q = Pressure at toe of dam.
u = Uplift at heel of dam.
W_u = Weight of dam per foot of length, diminished by excess uplift.

Subscripts e, f and u refer to "pond empty," "pond full" and "pond full with uplift," respectively.

Foundation Reaction, Pond Empty.—Pressure on foundation due to direct load plus or minus couple $W \times e_1$. See Fig. 2.

$$p_e = (W/L)(1+6e_1/L) \quad \ldots (1)$$
$$q_e = (W/L)(1-6e_1/L) \quad \ldots (2)$$

Foundation Reaction, Pond Full, No Uplift.—See Figs. 3 and 4. $Hd = pL^2/6 = qL^2/6$. Solving for p and q:—

$$p = q = 6Hd/L^2 \quad \ldots (3)$$
$$p_f = p_e - p \quad \ldots (4)$$
$$q_f = q_e + q \quad \ldots (5)$$

Also,
$$p_f = (W/L)(1-6e_2/L) \quad \ldots (6)$$
$$q_f = (W/L)(1+6e_2/L) \quad \ldots (7)$$

From this,
$$e_2 = (Hd/W) - e_1 \quad \ldots (8)$$

Foundation Reaction, Pond Full, With Uplift.—When $u \leqq p_f$, no change results due to uplift, as far as overturning moment is concerned, and equations 4 and 5 are to be used. For sliding, however, subtract $uL/2$ from W, since only that part of the dam which is in actual contact with the foundation produces friction.

When $u > p_f$ the procedure is as follows: The excess uplift BCD in Fig. 6 causes foundation reaction in Fig. 4, namely p_f and q_f, to change to p_u and q_u respectively in Fig. 6, so as to make the sum of all moments equal zero and the sum of all vertical forces equal zero.

Taking moments about A (see Fig. 6),
$$W(L/2-e_2) = uL^2/3+(q_u x/2)(x/3).$$

But $W = uL/2+q_u x/2$, and solving for x,
$$x = [6W(L/2-e_2)-2uL^2]/(2W-uL) \quad \ldots (9)$$

And
$$q_u = (2W-uL)/x$$
$$= (2W-uL)^2/[6W(L/2-e_2)-2uL^2] \quad \ldots (10)$$

From similar triangles:—
$$u/L = y/x, \text{ and } y = ux/L.$$

Also
$$(q_u-p_u)/L = (y-p_u)/(L-x).$$

And from these two equations,
$$p_u = u-q_u[(L-x)/x] \quad \ldots (11)$$
$$W - \Delta BCD = W_u = \tfrac{1}{2}(p_u+q_u)L \quad \ldots (12)$$

Since
$$q_u = (W_u/L)(1+6e_3/L),$$
and
$$p_u = (W_u/L)(1-6e_3/L),$$
$$e_3 = \tfrac{1}{6}L(q_u-p_u)/(q_u+p_u) \quad \ldots (13)$$

When $p_u = 0$, $e_3 = L/6$.

For sliding, subtract $\frac{1}{2}uL$ from W.

In conclusion it will be said that the uplift is usually assumed much larger than necessary. If a horizontal plane be cut through the base of the dam, the area of the voids in practice would not exceed 25% of the total area, except under extraordinary conditions, and since the water could only fill these voids, the unit uplift pressure exerted could only be 25% of the pressure assumed. The unit pressure of the dam against the foundation would then be $1 \div 0.75 = 133\%$ of the actual contact area, since only 75% of the base area would come in contact with the foundation.

PRESSURES IN PENSTOCKS CAUSED BY THE GRADUAL CLOSING OF TURBINE GATES*

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The theory of water-hammer in penstocks is one of the most intricate problems confronting engineers. Mr. Gibson has made a considerable addition to the knowledge of the subject; but exact solutions for all conditions have not yet been reached.

He has developed rational equations for penstocks of constant diameter and negligible friction head. In cases where friction head is relatively large, and particularly where the penstock has a varying diameter, we are still very far from a practical solution.

At any instant during gate closure, the discharge through the gate is a function of the static head, plus water-hammer head, less friction head, all measured at the gate.

Mr. Gibson makes the assumption that the friction head at the gate is proportional to the square of the velocity adjacent to the gate. This assumption appears to the writer to be only approximate, since it is well known that, at any instant during surges, the velocity is materially different at different points on the penstock.

It is the writer's opinion that, during the period $2\,L/a$, subsequent to a single small instantaneous closure, the friction head at the gate is not constant, although the velocity adjacent to the gate during that period is constant. This constantly varying friction head makes it very difficult to include this feature in the equations for water-hammer, and, in all probability, Mr. Gibson's methods are as close as can be obtained. It would be of interest to know how much difference the incorporation of friction head makes in ordinary problems.

For a penstock with varying diameter, auxiliary waves are set up each time a water-hammer wave passes a point of change in diameter. For this condition, Mr. Gibson's equations would not apply. His methods would apply, but they would be exceedingly difficult, if not impossible, of practical application.

It is evident that the maximum rise of pressure is materially influenced by the characteristics of the gate-closure curve. It is well known that modern turbine governors do not provide a uniform rate of gate closure throughout the stroke. It is also possible that governors of different types have different closure characteristics. Consequently, complete data for the determination of maximum water-hammer for specific cases must include the gate-closure characteristics of the governor, which is to be a part of the machinery.

*Discussion (presented to the American Society of Civil Engineers) of Norman R. Gibson's paper (see September 4th and 11th issues of *The Canadian Engineer*).

TO CONSIDER PAY FOR ESTIMATES

Tentative arrangements have been made for a conference to be attended by committees from the American Institute of Architects, the U.S. Engineering Council, and the Associated General Contractors of America, for the purpose of considering "payment for estimating." At present it is planned to hold the first meeting November 17th at Hotel La Salle, Chicago.

For the Associated General Contractors, a special committee has been appointed, consisting of the following: A. P. Greensfelder, of the Fruin-Colnon Contracting Co., St. Louis; James O. Heyworth, of the Chicago company of that name; and A. E. Wells, of the Wells Brothers Construction Co., Chicago. Mr. Greensfelder is the chairman of the A.G.C.'s Committee on Methods, which had general supervision of the discussions regarding "payment for estimating."

Engineering Council has named as its delegates, Ralph Modjeski, Samuel G. Neiler and Theodore L. Condron, while the Post-War Committee of the American Institute of Architects will be represented by George C. Nimmons, Richard E. Schmidt and Frederick W. Perkins, with Henry K. Holsman as alternate.

In a report recently made by F. A. Gaby, chief engineer of the Hydro-Electric Power Commission of Ontario, the estimated cost of the proposed hydro-radial from Toronto to Bowmanville, via Pickering, Whitby and Oshawa, is given as $8,360,794; the estimated annual revenue, $1,118,003; and the estimated annual operating expenses, including 5% interest on capital expenditure, $1,076,175. Of the $1,118,003 income, $873,140 is expected from passenger traffic and $244,863 from freight.