this moment be taken up entirely between truss B and the girder (and this is the assumption that places the heaviest load on B), the extra load on the truss is 18,000 ÷ 10 = 1,800 lbs. per lineal foot. The direct load described above

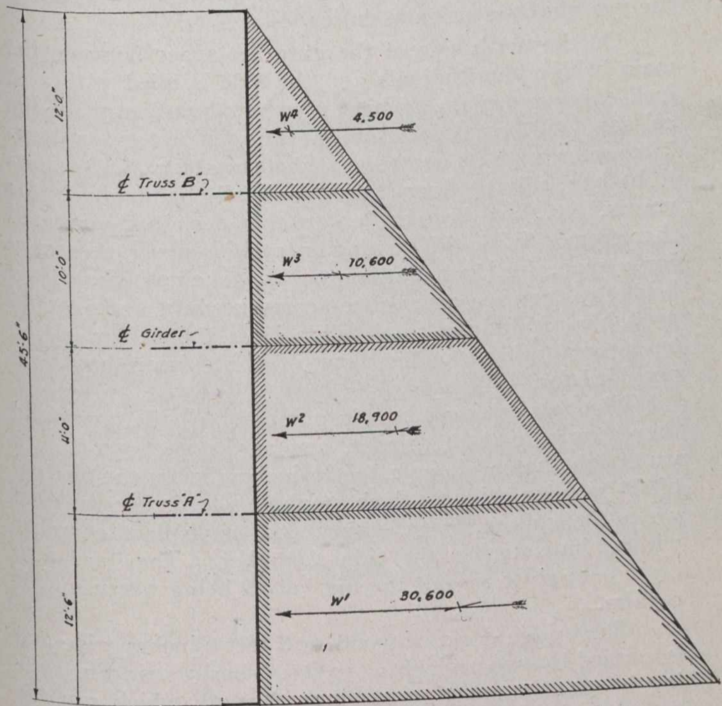

Fig. 2.—Hydrostatic Loading Against Rolling Caisson.

is 9,280 lbs. per lineal foot. Therefore, total load on truss is 11,080 lbs. per lineal foot.

The load on truss A is 24,600 lbs. per lineal foot. As mentioned previously, the unit stress specified was 12,000 lbs. per square inch, and this was interpreted by all parties to mean 12,000 lbs. per square inch tension and the same for compression, unless the Dominion Government Specifications '08 formulae would give less when reduced for $\frac{l}{r}$ ratio.

In summer, the compression chord of truss A would carry the water load between panel points and in winter the tension chord would do the same with, of course, a much smaller head. Owing to the full splicing of the chords the maximum bending moment assumed in them, due to flexure, was $1/10\ w\ l^2$. Preliminary calculations were made on two or three sections of both the compression and tension chords, and average values for extreme flexural fibre stress were assumed in order to fix the exact sections. The average amount deducted from the compression chord, including end post, was 3,200 lbs. per square inch. The $\frac{l}{r}$ ratio for all these sections was so small that no reduction of the 12,000 lbs. became necessary.

In the design of truss A very little need be mentioned except the large sizes of the members and gussets. The chords of this truss are the largest simple truss chords that have ever been built by the Dominion Bridge Company—attention being called to the fact that the gross area of the tension chord in truss A is 342 square inches. The magnitude of this may be realized when one recalls the fact that the cross-sectional area of the centre chord of one of the big Lachine trusses is 302 square inches and that of the lower chord of the St. John arch is 347.5 square inches. In the detailing, every precaution was taken to see that the rivets were capable of developing in each group the requisite amount of stress. The value assumed for rivets stressed in two directions simultaneously was adjusted accordingly.

The horizontal load on the girder is 14,600 lbs. per lineal foot and nothing has been taken from this figure to allow for the negative loading induced in it by the overturning effect of the upper water. The girder is stiffened underneath by 24-inch @ 80 lbs. I's in order to carry the load of tidal chamber when full of water. The cover plates are run far enough beyond their theoretical length to take up their value.

Owing to the magnitude of the loads and to the difficulty of developing the full stress in those portions of the cover plates and skin plates that act as flanges, it became essential to use 1-inch diameter rivets in all main connections of truss A and the girder.

The load on the upper truss B, as mentioned before, equals 11,080 lbs. per lineal foot, and in this truss no difficulty was experienced with the capacity of the rivets, hence ⅞-inch diameter rivets were employed. As in truss A, the working stresses used in the main chords and end post were reduced for flexure by the following amounts: Compression chords, 3,400 lbs. per square inch; end post, 3,000 lbs. per square inch; tension chord, 3,100 lbs. per square inch.

The distribution of the loads bearing on the vertical sides became at once not only very important but also one of the most difficult problems met in the design of the whole gate. The reactions were: Truss A, 1,512,000 lbs; girder, 900,000 lbs.; truss B, 680,000 lbs. The reaction of truss A, if spread over a length of 11.75 feet (½ of 11 + ½ of 12.5 = 11.75 feet its own proportion) and a width of 18 inches would give an average concentration of about

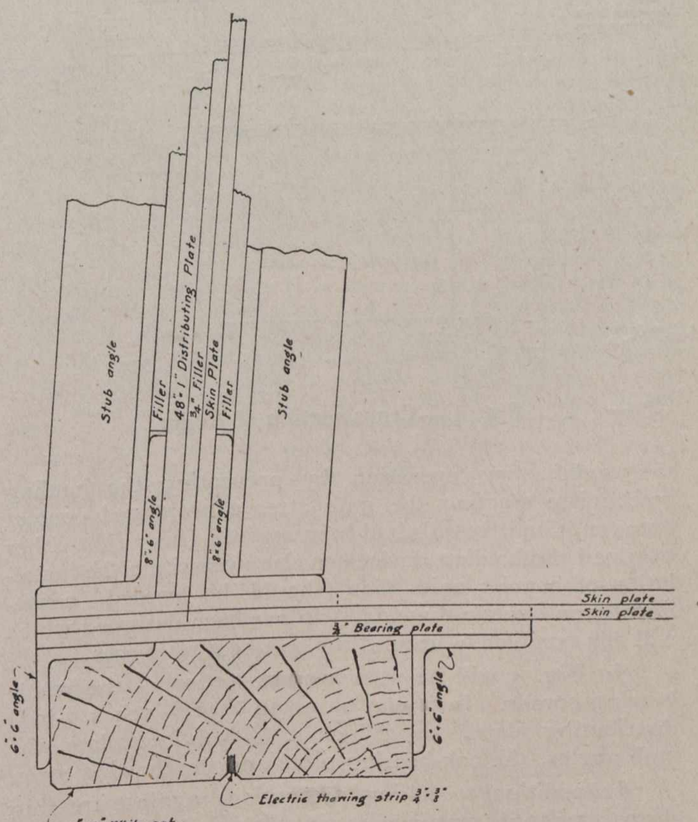

Fig. 3.—Bearing Corner of Rolling Caisson.

86,000 lbs. per square foot—600 lbs. per square inch—which is even then very large for sill pressures. It was felt that, though the width of the caisson was fairly good (19 feet) the end itself would, owing to its construction, be hardly stiff enough to really distribute the highly con-