U. S. Thesis

A DESIGN
for
STEEL MITERING LOCK GATE

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CONTENTS

INTRODUCTION.

Scope of the Thesis. - This thesis contains (1) a brief account of the services and the types of lock gates, (2) a general consideration of the basic principles for their design and (3) a design of a gate of ordinary size to illustrate the application. As the writer has not intended to make this thesis a purely descriptive presentation, consequently the first part will be treated with such an amount which seems necessary and proper for the subject. No originality will be claimed on the second part. It is compiled from various articles appeared in text-books, professional papers and journals. Too high mathematical treatment is not given. Elementary integrals and differential equations will be resorted to only when it is unavoidable. The time is so limited that a minute detailed design can not be very well done. The design is made on the main essentials of the structure and machinery, while details of riveting and connections are regrettedly omitted.

References and Acknowledgments. - The following list consists of some references on the subject. Most of them are consulted in preparing the thesis. While it is by no means complete, it may be useful to any body who likes to know more about Lock Gates.

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The writer wishes to acknowledge his indebtedness to the Instructing Staff of the Civil Engineering Department of the Massachusetts Institute of Technology, particularly to Professor Charles M. Spofford for the inspiring instruction in structures, and to Professor Dwight Porter, in hydraulics.

GENERAL ASPECT OF THE SUBJECT

Function of the Gate. - When two bodies of water having different levels, either occurring in nature or artificially made, are connected by a third passage, the ends of the passage must be provided with certain means to stop the violent torrent and to permit the safe transportation of vessels. The only means which can fulfill the requirement is to use gates. The built up passage provided with gates at both ends is called a lock.

When a vessel wishes to pass from higher level to lower, it enters to the lock, the gate at the down stream end being closed. The gate at the upper end is then shut behind the ship and the water in the lock is allowed to flow out until the level reaches that of stream below. The gates ahead of the ship are opened and the ship passes out. If the vessel is passing from lower to upper level, the process is similar except the water is let in-to the lock instead of being let out. Where the difference of level of two waters is too great for a good operation of one lock, two or more successive locks are often employed. An example is given at Fontenettes, France, where a series of five locks were built over a level difference of 13.13 m.

Gates are also used at the entrance to docks. It is necessary to maintain a fairly constant water level for the reception of shipping by vessels in the dock. Where there is a considerable tidal range to effect the level, gates are provided to impound the inside water at ebb. Many docks have only entrance gates; others are connected with locks. The latter can offer a great facility for the movements of ships since they can enter or leave at any time disregarding the tide; while the former only admit ships at rising tides. Sometimes the lock is subdivided into unequal sections by intermediate gates, so that it can accommodate to the reception of large or small vessels, as the case may be, with a minimum expenditure of water and time during the process.

It is evident that the service of the gate is highly valuable and its successful operation is of prime importance. The duty of the gate is to sustain the water pressure in the upper pool and transmit it to side walls and sills. Therefore, it must have sufficient strength and be watertight. Renewal of the gates will cause inconvenience to navigation, so its durability is important.

History. - The history of lock gate eventually goes with that of lock. While there are very few literatures on the evolution of the using of lock gates so far as the writer knows, we might be content by quoting several lines from "Lock and Lock Gates for Ship Canals" by Henry Goldmark, published in Journal of the Association of Engineering Societies, March 24, 1899. "The credit for building the first lock is claimed by both Holland and Italy, but the evidence as to time and place is conflicting. By some writers it is claimed that the first lock was built in 1481, near Padua, in Italy, while the advocates of Dutch priority feel confident that true canal locks were in use in the Netherlands before 1250. In both countries simple sluice or head gates were built long before the enclosed lock with enclosed chambers. Such gates are sometimes used for navigation, and are often confronted? with true locks by the early writers."

Although we do not know the exact date, the true lock gate, in modern sense, however, probably came into existence between the thirteenth and fourteenth centuries. The art of construction, however, was well developed in the nineteenth century. Many gates of size were built at this period. Gates of Manchester Ship Canal, England, that of Havre Dock, France, and that of lock at Sault Ste. Marie, America, are several of the notable cases. In this century, of course, the lock gates at Panama Canal are the most celebrated additions to this sort of structure.

Classification. - Gates may be mainly classified as (1) those having single leaf and (2) those consisting of two leaves. In the former case belong the swinging gate, rolling gate, Tumble gate, and lifting gate; in the latter, all are of mitering type and are only different in forms such as straight gates meeting at an angle, curved gates meeting at an angle and forming a Gothic arch, and curved gates, when closed, forming a continuous arc.

Swinging gates are adopted in many old and narrow locks. They are of the same shape as ordinary doors. They turn about vertical axis on the pivot. Usually they have a balance-beam extending back over the wall and serving as a lever for their movement.

Rolling gates can be seen in Europe and on the Upper Ohio River, in America. They are built of steel, trussed like a bridge. The gate is attached with rollers at its bottom and runs on rails. In opening it is moved laterally to a recess in sidewall with heavy chains. M. Eiffel, a French engineer, devised a peculiar rolling gate in Panama Canal when it was a French enterprise. In this gate the top is fitted with a turning bridge the bottom of which is higher than that of side wall. The bridge has rollers at its under side and moves on the rails laid at the top of the wall. (See Le Génie Civil 1887-88, p. 244; 1888-89, p. 71). The rolling type is said to be well adapted for wide and deep water. But its high cost in providing enough room for recess does not make its popular adoption.

Tumble gate turns about a horizontal axis built at lock floor. When open, it lies upon the platform, below and outside the sill. They are manoeuvered by means of either wire ropes or chains leading from the topmost member to winding apparatus. It is used on the

Erie Canal and at Havre dock near Tancarville, Françe.

Lifting gate was suggested by Mr. Tolkmitt, a German engineer, and advocated by some of his countrymen. Its principle is that of weighted window. By means of counterweights in side walls, the gate can be moved up and down with much ease. A recess is built at the bottom of the lock floor and the gate is dropped into it when open. The writer, however, is not aware of its actual application. (See Centralblatt der Bauverwaltung, 1886, pp. 92-93).

The advantages and disadvantages of single leaf gates are clearly stated in the Proceedings of the International Navigation Congress sitting at Brussels in 1898. On page 638, it says "they are not more expensive than mitred gates; they are subject to less strain, cause less loss of water, and are more easily adjusted, repaired and replaced; and their working is simpler and more regular. Nevertheless, the great expenditure of water, and the increase in the period of locking, resulting from the elongation of the chamber, are inconveniences which, as regards the lower gates, counterbalance and even outweigh the advantages mentioned above."

By far the most common type is the mitering gate. These gates are usually built of symmetrical leaves, - very rarely of unsymmetrical leaves as the entrance gates of a graving dock on the River Tyne, England. Each leaf turns about a vertical exis, or heel post, set in the hollow quoin of the side wall, and when closed shuts against the other leaf by the vertical edge called miter post and against the miter sill at lock floor. At the lower end of the heel post, a socket is made which fits into a hemispherical pivot built in the masonry; while at its top it is held by collars or anchor-bars fastened to the masonry and passing around a pin in the cap. In some old gates, a conical roller is placed at the bottom near the miter post and travels on a roller path to relieve the stress on the anchors due to the overhanging of heavy gate, especially when the variation of water level is great. Because the roller adds the friction in moving a gate, which is much increased by any irregularity or obstacles in the path, its use is gradually abondoned. The gate may be of single or double sheathing. The single sheathing is little cheaper; but it does not transmit stress symmetrically to the end posts, and, in case of ~~iron~~ wooden gates, it can not be provided with compartments to be filled with air and water to adjust its floatation. The total water pressure on the gate is transmitted through girders which may be either horizontal or vertical. Many engineers have the opinion that for wide lock with moderate depth of water, it is more economical to use vertical framing, otherwise horizontal framing is better. The sill is sometimes curved to suit the curvature of the gate; but more commonly it is made straight and a straight sill piece is fastened to the bottom of the curved gate.

With regard to the merits of different forms, circular arch shape is the most economical on the theoretical grounds of transmission of stress. But on account of the unavoidable inaccuracy in fitting and material and also the variation in temperature in addition to the necessary thickness of the gate, the ideal linear arch can never be realized. Besides that, the sharper is the curvature, the deeper will be the recess in the side wall. Consequently, in wide locks, the expense put on the land and masonry will offset the economy in gate work. For this reason some English engineers contend to use Gothic arch form. This has the adcantage of circular arch and lessens its defect, and it will be more watertight in the meeting place of two leaves. However, the fabrication of a curved gate increases its cost; and to conform a true shape is not an easy matter in actual practice. The tendency in America is to adopt straight gates which are only curved near the heel and miter posts. The examples are the gates at Panama Canal, and of a new lock at Sault Ste. Marie, Mich. (See Eng. Rec., Feb. 26, 1910, and March 1, 1913; and Eng. News, March 5, 1914).

<u>Manoeuvring of Gates</u>. - The details of the operation vary in different designs. But, for mitering gates

operated mechanically, the essential methods are either using chain or cable, or using spars. In the first case, two chains are used for each leaf, - one for opening and the other for closing. The chains are attached to the back and front of the gate, near the bottom and closing to the miter post. They pass through sheaves and end at a spiral drum. In the second case, a spar is pinned with its one end to the gate at about one-third of the length of the leaf from the miter post, and with its other end to a large gear wheel. When the wheel is turned by reducing gears, by the principle of link motion, the spar opens or closes the gate. In large locks the prime movers are generally water turbines operated by the water in the canal at the head equal to the lift of the lock. The power thus generated is transformed as hydraulic pressure, air compressure or electricity which actually controls the mechanism to move the gate.

Materials used. - Up to about 1820, all gates were built of wood. Cast iron and wrought iron were gradually used since that time. The employment of steel is comparatively recent. Very commonly the metal and wood are used in mixture such as wooden gates with metal framing or metal gates with wooden posts.

The chief advantages of wood are watertight, elastic, durable and capable of rough usage. Its disadvantages are low strength, susceptible to the attack of marine worms, complicate in its construction and expensive owing to the scarcity of large wood. It is said that the weight of wodeh gate required to withstand the same amount of water pressure would be more than twice as much as that of iron gate and hence it calls for more power to manoeuvre it. But its life is so long that many gates built sixty or seventy years ago are still in good condition without any repairing. Some gates, however, were destroyed owing to the deterioration of a strip which is alternately exposed to air and water. Besides that, the curving of wooden gates can only be accomplished with much difficulty; and the employment of compartments is not feasible.

The advantages and disadvantage of iron and steel are just opposite to that of wood. Iron and steel rust badly under water, especially salt water. Their life is seldom more than forty years. As the iron industry grows and metal fabrication becomes cheaper, steel is nowadays a dominating material used in gate construction.

In short, under ordinary conditions, wooden gates are more economical for smaller locks. Many continental engineers hold an opinion that 13 m. is the critical lock width in choosing the material of the gate. Below this value wood is more favorable and above which metal is better. Of course, the character of water and the feasibility of obtaining a particular material have a great influence in determining the best material to be used. To say one kind is always better than the other is simply prejudicial. Each case should be judged by its particular condition.

EXTERNAL AND INTERNAL STRESSES IN MITERING GATES

The external forces acting on the gate are

(1) Water pressure on the face of the gate

(2) Water pressure at the bottom of the gate

(3) Weight of the gate

(4) Reactions on the gate when shut

(5) Water pressure in manoeuvring the gate

(6) Reactions on the gate when open

(7) Wave action and the collision with passing boat

Water Pressure on the Face of the Gate.- It is a well known fact in hydraulics that the water pressure varies with depth. In locks the water level at upstream side is higher than that of downstream side when the gate is closed. The difference in level is generally called lift. Figure 1 shows diagrammatically a gate between different levels. The maximum water pressure per unit length at upstream side is WH and that at downstream side is Wh_2. The net total pressure is the difference of two total pressures at opposite sides and is shown graphically by the shaded area. It shows that the pressure below h_1 is uniform.

Water Pressure at the Bottom of the Gate.- In Figure 2, F is the uplifting force acting at the bottom of the gate. For gates of single sheathing, the weight of water at one side balance a part of the force and

$$F = WHdl - Whdl = Wdl\ (H-h) \quad \text{--------------}(1)$$

For that of double sheathing

$$F = WHdl \quad \text{------------------------------}(2)$$

In these equations, l and d are the length and depth of the gate.

We can easily see that higher is the lift greater is the upward pressure, the length and depth are also in proportion with it. In actual case, after the gate is built, l and d are fixed values and the force varies with H only. There is no adjustment in gate of single sheathing. The gates of double sheathing have air and water chambers used to equalize this upward pressure. This is one reason why the gates of latter class are favored especially at the place where there is a great range of tide.

Weight of the Gate.- The weight of the gate varies with materials used. The center of gravity of symmetrically built leaf is usually at its center. A long and heavy leaf has a tendency to droop at its toe when it is open. To overcome this condition, in wooden leaves, a diagonal strut extending from the foot of the quoin to the head of miter post is used to upright it. The water and air chambers of double sheathed leaf are generally arranged so as to bring its center of gravity nearer to the heel post. The uplifting pressure will counterbalance a great deal of dead weight.

Reactions on the Gate when shut.- Since the leaves are either straight or curved and are never built with irregular surfaces, the resultant of water pressure must be at the middle of each leave. The water pressure is same everywhere at one level. We may call the pressure per unit length at any particular level p, and the total pressure along the length l, pl. This load is to be equilibrated by the reactions at miter and heel posts. When the leaves are symmetrically placed the force at miter post is normal to the contact surface and hence perpendicular to the axis of the lock. This is shown as R, in Figure III. The reaction on heel post R_2 must follow the line joining the contact surface and the intersection of R_1 and Pl to establish a condition of equilibrium.

R_1 is evidently equal to R_2 according to force triangle. Taking moment about a, we get

$$fR_1 = \frac{1}{2} l^2 p$$

But $$f = l \sin \alpha$$

Therefore $$R_1 = \frac{lp}{2 \sin \alpha} \qquad \text{------- -----} \qquad (3)$$

R_1 can be resolved into two components, one along the length of the leaf and the other normal to it. The former will be $R_1 \sin\alpha = \frac{lp}{2}$, and the latter, $R_1 \cos\alpha = \frac{lp}{2} \cot\alpha$. The normal components of reactions produce bending stress while the longitudinal components, compressive stress.

The bottom of the gate receives sill reaction. This pressure can not be easily determined. In case of horizontal framing, the bending stress of the lowerest girder will be relieved. Owing to the vertical rigidity of the gate, the lower girders will decrease the deflection toward downstream side and thus the load on them will be lessened. The upper girders will consequently increase the deflection and the load is increased beyond that due to depth only. In vertical framing, the sill plays an important part in counterbalance the water pressure. The full discussion of the deflection of the gate modified by sill reaction may be referred to "Notes on Mitering Lock Gates" by H. F. Hodges.

Water Pressure in manoeuvring the Gate.- When a flat surface moves in still water, it is exerted by a pressure

$$p = \frac{cwv^2}{2g} \qquad \text{------------} \qquad (4)$$

Where C is a constant which is about 1.85 for salt water and 1.8 for fresh water, and p is in lbs per sq.ft. If the length of the gate leaf be l, and its angular velocity, when turning about the axis, is e, then the pressure at the surface of unit height and length dx will be

$$pdx = \frac{cw\ e^2x^2}{2g}\ dx$$

and that at the whole length is

$$\int_0^l pdx = \int_0^l \frac{cw\ e^2x^2}{2g}\ dx$$

$$= \frac{cw\ e^2l^3}{6\ g}$$

The total pressure at the leaf whose height is H will be

$$p = \frac{cw\ e^2l^3H}{6\ g} \qquad \text{------------------} \quad (5)$$

The moment about axis o (fig. IV) is

$$M = \int_0^l \frac{cwHe^2x^3}{2g}\ dx$$

$$= \frac{cwHe^2l^4}{8g} \qquad \text{-------------} \quad (6)$$

The distance from axis o to center of pressure is

$$\frac{M}{P} = \frac{cwHe^2l^4}{8g} \div \frac{cwe^2l^3H}{6g} = \frac{3}{4}\, l \text{ ------------(7)}$$

Besides that, the water level is higher at the side towards which the gate is moving than that at the opposite side. Consequently, the pressure at both sides are not balanced. The unbalanced pressure is

$$p = \frac{wlh^2_1}{2} - \frac{wlh^2_2}{2}$$

$$= \frac{wl}{2} (h_1 + h_2)\,(h_1 - h_2) \text{ ---------------(8)}$$

The difference of level is usually assumed. Since the intensity of this pressure is same at one level, its center must be at $\frac{l}{2}$ from axis o, and the moment is

$$M = \frac{wl^2}{4} (h_1 + h_2)\,(h_1 - h_2) \text{ ----------- (9)}$$

Both of the above pressures are taken into consideration in designing the operating machinery.

Reaction on the Gate when open.- In this case, the top pintle of the heel post and the pivot are in action. Referring to Fig. V., let W be the resultant of the weight of a double sheathed leaf and water ballast, F, the uplifting force, V and C, the horizontal and vertical reactions of the pivot, T, the tension of anchor bar. Then

$$T = C = \frac{aw - bF}{H} \qquad \text{------------------(10)}$$

$$\text{and} \qquad V = W - F \qquad \text{------------------(11)}$$

When the leaf is of single sheathing and made symmetrical with respect to its axis, then

$$a = b = \frac{l}{2}$$

$$\text{and} \qquad T = c = \frac{l}{2H}(W-F) \qquad \text{---------------(10a)}$$

$$\text{or} \qquad T = c = \frac{l}{2H} V \qquad \text{---------------(11a)}$$

Wave Action and Collision with passing Boat.- The magnitude of wave action can not be ascertained. It depends largely upon the locality, the weather and the extent of the water surface exposed to the wind. For this indeterminate stress, both in direction and amount, a reduced working stress of the material is used.

The chance that a gate may be stricken by a boat entering the lock is exceedingly rare. To design a gate which will stand such an abnormal stress is not necessary. Besides, the impact varies according to the momentum of moving ship and is not easily to be assumed. The abrading contact with the sides of passing boats, however, can not be wholly avoided, even though the gate is well recessed. Fender timbers are therefore always provided at the down stream side of leaves.

Besides the above mentioned external forces, there are others of unknown nature as striking at the sill in closing, slamming on the side wall in opening, encountering with any moving body, shocks caused by the irregular action of operating machinery, etc. All these can not be foreseen or determined. Nevertheless these are potent factors governing the life of the gate.

Let us now proceed to find the internal stresses of the different parts of the gate. Those parts whose stresses should be carefully analyzed are:

a. Horizontal girders

b. Vertical girders

c. Sheathing

d. Intercostals

e. Miter and heel posts.

Horizontal Girders.- The function of horizontal girder is to transmit the resulting water pressure to the end bearing of each leaf. The girders may be straight or curved according to the shape of the leaf. In curved girders the internal stress is entirely compressive. The stresses of straight girders are compressive at upstream side and tensile at downstream side.

The saving of material in curved girders is outweighed by the increase of recess in side walls and additional expense of shop work. Besides that, such a girder can not resist the blow of a ship which might happen in practice. All these points are clearly stated in the Report of Board of Engineering on Deep Waterways between Great Lakes and the Atlantic Tidewaters, Washington, D. C., 1900. The recent practice is to adopt straight girders. Therefore the following analysis will be confined to this form. Its principle, however, can be just as well applied to any other type.

Let ef (Fig. VI) be the axis of the girder of I shape; m n, the line joining the centers of reactions of heel and mitre posts; d, the thickness of the girder; c, the distance between ef and m n; l, the length of the leaf. The reactions are shown on the figure. At any distance x from n, the bending moment is

$$M = \frac{xpl}{2} - \frac{px^2}{2} - \frac{cpl\cot\alpha}{2} \qquad \text{------------(12)}$$

This must be equal to resisting moment of the material. Denote compression by K and tension, T. We have

$$\frac{d}{2}K+\frac{d}{2}T=\frac{xpl}{2}-\frac{px^2}{2}-\frac{cpl\cot\alpha}{2}\qquad\text{------(A)}$$

The web of the I-shape girder is sometimes considered not to take any bending stress. Therefore we may refer K and T as upstream and downstream flange stresses. Again, we have an equation

$$K-T=\frac{pl}{2}\cot\alpha\qquad\text{------------------------(B)}$$

By solving (A) and (B), we find

$$K=\frac{p}{2d}\left\{lx-x^2+l\cot\alpha\left(\frac{d}{2}-c\right)\right\}\qquad\text{-----------------(13)}$$

$$T=\frac{p}{2d}\left\{lx-x^2-l\cot\alpha\left(\frac{d}{2}+c\right)\right\}\qquad\text{----------------(14)}$$

The reason why m n is not made to coincide with e f will now be seen, for if c is greater, both K and T become smaller. When

$$x^2+l\cot\alpha\left(\frac{d}{2}+c\right)>lx$$

T will be negative, i.e. the tensile stress will be changed to compression. This will usually occur near the end of the girder, for then the value of x is small.

The value p is different for different girders. It varies more or less according to the spacing of the latter.

Strictly speaking, p is the reaction per unit length of each girder, when it acts as a horizontal support of a vertical continuous beam under triangular and uniform loading. This reaction may be obtained by taking a vertical strip of the leaf of unit length and full height and considering it as a continuous girder. By means of Clapeyron's Theorem of Three Moments we can find the exact value.

Fig. VII shows the section of the strip, water level at one side being h, and that at the other, h_2, Take the part below the lower level. The water pressure is uniform and equals W $(h_1 - h_2)$ per horizontal unit. The moments M_n, M_{n+1} and M_{n+2} at supports S_n, S_{n+1} and S_{n+2} are given in the equation,

$$M_n L_n + 2\, M_{n+1}\, (L_n + L_{n+1}) + M_{n+2}\, L_{n+1} =$$

$$-\frac{1}{4}\, W\ (h_1 - h_2)\ \left[L_n^3 + L_{n+1}^3\right) \text{ ----- } (15)$$

where L_n and L_{n+1} are the panel spaces between the supports. The derivation of the equation for uniform load may be found in C. M. Spofford's "The Theory of Structures."

Next take the part above the lower pool. The pressure here varies uniformly with the depth. Let the pressure at S_n (See Fig. VIII) be Wh and that at S_{n+1}, $w(h+L_n)$.

Then the equation is

$$M_nL_n+2M_{n+1}(L_n+L_{n+1})+M_{n+2}L_{n+1}=$$

$$L^2_n\int_0^{L_n} w(h+x)\left[\left(\frac{x}{L_n}\right)^3-\frac{x}{L_n}\right]dx$$

$$+L^2_{n+1}\int_0^{L_{n+1}} wdx\,(h+L_n+x)\left[3\left(\frac{x}{L_{n+1}}\right)^2-\left(\frac{x}{L_{n+1}}\right)^3-2\frac{x}{L_{n+1}}\right]$$

or $\quad M_nL_n+2M_{n+1}(L_n+L_{n+1})+M_{n+2}L_{n+1}=$

$$-\frac{w}{60}\left\{L^3_n(15h\ 8L_n)+L^3_{n+1}\left[15(h+L_n)+7L_{n+1}\right]\right\}\text{-------}(16)$$

When S_{n+1} is at the water surface of the lower pool, h will be $h_1-h_2-L_n$ and the pressure on L_{n+1} will be uniform. The equation is

$$M_nL_n+2M_{n+1}(L_n+L_{n+1})+M_{n+2}L_{n+1}=$$

$$-\frac{w}{60}\left\{L^3_n\left[15(h_1-h_2-L_n)+8L_n\right]\right\}-\frac{1}{4}w(h_1-h_2)L^3_{n+1}$$

Simplifying, we get

$$M_nL_n+2M_{n+1}(L_n+L_{n+1})+M_{n+2}L_{n+1}=$$

$$-\frac{w}{4}(h_1-h_2)(L^3_n+L^3_{n+1})+\frac{7}{60}wL^4_n\text{-------}(17)$$

If the panels are all equal, i.e. $L_n=L_{n+1}=L_{n+2}=L$, (1) becomes

$$M_n+4M_{n+1}+M_{n+2}=-\frac{1}{2}w(h_1-h_2)L^2\text{-------}(18)$$

equation (2) by substituting nL for h, becomes

$$M_n + 4M_{n+1} + M_{n+2} = -\frac{w}{2} L^3 (n+1) \text{-----------------} (19)$$

and equation (3) will be

$$M_n + 4M_{n+1} + M_{n+2} = -\frac{w}{2} L^2 \left(h_1 - h_2 - \frac{7}{30} L\right) \text{---} (20)$$

After the bending moment at different supports are found, to find the reactions is only a matter of applying ordinary moment equations at proper points.

It is to be noted here that in all these calculations, the upstream flange of the girders are assumed to lie on a vertical line. As a matter of fact this is not true for three reasons. First, the girders will deflect to a certain extent under hydrostatic pressure. The amount of deflection depends on the load and the cross-section of the girder. Second, even the girders are so spaced and made with such a proper section that they may deflect equally, the sill pressure will modify the condition. Third, in actual construction, the framework can never be made to conform to the theoretical requirements. Of course, if we could know the exact deviation of each girder from the vertical line after taking into account all the factors, we may find the exact uniform pressure each girder sustains by adding constants to the Three Moment Equations. But this will involve a very long and tedious work.

Therefore, the usual method adopted by English and American engineers is to assume each girder carrying a uniform hydrostatic pressure between the middle points of adjacent panels. This will simplify materially the calculations and, in fact, are not far from the true value. Because the spacing of girders is generally not very wide, about 2 1/2 to 4 feet apart; and as we know that more supports are used in continuous beam more nearly are their reactions equal to the imposing load between centers of adjacent spans.

Vertical Girders.- Vertical Girders are used in some old gates as the main frames to transmit the load. The top part lies against a stout horizontal girder while its bottom, against the sill. They are really a beam loaded with triangular loading and their stresses can be calculated by taking bending moments at different sections.

Many gates of hozizontal framing also employ vertical girders. Their duty is to stiffen the whole leaf, distribute the sill pressure and the reaction of operating strut or tension of cable. As stated somewhere above, the rigidity of the gate will render the sill pressure to be transmitted from lower to upper girders and consequently lessens the load on the latter and increases the load on the former. When vertical girders are used the rigidity is increased in vertical direction. The load will then be better distributed to all girders.

To find the distribution of load affected by verticals is not an easy matter. Mr. H. F. Hodges in his "Notes on Mitering Lock Gates" gives a simple formula deduced under the hypothesis that after deflection the resistance to further bending of any horizontal section is the same. His analysis, while meriting careful study, can not be briefly abstracted. It will be sufficient to reproduce his formula for the distributing load on any horizontal girder due to verticals:

$$p = \frac{w}{3}\left(H - \frac{h^3}{H^2}\right) \qquad (21)$$

where p is the pressure per sq. ft., H, the depth of water at higher pool and h, that at lower pool. It is seen that p is constant so long as H and h remains same.

Since the load brought to the horizontals by the vertical is originated from the sill pressure, the vertical will not influence the horizontal if the sill gives way. When the sill happens to fail, each horizontal will carry hydrostatic pressure only. Therefore in designing horizontal girders, we have to use the greater loading of two conditions.

The forces on different points of the vertical where it meets horizontal girders are the differences of the distributing loads and the loads due to hydrostatic pressure on the horizontals. Having found these forces the vertical can be designed as a beam with concentrated loadings.

Sheathing. - The sheathing has double functions. It transmits water pressure to horizontals and intercostals, and subjects to bending stress at the part where it acts as a part of girder flange. Since the great part of it is under water it is apt to rust very quickly. Therefore, too thin a plate is not safe to use, although it may meet the theoretical requirements. Thick plates can not be well calked and riveted. The thickness of plates used for this purpose generally varies from 5/16 inch to 1 inch.

If there are no intercostals between the horizontals, the plate merely acts as a beam end supported and its size can be easily found. But it is usually riveted at four edges, - two to intercostals and two to horizontals. The plate so riveted is stronger. To determine its maximum stress, we have to use flat plate formula. Some engineers use Bach's formula.

$$f = \frac{kb^2p}{2t^2\left\{1+(\frac{b}{a})^2\right\}} \qquad \text{-----------------(22)}$$

where f is the maximum stress, t the thickness of the plate, p the pressure per unit, a and b the length and breadth of the plate, and k is a constant varies from 0.75 to 1.13. Others use Grashof's formula

$$f_a = \frac{b^4a^2p}{2(a^4+b^4)t^2} \text{ and } f_b = \frac{a^4b^2p}{2(a^4+b^4)\ t^2} \quad \text{-----(23)}$$

where f_a and f_b are stresses along a and b. French engineers use DeLaharpe's formula

$$f\max = .92\ \frac{b^2p}{t^2\left\{1+(\frac{b}{a})^2\right\}^2} \quad \text{-------------- (24)}$$

Mr. Theodore Landsberg in his paper "Die eisernen Stemmthore der Schiffsschleusen" develops, by an elaborate demonstration, a formula, taking account of the water pressure due to depth,

$$t = \frac{ab}{2}\left\{\frac{w}{f}\ (\frac{2h+b}{a^2+b^2})\right\}^{\frac{1}{2}} \quad \text{----------------(25)}$$

where f is the allowable stress and h the depth of water at the top edge of the plate.

It is difficult to say which formula is the best and it seems that for practical design either one is as good as the other. But the first three are given for the plate uniformly loaded. Since all the formulas are equally complicate, there is no reason why Mr. Landsberg's formula is not more proper to be used to find the thickness of sheathing which is under varying load.

Intercostals. - Intercostals are light members perpendicular to main girders. Their function is to stiffen the sheathing plate and at the same time reduce its stress. They are made of angles or Z-bars either riveted to the flange or to the web of main girders. The load on them may be computed by M. Navier principle that a rectangular plate supported at four edges and loaded uniformly, carries its load to the supports in the ratio of the square of their length. Mr. Landsberg found that under varying load the maximum bending moment of the intercostal is

$$M = \frac{b}{3}(2d_o + d_1)\,z - d_o z^2 - \frac{(d_1 - d_o)\,z^3}{3b} \quad \text{----(26)}$$

while

$$z = \frac{b}{d_1 - d_o}\left[-d_o \pm \sqrt{d_o^2 + \frac{(2d_o + d_1)(d_1 - d_o)}{3}}\right]$$

where d_o and d_1 are the counter-pressuresper unit at the top and bottom edges of the plate, respectively

$$d_o = \frac{wab(3ah + bh + ab)}{2(a+b)(b+3a)}$$

$$d_1 = \frac{wab\,(3ah + bh + 2ab + b^2)}{2(a+b)\,(b+3a)}$$

h is the depth of water to the top edge; b and a, the height and width of the plate as shown in Fig. IX. This formula, though probably more exact, is too long for practical use. In actual construction, the spacing between intercostals is not larger than that between horizontals. Hence, the load they carry is not great. The practical size of the member used is often far more than necessary. If the average pressure is taken as the uniform load, M. Navier's principle can be well applied. To assume the maximum bending moment at the center of intercostal does not involve much error. For even in triangular loading the maximum deflection comes at .52 span from the heavier load. (See Beam Deflection under Triangular Loading, Eng. News, May 7, 1914).

Miter and Heel Posts.- These posts bear the same function. The only difference is that the miter transmits the pressure to the miter of the other leaf while the heel post, to the side wall through reaction bearing. Wood was used for these posts in old gates and is still largely employed in small gates nowadays. The advantages are watertight and elastic. But in large leaves the end thrust may be so great that it would require a large contact surface if wood were selected. Larger are the contact surfaces harder is to make them fit perfect. For this reason, steel castings with detachable contact pieces and yielding metal chusion are now generally used. The post is really a beam having concentrated loads at one side and reactions throughout the entire height at the other. As the concentrated loads from ends of horizontal are not equal, the reaction will not be uniform. We may make an assumption that along the distance between the middle points of two adjoining panels of each girder the reaction is uniform and its intensity is the thrust divided by the said distance. This is just the same sort of assumption adopted to estimate the uniform load on the horizontals.

Beside these essential parts of the gate, anchorages, pintles, pivots and miscellaneous details need be properly designed. All these vary in different cases and can not be said in general way. Their actual design, however, will be worked out in subsequent illustration.

INFORMATIONS

Data:-

Clear width of the lock....................100 ft.

Depth over the sill in the lower pool...... 18 ft.

Maximum lift............................... 20 ft.

Allowable Stresses:-

Tension of mild steel...........14,000 lbs.per sq.in.

Compression of mild steel......12,000 lbs.per sq.in.

Shear of mild steel............10,000 lbs.per sq.in.

Bending stress of cast steel...10,000 lbs.per sq.in.

Compression of concrete........ 450 lbs.per sq.in.

Bearing stress of greenheart
along the grain............. 1,300 lbs.per.sq.in.

Brief Specifications:-

(a) The framing is to be horizontal with vertical stiffeners

(b) The minimum thickness of sheathing plate shall not be less than 5/16 inch. The maximum thickness shall not be more than one inch.

(c) The maximum panel space between horizontal girders shall not be greater than 5 feet. The minimum space shall not be less than 2 1/2 feet.

(d) The spacing of rivets in the plates as regards securing water-tightness of the joints need not be closer than 5 inches nor greater than 8 inches.

(e) All the rivet spacing at connections made with rolled steel shapes shall conform to standard gage and usual shop practice.

(f) Provisions shall be made to facilitate the inspection, painting and repairing for all parts of the gate.

(g) The joints of sheathing plate shall be butt joints.

DESIGN

Preliminary Determination:- Where there exists a chance that the water in lower pool may be completely drawn out, it is always to consider that the gate is subjected to water pressure at its one side only. We will assume that there is a danger of losing all the water at downstream side. Each leaf will then be under triangular loading and the pressure at every point is proportional to the depth of water.

The height of the leaf from the sill may be made 39 feet, thus leaving one foot above the maximum level as a safe margin. Use the sill rise 1/4 of span, i.e. 1/4 x 100 = 25 ft. If the distance from the side of the lock wall to the contact surface at quoin be taken 3 ft., the axial length of the leaf is

$$(\overline{25}^2+\overline{53}^2)^{\frac{1}{2}} = 58.60 \text{ ft.}$$

For the ease in shopwork and convenience in operation, the downstream side of the leaf will be straight, while the upstream side is parallel to it except a short distance at each end where it is curved.

The upper portion of the leaf will be used as water chamber and the lower portion as air chamber. The sizes of two chambers are to be nearly equal. Several openings will be provided near the bottom of water chamber at upstream side so that the water can freely go in and out. The floatation of the gate is thus self-regulated.

Horizontals, Verticals, Sheathing and Intercostals.- Before the designing of girders the spacing between them should be assumed. Let us divide the total height of the gate into eleven spaces, the first three being at 4'-4" apart, next four at 3'-6" and last four at 3'. The load on the girders will thus be proportional to the water depth and the spacing.

When the verticals are employed the girders will carry the loads different from pure hydrostatic pressure. From the data and the condition for design, H and h in equation (21) are 39 and zero respectively, p then becomes $\frac{62.5}{3} \times 39 = 812.5$ lbs. per sq. ft.

The loads on the girders are given in Table A.

Table A.

At Girder No.	Spacing	Load per ft.length due to depth of water	Load per ft.length due to Verticals	Load on Verticals	Max. Load on Horizontals
1	4'-4"	2.17 x 62.5 x 1.08 = 147	812.5 x 2.17 = 1762	+1615	1762
2	4'-4"	4.33 x 62.5 x 4.33 = 1175	812.5 x 4.33 = 3520	+2345	3520
3	4'-4"	4.33 x 62.5 x 8.65 = 2345	812.5 x 4.33 = 3520	+1175	3520
4	3'-6"	3.92 x 62.5 x12.65 = 3100	812.5 x 3.92 = 3190	+ 90	3190
5	3'-6"	3.5 x 62.5 x16.5 = 3610	812.5 x 3.5 = 2845	- 765	3610
6	3'-6"	3.5 x 62.5 x20.0 = 4375	812.5 x 3.5 = 2845	-1530	4375
7	3'-6"	3.5 x 62.5 x23.5 = 5140	812.5 x 3.5 = 2845	-2295	5140
8	3'-0"	3.25 x 62.5 x26.8 = 5440	812.5 x 3.25 = 2640	-2800	5440
9	3'-0"	3. x 62.5 x30.0 = 5625	812.5 x 3. = 2440	-3185	5625
10	3'-0"	3. x 62.5 x33.0 = 6180	812.5 x 3. = 2440	-3740	6180
11	3'-0"	3. x 62.5 x36. = 6750	812.5 x 3. = 2440	-4310	6750
12		1.5 x 62.5 x38.3 = 3590	812.5 x 1.5 = 1220	-2370	3590

The economical depth of girder may be found by the equation given by Hodges

$$D = \sqrt{\frac{pl^2}{8ct}}$$

where p is hydrostatic pressure per unit area, l the length of the gate, c the allowable compression and t the thickness of sheathing for any girder. We may use the average pressure 5000 lbs. and a thickness of plate 1/2 inch. Then

$$D = \sqrt{\frac{5000 \times 58.6 \times 58.6}{8 \times 10000 \times 144 \times \frac{1}{2} \times \frac{1}{12}}}$$

$$= 5.9 \text{ say } 6 \text{ feet.}$$

The economical flange area of the girder will be obtained by having the line of reaction nearer to the downstream as much as possible. This can be made by arranging the contact surfaces at ends of girder to a proper place. But owing to the temperature change and irregularity either due to workmanship or to the unseen obstacle in operation, the line of reaction may shift to one side or the other. Suppose the position of this line of reaction changes within the limit between the axis of the lower flange and 2 feet from it toward the axis of the leaf. This will be conservative if the contact ends are of segmental surface. From equations (13) and (14), for two values of c, 3 and 1, we get the tension and compression at the mid-length of leaf for each case.

Case I $$K = \frac{p}{2 \times 6}\left\{\frac{\overline{58.6}^2}{2} - \frac{\overline{58.6}^2}{4}\right\} = 71.5\ p$$

$$T = \frac{p}{2 \times 6}\left\{\frac{\overline{58.6}^2}{2} - \frac{\overline{58.6}^2}{4} - 58.6 \times \frac{53}{25} \times 6\right\} = 9.42\ p$$

Case II $$K = \frac{p}{2 \times 6}\left\{\frac{58.6^2}{2} - \frac{58.6^2}{4} + 58.6 \times \frac{53}{25} \times 1\right\} = 81.8\ p$$

$$T = \frac{p}{2 \times 6}\left\{\frac{58.6^2}{2} - \frac{58.6^2}{4} - 58.6 \times \frac{53}{25} \times 4\right\} = 30.3\ p$$

In both cases the lower flange near the end of the girder is subjected to compression as shown by previous discussion. When the change of stress begins depends upon the value c. Evidently the change of stress comes sooner in Case I and consequently the maximum compressive stress is found there too. The ends of the girder are fixed to the miter and heel posts whose width may each be one foot long. The distance from the contact surface to the girder end is then 1'. Make the depth of the end of the girder 2 feet. The compression of the lower flange at end section will be

$$T = \frac{p}{2 \times 2}\left\{58.6 - 1 - 58.6 \times \frac{53}{25} \times 2\right\} = -\ 47.9\ p$$

The lower flange area is thus governed by compression.

The web of the girder is generally considered to take up all the shear. The maximum shearing stress is at end. The thickness of the web should be

$$\frac{58.6\ p}{10000 \times 2 \times 12 \times 2} = 0.1223\ \frac{p}{1000}$$

Table B gives the flange and web stresses and their theoretical areas required for the pressure on the gate

Table B

Girder No.	Top Flange Stress(81.8p)	Top Flange Area($\frac{k}{12000}$)	Lower Flange Stress(47.9p)	Lower Flange Area($\frac{5}{12000}$)	Thickness of Web $(0.1223\frac{p}{1,000})$
1	144,400 lbs.	12.01 sq.in.	84,400 lbs.	7.03 sq.in.	.216 sq.in.
2	288,000 "	24.00 " "	168,600 "	14.05 " "	.430 " "
3	288,000 "	24.00 " "	168,600 "	14.05 " "	.430 " "
4	261,000 "	21.75 " "	152,800 "	12.72 " "	.390 " "
5	295,500 "	24.60 " "	173,000. "	14.42 " "	.441 " "
6	358,000 "	29.80 " "	209,500 "	17.45 " "	.535 " "
7	420,000 "	35.00 " "	246,000 "	20.50 " "	.628 " "
8	445,000 "	37.10 " "	260,500 "	21.70 " "	.665 " "
9	460,000 "	38.35 " "	270,000 "	22.50 " "	.688 " "
10	506,000 "	42.20 " "	296,000 "	24.65 " "	.755 " "
11	552,500 "	46.00 " "	323,500 "	26.95 " "	.825 " "
12	294,000 "	24.50 " "	172,000 "	14.34 " "	.439 " "

The upper portion of the leaf above girder (6) will be filled with water ballast and the lower portion will be the air chamber. The horizontals in water chamber, though not subjecting to additional load, are liable to corrode as sheathings. Additional thickness should be provided for webs.

Girder (6) supports the weight of the water above it. Its web is under the combined stresses of tension and shear and also exposed to corrosion. The thickness needed to resist the vertical load may be found by equation (22) if the underside of the girder is strengthened by transvers beams. Let the beams be used; and they are spaced 3'-6" apart. The maximum depth of water above the girder is 20 feet. Therefore the pressure per sq.in. on the web is 20 x .433 = 8.66. Take the value of k in equation (22) as 1, then the additional thickness of the web required at the middle portion of the girder is

$$t = \frac{1}{2} \times 3.5 \times 6 \times 12 \sqrt{\frac{8.66}{(6^2 \;\; 3.5^2)14000}} = 0.452''$$

This value added to the thickness required for shear and for corrosion will be thickness used.

From the last column of Table B, the thickness of the web of girder (6) is .535 in. Adding this amount, the total thickness will be over one inch. It is to be noted that the value found in Table B is for the shearing stress at the end of the girder where the depth is only 2 ft. due to the curving at that place. To use the thickness thus determined for the whole web of this girder is wasting the material. For better economy, the thickness for the shearing stress can be obtained by considering the section through the tangent point of the curve; the remaining curved portion may be reinforced with plates to the required thickness. The distance from the end contact surface to the tangent point as shown in drawing is 12 ft. The shearing force at the section will be $\frac{58.6}{2}$ p - 12p = 17.3 p. Since p for this girder is 4375 lbs. and the depth of girder is 6 ft., the necessary thickness is found to be

$$t = \frac{17.3 \times 4375}{6 \times 12 \times 10000} = .105$$

The theoretical thickness of the web of the straight portion of the girder (6) then becomes 0.557 in., while that of curved portion, 0.987 in.

Girder (12) receives an uplifting force at its underside. If transverse beams are used and spaced at its upper side same as girder (6) the sizes of web can be similarly estimated. The maximum depth of water above it is 39 ft. The pressure per sq. in. on the web is 39x.433=16.9 lbs. The thickness required to resist this stress is

$$t = \frac{1}{2} \times 3.5 \times 36 \times 12\sqrt{\frac{16.9}{(6^2\ 3.5^2)\ 14000}} = 0.63 \text{ in.}$$

By the same reason as above the total web thickness of the straight portion may be reduced to $0.63+\frac{17.3 \times 2450}{6 \times 12 \times 10000} = 0.689$ in. and that at the end, reinforced to 1.069 in.

The flanges of these girders are under no additional stresses theoretically. But since the web plate will deflect no matter how well stiffened they are subjected to secondary stress of the pulling of web plate as shown in Fig. X. This pulling effect will put the flange into a shape of a beam uniformly loaded. How much is the secondary stress is not possible to determine. It is, however, not very large. It might be well to increase the flange areas 15 per cent to take care of this unknown force.

In figuring the transverse beams under girder (6) and above girder (12) we will assume they carry the weight of the water from the web. They are spaced at 3'-6 1/2" apart. Than, for girder (6), the uniform load per ft. is

$$20 \times 62.5 \times 3.54 = 4430 \text{ lbs.}$$

the maximum bending moment is

$$\frac{4430 \times 6 \times 6 \times 12}{8} = 239500 \text{ in. lbs.}$$

and section modulus should be

$$\frac{239500}{14000} = 17.1 \text{ in.}^3$$

Use 9" x 21# I-beam

For girder (12), the uniform load per ft. is

$$39 \times 62.5 \times 3.54 = 8630 \text{ lbs.}$$

the maximum bending moment is

$$\frac{8630 \times 6 \times 6 \times 12}{8} = 466000 \text{ in. lbs.}$$

and the section modulus should be

$$\frac{466000}{14000} = 33.3 \text{ in.}^3$$

Use 12" x 31# I-beam.

Table C gives the practical sizes of webs and flanges. Those parts of sheathing plate between the flange angles and cover plates are considered as flange plates, the width being same as that of cover plate.

Table C

Girder No.	Top Flange	Area sq.in.	Lower Flange	Area sq.in.	Web
1	2-4x4x3/8 L^s 2-9x3/8 plt.	12.47	2-4x4x3/8 L^s 1-9x3/8 plt.	9.09	7/16" plate
2	2-6x6x1/2 L^s 1-13x9/16plt 1-13x5/8 plt	24.45	2-4x4x1/2 L^s 1-9x1/2 plt 1-9x5/8 plt	17.63	1/2" plate
3	do	do	do	do	do
4	do	do	do	do	do
5	do	do	do	do	do
6	2-8x8x1/2 L^s 1-17x11/16plt 1-17x5/8 plt	37.75	2-6x6x3/8 L^s 1-13x11/16plt 1-13x1/2 plt	24.15	5/8" plate 4-1/2"x 11' plts
7	do	do	do	do	5/8" plate
8	do	do	do	do	11/16" plate
9	2-8x8x1/2 L^s 1-17x3/4 plt 1-17x5/8 plt	38.88	2-6x6x3/8 L^s 1-13x3/4 plt 1-13x1/2 plt	24.96	do
10	2-8x8x5/8 L^s 2-17x3/4 plt	42.52	do	do	3/4" plate
11	2-8x8x3/4 L^s 1-17x13/16plt 1-17x3/4 plt	48.33	2-6x6x3/8 L^s 1-13x13/16plt 1-13x5/8 plt	27.40	13/16" plate
12	2-6x6x1/2 L^s 1-13x13/16plt 1-13x5/8 plt	30.20	2-4x4x1/2 L^s 1-9x13/16plt 1-9x1/2 plt	19.31	11/16" plate 4-1/2"x 11' plts

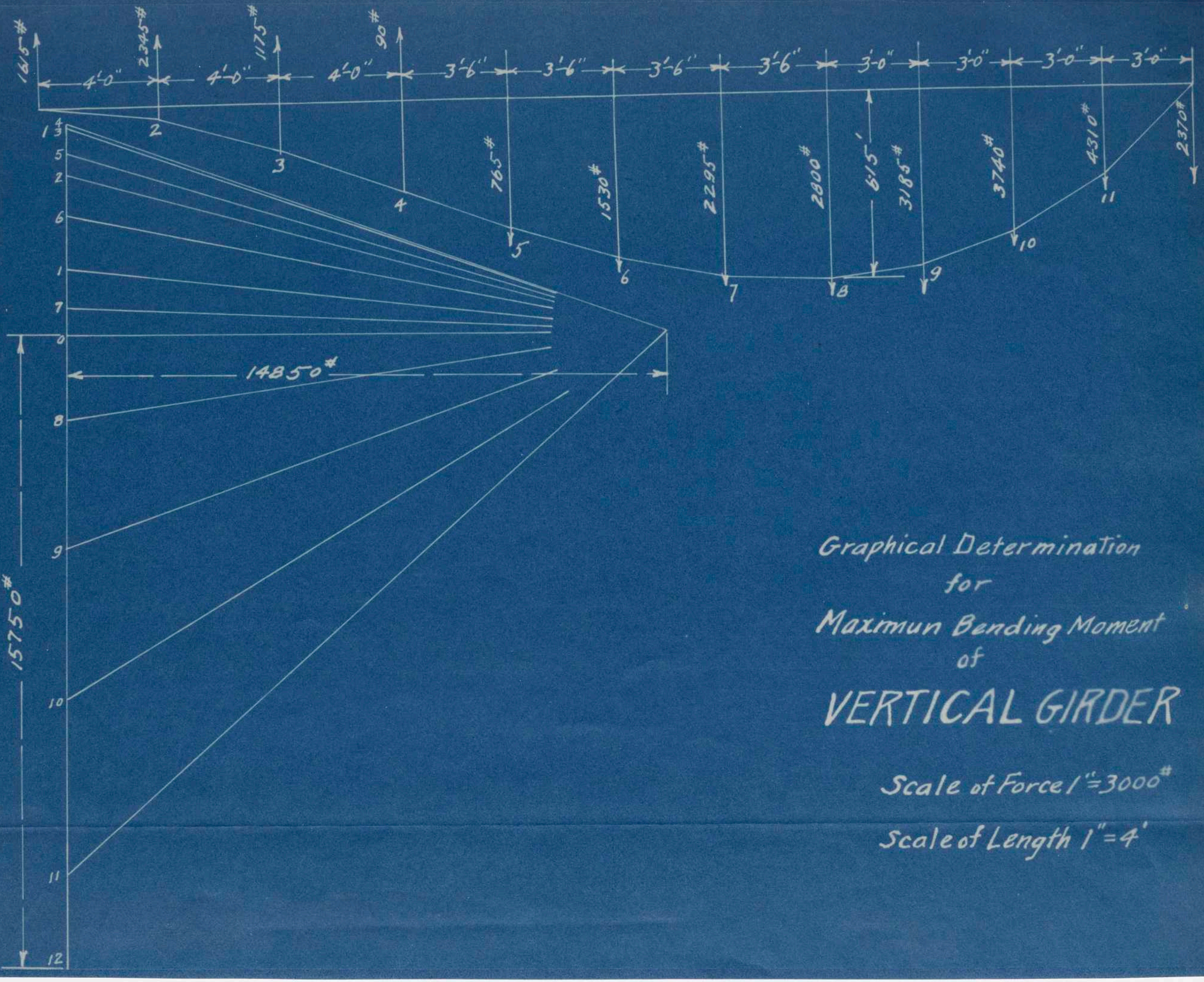
Web stiffeners are provided at intervals of 4 ft. and they are 2-3x4x1/2 L^s. Near the end of the girder diagonal stiffeners are used to secure good distribution of thrust and at the same time relieve the end thrust on the web.

The vertical girders may be easily designed by finding the maximum bending moments produced by the set of horizontal forces. These forces are given in the last second column of Table A. They are the differences of loads on horizontals due to the depth of water and the action of verticals. In other words they are reactions of horizontals on the verticals. The positive sign indicates the force pushing the vertical toward the downstream and the negative sign, that toward the upstream.

The moment will be determined by a well known graphical method as shown in accompanying diagram. The magnitude is found to be 6.15 x 14850 = 91400 ft-lbs. per lin. ft. Use three verticals and space them 14'-2" apart. Then the total moment will be 14.17 x 91400 = 1,295,000 ft-lbs.

Considering only the flanges of vertical girders take the bending stress and as the depth of girder between the centers of flanges will be about 5.5 ft., the area of flange required is $\frac{1295000}{5.5 \times 12000}$ = 19.6 sq. in. Therefore use 2-6"x6"x1/2" angles and 9/16" x 14" plate.

The sill pressure per ft. is 15750 lbs as measured from diagram. This value is same as maximum shearing stress on the girder. 18"x 22" manholes are made through the web at every panel. If the web takes all the shearing



Graphical Determination
for
Maximun Bending Moment
of
VERTICAL GIRDER

Scale of Force 1" = 3000#

Scale of Length 1" = 4'

stress the thickness required will be

$$t = \frac{15750 \times 14.17}{3 \times 12 \times 10000} = 0.62" \text{ say } 5/8"$$

The thickness of sheathing plate will be determined by equation (25), using allowable unit stress 12000 lbs. per sq. in. The most economical plate can be obtained by spacing intercostals at same distance as that between girders. But this can not always be accomplished for the distance between the verticals will influence the arrangement. We will space the intercostals in first three panels at 4'-9" and that in remaining panels at 3'-6 1/2".

The loads on the intercostals are found by Navier's principle which may be expressed in an equation

$W = \frac{b^2}{a^2+b^2}$ abwh where b is the length of intercostal, a the spacing between them, whab the total load on the rectangular sheathing plate and W the total load on the intercostal. The size of each intercostal can be calculated by considering it as a beam uniformly loaded and using unit stress 12000 lbs. per sq.,in.

Table D gives the theoretical and practical thickness of sheathing at every panel.

Table D

Space between	a in ft.	b in ft.	h in ft.	$t = \frac{ab}{2}\left\{\frac{w}{f}\,\frac{2h+b}{a^2+b^2}\right\}^{\frac{1}{2}}$	Thickness used
1 - 2	4.9	4.33	0	0.245 in.	3/8 in.
2 - 3	4.9	4.33	4.33	0.417 "	5/8 in.
3 - 4	4.9	4.33	8.67	0.539 "	do
4 - 5	3.54	3.5	13.0	0.488 "	do
5 - 6	3.54	3.5	16.5	0.543 "	do
6 - 7	3.54	3.5	20.0	0.594 "	11/16 in.
7 - 8	3.54	3.5	23.5	0.638 "	do
8 - 9	3.54	3.0	27.0	0.625 "	3/4 in.
9 - 10	3.54	3.0	30.0	0.656 "	do
10 - 11	3.54	3.0	33.0	0.687 "	13/16 in.
11 - 12	3.54	3.0	36.0	0.716 "	do

Table E gives the section modulus and size of intercostal at every panel.

Table E

Space between	a in ft.	b in ft.	h in ft.	$W=\frac{b^2}{a^2+b^2}$ abwh	$\frac{12\ b\ W}{8\text{x}12000}$	Size
1 - 2	4.90	4.33	2.17	1270 lbs.	0.688	4"x 13.8#
2 - 3	4.90	4.33	6.50	3800 "	2.06	"
3 - 4	4.90	4.33	10.83	6340 "	3.44	"
4 - 5	3.54	3.5	14.75	5650 "	2.47	"
5 - 6	3.54	3.5	18.25	7000 "	3.06	"
6 - 7	3.54	3.5	21.75	8330 "	3.64	"
7 - 8	3.54	3.5	25.25	9680 "	4.23	"
8 - 9	3.54	3.0	28.50	7920 "	2.97	"
9 - 10	3.54	3.0	31.50	8770 "	3.29	"
10 - 11	3.54	3.0	34.50	9600 "	3.60	"
11 - 12	3.54	3.0	37.50	10430 "	3.92	"

Heel and Miter Posts, Pivot and Pintle.- These posts receive the concentrated pressures at one side and reactions at other side. As stated somewhere above, the reactions may be assumed uniform between the mid-way of two panels next to the horizontal. Greater the depth of the post, nearer to the truth is the assumption. This can be easily seen by applying the principle of the mode of distributing wheel loads to road bed or railway ties. If the above assumption is followed, the determination of the size of post is quite easy. The critical section is at the low~~er~~est part, where the loads and consequently the reactions are the greatest. Fig. XI shows the portion of post adjacent to last two panels. The concentrated loads are

$$\frac{58.6}{2 \times \frac{25}{58.6}} \times 6180 = 425000 \text{ lbs.}, \quad \frac{58.6}{2 \times \frac{25}{58.6}} \times 6750 = 464000 \text{ lbs.}$$

and $\dfrac{58.6}{2 \times \frac{25}{58.6}} \times 3590 = 247000$ lbs. The uniform loads will

be $\dfrac{425000}{3} = 142000$, $\dfrac{464000}{3} = 154500$, and $\dfrac{247000}{1.5} = 164500$ lbs. per ft. The maximum bending moment in last panel will be

$$\frac{(232000\ \ 247000)3 \times 12}{8} = 2{,}155{,}000 \text{ in lbs. (approx.)}$$

Make the cross section of the post a form of T-rail having a depth of 12". Considering only the flanges take the bending stress and using allowable unit stress 13000 lbs.

per sq. in. for cast steel, the flange area is found to be

$$\frac{2155000}{10 \times 10000} = 21.6 \text{ sq. in.}$$

where 10 is the effective depth in inches.

The flange connected to the end of girder will be 2 ft. long and 1" thick. Thus the area is 24 in. which is ample. The other flange will be curved at the contact surface, the radius of curvature being 2 feet. The length of chord will be made 9" and the depth from the chord to inner edge 2.5", thus making a net area of 22.5" plus the curved portion.

For the shearing force of 247,000 lbs., we will make web 1" thick to resist it.

Both posts will be made equal. Their cross-sections are shown in drawing.

The pivot does not receive much load when the gate is shut, because by means of air chamber and water ballast the weight of gate may be greatly neutralized by the buoyancy. Besides that the friction along the edge of the leaf will counteract any force tending to move the gate in vertical direction.

When the gate is open the pivot will experience compressions both in vertical and horizontal direction. Their values are given in equations (10) and (11) or (10a) and (11a), for this design.

Let us assume that the water level is 18 feet above the sill when the gate is open. Then the buoyancing force is 62.5 x 18 x 58.6 x 6 = 396000 lbs.

The weight of sheathing	=	122000 lbs.
" " " horizontals =	=	202000 "
" " " verticals	=	34380 "
" " " intercostals	=	16140 "
" " " posts	=	14400 "
" " " rivets, gussets, footwalk, ladder, etc.	=	100000 " (assumed)
Total weight	=	489000 lbs.

The net vertical load on the pivot will be 93000 lbs.

The head of pivot is usually made spherical. Mr. Landsberg derived a formula to determine its size.

$$R = \sqrt{\frac{3P}{2\pi f}}$$

where R is the radius of the sphere, P the load and f the allowable stress which should be not more than 7500 lbs. By substitution we find

$$R = \left(\frac{3 \times 93000}{2 \times 3.14 \times 9000}\right)^{\frac{1}{2}} = 2.45''$$

Use 5" pin.

The horizontal load is $\frac{58.6}{2 \times 39} \times 93000 = 69900$ lbs.

We may allow 8000 lbs. for the shearing stress of the pivot.

5" ~~7-1/2"~~ pin can carry 19.64 ~~44.18~~ x 8000 = 157120 ~~353000~~ lbs. which is far exceeding the imposing load.

The pivot may fail by bending. The pivot head will be projected above the bed plate 5 in. ~~The~~ Considering the resultant is applied at the middle height, the moment is then 2.5 x 69900 = 175000 ~~350000~~ and the maximum fibre stress is $\frac{8 \times \cancel{350000}\ 175000}{\cancel{44.18 \times 7.5}\ 19.64 \times 5} = \cancel{7500}\ 14200\#$. Therefore the pivot is safe.

The center of pivot should not coincide with the center of heel post, for then the end contact surface will always be in touch with the wall surface thus introducing friction in operating the gate. Mr. Debauve gave a method to find the position of pivot as follows:-

In Fig. XII, let a b and a' b' be the positions of center line of contact when the gate is closed and open, o being the intersection. Bisect angle aob' and project the bisector to meet the axis of the leaf c e at p. Then p is the center of pin and g and g' the centers of post. Sometimes the point p is not possible to be made as center of pivot. Any point along o p may be selected.

The pintle is bolted on the top of heel post and fixed to the top girder. It is subjected to shearing force which is equal to the horizontal compression on the pivot as shown by equation (10). Fig. XIII shows a 5" pintle 6" long under a stress of 69900 lbs. The shear is

$\frac{69900}{19.64}$ = 3570 lbs. per sq. in.

Anchorage.- In many gates the anchorage is composed of eye-bars set in concrete or rock. There may be two or more anchor bars used. When two bars are employed, we can find the maximum stress in each of them.

In Fig. XIV, ox is the position of the axis of the leaf when gate is shut, oy that when open, ot any intermediate position, op and oq are anchor bars which are set permanent.

$$\text{Now } T \sin(a+b) = P \sin(d-90)+Q \sin(d+f-90)$$

$$T \cos(a+b) = P \cos(d-90)+Q \sin(d+f-90)$$

from which we get

$$Q = T\,\frac{\sin\left[90-(a+b)+d\right]}{\sin f}$$

$$P = T\,\frac{\sin(a+b+c)}{\sin f}$$

Q will be a compression when $90-(a+b)+d$ is greater than 180°. Also P will be compressive when $a+b+c > 180^{\circ}$. P and Q will have a smaller value if angle f is greater. It is better not to let the anchors subject to compressive stress. Make angles d and f both 90°. Then the maximum stress on OQ will occur when the gate is wide open and that on Op when it is shut tight. These two values will be

$$Q = 69900 \text{ lbs.}$$

$$\text{and} \quad P = 69900 \times \frac{53}{58.6} = 63200 \text{ lbs.}$$

The anchor bars will be set vertically into the side wall as shown in drawing and the force is transmitted through bars inclined at 45°. The actual stress in the vertical direction will therefore be

$$\frac{69900}{\sin^2 45^\circ} = 140000 \text{ lbs.}$$

and the area required is $\frac{140000}{14000} = 10$ sq.in.

Use 2-eye bars 4"x 1 1/8" for each anchor.

The size of anchor plate in concrete should be at least $\frac{140000}{450} = 311$ sq. in. Use a shoe plate 18"x 18".

Sill, Footwalk and Fender Timbers.- The sill is generally made of wood fastened to the lock floor. Greenheart is the best material for this purpose. Because it is not susceptible to marine worms and possesses great strength. The safe bearing value along the grain can be taken as 1300 lbs. per sq. in.

The pressure on the sill is 15750 lbs. per ft. Use a sill 8" deep and 6" wide. The load on the sill will be $\frac{15750}{12 \times 8} = 164$ lbs. per sq. in. The concrete can take 450 lbs. per sq. in.

Footwalk should be provided along the whole length of each leaf. The floor may be either of concrete or plank. A 4 ft. plank footwalk with its floor flushing to the ground line will be used. It is trussed at the topmost horizontal girder and provided with pipe rail at both sides. Ladder, manhole, air-shaft are provided for entering into the interior of the leaf.

Fender timbers are fastened on upper girder flanges at both down and upper stream sides. They are of greenheart and 5"x 6". A timber 8"x 6" is fitted at lowest girder flange on downstream side.

Machinery.- The machinery used in this gate will be similar to that used at Panama and at "the Soo". Fig. XV gives the arrangement of it. A is the turntable, B, B' and B", are the positions of the operating strut in manoeuvring and oa, ob, and oc the positions of leaf axis.

In turning the gate, three forces are to be overcome. These are (1) the inertia of the leaf, (2) hydrodynamic pressure at the face of the leaf, and (3) the friction at the contact surfaces, pivot and pintle.

The moment of inertia of the leaf may be calculated, without serious error, as if a weighted rectangle turns about the edge. Then

$$I = \frac{1}{3} Ml^2$$

when M is the mass of the leaf and l its length. The radius of gyration is

$$r = \sqrt{\frac{I}{M}} = 0.577\ l$$

The mass can be considered as concentrated at 0.577 l from the turning axis. If a force is applied at a distance x from the same origin, it will act on an equivalent mass

$$m = \frac{.577\ l\ M}{xl} = \frac{.577\ l}{x} M$$

The mass of the leaf will be greatest when the water chamber is full of water. This will happen when the gate is opened at high tide. Although openings are provided at the bottom of chamber, the water inside are very much restricted and can not be considered as in a condition of free water outside. In conservative calculation, the water will be assumed to move with the leaf.

The weight of water will be 20 x 6 x 62.5 x 57 = 428000 lbs. The weight of leaf is 489000 lbs.

Attach one end of the strut to the leaf at a distance 18 ft. from point of revolution. Then

$$m = \frac{0.577 \times 58.6 \times 907000}{18 \times 32.2} = 53000 \text{ lbs.}$$

The leaf when wide open is to be parallel to the axis of the lock. Thus the gate will be turned through an angle $90^\circ - \sin^{-1}\frac{25}{58.6} = 64^\circ - 45'$. If the gate is to be opened in 60 sec. and the uniform velocity is to be attained in 10 sec., the angular velocity is

$\frac{64^\circ - 45'}{55} = 1^\circ - 10.7'$ or 0.0206 radians, and the linear velocity of the equivalent mass is 0.0206 x 40.6 = 0.836 ft.

The force required to attain this velocity is

$$f_1 = 53000 \times \frac{0.836}{10} = 4440 \text{ lbs.}$$

Equation (6) gives the moment of the hydrodynamic pressure. The force necessary to overcome it is

$$f_2 = \frac{1.85 \times 62.5 \times 39 \times \overline{.0206}^2 \times \overline{58.6}^4}{8 \times 32.2 \times 18}$$

$$= 4890 \text{ lbs.}$$

The friction is greatest when the gate starts to open or at the time of closure, for then the contact surfaces of miter and heel post will play a part. When the leaf is at any other position the only friction is that due to pintle and pivot which is comparatively insignificant. Since the surfaces of contact at end posts are made segmental, even this friction is not very considerable. To make an accurate calculation we have to know the coefficient of friction, the area of total contact surface and the normal pressure on the surface. This can not be readily done, as we do not know how much the curved surface deflect and the law of variation of normal pressure on the different parts of the surface which are not uniform. Moreover, as the arm of the applying force is quite long, it will be ample enough to assume the force to overcome the friction.

$$F_3 = 1000 \text{ lbs.}$$

The total is 4890+4440+ 1000 = 10330 lbs. This force is normal to the leaf surface and is to be transmitted from the engine through the operating strut. In course of operation the strut inclines to the leaf axis with varying angles. The smallest angle as shown in Fig. XV is reached when the gate is shut. It is found 38^o. The maximum stress on the strut is then $\frac{10330}{\sin 38^o} = 16780$ lbs. The strut is under tension to open the gate and compression to close it. Compression governs the design. The strut is long and may subject to shock. We will therefore use a member whose length to radius of gyration is not larger than 80, i.e. $r \leq \frac{24 \times 12}{80} \leq 3.6$. Use 2-10"x 20# [s laced and spaced 10 in. apart. The maximum stress of the strut is only $\frac{20 \times 24 \times 24 \times 12}{8 \times 2 \times 15.7} + \frac{16780}{2 \times 5.88} = 1975$ lbs. per sq. in. The detail of the strut and its connections to turn-table and leaf are shown in drawing.

The turn-table is made of cast steel, 20 ft. diameter and center bearing. Half turn of the table opens or closes the gate, so that gear teeth are provided for 5/8 the circumference. A 3" pin is fixed 8 ft. from the center which is attached to one end of strut. The table is turned by the power of engine through a set of spurs and bevel gears.

The engine has not only to move the gate but also to overcome the inertia of table and friction of gearings. To cover uncertainties in various resistances we will use a 30 H.P. engine for each leaf.

Fig I.

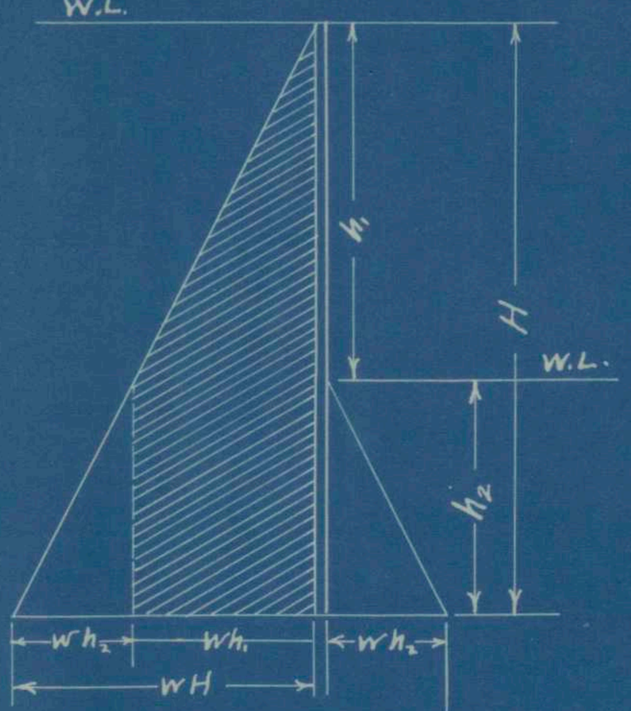


Fig II.

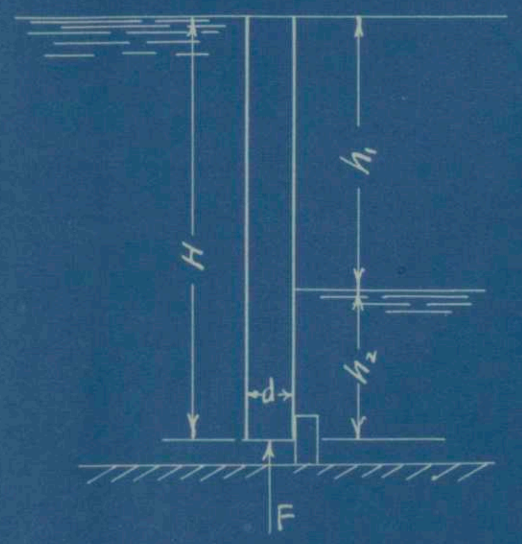


Fig III

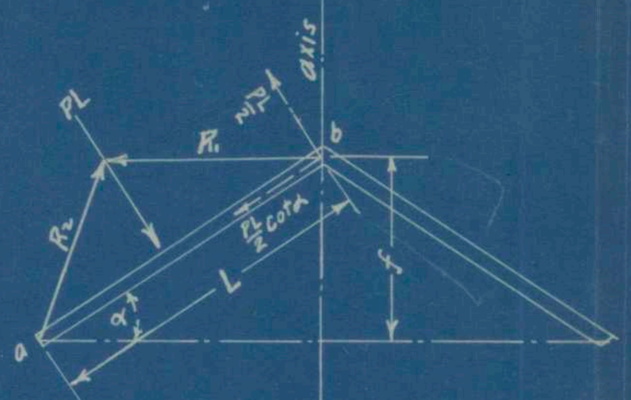


Fig IV

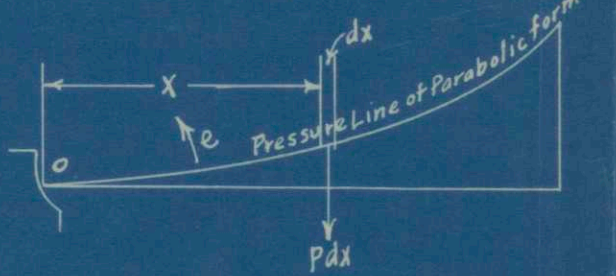


Fig V

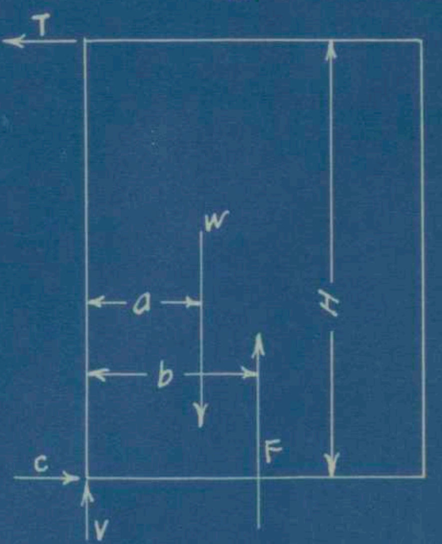


Fig VI

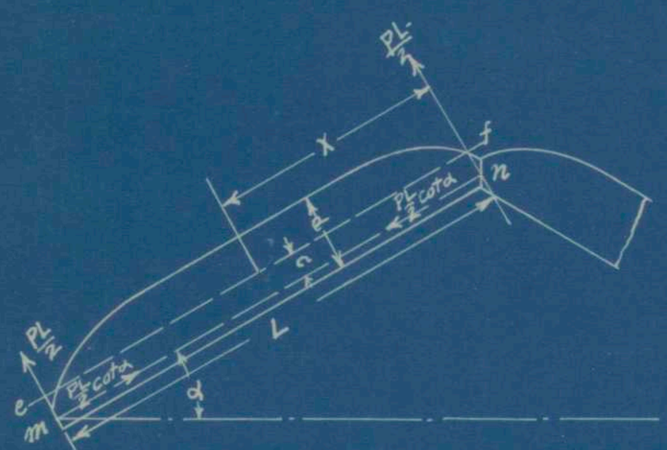


Fig VII.

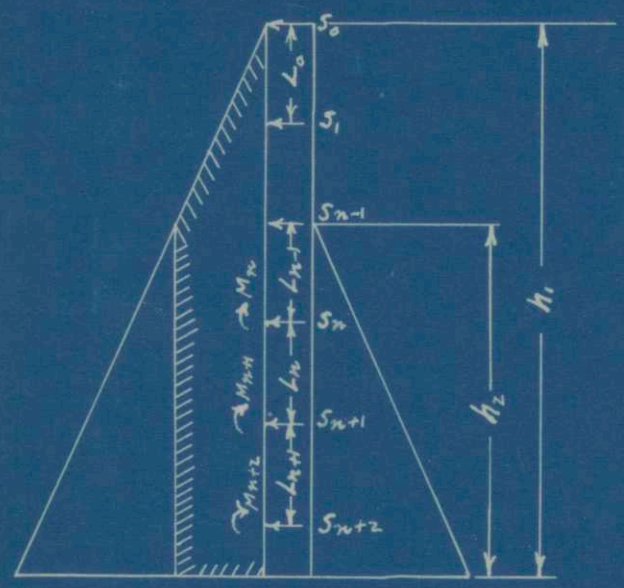


Fig VIII.

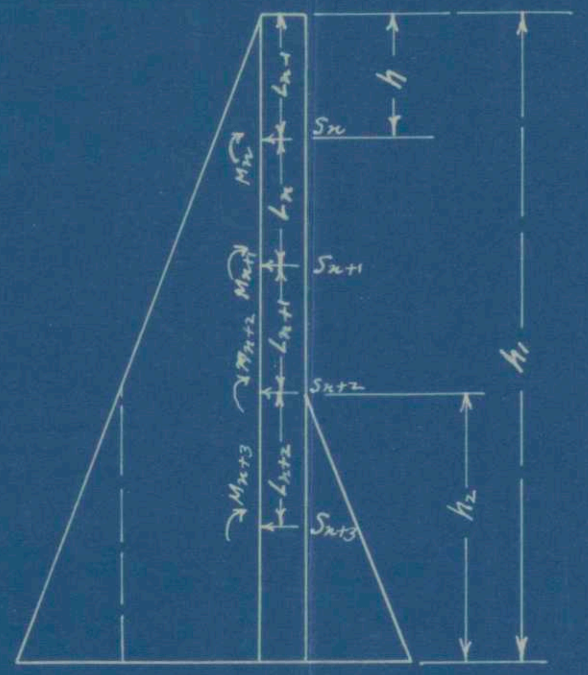


Fig IX

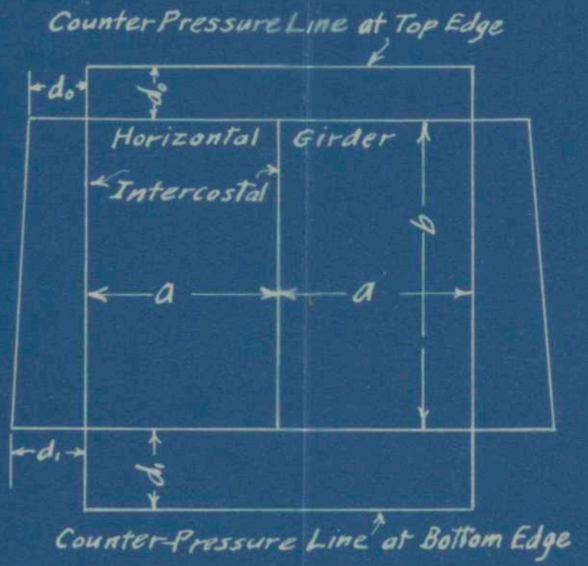


Fig XI

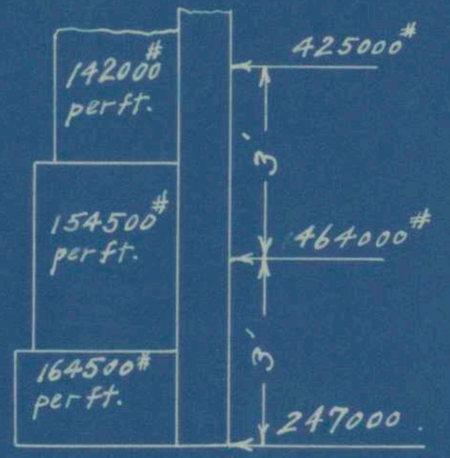


Fig XII.

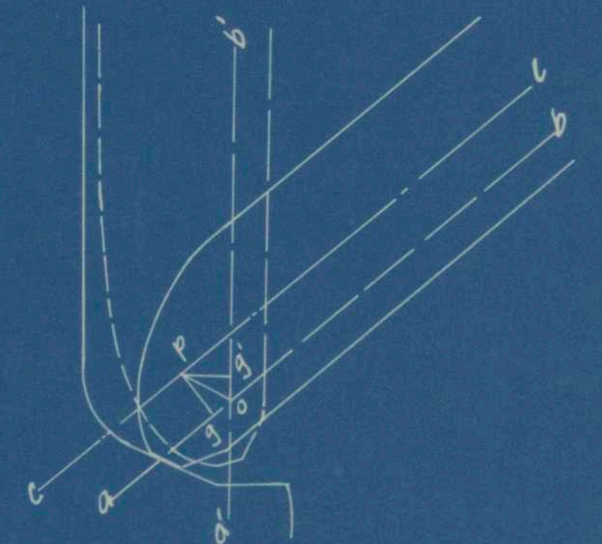


Fig X

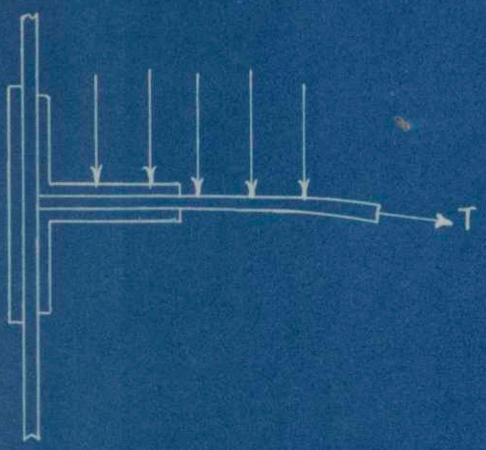


Fig XIV

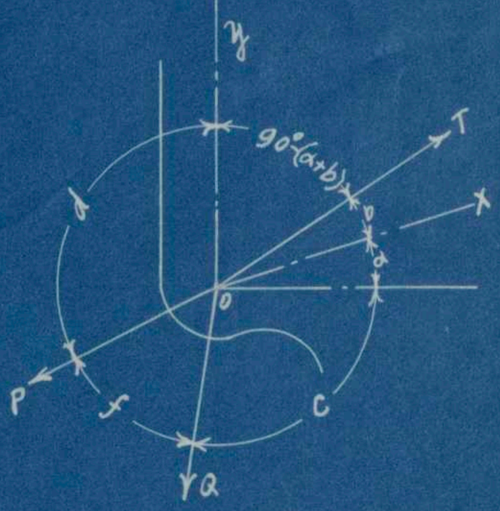


Fig XV

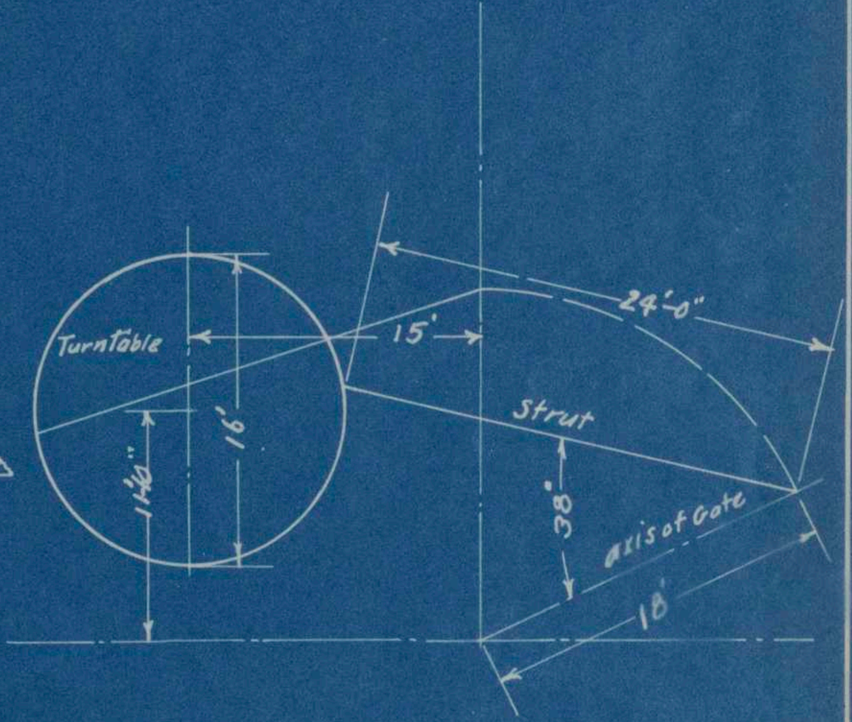
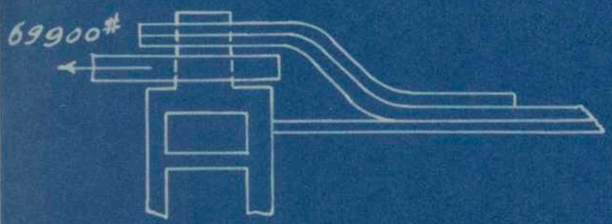


Fig XIII



Ground Line
1" Piperail
Upstream Elevation
Section A-A
Downstream Elevation
Section e-e
Section d-d
Side Elevation
(Toward Miter Post)
Downstream Face
Upstream Face
Detail of Turntable & Strut
Scale 1"=1/8
Detail of Top Pin
Scale 1"=1'
Detail of Top Plan of Miter Post
Scale 1"=1'
Anchorage for Top Hinge
Scale 1"=1'
Lock Wall Face
Water Chamber above this Line
Air Chamber below this Line
Sheathing Splice
Cover Plate Splice
Top Plan
Section b-b
Section C-C
Plank Foot-walk
Note:- Detail of Miter Post same as Heel Post.
Detail of Pivot
Scale 1"=1'
Center Section
Bronze Bush
Lock Floor
Steel Pl.
58'-7 3/4"
39'-0"
20'-0"
DESIGN
FOR A
MITERING LOCK GATE
Scale 3/8"=1'
May 1914