flood tide will then be filling in for 1.9 hours) and we have the head decreasing from 27½ ft. to 22 ft., but afterwards increasing to 23½ft. at 5.2 hours.

After this the head will gradually decrease to 13½ ft. at 9 hours, when the tide in the external bay having sufficiently ebbed, the water from the spillway will be again discharged into it and the head will steadily rise to its previous maximum of 28½ ft. at low tide, as shown by the curve of heads.

The average head, in this case, from low tide to low tide works out at 22¼ ft.

It will perhaps be wondered why I select a drop of 6 ins. per hr. for the high level basin, and what the difference would be if we selected a greater or lesser drop. Up to a certain maximum, which would occur at about 250,000 h.p. and at a 26 in. per hr. drop, the available horsepower increases with an increase in the hourly drop allowed. However, it must be borne in mind that as the hourly drop increases, the maximum head decreases, and the minimum head decreases more rapidly still, and we soon reach a case that cannot well be met by any ordinary turbines.

To illustrate this, a 4-in. drop in the high level basin gives maximum head of 30¼ ft., minimum of 20 ft., and average of 26½ ft., but 6 in. drop gives 28½ ft. maximum, 13½ ft. minimum and average of 22¼ ft., and this seems to be about the maximum relative range over which we can expect a single turbine to act; we could employ duplex turbines, mounted vertically on the same shaft to the same generator, but with separate draft tubes (as suggested by James Saunders), but this means considerable added cost and is to be avoided if possible.

We are thus forced to adopt one of those compromises that are often met with in engineering design, and must adopt a drop in our high level basin that will entail such a range of head that can be successfully met by the turbine designer. I am at present in correspondence with many of the leading turbine manufacturers and have no doubt they can largely overcome this difficulty of this variable head, in a moderate priced turbine, but the whole question of this variable head problem is so novel to them that the correspondence is necessarily lengthy to get them to understand the conditions involved in a tidal plant. However, one maker shows a turbine that gives 5,000 h.p. at 120 ft. head and efficiency of 86%, and also the same horsepower at 220 ft. head and 80% efficiency, all at a constant speed of 300 r.p.m., so I think we can have no doubt that once the conditions involved at Hopewell are understood by the manufacturer and designer, we can obtain turbines that will meet the assumed condition of a 6 in. hourly drop in our high level basin and with a probable average turbine efficiency of about 83%.

As a matter of fact the initial installation called for at the present time by the existing population, would be about 90,000 gross horsepower, and we can readily calculate that this corresponds to an hourly drop of 4 ins. in the high level basin. Now, as the population grows, and the demand for electricity increases from 90,000 to 120,000 h.p., we can steadily improve the ratio of the two basins and thus greatly improve our power output (up to say 200,000 h.p.) and yet never exceed the limit of 6 in. hourly drop which I have assumed as the present practical limit of single-turbine operation.

To improve this basin-ratio to the ideal condition in which the Memramcook would have half the effective area of the Petitcodiac, I propose to use electric shovels and an electric tramway (both operated by surplus power from the initial installation), expropriate by government charter the low-lying farms of the Memramcook valley at a fair and equitable rate, and shovel out the basin according to power requirements, removing the material by the electric railway and dumping it below the eastern dam where it would be largely carried away by the tide or could be formed into a useful embankment—wharf, railway terminal, or the like.

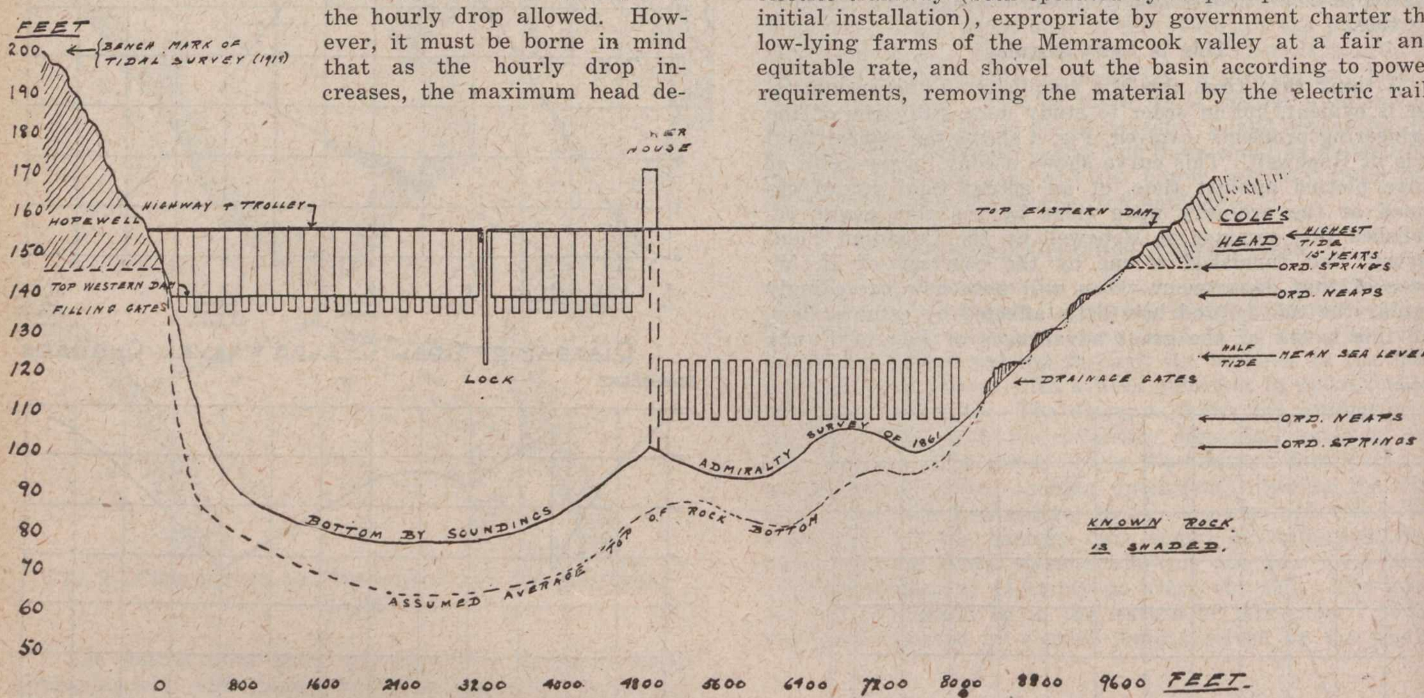

Fig. 6—Approximate Profile of Proposed Dams, Hopewell Hydro-Electric Plant

Fig. 6 shows the approximate profile of the western and eastern dams. Since it would not be policy to attempt to get the extra power that spring tides would give, the western dam need only have the height of high water, ordinary neap tides, but the height of the eastern dam would have to be carried up beyond the highest probable spring tides to insure that the low level basin would never be flooded at high tide. Fig. 6 shows this dam carried 3 ft. higher than the highest spring tide that occurred during a period of 15 years. As before mentioned, the tops of the dams could be utilized as highways, and would thus be of invaluable local benefit, and a light steel structure is indicated on the western dam for this purpose. The masonry width called for by the calculations would not be sufficient for a roadway, but a light steel structure could be winged out on top of the dams and made of sufficient width to carry a good roadway, an electric tram and the power transmission lines that would go both west to St. John and east to Halifax.

The flap-gates for filling the high level basin and emptying the low level basin are indicated in the sketch-profile. They should present no engineering difficulties, but they should be designed of sufficient size to pass readily the full volume of water required by the final and maximum development.

We now come to the question of the river bottom and the location of rock for a good dam foundation, and here I must apologize, for I cannot give any exact data. I took

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