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LOCKS ^{ON} DEEP WATERWAYS.

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for C. E. Degree. . . . June . 1900. —————

LOCKS ON DEEP-WATER-WAYS.

Being an outline of the course pursued in the investigation of the proper shape, size, and strength, of the walls, floors, miter-walls, &c for locks on Deep-Waterways, built to accommodate Lake and Ocean going vessels, of the greatest draughts and widths.

The locks investigated are for those on canals, having a minimum depth of thirty feet. This being assumed to be the depth necessary for largest vessels.

The lock consists of a closed chamber, with walls on the sides, and gates at the ends. Culverts in the walls, provide for the filling and emptying of this chamber, as occasion requires. The length is to be 740 ft. between the quoins of the lock gates, and all those investigated are of similar construction, differing only in the height of the lift, size of the walls, floors, &c. These are dependent upon,-

(1st.) the difference of level to be overcome;

(2nd.) the kind of material found at the lock-site; and

(3d.) where special conditions have to be met, such as

(a), one side of the lock adjacent to a body of water;

(b), being one of a flight of locks, and

(c), a wide difference between the high and low, or normal, water elevations, either above or below the locks.

The locks are to be of portland cement concrete throughout. Those points, where the concrete would probably suffer, by reason of being exposed to (1st.) the rush of the water in filling or emptying the lock,

(2nd.) the impact of a moving boat;

(3d.) or the wear or pressure of machinery,

will be faced and protected by granite masonry, designed and placed for the special protection it is to afford.

This is true at the culvert entrances and exits, the linings of the culverts, where they depart from a straight line, the tops of the miter-walls, bearings, for the quoin posts of the gates, and for the valves,

ends of the gate-recesses, &c.

The discussion of the lock will now be divided into its several parts, to show in a better manner how each was designed, and the duties assigned to each. They are: (a), Side-walls, (b), Floors, (c), Culverts (d), Miter-walls, Sills, &c., (e), Gates, and (f), Operating Plants.

Side-walls.

When by the aid of ordinary borings, the lock-site is shown to have a rock bottom, and by the aid of diamond drill borings, the rock is shown to be of such a character, as to leave no doubts to its ability to give a solid and firm foundation for the structure, then the side-walls are designed according to well known rules governing retaining walls. The earth pressures in such cases will vary, depending upon the weight per cu. ft., and the angle of repose assumed for any particular material. The pressures of the back-filling may be obtained by graphic methods or by formulae. One of the graphic methods, (Rebhan's), follows:-

Case 1.

$$\begin{aligned} AF^2 &= AD \cdot AC \\ FG &\parallel BD \\ FH &= FG \\ \triangle FGH &= \text{Press. } \triangle \end{aligned}$$

$$\begin{aligned} \gamma &= \phi + \epsilon \\ \phi &= \text{angle of repose.} \\ \text{Part at } \frac{2}{3} \end{aligned}$$

Special case, - AB Vert., $\phi = 0$, then $\alpha - \phi = 90^\circ - \phi$ & earth pr. = $\frac{1}{2} \gamma h^2 \tan^2 (45^\circ - \frac{\phi}{2})$.
If $\phi = 0$ (i.e. no friction between earth and wall), then for greatest pressure, the line of rupture will divide α ($\alpha - \phi$)

Case 2.
(where γ (above Case 1) is greater than $G \cdot B \cdot A$.)

$$\begin{aligned} BC &\parallel AX \\ AJ &= AG \\ GG_1 &\parallel AX \therefore AG = \text{Line of Rupture} \\ GL &\parallel AB \\ ML &\parallel AX \\ \text{Earth pr.} &= \frac{LM \times BB_1}{2} \gamma \end{aligned}$$

Rankine's Formula for earth pressures, $P = \frac{\gamma h^2}{2} \frac{1 - \sin \phi}{1 + \sin \phi}$,

This was most generally used, where a fair knowledge of the conditions at the lock site were at hand. In the above formula, h = height of the wall in feet; γ = Wt. of material per cu. ft.; and ϕ = the angle of repose of the material, behind the wall. Values of ϕ were used varying from 15 to 45 degrees. The following table gives the values of the constant, $\frac{1 - \sin \phi}{1 + \sin \phi}$, for different values of ϕ :-

15,	20,	25,	30,	35,	40,	45,
.588	.490	.406	.333	.271	.217	.172

These values were used in the above formula, varying them and γ to suit the conditions where the lock was to be placed, and letting P act at $h/3$ from the base of the wall, horizontally. Where the back of the wall was sloped, then the earth triangle resting upon the wall, was added to its effective weight.

Figures 1 and 2, Plate 2, shows the manner of finding a satisfactory section for a wall, under certain conditions. Fig. 1 was taken 40 ft. wide at the base, 87.5 ft. high, other dimensions as shown, and the results obtained were as follows:- The center of gravity of the section, plus the earth triangle on the back, (reduced to its masonry equivalent, with a wt. of 120# for the earth, and 140# for the concrete), equals 19.86 ft. from the face of the wall. Earth pressure by Rankine, with $h = 52$ ft., $\phi = 33$, gives 47,780 #, acting at 52 minus 35.5 ft. above base of wall. ($\gamma = 120$ # per cu. ft.) $P = 65,788$ #, using $\phi = 25$ degrees, $h = 52$ ft., and $\gamma = 120$ #, in the formula. Pressure of 83 ft. head of water = 215,381 #.

R is the resultant of wt. of wall plus wt. of the earth triangle, acting through gravity line, combined with the earth pressure of 52 ft., at 120 # per cu. ft. and $\phi = 25$ degrees. $R' = R$ combined with 35.5 ft. water pressure, acting at $\frac{35.5}{3}$ above base of wall, horizontally. R'' = resultant of wt. of wall plus earth triangle at 120#, combined with earth pressure at 120# per cu ft, and with $\phi = 33$ degrees. $R''' = R''$ combined with 35.5 ft. of water pressure, acting at $\frac{35.5}{3}$ from the base. R'''' combined with 83 ft. water pressure on face, throws R'''' outside of the middle third.

Then we take Fig. 2, using same ht. of wall, but a different width of base, and we get the following results:- C. G. of wall, plus earth triangle, (at 120#), equals 21.52 ft. from the face of the wall. Wt. of wall plus earth triangle = 512,163 #. Pressures are the same as for the first case considered. R = resultant of the wt. of wall and earth triangle, ($\gamma = 120\#$, $\phi = 25$, $h = 52$ ft.), with the pressure from the backing using the foregoing values in the formula. R' = resultant of R and 35.5 ft. of water pressure, acting at $\frac{35.5}{3}$ above base. R'' = resultant of wt. of wall and earth triangle at 120#, with the earth pressure found with $\gamma = 120\#$ and $\phi = 33$ degrees, acting at 52.8 above base of wall. R''' = resultant of R'' and 35.5 ft. of water pressure, acting at $\frac{35.5}{3}$ above base of wall. R^{IV} = resultant of R''' and 33 ft. water pressure on the face of the wall. R^{IV} does not go out of the middle third, and consequently, as it just strikes the outer edge of the middle third, it is taken as a satisfactory section.

In the manner above described, by several trials, walls were found that would be satisfactory under any given conditions. An approximate rule found to apply to most cases, found by trying walls of different heights, and for the severest cases of earth pressures liable to obtain with them, was to make the walls half their height in thickness at the base.

The main walls of the lock, (those not back of the gates or gate recesses), are all 12 ft. wide on top, and their general design is shown in Fig. 1, Plate 2. The upper five ft. on the back is sloped 2 in 5, thus producing a frost line, this being necessary in cold climates to preserve the upper part of the wall from the action of the frost.

The walls back of the gates are designed to take an additional thrust due to the pressure received from the gates, and for a distance of 60 ft. above and 40 ft. below the quoin, are made thicker. And owing to the need of gate-anchorage near the top of the wall, they are made of a form shown in Fig. 2, Plate 2. The amount of concrete in the walls can be readily obtained from the simple figure, or from tables easily prepared.

If the lock for good reasons, is to be placed where the rock is too far below the surface, for the walls to rest upon it, then a special treatment of the case becomes necessary. The strength of each wall separately cannot be considered, but the entire cross-section of the lock. When the strata is at all of a yielding character, cracks are apt to occur in the floor invert, especially near the center. These cracks are caused by the settlement of the side-walls, due to the greater pressure their weight throws upon the foundations, inducing a spreading or bearing action on the invert of the floor. This tendency to crack can only be remedied by distributing the weights over as large an area as possible, careful attention to the foundations, and the adjustment of the pressures due to weight of backing and side-walls combined, so that the resultant may fall somewhat nearer the front or toe, than is usually considered desirable in ordinary retaining walls. This will produce a slight inward tendency of the side-walls, which will however be resisted by the invert of the floor. Such cracks are not necessarily of a destructive character and after being closed with concrete are not likely to give further trouble, unless by a further settling of the side-wall the same cracks are caused to open or new ones to appear.

The nature of the foundations adopted for the walls must depend upon the nature of the soil to be built upon. When a firm stratum can be reached, then it is worth while to extend the excavations and foundations in order to build upon it. When however soft and silty soil extends for some distance below the level proposed for the foundations, it becomes necessary to adopt some other system than the mere deepening of the foundations. Bearing piles driven some distance, and connected at the top by grillage, a layer of concrete, or both, are employed to support the walls on soils too soft to bear the weight without settlement. The piles sustain the walls, either by resting on the hard stratum below, or by the adherence of their sides; and the grillage or concrete serve to distribute the load. The supporting power of soft soils

is augmented by enclosing it within sheet-piling, thus preventing its displacement, when the weight of the walls come upon it. This provision adds considerably to the cost of the foundation however.

A safe manner of construction is to build the lock walls first, and provide for an equal distribution of the earth reaction under them by using piles, grillage and concrete, as noted above, making them safe against settlement, overturning, and sliding, when the lock is empty. Then insert the floors, found by separate investigation to be strong enough to withstand the pressures liable to come upon it from below when the lock is empty.

The pile foundation, driven to bed rock or hard gravel, will give the nearest approximation to the desired results, i. e. to have a non-uniform distribution of the weight, so that the greatest part of the pressure shall come under the side-walls. There will be bending in the floor, but not to any dangerous extent. If the piles are in soft bottom there can be no question of entire rigidity, and in this case it would be better to confine the pile foundation to the walls, and omit it under the floor. A structure built in this way, even though the floor be built first, and the walls built on top of it later, would be somewhat safer against fracture in the floor, although it would be difficult to determine this mathematically, because the relative compression of the ground under the walls and under the floor would remain uncertain. It might also be possible to space the piles more closely under the walls than under the floor, and thus get a difference in the compressibility of the foundation.

At the present state of science the calculation becomes much more easy to carry out, if the side-walls are built first and the floor inserted afterward. In this way the exact distribution of pressures can be determined more closely, and the true position of the line of pressure can be got at with a fair degree of approximation.

The following figure shows the manner of building a pile foundation under a wall, capped with grillage and concrete.

FLOORS

The floors were investigated for heads of water, varying from 50 to 30 feet, and the minimum thickness of floor found, that gave a safe crushing strength at the crown of the arch.

Thus we see, for a low lift lock, say of ten feet, we use an eight foot floor, and increase its thickness for an increase of lift, so as to never have a greater press. at the crown, than the allowable limit, when

the lock is empty. For a fifty foot lift the floors are to be 14 foot thick.

Under the gates, where the invert is flattened or cut off, owing to the gate-recesses extending 2.5 ft. below the spring line of the floor arch, the floors are made from two to three feet thicker, to insure the stability of the same. This thickening extends beyond the gate recesses, and is usually made sixty ft., tapering back to the ordinary thickness in twenty feet. The floors are also thickened from two to three feet at the upper end of the lock, giving a larger factor of safety to 60 or 100 ft. of that part of the floor most liable to receive the pressure of the full head of the upper level. The floors above the upper guard gate will not receive any heavy pressures, as no great head can act upon it. They will be made five feet thick and as extra clearance is provided by the head-walls, they will be made flat.

DISCUSSION OF LOCK FLOORS CONSIDERED AS BEAMS.

In this discussion the floor will be considered as a beam, with fixed ends, and with a uniformly distributed load, consisting of the upward pressure of the water under the floor, due to a head equal to the rise to the surface of the upper level, less the weight of the floor. Consider a section of the floor, from side-wall to side-wall, and of unit width.

- Let l equal 80 ft., length between side-walls.
- " width = unity.
- " w = effective load per unit of area.

Then the bending moment at the middle of the beam, at C, $M_c = \frac{1}{24} w l^2$

$M_c = \frac{1}{24} w l^2$ (1).

The moment at A or B will be the maximum however, and in the opposite
(8).

direction. $M_A = M_B = - \frac{1}{2} w l^2 \dots \dots \dots (2).$

Consider the beam made of concrete, with a steel rib bedded in the tension side, capable of taking part of the tension. Assume for ease of computation, that the upper edge of the steel bar, of rectangular cross-section, coincides with the upper edge of the beam.

Let the modulus of elasticity of steel in tension = E_s .

" " " " " " concrete in tension = E_c .

" " cross-section of the steel member = A' .

" its width = D . Let width of beam = B , and its depth = D .

If the upper part of the beam is subjected to tension, the elongation of the concrete and steel must be equal, and the stresses taken by the steel must be to that portion taken by the concrete, as E_s is to E_c , or their stresses are directly proportional to their respective moduli.

Fig. 2.

Cross-section of beam, showing steel member.

The area of concrete that will take the same total tension as the steel is then = $A' \left(\frac{E_s}{E_c} \right) = A$.

We may consider the steel replaced by an area of concrete, capable of taking up its tension, for the same elongation, and at the same distance from the neutral axis. (See Fig. 3.)

Let this strip have the same depth, D' , as the steel, then its additional width, B' , after filling the space occupied by the steel = $\frac{A - A'}{D'} = B'$,

but $A = \frac{A' E_s}{E_c} \therefore B' = \frac{A' E_s}{D' E_c} - \frac{A'}{D'}$.
(9).

to locate the neutral axis of Fig. 3.

Let S = the unit stress at a unit distance from the neutral axis.

" x = any variable distance from the neutral axis.

" S_T = the unit stress in the outer fiber in tension.

" S_c = the unit stress in the outer fiber in compression.

" y = the distance to the outer fiber in compression from the neutral axis. Then:- $S_T = (D-y) S$, or $S = \frac{S_T}{(D-y)}$ } (4).
also, $S = yS$, from Fig. 3.

The total stress on the compression side = $SB \int_0^y x dx$.

" " " " " tension " = $SB \int_0^{(D-y)} x dx + SB \int_{D-(y+D')}^{(D-y)} x dx$.

Equating these stresses, and integrating and solving for y , we have

$$y = \frac{BD^2 + 2B'DD' - BD'^2}{2(BD + BD')} \dots \dots \dots (5).$$

This may be shown to be the ordinate of the center of gravity of the figure. The resisting moment of the beam, or the bending moment that

could produce a unit stress S , at the unit distance from the neutral

axis, is $R. M. = SB \int_0^y x dx + SB \int_0^{(D-y)} x dx + SB \int_{(D-y-D')}^{(D-y)} x dx \dots \dots \dots (6).$

In the above equation, let $A' = 12$ sq. inches; $D' = 6$ inches; $B = 12$ inches; $D = 120$ sq. inches; $E_s = 30,000,000$ #; $E_c = 1,000,000$; $S_T = 200$ sq. inches. Substituting in the above equations, we get

from (3), $B' = 58$ inches; from (5), $y = 71.1$ inches; from (4), $S = 4$ # per sq. inch, (approx.).

And from these the resisting moment, $R. M. = 10,567,100$ # inches (appr.)

If we consider the beam subjected to an upward hydrostatic pressure, due to a head of 80 ft. minus the weight of the beam, uniformly ten ft. thick, (weighing 140 # per cu. ft.), its length $l = 80$ ft., we obtain from (1), (see Fig. 1), Bending Moment at the middle $M_c = 960,000$ # ft. or 11,520,000 # inches.

Maximum moment at the ends, $M_A = 23,040,000$ # inches, in the opposite direction. Since the above resisting moment is less than the bending moment at the middle, the beam must be further strengthened at that point, and is not half strong enough at the ends.

Try $A' = 15$ in., the other dimensions remaining the same. We find,

$B = 72.5$ in., $y = 73.5$ in., $S = 4.27$, and the Resisting moment = 12,053,780 # inches, which exceeds slightly the bending moment. The flexure becomes zero and reverses in direction at a point about 23 inches from the walls or supports. To obtain a sufficient strength at the supports we must assume values and use them in the equations.

Trying $A' = 24$ in., we find, $B' = 116$ in., $y = 78.6$ in., $S = 4.83$, and R. M. = 15,709,064.
~~20,720,000~~ which is too small.

Assuming $D = 15$ ft., and $A' = 12$ sq. in. we find $B' = 58$ in., $y = 108.2$ in., $S = 2.39$, and R. M. = 23,167,430.

For a beam of 16 ft. depth, uniform, with the same head of water as above, we have from (2), $M_A = 17,664,000$.

For a 15 ft. beam, $M_A = 18,560,000$.

" " 14 " " " 19,456,000.

" " 13 " " " 20,352,000.

Such beams can be figured, as shown above, but their thickness becomes too thick for practical use. As those with an arched form answer the purpose in a better manner, and for less cost, they are adopted.

When the rock is found to be of sufficient firmness to warrant it, the floors will only be made of a thickness great enough to hold securely the heads of anchor bolts, which will be driven to a necessary depth to hold the floor in position, when the pressure is the greatest. Two and a half to three feet of concrete will be sufficient for this purpose. The number of such anchor bolts, their size, and distribution over the floor area, are all dependent upon the head of water, which may act to lift the floor.

30 foot head = $62.5 \times 30 = 1875$ #. 3.5 feet of concrete = $3.5 \times 140 = 490$ #.

Area of lock = $740 \times 80 = 59200$ sq. ft. Pressure on bottom = $59200(1875 - 490) =$

81,992,000 #. Taking allowable tension on an inch and a half anchor-bolt

ten tons = 20,000 #. No. of such bolts necessary to hold floor down = 4100.

Each bolt to take 20,000 # and be spaced 3 & 1/2 ft. crosswise, and 4 ft.

center to center lengthwise of the lock, which would require a mass of

stone $3.5 \times 4 \times 8.5 \times 170$ # to keep the floor from being forced up. Hence the

anchor bolts would be sunk to a depth of 8 & 1/2 ft. We neglect the shearing of the rock in place, also that of the floor, which two will add a factor of safety to prevent the floor from being raised due to the action of the water on the bottom.

In general, a rock of a character sound enough to stand such a strain, as would come upon it from the above case, will not be found; and then the thicker floors are used, being in themselves strong enough to withstand any upward pressure that may come upon them. The culverts for filling and emptying the locks, being in the side walls, (see culvert description), the tendency to lift the floors from this cause is not taken into account.

CULVERTS.

The locks will be filled and emptied through culverts in the side-walls, running through them, with the entrances between the upper guard and lock gates, and the outlet between the lower lock and guard gates. The dimensions of the culverts are shown in the following figure.

From the culvert, Ports, nine sq. ft. in area, (4.5 hor. by 2.0 vert.), lead into the lock chamber, the bottom of the port being flush with the springing line of the floor invert. The bottom of the culvert is two feet above the elevation of the spring line of the floor, thus allowing a two foot drop for the Ports. The time required to fill a lock with these culverts has been determined, and is given below.

$$\text{Formula, } t = \frac{\text{Area of lock}}{\text{Total cr. section of main culverts, } \sqrt{2g}} \frac{2 \sqrt{\text{lift}} \times 1}{c}.$$

where g = acceleration of gravity, c = a coefficient (less than 1), found by experiment. Case A = single lock, or end lock of a flight.

Case B = interior lock of a series in a flight.

For case A, $c = 0.75$; and for case B, $c = 0.66$.

Lift.	740 80.	740 80.	600 60.	600 60.	#
	Case A.	Case B	Case A.	Case B.	
10	6 m.00 s.	4 m.51 s.	3 m. 40 s.	2 m. 57 s.	
20	8 " 32 "	6 " 49 "	5 " 08 "	4 " 10 "	
30	10 " 24 "	8 " 20 "	6 " 16 "	5 " 04 "	
40	12 " 00 "	9 " 33 "	7 " 20 "	5 " 51 "	
51	13 " 36 "	11 " 59 "	8 " 20 "	6 " 40 "	

The lock will be filled in the times taken from the above table, and will be effected without subjecting the vessel to very much surging or straining. One object in adopting this plan of filling and emptying the chamber, over that of culverts in the floor, was the greater factor of safety secured for the floor, obtained by not decreasing the area of the floor cross-section for culverts, and by not subjecting them to the pressure of the rushing water. A reason for adopting this plan, over that of valves in the gates themselves, was the fact that such valves weaken the gates just where most strength is required; and weighs them down with cumbrous valve-gear. Besides the gate valves introduces the water for filling so as to strike the vessel heavily, creating a disturbance in the chamber, and a tendency to surge the vessel upon the upper or lower gates. The spacing of the port-openings from the culvert to the lock chamber, is shown below, and are so spaced to cause as little disturbance as possible, in the filling of the lock, thus preventing injury to the vessel or to the lock itself.

At the head and foot of each culvert, there is placed in a shaft, 9 by 4), operated from the coping a sluice of the "Stoney" pattern, to control the filling and emptying of the lock. These are introduced, because of their ease of operation, and the satisfaction which their use has given in the controlling of bodies of water on canals in Europe.

The method of filling and emptying locks through culverts in the side walls, is considered manifestly the best, and entirely better than any system of filling from beneath the floor.

The entrance to the culvert is divided by a granite stone pier, into two parts, of the same cross-section as the culverts proper, sloping

down-stream at an angle of 60 degrees with the face of the wall, providing ample space for the entrance of the water. The entrances and outfalls of the culverts, have a two and one half foot protection of granite, because of their exposed position to the rush of the water. The bends of the culvert, where they run from one lock to another in a flight, and bend around the gate recesses, are lined with stone, and the remainder is lined with brick. The port openings are cast iron pieces, made of a size and shape to fit accurately into the culvert, and the face of the wall, being built into the concrete. The dimensions of the casting is shown in the figure below.

The inside face of the culvert is kept ten feet from the face of the wall and the back of the gate-recesses. This necessitates a banding of the culverts, wherever they must pass around and back of a gate recess, equal to the depth of the recess. The valves for admitting and releasing the water, will be operated by power, the machinery being combined with that for moving the gates.

MITER WALLS: MITER SILLS, &c.

At the upper end of the lock, there is built across it and upon the flat floor, a head-wall, ten feet wide, rising eight feet from the floor for the purpose of preventing debris from washing into the locks. Its top is two feet below the required thirty feet draught.

The upper guard gates miter against a wall, built up from the flat floor of the lock to a height of seven feet, the top then being three feet below the required thirty foot draught. These three feet are allowed for the possible contingency of a fluctuation of the water, above the lock, due to an abnormal cause. This miter-wall is rounded on its face to fit the curve of the undercut gates that are to come against it.

The upper three feet of this wall consists of large granite blocks, anchored by bolts in their positions on the wall. These bolts are 12 ft.

long, two inches in diam. and are wedged at the bottom and secured in position by concrete. The walls were covered with these large stones and the stones securely anchored, for the purpose of preventing the sills or coping from being forced up, as has happened upon other locks, and caused great delay to navigation, annoyance and expense. This plan of miter-sill and miter-wall coping will, it is believed, fully obviate these difficulties. It is not possible under any imaginable circumstances to disarrange sills and coping held down as shown in the drawings of the lock. At the top on the upstream side of the wall, is placed an 18 by 18 in. timber, against which the bottom of the gates miter, and through which part of the gate pressure may be transmitted to the miter-wall. It will be noticed that these timbers are the only pieces of wood in the entire lock, except pieces that perform a similar function in the gates. The timbers are about 16 ft. long each, with six to a miter-sill, and are fastened to the stones in which they set, by four 12 ft. anchor bolts to each timber, the bolts running through the timber, the stone, and ^{into} the concrete. These bolts are supplied with nuts at the upper end, to permit of the timbers being tightened into place, if for any reason they should become loosened. The upper guard gate miter-wall is 32 ft. wide at its narrowest point, with its upstream face fitted to the gates at the top, and having a slope of 1 to three below the stone coping. The down-stream face is an arc of a circle with a radius of 58 ft., and a rise of 16 ft., width 80 feet.

The upper lock gate miter-wall is of the same shape as the upper guard gate wall, and has its top at the same elevation. It is 15 ft. at the narrowest part. The upstream face runs down to the floor of the forebay for the culvert entrances, at a slope of 1 to 3. The down stream face is the same as for all miter-walls, i. e. the arc of a circle, radius 58 ft. and rise 16 ft. The elevation of the floor of the fore-bay is the same as that of the lock chamber, for all lifts up to 26 ft. For lifts greater than this, another offset must be made at the upper lock gate miter-wall, equal to the excess of the lift over 26 ft.,

requiring a headwall of this excess of lift in height. The size of this head-wall is made the same as the floor under the forebay, in thickness, and is curved outward, so as to give the arch action to the wall, when the maximum pressure comes upon it, i.e. when the lock is pumped out. It has the same radius as that of the down-stream face of the miter-wall, of which it is a continuation. The quantity of concrete in these head-walls can be easily figured, when their height and thickness are known.

The lower lock and guard gate miter-walls are built together, as shown in the drawings. A double row of granite stone coping is placed on the upstream face, and a single row on the down-stream side, as a protection to them. Each of the outside row of stones are fastened by four anchor bolts, and those of the second row by two each. In the exceptional case, where the level of the water below the lock, may rise very much above the normal stage, then the down-stream side will have two rows of coping stones, as in the upper side.

The space between these rows of coping stones, is filled up to the level of the top of the miter-wall, (one ft. below the spring line of the floor invert), with concrete. The stones forming the coping, are three feet thick, and will weigh from ten to fifteen tons each. The miter-sills are the same as described for the upper gates. The upstream face of the miter-wall is straight to fit the kind of gate that miters against it. (See Gates.)

It will be observed that the lock is provided with a heavy head-wall, at its upper end, produced by carrying the miter-wall down to the floor of the lock fore-bay, which head-wall has a height equal to the lift, (nearly). These walls have been introduced for the purpose of removing the danger of accidents, namely, vessels carrying away the upper gates, such as have occurred on locks with all the gates of the same height. If a vessel goes too far ahead in these locks, she will strike against the head-wall, and damage herself instead of the gates.

INVESTIGATION OF THE UPPER LOCK GATE MITER-WALL.

The water pressure and the pressure from the gates, is here supposed to be sustained by the arch action of the head-wall. The Fig. below shows the form and dimensions of the head-wall. Fig. 2 is a vertical section through O-D.

Consider the miter-sill to take one half the pressure from the gates. Assume that this pressure is so distributed as to be taken up by the upper six feet of the head-wall, neglecting the batter on the face. The head on gates is taken as 39.5 feet, equal to the draught of the channel plus 9.5 feet of high water. Pressure per hor. ft. on the lock gate

$$\frac{39.5 \times 62.5}{2} \times 39.5 = 48,750 \text{ (closely).}$$

Pressure on upper six ft. of head-wall = $6 \times \frac{39.5 \times 62.5 + 45.5 \times 62.5}{2} = 15,938$.

Total = $\frac{48,750}{2} + 15,938 = 40,313$ per linear foot.

The resultant water pressure on the half arch ABDE, equals the above rate of pressure and is normal to its chord DB, applied to the length of DB.

$$\sin \phi = \frac{40}{73} = 0.548; \therefore \phi = 33^\circ - 14'; \text{ and } \phi/2 = 16^\circ - 37'.$$

$$\text{Length DB} = 2 \times 73 \sin \frac{\phi}{2} = 41.75 \text{ feet. Resultant R. W. P.} = 40313 \times 41.75 = 1,683,070 \text{ \#. Length AB} = \sqrt{73^2 - 40^2} - \sqrt{58^2 - 40^2} = 61 - 42 = 19.0 \text{ feet.}$$

To find value of P that will produce equilibrium, passing through upper edge of middle third of DE, with abutment reaction at lower edge of middle third, at C.

$$AC = \frac{19}{3} = 6 \frac{1}{3}. \text{ For moments about C, the arm of P} = 68 - (42 + 6 \frac{1}{3}) = 19 \frac{2}{3}$$

$$\text{In triangle BJC, } \sin 16^\circ 37' = \frac{BJ}{BG} = 0.286; BJ = \frac{41.75}{2} = 20.9; \therefore BG = \frac{20.9}{.286} = 73.1;$$

$$CG = 73.1 + 12 \frac{2}{3} = 85.8;$$

$$CH = 24.5 \text{ arm of R.W.P. about C.}$$

$$\text{Equating moments about C:- } 19.7 P - 24.5 \text{ R.W.P.} = 0.$$

$$\therefore P = \frac{24.5 \times 1,683,070}{19.7} = 2,093,160.$$

$$\text{Average thrust in section DE} = \frac{2,093,160}{6.15} = 23,257 \text{ \# per sq. foot. Thrust on}$$

outer element at $D = 2 \times 23,257 = 46,514$ # per sq. foot, which is well inside of safe limits.

In the lower part of the head-wall, considering a section one ft. deep it will be about 24 feet at the narrowest part, on account of the slope given to the face of 1 to 3. The head will be about 80 feet, and the water pressure will be about $80 \times 62.5 = 5,000$ # per sq. foot.

With the pressure from the gates, the upper six feet of head-wall carries a pressure of $40,313 \div 6 = 6,719$ # per sq. foot. So that as the bottom of the head-wall is thicker, it is amply safe.

The lower lock miter-wall cannot fail because, of its shape, its thickness, and its position near the floor.

GATES.

The successful operation of the lock depends largely upon the gates, and though representing relatively only a small part of the total cost, they are more complex in their construction than any other part of the lock. The standard form used in these locks are the mitering gates of two leaves each. They turn upon a vertical axis, (the quoins), like an ordinary door. When closed the leaves meet an obtuse angle, (the sally angle, which in these gates is 0.4), the miter-posts abutting against each other at the center, and the bottom resting against the timber miter sill. In this position the gates act as an arch, transmitting the water pressure to the side walls at the quoins.

The mitering gate having so many strong points in likeness, durability and facility of movement, are adopted as a basis for designs and estimates for the gates on all the locks considered. Steel mitering gates with a horizontal framed system, straight on the down stream side, and curved on the upstream side, with depths varying for different lifts of lock, and a rise of sill equal to $1/5$ of the span, are used. This is the so called Horizontal Girder type. It was found that it presents advantages of greater stiffness, requires a shallower recess in the side walls than other forms, and considering the state of the labor and steel markets at the present time, and for the past decade, it is actually

cheaper.

The advantages of iron and steel are durability, strength, and simplicity of construction, and in tidal waters freedom from the attacks of the worm. When designed to permit of inspection and painting, they are much more enduring than those of wood. Some of the early examples show little rusting after thirty years. In many of the locks investigated, the lift is so great, that it would be almost impossible to provide the necessary strength in a wooden gate. For these reasons, and because steel is structurally a much better material than wood, steel gates are designed for the locks under discussion.

The straight form of girder was adopted, as being both practically the best and the most economical for the width considered. The upstream face is straight in the middle part, the ends being circular arcs which intersect the central axis of the lock at 90 degrees, so as to simplify the details of the vertical posts. The horizontal girders consist of a web plate, four chord angles, and one cover plate on the upstream side, which also serves as a splice for the sheathing. The maximum stress allowed on the extreme fiber in the computations, was 9800 # per sq. inch; the web plate and a strip of sheathing adjacent to each girder, was considered as forming a part of the girder section in resisting compression and cross bending. The spaces between the frames are variable, being three feet three inches near the top, and less than two feet at the bottom of the gates. Their thickness for gates on locks of this size, is four and one half feet.

For investigation the gate can be divided into its several parts as follows:- the quoin post, the miter post, the framing between them, the sheathing covering the framing, the pivots upon which the leaves turn, the pin and anchorages which retain all in place, the bearings upon the sill, and the foot bridge on top of the gate.

The quoin post as designed consists of a very thick web plate, with four heavy flange angles, which forms the vertical girder acting transversely to the gate. This with its connections to the gate, distribute

the thrust of the horizontal frames. The hollow-quin is of stone, and the bearing piece of wood, where the magnitude of the thrust would allow. Wood is easily shaped, adjusts itself to any unevenness or error of workmanship, and is more easily repaired or replaced in case of accident.

The quin stones are cut so that the reactions at their backs will introduce a bending moment, which shall balance the thrust of the quin-post. In cases where the pressure per vertical inch of quin does not exceed 9,000 #, it is proposed to use wood for the quin post. The greatest intensity of pressure between the wood and stone will be about 400 # per sq. inch. In the cases where the thrust at the quin passes this limit, the quin post and bearing piece are of steel. The metal quins are intended for use on all gates that withstand a head of more than 30 feet, in these locks.

The functions of the miter post are practically the same as those of the quin post, the difference being that it bears against a similar post instead of against a hollow quin. Steel bearing blocks are used, as the pressures are so great, that too great a width would be necessary if wood were used.

Vertical framing is added to that of the horizontal girders, not for the purpose of carrying any part of the load but to stiffen the whole structure.

Sheathing plates are attached to the upstream face of the gates, which transmit the water pressure to frames. No plates were used less than 3/8 inches in thickness, and none more than a 1/2 inch were required.

The pivots on which the gates turn, consist of a cast steel base imbedded in the concrete floor, which holds the bearings of bronze upon polished steel, and this holds the pivot proper, which is a steel forging hemispherical on top. Bolted to the gate is another casting which holds a bronze hollow hemispherical cup, which fits the pivot.

The anchorages consist of eye-bars extending back into the masonry, to beams imbedded in the concrete. Sufficient masonry is embraced to preclude any danger of movement. Provision is made for adjustment by means of wedge shaped bars.

Contact between the gate and miter-sill is provided for by a straight timber bearing piece, bolted to the flange of the horizontal frame, nearest the bottom, and this timber closes against the timber miter-sill already described.

When the gates are closed, a head of water equal to the difference of level on the two sides of the gate, acts upon the bottom of the gate tending to lift it. This lifting effort upon the lower gate, is but slightly greater than the gate itself, so that the friction of the quoin posts in the hollow quoins, will overcome all tendency to upward movement. If the upper lock and guard gates, and the gates between locks in a flight, should expose so great a bottom area to a maximum water pressure, the uplift would be far in excess of the weight of the gate.

To meet this difficulty, the bottom frame of such gates is very much narrower than the others, and no timber bearing piece is used. A curved sill is required for this arrangement.

For the convenience of the workmen and others, a foot bridge is provided upon the top of the gate. It has a railing that is removable so that one man can handle it.

The guard gates at either end of the lock are for the purpose of allowing of the periodic examination and repairing to be done in the dry, and as a protection in case of accident to the lock gates proper.

OPERATING PLANTS.

The operating plants will derive their power from the development of electrical energy by water power, and will be placed in the position most economical for its purpose. This is to be judged in each case by a close examination of the ground surrounding the lock-site. As a lock is usually located where the conformation of the ground permit, or show it to be expedient, in most cases there will be nearby a good place for locating a power house, where a sufficient fall can be obtained to produce the necessary energy.

The power thus developed will be used in operating the machinery for opening and closing the gates, working the valves, lighting purposes,

operating pumps, and running winches to aid in the passing of boats.

Where necessary, extra canals will be constructed of the proper cross section for conducting the water from the canal proper, to the place selected as the best location for the plant. The cross section of such canal depends upon the amount of power to be generated, the head with which the water will act, and the maximum current allowed in the canal.

Below are two cases showing the manner of investigating different ways of developing power, and the approximate cost of the two schemes. By comparisons such as these, in regard to the cost and power obtained, the best can be chosen and adopted.

Discussion of the method of furnishing power for the double flight of locks on the Niagara Ship Canal, (proposed), where there is a difference of about 320 ft. in the levels of the Lakes, overcome by the locks.

Case 1. A riveted steel pipe buried in the concrete near the top of the left hand wall of the upper flight, to a point near the upper end of the lowest lock, (No. 6), thence into a shaft to the levels of the drainage tunnel of that lock, whence it is carried to the Niagara River and discharged. Approximate length of pipe including that in shaft = 4200 feet. Approximate head = 280 feet, length of shaft = 100 feet, No. of right angle turns in pipe = 12.

After some preliminary study, a two foot riveted steel pipe was decided on, as able to furnish the water necessary to generate the power required. The losses of head were then computed from the formulæ taken from a standard text on Hydraulics. For a velocity of 10 ft. a sec. $h' = .5 \times \frac{10^2}{2 \times 32.2} = 0.775 = \text{loss of head at entrance}$; $h'' = .014 \times \frac{4200}{2} \times \frac{10^2}{64.4} = 45.57 = \text{loss of head due to friction}$; $h''' = 3.48 \times \frac{10^2}{64.4} = 5.4 = \text{loss of head due to bends in pipe}$. Total loss of head = 52 ft., and effective head = 228 feet.

$Q = 31.4 \text{ cu. ft. per sec.}$; and $H.P. = \frac{31.4 \times 228 \times 62.5}{550} = 813 \text{ gross}$.

Similarly for velocities of 8, 15, and 20 ft. per sec., the gross H.P. was 703, 910, and 707 respectively. Thickness of pipe = $P = 280 \times 0.434 = 121.5 \text{ # per sq. inch}$. $T = \frac{121.5 \times 24}{20000} = 0.145 \text{ inches}$. For resisting coro-

sion, this was increased to $3/8$ inches for the first 2100 feet, and $1/2$ inch for the remainder. Circumference = 75.4 inches, and adding 5.6 inches for lap, get 81 inches. Wt. of $3/8$ inch pipe = $\frac{81 \times 3/8 \times 10 \times 2100}{3} = 212,625$ #. Wt. of $1/2$ inch pipe = $\frac{81 \times 1/2 \times 10 \times 2100}{3} = 283,500$ #. Total wt. = 496,125 #.

Cost of this plant:- 496,125 # @ $3 \frac{1}{2}$ ¢ = \$ 17,364.38.

Cost of 100 ft. shaft @ \$ 30.00 = \$ 3000.00, and total cost = \$ 20,364.38.

Case 2.

In this case a channel is cut from the main canal, above the upper lock, approximately following a creek that leads to the escarpment near the river, from which point a riveted steel pipe is proposed over the slope, to the plateau below, then into a shaft to the turbines discharging into the river.

The canal.

Length 2200 feet, width 10 ft., with vertical sides. Depth of bottom below the low water stage of the main canal nine feet, at the junction with the latter, and eleven feet at the junction with the steel pipe. The approximate average depth of cutting of this canal is 19 feet, giving an excavation of 15,500 cu. yards.

Riveted steel pipe and power developed:-

Length 660 ft., diam. $2 \frac{1}{2}$ ft., head 310 ft., approx.

For a velocity of 30 ft. per sec., H.P. = 4,370.

" " " " 40 " " " " = 5,000.

" " " " 66.7" " " " " = 2,570.

$P = 310 \times 0.434 = 134.5$ # per sq. inch. $T = \frac{134.5 \times 30}{20000} = 0.2$ inches. Thickness adopted = $1/2$ inch. Circumference = 94.2 inches, and add 5.8 inches for riveting get 100 inches.

Wt. = $\frac{100 \times 1/2 \times 10 \times 660}{3} = 110,000$ #.

Cost of case 2:- Canal, 15,500 yds. excavation @ \$ 75.00	\$ 11,625.00.
Pipe, 110,000 # @ 0.4 ¢	4,400.00.
Shaft, 130 ft. @ \$ 30.00	3,900.00.
Tunnel, 250 feet @ \$ 15.00	3,750.00.
	<u>23,675.00.</u>

The difference of cost as estimated, in the above two cases, is not great, but the greater H. P. obtained in case two leads to its adoption.

The cost of maintenance of the finished plant, will have to be looked into, when considering several cases, and may be the controlling feature that causes one of the plans to be adopted.

The above cases are but an example of the way of investigating power plants, and in something after the manner shown, each proposition at a lock-site must be worked out, until a satisfactory conclusion is reached for the case in hand. The amount of electrical power required to operate the locks, bridges, and other structures, and to light the canal properly throughout its entire length, can be carefully determined, and plants established, which shall be able to produce this power. The canal should be efficiently lighted, as it is of the greatest importance in securing safe navigation of the canals by night.

The capacity of the power producing plants, should be in excess of the actual requirements. To continually work up to the full power is false economy, and leaves no margin for contingencies. The appliances for producing light, operating the pumps, &c. should be in duplicate, or divided in such a manner that any breakdown may not affect the whole.

Accidents to pumps, engines, dynamos, or boilers will happen at the most unexpected times, and when they certainly should not happen according to paper arrangements. If at such times, duplicate power is not available, stoppage of navigation may result, possibly with very serious consequences.

INTERMEDIATE GATES FOR LOCKS.

If the quantity of water to supply the locks, and evaporation and leakage, is just about equal to the quantity obtainable, or is drawn from a storage reservoir, then for the sake of economy, intermediate gates are put in the locks, to allow the smaller class of vessels to be locked through, without the necessity of using an entire lockfull for each operation. In such cases, intermediate gates will be placed at 500 ft. from the upper lock gate quoin, leaving 240 ft. between its

quoin and that of the lower lock gate. Thus by using the intermediate gate, 240 feet of the 740, (about $1/3$), will be saved. This is a matter of great importance where the water supply is taken from reservoirs, and must be carefully considered.

The floors will have to be made thicker under these gates, in the same manner as under the lower gates, and the same amount; but need only be thickened for a distance of 60 ft. above the quoin to 35 ft. below.

This insures the safety of that part of the floor where the gates recesses are cut out, and for a short distance below the miter wall where extra pressure would be liable to occur. The walls back of the quoins in these gates, will be made thicker, for the same reasons as in the regular lock gates. As the pressure these walls will have to carry, is that due to the gates and enclosed water of the same depth as comes upon the lower gates, hence they will be made of the same thickness as those back of the latter. The culverts will bend around the gate recesses so as to keep ten feet from the face of the wall at every point.

A new arrangement of the port openings becomes necessary, and is shown in the figure below.

Valves on each side will be necessary to open and close the culverts, allowing the shorter lock to be used. They are placed above the quoins where the walls will not be weakened by them, and be on a straight part of the culvert below the last port opening from the culvert to the (shorter) lock chamber.

The miter wall is to be 12 feet at the narrowest part, and of the same form upstream as that of the lower lock gate miter wall. Its top will be one foot below the spring line of the floor invert, and have a three foot coping of granite, bolted to the concrete beneath. The downstream face of this miter wall will have a radius of 58 feet, similar to that of the upper guard and lock gates.

ON EMPTYING THE LOCKS.

At the lower end of the locks, above the lower lock gate miter wall, a drain pipe is provided, built into the concrete, leading to a sump-hole or well, from which by natural drainage or pumps, the water is carried away. These drains are provided for the purpose of emptying the locks, when inspection or repairs are necessary. A three foot pipe is used for a lock 80 foot wide, and a 2.5 foot pipe for the 60 foot lock in the flights, and are all laid with a slope of one foot in fifty feet.

The lock floors are level throughout, as it is more convenient and not more costly, and longitudinal drainage is provided for by having the upper end of the drain pipes on a level with the bottom of the invert.

As the locks will only be emptied at long intervals, and when navigation is practically closed, the time of emptying them, does not figure so prominently in the calculations for the size of the pumping plant, as it would in the case of a dry-dock, where the time element is a matter of first importance. An arrangement of the plant, so as to empty the lock in 24 hours, is amply sufficient, for the general case of locks. Special conditions may alter this proposition and necessitate a pumping plant that could empty the chamber in much less time; but these must be the subject for separate investigation.

ON FLIGHTS OF LOCKS.

In some cases where a large difference of elevation occurs, on a canal route, and the nature of the soil or rock makes it necessary or expedient, two or more locks will be built together forming a flight. In such a flight, the locks are of the same general character as a single lock, and are built double with a thick wall between them. The upper and lower ends of the flight are the same as for the single locks described above. Between the locks of the flight, gates are required also another miter-wall and head wall. The size of these is dependent upon the lift. The walls and the floors are made thicker, in a similar manner to those at the intermediate gate.

A vertical curve will be necessary in the culverts, to carry them from the level of one lock to that of the other. These vertical curves will be lined throughout with stone. The center wall which separates the double locks, will be 52 feet wide, and be built up solid. This is the smallest width allowable to insure safety of the walls, with the culverts running through them, at the points where the gate recesses cut off part of the wall. They are thicker than necessary along the main wall of the lock chamber, but it is desirable to have them straight throughout, so as to keep the lock capacity as small as possible, so as not to have the time of lockages too great.

It is also proposed to make the lock on the left hand side only 60 feet wide, thus again shortening the time of lockages by using the smaller lock as much as possible. The design of the miter-walls and other parts of the lock for the 60 foot side, are similar to those described for the 80 foot side.

Drawings and prints accompany this article, showing details.

In closing this paper on locks, grateful acknowledgement is due, for information gained, to the U.S. Board of Engineers on Deep-Waterways, and their efficient assistants, (among whom the writer has been an humble worker for the past two years,) who have been making drawings and estimating on locks, such as described for the proposed Deep-Waterway from the lakes to the Atlantic.

by,

Henry Howell Lott
Mobile, Ala.

Columbia, Mo, 5/21/1900

This ~~thesis~~ thesis, submitted for the degree of
Civil Engineer by Mr. H. H. Latta, is hereby
approved and accepted as satisfying the
thesis requirement for said degree so far
as I am concerned

H. J. Conroy Prof of Civ Eng

378.7M71
XL 917

Fig. 1.

Fig. 2.

Fig. 3.

Scale. 0' 20' 40' 60' 80' 100'

Gaylord
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