The TOTAL HEAD of a given particle of water is found by adding together the following quantities:—

The *head of pressure*, or intensity of the pressure exerted by the particle, expressed in feet of water.
The *head of elevation*, or actual height of the particle above some fixed or "*datum*" level.

In stating the pressure or head of a particle of water, it is usual *not* to include the *atmospheric pressure*, so that the *absolute* or true pressure exceeds the pressure as stated in the customary way by one atmosphere. When the absolute pressure is exactly one atmosphere, the pressure as stated in the customary way is *nothing;* when the absolute falls short of the atmospheric pressure by so many lbs. on the square inch, or so many feet of water, the customary mode of stating that fact is to say that there are so many lbs. on the square inch, or so many feet, of *vacuum*.

The atmospheric pressure, at the level of the sea, varies from about 32 to 35 feet of water, and diminishes at the rate nearly of 1-100th part of itself for each 262 feet of elevation above that level.

444. **Volume and Mean Velocity of Flow.**—The *volume of flow* or *discharge* of a stream of water is expressed in units of volume per unit of time.

The most convenient unit of volume is the *cubic foot;* but in calculations relating to the water supply of towns it is customary to use the *gallon*.

The following is the relation between those units:—

One gallon = 0·1604 cubic foot (being 10 lbs. of water); and
One cubic foot = 6·2355 gallons;

but in ordinary calculations respecting water-works it is sufficiently accurate to make one gallon = 0·16 cubic foot, and one cubic foot = $6\frac{1}{4}$ gallons.

Of different *units of time*, the *second* is the most convenient in mechanical calculations; the *minute* is the customary unit in stating the discharge of streams; the *hour*, the *day*, and longer periods are used in calculations as to drainage and water supply.

The variety of *units of discharge* is thus very great. The *cubic foot per second* is the most convenient in mechanical calculations.

The *mean velocity* of a stream at a given cross-section is found by dividing the discharge, or volume of flow, by the area of the cross-section, and is most conveniently expressed in feet per second.

445. **Greatest and Least Velocities.**—Inasmuch as every stream of fluid that flows in a channel is retarded by friction against the

material of the channel, the velocity of the fluid particles is different at different points of the same cross-section, being greatest in the centre and least at the border. In open channels, like those of rivers, the ratio of the mean velocity to the greatest or central velocity is given approximately by the following formula of Prony :—

$$\frac{\text{mean velocity}}{\text{greatest velocity}} = \frac{\text{greatest velocity} + 7{\cdot}71 \text{ feet per second}}{\text{greatest velocity} + 10{\cdot}28 \text{ feet per second}}. \quad (1.)$$

The least velocity, or that of the particles in contact with the bed, is about as much less than the mean velocity as the greatest velocity is greater than the mean. In ordinary cases, the least, mean, and greatest velocities may be taken as bearing to each other nearly the proportions of 3, 4, and 5. In very slow currents they are nearly as 2, 3, and 4.

446. **General Principles of Steady Flow.**—The *steady* motion of a mass of fluid, as distinguished from unsteady motion, means that kind of motion in which the velocity and direction of motion of a particle depend on its *position* alone, and not jointly on position and time; so that each particle of the series of particles which successively come to a given point, assumes a certain velocity and direction of motion proper to that point. It is, in short, the motion of a *permanent current*, as distinguished from that of a varying current, or that of a wave.

In order to acquire velocity from a state of rest, or an increase of velocity, a fluid particle must pass from a place of *greater total head* to a place of *less total head.* This it may do either by actual descent from a higher to a lower level, or by passing from a place of more intense pressure to a place of less intense pressure, or by both those changes combined. The *loss of head* thus incurred is connected with the velocity produced by the following laws :—

I. In a liquid without friction the loss of head in producing a given increase of velocity is equal to the height of vertical fall which would produce the same increase of velocity in a body falling freely; in other words, the loss of head is equal to the *height due to the acceleration;* and if the particle starts from a state of rest, that height is called the *height due to the velocity*, and is given by the following formula, where v is the velocity in feet per second :—

$$\text{height in feet} = v^2 \div 64{\cdot}4 \ldots\ldots\ldots\ldots\ldots\ldots (1.)$$

II. If the motion of the liquid is impeded by friction, there is an additional loss of head, bearing to the height due to the velocity of flow a certain proportion, depending on the figure and dimensions of the channel and openings traversed by the stream, and other circumstances.

The combination of those two principles may be thus expressed: Let h denote the *loss of head*, in feet; then

$$h = (1 + F) \frac{v^2}{64 \cdot 4}; \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (2.)$$

in which F is a factor, determined by experiment, expressing the proportion which the loss of head by friction bears to the height due to the velocity.

The inverse formula, for finding the velocity from the loss of head, is as follows:—

$$v = 8 \cdot 025 \sqrt{\frac{h}{1 + F}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (3.)$$

The velocity computed from a given height, on the supposition that there is no friction, by the formula $v = 8 \cdot 025 \sqrt{h}$, is sometimes called the "theoretical velocity."

In an open channel the loss of head h consists wholly in diminution of the "head of elevation," and is the *actual fall* of the upper surface of the stream. In a close pipe it may consist wholly or partly of diminution of the "head of pressure," and is then called *virtual fall.* To express this in symbols,

Let z_1 denote the elevation above a fixed datum, and
p_1, the head of pressure at a point in the reservoir from which a pipe is supplied, the velocity at that point being insensible, so that
$z_1 + p_1$ is the *total head in still water;* also let
z denote the elevation above the datum, and
p, the head of pressure at a given point in the pipe, at which the loss of head, as computed by equation 2, is h; then the total head at this point is,

$$z + p = z_1 + p_1 - h; \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (4.)$$

and the pressure, in feet of water, is

$$p = z_1 + p_1 - z - h. \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (5.)$$

The pressure of flowing water, as thus diminished by loss of head, is called HYDRAULIC PRESSURE, to distinguish it from the pressure of still water, called *hydrostatic pressure.*

In an open channel, equation 5 is simplified by the fact that for the upper surface of the stream, and all surfaces parallel to it, h is simply $= z_1 - z$; so that $p = p_1$, if the two points are at equal depths below the surface.

If the water has a sensible velocity of flow *at the starting point*, the loss of head required is diminished to the extent of the height due to that *velocity of approach*, as it is called. Thus, let v_0 be the velocity of approach; then, instead of equation 2, we must use the following:—

$$h = (1 + \mathrm{F})\frac{v^2}{64 \cdot 4} - \frac{v_0^2}{64 \cdot 4}; \dots\dots\dots\dots\dots (6.)$$

and if v_0 bears *a known ratio* to v, let that ratio be $v_0 \div v = r$; then the above equation becomes,

$$h = (1 + \mathrm{F} - r^2)\frac{v^2}{64 \cdot 4}; \dots\dots\dots\dots\dots (7.)$$

which gives, for the inverse formula,

$$v = 8 \cdot 025 \sqrt{\frac{h}{1 + \mathrm{F} - r^2}}. \dots\dots\dots\dots\dots (8.)$$

When a stream flows with an *uniform speed* down an *uniform channel*, and two cross-sections of that channel are compared together, the velocities v_0 and v are equal, and $r = 1$; in this case, the *whole* loss of head between the two cross-sections is expended in overcoming friction; and equations 7 and 8 are reduced to the following:—

$$h = \mathrm{F}\, v^2 \div 64 \cdot 4; \dots\dots\dots\dots\dots (9.)$$

$$v = 8 \cdot 025 \sqrt{h \div \mathrm{F}}. \dots\dots\dots\dots\dots (10.)$$

The following table gives examples of heights in feet due to velocities in feet per second, as computed by equation 1. It is exact for latitude $54°\frac{3}{4}$, and near enough to exactness for practical purposes in all latitudes. The most convenient table, however, for calculating either heights from velocities or velocities from heights is an ordinary table of squares and square roots:—

v	h	v	h	v	h	v	h	v	h
1	·01553	17	4·4876	32·2	16·100	48	35·776	76	89·688
2	·06211	18	5·0311	33	16·910	49	37·283	78	94·472
3	·13975	19	5·6056	34	17·950	50	38·820	80	99·379
4	·24845	20	6·2112	35	19·022	52	41·987	82	104·41
5	·38820	21	6·8478	36	20·124	54	45·280	84	109·56
6	·55901	22	7·5155	37	21·257	56	48·695	86	114·84
7	·76087	23	8·2143	38	22·422	58	52·235	88	120·25
8	·99379	24	8·9441	39	23·618	60	55·901	90	125·78
9	1·2578	25	9·7050	40	24·845	62	59·688	92	131·43
10	1·5528	26	10·497	41	26·102	64	63·602	94	137·20
11	1·8789	27	11·320	42	27·391	64·4	64·400	96	143·10
12	2·2360	28	12·174	43	28·711	66	67·640	98	149·13
13	2·6242	29	13·059	44	30·062	68	71·800	100	155·28
14	3·0435	30	13·975	45	31·444	70	76·087		
15	3·4938	31	14·922	46	32·857	72	80·496		
16	3·9752	32	15·901	47	34·301	74	85·029		

447. **Friction of Water.**—The following are the values of the factor of friction F in the formulæ of Article 446, as ascertained by experiment, for the cases of most common occurrence in practice.

I. *Friction of an orifice in a thin plate*—

$$F = 0{\cdot}054. \qquad (1.)$$

II. *Friction of mouthpieces or entrances from reservoirs into pipes.*—Straight cylindrical mouthpiece, perpendicular to side of reservoir—

$$F = 0{\cdot}505. \qquad (2.)$$

The same mouthpiece making the angle θ with a perpendicular to the side of the reservoir—

$$F = 0{\cdot}505 + 0{\cdot}303 \sin \theta + 0{\cdot}226 \sin^2 \theta. \qquad (3.)$$

For a mouthpiece of the form of the "contracted vein," that is, one somewhat bell-shaped, and so proportioned that if d be its diameter on leaving the reservoir, then at a distance $d \div 2$ from the side of the reservoir it contracts to the diameter $\cdot 7854\, d$,—the resistance is insensible, and F nearly $= 0$.

III. *Friction at sudden enlargements.*—Let A_1 be the sectional area of a channel, discharging Q cubic feet of water per second, in which a sluice, or slide valve, or some such object, produces a sudden contraction to the smaller area a, followed by a sudden enlargement to the area A_2. Let $v = Q \div A_2$ be the velocity in the second enlarged part of the channel. The *effective* area of the orifice a will be $c\,a$, c being a *co-efficient of contraction* of the stream flowing through it, whose value may be taken at $\cdot 618 \div \sqrt{1 - \cdot 618 \frac{a^2}{A_1^2}}$. Let the ratio in which the effective area of the channel is suddenly enlarged be denoted by

$$r = A_2 \div c\,a = \frac{A_2}{a} \sqrt{\left(2{\cdot}618 - 1{\cdot}618 \frac{a^2}{A_1^2}\right)}. \qquad (4.)$$

Then $r\,v$ is the velocity in the most contracted part. It appears that all the energy due to the *difference* of the velocities, $r\,v$ and v, is expended in fluid friction, and consequently that there is a loss of head given by the formula—

$$(r - 1)^2 \cdot \frac{v^2}{2\,g}; \qquad (5.)$$

so that in this case

$$F = (r - 1)^2. \qquad (6.)$$

IV. *Friction in pipes and conduits.*—Let A be the sectional area of a channel; b its *border*, that is, the length of that part of its girth which is in contact with the water; l the length of the channel, so that $l\,b$ is the frictional surface; and for brevity's sake let $A \div b = m$; then, for the friction between the water and the sides of the channel—

$$F = f \cdot \frac{l\,b}{A} = \frac{f\,l}{m}; \text{ ....}(7.)$$

in which the co-efficient f has the following values:—

$$\text{For iron pipes (Darcy), ...} f = 0{\cdot}005 \left(1 + \frac{1}{48\,m\,(\text{feet})}\right); (8.)$$

$$\text{For open conduits (Weisbach), } f = 0{\cdot}0074 + \frac{0{\cdot}00023}{v}. \text{ ....}(9.)$$

The quantity $m = A \div b$ is called the "*hydraulic mean depth*" of channel, and for cylindrical and square pipes running full is obviously *one-fourth* of the diameter; and the same is its value for a semicylindrical open conduit, and for an open conduit whose sides are tangents to a semicircle of a diameter equal to twice the greatest depth of the conduit.

In an open conduit, the loss of head,

$$h = \frac{f\,l}{m} \cdot \frac{v^2}{2\,g}, \text{ ....}(10.)$$

takes place as an actual fall in the surface of the water, producing a declivity at the rate

$$i = \frac{h}{l} = \frac{f}{m} \cdot \frac{v^2}{2\,g}; \text{ ....}(11.)$$

and by the last two formulæ are to be determined the fall and the rate of declivity of open channels which are to convey a given flow. In close pipes, the loss of head takes place in the total head; and the ratio $i = h \div l$ is called the *virtual declivity*.

V. For *bends in circular pipes*, let d be the diameter of the pipe, ϱ the radius of curvature of its centre line at the bend, θ the angle through which it is bent, π two right angles; then, according to Professor Weisbach,

$$F = \frac{\theta}{\pi} \left\{ 0{\cdot}131 + 1{\cdot}847 \left(\frac{d}{2\,\varrho}\right)^{\frac{7}{2}} \right\} \text{ ....}(12.)$$

VI. For *bends in rectangular pipes*,

$$F = \frac{\theta}{\pi} \left\{ 0{\cdot}124 + 3{\cdot}104 \left(\frac{d}{2\,\varrho}\right)^{\frac{7}{2}} \right\} \text{ ....}(13.)$$

VII. For *knees*, or sharp turns in pipes, let θ be the angle made by the two portions of the pipe at the knee; then

$$F = 0{\cdot}946 \sin^2 \frac{\theta}{2} + 2{\cdot}05 \sin^4 \frac{\theta}{2}. \qquad (14.)$$

VIII. *Summary of losses of head.*—When several successive causes of resistance occur in the course of one stream, the losses of head arising from them are to be added together; and this process may be extended to cases in which the velocity varies in different parts of the channel, in the following manner:—

Let the final velocity at the cross section, where the loss of head is required, be denoted by v;

Let the ratios borne to that velocity by the velocities in other parts of the channel be known; $r_0 v$ being the "velocity of approach" (Article 446, p. 676), $r_1 v$ the velocity in the first division of the channel, $r_2 v$ in the second, and so on; and let F_1 be the sum of all the factors of resistance for the first division, F_2 for the second, and so on; then the loss of head will be—

$$h = \frac{v^2}{64{\cdot}4}(1 - r_0^2 + F_1 r_1^2 + F_2 r_2^2 + \&c.);$$

an expression which may be abbreviated into the following: (15.)

$$h = \frac{v^2}{64{\cdot}4}(1 - r_0^2 + \Sigma \cdot F r^2).$$

448. **Contraction of Stream from Orifice—Co-efficients of Discharge.**—The fact of the contraction of a jet or stream that flows from an orifice has already been referred to. It is caused by the convergence of the particles towards the orifice before they pass through it, which convergence continues for a time after the particles pass the orifice. The result is, that the *effective* area of the orifice, or area of the "*contracted vein*," which is to be used in computing the discharge, is less than the total area in a proportion which is called the *co-efficient of contraction.*

Sometimes it is impossible to distinguish between the effect of friction in diminishing the velocity (expressed by $1 \div \sqrt{1+F}$), and that of contraction in diminishing the area of the stream. In such cases the ratio in which the actual discharge is less than the product of the "theoretical velocity" (Article 446, p. 675) and the total area of the orifice, is called the *co-efficient of efflux* or *of discharge.*

The quantities given in the following statements and tables are some of them real co-efficients of contraction, and some co-efficients of discharge. In hydraulic formulæ, such co-efficients are usually denoted by the symbol c.

In sharp-edged orifices the friction is almost inappreciable (see Article 447, Case I.); in those with flat or rounded borders its effects become sensible, and in tubes or other channels of such length as to guide all the particles along their sides there is no contraction, and friction operates alone in diminishing the discharge.

In all the *sharp-edged orifices* here mentioned the edge is supposed to be formed at the *inner* or up-stream side of the plate by chamfering or bevelling the outer side. Were the inner side of the plate chamfered, it would guide the stream, and alter the contraction to an uncertain amount.

I. *Sharp-edged circular orifices in flat plates*; $c = \cdot 618$....(1.)

II. *Sharp-edged rectangular orifices in vertical flat plates.*—In this case the co-efficient depends partly on the proportions of the dimensions of the orifice to each other, and partly on the proportion borne by the breadth of the orifice to the *charge* or head. The co-efficient is intended to be used in the following formula for the discharge in cubic feet per second, A being the area of the orifice in square feet; and h the head, measured from the *centre* of the orifice to the *level of still water*.

$$Q = 8{\cdot}025\, c \, A \sqrt{h}. \quad\ldots\ldots\ldots\ldots\ldots\ldots (2.)$$

The co-efficients are given on the authority of experiments of Poncelet and Lesbros on orifices about 8 inches wide. They have not been reduced to a general formula.

CO-EFFICIENTS OF DISCHARGE FOR RECTANGULAR ORIFICES.

Head ÷ Breadth.	Height of Orifice ÷ Breadth. 1	0·5	0·25	0·15	0·1	0·05
0·05	...	...	...	...	...	·709
0·10	...	...	...	...	·660	·698
0·15	...	...	...	·638	·660	·691
0·20	...	...	·612	·640	·659	·685
0·25	...	...	·617	·640	·659	·682
0·30	...	·590	·622	·640	·658	·678
0·40	...	·600	·626	·639	·657	·671
0·50	...	·605	·628	·638	·655	·667
0·60	·572	·609	·630	·637	·654	·664
0·75	·585	·611	·631	·635	·653	·660
1·00	·592	·613	·634	·634	·650	·655
1·50	·598	·616	·632	·632	·645	·650
2·00	·600	·617	·631	·631	·642	·647
2·50	·602	·617	·631	·630	·640	·643
3·50	·604	·616	·629	·629	·637	·638
4·00	·605	·615	·627	·627	·632	·627
6·00	·604	·613	·623	·623	·625	·621
8·00	·602	·611	·619	·619	·618	·616
10·00	·601	·607	·613	·613	·613	·613
15·00	·601	·603	·606	·607	·608	·609

The co-efficients in the preceding table include a correction for the error occasioned by measuring the head from the *centre* of the orifice instead of from the point where the mean velocity occurs, which is somewhat above the centre. That correction is inappreciable when the head exceeds 3 times the height of the orifice.

III. *Sharp-edged rectangular notches* (or orifices extending up to the surface) *in flat vertical weir boards.*—The area of the orifice is measured up to the *level of still water* in the pond behind the weir.

Let b = breadth of the notch;
B = total breadth of the weir; then

$$c = \cdot 57 + \frac{b}{10\,\mathrm{B}}; \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (3.)$$

provided b is not less than B $\div$ 4.

IV. *Sharp-edged triangular or* V*-shaped notches in flat vertical weir boards* (from experiments by Professor James Thomson.)—Area measured up to the level of still water.

$$\left.\begin{array}{l}\text{Breadth of notch} = \text{depth} \times 2;\ c = \cdot 595;\\ \text{Breadth of notch} = \text{depth} \times 4;\ c = \cdot 620.\end{array}\right\} \ldots\ldots (4.)$$

V. *Partially-contracted sharp-edged orifice.* (That is to say, an orifice towards part of the edge of which the water is guided in a direct course, owing to the border of the channel of approach partly coinciding with the edge of the orifice).

Let c be the ordinary co-efficient;
n, the fraction of the edge of the orifice which coincides with the border of the channel;
c', the modified co-efficient; then

$$c' = c + \cdot 09\, n. \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (5.)$$

VI. *Flat or round-topped weir*, area measured up to the level of still water—

$$c = \cdot 5 \text{ nearly}. \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (6.)$$

VII. *Sluice in a rectangular channel*—

$$\left.\begin{array}{r}\text{vertical; } c = 0\cdot 7;\\ \text{Inclined backwards to the horizon at } 60^\circ;\ c = 0\cdot 74;\\ \text{,, \quad ,, \quad ,, \quad at } 45^\circ;\ c = 0\cdot 8.\end{array}\right\} (7.)$$

VIII. *Incomplete contraction;* see Article 477, Division III., p. 677.

449. **Discharge from Vertical Orifices, Notches, and Sluices.**—When the height of an orifice in the vertical side of a reservoir

does not exceed about one-half or one-third of its depth below the surface, the head measured from the centre of the orifice to the level of still water may be used, without sensible error, to compute the mean velocity of a flow, and the discharge; so that the formula for the discharge is

$$Q = 8{\cdot}025\, c\, A \sqrt{h}; \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (1.)$$

A being the total area of the orifice, and c the proper co-efficient of contraction.

When the height of the orifice exceeds about one-half of the head of water, and especially when the orifice is a *notch* extending to the surface, it is not sufficiently accurate to measure the head simply from the level of still water to the centre of the orifice; but the area of the orifice is to be conceived as divided into a number of horizontal bands, the area of each such band multiplied by the velocity due to its depth below the surface of still water, the products summed or integrated, and the sum or integral multiplied by a suitable co-efficient of contraction.

To express this in symbols, let b be the breadth, $d\,h$ the height of one of the horizontal bands, so that $b\ d\ h$ is its area; h, the depth of its centre below the level of the surface of still water in the reservoir; h_0, the depth of the upper edge of the orifice, and h_1 that of its lower edge, below the same level; c, the co-efficient of contraction; Q, the discharge in cubic feet per second; then

$$Q = 8{\cdot}025\, c \int_{h_0}^{h_1} b \sqrt{h} \cdot d\,h. \quad \ldots\ldots\ldots\ldots\ldots\ldots (2.)$$

For co-efficients of contraction, see Article 448.

The following are the most important cases:—

I. *Rectangular orifice;* $b =$ constant.

$$Q = 8{\cdot}025\, c \times \frac{2}{3}\, b \left(h_1^{\frac{3}{2}} - h_0^{\frac{3}{2}}\right) = 5{\cdot}35\, c\, b \left(h_1^{\frac{3}{2}} - h_0^{\frac{3}{2}}\right). \ (3.)$$

It is seldom necessary to use this formula in practice; for the co-efficients in the table by Poncelet and Lesbros (see p. 680) comprehend, as has been stated, the correction for the error arising from using the head at the centre of the orifice simply, as in equation 1.

II. *Rectangular notch, with a still pond;* $b =$ constant, $h_0 = 0$; h_1 measured from the lower edge of the notch to the level of still water.

$$\left.\begin{aligned} Q &= 8{\cdot}025\, c \times \frac{2}{3}\, b\, h_1^{\frac{3}{2}} = 5{\cdot}35\, c\, b\, h_1^{\frac{3}{2}} \\ &= \left(3{\cdot}05 + {\cdot}535\, \frac{b}{B}\right) b\, h_1^{\frac{3}{2}} \end{aligned}\right\} \ldots\ldots\ldots\ldots (4.)$$

The last expression is founded on the formula for the co-efficient c, given in Article 448, Division III., p. 681, B being the whole breadth of the weir.

TABLE OF VALUES OF c AND $5{\cdot}35\,c$.

$\frac{b}{B}$,......	1·0	0·9	0·8	0·7	0·6	0·5	0·4	0·3	0·25
c,......	·67	·66	·65	·64	·63	·62	·61	·60	·595
$5{\cdot}35\,c$,	3·58	3·53	3·48	3·42	3·37	3·32	3·26	3·21	3·18

The *cube of the square root of the head*, $h_1^{\frac{3}{2}}$, is easily computed as follows, by the aid of an ordinary table of squares and cubes: look in the column of squares for the nearest square to h_1; then opposite, in the column of cubes, will be an approximate value of $h_1^{\frac{3}{2}}$.

III. *Rectangular notch, with current approaching it.*—When still water cannot be found, to measure the head h_1 up to, let v_0 denote the velocity of the current at the point up to which the head is measured, or *velocity of approach:* compute the height due to that velocity as follows:—

$$h_0 = v_0^2 \div 64{\cdot}4;$$

then the flow is the difference between that from a still pond due to the height $h_1 + h_0$, and that due to the height h_0; so that it is given by the formula

$$Q = 5{\cdot}35\, c\, b \left\{(h_1 + h_0)^{\frac{3}{2}} - h_0^{\frac{3}{2}}\right\} \ldots\ldots\ldots\ldots\ldots(5.)$$

When v_0 cannot be directly measured, it can be computed approximately by taking an approximate value of Q from equation 4, and dividing by the sectional area of the channel at the place up to which the head is measured from the lower edge of the notch.

IV. *Triangular or* V*-shaped notch, with a still pond;* h_1 measured from the apex of the triangle to the level of still water.

Let a denote the ratio of the *half-breadth* of the notch at any given level to the height above the apex, so that, for example, at the level of still water, the whole breadth of the notch is $2\,a\,h_1$.

$$Q = 8{\cdot}025\, c \times \frac{8}{15}\, a\, h_1^{\frac{5}{2}} = 4{\cdot}28\, c\, a\, h_1^{\frac{5}{2}}; \ldots\ldots\ldots(6.)$$

and adopting the values of c already given in Article 448, p. 681, we have,

$$\text{for } a = 1;\ Q = 2{\cdot}54\, h_1^{\frac{5}{2}}; \ldots\ldots\ldots\ldots\ldots(6\text{ A.})$$

$$\text{for } a = 2;\ Q = 5{\cdot}3\, h_1^{\frac{5}{2}}. \ldots\ldots\ldots\ldots\ldots\ldots(6\text{ B.})$$

In the absence of sufficiently extensive tables of squares and fifth powers, the best method of computing the fifth power of the square root of the head is by the aid of logarithms.

V. *Drowned orifices* are those which are below the level of the water in the space into which the water flows as well as in that from which it flows. In such cases the difference of the levels of still water in those two spaces is the head to be used in computing the flow.

VI. *Drowned rectangular notch.*—Let h_1 and h_2 be the heights of the still water above the lower edge of the notch at the up-stream and down-stream sides of the notch-board respectively; the following formula gives the flow in cubic feet per second:—

$$Q = 5{\cdot}35\, c\, b \left(h_1 + \frac{h_2}{2}\right) \sqrt{(h_1 - h_2.)} \text{..........}(7.)$$

VII. For *weirs with broad flat crests*, drowned or undrowned, the formulæ are the same as for rectangular notches, except that the co-efficient c is about ·5, as has been stated.

VIII. *Computation of the dimensions of orifices.*—The whole of the preceding formulæ (with the exception of equations 5 and 7) can easily be used in an inverse form, in order to find the dimensions of orifices that are required to discharge given volumes of water per second.

For example, if equation 1 is applicable, we have for the area of the orifice,

$$A = Q \div 8{\cdot}025\, c \sqrt{h.} \text{....................}(8.)$$

If equation 3 is applicable, the breadth of the orifice is given as follows:—

$$b = Q \div 5{\cdot}35\, c\, (h_1^{\frac{3}{2}} - h_0^{\frac{3}{2}}). \text{................}(9.)$$

If equation 4 is applicable, the depth of the bottom of the notch below still water is given by the equation,

$$h_1 = \left\{Q \div 5{\cdot}35\, c\, b\right\}^{\frac{2}{3}}; \text{..................}(10.)$$

if equation 6 is applicable,

$$h_1 = \left\{Q \div 4{\cdot}28\, c\, a\right\}^{\frac{2}{5}}. \text{..................}(11.)$$

IX. *Sluices.*—The opening of a sluice generally acts as a rectangular orifice, drowned or undrowned as the case may be; the value of c being as given in Article 448, p. 681.

450. **Computation of the Discharge and Diameters of Pipes.**—The loss of head by a stream of the velocity v in traversing the length

l of a pipe of the uniform diameter d is given by the following formula, deduced from equations 8 and 10 of Article 447, by putting $d \div 4$ for the hydraulic mean depth m:—

$$h = \frac{4fl}{d} \cdot \frac{v^2}{64 \cdot 4} = 0 \cdot 02 \left(1 + \frac{1}{12\,d\,(\text{feet})}\right) \frac{l}{d} \cdot \frac{v^2}{64 \cdot 4} \ldots (1.)$$

From this equation are deduced the solutions of the following problems:—

I. *To compute the discharge of a given pipe;* the data being h, l, and d, all in feet.

For a rough approximation, it is usual to assume an average value for $4f$; say, 0·0258. This gives for the approximate velocity, in feet per second,

$$v = 8 \cdot 025 \sqrt{\frac{h\,d}{\cdot 0258\,l}} = 50 \sqrt{\frac{h\,d}{l}}; \ldots\ldots\ldots\ldots (2.)$$

or *a mean proportional between the diameter and the loss of head in* 2,500 *feet of length;* and for the discharge, in cubic feet per second,

$$Q = \cdot 7854\,v\,d^2 = 39 \sqrt{\frac{h}{l}} \cdot d^{\frac{5}{2}}, \text{ nearly.} \ldots\ldots (2\text{A.})$$

When greater accuracy is required, make

$$4f = 0 \cdot 02 \left(1 + \frac{1}{12\,d\,(\text{feet})}\right); \ldots\ldots\ldots\ldots\ldots (3.)$$

and find the velocity in feet per second by the formula

$$v = 8 \cdot 025 \sqrt{\frac{h\,d}{4fl}}; \ldots\ldots\ldots\ldots\ldots\ldots\ldots (4.)$$

and the discharge, in cubic feet per second, by the formula

$$Q = \cdot 7854\,v\,d^2 = 6 \cdot 3 \sqrt{\frac{h}{4fl}} \cdot d^{\frac{5}{2}}. \ldots\ldots\ldots\ldots (4\text{A.})$$

II. *To find (in feet) the diameter* d *of a pipe, so that it shall deliver* Q *cubic feet of water per second, with a loss of head at the rate of* h *feet in each length of* l *feet.*

Supposing the value of $4f$ known,

$$d = \left(\frac{4fl\,Q^2}{39 \cdot 73\,h}\right)^{\frac{1}{5}}. \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (5.)$$

But $4f$ depends on the diameter sought. Therefore assume, in the first place, an approximate value for $4f$; say, $4f' = \cdot0258$. Then compute a first approximation to the diameter by the following formula :—

$$d' = 0{\cdot}23 \left(\frac{l\,Q^2}{h}\right)^{\frac{1}{5}}. \quad\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(6.)$$

From the approximate diameter, by means of equation 3 of this Article, calculate a second approximation, $4f''$, to the value of $4f$. If this agrees with the value first assumed, d' is the true diameter; if not, a corrected diameter is to be found by the following formula :—

$$d = d' \cdot \left(\frac{f''}{f'}\right)^{\frac{1}{5}} = d \cdot \left(\frac{4}{5} + \frac{f''}{5f'}\right) \text{ nearly.} \quad\ldots\ldots\ldots(7.)$$

In the preceding formulæ the pipe is supposed to be free from all curves and bends so sharp as to produce appreciable resistance. Should such obstructions occur in its course, they may be allowed for in the following manner :—Having first computed the diameter of the pipe as for a straight course, calculate the additional loss of head due to curves by the proper formula (Article 447, p. 678); let h'' denote that additional loss of head; then make a further correction of the diameter of the pipe, by increasing it in the ratio of

$$1 + \frac{h''}{5\,h} : 1. \quad\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(8.)$$

By a similar process an allowance may be made for the loss of head on first entering the pipe from the reservoir, viz. :—

$(1 + F)\,v^2 \div 64{\cdot}4$; F being the factor of friction of the mouthpiece.

To the diameter of a pipe, as computed by the formulæ, an addition is commonly made in practice, in order to allow for accidental obstructions, for the incrustation of the interior of the pipe, &c. According to some authorities about one-sixth is to be added to the diameter of the pipe for this purpose; but experience seems to show that in general the incrustation, if any, is of equal thickness in pipes of all diameters exposed for equal times to the action of the same water; and therefore that, in a given system of water-pipes, an equal absolute allowance should be made for possible incrustation in pipes of all diameters. In ordinary cases it appears that about *one inch* is sufficient for that purpose.

451. **Discharge and Dimensions of Channels.**—The rate of declivity required for the surface of the current in an uniform

conduit or river-channel is found by dividing the loss of head h (which is all actual fall) by the length l of the channel, and is expressed by the following equation, deduced from equation 11 of Article 447, p. 678:—

$$i = \frac{h}{l} = \frac{f}{m} \cdot \frac{v^2}{64{\cdot}4} = \left({\cdot}0074 \quad + \frac{{\cdot}00023}{v} \right) \cdot \frac{v^2}{64{\cdot}4\, m}; \quad (1.)$$

m being the "hydraulic mean depth." This equation enables the following problems to be solved:—

I. *To compute the discharge of a given stream*, the data being i, m, and the sectional area A. The first step is to find the velocity, which might be done by means of a quadratic equation; but it is less laborious to find it by successive approximations. For that purpose assume an *approximate value* for the co-efficient of friction, such as

$$f' = {\cdot}007565;$$

then the *first approximation* to the velocity is

$$v' = 8{\cdot}025 \sqrt{\frac{i\, m}{{\cdot}007565}} = \sqrt{8512\, i\, m} = 92{\cdot}26 \sqrt{i\, m}; \quad (2.)$$

or, *a mean proportional between the hydraulic mean depth and the fall in* 8,512 *feet.* A *first approximation to the discharge* is

$$Q' = v' \text{ A.} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (3.)$$

These first approximations are in many cases sufficiently accurate. To obtain second approximations, compute a corrected value of f according to the expression in brackets in equation 1; should it agree nearly or exactly with f', the first assumed value, it is unnecessary to proceed further; should it not so agree, correct the values of the velocity and discharge by multiplying each of them by the factor,

$$\frac{3}{2} - \frac{f}{{\cdot}01513} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (4.)$$

II. *To determine the dimensions of an uniform channel, which shall discharge* Q *cubic feet of water per second with the declivity* i. —To solve this problem, it is necessary, in the first place, to assume a figure for the intended channel, so that the *proportions* of all its dimensions to each other, and to the hydraulic mean depth m, may be fixed. This will fix also the proportion $A \div m^2$ of the sectional area to the square of the hydraulic mean depth, which will be known although those areas are still unknown; let it be denoted by n.

[The following are examples of the values of n for different figures of cross-section:—

for a semicircle, $n = 6{\cdot}2832$;
for a half-square, $n = 8$;
for a half-hexagon, $n = 4\sqrt{3} = 6{\cdot}928$;
for a section (proposed by Mr. Neville) bounded below and at the sides by three straight lines, all tangents to one semicircle which has its centre at the water level, the bottom being horizontal, and the sides sloping at any angle θ (see fig. 288);

$$n = 4\left(\operatorname{cosec}\theta + \tan\frac{\theta}{2}\right).$$

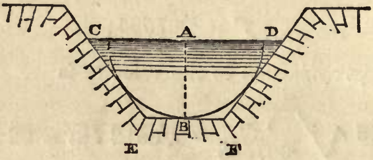

Fig. 288.

In each of the four figures mentioned above, m is one-half of the greatest depth.]

Compute a *first approximation* to the required hydraulic mean depth as follows:—

$$m' = \left(\frac{Q^2}{8{,}512\, n^2 i}\right)^{\frac{1}{5}}; \ldots\ldots\ldots\ldots\ldots\ldots\ldots (5.)$$

also a first approximation to the velocity,

$$v' = \frac{Q}{n\, m'^2}; \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (6.)$$

from these data, by means of equation 1 of this article, compute an *approximate declivity* i′. If this agrees exactly or very nearly with the given declivity, i, the first approximation to the hydraulic mean depth is sufficient; if not, a *corrected hydraulic mean depth* is to be found by the following formula:—

$$m = m'\left(\frac{4}{5} + \frac{i'}{5\, i}\right). \ldots\ldots\ldots\ldots\ldots\ldots\ldots (7.)$$

From the hydraulic mean depth, all the dimensions of the channel are to be deduced, according to the figure assumed for it.

452. **Elevation Produced by a Weir.**—When a weir or dam is erected across a river, the following formulæ serve to calculate the height h_1, in feet, at which the water in the pond, close behind the weir, will stand above its crest; Q being the discharge in cubic feet per second, and b the breadth of the weir in feet:—

I. *Weir not drowned*, with a flat or slightly rounded crest—

$$h_1 = \left(\frac{Q^2}{7\,b^2}\right)^{\frac{1}{3}}, \text{ nearly. } \ldots\ldots\ldots\ldots\ldots\ldots (1.)$$

II. *Weir drowned.*—Let h_2 be the height of the water in front of the weir above its crest.

$$\textit{First approximation; } h'_1 = h_2 + \left(\frac{Q^2}{7\,b^2}\right)^{\frac{1}{3}}. \ldots\ldots (2.)$$

$$\textit{Second approximation; } h''_1 = h'_1 - h_2\left(1 - \frac{5}{4}\cdot\frac{h_2}{h'_1 - h_2}\right). \ (3.)$$

Closer approximations may be obtained by repeating the last calculation.

453. **Backwater** is the effect produced by the elevation of the water-level in the pond close behind the weir, upon the surface of the stream at places still farther up its channel.

For a channel of uniform breadth and declivity, the following is an approximate method of determining the figure which a given elevation of the water close behind a weir will cause the surface of the stream farther up to assume.

Let i denote the rate of inclination of the *bottom* of the stream, which is also the rate of inclination of its surface before being altered by the weir.

Let δ_0 be the natural depth of the stream, before the erection of the weir.

Let δ_1 be the depth as altered, close behind the weir.

Let δ_2 be any other depth in the altered part of the stream.

It is required to find x, the distance from the weir in a direction up the stream at which the altered depth δ_2 will be found.

Denote the ratio in which the depth is altered at any point by

$$\delta \div \delta_0 = r;$$

and let φ denote the following function of that ratio:—

$$\left.\begin{aligned} \varphi = \int \frac{d\,r}{r^3 - 1} = \frac{1}{6}\,\text{hyp. log.}\left\{1 + \frac{3\,r}{(r-1)^2}\right\} \\ + \frac{1}{\sqrt{3}}\,\text{arc. tan.}\,\frac{2\,r + 1}{\sqrt{3}} \end{aligned}\right\} \ldots\ldots (1.)$$

A convenient approximate formula for computing φ is as follows:—

$$\varphi \text{ nearly} = \frac{1}{2\,r^2} + \frac{1}{5\,r^5} + \frac{1}{8\,r^8}. \quad \ldots\ldots\ldots\ldots\ldots (1\text{ A.})$$

Compute the values, φ_1 and φ_2, of this function, corresponding to the ratios

$$r_1 = \delta_1 \div \delta_0 \text{ and } r_2 = \delta_2 \div \delta_0.$$

Then

$$x = \frac{\delta_1 - \delta_2}{i} + \left(\frac{1}{i} - 264\right) \cdot (\varphi_1 - \varphi_2)\,\delta_0 . \quad \ldots\ldots\ldots (2.)$$

The following table gives some values of φ:—

r	φ	r	φ
1·0	∞	1·8	·166
1·1	·680	1·9	·147
1·2	·480	2·0	·132
1·3	·376	2·2	·107
1·4	·304	2·4	·089
1·5	·255	2·6	·076
1·6	·218	2·8	·065
1·7	·189	3·0	·056

The first term in the right-hand side of the formula 2 is the distance back from the weir at which the depth δ_2 would be found if the surface of the water were level. The second term is the additional distance arising from the declivity of that surface towards the weir. The constant 264 is an approximation to $2 \div f$, f being the co-efficient of friction. For a natural declivity of 1 in 264 the second term vanishes. For a steeper declivity it becomes negative, indicating that the surface of the water rises towards the weir; but although that rise really takes place in such cases, the agreement of its true amount with that given by the formula is somewhat uncertain, inasmuch as the formula involves assumptions which are less exact for steep than for moderate natural declivities. It is best, therefore, in cases of natural declivities steeper than 1 in 264, to compute the extent of backwater simply from the first term of the formula.

454. **Stream of Unequal Sections.**—The preceding rule for determining the figure and extent of backwater is the solution of a particular case of the following general problem:—*Given the form of the bed of a stream, the discharge* Q, *and the water-level at one cross-section; to find the form assumed by the surface of the water in an* up-stream *direction from that cross-section.*

In this case the loss of head between any two cross-sections is the sum of that expended in overcoming friction, and of that due to change of velocity, when the velocity increases, or the difference of those two quantities when the velocity diminishes, which difference may be positive or negative, and may represent either a loss or a gain of head. In parts of the stream where the difference is negative, the surface slopes the reverse way. In fig. 289, let O Z be the vertical plane of the cross-section at which the water-level is given; let horizontal abscissæ, such as O X = x, be measured *against* the direction of flow, and vertical ordinates to the surface of the stream, such as X B = z, upwards from a horizontal datum plane. Consider any indefinitely short portion of the stream whose length is $d\,x$, hydraulic mean depth m, and area of section A. The fall in that portion of the stream is $d\,z$, and the acceleration $-\,d\,v$, because of v being opposite to x. Then,

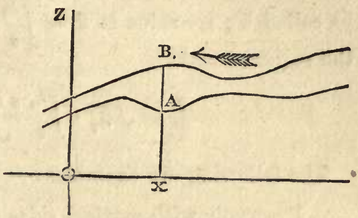

Fig. 289.

$$d\,z = -\frac{v\,d\,v}{32{\cdot}2} + \frac{f\,d\,x}{m} \cdot \frac{v^2}{64{\cdot}4}. \quad \ldots\ldots\ldots\ldots\ldots(1.)$$

In applying this differential equation to the solution of any particular problem, for v is to be put Q ÷ A, and for A and m are to be put their values in terms of x and z. Thus is obtained a differential equation between x and z, and the constant quantity, Q, which equation, being integrated, gives the relation between x and z, the co-ordinates of the surface of the stream.

455. The **Time of Emptying a Reservoir** is determined by conceiving it to be divided into thin horizontal layers at different heights above the outlet, finding the velocity of discharge for each layer, and thence the time of discharge, and summing or integrating the results.

Let s be the area of any given layer, $d\,h$ its depth, A the effective area of the outlet, h the height of the layer above the outlet; then the velocity of outflow for that layer is C $\sqrt{h}$, C being a multiplier taken from the proper formula in Articles 449, 450, or 451. The time of discharge of the layer is

$$d\,t = \frac{s\,d\,h}{\mathrm{A\,C}\sqrt{h}}; \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots(1.)$$

and if h_1 be the height of the top water, the whole time is,

$$t = \frac{1}{\mathrm{A\,C}} \int_0^{h_1} \frac{s\,d\,h}{\sqrt{h}}. \quad \text{.......................(2.)}$$

One of the most convenient ways of expressing this result is to state the ratio which the time of emptying bears to the time of discharging a quantity of water equal to the contents of the reservoir $\left(\text{that is, } \int_0^{h_1} s\,d\,h\right)$, supposing it kept always full. Let that time be called T; its value is $\mathrm{T} = \int_0^{h_1} s\,d\,h \div \mathrm{A\,C}\sqrt{h_1}$, and that of the required ratio is

$$\frac{t}{\mathrm{T}} = \sqrt{h_1} \cdot \int_0^{h_1} \frac{s\,d\,h}{\sqrt{h}} \div \int_0^{h_1} s\,d\,h. \quad \text{.............(3.)}$$

The following are examples:—

Reservoir with vertical sides (s = constant); $t \div \mathrm{T} = 2$.
Wedge-shaped reservoir (s = constant $\times h$); $t \div \mathrm{T} = 1\frac{1}{3}$.
Pyramidal reservoir, the base of the pyramid being the surface, the apex at the outlet (s = constant $\times h^2$); .... $t \div \mathrm{T} = 1\frac{1}{5}$.

The division of the reservoir into layers may be facilitated by a plan with contour-lines at a series of different levels.

The time required to empty part of a reservoir is found by computing the time required to empty the whole, and subtracting from it the time which would be required to empty the remaining part.

The time required to *equalize the level of the water* in two adjoining basins with vertical sides (such as lock-chambers on canals), when a communication is opened between them under water, is the same with that required to empty a vertical-sided reservoir of a volume equal to the volume of water *transferred* between the chambers, and of a depth equal to their greatest difference of level.

SECTION II.—*Of the Measurement and Estimation of Water.*

456. **Sources of Water in General—Rain-fall, Total and Available.** —The original source of all supplies of water is the rain-fall. The rain-water which escapes evaporation and absorption by vegetables either runs directly from the surface of the ground or from the pores of the surface-soil into *streams*, or it sinks deeper into the ground, flows through the crevices of porous strata, and escapes at their out-crop in *springs*, or collects in such porous strata, from which it is drawn by means of *wells*.

In what manner soever the water is collected, and whether it is to be used for irrigation, for driving machinery, for feeding a canal, or for the supply of a town, or to be got rid of as in works of mere drainage, the measurement of the rain-fall of the district whence it comes is of primary importance. To complete that measurement two kinds of data are required,—the area of the district, called the *drainage area*, or *catchment-basin*, or *gathering-ground;* and the depth of rain-fall in a given time.

I. *A Drainage Area*, or *Catchment-basin*, is, in almost every case, a district of country enclosed by a *ridge* or *water-shed line* (see Article 58, p. 93), continuous except at the place where the waters of the basin find an outlet. It may be, and generally is, divided by branch ridge-lines into a number of smaller basins, each drained by its own stream into the main stream. In order to measure the area of a catchment-basin a plan of the country is required, which either shows the ridge-lines or gives data for finding their positions by means of detached levels, or of contour-lines. (Article 59, p. 95.)

When a catchment-basin is very extensive it is advisable to measure the several smaller basins of which it consists, as the depths of rain-fall in them may be different; and sometimes, also, for the same reason, to divide those basins into portions at different distances from the mountain-chains, where rain-clouds are chiefly formed.

The exceptional cases, in which the boundary of a drainage area is not a ridge-line on the surface of the country, are those in which the rain-water sinks into a porous stratum until its descent is stopped by an impervious stratum, and in which, consequently, one boundary at least of the drainage area depends on the figure of the impervious stratum, being, in fact, a ridge-line on the upper surface of that stratum, instead of on the ground, and very often marking the upper edge of the outcrop of that stratum. If the porous stratum is partly covered by a second impervious stratum, the nearest ridge-line on the latter stratum to the point where the porous stratum crops out, will be another boundary of the drainage area. In order to determine a drainage area under these circumstances it is necessary to have a geological map and sections of the district.

II. The *Depth of Rain-fall* in a given time varies to a great extent at different seasons, in different years, and in different places. The extreme limits of annual depth of rain-fall in different parts of the world may be held to be respectively nothing and 150 inches. The average annual depth of rain-fall in different parts of Britain ranges from 22 inches to 140 inches, and the least annual depth recorded in Britain is about 15 inches.

The rain-fall in different parts of a given country is, in general, greatest in those districts which lie towards the quarter from which the prevailing winds blow; in Britain, for example, the western districts are the most rainy. Upon a given mountain-ridge, however, the reverse is the case, the greatest rain-fall taking place on that side which lies to leeward, as regards the prevailing winds: thus, in Britain, more rain falls in general on the eastern than on the western slope of a range of hills. The cause of this is probably the fact that the condensation of watery vapour in the atmosphere into rain-clouds arises in general from the ascent of moist and warm air up the slopes of mountains into a cold region; the clouds thus formed are drifted by the wind to the leeward side of the mountains, and there fall in rain. To the same cause may be ascribed the fact that the rain-fall is greater in mountainous than in flat districts, and greater at points near high mountain-summits than at points further from them.

The elevation of the locality where the rain-fall is measured does not appear materially to affect the depth, except in so far as elevation is an usual accompaniment of nearness to a mountain-chain.

A vast amount of detailed information has been collected as to the depth of rain-fall in different places at different times; but there does not yet exist any theory from which a probable estimate of the rain-fall in a given district can be deduced independently of direct observation.

The most important data respecting the depth of rain-fall in a given district, for practical purposes, are the following:—

(1.) The least annual rain-fall.
(2.) The mean annual rain-fall.
(3.) The greatest annual rain-fall.
(4.) The distribution of the rain-fall at different seasons, and, especially, the longest continuous drought.
(5.) The greatest flood rain-fall, or continuous fall of rain in a short period.

The order of importance of these data depends on the purpose of the proposed work. If it is one of water-supply, the least annual rain-fall and the longest drought are the most important data; if it is a work of drainage, the greatest annual rain-fall and the greatest flood are the most important.

Experience shows that to obtain those data completely and exactly for a given district requires at least 20 years of daily rain-gauge observations, if not more. But it very seldom happens that so long a series of observations has been made in the precise spots to which the inquiries of the engineer are directed, and in the absence of such records he may proceed as follows:—

(1.) Obtain a copy of the records of the observations made at the nearest station where the rain-fall has been observed for a long series of years, and from them ascertain the longest drought, and compute the mean annual rain-fall at that station, the greatest and least annual rain-fall, the greatest flood rain-fall, &c. The station in question may be called the "standard station."

(2.) Establish rain-gauges in the district to be examined, at places which may be called the "catchment stations," and have them observed daily by trustworthy persons, taking care to obtain a copy of the records of the observations made at the same time at the standard station; and let that series of simultaneous observations be carried on as long as possible.

(3.) Compute from those simultaneous observations the proportions borne to the rain-fall at the standard station by the rain-fall in the same time at the several catchment stations; multiply the greatest, least, and mean annual depths of rain-fall, the greatest flood, &c., at the standard station by those proportions, and the results will give probable values of the corresponding quantities at the catchment stations.

The positions of the catchment rain-gauge stations must, to a considerable extent, be regulated by the practicability of having them observed once a-day; but they should, as far as practicable, be distributed uniformly over the gathering-ground. If it consists of a number of branch basins, there should, if possible, be one or more rain-gauges in each of them. If it is bounded or traversed by high hills, some gauges should be placed on or near their summits, and others at different distances from them.

Each rain-gauge should be placed in an open situation, that it may not be screened by rocks, walls, trees, hedges, or other objects. Its mouth should be as near the level of the ground as is consistent with security. It may be surrounded with an open timber or wire fence to protect it from cattle and sheep.

A rain-gauge for use in the field consists, in general, of a conical funnel, with a vertical cylindrical rim, very accurately formed to a prescribed diameter, such as 10 or 12 inches, and a collecting vessel for the water, usually cylindrical, and smaller in area than the mouth of the funnel. If this vessel is to be used as a measuring vessel also, the ratio of its area to that of the mouth of the funnel is accurately ascertained, and the depth at which the water stands in it is shown by means of a float with a graduated brass stem rising above the mouth of the gauge. Sometimes the rain collected is measured by being poured into a graduated glass measure, which the observer carries in a case. The most accurate method of graduating the measure is by putting known weights of water into it, and marking the height at which they stand (as recommended by Mr. Haskoll in his *Engineering Field-Work*). In performing this process, the weight of a cubic inch of pure water, at 62° Fahr., may be taken as

$$252{\cdot}6 \text{ grains.*}$$

The glass measure may either be graduated to cubic inches, which, being divided by the area of the funnel in square inches, will give the depth of rain-fall in inches; or it may be graduated to show at once inches of rain-fall in a funnel of the area employed.

Observations of rain-fall in the field are usually recorded to two decimal places of an inch.

It may be stated as a result of experience, that the proportions of the least, mean, and greatest annual rain-fall at a given spot usually lie between those of the numbers 2, 3, and 4, and those of the numbers 4, 5, and 6.

III. The *Available Rain-fall* of a district is that part of the total rain-fall which remains to be stored in reservoirs, or carried away by streams, after deducting the loss through evaporation, through permanent absorption by plants and by the ground, &c.

The proportion borne by the available to the total rain-fall varies very much, being affected by the rapidity of the rain-fall and the compactness or porosity of the soil, the steepness or flatness of the ground, the nature and quantity of the vegetation upon it, the temperature and moisture of the air, the existence of artificial drains, and other circumstances. The following are examples:—

Ground.	Available Rain-fall. ÷ Total Rain-fall.
Steep surfaces of granite, gneiss, and slate,	nearly 1
Moorland and hilly pasture, ...	from ·8 to ·6
Flat cultivated country, ...	from ·5 to ·4
Chalk,..	0

Deep-seated springs and wells give from ·3 to ·4 of the total rain-fall.

Such data as the above may be used in roughly estimating the probable available rain-fall of a district; but a much more accurate and satisfactory method is to measure the actual discharge of the streams at the same time that the rain-gauge observations are made, and so to find the actual proportion of available to total rain-fall.

457. **Measurement and Estimation of the Flow of Streams.**—There

* This is deduced from the value already given in p. 161 for the weight of a cubic foot of pure water at 62° Fahr., viz., 62·355 lbs. avoirdupois, or 436,495 grains. That value is based on data given in Professor Miller's paper on the "Standard Pound" (*Philosophical Transactions*, 1856); it differs slightly from that formerly fixed by statute but since abolished.

are three methods of measuring the discharge of a stream—by weir-gauges, by current meters, and by calculation from the dimensions and declivity.

I. The use of *Weir-gauges* is the most accurate method, but it is applicable to small streams only. The weir is constructed across the stream so as to dam up a nearly still pond of water behind it, from which the whole flow of the stream escapes through a notch or other suitable sharp-edged orifice in a vertical plate or board, the elevation of still or nearly still water being observed on a vertical scale in the pond, whose zero-point is on a level with the bottom of the notch, or with the centre of a round or rectangular orifice. For the laws of the discharge of water through vertical orifices, see Article 449, p. 681.

For streams of very variable flow, it appears from the experiments of Mr. James Thomson, that the right-angled triangular notch is the best form of orifice (see *Reports of the British Association for* 1861), as it measures large and small quantities with equal precision, and has a sensibly constant co-efficient of contraction. Where one such notch is insufficient, he recommends the use of a row of them. The pond may have a flat floor of planks, on a level with the bottom of the triangular notch.

When orifices wholly immersed are used, round or square holes are the best, because their co-efficients of contraction vary less than those of oblong holes (see p. 680). If one round or square hole is insufficient, a horizontal row of them may be used.

A weir-gauge should be placed on a straight part of the channel, because if it is placed on a curved part the rush of water from the outlet may undermine the concave bank of the stream. To prevent the weir itself from being undermined in front, the bottom of the channel below the outlet should be protected by an apron of boards, or a stone pitching, or by carrying the water some distance forward in a wooden shoot or spout, placed so low as not to drown any part of the outlet.

Stream-gauges ought to be observed once a-day at least, and oftener when the flow of the stream is in a state of rapid variation, as it is during the rise and fall of floods.

II. *By Current Meters.*—In large streams the flow can in general be measured only by finding the area of cross-section of the stream, measuring by suitable instruments the velocities of the current at various points in that cross-section, taking the mean of those velocities, and multiplying it by the sectional area. The most convenient instrument for such measurements of velocity is a small light revolving fan, on whose axis there is a screw, which drives a train of wheel-work, carrying indexes that record the number of revolutions made in a given time. The whole apparatus is fixed at the end of a pole, so that it can be immersed to different depths in different parts of the channel. The relation between the number of revolutions of the fan per minute, and the corresponding velocity of the current, should be determined experimentally by moving the instrument with different known velocities through a piece of still water, and noting the revolutions of the fan in a given time.

A straight and uniform part of the channel should always be chosen for experiments on the velocity of a stream.

When from the want of the proper instrument, or any other cause, the velocity of the current cannot be measured at various points, the velocity of its swiftest part, which is at the middle of the surface of the stream, may be measured by observing the motions of any convenient body floating down, and from that greatest velocity the mean velocity may be computed by the formula given in Article 445, p. 674.

III. *By Calculation from the Declivity.*—For this purpose a portion of the stream must be carefully levelled, cross-sections being taken at intervals; and the discharge is to be calculated by the rules of Division I. of Article 451, p. 687. In order that the result may be accurate, the part of the stream chosen should have, as nearly as possible, an uniform cross-section and declivity, and should be free from obstruction to the current, and, above all, from weeds, which have been known to increase the friction nearly tenfold.

IV. *Estimation of Flow in Different Years.*—The discharge of a stream during a certain period of observation having been ascertained, may be used to compute probable values of its least, mean, and greatest discharge in a series of years, by multiplying it by the proportions borne by the rain-fall in those years as observed at the "standard station" (see Article 456, p. 695) to the rain-fall at the same station during the period when the stream was gauged.

458. **Ordinary Flow and Floods.**—Questions often arise between the promoters of a water-work and the owners and occupiers of land on the banks of a stream as to the distinction between the "ordinary" or "average summer discharge" of a stream and the "flood discharge." The distinction is in general not difficult to draw by an engineer who personally inspects the stream at each time that its flow is gauged; but to provide for the case of such inspection being impracticable, Mr. Leslie has proposed an arbitrary rule for drawing that distinction, which many engineers have adopted. It is as follows:—

Range the discharges as observed daily in their order of magnitude.

Divide the list thus arranged into an upper quarter, a middle half, and a lower quarter.

The discharges in the upper quarter of the list are to be considered as *floods.*

For each of the flood discharges thus distinguished substitute the *average of the middle half of the list,* and take the mean of the whole list, as thus modified, for the *ordinary* or *average discharge, exclusive of flood-waters.*

It appears that the ordinary discharge, as computed by this method in a number of examples of actual streams in hilly districts, ranges from *one-third* to *one-fourth* of the *mean discharge, including floods;* being a result in accordance with those arrived at by engineers who have distinguished floods from ordinary discharges to the best of their judgment, without following rules.

459. **Measurement of Flow in Pipes.**—The *Water Meters,* or instruments for measuring the flow in pipes, now commonly used, may be divided into two classes—piston meters and wheel meters.

A piston meter is a small double-acting water-pressure engine, driven by the flow of water to be measured. That of Messrs. Chadwick and Frost records the *number of strokes* made by the piston, each stroke corresponding to a certain volume of water. That of Mr. Kennedy is so constructed that, by means of a rack on the piston-rod driving pinions, the *distance* traversed by the piston is recorded by a train of wheel-work, with dial-plates and indexes.

An example of a wheel meter is that of Mr. Siemens, being a small *reaction turbine* or "Barker's mill," driven by the flow. The revolutions are recorded by a train of wheel-work, with dial-plates and indexes.

Another example of a wheel meter is that of Mr. Gorman, being a small *fan turbine* or *vortex wheel* driven by the flow, and driving the indexes of dial-plates.

The ordinary errors of a good water meter are from $\frac{1}{2}$ to 1 per cent.; in extreme cases of variation of pressure and speed errors may occur of $2\frac{1}{2}$ per cent.

The value of the revolutions of a wheel meter should be ascertained experimentally, by finding the number of revolutions made during the filling of a tank of known capacity.

For descriptions of several kinds of water meters, see the *Transactions of the Institution of Mechanical Engineers for* 1856.

Section III.—*Of Store Reservoirs.*

460. **Purposes and Capacity of Store Reservoirs.**—A store reservoir is a place for storing water, by retaining the excess of rain-fall in times of flood, and letting it off by degrees in times of drought. It effects one or more of the following purposes:—

To prevent damage by floods to the country below the reservoir.

To prevent the evil consequences of droughts.

To increase the ordinary or available flow of a stream by adding to it the whole or part of the flood-waters.

To enable water to be diverted from a stream without diminishing the "ordinary" or "average summer flow," as defined in Article 458, p. 698.

To allow mechanical impurities to settle.

The *available capacity* or *storage-room* of a reservoir is the volume contained between the highest and lowest working water-levels, and is less than the *total capacity* by the volume of the space below the lowest working water-level, which is left as a place for the collection of sediment, and which is either kept always full, or only emptied when it is absolutely necessary to do so for purposes of cleansing and repair. It is impossible to lay down an universal rule for the volume of the space so left, or "bottom" as it is called; but in some good examples of artificial reservoirs it occupies about one-sixth of the greatest depth of water at the deepest part of the reservoir.

The absolute storage-room required in a reservoir is regulated by two circumstances:—the *demand* for water, and the extent to which the *supply* fluctuates.

The demand is usually a certain uniform quantity per day. Experience has shown that about 120 *days' demand* is the least storage-room that has proved sufficient in the climate of Britain; in some cases it has proved insufficient; and even a storage equal to 140 days' demand has been known to fail in a very dry season; and consequently some engineers advise that every store reservoir should if possible contain *six months' demand.*

From data respecting various existing reservoirs and gathering-grounds, given by Mr. Beardmore (*Hydraulic Tables*), it appears that the storage-room varies *from one-third to one-half of the available annual rain-fall.*

The best rule for estimating the available capacity required in a store reservoir would probably be one founded upon taking into account the supply as well as the demand. For example—

180 *days of the excess of the daily demand, above the least daily supply*, as ascertained by gauging and computation in the manner described in the preceding section.

In order that a reservoir of the capacity prescribed by the preceding rule may be efficient, it is essential that the *least available annual rain-fall* of the gathering-ground should be sufficient to supply *a year's demand for water.*

To enable the gathering-ground to supply a demand for water corresponding to the *average available annual rain-fall*, the *greatest* *total deficiency* of available rain-fall below such average, whether confined to one year or extending over a series of years, must be ascertained, and an addition equal to such deficiency made to the reservoir room; but it is in general safer, as well as less expensive, to extend the gathering-ground so that the least annual supply may be sufficient for the demand.

The foregoing principles as to capacity have reference to those cases in which the water is to be used to supply a demand for water. When the sole object of the reservoir is to prevent floods in the lower parts of the stream, it ought to be able to contain the ascertained greatest total excess of the available rain-fall during a season of flood above the greatest discharging capacity of the stream consistent with freedom from damage to the country.

461. **Reservoir Sites.**—In choosing the site of a reservoir, the engineer has three things chiefly to consider: the elevation, the configuration of the ground, and the materials, especially those which will form the foundations of the embankment or embankments by which the water is to be retained.

I. The *Elevation* of the site must at once be so high that from the lowest water-level there shall be sufficient fall for the pipes, conduits, or other channels by which the water is to be discharged, and at the same time so low that there shall be a sufficient gathering-ground above the highest water-level.

II. The *Configuration of the Ground* best suited for a reservoir site is that in which a large basin can be enclosed by embanking across a narrow gorge. To enable the engineer to compare such sites with each other, and to calculate their capacities, plans with frequent contour-lines are very useful (Article 59, p. 95), or in the absence of contour-lines, numerous cross-sections of the valleys. The water's edge of the reservoir is itself a contour-line. After the site of a reservoir has been fixed, a plan of it should be prepared with contour-lines numerous and close enough to enable the engineer to compute the capacity of every foot in depth from the lowest to the highest water-level, so that when the reservoir is constructed and in use, the inspection of a vertical scale fixed in it may show how much water there is in store.

Care should be taken to observe whether the basin of a projected reservoir site has, besides its lowest outlet, higher outlets through which the water may escape when the lowest outlet is closed, unless they also are closed by embankments.

The figure of the ground at the site of a proposed reservoir embankment must be determined with care and accuracy, by making not only a longitudinal section along the centre line of the embankment (which section will be a cross-section as regards the valley), but several cross-sections of the site of the embankment, which should be at right angles to the longitudinal section, unless there is some special reason for placing them otherwise. One of these cross-sections of the embankment site should run along the course of the existing outlet of the reservoir site (usually a stream), and another along the course of the intended outlet (usually a culvert containing one or more pipes).

III. *Material.*—The materials of the site of the intended embankment should be either impervious to water or capable of being easily removed so far as they are pervious, in order to leave a water-tight foundation; and their nature is to be ascertained by borings and trial pits, as to which, see Article 187, p. 331, and Article 391, p. 598; and, if necessary, by mines. (Article 392, p. 594.) In many cases it is not sufficient to confine this examination to the site of the embankment; but the bottom and sides of the reservoir-basin must be examined also, in order to ascertain whether they do not contain the outcrop of porous strata, which may conduct away the impounded water. The best material for the foundation of a reservoir embankment is clay, and the next, compact rock free from fissures. Springs rising under the base of the embankment are to be carefully avoided.

The engineer should ascertain where earth is to be found suitable for making the embankment, and especially clay fit for puddle.

462. **Land Awash** means land which lies near the margin of a reservoir, at a height not exceeding three feet above the top water-level, and whose drainage is consequently injured. The promoters of the reservoir are sometimes obliged to purchase such land. Its boundary is of course a contour-line.

463. **Construction of Reservoir Embankments.**—I. *General Figure and Dimensions.*—A reservoir embankment rises at least 3 feet above the top water-level, and in some cases 4, 6, or even 10 feet. It has a level top, whose breadth may be in ordinary cases about *one-third* of the greatest height of the embankment; the outer slope, or that furthest from the water, may have an inclination regulated by the stability of the material, such as $1\frac{1}{2}$ to 1, or 2 to 1; the inner slope, or that next the water, is always made flatter, its most common inclination being 3 to 1.

II. The *Setting-out* of the boundaries of the embankment on the ground (see Article 67, p. 113) is to be performed with great accuracy, by the aid of the cross-sections already mentioned in a preceding article. The following method also has been found convenient in suitable situations. On the side of the valley, at one end of the proposed embankment, erect upon props a wooden rail, with its upper edge exactly horizontal, and exactly in the plane of the slope to be set out. At a convenient distance back from the rail as regards the slope, set up a prop supporting a sight having a small eye-hole, also exactly in the plane of the slope to be set out. A row of pegs ranged from the sight so as to mark points on the ground in a line with the upper edge of the rail will give the foot of the slope.

The same rail (with two different sights) may be used to set out both slopes, if its upper edge coincides with their line of intersection. Let the inner slope be s to 1, the outer s' to 1, the breadth of the top of the embankment b; then the height of that line of intersection above the top of the embankment is,

$$b \div (s + s'); \text{..........................} (1.)$$

and its horizontal distance outwards from the centre line of the embankment is,

$$b\,(s - s') \div 2\,(s + s'). \text{...................} (2.)$$

An instrument consisting of a bar with two sights capable of turning about an axis adjusted so as to be perpendicular to the slope to be ranged has been used for the same purpose.

III. *Preparing the Foundation.*—The foundation is to be prepared by stripping off the soil, and excavating and removing all porous materials, such as sand, gravel, and fissured rock, until a compact and water-tight bed is reached.*

IV. The *Culvert* for the outlet-pipes is next to be built in cement or strong hydraulic mortar, resting on a base of hydraulic concrete. Its internal dimensions must be sufficient to admit of the access of workmen beside the pipe or pipes which it is to contain. The principles which should regulate its figure and thickness are those which have been explained in Article 297 A, p. 433. The outer or down-stream end of the culvert is usually open, and often has wing-walls sustaining the thrust of part of the outer slope of the embankment; the inner or up-stream end is usually closed with water-tight masonry, through which the lowest or scouring outlet-pipe passes. In some reservoirs there is a water-tight partition of masonry at an intermediate point in the culvert. The culvert is to be well coated with clay puddle. (Article 206, p. 344.) In the best constructed reservoirs a *tower* stands on the inner end of the culvert, to contain outlet-pipes for drawing water from different levels, with valves, and mechanism for opening and shutting them.

Sometimes a cast iron pipe is laid without any culvert.

* The following method was used by Jardine to clear unsound pieces away from the rock foundation of the embankment of Glencorse reservoir, near Edinburgh. A layer of clay puddle was spread and well rammed over the surface of the rock, and was then torn off, when all the fissured fragments came away adhering to the sheet of puddle, leaving a surface of sound rock for the foundation of the embankment.

V. *Making the Embankment.*—The embankment is to be made of clay in thin horizontal layers, as described in Article 199, Division III., p. 341. The central part of the embankment should be a "*puddle wall*," of a thickness at the base equal to about one-third of its height; it may diminish to about two-thirds or one-half of that thickness at the top. Great care must be taken that the puddle wall makes a perfectly water-tight joint with the ground throughout the whole of its course, and also with the puddle coating of the culvert.*

During the construction of a reservoir embankment care should be taken to provide a temporary outlet for the water of its gathering-ground, sufficient to carry away the greatest flood-discharge. This may be done either by having a pipe sufficient for the purpose traversing the culvert, or by completing a sufficient bye-wash before the embankment is commenced.

VI. *Protection of Slopes and Top.*—The outer slope is usually protected from the weather by being covered with sods of grass. The inner slope is usually *pitched* or faced with dry stone set on edge by hand, about a foot thick, up to about three feet above the top water-level, and as much higher as waves and spray are found to rise. The top of the embankment may be covered with sods like the outer slope; but it is often convenient to make a roadway upon it; in either case it should be dressed so as to have a slight convexity in the middle, like that given to ordinary roads, in order that water may run off it readily.

No trees or shrubs should be allowed to grow on a reservoir embankment, as their roots pierce it and make openings for the penetration of water. For the same reason no stakes should be driven into it.

464. **Appendages of Store Reservoirs.** — I. The *Waste-weir* is an appendage essential to the safety of every reservoir. It is a weir at such a level, and of such a length, as to be capable of discharging from the reservoir the greatest flood-discharge of the streams which supply it, without causing the water-level to rise to a dangerous height. (As to the discharge over a weir, see Article 449, Divisions II., III., VI., and VII., pp. 682 to 684.) The water discharged over the weir is to be received into a channel, open or covered, as the situation may require, and conducted into the natural water-course below the reservoir embankment. The weir is to be built of ashlar or squared hammer-dressed masonry; the bottom of the waste-

* The late Mr. Smith of Deanston rammed and puddled each successive layer of a reservoir embankment by erecting a rail-fence along each side of it, and driving a flock of sheep several times backwards and forwards along it.

Clay puddle may be protected against the burrowing of rats by a mixture of engine ashes, care being taken not to add so much as to make it pervious to water.

channel, directly in front of it, is best protected by a series of rough stone steps, which break the fall of the water. Instead of a waste-weir, a *waste-pit* has in some cases been used; that is to say, a tower rising through or near the embankment to the top water-level; the waste water falls into this tower and is carried away by a culvert from its bottom; but the efficiency and safety of this contrivance are very questionable, for it seldom can have a sufficient extent of overfall at the top.

II. *Waste-sluices* may be opened to assist the waste-weir in discharging an excessive supply of water. They may either be under the control of a man in charge of the reservoir, or they may be self-acting. The simplest and best self-acting waste-sluice is that of M. Chaubart, as to which, see *A Manual of the Steam Engine and other Prime Movers*, Article 139, p. 153.

III. *Culvert, Valve-Tower, Bridge, Outlet-Pipes and Valves.*—The culvert and its tower have been mentioned in the preceding article. When the tower is imbedded in the embankment, as it sometimes is, it is called the *valve-pit;* but the best position for it is in the reservoir, just clear of the embankment; and then a light *foot-bridge* is required to give access to it from the top of the embankment.

When the object of a store reservoir is simply to equalize the flow of a stream, in order to protect the lower country from floods, and to obtain an increased ordinary flow available for irrigation and water-power, one outlet-pipe may be sufficient, discharging into the natural water-course below the embankment; but if the water is to be used for the supply of a town, or for any other purpose to which cleanness is essential, there must be at least two outlet-pipes,—the ordinary *discharge-pipe*, which takes the water from a point or points not below the lowest water-level of the reservoir, in order to conduct it to the town or place to be supplied; and the *cleansing-pipe*, which takes the water at or near the lowest point in the reservoir, and discharges it into the natural water-course below the embankment, and is only opened occasionally in order to scour away sediment. The water-course, where such scouring discharge falls into it, must have its bottom protected by a stone pitching. As to the discharge of pipes, see Article 450, p. 684.

The mouthpieces of such pipes should be guarded against the entrance of stones, pieces of wood, or other bodies which might obstruct them or injure the valves, by means of convex gratings. The valves best suited for them are slide valves, as to which, see *A Manual of the Steam Engine and other Prime Movers*, Article 120, p. 124.

IV. The *Bye-wash* is a channel sometimes used to divert past the reservoir the waters of the streams which supply it, when these are turbid or otherwise impure. Its dimensions are fixed according to the principles of Article 451, p. 685. Its course usually lies near one margin of the reservoir, and is then conveniently situated for receiving the water discharged by the waste-weir.

In some cases, when a reservoir has been made under a stipulation that only the surplus above a certain quantity was to be allowed to flow into it from the streams, the whole of the streams have been conducted past the reservoir in a bye-wash, having weirs or overfalls along its margin, at certain points in its course above the top water-level of the reservoir. The levels of those weirs were so adjusted that when no more than the prescribed quantity flowed down the bye-wash none escaped over the weirs; but when there was any surplus flow in the bye-wash, the water in it rose above the crests of the weirs, and the surplus escaped over them into the reservoir.

V. *Diversion-cuts* are permanent bye-washes for streams that are so impure as to be rejected altogether.

VI. *Feeders* are small channels for diverting either streams or surface drainage *into* the reservoir, and so increasing its gathering-ground. When used to catch surface drainage, they have been found to conduct to the reservoir from *one-quarter* to *one-half* of the rain-fall.

In connection with feeders for diverting streams into the reservoir may be mentioned what may be called a *separating-weir*, the invention of an assistant of Mr. Bateman, and first used in the Manchester water-works. A weir built across the channel of a stream has in front, and parallel to its crest, a small conduit running along its front slope at such a level that when the stream is in flood, and therefore turbid, the cascade from the top of the weir over-leaps the conduit, and runs down the front slope into the natural channel, which conveys it to a reservoir for the supply of mills; but when the flow is moderate, the cascade falls into the small conduit, which leads it into a feeder of the store reservoir for the supply of the city.

The horizontal distance x to which a cascade from the crest of a weir will leap in the course of a given fall z below that crest may be thus calculated. The mean velocity with which the cascade shoots from the weir-crest is nearly

$$v = \frac{2}{3} \times 8{\cdot}025 \sqrt{h_1} = 5{\cdot}35 \sqrt{h_1}; \text{ ....(3.)}$$

h_1 being the height from the weir-crest to still water in the pond. Then

$$x = \frac{2\, v \sqrt{z}}{8{\cdot}025} = \frac{4}{3} \sqrt{z\, h_1}. \text{ ....(4.)}$$

465. **Reservoir Walls.**—Retaining walls are often used at the foot of the slopes of a reservoir embankment; they are of course to be built in strong and durable hydraulic mortar, especially at the foot of the inner slope. As to their stability and construction, see Articles 265 to 271, pp. 401 to 410.

When the gorge to be closed has a bottom of sound rock, a wall of rubble masonry, built in strong hydraulic mortar, may with great advantage, in point of durability, be substituted for an earthen embankment; and this is especially the case when the depth is great, such as 100 feet and upwards. The masonry should be built with great care; and continuous courses should be avoided; for the bed-joints of such courses tend to become channels for the leakage of the water. In designing the profile of the wall, with a view to stability, strength, and economy of material, the following principles are to be followed:—

(1.) The inner face of the wall to be nearly vertical.

(2.) At each horizontal section, the centre of resistance not to deviate from the middle of the thickness, inward when the reservoir is empty, outward when full, to such an extent as to produce appreciable tension at the further face of the wall.

(3.) The intensity of the vertical pressure at the inner face of the wall, when the reservoir is empty, and at the outer face when the reservoir is full, not to exceed a safe limit. That limit may be estimated as nearly equivalent to the weight of a column of masonry —160 feet high for the inner face, and about 125 feet high for the outer face; the reason for making the latter value the smaller being, that owing to the batter of the outer face, the resultant pressure may be considerably greater than the vertical pressure, especially near the base of the wall.

466. **Lake Reservoirs.**—To convert a natural lake into a reservoir it must be provided with a waste-weir, and with one or more outlets at the intended lower water-level, controlled by valves. The outlet or outlets may be made either by building a culvert with pipes in an excavation of sufficient depth, or by tunnelling through one of the ridges that enclose the lake.

Section IV.—*Of Natural and Artificial Water-Channels.*

467. **Surveying and Levelling of Water-Channels.**—The principles which connect the dimensions, figure, declivity, velocity of current, and discharge of a water-channel have already been fully set forth in Articles 444 and 445, pp. 673 to 674, and Articles 451 to 454, pp. 686 to 691. In the present section are to be explained the principles according to which such-channels, whether natural or artificial, are constructed, preserved, and improved.

The plans of an existing or intended water-channel require no special remark beyond what has already been stated as to plans in general in the first part of this work, except that in the case of existing streams liable to overflow their banks, they should show the boundaries of lands liable to be flooded, and also of those liable to be laid *awash* (see Article 462, p. 702), and that their utility will be greatly increased by contour-lines. The longitudinal section should be made along the centre line of a proposed channel, and along the line of the most rapid current in an existing channel; and it should show the levels of both banks as well as those of the bottom of the channel, and of the surface of the current in its lowest, ordinary, and flooded conditions. It should be accompanied by numerous cross-sections, especially in the case of existing streams of variable sections; and of those cross-sections a sufficient number should extend completely across the lands flooded and awash, to show the figure of their surface. They should include accurate drawings of the archways, roadways, and approaches of existing bridges, also of existing weirs and other obstructions. The nature of the strata should be ascertained, as for any piece of earthwork, by sinking pits and borings, and, in the case of an existing channel, by probing its bottom also, and the results should be shown on the section and plan.

468. **Regime or Stability of a Water-Channel.**—A water-channel is said to be in a state of *régime* or *stability* when the materials of its bed are able to resist the tendency of the current to sweep them forward. The following table shows, on the authority of Du Buat, the greatest velocities *of the current close to the bed*, consistent with the stability of various materials:—

Soft clay,....................................	0·25 foot per second.
Fine sand,....................................	0·50 " "
Coarse sand, and gravel as large as peas,	0·70 " "
Gravel as large as French beans,.........	1·00 " "
Gravel 1 inch in diameter,..................	2·25 feet per second.
Pebbles 1½ inch diameter,..................	3·33 " "
Heavy shingle,..............................	4·00 " "
Soft rock, brick, earthenware,..............	4·50 " "
Rock, various kinds,........................	6·00 " " and upwards.

As to the relation between the surface velocity, the mean velocity, and the velocity close to the bed, see Article 445, p. 674.

The condition of the channels of streams which have a rocky bed is generally that of stability. When the bed is stony or gravelly the condition is most frequently that of stability in the ordinary state of the river, and instability in the flooded state.

When the bed is earthy its usual condition is either *just stable and no more*, or *permanently unstable.* The former of these conditions arises from the fact of the stream carrying earthy matter in suspension, so that the bed consists of particles which are just heavy enough to be deposited, and which any slight increase of velocity would sweep away.

The bottom of a river in a permanently unstable condition presents, as Du Buat pointed out, a series of transverse ridges, each with a gentle slope at the up-stream side and a steep slope at the down-stream side. The particles of the bed are rolled by the current up the gentle slope till they come to the crest of the ridge, whence they eventually drop down the steep slope to the bottom of a furrow, where they become covered up, and remain at rest till the gradual removal of the whole ridge leaves them again exposed.

When the banks, as well as the bottom, are unstable, the river-channel undergoes a continual alteration of form and position. If the banks are straight, they soon become curved, for a very slight accidental obstacle is sufficient to divert the main current so that it acts more strongly on one bank than on the other: the former bank is scooped away, and becomes concave, and the earthy matter suspended in the stream is deposited in the less rapid part, so as to make the opposite bank convex. A curved part of a river-channel tends to become continually more and more curved; for the centrifugal force (or rather the tendency of the particles of water to proceed in a straight line) causes the particles of water to accumulate towards the concave bank; the current is consequently more rapid there than towards the convex bank, and it scoops away both the bank and the bottom (unless they are able to resist it), and deposits the material in some slower part of the stream: thus the *line of the strongest current* is always *more* circuitous than the centre line of the channel; and the action of the current tends to make the concave banks more concave, the convex banks more convex, and the whole course of the river more serpentine. This goes on until the current meets some material which it cannot sweep away, or until, by the lengthening of the course of the stream and the consequent flattening of its declivity, its velocity is so much reduced that it can no longer scoop away its banks, and stability is established. In some cases stability is never established; but the river presents a serpentine channel which continually changes its form and position.

One of the chief objects of engineering, in connection with the channels of streams, is to protect their banks against the wearing action of the current, so as in some cases to give them that stability which they want in their natural condition, and in other cases to give them the additional stability that is required in order to resist an increased velocity of current, produced by improvements in the course and form of the channel.

469. **Protection of River-Banks.**—The most efficient protection to the banks of a stream is a thick growth of water-plants; but as these form a serious impediment to the current, artificial protection must be substituted for them, at least below the average water-level. Above that level a plantation of small willows forms a good defence against the destructive action of floods; but it is not applicable where there is a towing-path. The means of artificially protecting river-banks may be thus classed:—I. Fascines. II. Timber sheeting. III. Iron sheeting. IV. Crib-work. V. Stone pitching. VI. Retaining walls. VII. Groins.

I. *Fascines*, already referred to in Article 417, p. 625, are bundles of willow twigs from 9 to 12 inches in diameter: the largest are about 20 feet long, but 12 feet is a more common length: they are tied at every 4 feet, or thereabouts. For the protection of a river-bank *below the low water-level* an "apron" or "beard" is laid, consisting of fascines lying with their length up and down the slope of the bank; the upper ends are fastened down to the bank with stakes about 4 four feet long; the lower ends are sunk, and held down under water by loading them with stones. To protect the bank *above the low water-level* fascines are laid horizontally in layers, with their butt ends towards the stream, so as to form a series of steps rising at the same rate with the slope of the lower part of the bank, or nearly so (say from 1 to 1 to 3 to 1); each layer is fastened down with three rows of stakes 4 feet long; the heads of the stakes rise 8 inches or thereabouts above the fascines, and are laced or wattled with wicker-work, so as to form a crib for the retention of a layer of gravel.

Fascines usually last 6 years above the low water-level and 10 years below.

II. *Timber Sheeting* may consist either of sheet-piles (already described in Article 404, p. 605) or of guide-piles and horizontal planks, described in Article 409, p. 613. The wales of the sheet-piling or the guide-piles of the planking must be tied back to anchoring-plates made of planks buried in a firm stratum of earth at a sufficient distance back from the bank. The holding power of such anchoring-plates depends on the same principles as that of iron anchoring-plates, as to which, see Article 272, p. 410.

III. *Iron Sheeting* has already been described in Article 404, p. 606. It is sometimes used for the faces of quays in navigable rivers, being tied back to anchoring-plates. (Article 272, p. 410.)

IV. As to *Crib-work*, see Article 409, p. 614. When used for a quay or river-bank its interstices are rammed full of clay and gravel.

V. *Dry Stone Pitching* is used to protect earthen banks, of slopes ranging from that of 1 to 1 to that of 2 to 1, or flatter. It consists of stones roughly squared, and laid by hand in courses. Its thickness is usually from 8 to 12 inches at the top, and increases in going down at the rate of 2 or 3 inches per yard. The foot of the pitching must abut against a foundation sufficient to prevent it from slipping. Such a foundation may be made by sinking a row of oblong baskets, each containing about 2 cubic yards of gravel, or by driving a row of piles with horizontal wales at the inner side of their heads; the strength of the wales is a matter of calculation; they have to resist a maximum pressure = weight of pitching × rise of slope ÷ length of slope, the friction of the pitching on the earth being neglected for the sake of security.

VI. *Retaining Walls* are used chiefly where quays are required, and will be again mentioned further on.

VII. *Groins* are small dykes projecting at right angles to the bank to be protected, and are made either of loose stones, of piles and planks, or of wattled stakes. Each groin protects a portion of the bank of about *five* times its own length, and usually causes the current that sweeps round its point to scoop out an excavation in the bottom of the channel of a breadth equal to about one-quarter of the length of the groin, the material scooped out being deposited in the space between the groins. Groins, besides being an obstruction to the current, are injurious to the regularity of figure and stability of the bottom of the channel, and should only be used as a temporary expedient to protect the banks, until works of a better description can be completed.

470. **Improvement of River-Channels.**—The defects in a river-channel which are to be removed by improvements are usually of the following kinds:—The channel may be too shallow, either generally or in particular places; it may be too narrow, either generally or in particular places; it may even in particular places be too wide, if the breadth is so great as to cause the formation of shoals by enfeebling the current; its declivity may be too flat, either from the existence of obstacles, such as shoals, islands, weirs, ill-constructed bridges, or the like, or from its course being too circuitous; occasionally, but rarely, the declivity may be too steep at particular places, giving rise to a current so rapid as to make it impossible to preserve the stability of the bed; but this defect generally arises from the declivity being too flat elsewhere; it may contain sharp turns, injurious to the stability of the banks; it may be divided into branches, so as to enfeeble the current.

Setting aside for the present *diversions of the course* of a river, which will be considered in the next article, the works for the improvement of the channel consist mainly of:—I. Excavations to remove islands and shoals, and widen narrow places. II. Regulating dykes, to contract wide shallows. III. Works for stopping useless branches.

Before commencing alterations of any kind in a river-channel careful calculations should be made, according to the principles explained in Section I. of this chapter, of the probable effect of such alterations on the level, declivity, and velocity of the current in different states of the river. The object kept in view should be to obtain a channel either of nearly uniform section, or of a section gradually enlarging from above downwards, with a current that shall be sufficient to discharge flood-waters without overflowing the banks more than can be avoided, and at the same time not so rapid as to make it difficult or impossible to preserve the stability of the channel.

All improvements of river-channels should be begun at the lowest point to be altered, and continued upwards; because every improvement takes effect on the parts of the stream above it.

I. *Excavation* under water, by hand dredging, machine dredging, and blasting, has been described in Article 410, p. 614. When the current is at a low level, it may occasionally be advantageous to excavate parts of the bed by enclosing them with temporary dams as if for foundations (Article 409, p. 611), and laying them dry. Excavation of a muddy, sandy, or gravelly bottom, by the aid of the current, is performed by mooring at the place to be deepened a boat, furnished with a transverse projecting frame covered with boards or canvas; this frame descends to within 3 or 4 inches of the bottom of the channel, and the current, forced through that narrow opening, scoops out the material and sweeps it away. From 30 to 70 cubic yards per day have been excavated in this manner with a single boat.

II. *Regulating Dykes* should be adopted with great caution, and only where the excessive width of the channel is an undoubted cause of shallowness. They should not in any case rise much above the low water-level, lest they contract too much the space for flood-waters. They may be built either of dry stone, with a slope of about 1 to 1, or of wattled piles and gravel. The ordinary rules for the construction of dykes of the latter kind are as follows:—The piles in a double row to be driven into the ground to a depth equal to twice the depth of water; their diameter not less than 1-20th of their length; their distance apart longitudinally to be equal to the depth of water; the distance transversely between the rows of piles to be once and a-half the depth of water. They are to be tied together transversely, and wattled with willow twigs, and the space between the two rows filled with gravel.

III. The *Stopping of Branches* should be performed at their upper ends. In a gentle current it may be effected by means of an embankment of stones and gravel, advancing simultaneously from the two banks until it is closed in the centre; in a more rapid stream a dyke of wattled piles and gravel, made as already described, may be used; should the current be too strong for either of these plans, a raft, boat, or caisson (Article 409, Division III., p. 613), or a crib-work dam (Article 409, Division IV., p. 614), loaded with stones, is to be moored across the stream and sunk. The branch channel having had its current stopped will silt up of itself.

471. **Diversions of River-Channels** are usually adopted for the purpose of rendering the course less circuitous. In designing them regard should be had to the principles already explained in Section I. of this chapter, and in the preceding articles of this section; and care should be taken not to make the course *too direct*, lest the current be rendered too rapid for the stability of the bed. A slightly curved channel is always better than a straight channel; because in the former the main current takes a definite course, being always nearest the concave bank; whereas in a straight channel its course is liable to keep continually changing.

The form of cross-section with a horizontal base and sloping sides which gives the least friction with a given area has already been described in Article 451, p. 688, and it may be adopted if the stream is to act solely as a conduit for the conveyance of water; but should it be navigable, a figure must be adopted suited to the convenience of the navigation. This will be further considered in Chapter III. of this part.

472. A **Weir** is an embankment or dam, usually of stone, sometimes of timber, constructed across the channel of a stream. As to its effect on the water-level, see Articles 452 and 453, pp. 689 and 670.

When erected for purposes of water-power or water-supply, the object of a weir is partly to make a small store reservoir, but principally to prolong a high top water-level from its natural situation at a place some distance up the stream, to a place where water is to be diverted from the stream to drive machinery, or for some other purpose. When erected for purposes of navigation, the object of a weir is to produce a long reach or pond of deep and comparatively still water, in a place where the river is naturally shallow and rapid.

In planning a weir three things are to be considered: its line and position, its form of cross-section, and its construction.

I. *Line and Position of a Weir.*—It is best to avoid sharply curved parts of a river-channel in choosing the site of a weir, lest the rapid current which rushes down its face in times of flood should undermine the concave bank. For the protection of the banks in any case, it is advisable so to form the weir that the cascade from the lateral parts of the crest shall be directed from the banks, and towards the centre of the channel. This may be effected either by making the weir slightly curved in plan, with the concavity at the down-stream side, or by making it like a V in plan, with the angle pointing up stream. Another mode of protecting the banks is to make the crest of the weir slightly higher at the ends than in the middle, so that the lateral parts of the cascade may be too feeble to do damage.

In order to diminish the height and extent of backwater during floods, the crest of the weir is often made considerably longer than the breadth of the channel; this is effected either by making it cross the channel obliquely, or by using the V-shape already described, the latter method being the best for the stability of the banks. The practical advantage of such increased length is doubtful.

II. *Form of Cross-section.*—The back or up-stream side of a weir is usually steep, ranging from vertical to a slope of about 1 to 1; the top is either level or slightly convex, and not less than about 2 or 3 feet broad. In designing the front or down-stream slope of a weir, the principal object is to prevent the cascade that rushes over it from undermining its base. The commonest method is to use a long flat slope of 3 to 1, 4 to 1, or 5 to 1, in order that the speed of the current may be diminished by friction, and that it may strike the bottom of the channel very obliquely. A further protection is given to the river-bed by continuing the front slope a short distance below the bottom of the channel, and then curving it slightly upwards. Another method is to make the front of the weir present a steep or nearly vertical face, over which the water falls on a nearly level apron or pitching of timber or stone. Probably the best method would be to form the front of the weir into a series of steps, presenting steep faces and flat platforms alternately, the general inclination being about 3 to 1; thus a great fall might be broken up into a series of small falls, each incapable of damaging the platform which receives it.

III. *Construction.*—In order that the water of the pond may not force its way under the base of a weir, or round its "roots" (as the ends which join the banks of the stream are called), its foundation should be examined, chosen, and formed with precautions similar to those used in the case of a reservoir embankment, as to which, see Articles 461 and 463, pp. 701 to 704.

To make a weir of *timber*, or of timber, stones, and clay combined, any of the methods may be employed which have been described under the head of "Dams," in Article 409, Divisions II., III., and IV., with the addition that the back, crest, and front of the dam are to be covered with planking laid parallel to the current, to form an overfall for the water; and that the bottom of the channel at the foot of the weir is to be protected either by a platform of planks resting on a timber grating or on piles, or by a stone pitching.

A weir of *fascines* may be built of horizontal layers of fascines, staked down with mixed clay and gravel packed between them, in the manner described under the head of the protection of river-banks, Article 469, p. 710, the crest, front, and foot of the dam being protected with an apron of fascines, like that described in the same article.

A *dry stone* weir is formed like the stone embankments mentioned in Article 412, p. 617, with a steep slope at the bank and a long gentle slope in front, pitched or faced with roughly squared stones set in courses, as in the pitching of a river-bank, Article 469, p. 711. Sometimes a skeleton crib of timber, consisting of piles and longitudinal and transverse horizontal wales is constructed in order to keep the stones of the pitching in their places. As to the pressure against the longitudinal wales, see the article just quoted.

A weir of *solid masonry* may be founded, like other structures under water, on the natural ground, on a bed of concrete, on a timber platform, or on piles, according to circumstances. (See Part II., Chapter VI., Section II., p. 601.) When it has a timber foundation, a row of sheet-piles at the base of the up-stream side will in general be necessary to prevent the passage of water under it; and in the grating of the platform, pieces of timber running continuously through the weir in the direction of the stream should be avoided, lest they should conduct water along their sides. The masonry should be built in cement, or in quickly-setting hydraulic mortar; the heart of the weir may be of coursed rubble, or of concrete laid in layers; but the facing should be of good block-in-course, or of hammer-dressed ashlar, and the crest should form a coping of large stones, all headers, dowelled to each other.

One of the most effectual ways of preventing filtration round the "roots" of a weir is to carry them a considerable distance into the bank; but in the case of a weir of masonry the ends often abut upon a pair of side-walls, running along the banks of the stream, and having counterforts behind them to interrupt filtration.

IV. *Appendages of a Weir—Sluices and Floodgates—Salmon-*

stair.—When a weir is built across a navigable river, it requires a lock for the passage of vessels, which will be again mentioned further on. It may have one or more outlets with valves, like those of a reservoir embankment (Article 464, p. 705), according to the purpose for which it is intended.

It is almost always necessary to provide a weir with waste-sluices or floodgates, to be opened when the river is high, in order to prevent too great a rise of backwater. A sluice is a sliding valve of timber or iron, moving in guides, which are in general vertical, set in a rectangular passage of timber or masonry, and opened and shut by means of a screw, or of a rack and pinion. It is advisable not to make any sluice wider than about 4 or 5 feet. Should a greater width of opening be required, the passage through the weir is to be divided by walls or piers into a sufficient number of parallel passages, each furnished with a sluice. As to the discharge through a sluice, see Articles 448, 449, p. 681.

Another mode of opening and closing floodgates in a weir is by means of *needles,* as they are called. A rectangular channel through the weir is crossed at the bottom by a fixed timber sill, and near the top by a moveable timber sill, resting in two notches. The strength of the sills is a matter of calculation: they have to withstand the pressure of the water on a flat surface closing the passage. That surface is made up of the "needles," which are a set of square bars of wood strong enough to withstand the pressure, which are ranged close together side by side in a vertical position at the up-stream side of the sills. Each needle has a cylindrical handle at its upper end, to hold it by in removing and replacing it.

As to *self-acting waste-sluices,* see *A Manual of the Steam Engine and other Prime Movers,* Article 139, p. 153.

A weir across a river frequented by salmon requires a passage or channel to enable those fish to ascend its front slope. Mr. Smith of Deanston introduced the practice of making that channel of a zig-zag form, so as to reduce its rate of declivity and bring the speed of the current in it within moderate limits.

A *moveable weir* consists in general of a water-tight planked timber gate, placed in a rectangular passage of masonry or timber, and capable of turning upon a horizontal hinge at the floor of the passage, so as to be either laid flat when the channel is to be left clear, or set at any required angle of elevation, sloping against the declivity of the stream, with oblique struts to prop it at the down-stream side. In one ingenious modification of this weir the duty of the struts is performed by a second and smaller gate, also turning on a horizontal hinge at the floor of the passage, but so as to slope *with* the stream. When the passage is clear, both gates lie flat in a horizontal recess in the floor of the passage, the smaller gate undermost and the upper surface of the larger gate flush with the floor. When the weir is to be raised, water is admitted through a valve and culvert from the up-stream side of the weir passage into the recess below the gates; its pressure lifts them both until they form a weir of a triangular section, the larger gate making the up-stream slope and the overfall, and the smaller making the down-stream slope, and acting at the same time as a strut to prop the larger gate. When the weir is to be lowered, the mass of water contained below the gates is allowed to escape by opening a valve in a culvert which leads to the down-stream side of the weir; and both gates then fall flat into the recess of the floor.*

473. **River Bridges.**—The construction of the foundations on land and in water, and of the superstructures, of bridges of various materials having been explained in Part II. of this work, and their adaptation to roads and railways in the preceding chapter, it is now only necessary to state those principles which are specially applicable to bridges over rivers.

In choosing the site of a bridge which is to have piers in the river, sharply curved parts of the channel should be avoided, lest the increased rapidity of the current caused by the narrowing of the water-way should undermine the concave bank.

The current should be crossed at right angles, or as nearly so as practicable. The abutments should not contract the water-way.

The piers, if any, should stand with their length exactly in the direction of the current; they should have pointed or cylindrical cutwaters at both ends, to diminish the obstruction to the current which they produce; and they should be no thicker than is necessary for the safety of the bridge. (As to stone piers in particular, see Article 293, p. 428.)

The springing of the arches should be above the highest ordinary water-level, and as much higher as the convenience of the navigation may require; and care should be taken that sufficient water-way is provided for the greatest floods. The crown of the lowest arches should be at least three feet above the flood-level, that they may allow floating bodies to pass through.

It may here be observed that the figure of arch which gives the greatest water-way for a given rise and span is the "hydrostatic arch." (See Article 283, p. 419.)

* In order to do away as far as possible with the obstruction occasioned by weirs, it has been proposed by Hugh Mackenzie, Esq. of Ardross, that in those cases in which the fall of the stream is sufficiently rapid, and the country in other respects suitable, the diversion of water from a stream for the purpose of obtaining power should be effected by making a tunnel with suitably formed grated apertures in its roof, under the bed of the stream, at a point where its water-level has sufficient elevation, and so conducting the water into a mill-lead of sufficiently large size and moderate declivity.

Should it appear, upon an examination of the land subject to inundation at and near the site of the intended bridge, that such land acts not merely as a reservoir for flood-waters, but as a wide temporary channel for their discharge, that land should be crossed by a viaduct, and not by embanked approaches.

In designing a bridge for carrying an ordinary road over a river, it is usual, in order to obtain the greatest headroom possible consistent with economy in forming the approaches, to give the roadway an ascent from the ends of the approaches to the middle of the bridge, at a rate not exceeding the ruling gradient of the road; and to suit the arches, when there are more than one, to the form of the roadway, the centre arch is made the largest, and the others gradually diminish in size towards the ends of the bridge. They should, at the same time, be so proportioned as to exert as nearly as possible equal horizontal thrust.

Swing bridges for navigable rivers will be again mentioned further on.

Ice-breakers are required for the protection of the piers of bridges across rivers which bring down large masses of ice.

A *stone ice-breaker* usually forms part of the up-stream cutwater of the pier to which it belongs, presenting to the current a ridge sloping at about 45°, up which the flat sheets of ice slide, and break asunder by their own weight. Examples of such ice-breakers are shown in the view of the Victoria Bridge, fig. 249, p. 533.

A *timber ice-breaker* stands usually separate from the pier which it protects, at a short distance up-stream. The sloping ridge is formed by a beam of 12 or 14 inches square, covered with sheet iron. Its base consists of piles, ranged in the form of a long sharp triangle with the point up-stream, connected with the ridge by a strong framework of uprights and diagonals, which are protected against the ice by projecting horizontal wales.

(On the subject of river bridges, see Telford's and Smeaton's *Reports*, and the work *On Bridges* by Mr. Hosking and others).

474. **Artificial Water-Channels — Conduits.**—In laying out and designing artificial water-channels it is advisable, if possible, so to fix the declivity with reference to the length, that the velocity shall not be less than about one foot per second (lest the conduit silt up), nor greater than about four feet per second (lest the current should sweep stones along, and injure the bed).

As to the larger-sized artificial water-channels, and as to those of all sizes which are merely to be used as open drains, when they are wholly in cutting, it is unnecessary to add anything to what has already been stated respecting river-channels, and especially respecting their diversions, Article 471, p. 713. Artificial earthen channels in embankment will be considered under the head of canals.

When a channel is to convey water for the supply of a town, it is usual, with a view to the clearness and purity of the water, as well as to the preservation of the channel, to line it throughout with brick or stone built in cement; and in most cases it is necessary to cover it also, especially if it traverses districts where the air is smoky and otherwise impure. When brick or porous stone is used, the water-way may be lined throughout with a coating of cement, calcareous or asphaltic.

The water-way of a *stone or brick conduit* should be made of one of those forms which give the greatest hydraulic mean depth for a figure of given class and a given area; that is to say, the semicircle, the half-square, or the half-hexagon, already referred to in Article 451, p. 688. To preserve a constant definite flow it may have a series of waste-weirs along its sides, placed in positions where there are convenient channels at hand for discharging the waste water. Should it be necessary to carry it along an embankment, that embankment should be formed in thin layers, each well rammed, and should if possible contain a large mixture of stones with the earth; the breadth at the top should be from 4 to 6 feet at each side of the conduit, so that the total breadth at the brink of the conduit will be = breadth of water-way + from 8 to 12 feet, and the masonry of the conduit should be imbedded in puddle or in hydraulic concrete.

The best form for a *covered conduit* to convey a constant flow, as for the supply of a town, is cylindrical. To guard it against frost it should be completely covered with earth to the depth, in Britain, of about 3 feet, the bank being faced with sods. When it forms a tunnel, or is placed in deep cutting and covered with earth, its strength is regulated by the principles of Article 297 A, p. 433.

One of the largest cylindrical conduits yet executed is that of the Loch Katrine Water-Works, 8 feet in diameter.

A covered conduit should be provided, like a tunnel, with grated ventilating shafts, which will also serve to admit men for the purpose of repairing it.

When the flow varies very much, as in sewers, an egg-shaped section with the small end down is preferred.

A recent invention in conduits is that of Mr. Richardson, in which a cylinder of sheet iron is lined with brickwork in cement. It is suitable for making large conduits possessing great strength and stability with a moderate quantity of materials.

475. **Junctions of Water-Channels.**—In all cases in which a pair of water-channels join together into one, their centre lines, if

possible, should be a pair of curves, or a curve and a straight line touching each other at the junction; or should an angle at the junction be unavoidable, that angle ought to be as acute as possible. This principle applies also to the *divergence* of a branch from a main channel, and to pipes as well as to free channels.

476. **Aqueduct Bridges** differ from viaducts only in supporting a water-conduit instead of a road or a railway, and the mechanical principles of their construction involve nothing that has not been already explained in the Second Part of this treatise.

The water conduit or trough is usually of the same material with the rest of the structure. For example, in a stone aqueduct the conduit is of masonry, imbedded in a mass either of puddle or of concrete, resting on the arch and contained between the external spandril walls.

In some recent examples of wrought iron aqueducts introduced by Mr. Simpson, the water-channel has been made self-supporting by constructing it as a plate iron tubular girder of oval section. In this case the interior of the tube should be smooth, that it may offer no impediment to the current. All T-iron stiffening-ribs, &c., should project outside only.

Pipe-aqueducts will be mentioned further on.

477 **Water-Pipes.**—The diameters of water-pipes are fixed with reference to the vertical declivity and the intended greatest discharge, according to the rules explained in Article 450, p. 684. The materials principally used in making pipes for the conveyance of large quantities of water are earthenware and iron.

I. *Earthenware Pipes* are of various qualities as to texture, from a porous material like that of red bricks, to a hard and compact material, which is glazed to make it water-tight. They are made of various diameters, from 2 inches to nearly 3 feet, and in lengths of from 1 foot to 3 feet. The harder kinds have considerable tenacity, and are capable of bearing the dead pressure of a high column of water; but they are so easily broken by sharp blows and sudden shocks that it is not advisable to expose them to high pressures in situations where their bursting might cause damage or inconvenience. Hence their chief use is as *small covered conduits* for purposes of drainage. Their joints are most commonly of the spigot and faucet form, being made tight, if necessary, with cement, or with a bituminous mastic. (Article 234, p. 376.) Another form, very useful to facilitate laying and lifting is the *thimble-joint*. The lengths of pipe are plain hollow cylinders, and the thimble is a ring embracing and loosely fitting the adjoining ends of a pair of lengths. Sometimes the thimble is in two semicircular halves; and sometimes each pipe has on one end a half-faucet, which is laid downwards; the end of the adjoining pipe rests in the half-faucet, and the joint is completed by a half-thimble above. Curved and acute-angled junction-pieces are made: so also are right-angled junction-pieces; but these last should never be used.

II. *Cast Iron Pipes* should be made of a soft and tough quality of cast iron. (See Article 353, p. 499.) Great attention should be paid to moulding them correctly, so that the thickness may be exactly uniform all round. Each pipe should be tested for air-bubbles and flaws by ringing it with a hammer, and for strength by exposing it to double the intended greatest working pressure.

Cast iron water-pipes are made of various diameters or bores, from 2 inches to 4 feet.

They are usually moulded and cast horizontally, the sand core being supported by a strong horizontal bar with projecting teeth; but advantages in point of accuracy and soundness are possessed by the process of casting them vertically, the faucet being turned downwards, and the plain end upwards.* The pipe is cast with an additional length at the upper end, which acts as a *head* (Article 354, p. 503), compressing the mass below, and receiving the air-bubbles; this *head* is afterwards cut off.

The rule for computing the thickness of a pipe to resist a given working pressure (the factor of safety being *six*) has already been given in Article 150, equation 2, p. 228, the pressure and the tenacity of the iron being expressed in lbs. per square inch; but as it is more convenient to express those quantities in *feet of water*, the following rule is given:—

$$\frac{\text{thickness}}{\text{diameter}} = \frac{\text{greatest working pressure in feet of water}}{12{,}000}. \quad (1.)$$

There are limitations, however, arising from difficulties in casting, and from the fact that the most severe strain on a pipe is often produced by shocks from without, which cause the thickness of cast iron pipes to be often made considerably greater than that given by the above rule. The following empirical rule expresses very accurately the *limit to the thinness of cast iron pipes*, in ordinary practice:—

The thickness of a cast iron pipe is never to be less than a mean proportional between its internal diameter and one-forty-eighth of an inch.

It is very seldom, indeed, that a less thickness than 3-8ths of an inch is used for any pipe, how small soever.

Cast iron pipes are made of various lengths; but the most common length is 9 feet, exclusive of the faucet or socket on

* Introduced by Mr. D. Y. Stewart.

one end of each length, for receiving the plain end of the next length. The faucet adds from one-twentieth to one-tenth to the weight of the pipe. The joints are sometimes run up with melted lead, sometimes turned so that the plain end and the faucet fit exactly, and made water-tight with red lead paint. The latter is the easier and quicker process; but the former admits of a greater amount of yielding to expansion and contraction, and to the unequal settlement of the ground, which is an advantage in point of safety.

III. The best *preservative* for cast iron pipes against corrosion is a coating of pitch, applied both inside and out, by a process which makes it penetrate the pores of the iron to a certain extent, and adhere very firmly. This coating appears to diminish sensibly the friction of the water.

IV. In estimating the greatest working pressure which a water-pipe should be capable of resisting, the *hydrostatic pressure* due to the whole depth below top-water of the reservoir whence the supply enters the pipe, and not the mere *hydraulic pressure* when the water is in motion (Article 446, p. 675), should be taken into account, in order to provide for the contingency of the flow of the water being checked by an obstruction in the pipe.

V. The *loss of head* during the most rapid discharge should be computed for a series of points in the course of an intended pipe by the principles explained in the First Section of this chapter, so as to determine the *line of virtual declivity*, which will commence at a point vertically above the mouthpiece of the pipe, and at a depth below the top-water of the reservoir equal to the loss of head due to the velocity of flow in the pipe and the friction of the mouthpiece. The object of determining that line is to insure that in laying out the levels of the pipe *no part of it shall be made to rise above the line of virtual declivity.* The reason for this rule is, that at all points in a pipe which are above that line, the pressure, when the water is flowing, becomes less than that of the atmosphere (a fact commonly described by saying that there is a "partial vacuum," see Article 443, p. 673); in consequence of which the air, which all water contains in a diffused state, escapes from the water in bubbles, and eventually accumulates in the highest part of the pipe so as to obstruct the flow of the water.

A pipe thus rising above the line of virtual declivity is called a *siphon,* and is incapable of continuously conveying water unless the air be from time to time exhausted from the summit of the pipe.

Air collects to a certain extent at the *summits* of an undulating pipe even when they are below the line of virtual declivity; but as it exerts a pressure greater than that of the atmosphere, it is easily expelled. A small cylindrical receiver, called an *air-lock,* is placed above the pipe at each such summit, to collect the air, which is from time to time discharged through a valve. That valve may either be opened by hand occasionally, or it may be loaded with a weight equivalent to the hydraulic pressure, and made self-acting.

VI. At the *lowest points* in an undulating line of water-pipe sediment collects, and is to be discharged from time to time through a *cleansing* or *scouring* cock or valve.

VII. As to *slide-valves, double-beat-valves,* and other valves and cocks used in connection with water-pipes, see *A Manual of Prime Movers,* Article 116, p. 120, and Articles 119 to 123, pp. 123 to 126.

VIII. *Sheet Iron Water-Pipes* lined with pitch have lately been used in France.

478. **Pipe-Track—Pipe-Aqueducts.**—Care should be taken to bed water-pipes on a firm foundation, and to cover them to a sufficient depth to prevent the action of frost; that is, in Britain, about 2 or 3 feet.

When a water-pipe crosses a valley, or a river-channel, or a line of communication, it may sometimes be advisable to carry it above ground by means of an aqueduct. This may be a bridge of any convenient construction, or it may consist simply of the pipe itself lying on a series of piers, and cased outside with wood, or other non-conducting material, for protection against heat and cold. For a pipe-aqueduct of wide span, the pipe itself may be made to form a catenarian arch.*

The total thrust at the springing of the arch under an uniform load is to be computed in the usual way, being,

load per foot of span × radius of curvature at crown in feet × secant of inclination at springing;

from which has to be deducted the thrust borne by the water, viz.,

pressure of water × sectional area of pipe;

and the *remainder* only of the thrust has to be borne by the iron of the pipe. In fact, the *arch of water* bears a part of the load.

If the arched pipes be made to carry a roadway, the whole of the stress produced by a partial or travelling load will fall on them; and their strength is to be computed by the formulæ of Article 180, Problems IV. and V., pp. 303 to 308, as explained in treating of cast iron arched ribs, Article 374, Case I., p. 539.

The *wooden lining* referred to as a protection against frost

* Of this there is an example on the Washington Water-Works, designed by General Meigs of the United States' Engineers. The arch is of 200 feet span, and consists of two parallel cast iron pipes of 4 feet diameter.

consists of oaken staves about 3 inches thick, packed in a cylindrical form round the interior of each pipe. It is likely to prove more lasting than an outside casing, because it is constantly wet, instead of being alternately wet and dry.

SECTION V.—*Of Systems of Drainage.*

479. **General Principles as to Land Drainage.**—The engineer who examines a district with a view to the improvement of its drainage requires the information respecting the features, extent, and levels of the district, its rain-fall, and the course, dimensions, levels, and discharge of its streams, which have already been specified in Articles 456, 457, and 458, pp. 692 to 699, and in Article 467, p. 707. In some cases it is necessary to attend to the question, whether the water to be carried off by the system of drainage comes merely from the apparent gathering-ground bounded by the ridges that surround the district, or whether some of it is brought to the district through porous strata, which have their gathering-ground wholly or partly beyond such ridges.

In order that a district may be in a perfect state as to drainage, the water-level in the branch drains, which directly receive the discharge of the field drains, should be at least about 3 feet below the level of the ground at all times. When it rises above that level the ground becomes *awash* or *flooded*, according as the water-level is below or above its surface.

Each water-channel must have sufficient area and declivity, when at its fullest flow, to discharge all the water that it receives as fast as such water flows in, without its water-level rising so high as to obstruct the flow of the branches it receives, or to lay land awash.

Should it be impossible absolutely to fulfil these conditions, means are to be taken to make the deviation from them as small in extent and as short in duration as possible.

480. **Questions as to Improvement of Drainage.**—Should the drainage of a district be found defective, the engineer will in general have to consider questions of the following kind, as to the causes of such defective condition, and the means of improving it:—

I. Whether, and to what extent, it is practicable to diminish or prevent floods by the construction of store reservoirs.

II. Whether the channels of the streams contain *removable obstructions* such as shelves of rock or other shallows, narrow places, islands, ill-designed weirs and bridges, &c., and how such obstructions are to be removed. This may involve questions as to rebuilding weirs and bridges according to improved designs.

III. Whether the channels are defective and liable to be obstructed through the instability of their beds, and how such instability is to be prevented.

IV. In the case of a smaller stream having too little declivity, which falls into a larger stream, whether that declivity can be increased by diverting the course of the smaller stream so as to remove its outfall to a lower part of the larger stream.

V. Whether the course of a stream, being too circuitous, can be improved by a diversion; and whether, in the event of improvements being required in the channel of a stream, it is best to execute them in the existing channel, or to make a new channel, independently of the question of circuitousness.

All the preceding questions relate to matters which have already been treated of in Sections III. and IV. of this chapter, but the following involve subjects which will be treated of in the ensuing articles:—

VI. Whether the branch drains are of sufficient discharging capacity.

VII. To what extent the water-channels are capable of acting as temporary reservoirs for moderating the rapidity with which flood-waters descend from them into lower and larger channels.

VIII. To what extent the lands adjoining a river which are liable to inundation act in the capacity of a reservoir, and what will be the effect upon the part of the river below them of preventing or diminishing such action.

IX. Whether the drainage can be sufficiently improved by improvements on the water-channels alone, or whether, on the other hand, it is advisable to use embankments for the confinement of floods within certain limits.

481. **Discharging Capacity of Branch Drains.**—If the rain-fall found its way at once from the surface of the ground to the drains, each of these would require to have dimensions and declivity sufficient to discharge the most rapid fall of rain known to take place for any time how short soever. The following data as to the most rapid rain-fall in Britain are given on the authority of Mr. Phillips; they illustrate how the greatest *rate* of rain-fall diminishes according as the period for which it is reckoned is increased:—

Period.	Total depth of Rain-fall. Inches.	Rate of Rain-fall. Inches per Hour.
One hour,	1	1·0
Four hours,	2	0·5
Twenty-four hours,	5	0·2 nearly.

The soil, however, acts as a sort of reservoir to an extent depending on its texture; it keeps from the drains altogether a portion of the rain-fall, which passes off by evaporation, or is absorbed by plants, as stated in Article 456, p. 692; and it discharges the remainder into the drains more or less gradually. The branch drains in country drainage should be made capable of discharging at an uniform rate the greatest *available* rain-fall known to take place in a period whose length is greater according as the soil is more retentive. It is probable that in most cases of cultivated land *twenty-four hours* will be found a sufficiently short period: that is, each drain which directly receives water from the fields should be capable of discharging, in twenty-four hours, the greatest available rain-fall of twenty-four hours; for steep and rocky ground the period must be shortened, in some cases, it is probable, to four hours; but the best method in each case is to ascertain the period by an experimental comparison of the rain-fall with the discharge of drains.

482. **Action of Channels and Flooded Lands as Reservoirs.**—The volume of the space contained between the ordinary water surface of a given portion of a stream and the flood-water surface, whether such space be wholly contained between the banks of that portion of the stream, or partly between such banks and partly over adjoining lands liable to inundation, constitutes a reservoir for retaining *the excess of the total supply of water during a period of flood rain-fall from the district drained by that portion of the stream, above the greatest quantity that the stream is capable of discharging in the same period*, until the flood rain-fall is over, when that excess flows away by degrees. The existence of that reservoir-room thus renders sufficient a water-channel of less discharging capacity than would otherwise be necessary; and if such reservoir-room is diminished, either by improving the channel so as to lower the flood-water surface, or by contracting the space by means of embankments, care should be taken that the discharging capacity of the channel *below* the district in question is increased to a corresponding extent, otherwise the effect of diminishing the extent of floods in that district may be to increase it in some district further down the river. This is one of the reasons for the rule already stated in Article 470, p. 712, that works of river improvement should proceed from below upwards.

483. **River Embankments.**—When the land adjoining a stream cannot be sufficiently guarded from inundation by improvements in the channel, embankments may be erected. In determining the course and site of such embankments regard must be had to the principle stated in the last article—of leaving sufficient reservoir-room between them for flood-water. In some cases there may be sufficient room even when the embankments are erected close to the natural banks of the channel; but in general it is advisable to

leave a wider space; and when the river follows a serpentine course sufficient reservoir-room may in many cases be provided by carrying the embankments along the general course of the valley, so as to enclose the windings of the stream without following them, and thus to form not only a reservoir, but a wide and direct channel for the discharge of floods.

The tributary streams which flow into the main streams will in general require branch embankments. Where a main embankment extends for a long distance uninterrupted by a tributary stream, the land protected by it is often divided into portions by means of branch embankments, called "*land arms*," diverging from the main embankment, the object of which is, that, in the event of a breach being made in the main embankment, the inundation may be confined to a limited extent of ground. These "land arms" generally run along the boundaries of separate holdings.

Behind and parallel to each main embankment there runs a "*back drain*," the material dug from which, if suitable, may be used in making the embankment. The use of this back drain is to act not only as a channel for the drainage of the land protected by the embankment, but as a reservoir to collect that drainage when the river is in a state of flood, and its dimensions are to be regulated accordingly. The waters of the back drain are discharged into the river (when its surface is low enough) through a series of pipes traversing the embankment, and having flap-valves opening outwards to prevent the return of water from the river. These valves are made sometimes of iron, sometimes of wood; one of the most efficient consists of an iron grating or perforated plate, covered with a flap of vulcanized indian-rubber. As to the computation of the time required to discharge a given accumulation of water from the back drain through a given outlet, see Article 455, p. 691.

The embankments are to be made of clay rammed in layers one foot deep, or thereabouts. When of moderate height, and not exposed to great pressure, they may have slopes of $1\frac{1}{2}$ to 1 or 2 to 1. When they are liable to be acted upon by a strong current they should be pitched with stone, or otherwise defended like river-banks (Article 469, p. 710): elsewhere they should be covered with sods, and no trees, shrubs, or hedges should be suffered to grow upon them.

484. **Tidal Drainage** is the drainage of lands which are above the low-water-mark of ordinary tides, and either below high-water-mark, or so near that level that their drainage waters can only be discharged in certain states of the tide. Such lands are defended against inundation by the sea by means of embankments, which will be treated of further on.

The best mode of draining a district of this sort is by means of a canal extending completely through it, which acts alternately as a reservoir and as a channel. The *top-water-level* of the canal is to be fixed so as to give sufficient declivity to the branch drains. Its *low-water-level* will be above that of low-water of neap tides to the extent of 1-15th part of the rise of such tides. The space contained in the canal between those levels is the *reservoir-room;* and inasmuch as the length and depth of that space are fixed, the breadth midway between those levels is to be made sufficient to give reservoir-room for the greatest quantity of drainage water that ever collects during one tide. The depth of the canal must be made at least sufficient to enable the whole of that quantity of water to be discharged in the interval between 1 hour before and 1 hour after low-water, the *mean velocity of outflow* being assumed to be about equal to that due to a declivity of the height between high and low-water-levels in the whole length of the canal, and to its hydraulic mean depth when full up to its middle water-level. The outer end of the canal is to have large floodgates capable of throwing its whole width and depth open at once; or a row of large siphon-pipes, passing over the tidal embankment, and having suitable apparatus for exhausting the air from their summits. (See p. 741.)

485. **Drainage by Pumping** is extensively employed in lands below high-water-mark, especially in Holland. In former times windmills were chiefly used for this purpose, but now they are to a great extent replaced by steam engines. The most economical mode of conducting drainage in this manner is to provide reservoir-room for the greatest floods, and pump constantly at an uniform rate. To provide for the repair of engines, and for accidental stoppages, engines are to be kept in reserve, of power equal to from one-half to the whole of the power of those that are kept at work.

486. **Town Drainage.**—Plans for systems of town drainage require to be on a larger scale, and to have closer contour-lines, than those of any other description of work. (See Article 59, p. 96.) The discharge to be provided for is the natural drainage of the basin which the town occupies, added to the water supply artificially brought into the town.

Inasmuch as the rain-fall in towns finds its way into the sewers almost instantly, their dimensions and declivity must be suited to the heaviest rain-fall in a short period. Authorities differ whether that rain-fall is to be estimated at *one inch* or at *half-an-inch* in depth per hour.

The treatment and disposal of the drainage of towns, after it has been collected by means of a system of sewers, involves chemical and physiological questions into which it is impossible to enter in this treatise.

487. **Sewers,** or main drains of towns, are underground arched brick conduits, designed, laid out, and constructed according to the principles already explained or referred to in Articles 474, 475, pp. 718 to 720. As to their strength, see Article 297 A, p. 433. The cross-section preferred for them in Britain is an oval, with the small end downwards. In order that men may be able to enter them for purposes of cleansing and repair, no sewer should have a less breadth than 2 feet.

The velocity of the current should be not less than 1 foot per second, or more than about $4\frac{1}{2}$ feet per second.

As to the drainage of streets into the sewers, see Article 417, p. 626. Owing to the quantity of mud that is swept into sewers, they are peculiarly liable to be obstructed by collections of sediment: these are swept away by an operation called *flushing* or *flashing*, which consists in placing a temporary dam of timber above the spot where the deposit is, so as to collect a quantity of water, which is allowed suddenly to escape with great speed in order to scour away the deposit.

As the pipes leading into the sewers from the channels of the streets, and also those from the houses, either are or ought to be "trapped" by means of valves or inverted siphons, so as to prevent the escape of foul gas from the sewers, such gas must have openings provided for its escape, either by building chimneys for the purpose, or by connecting the sewer with existing chimneys. Passages for the admission of fresh air to the sewers are also required, and subterranean entrances with trap-doors to give men access to them. As to the use of "side-trenches" and "subways," see Article 421, pp. 629, 630.

488. **Pipe-Drains.**—The earthenware pipes used for drainage have already been described in Article 477, p.720. In town drainage they are chiefly used for the branch drains leading from houses and from the adjoining ground into the main sewers; and they usually range from 4 inches to 18 inches in diameter, according to the quantity which they are to discharge. It is not advisable in any case to use drain-pipes of less than 4 inches in diameter. They should all be laid, as far as possible, at such declivities as to insure a velocity of flow of $4\frac{1}{2}$ feet per second, in order that the formation of deposit may be impossible; and when their proper levels and declivities have been determined by calculation, great care should be bestowed on seeing that they are accurately laid at those levels and declivities: the smaller the diameter of the pipe, the worse is the effect of any inaccuracy in this respect. Obstructions are most likely to occur at the junctions. The importance of making these either curved or acute-angled has already been mentioned; but even at curved or acute-angled junctions deposits may some-

times take place, and a good safeguard against this, when the levels are such as to render it practicable, is to make the junction in a vertical or transversely inclined, instead of a nearly horizontal plane.

The *inverted siphon air-trap*, for preventing the entrance of foul gas from a sewer into a building through a drain-pipe, is an U-shaped tube, in the lower part of the bend of which water lodges, so as to prevent the passage of gas. To insure the efficiency of this trap, it is essential that the sewer should have chimneys for the escape of gas; otherwise the pressure may become sufficient to enable the gas to force its way past the water in the tube.

SECTION VI.—*Of Systems of Water Supply.*

489. **Irrigation.**—It appears that the supply of water required for the irrigation of a district ranges from ·013 to ·008 of a cubic foot of water per second for each acre irrigated; and this is the *demand* to be provided for by reservoirs, or by the use of weirs to divert water from rivers. (Article 460, p. 699; Article 472, p. 713.) The channels by which the water is distributed are to be carried at the highest levels compatible with the minimum velocity of 1 foot per second, in order that as great an area of land as possible may be commanded by them. Their dimensions and declivity are to be determined by the principles of Article 451, p. 686, and they are to be constructed according to the principles of Section IV. of this chapter, especially Article 474, p. 718. When they run between earthen embankments, as is often the case, each embankment should have a vertical puddle wall in its centre, from 2 to 3 feet thick, and the tops of the embankments should not be less than 4 feet wide.

The method of delivering specified supplies of water from an irrigation canal to holders of land is the following:—A small tank at one side of the canal is supplied through a sluice, and the water in it is kept at a constant level by regulating the opening of that sluice. The water is delivered out of the tank through a square or round orifice of constant size under a constant head. Different quantities of water are delivered by varying the *number* of the orifices, and not their dimensions nor the head which causes their discharge.

490. **Water Supply of Towns — Estimation of Demand as to Quantity.**—The supply of water to towns ranges in extreme cases from about 2 gallons to 600 gallons per inhabitant per day. (Gordon *On Civil Engineering.*) In town water-works executed with a due regard to sufficiency of supply on the one hand and economy of

cost on the other, and with a moderate amount of waste, the following may be regarded as fair estimates of the real daily demand for water per inhabitant amongst inhabitants of different habits as to the quantity of water they consume, (having been verified by the experiments of Mr. J. M. Gale, C.E.)

	Gallons per Day. Least.	Average.	Greatest.
Used for domestic purposes, ...	7	10	15
Washing streets, extinguishing fires, supplying fountains, &c., ...	3	3	3
Trade and manufactures, ...	7	7	7
Total usefully consumed, ...	17	20	25
Waste, under careful regulation, say........	2	2	$2\frac{1}{2}$
Total demand, ...	19	22	$27\frac{1}{2}$

A liberal supply of water has a tendency to increase its use, and at the same time to bring the daily consumption per head amongst different classes of persons more nearly to an equality; so that, with a view to such improvement in the habits of the population, it is advisable in projecting new water-works to take somewhat more than the highest of the preceding estimates of the demand; that is to say, about 30 gallons per head per day, supposing waste of water to be as far as possible prevented.

The quantity of water run to waste, however, frequently exceeds enormously that allowed for in the preceding estimate, through ill-constructed fittings and carelessness. A quantity equal to that used is not uncommon, and in one case, where 7 gallons of water per head per day were actually used, 18 gallons ran to waste. The most effectual means of preventing such waste are, the establishment of a regulation or enactment, that domestic water-fittings shall be executed to the satisfaction of the engineer or manager of the water-works; the carrying out, as far as practicable, of the system of selling water by measure to those who require it for other than ordinary domestic purposes (as to water meters, see Article 459, p. 699); and the prevention of excessive pressure in the service-pipes from which houses are directly supplied.

The preceding statements have reference to the daily demand. Regard must also be had to the hourly demand, which fluctuates very much at different times of the day, chiefly because the inhabitants draw nearly the whole of their supply for domestic purposes during a limited number of hours. It is estimated that the most rapid draught for domestic purposes is at such a rate that, if kept up continuously, it would exhaust the whole daily supply for these purposes in 8 hours; that is to say, the maximum hourly demand for domestic purposes is *three times* the average hourly demand.

The effect of this on the greatest hourly demand *for all purposes* is to make it in different cases range from twice to $2\frac{1}{3}$ times the average hourly demand.

491. **Estimation of Demand as to Head.**—It is considered that the head of pressure in each of the street mains ought, when the flow is most rapid, to be equivalent to an elevation of about 20 feet above the tops of the adjoining houses, in order that their uppermost stories may be directly supplied, and that it may be possible to throw a jet to the top of the highest building without the aid of a fire-engine.

The required virtual head in various districts of the town being fixed, the virtual declivity from the source to each of those districts is to be made as nearly uniform as circumstances will permit, if pipes are used throughout. Should a conduit be used for part of the distance, and pipes for the remainder, the pipes should have the steeper virtual declivity, and consequently the greater share of the total virtual fall in proportion to their length, in order that they may be smaller than the conduit; because their cost is greater in proportion to their size than that of the conduit. No precise rule can be laid down for this distribution of fall between pipes and conduit; but in some good examples the virtual declivity of the pipes has been made *eight times* as steep as the actual declivity of the conduit. As to the discharging capacity and construction of conduits and pipes, see Articles 450, 451, pp. 684 to 688, and Articles 474 to 478, pp. 718 to 724.

In a town of irregular levels, or of great extent, the same virtual declivity which is required in order to give sufficient head of pressure in the higher parts of the town, or in those more distant from the source, may give excessive pressure in the lower or nearer parts. In such cases the excessive pressure in the branch mains and distributing pipes of the latter districts may be moderated by any convenient means of causing loss of head at their inlets, such as passing the water through small orifices, or loaded valves; the latter being the more accurate method in its working.

492. **Compensation Water** is the supply of water which is secured to the owners and occupiers of land and mills, and other parties interested in the sources from which water is diverted to supply a town, in order that they may not suffer damage by such diversion. It must be at least equal to the supply which was beneficially available for their use before the execution of the water-works, or else they must receive compensation in money for the deficiency.

The only means of enabling a source of water to supply a town, besides providing the landholders with compensation water, according to the preceding principle, is to store in reservoirs and discharge by degrees the flood-waters which previously ran to waste. (See Section III. of this chapter, p. 699.)

In providing the daily supply of compensation water to which the landholders on the course of a stream are entitled, different principles have been followed in different cases. The following are three of them:—

I. To secure them the *average summer discharge, exclusive of floods,* as ascertained by gauging. (As to the distinction between flood discharges and ordinary discharges, see Article 458, p. 698).

II. To give them a proportion fixed by agreement (usually *one-third,* or thereabouts) of the whole water impounded.

In some cases a special arrangement has been come to, by which the landholders, on condition of a certain supply being delivered down the stream during the day, have agreed to a less supply being delivered during the night.

III. To make a special compensation reservoir, receiving the discharge from a certain proportion of the gathering-ground, and to hand it over to the landholders, to be managed under their own control.

The usual method adopted in delivering a fixed daily quantity of water into the natural channel of a stream is to construct a tank in which the water is kept at a fixed level by means of the sluice or sluices through which it is supplied, and let the water flow out of that tank through an outlet or outlets of a fixed area and figure, under a fixed head.

493. **Storage-Works** consist of reservoirs with their appurtenances, as described in Section III. of this chapter. In estimating the extent of gathering-ground and capacity of the reservoirs required, regard must be had to the demand of water for compensation (Article 492), as well as for the supply of the town.

In most cases in which a town is supplied from works of this class, the best economy consists in choosing the sites of the store reservoirs, and designing the conduits and principal main pipes, so as to supply every part of the town by means of the gravitation of the water alone. But exceptional cases sometimes occur, in which a great saving may be effected in capital outlay, and especially in the cost of conduits and pipes, by incurring a comparatively small additional annual expenditure in order to supply some limited district that is highly elevated above the rest of the town by means of a pumping steam engine, instead of giving the conduits and principal main pipes the dimensions required in order to supply that limited district by gravitation.

494. **Springs** in many cases are so variable in their discharge that they can only be classed amongst the sources whose waters require to be stored in a reservoir. But occasionally springs are met with which are the outlets of extensive porous strata, forming underground natural reservoirs that maintain a nearly uniform discharge independently of artificial storage. (See Article 456, p. 696.) When the waters of such springs are diverted from the streams into which they naturally flow in order to supply a town, the ordinary summer flow of those streams must be maintained at its original volume by the aid of the flood-waters of a gathering-ground, stored in a reservoir.

495. **River-Works—Pumping.**—A large river may be used for the supply of a town, independently of storage-works, provided the volume of water brought down by it is at all times so great, that the temporary abstraction of a volume sufficient to supply the town will cause no injury to its navigation, or the interests of the inhabitants of its banks.

The works required in order to supply a town from such a river usually comprise a *weir*, for maintaining part of the river at a nearly constant level (Article 472, p. 713); two or more *settling-ponds*, into which the water is conducted, or if necessary, pumped, or otherwise raised by machinery; filtering apparatus; and a sufficient establishment of pumping engines.

It would be foreign to the plan of the present work to enter into details as to the construction and working of pumping steam engines. The following principles, however, must be stated as specially applicable to their use for the supply of a town.

I. The *effective power* required to be in operation may be computed in *foot-pounds per hour*, by multiplying the *weight* of water to be delivered per hour by the *total head* at the engines in feet; such head being measured from the level of the water in the tank whence the engines draw it, to the virtual elevation required in order to give sufficient head in the town and sufficient virtual declivity in the principal main pipes. To find the *effective horse-power*, divide the effective power in foot-pounds per hour by 1,980,000. The *indicated horse-power* is about *once and a-quarter* the effective horse-power.

II. *Reserve power* should be provided to an amount equal to at least one-half of the working power; for example, of three engines of equal power, two are to be kept at work and the third in reserve.

III. *Air-vessels* and *stand-pipes* are contrivances to prevent the shocks to which the pipes would be exposed by the intermittent action of the pumps, and to maintain an uniform head of pressure and velocity of flow in the pipes.

An air-vessel is an air-tight receiver, usually of cast iron, and of the figure of a cylinder standing vertically, with a hemispherical top and bottom. At its lower end are two openings, an inlet through which water enters from a pump, and an outlet from which the water is discharged along a pipe. Its upper portion contains compressed air, which tends continually to diminish in quantity, partly by leakage and partly by absorption in the water, so that a small supply of air should be forced in from time to time by suitable apparatus. The effect of the air-vessel in moderating fluctuations of pressure is expressed by the following proportion:—

mean volume of air in the vessel : volume of the pump
: : mean head of pressure : greatest fluctuation of the head of pressure.

In some good practical examples, the capacity of the air-vessel is about *fifty times* that of the pump.

A *single stand-pipe* is a vertical cast iron pipe, rising a little higher than the elevation due to the head of pressure, and open at the top. It has at its base an inlet through which it receives water from the pumps, and an outlet or outlets through which it discharges water into the horizontal supply-pipes. Its sectional area varies from once to twice that of its outlets, or thereabouts. It equalizes the pressure and flow even more effectually than an air-vessel, for the rapid entrance of the quantity of water due to one stroke of a pump produces but a slight elevation of the surface of the water in the stand-pipe as compared with its total height.

A *double stand-pipe* has two branches, in one of which the water ascends from the pump, while in the other it descends to the mains: the two branches unite at the top into a vertical stem, which is open above. This construction effects a constant renewal of the water in the stand-pipe.

In estimating the dimensions and speed required for the piston or plunger of a pump that is to deliver a given volume of water in a given time, it is usual to add about *one-fifth* to that volume as an allowance for "*slip*;" that is, water which runs back through the pump-clacks while they are in the act of closing. It appears, however, from experiment, that in the best pumps the slip is not practically appreciable.*

* The cost of pumping large quantities of water, as ascertained from the accounts of the expenditure of the former Glasgow Water-Works (since superseded by the Loch Katrine Works), during a long series of years, was at the rate of almost exactly 400,000 *gallons raised one foot for a penny;* that is to say, 4,000,000 *foot-pounds of effective work for a penny.*

496. **Wells** may be used as sources for a supply of water, where a water-bearing stratum exists into which they can be sunk. The water in such a stratum has always either an actual or a virtual declivity towards the place where, by the outcrop of the stratum, it makes its escape into a river, or into the sea. Should the water-bearing stratum have its gathering-ground at a high elevation, and should it be covered, in a district far distant from its final outlet, by an impervious stratum, the line of virtual declivity may be above the surface of the ground in that district; so that, on boring or sinking a well through the impervious stratum, the water will spout up in a jet. Such wells are called "Artesian Wells." In other cases the line of virtual or actual declivity is below the surface of the ground, and the water must be raised by pumping (as to which, see the preceding article).

The raising of a large quantity of water from a water-bearing stratum has always the effect of depressing the water-level to an extent which cannot be estimated beforehand.

The quantity of water which a water-bearing stratum is capable of yielding may be estimated in the manner explained in Article 456, p. 696, provided the position and extent of its gathering-ground can be ascertained; but that can seldom be done with precision.

In sinking or boring for well water, it is in general advisable to prevent the surface water from mixing with that of the well. This is done, in the case of a bore, by lining it with iron pipes, and in the case of a shaft, by lining it with brickwork laid in cement.

As to boring and shaft-sinking, see Article 187, p. 331, and Article 391, p. 589.

497. The **Purity of Water** is a subject of which the detailed consideration belongs to chemistry and physiology rather than to engineering. The following general principles, however, may be stated.

For purposes of cleansing, cookery, chemistry, and manufactures, the best water is that which approaches nearest to absolute purity. Such is the water which flows from mountain districts, where granite, gneiss, and slate prevail. Such water usually contains a large quantity of diffused oxygen and carbonic acid. It is the most wholesome for drinking, and the most agreeable to those whose taste does not prefer a certain admixture of earthy salts.

The most common mineral impurities of water are salts of lime and iron, which injure it for all purposes except drinking. Salts of lime, especially the bicarbonate, are the principal causes of the property called "hardness." The bicarbonate of lime can be removed by adding to the water as much lime-water as contains a quantity of lime equal to that already contained in the bicarbonate of lime present. The additional lime thus added combines with one-half of the carbonic acid, thus becoming chalk itself, and reducing the bicarbonate to chalk also; and the chalk, being insoluble, settles, and leaves the water softened. This is Dr. Clarke's process of softening water. The *degrees of hardness* of a specimen of water means the number of grains of chalk which the lime held in solution in a gallon of the water (or 70,000 grains) is capable of forming. Water of less than 5 degrees of hardness may be considered as comparatively soft; that of 12 or 13, as decidedly hard.

The waters collected directly from gathering-grounds are usually the softest, those of rivers harder, those of springs and wells hardest of all.

The drainage waters of cultivated and populous districts, and above all, those of towns and their neighbourhood, are to be avoided, as containing organic matter in the act of decomposition, and being therefore unwholesome, and sometimes highly dangerous.

The taste and smell of a person accustomed to drink pure water and breathe pure air may in general be relied upon for the detection of the presence of impurities in water, though not of their nature or amount; but in persons who have for some time habitually drunk impure water and breathed a foul atmosphere those senses become blunted.

The colouring matter of peat moss, which is a compound of carbon with oxygen and hydrogen, unfits water for many manufacturing purposes. It does not render it unfit for drinking, unless present in considerable quantity, when it produces an unpleasant flatness of taste; but whether that substance is unwholesome or not has not been ascertained. Its appearance is strongly objected to by the inhabitants of most towns. Long exposure to light and air destroys it, probably by oxidating its carbon.

The long-continued action of oxygen decomposes and destroys organic matter in water, and is the principal means of purifying originally impure water. In store reservoirs the presence of a moderate quantity of living plants is favourable to purity of the water, provided there are also animals enough to consume them, so that they may not die and decompose, and that a proper balance is kept up amongst animals of different kinds. The destruction of the fish in a reservoir has been known to lead to an excessive multiplication of the small crustaceous animals upon which the fish had fed, to such an extent that the water acquired a nauseous flavour from the oil which those minute creatures contained. The only remedy was to re-stock the reservoir with fish.*

* This case was examined into and reported upon, and the remedy discovered, by Dr. H. D. Rogers.

Shallow reservoirs are unfavourable to purity, because the warmth of the water produced by the sun's heat encourages the growth of an excessive quantity of vegetation, most of which dies and decomposes.

On the subject of the purity of water, see Dr. R. Angus Smith's "Report on the Air and Water of Towns," in the *Reports of the British Association* for 1851.

498. **Settling and Filtration.**—A store reservoir generally answers the purpose of a settling-pond also, to clear the water of earthy matter held in suspension. Water pumped from a river generally requires to rest for a time in a settling-pond.

The water both of rivers and of gathering-grounds in most cases requires to be filtered. A filter-bed for that purpose consists of a tank about 5 feet deep, having a paved bottom, covered with open-jointed tubular drains leading into a central culvert; the drains are covered with a layer of gravel about 3 feet deep, and that with a layer of sand 2 or 3 feet deep. The water is delivered upon the upper surface of the sand very slowly and uniformly; it gradually descends, and is collected by the drains into the central culvert. The area of the filter should be such that the water to be filtered may not descend vertically with more than a certain speed; for the whole efficiency of the filtering process depends on its slowness. The speed of vertical descent recommended by the best authorities is *six inches an hour;* in some cases a speed as high as *one foot an hour* has been used.

There should be a sufficient number of filter-beds to enable some to be cleansed whilst others are in use. The cleansing is performed by scraping from the surface of the sand a thin layer, in which all the dirt collects.

It appears that proper filtration not merely removes mechanical impurities from the water, but even organic impurities, by causing their oxidation.

499. **Distributing-Basins or Town Reservoirs.**—It has been explained in Article 490, p. 730, that the *greatest hourly demand* for water is about double of the *average hourly demand;* from which it follows, that the pipe or conduit which *directly* supplies a given town, or part of a town, must have about double the discharging capacity that it would require if the hourly demand were uniform.

The great additional expense which this would cause in the principal conduits and main pipes is saved by the use of *distributing-basins* or *town reservoirs.*

A distributing-basin for a given district is a small reservoir, capable of containing a volume of water *at least* equal to the whole excess of the demand for water during those hours of the day when such demand exceeds the average rate above a supply during the same time at the average rate. The smallest capacity which will enable a distributing-basin to fulfil that condition is about one-half of the daily demand of the district to which it belongs; but to provide for unforeseen contingencies, it may be made to contain a whole day's demand, or even more. It is supplied with water at an uniform rate, by a principal main pipe, which thus only needs to be made capable of supplying the average hourly demand, the distributing-pipes alone requiring to be adapted to the greatest hourly demand. During the night, when the supply exceeds the demand, the water accumulates in the distributing-basin; during the day, when the demand exceeds the supply, that accumulated water is expended.

The area of a distributing-basin should be such, that the variation of its water-level may not cause an inconvenient variation of the head of pressure in the pipes, nor in their virtual declivity.

It may be built and paved with masonry or brickwork lined with cement, in which case the stability of its walls will depend on the principles cited in Article 465, p. 707; or it may be made of rectangular cast iron plates, flanged and bolted together, the opposite sides of the reservoir being tied together by means of wrought iron rods, to enable them to resist the pressure. The figure in plan will in general be regulated by that of the site; but should the engineer be free to choose any figure, the circular figure is obviously the best.

The elevation of the site should be such as to command the district to be supplied from the basin, according to the principles of Article 491, p. 732, and it should be as near that district as possible.

Every distributing-basin should be roofed, that the water may be protected against heat, frost, and the dust and soot which float in the air of populous districts. The most efficient protection against heat and frost is that given by a vaulted roof of masonry or brick, covered with asphaltic concrete to exclude surface water, and with two or three feet of soil, and a layer of turf.

When water is brought to a city from a great distance, it may be useful to construct in the neighbourhood of the city (should the ground afford a suitable site), a large town reservoir or auxiliary store reservoir, capable of holding a store of water for about a month's demand, to be used in the event of an accident happening to the more distant part of the main conduit, until the damage is repaired. From that reservoir to the town the main pipes may form a double line, so that in the event of a failure of one line, a supply, although a diminished one, may be conveyed through the other line until the first line is repaired. The construction of such an auxiliary store reservoir will in general be similar to that of the reservoirs described in Section III. of this chapter.

500. **Distributing-Pipes** must be adapted to the *greatest* hourly demand for water, and to the requisite head in the streets, as already explained in Articles 490 and 491, pp. 730 to 733. In large cities the total length of distributing-pipes required is about a mile for every 2,000 or 3,000 inhabitants. The smaller the town, the smaller in general is the *proportionate* extent of distributing-pipes required.

The distributing-pipes which are laid along the street are classed as *mains* and *service-pipes;* the chief distinction being, that a main either conveys, or is capable of conveying, water along a street to some place beyond it; while a service-pipe is a branch diverging from a main, in order to supply a single or double row of buildings. In wide streets, and in those of great traffic, it is best to have two service-pipes, one for each side, in order that they may be laid so as to be accessible without interrupting the traffic of the street (see Article 421, p. 629), and in order that the house water-pipes may be as short as possible, and may lie as little as possible under the carriage-way.

When a general rate of virtual declivity has been fixed for the distributing-pipes of a town or of a district of a town, and the diameters of the more important mains have been computed by the proper formula, those of all branch mains and service-pipes are easily deduced from them by the rule, that, with equal virtual declivities, the diameters of pipes are to be proportional to the *squares of the fifth roots* of the quantities of water that they are to convey.

When a pipe of uniform diameter has a series of branches diverging from it, so that the flow of water through it becomes less and less at an uniform rate, until the pipe terminates at a "*dead end*," the virtual declivity goes on diminishing, being proportional to the *square of the distance from the dead end;* the excess of the head at any point above the head at the dead end is proportional to the *cube of the distance from the dead end;* and the total virtual fall, from the commencement of the pipe to the dead end, is *one-third* of what it would have been had the whole quantity of water flowed along the pipe without diverging into branch pipes.

All *dead ends* of pipes should be provided with *scouring-valves*, which should be opened from time to time to prevent the accumulation of deposit there. Pipes should be laid out and connected with each other so as to have as few dead ends as possible; and with that view it is desirable that service-pipes should, if practicable, be connected at both ends with mains.

The use of loaded valves to moderate pressure has already been mentioned in Article 491, p. 732.

The system called that of *constant service*, according to which all distributing-pipes are kept charged with water at all times, is the best, not only for the convenience of the inhabitants, but also for the durability of the pipes, and for the purity of the water; for pipes, when alternately wet and dry, tend to rust; and when emptied of water, they are liable to collect rust, dust, coal-gas, and the effluvia of neighbouring sewers, which are absorbed by the water on its re-admission. In order, however, that the system of constant service may be carried out with efficiency and economy, it is necessary that the diameters of the pipes should be carefully adapted to their discharges, and to the elevation of the district which they are to supply, and that the town should be sufficiently provided with town reservoirs. When these conditions are not fulfilled, it may be indispensable to practise the system of *intermittent service*, especially as regards elevated districts; that is to say, to supply certain districts in succession, during certain hours of the day. The adoption of this system makes it necessary for the inhabitants to have cisterns in their houses for the purpose of holding the daily store of water. In the poorer districts of towns, it is often advisable to have one large tank for a group of small houses, instead of a cistern in each house; the tank may be under the control of the water-work officials, and may be filled once a day, and the householders may be supplied from it through small pipes constantly charged, and may thus have the convenience of constant service although the supply to the tank is intermittent.

500 A. On the subject of the collection, conveyance, and distribution of water generally, special reference may be made to the works of Du Buat, M. D'Aubuisson, Mr. Neville, and Mr. Downing, *On Hydraulics;* Tredgold's *Hydraulic Tracts;* Mr. Beardmore's *Hydraulic Tables;* Professor Becker's "*Wasserbau;*" and Dr. Hagen's "*Handbuch der Wasserbaukunst,*" (Königsberg, 1853 to 1857); and on that of the water supply of towns, to the *Parliamentary Reports* on the supply of water to the metropolis, and to the *Reports* of the Board of Health on the same subject.

ADDENDUM to Article 484, p. 728.—**Siphons for Tidal Drainage.**—The waters of the Middle-Level Drainage Canal are discharged over the top of an embankment through sixteen parallel siphons, each of $3\frac{1}{2}$ feet bore and $1\frac{1}{8}$ inch thick. The summits of the siphons are 20 feet above, and their lower ends $1\frac{1}{2}$ foot below, low water of spring-tides. They have flap-valves, opening down stream, at both ends; the lower valve can be made fast with a bridle when required. The air is exhausted from their summits, when required, by an air-pump having three cylinders of 15 inches diameter and 18 inches stroke, driven by a high-pressure steam engine of ten horse power. The floor of the canal at the inlets and outlets is protected by a wooden apron. (J. Hawkshaw, C.E., F.R.S., in the *Proceedings of the Institution of Civil Engineers*, April, 1863.)

CHAPTER III.

OF WORKS OF INLAND NAVIGATION.

SECTION I.—*Of Canals.*

501. **Canals Classed—Selection of Line and Levels.**—Canals may be divided into three classes—

I. *Level Canals,* or *Ditch Canals,* consisting of one *reach* or *pond,* which is at the same level throughout. The most economical course for a canal of this sort is obviously one which nearly follows a contour-line, except where opportunities occur of saving expense by crossing a ridge or a valley so as to avoid a long circuit.

II. *Lateral Canals,* which connect two places in the same valley, and in which, therefore, there is no summit level, the fall taking place in one direction only. A lateral canal is divided into a series of level reaches or ponds, connected by sudden changes of level, at which there are either single locks or flights of locks, or some other means of transferring boats from one level to another. The "lift" of a single lock ranges from 2 feet to 12 feet, and is most commonly 8 or 9 feet. Each level reach is to be laid out on the same principles with a level canal. In fixing the lengths of the reaches and the positions of the locks, the engineer should have regard to the fact that economy of water is promoted by distributing a given fall amongst single locks with reaches between them, rather than concentrating the whole fall at one flight of locks.

III. *Canals with Summits* have to be laid out with a view to economy of works at the passes between one valley and another, and with a view also to the obtaining of sufficient supplies of water at the summit reaches. The subject of the supply of water to canals will be considered further on.

502. **Form and Dimensions of Water-way.**—Although, for the sake of saving expense in aqueducts and bridges, short portions of a canal may be made wide enough for the passage of one boat only, the general width ought to be sufficient to allow two boats to pass each other easily. The depth of water and sectional area of water-way should be such as not to cause any material increase of the resistance to the motion of the boat beyond what it would encounter in open water. The following are the general rules which fulfil these conditions:—

Least Breadth at Bottom = 2 × greatest breadth of a boat.
Least Depth of Water = $1\frac{1}{2}$ foot + greatest draught of a boat.
Least Area of Water-way = 6 × greatest midship section of a boat.

The bottom of the water-way is flat. The sides, when of earth (which is generally the case), should not be steeper than $1\frac{1}{2}$ to 1; when of masonry, they may be vertical; but, in that case, about 2 feet additional width at the bottom must be given to enable boats to clear each other, and if the length traversed between vertical sides is great, as much more additional width as may be necessary in order to give sufficient sectional area.

The customary dimensions of canal-boats have been fixed with a view to horse-haulage. The most economical use of horse-power on a canal is to draw heavy boats at low speeds. The heaviest boat that one horse can draw at a speed of from 2 to $2\frac{1}{2}$ miles an hour weighs, with its cargo, about 105 tons, is about 70 feet long and 12 feet broad, and draws about $4\frac{1}{2}$ feet of water when fully loaded. Smaller boats, which a horse can draw at $3\frac{1}{2}$ or 4 miles an hour, are of about the same length, 6 or 7 feet broad, and draw about $2\frac{1}{2}$ feet of water.

Boats of the greater breadth above-mentioned can easily be adapted to the various methods of propulsion by steam, whether by means of the screw propeller or the warping chain, or fixed engines and endless wire ropes (Mr. Liddell's system).

Ordinary canals are suited to boats such as the above. A larger class of canals are suited to sea-going vessels.

The following are examples of the extreme and ordinary dimensions of canals:—

	Breadth at Bottom.	Breadth at Top-water.	Depth of Water.
Small canal,.......	12 feet, ...	24 feet, ...	4 feet.
Ordinary canal,....	25 „	40 „	5 „
Large canal, ...	50 „	110 „	20 „

503. **Construction of a Canal.**—The least expensive parts of a canal are those in which the upper part of the water-way is contained between two embankments, and the lower part in a cutting, the earth dug from which, together with that dug from the side-drains at the foot of the outer slopes, is just sufficient to form the embankments.

All canal embankments should be formed and rammed in thin layers. (Article 203, p. 341.) The width of the embankment which carries the towing-path is usually about 12 feet at the top; that of the opposite embankment at least 4 feet, and sometimes 6 feet. Each embankment has a vertical puddle wall in its centre from 2 to 3 feet thick.

In cutting, there should be a bench or berm of 12 or 14 feet wide, at one side, for the towing-path, and on the opposite side a bench about 3 or 4 feet wide at the same level. At the feet of the slopes, which terminate at those benches, there are a pair of side-drains, as described in Article 193, p. 335. These side-drains discharge their water at intervals into the canal through tubes.

The surface of the towing-path is usually about 2 feet above the water-level. It is made to slope slightly in a direction away from the canal, in order to give a better foot-hold for the horses, as they draw in an oblique direction.

The slopes are to be pitched with dry stone from 6 to 9 inches thick.

Occasionally it may be necessary to line a canal with concrete, or to face the sides with rows of sheet-piling, in order to retain the water.

Natural water-courses are to be carried below the canal by means of bridges and culverts, and, if necessary, by inverted siphons of masonry or iron. Where such water-courses are above the level of the canal, their waters may be partly used for supplying it; but means should be provided for carrying such waters wholly across the canal when required.

Each reach of a canal should be provided with waste-weirs in suitable positions, to prevent its waters from rising to too high a level; also with sluices, through which it may be wholly emptied of water for purposes of repair; and in a reach longer than two miles, or thereabouts, there may be stop-gates at intervals, so that one division of the reach may be emptied at a time, if necessary. The rectangular channel under a bridge or over an aqueduct is a suitable place for such gates.

Leaks in canals may sometimes be stopped by shaking loose sand, clay, lime, chaff, &c., into the water. The particles are carried into the leaks, which they eventually choke by their accumulation.

504. **Canal Aqueducts and Fixed Bridges.**—A canal aqueduct, like the aqueducts for conduits already mentioned in Article 476, p. 720, is a bridge supporting a water-channel. The trough or channel, for economy's sake, is usually made wide enough for one boat only. Its bottom is flat, or nearly so; its sides vertical or slightly battering. In aqueducts of masonry, the total thickness of material, from the side of the trough to the face of the spandril-wall, is usually 4 feet at least at the side furthest from the towing-path; at the towing-path side it is sufficient for a towing-path of from 6 to 10 feet wide, and a parapet from 15 to 18 inches thick.

In Telford's cast iron aqueduct, known as Pont-y-Cysylte, the channel is a rectangular trough of cast iron, supported on cast iron segmental arched ribs of 45 feet span. The trough is of the whole width of the bridge, about 12 feet, and the towing-path, 5 feet 8 inches wide, covers part of the trough.

The principle of the *suspension bridge* is peculiarly well adapted to aqueducts, because, as each boat displaces its own weight of water, the only disturbance of the uniform distribution of the load is that arising from the passage of men and horses along the towing-path. An aqueduct of this sort, designed by Mr. Roebling, with seven spans of 160 feet, carries a canal 16 feet wide and 8 feet deep, over the Alleghany River at Pittsburg.

Fixed bridges over canals require no special explanation, except to state that, in the older examples of them, the water-way is contracted so as to admit one boat only, and the towing-path is only 6 feet wide, or thereabouts, the headroom over it being about 10 feet. Sometimes the archway admits the water-channel alone, while the towing-path ascends to the approach of the bridge and descends again, the tow-rope being cast loose while the horse passes over. As to bridges for carrying railways over canals, see Article 436, p. 663.

Tunnels for canals usually have the water-way and towing-path contracted as already described; and sometimes the towing-path is dispensed with, the boats being pushed through by means of poles, or by the hands and feet of the boatmen, with the aid of notches in the brickwork, or by means of the various methods of steam propulsion.

505. **Moveable Bridges** cross a canal near its water-level are made of timber or of iron, and are capable of being opened so as to leave the navigation clear, and closed so as to form a passage for a road or railway by one or other of five kinds of movement, viz., I. By turning about a horizontal axis; II. By turning about a vertical axis; III. By rolling horizontally; IV. By lifting vertically; V. By floating in the canal. As regards the adaptation of the strength and stiffness of a moveable bridge to the greatest load which it has to bear when closed, it differs in no respect from a fixed bridge. But, besides having the strength and stiffness required in a fixed bridge, it must fulfil some other conditions, which are as follows:—If it turns about an axis, it must be so balanced that its centre of gravity shall always lie in that axis; if it rolls backwards and forwards it must be so balanced that its centre of gravity shall always lie over the base or platform on which it rolls: in either of those cases it must have strength sufficient to support safely the *overhanging* part of its own structure, when deprived of direct support; if it is lifted vertically, it must be counterpoised; and if it is carried by a pontoon or float, that float must displace a mass of water equal in weight to the bridge, and must have sufficient stability.

I. A bridge which turns about a horizontal axis near an end of its span is called a *draw-bridge.* It is opened by being raised into a vertical position by means of a pinion driving a toothed sector. It is best suited for small spans.

II. A bridge which turns about a vertical axis is called a *swing-bridge.* Its principal parts are as follows :—

A pier of masonry or iron, supporting a circular base-plate of a diameter equal, or nearly equal, to the breadth of the bridge. That base-plate has a pivot in the centre, and a circular race or track for rollers round the circumference, as in a railway turntable:

A roller frame turning about the central pivot, with a set of conical rollers resting on the race:

A circular revolving platform resting on the pivot and rollers:

A toothed arc fixed to the revolving platform, with suitable wheel-work for giving it motion:

A set of parallel girders, resting on and fastened to the revolving platform, of the strength and stiffness required by the principles already stated, and supporting a roadway.

The ends of the superstructure are bounded by arcs of circles, described about the axis of motion, and the ends of the roadway of the approaches must be formed to fit them.*

III. A *rolling bridge* has a strong frame, supported by wheels upon a line of rails, and having an overhanging portion sufficient to span the water-way. When closed, by being rolled forward, the rolling frame leaves a gap between its platform and that of one of the approaches, which gap is filled by rolling in another rolling frame that moves sideways. The latter rolling frame is rolled out of the way before opening the bridge.

IV. A *lifting bridge* is hung by the four corners to four chains, which pass over pullies, and have counterpoises at their other ends.

V. A *floating swing-bridge* rests on a caisson or pontoon: it is opened and closed by means of chains and windlasses, and, when open, lies in a recess in the side of the canal made to receive it. The pontoon, being made of sheet iron, is so designed as to act as a tubular girder when the bridge is closed.

506. **Canal Locks.**—Figs. 290, 291, and 292, show the general arrangement of the parts of a canal lock. Fig. 290 is a longitudinal section, fig. 291, a plan, and fig. 292 a cross-section, looking upwards.

* For an example of a swing-bridge on a great scale, reference may be made to one planned by Mr. Hemans and constructed by Messrs. Fairbairn, which carries the Midland Great Western Railway of Ireland over the entrance to Lough Atalia. It has two spans of 60 feet each, and is balanced on a central pier of 34 feet diameter. It is described in detail in Mr. Humber's work *On Iron Bridges.*

A is the *lock-chamber;* *a, a,* its side walls; E, its floor, or invert.

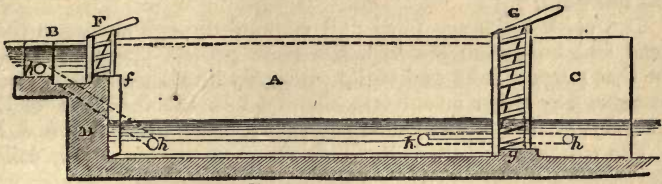

Fig. 290.

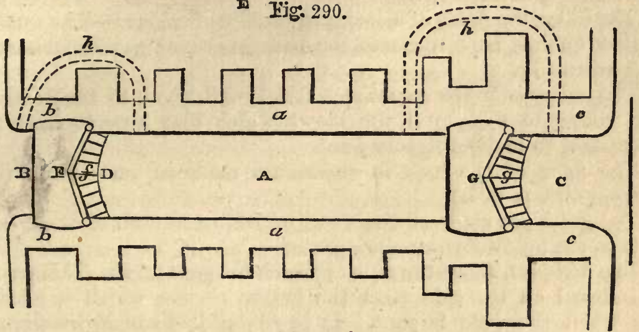

Fig. 291.

Its clear length should be at least equal to that of the longest vessel used on the canal, including the rudder; its clear breadth, one foot more than the greatest breadth of a vessel; its greatest depth of water should be = $1\frac{1}{2}$ foot + greatest draught of a vessel + lift of the lock. Its depth from the cope of the side walls to the bottom may be about 2 feet more.

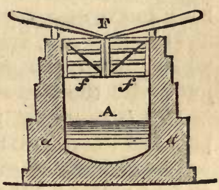

Fig. 292.

The side walls and floor are recessed to admit of the opening of the "tail-gates."

The floor is level with the bottom of the lower of the two ponds to be connected.

B is the *head-bay,* with its side walls and floor, which are recessed to admit of the opening of the "head-gates." The side-walls end in curved wings. The floor is level with the bottom of the upper pond.

C, the *tail bay,* with its side walls and floor. The side walls end in curved wings: the floor in a dry stone pitching or *apron.*

D, the *lift-wall,* which is usually built like a horizontal arch.

F, the *head-gates,* whose lower edges, when shut, press against the *head mitre-sill, f.*

G, the *tail-gates,* whose lower edges, when shut, press against the *tail mitre-sill,* g.

The older locks are filled and emptied through sluices in their head and tail-gates; but now the more general practice is to use for that purpose inlet and outlet passages with slide-valves. These passages may either be culverts contained in the thickness of the masonry, or iron pipes in such positions as those marked *h, h, h, h.*

The cylindrical recesses in which the gates are hinged are called the *hollow quoins.*

The following parts of a lock are usually of ashlar:—The quoins, hollow quoins, cope, recesses for the gates (or "gate-chambers"), and mitre-sills.

The mitre-sills are sometimes faced with wood, to enable them the better to withstand the blows which they receive from the gates, and to make a tighter joint.

The floor of the lock is sometimes made of cast iron. (See Article 400, p. 601.)

The gates are made of timber or of iron, and each of them consists of the following principal parts:—

The *heel-post,* about the axis of which the gate turns. This post is cylindrical on the side next the hollow quoins, which it exactly fits when the gate is shut. It is advisable to make it slightly eccentric, so that when the gate is opened, it may cease to rub on the hollow quoins. At its lower end it rests on a pivot, and its upper end turns in a circular collar, which is strongly anchored back to the masonry of the side walls:

The *mitre-post,* forming the outer edge of the frame of the gate, which, when the gate is shut, abuts against and makes a tight joint with the mitre-post of the opposite leaf:

The *cross-pieces,* which extend horizontally between the heel-post and mitre-post:

The *cleading* or covering, which may consist of timber planking or iron plates. When it consists of planks, they run either vertically or diagonally:

The *diagonal bracing,* which, in its simplest form, may consist either of a timber strut extending from the bottom of the heel-post to the top of the mitre-post, or of an iron tie-bar extending from the top of the heel-post to the bottom of the mitre-post.

The gates shown in the sketch are provided with *balance-bars.* A balance-bar is bolted to the top of the mitre-post, slopes slightly upwards, and crosses over the top of the heel-post, which is mortised into it, and has a long and heavy overhanging end, which acts as a counterpoise to bring the centre of gravity of the gate near the heel-post, and as a lever to open and shut it by.

Sometimes the balance-bar is dispensed with, and each gate has one or more *rollers* under its lowest cross-bar, to assist the pivot in supporting its weight. Each of those rollers runs upon a quadrantal iron rail on the floor of the gate-chamber. This mode of construction is almost always adopted in large and heavy gates that require chains and windlasses to open and shut them.

The following are some of the ordinary dimensions and proportions of locks, in addition to those already stated:—

The mitre-sills rise from 6 to 9 inches above the floor:

Versed-sine of mitre-sill, from $\frac{1}{7}$ to $\frac{1}{5}$ of breadth of lock:

Clearance in depth of the recesses for the gates, $\frac{1}{10}$ of thickness of gate; clearance in length, $\frac{1}{7}$ of length of gate:

Least thickness of the side walls at the top, about 4 feet. Greatest thickness at the base, fixed according to the principles of the stability of walls, usually from $\frac{1}{4}$ to $\frac{1}{2}$ of the height:

Length of side walls of head-bay above gate-chamber, about $\frac{1}{3}$ of breadth of lock:

Large counterforts opposite hollow quoins to have stability enough to withstand the calculated *transverse thrust* of the gates.

The *longitudinal thrust* of the head-gates is borne by the side walls of the lock-chamber; that of the tail-gates by the side walls of the tail-bay. To give the latter walls sufficient stability, the rule is to make their length as follows:—

$$\text{Breadth of lock} \times \text{greatest depth of water} \div 15 \text{ feet.}$$

Versed-sine of lift-wall, from 1-12th to 1-7th of breadth of lock.

Floor of head-bay: least thickness, from 10 inches to 14 inches.

Floor of lock-chamber: versed-sine, about 1-15th of breadth; thickness, from 1-15th to 1-3rd of breadth, according to the nature of the foundation.

Foundations of various kinds have been sufficiently explained. It has only to be added that, when a lock is founded on a timber platform, longitudinal pieces of timber extending along the whole length of the foundation are to be avoided, lest they guide streams of water along their sides; that transverse trenches under the foundation, filled with hydraulic concrete, are a good means of preventing leakage; and that, in porous soils, the whole space behind the lift-wall and under the floor of the head-bay may be filled with a mass of concrete.

Length of apron from 15 to 30 feet.

The dimensions of the different parts of the gates are to be computed according to the principles of the strength of materials. It appears that the factor of safety in many actual lock-gates is as low as 3 or 4. This can only be sufficient by reason of the perfect steadiness of the load.

507. **Inclined Planes on Canals.**—To save the time and water

expended in shifting boats from one level to another by means of locks, inclined planes are used on some canals. Their general arrangement is as follows:—The upper and lower reach of the canal, at the places which are to be connected by inclined planes, are deepened sufficiently to admit of the introduction of water-tight iron caissons, or moveable tanks, under the boats. Two parallel lines of rails start from the bottom of the lower reach, ascend an inclined plane up to a summit a little above the water-level of the upper reach, and then descend down a short inclined plane to the bottom of the upper reach. There are two caissons, or moveable tanks on wheels, each holding water enough to float a boat. One of these caissons runs on each line of rails; and they are so connected, by means of a chain, or of a wire rope, running on moveable pullies, that when one descends the other ascends. These caissons balance each other at all times when both are on the long incline, because the boats, light or heavy, which they contain, displace exactly their own weight of water. There is a short period when both caissons are in the act of coming out of the water, one at the upper and the other at the lower reach, when the balance is not maintained; and, in order to supply the power required at that time, and to overcome friction, a steam engine drives the main pully, as in the case of fixed-engine planes on railways.*

Boats may be hauled up on wheeled cradles without using caissons; but this requires a greater expenditure of power. Mr. Thomas Grahame has proposed a method of performing this process which would enable a fixed engine to be dispensed with where steamboats are used. It consists in providing each steamer with a windlass, driven by its engine, and the inclined plane simply with a rope, whose upper end is made fast while its lower end is loose. The boat is floated on to the cradle at the bottom of the plane; the loose end of the rope is laid hold of and attached to the windlass, which, being driven by the engine, causes the boat to haul itself up the inclined plane.

On some canals vertical lifts with caissons are used instead of inclined planes.†

508. **Water Supply of Canals.**—Canals are supplied with water from gathering-grounds, springs, rivers, and wells, by the aid of reservoirs and conduits; and their supply involves the same questions of rain-fall, demand, compensation, &c., which have already been treated of in Chapter II. of this Part.

* For a description of an inclined plane of this sort, used on the Monkland Canal near Glasgow, see the *Transactions of the Royal Scottish Society of Arts* for 1852.

† An improved system of apparatus for such lifts, proposed by Mr. George Simpson, is described in the *Transactions of the Institution of Engineers in Scotland* for 1860-61.

The *demand* for water, in the case of a canal, may be estimated as follows:—

I. *Waste of Water* by leakage of the channel, repairs, and evaporation, *per day* = area of surface of the canal × $\frac{1}{6}$ of a foot, nearly.

II. *Current* from the higher towards the lower reaches, produced by leakage at the lock gates, *per day*, from 10,000 to 20,000 cubic feet, in ordinary cases.

III. *Lockage*, or expenditure of water in passing boats from one level to another.

Let L denote a *lockful* of water; that is, the volume contained in the lock-chamber, between the upper and lower water-levels.

B, the volume displaced by a boat.

Then the quantities of water discharged from the upper pond, at a lock or a flight of locks, under various circumstances, are shown in the following tables. The sign — prefixed to a quantity of water denotes that it is displaced *from the lock into the upper pond.*

SINGLE LOCK.	Lock found,	Water discharged.	Lock left,
One boat descending.........	empty,............	$L - B$	empty.
" "	full, ...	$- B$	
One boat ascending,.........	empty or full,....	$L + B$	full.
2 *n* boats, descending and ascending alternately,	descending full, ascending empty	$n L$	descending empty. ascending full.
Train of *n* boats descending,	empty,............	$n L - n B$	empty.
" " "	full, ...	$(n - 1) L - n B$	
Train of *n* boats ascending,	empty or full,...	$n L + n B$	full.
Two trains, each of *n* boats, the first descending, the second ascending,	full, ...	$(2 n - 1) L$	full.

FLIGHT OF *m* LOCKS.	Locks found,	Water discharged.	Locks left,
One boat descending,.........	empty,............	$L - B$	empty.
" "	full, ...	$- B$	
One boat ascending,.........	empty,............	$m L + B$	full.
" "	full, ...	$L + B$	
2 *n* boats, descending and ascending alternately,	descending full, ascending empty	$m n L$	descending empty. ascending full.
Train of *n* boats descending,	empty,............	$n L - n B$	empty.
" " "	full, ...	$(n - 1) L - n B$	
Train of *n* boats ascending,	empty,............	$(m + n - 1) L + n B$	full.
" " "	full, ...	$n L + n B$	
Two trains, each of *n* boats, the first descending, the second ascending,	full, ...	$(m + 2 n - 2) L$	full.

From these calculations it appears, as has been already stated, that single locks are more favourable to economy of water than flights of locks; that at a single lock single boats ascending and descending alternately cause less expenditure of water than equal numbers of boats in trains; and that, on the other hand, at a flight of locks, boats in trains cause less expenditure of water than equal numbers of boats ascending and descending alternately.

For this reason, when a long flight of locks is unavoidable, it is usual to make it double; that is, to have two similar flights side by side—using one exclusively for ascending boats and the other exclusively for descending boats.

Water may be saved at flights of locks by the aid of *side ponds* (sometimes called "lateral reservoirs"). The use of a side pond is to keep for future use a certain portion of the water discharged from a lock, when the locks below it in the flight are full, which water would otherwise be wholly discharged into the lower reach. Let a be the horizontal area of a lock-chamber, A that of its side pond; then the volume of water so saved is—

$$\mathrm{L\,A} \div (\mathrm{A} + a).$$

SECTION II.—*Of River Navigation.*

509. An **Open River** is one in which the water is left to take a continuous declivity, being uninterrupted by weirs. On the subject of such streams little has here to be added to what has already been stated in articles 467 to 471, pp. 707 to 713. The towing-path required, if horse haulage is to be employed, is similar to that of a canal.

The effect of the current of the stream on the load which one horse is able to draw against it at a walk may be roughly estimated as follows:—

$$\text{Load drawn against current} = \text{load drawn in still water} \times \left(\frac{3{\cdot}6}{3{\cdot}6 + v}\right)^2;$$

v being the velocity of the current in feet per second.

It would be foreign to the subject of this work to discuss the principles of the propulsion of vessels by steam and sails.

510. A **Canalized River** is one in which a series of ponds or reaches, with a greater depth of water and a slower current than the river in its natural state, have been produced by means of weirs. The construction and effect of weirs have been explained in Article 472, p. 713, and the previous articles there referred to.

Each weir on a navigable river requires to be traversed by a lock for the passage of vessels, the most convenient place for which is usually near one end of the weir, next the bank where the towing-path is. River locks differ from canal locks in having no lift-wall, so that the head-gates and tail-gates are of equal height.

511. **Moveable Bridges over Rivers** are identical in principle with those over canals, and differ from them only in being of greater size. Examples of them have already been cited in Article 505, p. 706.

CHAPTER IV.

OF TIDAL AND COAST WORKS.

Section I.—*Of Waves and Tides.*

512. **Motion of Ordinary Waves.**—The following description of wave-motion in water is founded chiefly on the theoretical investigations of Mr. Airy and others, and the observations of the Messrs. Weber and of Mr. Scott Russell, with a few additions founded on later researches.

Rolling waves in water are propagated horizontally; the motion of each particle takes place in a vertical plane, parallel to the direction of propagation; the path or orbit described by each particle is approximately elliptic (see fig. 293), and in water of uniform depth the longer axis of the elliptic orbit is horizontal, and the shorter vertical; the centre of that orbit lies a little above the position that the particle occupies when the water is undisturbed; when at the top of its orbit, the particle moves *forwards* as regards the direction of propagation; when at the bottom, backwards, as shown by the curved arrows in fig. 293, in which the straight feathered arrow denotes the direction of propagation.

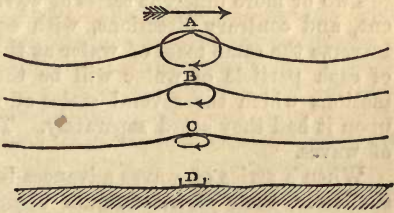

Fig. 293.

The particles at the surface of the water describe the largest orbits; the extent of the motion, both horizontally and vertically, diminishes as the depth below the surface increases; but that of the vertical motion more rapidly than that of the horizontal motion, so that the deeper a particle is situated the more flattened is its orbit, as indicated at A, B, and C; a particle in contact with the bottom moves backwards and forwards in a horizontal straight line, as at D.

In water that is deep, as compared with the length of a wave (or distance between two successive ridges on the surface of the water), the orbits of the particles are nearly circular, and the motion at great depths is insensible.

The *period* of a wave is the time occupied by each particle in making one revolution, and is also the time occupied by a wave in travelling a distance equal to its length. Hence we have the following proportion:—

$$\frac{\text{mean speed of a particle}}{\text{speed of the waves}} = \frac{\text{circumference of particle's orbit}}{\text{length of a wave}}.$$

The *speed of the waves* depends principally on their length and on the depth of water, being greatest for long waves and deep water. When the depth of water is greater than the length of a wave the speed is not sensibly affected by the depth, and is almost exactly equal to the velocity acquired by a body in falling through *half of the radius of a circle whose circumference is the length of a wave.* In water that is very shallow, compared with the length of the waves, the velocity is nearly independent of the length, and is nearly equal to that acquired by a heavy body in falling through *half the depth of the water added to three-fourths of the height of a wave.*

Two or more different series of waves moving in the same, different, and contrary directions, with equal or unequal speeds, may traverse the same mass of water at the same time, and the motion of each particle of water will be the resultant of the respective motions which the several series of waves would have impressed upon it had they acted separately. This is called the *interference* of waves.

When a series of waves advances into water gradually becoming shallower, their *periods* remain unchanged, but their *speed*, and consequently their *length*, diminishes, and their slopes become steeper. The orbits of the particles of water become distorted, as

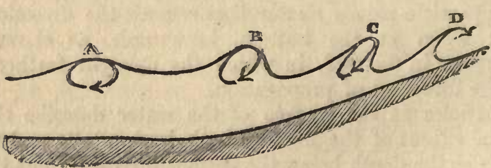

Fig. 294.

at B, C, D, fig. 294, in such a manner that the front of each wave gradually becomes steeper than the back; the crest, as it were, advancing faster than the trough. At length the front of the wave curls over beyond the vertical, its crest falls forward, and it *breaks* into surf on the beach.

As the energy of the motion of a given wave which advances into shallowing water, or up a narrowing inlet, is successively communicated to smaller and smaller masses of water, there is a *tendency* to throw those masses into more and more violent agitation: that tendency may either take effect, or it may be counteracted, or more than counteracted, by the loss of energy which takes place through the production of eddies and surge at sudden changes of depth, and through friction on the bottom.

When waves roll straight against a vertical wall, as in fig. 295, they are reflected, and the particles of water for a certain distance in front of the wall have motions compounded of those due to the direct and to the reflected waves. The results are of the following kind:—The particles in contact with the wall, as at A, move up and down through a height equal to *double* the original height of the waves, and so also do those at half a wave length from the wall, as at C; the particles at a quarter of a wave length from the wall, as at B, move backwards and forwards horizontally, and intermediate particles oscillate in lines inclined at various angles.

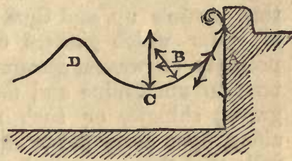

Fig. 295.

In order that a surface may reflect the waves, it is not essential that it should be exactly vertical; according to Mr. Scott Russell, it will do so even with a batter of 45°.

A vertical or steep surface which is wholly covered by the water reflects the wave-motion of those layers of water which lie below its level, and thus a sunken rock or breakwater, even though covered with water to a considerable depth, causes the sea to break over it, and so diminishes the energy of the advancing waves.

The *greatest length* of waves in the ocean is estimated at about 560 feet, which corresponds to a speed of about 53 feet per second, and a period of about 11 seconds. Their *greatest height* is given by Scoresby as about 43 feet, and this, with the period just stated, gives 12 feet per second as the velocity of revolution of the particles of water. (See p. 766.)

In smaller seas the waves are both lower and shorter, and less swift; and, according to Mr. Scott Russell, waves in an expanse of shallow water of nearly uniform depth never exceed in height the undisturbed depth of the water. But the concentration of energy upon small masses of water, which occurs on shelving coasts in the manner already stated, produces waves of heights greatly exceeding those which occur in water of uniform depth, as the following examples show.

Pressures of waves against a vertical surface, at Skerryvore as observed by Mr. Thomas Stevenson:—

	Summer average.	Winter average.	Storms.
In lbs. per square foot, ...	611	2086	6083
In feet of water, ...	9·8	33	97

Greatest height of breakers on the south-west coast of Ireland, as observed by the Earl of Dunraven, 150 feet.

Recent investigations tend towards the conclusion, which is in accordance with observation, that every wave is more or less a "wave of translation," setting down each particle of water, or of matter suspended in water, a little in advance of where it picked that particle up, and thus by degrees producing that heaping up of water which gathers on a lee shore during a storm. This property of waves accounts for the facts, that although they tend to undermine and demolish steep cliffs, they heap up sand, gravel, shingle, or such materials as they are able to sweep along, upon every flat or sloping beach against which they directly roll; that they carry such materials into bays and estuaries; and that when they advance obliquely along the coast they make the materials of the beach travel along the coast in the same direction.

513. **Tides in General.**—The general motion of the tides consists in an alternate vertical rise and fall, and horizontal ebb and flow, occupying an average period of half a lunar day, or about 12·4 hours, and transmitted from place to place in the seas like a series of very long and swift waves, in which the extent of the horizontal motion is very much greater than that of the vertical motion. The extent of motion, both vertical and horizontal, undergoes variations between spring and neap, whose period is half a lunation, and other variations whose periods are a whole lunation and half-a-year. The propagation of the tide-waves is both retarded and deflected in gradually shallowing water, the crests of the waves having a tendency to become parallel to the line of coast which they are approaching.

Tides in narrow seas, and in the neighbourhood of land generally, are modified by the interference of different series of waves arriving by different routes, so as sometimes to present very complex phenomena. (See Mr. Airy's treatise "On Tides and Waves," in the *Encyclopædia Metropolitana.*) In the following examples simple cases only are described.

514. **Tidal Waves in a Clear and Deep Channel** are analogous to ordinary waves, as represented in fig. 293, p. 753; but with the modification that, owing to the enormous length of the waves as compared with the depth of the sea, the extent of horizontal motion is nearly equal at all depths, and the extent of vertical motion in any layer is nearly in the simple proportion of its height above

the bottom. The orbit of each particle is a very long and flat ellipse.

Supposing such a channel as that here considered to have a beach of moderately steep slope at one side, the depth being elsewhere uniform, the particles near that beach move in ellipses situated in planes inclined so as to be nearly parallel to the beach, as represented in plan in figs. 296 and 297. In each of these figures the

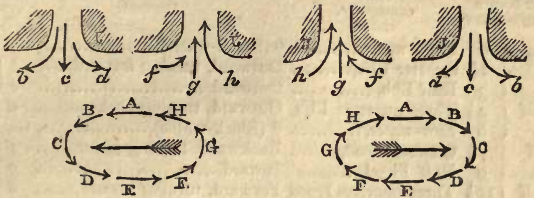

Fig. 296. Fig. 297.

beach is supposed to be towards the top of the page; in fig. 296 it lies to the right hand of the direction of advance of the tide wave (represented by the feathered arrow); in fig. 297, to the left of that direction. The following are the motions of a particle at different times of the tide:—

Lunar Hours after High-water.	Time commonly called.	Current.	Reference to the Figures.
0	High-water,....................	Forward,...........................	A
1½	Quarter Ebb,....................	Forward and Seaward,............	B
3	Half Ebb,........................	Seaward,...........................	C
4½	Three-quarters Ebb,............	Backward and Seaward,..........	D
6	Low-water,........................	Backward,...........................	E
7½	Quarter Flood,....................	Backward and Shoreward,.......	F
9	Half Flood,........................	Shoreward,..........................	G
10½	Three-quarters Flood,...........	Forward and Shoreward,.........	H
12	High-water,.......................	Forward,.............................	A

515. The **Tide in a Short Inlet,** or in any bay, gulf, or estuary of such dimensions and figure that high and low-water occur in all parts of it sensibly at the same instant, is somewhat analogous to a wave rising and falling against a steep wall (fig. 295, p. 755), or to the emptying and filling of a reservoir. Each particle of water moves alternately outwards and inwards during the fall and rise of the tide respectively; and the current is swifter and stronger when the depth of water is greater, that is, *during the second half of flood and the first half of ebb.*

Supposing that the entrance to such an inlet runs at right angles to the line of coast described in the preceding article, the combination of the tidal currents of the inlet with those of the offing, or sea outside, produces the results, as regards the currents at the entrance, indicated by the arrows marked *b, c, d, f, g, h,* in figs. 296 and 297 (whose lengths denote the strength of the current), and explained in the following table, in which *outward* and *inward* refer to the entrance of the inlet, and *forwards* and *backwards* to the directions of currents as compared with that of the flood-current along the coast:—

Lunar Hours after High-water.		Time commonly called	Current.	Reference to the Figures.	
First half of Ebb.	0	High-water,..........	0 (Slack-water),..................	—	
	1½	Quarter Ebb,.........	Outward, turning forward,.....	*b*	Strong.
	3	Half Ebb, ...	Outward, ...	*c*	Strong.
Second half of Ebb.	4½	Three-quarters Ebb,	Outward, turning backward,...	*d*	Weak.
	6	Low-water, ...	0 (Slack-water),..................	—	
First half of Flood.	7½	Quarter Flood,........	Backward, turning inward,....	*f*	Weak.
	9	Half Flood,...........	Inward,............................	*g*	Strong.
Second half of Flood.	10½	Three-quarters Flood	Forward, turning inward,......	*h*	Strong.
	12	High-water,..........	0 (Slack-water),..................	—	

The letter J in each figure marks the *up-stream corner* of the entrance as regards the flood-current along the coast.

The *volume of water* which flows alternately in and out at the entrance of a short inlet is nearly equal to the space between the surfaces of high and low-water, as ascertained by levelling and tide-gauges. The *mean velocity* of the current through the entrance is nearly equal to that volume divided by the mean sectional area of the entrance, and by the time of rise or fall; and the *greatest velocity* is nearly equal to $1\cdot57 \times$ mean velocity. It is best to use such calculations only for the purpose of computing the probable effect of alterations. The velocities of actual currents should be found by observation.

516. The **Tides in Long Inlets** are compounded of a simple emptying and filling current like that in a short inlet, and a series of branch tidal waves, propagated up the channel from the waves of the offing. In *river-channels* the alternate currents due to the tides are combined with the downward current due to the flow of fresh water.

The tidal wave which is propagated up a long inlet or river-channel is analogous to those represented as advancing into shallow water in fig. 294, p. 754. It diminishes in length and increases in height until it reaches a limit where its further increase in height is stopped by friction. Its front becomes shorter and steeper, and its back longer and flatter; in other words, the rise of tide occupies a shorter time, and the fall a longer time, as the wave advances up the channel. When a high tidal wave advances into very shallow water, its front sometimes shortens and steepens, until at length it curls over, like the breaker D in fig. 294, and continues to advance rolling and breaking into surf, followed by a very long flat back. The tidal wave is then called a "*bore.*" The back of the wave sometimes breaks up into two or three smaller waves, and then the fall of the tide is interrupted by short intervals of rise.

To estimate by calculation the velocity of the flood and ebb-currents at a given cross-section of a river-channel or other long inlet, two longitudinal sections of the surface of the water must be prepared from two sets of simultaneous tide-gauge observations, made at a series of stations along the channel and above that cross-section, *at the two instants of slack-water at the given cross-section* respectively. The volume contained between the two surfaces thus determined will be the volume of tidal water which runs in and out through the given cross-section; and this, being divided by the duration of flood and ebb respectively, and by the area, will give the probable mean velocities of the currents, which, being multiplied by 1·57, will give, approximately, the probable maximum velocities. The velocity due to the fresh-water stream, if any, is to be subtracted from the flood and added to the ebb. (See the remark at the end of the preceding article.)

The tidal waves in rivers are propagated up the declivity of the stream, which they often affect at points above the level of high water in the sea.

517. **Actions of Tides on Coasts and Channels.**—The flowing tide augments, and the ebbing tide diminishes, the speed and force of storm waves; and hence the observed fact, that the most powerful action of such waves on the coast occurs after half-flood, when the shoreward current is strong. The tidal currents sweep along with them silt or mud, sand, gravel, and other materials, according to the laws already stated with reference to river currents (Article 468, p. 708); hence the ebbing tide tends to scour and deepen inlets, and the flowing tide to silt them up. From what has been explained in the preceding article, it appears that in shallow water there is a tendency for the flowing tide to become more rapid, and therefore stronger in its action, than the ebbing tide, unless opposed by a sufficiently strong fresh-water current; and hence the prevailing tendency of the tides, like that of the waves, is to choke and fill up estuaries, river-channels, and other inlets, especially such as are already shallow.

A strong fresh-water current may maintain a deep channel against this action of the sea, so far as it is limited in breadth; but where that current escapes into the open sea, and is either enfeebled by spreading laterally, or has its action on the bottom prevented by floating on the salt water, a *bar* is formed by the action of the waves and tides.

One of the chief objects of harbour engineering is so to manage and modify the action of the tidal currents that the ebb shall become stronger than the flood, and shall scour deep channels and remove bars. (See p. 766.)

Section II.—*Of Sea Defences.*

518. **Groins,** running out at right angles to the coast, are constructed in the same manner with groins for river-banks, but more strongly. (Article 469, p. 711.) They not only interrupt the *travelling* of the materials of the beach along the shore under the influence of oblique waves and of the flowing tide, but they also cause a permanent deposit of such materials, and, if gradually extended seaward in shallow water, produce a gain of ground from the sea. After the spaces between the groins have been filled up, the travelling of shingle goes on past their ends as before.

Groins are amongst the most efficient means of protecting dykes, cliffs, and sea-walls, against the undermining action of the sea.

519. An **Earthen Dyke** has usually a long flat slope towards the sea, its inclination ranging from that of 3 to 1 to that of 12 to 1. The top is level, and usually has a roadway upon it: its average usual height above high-water-mark of spring tides, is about 6 feet; it should, if possible, be above the reach of the waves. The back slope has an inclination ranging from that of $1\frac{1}{2}$ to 1 to that of 3 to 1. Behind the dyke is a back drain, or ditch, for the drainage of the land, constructed on the same principles with the back drains mentioned in Article 483, p. 727, and Article 484, p. 728.

In the heart of the dyke is a rectangular wall of fascines, constructed like the fascine-work of a river-bank. (Article 469, p. 710.) The fascines may be made of willow twigs or of reeds. The seaward slope is faced with fascines. If the top is above the reach of the waves the back slope may be turfed; if waves sometimes break over it, the top and back require stone pitching.

520. **Stone Bulwarks** withstand the waves best when either very flat or very steep. They are of two principal kinds—those with a long slope, on which the waves break, as in fig. 294, p. 754, and those with a steep face, which reflect the waves as in fig. 295, p. 755.

I. *Long-sloping Bulwarks* have an inclination which ranges from 3 to 1 to 7 to 1. They are made internally of earth and gravel, or of loose stones, according to the situation, and are faced with blocks, each of which should be able to withstand independently the lifting action of the waves. As to this, see Article 412, p. 618. The foot or "toe" of the slope may be slightly turned up, like that of a weir, to prevent the undermining action of the returning current, or "undertow" from the breakers. (See Article 472, p. 713.)

To prevent breakers or spray from gliding up to the top of the slope, and dashing over the summit of the bulwark, the top of the slope is sometimes curved upwards, so as to present a concave face to the waves; but this is sometimes liable to be knocked down by the shocks which it receives; and in that case it is best to carry up the slope in one plane, with a level *berm* or bench at the top of it, paved with large blocks, and on that berm to erect a strong parapet, set so far back that its cope is below the plane of the slope. A series of level berms, alternating with flat slopes of the same length with the berms, or thereabouts, are very effective in breaking the waves and exhausting their energy; the blocks at the edges of the berms must be larger than the rest.

The largest blocks in the facing of the slope should be at and near half-tide level, because the waves are largest at half-flood.

When a sloping bulwark stands in deep water, the part below low-water-mark may have a steeper slope than that above, as being less violently acted upon by the waves: for example, from 1 to 1 to 3 to 1 below, and from 4 to 1 to 7 to 1 above. The waves will partially break and lose their energy in passing over the place where the inclination changes.

II. A *Steep-faced Bulwark or Sea-Wall* should be proportioned like a reservoir wall. (See Article 465, p. 707.) As to the manner in which it reflects the waves, see Article 512, p. 755. Its cope should either rise above the crests of the highest waves, augmented as they are in height by the reflection, or, should that be impracticable, that cope should be made of stones, each large enough to resist being lifted by the pressure due to the greatest height of a wave above its bed, and dowelled to the adjoining cope-stones. The front edge of the cope should not project beyond the face of the wall, lest the waves overturn it. The remainder of the wall may have a hammer-dressed ashlar or a block-in-course face, backed with coursed rubble or with strong concrete, the whole built in strong hydraulic mortar, and the outer edges of the joints *laid* in cement. (Article 248, p. 389.) The chief danger to the face of such a wall is that air and water should penetrate the joints, and, by their pressure and elasticity, cause stones to jump out after receiving the blow of a wave.

The undermining action of the waves on the ground at the foot of a steep wall is very severe, and should be resisted by a flat stone pitching (which should have no bond or connection with the wall), and by a series of groins. The undermining action may be some what moderated by forming the face of the wall into steps, so as to interrupt the vertical descent of the water.

There are good grounds for believing it to be advantageous to build sea-walls in courses of stones which stand nearly on edge, instead of lying horizontal, in order that each stone may always be loaded with the whole weight of those directly above it.

When there is an earthen embankment behind a sea wall, it should have a retaining wall at the landward side also, to prevent the earth from being washed away by water which may collect on the top.

III. *Combined Wall.*—As the expense of erecting a steep or vertical wall in deep water is very great, it is sometimes combined in such situations with a long slope, in the following manner:—From the bottom up to near low-water-mark extends a slope of 2 to 1 or 3 to 1, terminating in a long level or nearly level *berm* or "*foreshore;*" and on that berm, as on a beach in shallow water, is built a steep wall, at a distance back from the edge of the slope equal to twice or thrice the length of the slope.

521. A **Breakwater**, being placed so as to defend a harbour or roadstead from the waves, differs from a bulwark by having sea at both sides of it. The site of a breakwater should be so chosen as to present a barrier to the waves of the prevailing storms, and especially to those which come along with the flood-current. It may be isolated, and in the midst of the entrance of a bay, as at Plymouth and Cherbourg, or it may run out from the shore into deep water. In the latter case, the best position for the junction of a single breakwater with the land is in general at the *up-stream corner* of the entrance to the inlet or harbour (see Article 515, p. 758), for in that position it opposes the strongest flood-current, and does not interfere with the strongest ebb-current. The principles of the construction of the front of a breakwater are the same with those described in the preceding article with reference to bulwarks in deep water. The back of a vertical-fronted breakwater is usually vertical also; that of a sloping or combined breakwater, if intended to be used as a quay, is vertical; in other cases it differs from the front only in having a steeper slope (from 1 to 1 to $1\frac{1}{2}$ to 1) and being faced with smaller blocks. As to embanking and building under water, see Article 412, p. 617. When a stage supported on screw piles is used to tip the stones from, those piles remain imbedded in the breakwater. Their diameter should be about $\frac{1}{20}$th of their height, so that, in very deep water, they may require to be built of several balks of timber hooped together, as at Portland.

Fig. 298 is a section of the Cherbourg breakwater, which combines the long slope and vertical face. The base A F is about 300 feet; the slope A B is $2\frac{1}{2}$ to 1; B C is $5\frac{1}{2}$ to 1; E F, 1 to 1; C D is a nearly level platform, on which stands the wall G, 36 feet thick at its base. Ordinary spring tides rise 19 feet, the depth at low-water being 40 feet.

Fig. 299, a section of the Plymouth breakwater, illustrates the

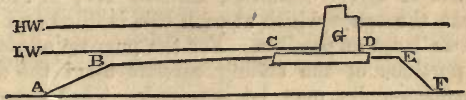

Fig. 298.

principle of alternate slopes and berms. A B is 3 to 1, B C level, C D 5 to 1, D E level, E F $1\frac{1}{2}$ to 1.

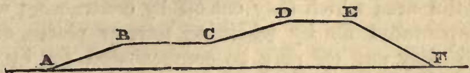

Fig. 299.

(As to breakwaters, and sea defences generally, may be consulted the works of Smeaton and Telford, Sir John Rennie's works *On the Plymouth Breakwater* and *On Harbours*, the *Proceedings of the Institution of Civil Engineers* since the commencement, and Mr. Burnell's *Treatise on Marine Engineering*.) (See also pp. 766, xvi.)

522. **Reclaiming Land.**—The process of reclaiming or gaining land from the sea is to be undertaken with great caution, especially in river-channels and estuaries, lest it should diminish the tidal scour, and so cause the silting up of channels and harbours; and, in particular, care should be taken that the space for tidal water which is to be lost through the reclaiming of the land, is exactly made up for by deepening or otherwise improving other parts of the estuary or channel. In every instance in which that precaution has been neglected, the damage, and in some cases the ruin, of the harbour has followed. (See *Reports of the Tidal Harbours Commission*.)

The first operation in reclaiming land is usually to raise its level as much as possible by *warping*, or deposition of sediment from the tidal water; with a view to which the land to be reclaimed is intersected by a network of transverse wattled groins, and of longitudinal dykes of the same construction.

The ground having been raised as far as practicable by warping, is enclosed with sea-dykes, and drained in the manner described in Article 484, p. 727.

Section III.—*Of Tidal Channels and Harbours.*

523. The **Improvement of Tidal Rivers and Estuaries** depends mainly on the strengthening of the ebbing current, as stated in Article 517, p. 760. With that view, the measures to be adopted

are nearly the same with those already described under the head of Improvements of River-Channels, Article 470, p. 711, with the addition that the space which at each tide is filled and emptied is to be kept as large as possible. For the purpose of concentrating the latter portions of the ebbing current upon the deep-water-channel, *training-dykes* may be required. That these may not diminish the quantity of scouring-water, they should rise but little, if at all, above low-water-mark of ordinary spring-tides, their position being marked by means of rows of beacons.

Should bulwarks or quays be erected, they should either be so placed that the area which they cut off by contracting wide places may be compensated for by widening narrow places, or that the space which they cut off may be compensated for by deepening that part of the space in front of them which is above low-water-mark.

The most important effect of making a deep, direct, and regular channel for a tidal river consists in the increase in the extent of rise and fall of the tide, and the diminution of that steepening action of a shallow channel on the front of the tide-wave which has been described in Article 516, p. 758.

In order to increase the depth over a *bar*, piers or breakwaters must be carried out so as to concentrate the current over it, and it is best, if possible, to make the space between those piers *widen inwards*, in order both to hold scourage-water and to serve as a "wave-trap," or space for storm-waves which roll in at the entrance to spread and expend themselves in. When there is only one pier, it should run from the *up-stream* corner of the entrance, for the reason explained in Article 521, p. 762, observing that in deciding which is the *up-stream* corner, regard must be had to the flood-current *along the shore*, in case, through the action of headlands, its direction should be different from that of the flood-current in the open sea.

The bar may thus be swept into deeper water, although it is in general impossible to remove it altogether.

524. A **Scouring-Basin** is a reservoir by means of which the tidal water is stored up to a certain level, and let out through sluices, in a rapid stream, for a few minutes at low-water, to scour a channel and its bar. The outlets of the basin should face as nearly as possible directly along the channel to be scoured; they should be distributed throughout its whole cross-section, that they may produce an uniform steady current in it like a river, and may not concentrate their action on a few spots. To carry away gravel and large shingle, the scouring stream should flow at 4 or 5 feet per second, and the dimensions of the outlets should be regulated accordingly. One of the best examples of such an arrangement is

at the south entrance of the harbour of Sunderland, described by the engineer, Mr. Murray, in the *Proceedings of the Institution of Civil Engineers* for 1856. The current is let out for 15 minutes at low-water; it runs at about 5 feet per second, and is sensible in the sea 2,000 yards off, although it is confined by piers for 350 yards only.

525. **Quays** of masonry are to be regarded as a class of retaining walls, the stability of which has been treated of in Articles 265 to 269, pp. 401 to 408, and their construction in Articles 271, 272, pp. 409 to 411. Their ordinary thickness at the base is from $\frac{1}{3}$ to $\frac{1}{2}$ of their height. When founded on piles, the timber-work should be always immersed. (See Part II., Chapter VI., Section II., p. 601.) The face of a stone quay is usually protected against being damaged by vessels by means of a network of upright *fender-piles* and horizontal *fender-wales*.

As to timber and iron quays, see Article 469, p. 710, and the other articles there referred to.

The inner side of a breakwater may form a quay, as already mentioned.

526. **Piers** of masonry running out into the sea are to be regarded as upright breakwaters combined with quays, and require here no additional explanation. Those of timber and iron are best formed of a skeleton framework, supported by screw-piles. A timber skeleton-pier is often combined with a loose stone breakwater, in which the lower parts of the posts are imbedded.

527. **Basins and Docks.**—A *deep-water-basin* is a reservoir surrounded by quay-walls, in which the water is retained when the tide falls below a certain level (usually somewhat above half-tide) by a pair of lock-gates opening inwards, of sufficient size and strength. Should the entrance be exposed to waves, a pair of *sea-gates,* or gates opening outwards, are also required, to be closed during storms. A deep-water-basin may also be used as a scouring-basin. (Article 524, p. 764.)

A *dock* differs from a basin in having a *lock* at its entrance, through which ships can pass in all states of the tide. (As to locks, see Article 506, p. 746.) A harbour-lock, like a river-lock, has no lift-wall. In order that vessels may pass easily in and out, the entrances of docks from a river-channel should slant *up-stream as regards the ebb-current.*

One of the best forms of gate for basins and docks is a *caisson-gate,* being a water-tight vessel of plate-iron, which can be floated to or from its seat in the masonry of the entrance, being placed in a recess when open. When closed it is sunk by loading it with water, which is run into a tank on the top of the caisson. In order to open it, it is floated by emptying that tank.

It is often convenient, when practicable, to conduct a supply of fresh water into basins or docks, care being taken that such supply is pure.

528. **Lighthouses.**—The principles which regulate the placing and illuminating of lighthouses form a subject which can be fully considered in a special treatise only, such as that by Mr. Thomas Stevenson. When a lighthouse is exposed to the waves, it may be either a round tower of masonry, built of hewn stones, dove-tailed, tabled, and dowelled to each other, as described in Article 412, p. 618, solid up to the level of high-water of spring tides, and as much higher as ordinary waves rise, and high enough in all to keep the lantern clear of the highest breaking and reflected storm-waves, with an overhanging curved cornice to throw their crests back; or it may consist of a skeleton frame of screw-piles and diagonal bracing, supporting a timber or iron house and platform; and in this case the platform needs only to be high enough to clear the tops of the natural unreflected waves. On the subject of the strength and stability of frames supported on screw-piles, see Article 403, p. 605. In designing the frame of a lighthouse to be supported on them, regard must be had to the pressure of the wind, whose greatest recorded intensity, in Britain, is 55 lbs. per square foot of a flat surface, and about one-half of that intensity per square foot of the plane projection of a cylindrical surface.

Additional Authorities on Harbour and Sea Works.

Minard—*Ouvrages Hydrauliques des Ports de Mer.* Bremner *On Harbours* (Wick, 1845). Thomas Stevenson *On the Design and Construction of Harbours.*

Lighthouses.—Smeaton's *Account of the Eddystone Lighthouse.* Robert Stevenson *On the Bell Rock Lighthouse.* Alan Stevenson *On the Skerryvore Lighthouse.* Alan Stevenson, *Rudimentary Treatise on Lighthouses.* Thomas Stevenson *On Lighthouse Illumination.* Mitchell's "Account of Lighthouses on Screw Piles," in the *Proceedings of the Institution of Civil Engineers* for 1848.

Waves.—J. Scott Russell; *Reports of the British Association* for 1844. G. G. Stokes; *Cambridge Transactions*, 1842, 1850. Earnshaw; *ib.*, 1845. W. Froude; *Transactions of the Institution of Naval Architects*, 1862. Rankine; *Philosophical Transactions*, 1863. Watts, Rankine, Napier, and Barnes, *On Shipbuilding*, 1864. Cialdi; *Sul Moto ondoso del Mare*, 1866. Caligny; *Liouville's Journal*, June and July, 1866.

Addendum to Article 512, p. 755.—Height of Waves.—The height of the waves depends on what is called the "Fetch;" that is, the distance from the weather shore, where their formation commences. According to Mr. Thomas Stevenson, the following formula is nearly correct during heavy gales, when the fetch is not less than about six nautical miles; height in feet $= 1 \cdot 5 \times \sqrt{\text{(fetch in nautical miles)}}$.

Addendum to Article 517, p. 759.—Scouring Action of Tide.—According to Mr. Thomas Stevenson, the sectional area of many estuaries at *low water* bears a nearly constant proportion to the volume of water which runs in and out at each tide, being from 7½ to 10 square feet of area for each 1,000,000 cubic feet of tidal water.

APPENDIX.

I.

TABLE OF THE RESISTANCE OF MATERIALS TO STRETCHING AND TEARING BY A DIRECT PULL, *in pounds avoirdupois per square inch.*

MATERIALS.	Tenacity, or Resistance to Tearing.	Modulus of Elasticity, or Resistance to Stretching.
STONES, NATURAL AND ARTIFICIAL:		
Brick, Cement,	280 to 300	
Glass,	9,400	8,000,000
Slate,	9,600 to 12,800	13,000,000 to 16,000,000
Mortar, ordinary,	50	
METALS:		
Brass, cast,	18,000	9,170,000
" wire,	49,000	14,230,000
Bronze or Gun Metal (Copper 8, Tin 1),	36,000	9,900,000
Copper, cast,	19,000	
" sheet,	30,000	
" bolts,	36,000	
" wire,	60,000	17,000,000
Iron, cast, various qualities,	13,400 to 29,000	14,000,000 to 22,900,000
" average,	16,500	17,000,000
Iron, wrought, plates,	51,000	
" joints, double rivetted,	35,700	
" " single rivetted,	28,600	
" bars and bolts,	60,000 to 70,000	29,000,000
" hoop, best-best,	64,000	
" wire,	70,000 to 100,000	25,300,000
" wire-ropes,	90,000	15,000,000
Lead, sheet,	3,300	720,000
Steel bars,	100,000 to 130,000	29,000,000 to 42,000,000
Steel plates, average,	80,000	
Tin, cast,	4,600	
Zinc,	7,000 to 8,000	

Materials.	Tenacity, or Resistance to Tearing.	Modulus of Elasticity, or Resistance to Stretching.
Timber and other Organic Fibre:		
Acacia, false. See "Locust."		
Ash (*Fraxinus excelsior*),..........	17,000	1,600,000
Bamboo (*Bambusa arundinacea*),	6,300	
Beech (*Fagus sylvatica*), ...	11,500	1,350,000
Birch (*Betula alba*), ...	15,000	1,645,000
Box (*Buxus sempervirens*),........	20,000	
Cedar of Lebanon (*Cedrus Libani*),	11,400	486,000
Chestnut (*Castanea Vesca*),.......	10,000 to 13,000	1,140,000
Elm (*Ulmus campestris*),..........	14,000	700,000 to 1,340,000
Fir: Red Pine (*Pinus sylvestris*),	12,000 to 14,000	1,460,000 to 1,900,000
" Spruce (*Abies excelsa*),.......	12,400	1,400,000 to 1,800,000
" Larch (*Larix Europæa*),.....	9,000 to 10,000	900,000 to 1,360,000
Flaxen Yarn,..................about	25,000	
Hazel (*Corylus Avellana*),.........	18,000	
Hempen Ropes,..........from 12,000 to	16,000	
Hide, Ox, undressed,..............	6,300	
Hornbeam (*Carpinus Betulus*),...	20,000	
Lancewood (*Guatteria virgata*),...	23,400	
Leather, Ox,.........................	4,200	24,300
Lignum-Vitæ (*Guaiacum officinale*),..............................	11,800	
Locust (*Robinia Pseudo-Acacia*),	16,000	
Mahogany (*Swietenia Mahagoni*),	8,000 to 21,800	1,255,000
Maple (*Acer campestris*), ...	10,600	
Oak, European (*Quercus sessiliflora* and *Quercus pedunculata*),	10,000 to 19,800	1,200,000 to 1,750,000
" American Red (*Quercus rubra*),..............................	10,250	2,150,000
Silk Fibre,...........................	52,000	1,300,000
Sycamore (*Acer Pseudo-Platanus*),	13,000	1,040,000
Teak, Indian (*Tectona grandis*),	15,000	2,400,000
" African, (?).................	21,000	2,300,000
Whalebone,.........................	7,700	
Yew (*Taxus baccata*),..	8,000	

II.

TABLE OF THE RESISTANCE OF MATERIALS TO SHEARING AND DISTORTION, *in pounds avoirdupois per square inch.*

MATERIALS.	Resistance to Shearing.	Transverse Elasticity, or Resistance to Distortion.
METALS:		
Brass, wire-drawn,		5,330,000
Copper,		6,200,000
Iron, cast,	27,700	2,850,000
„ wrought,	50,000	8,500,000 to 10,000,000
TIMBER:		
Fir: Red Pine,	500 to 800	62,000 to 116,000
„ Spruce,	600	......
„ Larch,	970 to 1,700	......
Oak,	2,300	82,000
Ash and Elm,	1,400	76,000

III.

TABLE OF THE RESISTANCE OF MATERIALS TO CRUSHING BY A DIRECT THRUST, *in pounds avoirdupois per square inch.*

MATERIALS.	Resistance to Crushing.
STONES, NATURAL AND ARTIFICIAL: (see also page 361).	
Brick, weak red,	550 to 800
„ strong red,	1,100
„ fire,	1,700
Chalk,	330
Granite,	5,500 to 11,000
Limestone, marble,	5,500
„ granular,	4,000 to 4,500
Sandstone, strong,	5,500
„ ordinary,	3,300 to 4,400
„ weak,	2,200
Rubble masonry, about four-tenths of cut stone.	
METALS:	
Brass, cast,	10,300
Iron, cast, various qualities,	80,000 to 145,000
„ „ average,	112,000
„ wrought,	about 36,000 to 40,000

Materials.	Resistance to Crushing.
Timber,* Dry, crushed along the grain:	
Ash,	9,000
Beech,	9,360
Birch,	6,400
Blue-Gum (*Eucalyptus Globulus*),	8,800
Box,	10,300
Bullet-tree (*Achras Sideroxylon*),	14,000
Cabacalli,	9,900
Cedar of Lebanon,	5,860
Ebony, West Indian (*Brya Ebenus*),	19,000
Elm,	10,300
Fir: Red Pine,	5,375 to 6,200
,, American Yellow Pine (*Pinus variabilis*),	5,400
,, Larch,	5,570
Hornbeam,	7,300
Lignum-Vitæ,	9,900
Mahogany,	8,200
Mora (*Mora excelsa*),	9,900
Oak, British,	10,000
,, Dantzic,	7,700
,, American Red,	6,000
Teak, Indian,	12,000
Water-Gum (*Tristania nerifolia*),	11,000

IV

Table of the Resistance of Materials to Breaking Across, *in pounds avoirdupois per square inch.*

Materials.	Resistance to Breaking, or Modulus of Rupture.†
Stones:	
Sandstone,	1,100 to 2,360
Slate,	5,000

* The resistances stated are for *dry* timber. Green timber is much weaker, having sometimes only half the strength of dry timber against crushing.

† The modulus of rupture is eighteen times the load which is required to break a bar of one inch square, supported at two points one foot apart, and loaded in the middle between the points of support.

MATERIALS.	Resistance to Breaking, or Modulus of Rupture.
METALS:	
Iron, cast, open-work beams, average, ...	17,000
„ „ solid rectangular bars, var. qualities,	33,000 to 43,500
„ wrought, plate beams, ...	42,000
Steel, average,......................................	80,000
TIMBER:	
Ash,...	12,000 to 14,000
Beech,..	9,000 to 12,000
Birch, ...	11,700
Blue-Gum,..	16,000 to 20,000
Bullet-tree, ...	15,900 to 22,000
Cabacalli,...	15,000 to 16,000
Cedar of Lebanon,.............................	7,400
Chestnut,..	10,660
Cowrie (*Dammara australis*), ...	11,000
Ebony, West Indian, ...	27,000
Elm,..	6,000 to 9,700
Fir: Red Pine, ...	7,100 to 9,540
„ Spruce,..	9,900 to 12,300
„ Larch,..	5,000 to 10,000
Greenheart (*Nectandra Rodiœi*),.................	16,500 to 27,500
Lancewood, ...	17,350
Lignum-Vitæ,......................................	12,000
Locust, ...	11,200
Mahogany, Honduras,..............................	11,500
„ Spanish, ...	7,600
Mora,...	22,000
Oak, British and Russian,........................	10,000 to 13,600
„ Dantzic, ...	8,700
„ American Red,..................................	10,600
Poon,...	13,300
Saul,..	16,300 to 20,700
Sycamore, ...	9,600
Teak, Indian,......................................	12,000 to 19,000
„ African,..	14,980
Tonka (*Dipteryx odorata*), ...	22,000
Water-Gum, ...	17,460
Willow (*Salix*, various species),..................	6,600

V.—Comparative Table of French and British Measures.

	No.	Log.	Log.	No.	
Grains in a gramme,...	15·43235	1·188432	$\bar{2}$·811568	0·064799	Gramme in a grain.
Pounds avoird. in a kilogramme,	2·20462	0·343334	$\bar{1}$·656666	0·453593	Kilog. in a lb. avoirdupois.
Ton in a tonne,....................	0·984206	$\bar{1}$·993086	0·006914	1·01605	Tonnes in a ton.
Feet in a mètre, ...	3·2808693	0·515989	$\bar{1}$·484011	0·3047972 1	Mètres in a foot.
Inch in a millimètre,............	0·03937043	$\bar{2}$·595170	1·404830	25·39977	Millimètres in an inch.
Mile in a kilomètre, ...	0·621377	$\bar{1}$·793355	0·206645	1·60933	Kilomètres in a mile.
Square feet in a square mètre,..	10·7641	1·031978	$\bar{2}$·968022	0·0929013	Square mètre in a square foot.
Square inch in a square millimètre, ...	0·00155003	$\bar{3}$·190340	2·809660	645·148	Square millim. in a square inch.
Cubic feet in a cubic mètre,.....	35·3156	1·547967	$\bar{2}$·452033	0·0283161	Cubic mètre in a cubic foot.
Foot-pounds in a kilogrammètre,	7·23308	0·859323	$\bar{1}$·140677	0·138254	Kilogrammètre in a foot-pound.
Pounds-to-the-foot in a kilogramme-to-the-mètre,......	0·671963	$\bar{1}$·827345	0·172655	1·48818	Kilogrammes-to-the-mètre in a pound-to-the-foot.
Pounds-to-the-square-foot in a kilogramme-to-the-square-mètre, ...	0·204813	$\bar{1}$·311356	0·688644	4·88252	Kilogrammes-to-the-square-mètre in a pound-to-the-square-foot.
Pounds-to-the-square-inch in a kilog.-to-the-square-millimètre,........................	1422·31	3·152994	$\bar{4}$·847006	0·000703083	Kilog.-to-the-square-millimètre in a pound-to-the-square-inch.
Pounds-to-the-cubic-foot in a kilogramme-to-the-cubic-mètre,........................	0·062426	$\bar{2}$·795367	1·204633	16·019	Kilogrammes-to-the-cubic-mètre in a pound-to-the-cubic-foot.
Fahrenheit-degrees in a centigrade-degree,.................	1·8	0·255273	$\bar{1}$·744727	0·5555$\dot{5}$	Centigrade-degree in a Fahrenheit degree.
British units of heat in a French unit,..................	3·96832	0·598607	$\bar{1}$·401393	0·251996	French units of heat in a British unit.

VI.

TABLE OF SPECIFIC GRAVITIES OF MATERIALS.

GASES, at 32° Fahr., and under the pressure of one atmosphere, of 2116·3 lb. on the square foot:	Weight of a cubic foot in lb. avoirdupois.
Air,	0·080728
Carbonic Acid,	0·12344
Hydrogen,	0·005592
Oxygen,	0·089256
Nitrogen,	0·078596
Steam (ideal),	0·05022
Æther vapour (ideal),	0·2093
Bisulphuret-of-carbon vapour (ideal),	0·2137
Olefiant gas,	0·0795

	Weight of a cubic foot in lb. avoirdupois.	Specific gravity, pure water = 1.
LIQUIDS at 32° Fahr. (except Water, which is taken at 39°·1 Fahr.):		
Water, pure, at 39°·1,	62·425	1·000
" sea, ordinary,	64·05	1·026
Alcohol, pure,	49·38	0·791
" proof spirit,	57·18	0·916
Æther,	44·70	0·716
Mercury,	848·75	13·596
Naphtha,	52·94	0·848
Oil, linseed,	58·68	0·940
" olive,	57·12	0·915
" whale,	57·62	0·923
" of turpentine,	54·31	0·870
Petroleum,	54·81	0·878
SOLID MINERAL SUBSTANCES, non-metallic:		
Basalt,	187·3	3·00
Brick,	125 to 135	2 to 2·167
Brickwork,	112	1·8
Chalk,	117 to 174	1·87 to 2·78
Clay,	120	1·92
Coal, anthracite,	100	1·602
" bituminous,	77·4 to 89·9	1·24 to 1·44
Coke,	62·43 to 103·6	1·00 to 1·66
Felspar,	162·3	2·6
Flint,	164·2	2·63

	Weight of a cubic foot in lb. avoirdupois.	Specific gravity, pure water = 1.
SOLID MINERAL SUBSTANCES—*continued.*		
Glass, crown, average,...........	156	2·5
„ flint, „	187	3·0
„ green, „	169	2·7
„ plate, „	169	2·7
Granite, ...	164 to 172	2·63 to 2·76
Gypsum,.........................	143·6	2·3
Limestone (including marble),..	169 to 175	2·7 to 2·8
„ magnesian,..........	178	2·86
Marl,..............................	100 to 119	1·6 to 1·9
Masonry,.........................	116 to 144	1·85 to 2·3
Mortar,...........................	109	1·75
Mud,..............................	102	1·63
Quartz,............................	165	2·65
Sand (damp),.....................	118	1·9
„ (dry),........................	88·6	1·42
Sandstone, average,...............	144	2·3
„ various kinds,........	130 to 157	2·08 to 2·52
Shale,.............................	162	2·6
Slate,.............................	175 to 181	2·8 to 2·9
Trap,..............................	170	2·72
METALS, solid:		
Brass, cast,........................	487 to 524·4	7·8 to 8·4
„ wire,........................	533	8·54
Bronze,............................	524	8·4
Copper, cast,......................	537	8·6
„ sheet,......................	549	8·8
„ hammered,...............	556	8·9
Gold,..............................	1186 to 1224	19 to 19·6
Iron, cast, various,...............	434 to 456	6·95 to 7·3
„ average,...............	444	7·11
Iron, wrought, various,..........	474 to 487	7·6 to 7·8
„ average,.........	480	7·69
Lead,..............................	712	11·4
Platinum,.........................	1311 to 1373	21 to 22
Silver,.............................	655	10·5
Steel,..............................	487 to 493	7·8 to 7·9
Tin,................................	456 to 468	7·3 to 7·5
Zinc,..............................	424 to 449	6·8 to 7·2

Timber:*	Weight of a cubic foot in lb. avoirdupois.	Specific gravity, pure water = 1.
Ash,	47	0·753
Bamboo,	25	0·4
Beech,	43	0·69
Birch,	44·4	0·711
Blue-Gum,	52·5	0·843
Box,	60	0·96
Bullet-tree,	65·3	1·046
Cabacalli,	56·2	0·9
Cedar of Lebanon,	30·4	0·486
Chestnut,	33·4	0·535
Cowrie,	36·2	0·579
Ebony, West Indian,	74·5	1·193
Elm,	34	0·544
Fir: Red Pine,	30 to 44	0·48 to 0·7
„ Spruce,	30 to 44	0·48 to 0·7
„ American Yellow Pine,	29	0·46
„ Larch,	31 to 35	0·5 to 0·56
Greenheart,	62·5	1·001
Hawthorn,	57	0·91
Hazel,	54	0·86
Holly,	47	0·76
Hornbeam,	47	0·76
Laburnum,	57	0·92
Lancewood,	42 to 63	0·675 to 1·01
Larch. See "Fir."		
Lignum-Vitæ,	41 to 83	0·65 to 1·33
Locust,	44	0·71
Mahogany, Honduras,	35	0·56
„ Spanish,	53	0·85
Maple,	49	0·79
Mora,	57	0·92
Oak, European,	43 to 62	0·69 to 0·99
„ American Red,	54	0·87
Poon,	36	0·58
Saul,	60	0·96
Sycamore,	37	0·59
Teak, Indian,	41 to 55	0·66 to 0·88
„ African,	61	0·98
Tonka,	62 to 66	0·99 to 1·06
Water-Gum,	62·5	1·001
Willow,	25	0·4
Yew,	50	0·8

* The Timber in every case is supposed to be dry.

TABLE OF SQUARES AND FIFTH POWERS.

	Square.	Fifth Power.		Square.	Fifth Power.
10	1 00	1 00000	55	30 25	5032 84375
11	1 21	1 61051	56	31 36	5507 31776
12	1 44	2 48832	57	32 49	6016 92057
13	1 69	3 71293	58	33 64	6563 56768
14	1 96	5 37824	59	34 81	7149 24299
15	2 25	7 59375	60	36 00	7776 00000
16	2 56	10 48576	61	37 21	8445 96301
17	2 89	14 19857	62	38 44	9161 32832
18	3 24	18 89568	63	39 69	9924 36543
19	3 61	24 76099	64	40 96	10737 41824
20	4 00	32 00000	65	42 25	11602 90625
21	4 41	40 84101	66	43 56	12523 32576
22	4 84	51 53632	67	44 89	13501 25107
23	5 29	64 36343	68	46 24	14539 33568
24	5 76	79 62624	69	47 61	15640 31349
25	6 25	97 65625	70	49 00	16807 00000
26	6 76	118 81376	71	50 41	18042 29351
27	7 29	143 48907	72	51 84	19349 17632
28	7 84	172 10368	73	53 29	20730 71593
29	8 41	205 11149	74	54 76	22190 06624
30	9 00	243 00000	75	56 25	23730 46875
31	9 61	286 29151	76	57 76	25355 25376
32	10 24	335 54432	77	59 29	27067 84157
33	10 89	391 35393	78	60 84	28871 74368
34	11 56	454 35424	79	62 41	30770 56399
35	12 25	525 21875	80	64 00	32768 00000
36	12 96	604 66176	81	65 61	34867 84401
37	13 69	693 43957	82	67 24	37073 98439
38	14 44	792 35168	83	68 89	39390 40643
39	15 21	902 24199	84	70 56	41821 19424
40	16 00	1024 00000	85	72 25	44370 53125
41	16 81	1158 56201	86	73 96	47042 70176
42	17 64	1306 91232	87	75 69	49842 09207
43	18 49	1470 08443	88	77 44	52773 19168
44	19 36	1649 16224	89	79 21	55840 59449
45	20 25	1845 28125	90	81 00	59049 00000
46	21 16	2059 62976	91	82 81	62403 21451
47	22 09	2293 45007	92	84 64	65908 15232
48	23 04	2548 03968	93	86 49	69568 83693
49	24 01	2824 75249	94	88 36	73390 40224
50	25 00	3125 00000	95	90 25	77378 09375
51	26 01	3450 25251	96	92 16	81537 26976
52	27 04	3802 04032	97	94 09	85873 40257
53	28 09	4181 95493	98	96 04	90392 07968
54	29 16	4591 65024	99	98 01	95099 00499

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Addendum to Article 57, p. 91.

Levelling by the Barometer.—To correct the difference of level given by the formula in the text for variations in the force of gravity, multiply by

$$1 + 0{\cdot}00284 \cos. 2\lambda + \frac{h}{10,450,000};$$

in which λ is the mean latitude of the two stations, and h the mean of their heights in feet above the level of the sea.

Addendum to Article 406, p. 607.

Tubular Foundations.—For excavating the earth inside iron cylinders that are being sunk for foundations, Mr. Milroy introduced the following digging apparatus. A polygonal iron frame is suspended in a horizontal position by means of chains. It has hinged to it, by their broad ends, a set of triangular, or nearly triangular shovels, which, when they are supported by catches in a horizontal position, with their small ends meeting at the centre of the frame, form a sort of flat tray. When the catches are let go, the shovels hang with their points downwards. In this position they are lowered, and forced into the earth at the bottom of the cylinder. The points of the shovels are then hauled together by means of chains, so as to form the tray, which is wound up with its load of earth by means of a steam windlass; a truck is wheeled upon rails over the mouth of the cylinder, so as to be under the tray; the catches are let go, so as to drop the shovels, and let the earth fall into the truck, which is wheeled away; and the apparatus is ready to be lowered again. By means of this apparatus, cylinders 8 feet 4 inches in diameter, together with the excavation inside, have been sunk at the average rate of about a foot an hour, including stoppages to put on new lengths of cylinder. The numbers of men employed were, one at the winding steam engine, six at rollers for hauling chains to force the shovels into the ground, and afterwards to pull their points together; three at the truck, and one with a shovel and barrow; in all, eleven men; but several of those men might be saved by working the chains from the steam engine.

Addendum to Article 435, p. 656.

Narrow Gauge Railways.—Railways of gauges smaller than that commonly called the "narrow gauge" are used where the traffic is light, and cheapness of first cost is important. Some Norwegian railways have a gauge of 3½ feet.

The Festiniog Railway in North Wales has a gauge of only 2 feet. The rails weigh 30 lbs. to the yard, and are in lengths of 18 and 21 feet. The intermediate chairs weigh 10 lbs.; the joint chairs, 13 lbs. The sleepers are of larch, 4 feet 6 inches long, from 9 inches to 10 inches broad, and from 4½ inches to 5 inches deep. At each side of a joint they are 1 foot 6 inches apart from centre to centre; elsewhere, 2 feet 8 inches. Clear width of roadway for a single line, 12 feet; central space of a double line, 7 feet; clear width, 21 feet. Sharp curves, from 2 to 4 chains radius. Steepest gradient on passenger line, 1 in 80 nearly; elsewhere, 1 in 60. Passenger carriages 10 feet long, 6 feet 3 inches wide, 6 feet 6 inches high; four wheels, 1 foot 6 inches diameter; wheel base, 4 feet; carry 10 passengers, in two rows, back to back. Engine weighs when full, 7½ tons; four wheels, coupled, 2 feet diameter; wheel base, 5 feet. Two outside cylinders, 8 inches diameter, 12-inch stroke; greatest steam-pressure, 200 lbs. on the square inch above atmosphere. Water carried in a tank over the boiler; fuel in a 4-wheeled tender. The ordinary speed ascending 1 in 80, with a gross load of 50 tons, exclusive of engine and tender, is 10 miles an hour. As to "Fairlie" engines, which are well suited for narrow gauge railways, see p. 649.

Addendum to Article 430, p. 639.

Wire Tramways.—The following description of Hodgson's Wire-rope Transport System is abridged from a published pamphlet on that subject:—

"Lines of this system . . . may be described as consisting of an endless wire-rope, supported on a series of pulleys carried by substantial posts, which are ordinarily about 300 feet apart; but, where necessary, much longer spans are taken, in many cases amounting to 1,000 feet. This rope passes at one end of the line round a drum, driven by a steam engine, or other available power, at a speed of from four to eight miles an hour. The boxes in which the load is carried are hung on the rope at the loading end, the attachment consisting of a pendant of peculiar shape, which maintains the load in perfect equilibrium, and at the same time enables it to pass the supporting pulleys with ease. Each of these boxes carries from 1 cwt. to 10 cwt., and the delivery is at the rate of about 200 boxes per hour for the entire distance. . . . A special arrangement is made at each end of the line, consisting of rails so placed as to receive the small wheels with which the boxes are provided, and deliver them from the rope. The boxes thus become suspended from a fixed rail instead of the moving rope, and can be run to any point to which the rail is carried, for loading or delivering, and again run on to the rope, for returning. The succession is continuous, and the rope is never required to stop. . . . Curves are easily passed, and inclines of 1 in 6 or 7 are admissible. . . . The rope being continuous, no power is lost on undulating ground. . . ."

BELL AND BAIN, PRINTERS, GLASGOW.