a flow of 15,555 gallons per minute is required, as it varies approximately as the fifth power of the diameter, the requisite diameter will be about

[*Math: \sqrt[5]{\frac{30^5 \times 15,555}{7575}] = 34.64 inches.

Now assume a diameter of 40 in, and repeat the calculations.
Then head necessary to produce velocity

[*Math: = \frac{15,555^2}{215 \times 40^4}] = 0.044 ft, and head to overcome friction =

[*Math: \frac{15,555^2 \times 2042}{240 \times 40^5}]

= 20.104 ft Then 0.044 + 20.104 = 20.148, say 20.15 ft, and the true flow will therefore be about

[*Math: 15,555 * \sqrt{\frac{20.42}{20.15}}]

= 15,659 gallons, and the requisite diameter about

[*Math: \sqrt[5]{\frac{40^5 * 15,555}{15,659}}]

= 39.94 inches.

When, therefore, a 30 in diameter pipe is assumed, a diameter of 34.64 in is shown to be required, and when 40 in is assumed 39.94 in is indicated.

Let *a* = difference between the two assumed diameters. *b* = increase found over lower diameter. *c* = decrease found under greater diameter. *d* = lower assumed diameter.

Then true diameter =

[*Math: d + \frac{ab}{b+c} = 30 + \frac{10 \times 4.64}{4.64+0.06} = 30 + \frac{46.4}{4.7} = 39.872],

or, say, 40 in, which equals the required diameter.

A simpler way of arriving at the size would be to calculate it by Santo Crimp's formula for sewer discharge, namely, velocity in feet per second =

[*Math: 124 \sqrt[3]{R^2} \sqrt{S}],

where R equals hydraulic mean depth in feet, and S = the ratio of fall to length; the fall being taken as the difference in level between the sewage and the sea after allowance has been made for the differing densities. In this case the fall is 20.42 ft in a length of 6,126 ft, which gives a gradient of 1 in 300. The hydraulic mean depth equals

[*Math: \frac{d}{4}];

the required discharge, 2,497 cubic feet per min, equals the area,

[*Math: (\frac{\pi d^2}{4})]

multiplied by the velocity, therefore the velocity in feet per second = 4/(pi d^2) x 2497/60 = 2497/(15 pi d^2) and the formula then becomes

2497/(15 pi d^2) = 124 x * 3rd_root(d^2)/3rd_root(4^3*) x sqrt(1)/sqrt(300)

or d^2 x 3rd_root(d^2) = 3rd_root(d^6) = (2497 x 3rd_root(16) x sqrt(300)) / (124 x 15 x 3.14159*)

or (8 x log d)/3 = log 2497 + (1/3 x log 16) + (* x log 300) - log 124 - log 15 - log 3.14159;

or log d = 3/8 (3.397419 + 0.401373 + 1.238561 - 2.093422 - 1.176091 - 0.497150) = 3/8 (1.270690) = 0.476509.

* d = 2.9958* feet = 35.9496, say 36 inches.

As it happens, this could have been obtained direct from the tables where the discharge of a 36 in pipe at a gradient of 1 in 300 = 2,506 cubic feet per minute, as against 2,497 cubic feet required, but the above shows the method of working when the figures in the tables do not agree with those relating to the particular case in hand.

This result differs somewhat from the one previously obtained, but there remains a third method, which we can now make trial of—namely, Saph and Schoder's formula for the discharge of water mains, V = 174 3rd_root(R^2) x S^.51*. Substituting values similar to those taken previously, this formula can be written

2497/(15 pi d^2) = 174 x 3rd_root(d_2)/3rd_root(4^2) x 1^.51/300^.51

or d^2 x 3rd_root(d^2) = 3rd_root(d^6) = (2497 x 3rd_root(16) x 300^.51) / (174 x 15 x 3.14159)

or* log d = 3/8 (3.397419 + 0.401373 + (54 x 2.477121) - 2.240549 - 1.176091 - 0.497150) = 3/8 (1.222647) = 0.458493

* d = 2.874* feet = 34.388 say 34 1/2 inches.

By Neville's general formula the velocity in feet per second = 140 SQRT(RS)-11(RS)^(1/3) or, assuming a diameter of 37 inches,

V = 140 X SQRT(37/(12 x 4) x 1/300) - 11 (37/(12x4x300))^(1/3)

= 140 x SQRT(37/14400) - 11 (37/1440)^(1/3)

= 7.09660 - 1.50656 = 5.59 feet per second.

Discharge = area x velocity; therefore, the discharge in cubic feet per minute

= 5.59 x 60 x (3.14159 x 37^2)/(4*12^2) = 2504 compared with

2,497 c.f.m, required, showing that if this formula is used the pipe should be 37 in diameter.

The four formulæ, therefore, give different results, as follows:—

Hawksley = 40 in
Neville = 37 in
Santo Crimp = 36 in
Saph and Schoder = 34-1/2 in

The circumstances of the case would probably be met by constructing the outfall 36 in in diameter.

It is very rarely desirable to fix a flap-valve at the end of a sea outfall pipe, as it forms a serious obstruction to the flow of the sewage, amounting, in one case the writer investigated, to a loss of eight-ninths of the available head; the head was exceptionally small, and the flap valve practically absorbed it all. The only advantage in using a flap valve occurs when the pipe is directly connected with a tank sewer below the level of high water, in which case, if the sea water were allowed to enter, it would not only occupy space required for storing sewage, but it would act on the sewage and speedily start decomposition, with the consequent emission of objectionable odours. If there is any probability of sand drifting over the mouth of the outfall pipe, the latter will keep free much better if there is no valve. Schemes have been suggested in which it was proposed to utilise a flap valve on the outlet so as to render the discharge of the sewage automatic. That is to say, the sewage was proposed to be collected in a reservoir at the head of, and directly connected to, the outfall pipe, at the outlet end of which a flap valve was to be fixed. During high water the mouth of the outfall would be closed, so that sewage would collect in the pipes, and in the reservoir beyond; then when the tide had fallen such a distance that its level was below the level of the sewage, the flap valve would open, and the sewage flow out until the tide rose and closed the valve. There are several objections to this arrangement. First of all, a flap valve under such conditions would not remain watertight, unless it were attended to almost every day, which is, of course, impracticable when the outlet is below water. As the valve would open when the sea fell to a certain level and remain open during the time it was below that level, the period of discharge would vary from, say, two hours at neap tides to about four hours at springs; and if the two hours were sufficient, the four hours would be unnecessary. Then the sewage would not only be running out and hanging about during dead water at low tide, but before that time it would be carried in one direction, and after that time in the other direction; so that it would be spread out in all quarters around the outfall, instead of being carried direct out to sea beyond chance of return, as would be the case in a well- designed scheme.

When opening the valve in the reservoir, or other chamber, to allow the sewage to flow through the outfall pipe, care should be taken to open it at a slow rate so as to prevent damage by concussion when the escaping sewage meets the sea water standing in the lower portion of the pipes. When there is considerable difference of level between the reservoir and the sea, and the valve is opened somewhat quickly, the sewage as it enters the sea will create a "water-spout," which may reach to a considerable height, and which draws undesirable attention to the fact that the sewage is then being turned into the sea.

Chapter XIV

TRIGONOMETRICAL SURVEYING.

In the surveying work necessary to fix the positions of the various stations, and of the float, a few elementary trigonometrical problems are involved which can be advantageously explained by taking practical examples.

Having selected the main station A, as shown in Fig. 35, and measured the length of any line A B on a convenient piece of level ground, the next step will be to fix its position upon the plan. Two prominent landmarks, C and D, such as church steeples, flag-staffs, etc., the positions of which are shown upon the ordnance map, are selected and the angles read from each of the stations A and B. Assume the line A B measures ll7 ft, and the angular measurements reading from zero on that line are, from A to point C, 29° 23' and to point D 88° 43', and from B to point C 212° 43', and to point D 272° 18' 30". The actual readings can be noted, and then the arrangement of the lines and angles sketched out as shown in Fig. 35, from which it will be necessary to find the lengths AC and AD. As the three angles of a triangle equal 180°, the angle B C A = 180°- 147° 17'-29° 23'= 3° 20', the angle B D A = 180°-87° 41' 30"- 88° 43'= 3° 35' 30". In any triangle the sides are proportionate to the sines of the opposite angles, and vice versa; therefore,

A B : A C :: sin B C A : sin A B C, or sin B C A : A B :: sin ABC : A C, nr A C = (A B sin A B C) / (sin B C A) = (117 x sin 147° 17') / (sin 3° 20')

or log A C = log 117 + L sin 147° 17' - L sin 3° 20'.

The sine of an angle is equal to the sine of its supplement, so that sin 147° 17' = sin 32° 43', whence log A C = 2.0681859 + 9.7327837-8.7645111 = 3.0364585

Therefore A C = 1087.6 feet.

Similarly sin B D A: A B :: sin A B D: A D

$$\text{therefore } A\,D = \frac{A\,B \sin A\,B\,D}{\sin B\,D\,A} = \frac{117 \times \sin 87^\circ\, 41'\, 30''}{\sin 3^\circ\, 35'\, 30''}$$

whence log A D = log ll7 + L sin 87° 41' 30" - L sin 3° 35' 30"
= 2.0681859 + 9.99964745 - 8.79688775
= 3.2709456

Therefore AD = 1866.15 feet.

The length of two of the sides and all three angles of each of the two triangles A C B and A D B are now known, so that the triangles can be drawn upon the base A B by setting off the sides at the known angles, and the draughtsmanship can be checked by measuring the other known side of each triangle. The points C and D will then represent the positions of the two landmarks to which the observations were taken, and if the triangles are drawn upon a piece of tracing paper, and then superimposed upon the ordnance map so that the points C and D correspond with the landmarks, the points A and B can be pricked through on to the map, and the base line A B drawn in its correct position.

If it is desired to draw the base line on the map direct from the two known points, it will be necessary to ascertain the magnitude of the angle A D C. Now, in any triangle the tangent of half the difference of two angles is to the tangent of half their sum as the difference of the two opposite sides is to their sum; that is:—

Tan 1/2 (ACD - ADC): tan 1/2 (ACD + ADC)::
AD - AC : AD + AC,

but ACD + ADC = l80° - CAD = 120° 40',
therefore, tan 1/2 (ACD - ADC): tan 1/2 (120° 40')::
(1866.15 - 1087.6): (1866.15 + 1087.6),

$$\text{therefore, } \tan 1/2\,(ACD - ADC) = \frac{778.55 \tan 60° 20'}{2953.75}$$

or L tan 1/2 (ACD - ADC) = log 778.55 + L tan 60° 20'
- log 2953.75 .
= 2.8912865 + 10.2444l54 - 3.4703738
= 9.6653281 .. 1/2 (ACD - ADC) = 24° 49' 53"
.. ACD - ADC = 49° 39' 46". Then algebraically

$$ADC = \frac{(ACD + ADC) - (ACD - ADC)}{2}$$

$$.. ADC = \frac{120° 40' - 49° 39' 46"}{2} = \frac{71° 0' 14"}{2} = 35° 30' 7",$$

ACD = 180° - 35° 30' 7" - 59° 20' = 85° 9' 53".

[Illustration: Fig. 35.—Arrangement of lines and Angles
Showing Theodolite Readings and Dimensions.]

Now join up points C and D on the plan, and from point D set off the line D A, making an angle of 35° 30' 7" with C D, and having a length of l866.15 ft, and from point C set off the angle A C D equal to 85° 9' 53". Then the line A C should measure l087.6 ft long, and meet the line A D at the point A, making an angle of 59° 20'. From point A draw a line A B, ll7 ft long, making an angle of 29° 23' with the line A C; join B C, then the angle ABC should measure 147° 17', and the angle B C A 3° 20'. If the lines and angles are accurately drawn, which can be proved by checking as indicated, the line A B will represent the base line in its correct position on the plan.

The positions of the other stations can be calculated from the readings of the angles taken from such stations. Take stations E, F, G, and H as shown in Fig. 36*, the angles which are observed being marked with an arc.

It will be observed that two of the angles of each triangle are recorded, so that the third is always known. The full lines represent those sides, the lengths of which are calculated, so that the dimensions of two sides and the three angles of each triangle are known. Starting with station E,

Sin A E D: A D:: sin D A E: D E

A D sin D A E

D E = ———————
sin A E D

or $\log DE = \log AD + L \sin DAE - L \sin AED$.

From station F, E and G are visible, but the landmark D cannot be seen; therefore, as the latter can be seen from G, it will be necessary to fix the position of G first. Then,

$\sin EGD : DE :: \sin EDG : EG$,

$$\text{or } EG = \frac{DE \sin EDG}{\sin EGD}$$

Now, $\sin EFG : EG :: \sin FEG : FG$

$$FG = \frac{EG \sin FEG}{\sin EFG}$$

thus allowing the position of F to be fixed, and then

$\sin FHG : FG :: \sin FGH : FH$

$$FH = \frac{FG \sin FGH}{\sin FHG}$$

[Illustration: FIG 36.—DIAGRAM ILLUSTRATING TRIGONOMETRICAL SURVEY OF OBSERVATION STATIONS.]

In triangles such as E F G and F G H all three angles can be directly read, so that any inaccuracy in the readings is at once apparent. The station H and further stations along the coast being: out of sight of landmark D, it will be as well to connect the survey up with another landmark K, which can be utilised in the forward work; the line K H being equal to

$$\frac{FH \sin KFH}{\sin FKH}$$

The distance between C and D in Fig. 35 is calculated in a similar manner, because $\sin ACD : AD :: \sin CAD : CD$,

$$\text{or } CD = \frac{AD \sin CAD}{\sin SCD} = \frac{1866.15 \sin 59° 20'}{\sin 85° 9' 53''}$$

or $\log CD = \log 1866.15 + L \sin 59° 20' - L \sin 85° 9' 53''$

$= 3.2709456 + 9.9345738 - 9.9984516$

$= 3.2070678$. ' . $CD = 1610.90$ ft

The distance between any two positions of the float can be obtained by calculation in a similar way to that in which the length C D was obtained, but this is a lengthy process, and is not necessary in practical work. It is desirable, of course, that the positions of all the stations be fixed with the greatest accuracy and plotted on the map, then the position of the float can be located with sufficient correctness, if the lines of sight obtained from the angles read with the theodolites are plotted, and their point of intersection marked on the plan. The distance between any two positions of the float can be scaled from the plan.

The reason why close measurement is unnecessary in connection with the positions of the float is that it represents a single point, whereas the sewage escaping with considerable velocity from the outfall sewer spreads itself over a wide expanse of sea in front of the outlet, and thus has a tangible area. The velocity of any current is greatest in the centre, and reduces as the distance from the centre increases, until the edges of the current are lost in comparative still water; so that observations taken of the course of one particle, such as the float represents, only approximately indicate the travel of the sewage through the sea. Another point to bear in mind is that the dilution of the sewage in the sea is so great that it is generally only by reason of the unbroken fæcal, or other matter, that it can be traced for any considerable distance beyond the outfall. It is unlikely that such matters would reach the outlet, except in a very finely divided state, when they would be rapidly acted upon by the sea water, which is a strong oxidising agent.

CHAPTER XV.

HYDROGRAPHICAL SURVEYING.

Hydrographical surveying is that branch of surveying which deals with the complete preparation of charts, the survey of coast lines, currents, soundings, etc., and it is applied in connection with the sewerage of sea coast towns when it is necessary to determine the course of the currents, or a float, by observations taken from a boat to fixed points on shore, the boat closely following the float. It has already been pointed out that it is preferable to take the observations from the shore rather than the boat, but circumstances may arise which render it necessary to adopt the latter course.

In the simplest case the position of the boat may be found by taking the compass bearings of two known objects on shore. For example, A and B in Fig. 37 may represent the positions of two prominent objects whose position is marked upon an ordnance map of the neighbourhood, or they may be flagstaffs specially set up and noted on the map; and let C represent the boat from which the bearings of A and B are taken by a prismatic compass, which is marked from 0 to 360°. Let the magnetic variation be N. 15° W., and the observed bearings A 290, B 320, then the position stands as in Fig. 38, or, correcting for magnetic variation, as in Fig. 39, from which it will be seen that the true bearing of C from A will be 275-180=95° East of North, or 5° below the horizontal, and the true bearing of C from B will be 305-180=120° East of North, or 35° below the horizontal. These directions being plotted will give the position of C by their intersection. Fig. 40 shows the prismatic compass in plan and section. It consists practically of an ordinary compass box with a prism and sight-hole at one side, and a corresponding sight-vane on the opposite side. When being used it is held horizontally in the left hand with the prism turned up in the position shown, and the sight-vane raised. When looking through the sight-hole the face of the compass-card can be seen by reflection from the back of the prism, and at the same time the direction of any required point may be sighted with the wire in the opposite sight vane, so that the bearing of the line between the boat and the required point may be read. If necessary, the compass-card may be steadied by pressing the stop at the base of the sight vane. In recording the bearings allowance must in all cases be made for the magnetic pole. The magnetic variation for the year 1910 was about l5 1/2° West of North, and it is moving nearer to true North at the rate of about seven minutes per annum.

[Illustration: FIG. 37.—POSITION OF BOAT FOUND BY COMPASS BEARINGS.]

[Illustration: FIG. 38.—REDUCTION OF BEARINGS TO MAGNETIC NORTH.]

[Illustration: FIG. 39.—REDUCTION OF BEARINGS TO TRUE NORTH.]

There are three of Euclid's propositions that bear very closely upon the problems involved in locating the position of a floating object with regard to the coast, by observations taken from the object. They are Euclid I. (32), "The three interior angles of every triangle are together equal to two right angles"; Euclid III. (20),

"The angle at the centre of a circle is double that of the angle at the circumference upon the same base—that is, upon the same part of the circumference,"

or in other words, on a given chord the angle subtended by it at the centre of the circle is double the angle subtended by it at the circumference; and Euclid III. (21),

"The angles in the same segment of a circle are equal to one another."

[Illustration: Fig. 40.—Section and Plan of Prismatic Compass.]

Having regard to this last proposition (Euclid III., 21), it will be observed that in the case of Fig. 37 it would not have been possible to locate the point C by reading the angle A C B alone, as such point might be amywhere on the circumference of a circle of which A B was the chord. The usual and more accurate method of determining the position of a floating object from the object, itself, or from a boat alongside, is by taking angles with a sextant, or box-sextant, between three fixed points on shore in two operations. Let A B C, Fig. 41, be the three fixed points on shore, the positions of which are measured and recorded upon an ordnance map, or checked if they are already there. Let D be the floating object, the position of which is required to be located, and let the observed angles from the object be A D B 30° and B D C 45°. Then on the map join A B and B C, from A and B set off angles = 90 - 30 = 60°, and they will intersect at point E, which will be the centre of a circle, which must be drawn, with radius E A. The circle will pass through A B, and the point D will be somewhere on its circumference. Then from B and C set off angles = 90-45 = 45°, which will intersect at point F, which will be the centre of a circle of radius F B, which will pass through points B C, and point D will be somewhere on the circumference of this circle also; therefore the intersection of the two circles at D fixes that point on the map. It will be observed that the three interior angles in the triangle A B E are together equal to two right angles (Euclid I. 32), therefore the angle A E B = 180 - 2 x (90 - 30) = 600, so that the angle A E B is double the angle A D B (Euclid III., 20), and that as the angles subtending a given chord from any point of the circumference are equal (Euclid III, 21), the point that is common to the two circumferences is the required point. When point D is inked in, the construction lines are rubbed out ready for plotting the observations from the next position. When the floating point is out of range of A, a new fixed point will be required on shore beyond C, so that B, C, and the new point will be used together. Another approximate method which may sometimes be employed is to take a point on a piece of tracing paper and draw from it three lines of unlimited length, which shall form the two observed angles. If, now, this piece of paper is moved about on top of the ordnance map until each of the three lines passes through the corresponding fixed points on shore, then the point from which the lines radiate will represent the position of the boat.

[Illustration: Fig. 41. Geometrical Diagram for Locating
Observation Point Afloat.]

The general appearance of a box-sextant is as shown in Fig. 42, and an enlarged diagrammatic plan of it is shown in Fig. 43. It is about 3 in in diameter, and is made with or without the telescope; it is used for measuring approximately the angle between any two lines by observing poles at their extremities from the point of intersection. In Fig. 43, A is the sight- hole, B is a fixed mirror having one-half silvered and the other half plain; C is a mirror attached to the same pivot as the vernier arm D. The side of the case is open to admit rays of light from the observed objects. In making an observation of the angle formed by lines to two poles, one pole would be seen through the clear part of mirror B, and at the same time rays of light from the other pole would fall on to mirror C, which should be moved until the pole is reflected on the silvered part of mirror B, exactly in line, vertically, with the pole seen by direct vision, then the angle between the two poles would be indicated on the vernier. Take the case of a single pole, then the angle indicated should be zero, but whether it would actually be so depends upon circumstances which may be explained as follows: Suppose the pole to be fixed at E, which is extremely close, it will be found that the arrow on the vernier arm falls short of the zero of the scale owing to what may be called the width of the base line of the instrument. If the pole is placed farther off, as at F, the rays of light from the pole will take the course of the stroke-and-dot line, and the vernier arm will require to be shifted nearer the zero of the scale. After a distance of two chains between the pole and sextant is reached, the rays of light from the pole to B and C are so nearly parallel that the error is under one minute, and the instrument can be used under such conditions without difficulty occurring by reason of error. To adjust the box-sextant the smoked glass slide should be drawn over the eyepiece, and then, if the sun is sighted, it should appear as a perfect sphere when the vernier is at zero, in whatever position the sextant may be held. When reading the angle formed by the lines from two stations, the nearer station should be sighted through the plain glass, which may necessitate holding the instrument upside down. When the angle to be read between two stations exceeds 90°, an intermediate station should be fixed, and the angle taken in two parts, as in viewing large angles the mirror C is turned round to such an extent that its own reflection, and that of the image upon it, is viewed almost edgeways in the mirror B.

[Illustration: Fig. 42.—Box-Sextant.]

It should be noted that the box-sextant only reads angles in the plane of the instrument, so that if one object sighted is lower than the other, the angle read will be the direct angle between them, and not the horizontal angle, as given by a theodolite.

The same principles may be adopted for locating the position of an object in the water when the observations have to be taken at some distance from it. To illustrate this, use may be made of an examination question in hydrographical surveying given at the Royal Naval College, Incidentally, it shows one method of recording the observations. The question was as follows:—

[Illustration: Fig. 43.—Diagram Showing Principle of Box-Sextant]

"From Coastguard, Mound bore N. 77° W. (true) 0.45 of a mile, and Mill bore, N. 88° E, 0.56 of a mile, the following stations were taken to fix a shoal on which the sea breaks too heavily to risk the boat near:—

Mound 60° C.G. 47° Mill.
[Greek: phi]
Centre of shoal
Mound 55° C.G. 57° 30' Mill.
[Greek: phi]
Centre of shoal.

Project the positions on a scale of 5 in = a mile, giving the centre of the shoal." It should be noted that the sign [Greek: phi] signifies stations in one line or "in transit," and C G indicates coastguard station. The order of lettering in Fig. 44 shows the order of working.

[Illustration: Fig. 44.—Method of Locating Point in Water When Observations Have to Be Taken Beyond It.]

The base lines A B and A C are set out from the lengths and directions given; then, when the boat at D is "in transit" with the centre of the shoal and the coastguard station, the angle formed at D by lines from that point to B and A is 60°, and the angle formed by lines to A and C is 47°. If angles of 90° - 60° are set up at A and B, their intersection at E will, as has already been explained, give the centre of a circle which will pass through points A, B, and D. Similarly, by setting up angles of 90°-47° at A and C, a circle is found which will pass through A C and D. The intersection of these circles gives the position of the boat D, and it is known that the shoal is situated somewhere in the straight line from D to A. The boat was then moved to G, so as to be "in transit" with the centre of the shoal and the mound, and the angle B G A was found to be 55°, and the angle A G C 57° 30'. By a similar construction to that just described, the intersection of the circles will give the position of G, and as the shoal is situated somewhere in the line G B and also in the line A D, the intersection of these two lines at K will give its exact position.