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The Canadian Engineer

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HAMILTON ENTRANCE OF TORONTO-HAMILTON HIGHWAY

PRESENT ROUTE HAS DANGEROUS CURVES AND BAD GRADES—TWO ALTERNATIVE ROUTES CALLING FOR BRIDGES AND ONE REQUIRING FILL — DISCUSSION OF THEIR RELATIVE MERITS.

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At the time construction work was commenced on the Toronto-Hamilton concrete highway, there were certain sections, particularly at the western end, which had not been definitely located. For the section between Burlington and Hamilton it was finally decided to follow what is known as the Plains Road, which lies north of Burlington Bay, about one-eighth to one-quarter of a mile from the shore.

This road enters Hamilton across a deep, wide valley, which forms the end of the bay and the present route is a narrow, steep and tortuous one, winding down to a low bridge past an old wayside hotel which gives to this part of the road its name—the Valley Inn Road. On its climb up the western side of the valley the road passes under the Grand Trunk Railway main line tracks to Toronto, and four hundred yards further on it runs over the Grand Trunk Hamilton - London branch and the Canadian Pacific Guelph Junction branch, and comes into the city over what is known as the High Level bridge via York Street. The plan of this present route is shown on the accompanying map (Fig. 2) and a profile of it on Fig. 5. The total drop in this section is about one hundred feet and in a distance of 4,000 feet there is 3,600 feet on a grade averaging 5½ per cent., being as steep as 11½ per cent. at one point. It has more than 620 degrees of curvature in this distance. At the overhead Grand Trunk Railway crossing there is a sharp reverse curve on a 7½ per cent. grade where a driver has a view ahead of not more than 50 feet. (See Fig. 3.) This view is often still further obstructed by dust and smoke from passing trains, while the noise from them prevents any signals being heard. For traffic such as is expected on the Toronto-Hamilton highway, the whole section, and this spot in particular, would be dangerous, and in spite of any little local improvements which might be made, it would greatly impair the usefulness and efficiency of the whole scheme as far as Hamilton and the west are concerned, to attempt to utilize the present location.

While only half of the Valley Inn Road lies inside of the city of Hamilton, its improvement is not only of interest to the city in connection with the Toronto-Hamilton highway but the scheme of improvement adopted will have an important bearing on the whole problem of future entrance routes into the city from the north. So limited and difficult is the approach of Hamilton from this direction that the whole question deserves special attention, and it is very desirable that whatever is now done may be made to work into a general scheme which will ultimately provide for all future needs.

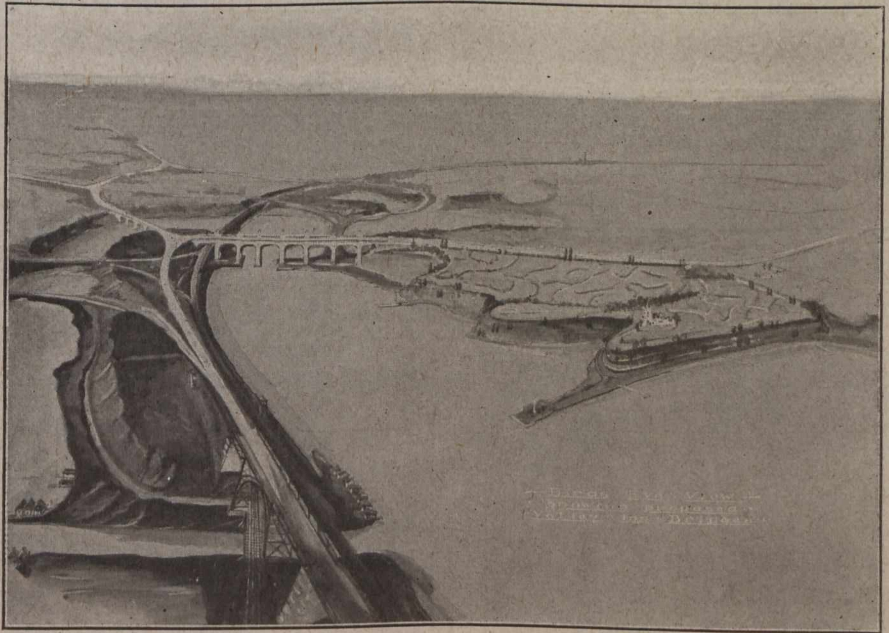


Fig. 1.—Bird's-eye View Showing Proposed Valley Inn Bridges, Hamilton (Route No. 3).

The proposal first submitted by the engineers of the Highway Commission is shown on the map and profile as Route 1. Its principal feature is the large fill and the corresponding heavy cuts. It would save about 38 feet of the 100-foot drop and improve the curves of the road. The present Grand Trunk Railway overhead crossing would still be utilized and made somewhat less dangerous. There would still be 1,750 feet of 4 per cent. and 1,150 feet of 4.75 per cent. grades; this latter being steeper than the standard maximum grade adopted for the rest of the highway.

Route No. 2 has been developed by the engineers of the Hamilton Bridge Works Co. at the request of the city Board of Control. It involves the building of a high level bridge 1,620 feet long. By it the shortest practicable route is obtained and grades and curves are all but eliminated. Considering only the problem of bringing the highway into Hamilton it probably presents the ideal solution.

Route No. 3 has been proposed by J. J. Mackay, O.L.S., president of the Canadian Engineering and Contracting Co., acting in the capacity of secretary of the local Town Planning Commission. This route is also a high level one, but follows the old road across the valley and is 15 feet lower than Route 2. Its principal structure would be a bridge 1,200 feet long, but in addition to this there would be two overhead railroad crossings and considerable cut and fill. The grades and curves would be very slight.

There have been other routes or modifications suggested but they have all been abandoned for one reason or another.

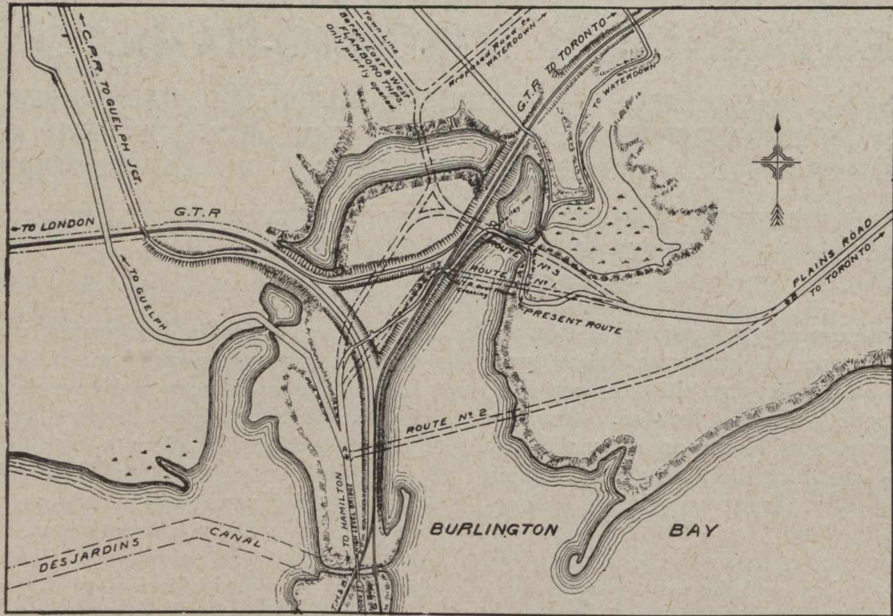


Fig. 2.—Map Showing Present and Proposed Routes of Toronto-Hamilton Highway Entrance into Hamilton.

In comparing these routes from a practical point of view there are many things to take into consideration. While Route No. 1 is apparently the simplest and cheapest, Hamiltonians look dubious at the mention of a fill. It is remembered that the Grand Trunk, when the line was first brought into Hamilton, found it an extremely expensive and difficult piece of work to construct a fill across this same ravine at a point only 300 yards away. Old-timers yet tell how on four separate occasions their fill sunk out of sight into the water over night from a height of about 60 feet; how that great quantities of brush, logs, whole trees, and at last many sacks of wool were thrown into the embankment in a vain attempt to hold it. A boring made near the site of the proposed fill shows 65 feet of silt before hard clay is reached and what is supposed to be rock at 96 feet.

Fig. 3.—The G.T.R. Overhead Crossing on the Valley Inn Road. The Fill Called for by Route No. 1 Would Begin at the Far Side of This Bridge.

Under these conditions a fill 40 feet high might easily over-run by a large amount the estimated cost. This uncertainty, coupled with the fact that so little would be accomplished toward eliminating grades and that the route would be practically useless for electric railway development, make this solution unsatisfactory.

The principal objection that can be raised against Route No. 2 is its cost. Foundations for a bridge on this formation would be a rather expensive item even for a steel bridge, while a concrete structure would be out of the question.

By adopting Route No. 3 the size of the main structure would be much reduced and as it would rest on the old fill which now forms the Valley Inn Road it is probable that better foundations could be obtained. Two additional bridges would be required, however, for the railway crossings, one a 172-foot and one a 207-foot bridge. These would have to be of shallow floor construction and expensive for their size.

Other details which enter into a comparison of the two routes are given below. In order to reduce them to the same basis the whole route between the points marked A and B (Fig. 2) is considered in all cases. The profiles (Fig. 5) are also drawn on this basis.

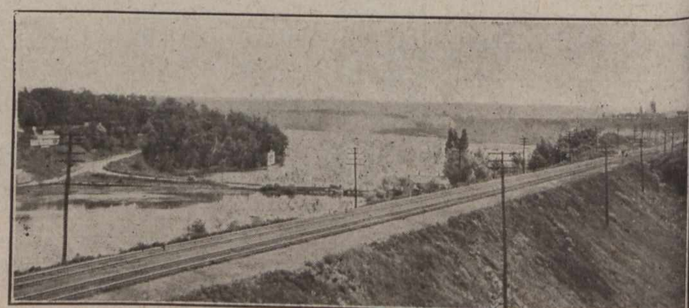


Fig. 4.—The End of Burlington Bay, Looking South Toward Hamilton, Showing the Valley Inn Road and the G.T.R. Fill.

	Length.	Extra land required.
Present route	6,700 feet	
Route No. 1	5,940 feet	3.21 acres
Route No. 2	4,830 feet	4.14 acres
Route No. 3	6,770 feet	2.70 acres

It will be seen that Route No. 3 is 1,940 feet longer than Route No. 2; or, in other words, it would add over a third of a mile on the distance between Hamilton and Toronto.

Until a thorough investigation has been made and a careful detail estimate of costs prepared it will be impossible to properly compare the advantages of the various routes.

There is still another consideration which will have an influence on the choice. It may be possible for the proposed Hydro-Electric Radial Lines to utilize to good advantage part of Route No. 3, including the two overhead crossings. If this could be arranged it might so reduce the cost of Route No. 3 to the city and the Highway Commission as to give it a strong preference. Furthermore, if in conjunction with the Hydro-Electric Radials and the municipalities lying north of Hamilton a plan could be agreed upon which would give a high level entrance for all this district, and form a starting point for the continuance of the provincial road system westward, the extra

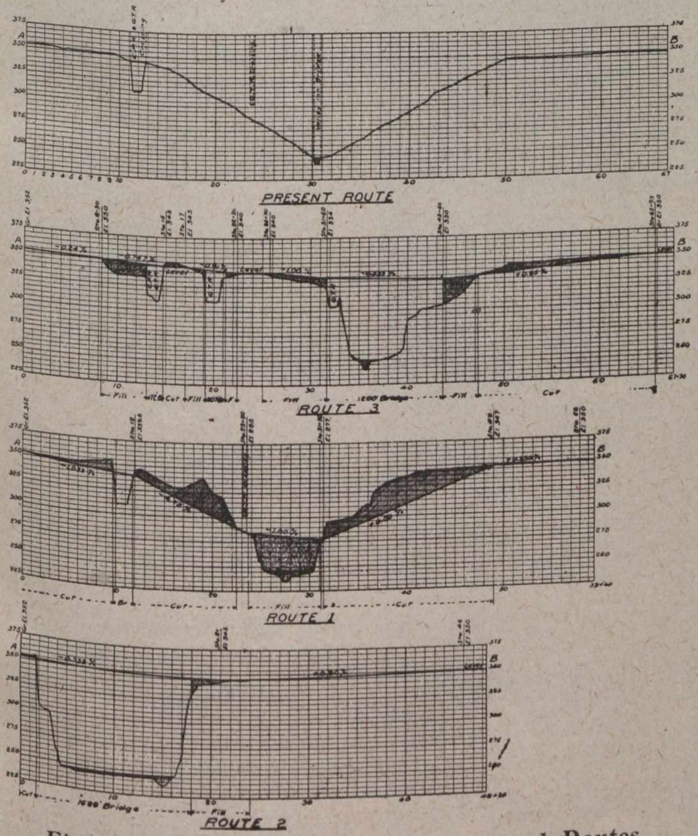


Fig. 5.—Profile of Present and Proposed Routes, Toronto-Hamilton Highway Entrance.

cost of Route No. 3 would be a good investment. This side of the question is now being investigated but definite information regarding it is not available at the time of writing.

Finally, no investigation can be considered thorough that does not take into consideration the artistic point of view. The scenery in this neighborhood is very picturesque and the proposed bridge would not only be seen by all travellers entering the city from this direction but it would be in full view from many points in the city and on the "mountain." In fact, it could easily be made part of, or a continuation of, Hamilton's beautiful park system, while the completion of the highway will open up the whole north shore of Burlington Bay which promises to become one of the finest park and residential sites in this district. Fig. 4 gives some idea of the possibilities along this line.

The artist, Lawrence Munro, of Hamilton, has pictured a bird's eye view of the territory shown on the map (Fig. 2) as it would appear if the Route No. 3 is adopted, and this is reproduced as Fig. 1.

Whether it is worth while for Hamilton to spend something in thus building an imposing portal, and how much, should be considered in the final decision.

The writer is greatly indebted to R. K. Palmer, C.E., chief engineer of the Hamilton Bridge Works Co., and J. J. Mackay, president of the Canadian Engineering and Contracting Co., for surveys and other data used in the compilation of this article.

CANADIAN RAILROAD FINANCING IN NEW YORK

As a result of Premier Borden's visit to New York, on his way to Washington, Wall Street has many stories as to forthcoming Canadian railroad loans. A New York despatch of last week, read as follows: "Prospects of a large loan by a group of New York bankers to Canadian railways, approximating $100,000,000, was the chief topic of discussion in Wall Street banking circles to-day. Canadian borrowing in this market has met with popular approval here, and it was predicted that if the negotiations for the loan were carried through the issue would be marketed with as great success as the numerous Dominion and municipal issues which are now closely held by American investors.

"Members of a group of bankers, it was said, have assured Premier Borden that they stand ready and willing to furnish the necessary capital, providing the Canadian government will guarantee the issue when it is made. These bankers, it is said, assured Premier Borden and his committee that the loan with Canada in back of it could be sold to investors here on an interest basis almost as low as the Canadian government bonds."

Premier Borden journeyed to the United States with the object of obtaining the advice and services of practical railroad men in regard to the Canadian railroad situation. A commission of railroad experts, probably three, will study this matter and report to the government. There is little doubt that the premier discussed with leading New York bankers the financial phases of our railroad problem, which, whether or not nationalization is ultimately favored, are among the most important factors and those leading to the present situation. With the good record of Canadian securities and credit generally, especially in the New York market, since the outbreak of war, it is quite probable that the premier received assurances that if a large Canadian railroad loan were decided upon, it would have the support of leading New York bankers. Inquiries by *The Canadian Engineer* at Ottawa seem to indicate that the question has not been under consideration by the government.

A Canadian Northern issue of $6,000,000 6 per cent. 1 and 2-year notes was sold by Wm. A. Read and Company, New York, and the Dominion Securities Corporation, Toronto, in the New York market last week in one day, and other financing, of which *The Canadian Engineer* is not at liberty to write at present, is in contemplation.

In the category of world-famous dams Elephant Butte dam stands pre-eminent as a flood conserver. Its height, 318 ft., is topped by Arrowrock dam, 350 ft., and Shoshone dam, 328 ft., are service achievements. Elephant Butte in the capacity of its reservoir, however, ranks every other storage dam in the world completely controlled by man. Its reservoir when full will contain 856,000,000,000 gal., or enough to cover Delaware 2 ft. deep. This is a third more than is stored by the $17,000,000 Assuan dam in Egypt and two-thirds more than the combined capacity of all the reservoirs built or projected for Greater New York City.

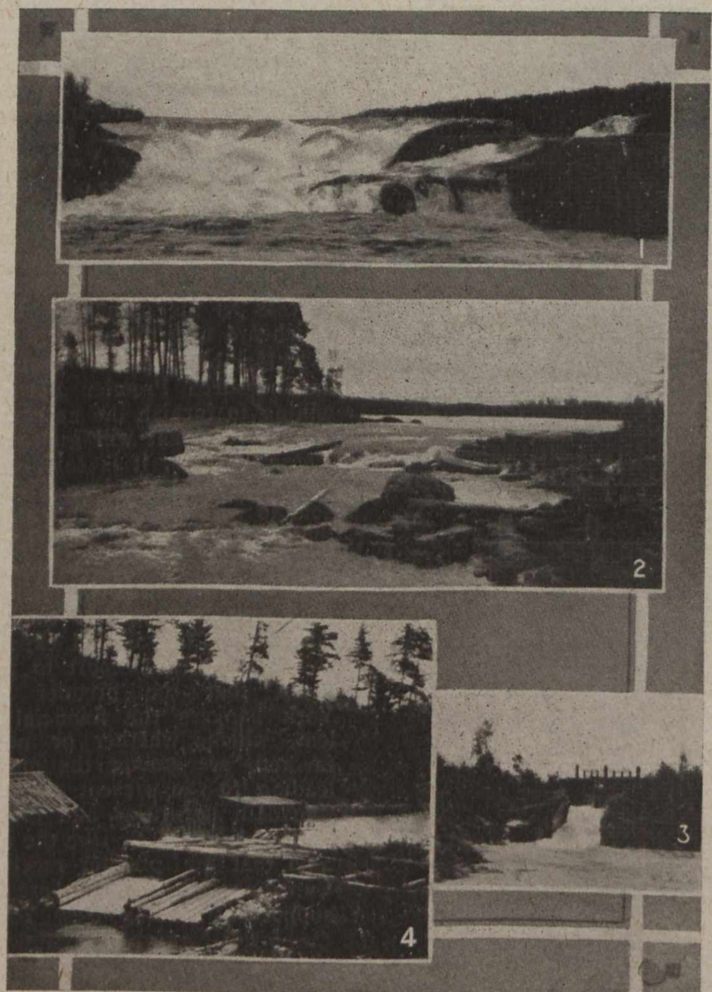
TWO METHODS OF REGULATING RAINY LAKE

SHOULD STORAGE BE REGULATED TO AUGMENT FLOW OVER EXTREME LOW-WATER PERIOD OF POSSIBLY FEW YEARS, OR ONLY TO AUGMENT TO GREATEST POSSIBLE EXTENT THE ANNUAL LOW-WATER FLOW?—ONE OF THE ENGINEERING QUESTIONS INVOLVED IN LAKE OF THE WOODS INVESTIGATION.

TO most people the Lake of the Woods district is a comparatively unknown region, and the popular impression probably is that it is of little or no importance to the inhabitants of either the United States or Canada other than those who live in or own interests in the immediate vicinity. That is not the case.

"The investigation which the International Joint Committee has carried out in regard to regulating the levels and outflow of the Lake of the Woods," said Lawrence J. Burpee, Canadian secretary of the International Joint Commission, in a recent article in the American Review of Reviews, "shows among other things that the navigation, power and other interests that will be affected by the Commission's decision, have invested something over one hundred million dollars in the Lake of the Woods district; that the natural resources of the region are enormous and only beginning to be developed; and that communities so far apart as Duluth and Winnipeg are more or less directly interested in the fixing of a level on this lake that will give the maximum benefit to the people on both sides of the boundary."

In speaking of the importance of the interests involved in the regulation of matters in general pertaining to boundary waters, Mr. Burpee says that "when it is remembered that these boundary waters support a population of over seven million people, American and Canadian; that the navigation interests alone of the Great Lakes represent an enormous investment; that approximately ninety-five million tons of freight, valued at more than eight hundred million dollars and carried by twenty-six thousand vessels, are transported on these waters annually—more than three times the volume of freight taken through the Suez Canal; when you add to this the rapidly increasing power interests along these waters, and all that depend upon them, and the vital uses of the Great Lakes and their connecting waterways for domestic and sanitary purposes, it is not very difficult to appreciate the opportunities for usefulness that lie before the International Joint Commission. * * *



1—Curtain Falls at Outlet of Crooked Lake; 2—Falls at Outlet of Basswood Lake; 3—Dam at Outlet of Birch Lake on International Boundary; 4—Dam at Outlet of Knife Lake.

"Of the questions that have already been referred to the Commission by the governments of the United States and Canada for investigation and report, two are of of special importance. The first requires the Commission to report what levels of water in the Lake of the Woods can be maintained which will best meet the needs of the various interests on both sides of the boundary: navigation, agriculture, fishing, lumbering and power."

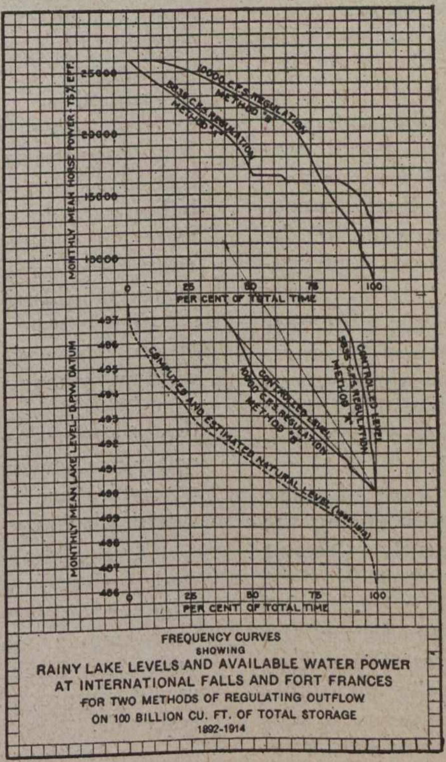
The Lake of the Woods and its tributaries derive their water supply from an area of 26,750 square miles, almost equally divided between the United States and Canada; an area that is greater than New England, exclusive of Maine. "For almost twenty years the obstruction and use of these waters at the outlet of the lake for power purposes, which outlet is wholly in Canada, some seventy-five miles from the boundary, have been a source of a great deal of irritation and complaint upon the part of the people, whose land on its south shore, it is claimed, has been injuriously affected by this obstruction at the lake's outlet," said Charles A. Magrath, chairman of the Canadian section of the International Joint Commission, in a speech before the Canadian Club at Ottawa last fall.

"This dispute has been the source of ineffective diplomatic negotiations all these years. In 1912 the United States and Canada referred the whole question to the Commission. Since then the Commission has conducted a most extensive investigation into all questions involved into the controversy, and we have at this time every reason to believe that we will be able to submit to the two governments unanimous conclusions and written recommendations which they will accept and adopt, and thus secure for the people of both countries the most advantageous use of these boundary waters, which are so essential to the growth and development of the country on both sides of the line, and invaluable to the people in the carrying on of their industries and other pursuits."

One of the first acts of the International Joint Commission in conducting the Lake of the Woods investigation, was to appoint Arthur V. White, of Toronto, and Adolph F. Meyer, of Minneapolis, as consulting engineers to report fully upon all the engineering and scientific phases of the problem.

After many months of field and office work, these consulting engineers made a preliminary report in three volumes. The introduction to this report, containing a review of all the causes that led up to the investigation, was summarized in last week's issue of *The Canadian Engineer*.

After explaining in clear detail the need for physical data, and the field operations which were undertaken to supply same, the report gives the complete results of those operations and an analysis of same. Voluminous evidence is presented regarding observed water levels, and outflows and run-offs from the various lakes and watersheds in the district. The engineering features of the problems, as based upon this data, are then discussed under the general headings of "Reservoir Control in General," "Regulation of Outflow from Rainy Lake," "Regulation of Levels and Outflow, Lake of the Woods," "Outflow Capacity Required Under Various Methods of Regulation, Lake of the Woods," and "Desirability and Practicability of Regulating Levels and Outflows, Lake of the Woods."



FREQUENCY CURVES
SHOWING
RAINY LAKE LEVELS AND AVAILABLE WATER POWER
AT INTERNATIONAL FALLS AND FORT FRANCES
FOR TWO METHODS OF REGULATING OUTFLOW
ON 100 BILLION CU. FT. OF TOTAL STORAGE
1892-1914

Fig. 1.

Following is a summary of the chapters regarding reservoir control in general and regulation of outflow from Rainy Lake:—

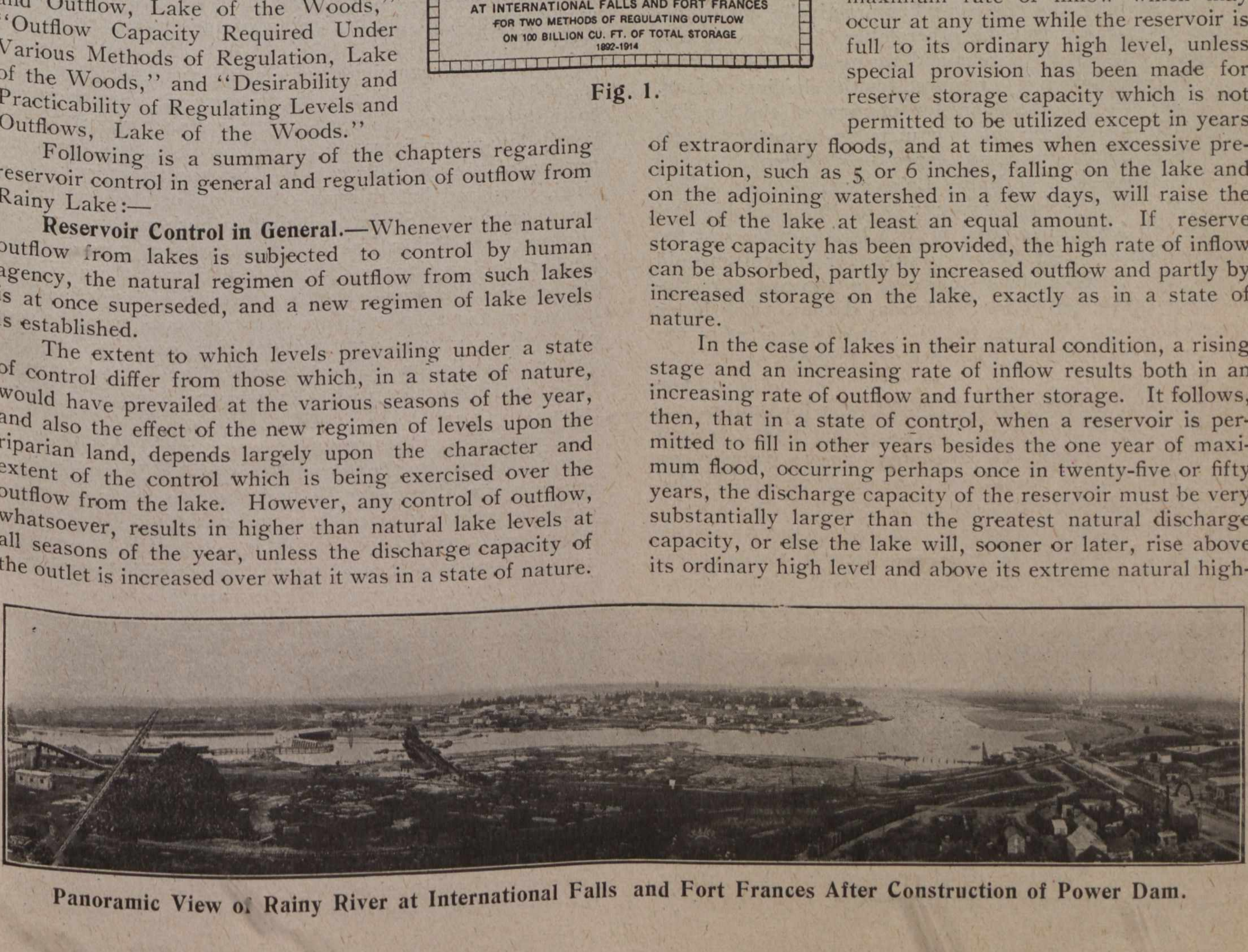
Reservoir Control in General.—Whenever the natural outflow from lakes is subjected to control by human agency, the natural regimen of outflow from such lakes is at once superseded, and a new regimen of lake levels is established.

The extent to which levels prevailing under a state of control differ from those which, in a state of nature, would have prevailed at the various seasons of the year, and also the effect of the new regimen of levels upon the riparian land, depends largely upon the character and extent of the control which is being exercised over the outflow from the lake. However, any control of outflow, whatsoever, results in higher than natural lake levels at all seasons of the year, unless the discharge capacity of the outlet is increased over what it was in a state of nature.

Control of the outflow from any lake, with a view to its equalization, presupposes the storage of flood water. Such a lake, then, acting as a storage reservoir, will frequently become filled at a time when the rate of inflow is high. The frequency with which this condition will recur will depend upon the extent to which the outflow can be equalized by the available storage, *i.e.*, the filling will recur more frequently with a reservoir of relatively small storage capacity than with one of relatively large capacity. The extent to which flood water can economically be utilized in the equalization of stream flow depends upon the cost of flowage rights, and the cost of impounding and other works required for the utilization of the stored water, as compared with the benefits accruing to the various interests utilizing this water.

In the operation of all storage reservoirs whose capacity is insufficient to completely equalize the outflow over a long period of years,—and in principle this means essentially all of them—a time will come, if the ordinary high level is not to be exceeded, when the discharge of water, even under the most intelligent operation, must be substantially greater than the maximum outflow in a state of nature. The maximum rate of outflow to be expected under a state of control will equal the maximum rate of inflow which may occur at any time while the reservoir is full to its ordinary high level, unless special provision has been made for reserve storage capacity which is not permitted to be utilized except in years of extraordinary floods, and at times when excessive precipitation, such as 5 or 6 inches, falling on the lake and on the adjoining watershed in a few days, will raise the level of the lake at least an equal amount. If reserve storage capacity has been provided, the high rate of inflow can be absorbed, partly by increased outflow and partly by increased storage on the lake, exactly as in a state of nature.

In the case of lakes in their natural condition, a rising stage and an increasing rate of inflow results both in an increasing rate of outflow and further storage. It follows, then, that in a state of control, when a reservoir is permitted to fill in other years besides the one year of maximum flood, occurring perhaps once in twenty-five or fifty years, the discharge capacity of the reservoir must be very substantially larger than the greatest natural discharge capacity, or else the lake will, sooner or later, rise above its ordinary high level and above its extreme natural high-



Panoramic View of Rainy River at International Falls and Fort Frances After Construction of Power Dam.

water stage. Without increased outflow capacity, any control of outflow, whatsoever, must inevitably result in raising the lake to a higher level than the highest stage which would have occurred during the given period in a state of nature.

Regulation of Outflow from Rainy Lake.—Having obtained what we believe to be substantially correct values of run-off from the Upper Rainy watershed from 1892 to 1914, we made a study of the extent to which it appears practicable and desirable to regulate the outflow from Rainy Lake in order to aid in regulating the levels and outflow from the Lake of the Woods; and at the same time to ensure the advantageous use of the waters of Rainy Lake itself. The necessity for this special consideration of Rainy Lake will be appreciated when it is realized that the Rainy Lake watershed at Fort Frances and International Falls constitutes 54.2 per cent. of the total drainage area of the Lake of the Woods and contributes 62.0 per cent. of the total run-off.

Although the inflow into Rainy Lake for a number of years has, to some extent, been controlled by dams located at the outlets of a number of tributary lakes, yet, so far as our present study is concerned, the effect of these dams—most of which are used in connection with logging operations—has been relatively small.

In general, the method of operating most of these logging dams consists, first, in storing water during the fall, winter and early spring; and subsequently releasing it soon after the spring break-up,—usually about May 1st —when the sluicing of logs begins. On a number of lakes, the storage capacity compared with the area of the tributary watershed is so small that the effect of the dams on the distribution of the run-off is practically negligible. In the case of some of the other lakes, however, a substantial portion of the winter and spring flow is held back until the period when it is required for the sluicing of logs. The usual effect of the operation of most of the logging dams is to accentuate the irregularities in outflow from those lakes which the dams respectively control.

The mean area of Rainy Lake is 345 square miles, consequently its storage capacity is 9.6 billion cubic feet per foot of depth, or a total of 67.2 billion cubic feet for a fluctuation of 7 feet. Such a range, for example, would be afforded between elevations, say, of 490 and 497, Department of Public Works, Canada, datum. The latter elevation, it may be observed, is that of the crest of the dam at International Falls and Fort Frances.

During the past few years, while the outflow from Rainy Lake has been under control, the fluctuation has actually been greater than 7 feet, so that under present conditions of control, the available storage capacity on Rainy Lake may be considered to be about 70 billion cubic feet. Since the construction of the Kettle Falls dams at the outlets of Lake Namakan, in 1914, about 100 square miles of additional lake area has become available for storage. This area embraces Namakan, Kabetogama, Sand Point, Crane and Little Vermilion Lakes, and their connecting waters. As the storage per foot depth on these lakes is 2.8 billion cubic feet, a total storage capacity of 30 billion cubic feet could be obtained by a draft of a little less than 11 feet. Contemplated power developments on the Vermilion and the Kawishiwi Rivers would provide additional storage capacity.

A storage capacity of 150 billion cubic feet could be obtained by increased storage on Rainy Lake, and the lakes controlled by the Kettle Falls dams, or else by means of the secondary storage available on such lakes as Vermilion, Birch, Manitou, Otukamamoan, White Otter, Lac La Croix, Basswood, and Saganaga.

The storage, therefore, which may be considered available under present conditions—that is, by reason of existent structures—is not less than 100 billion cubic feet. This storage may be expected to be available even though it should become desirable, in the future, to reduce the range of fluctuation in level of Rainy Lake, or of the lakes above Kettle Falls, or both.

It is well known to those familiar with meteorological studies, that consecutive years of low precipitation and consequently low stream flow, occur in more or less readily distinguishable cycles of twenty-five or more years. In the period of years under consideration, 1892-1914, the extreme low-water period extends from July, 1910, to July, 1913, when considering the case of 100 billion cubic feet of total storage capacity; and for the case of 150 billion cubic feet of total storage capacity the extreme low-water period extends from July, 1910, to at least July, 1915. The greatest depletion, in each case, would have occurred in April, 1912.

The characteristic feature of the first method of regulation, which we will term "Method A," is to limit the outflow, whenever the reservoir storage is being drawn upon, to the highest possible rate which, by the judicious use of storage, can be maintained during the extreme low-water period, extending over some years. Whenever the reservoirs are full and water is being wasted, a higher rate of discharge may, of course, be economically utilized.

Under Method A, an effort is made to fill the reservoir each spring on the assumption that it cannot be foretold whether the succeeding summer or, indeed, the succeeding years, may or may not be the exceptionally dry period over which it is intended to equalize the flow, and to do which would require, at the beginning of the dry period, a full reservoir.

Under the second method of regulation, which will be termed "Method B," the aim, in general, is to augment to the greatest possible extent the ordinary recurrent, or yearly low-water flow, and to utilize the largest possible portion of the available run-off from the tributary watershed, insuring that this run-off maintains, for much the larger proportion of the total time, a high uniform rate of outflow from Rainy Lake. In the present case in particular,—unless danger of total depletion of the available storage is imminent—effort is made to maintain and to utilize, as far as possible, a minimum outflow of 10,000 c.f.s. This rate is approximately the capacity of the present installation at Fort Frances and International Falls, and almost equal to the average flow for the past twenty-two years.

Comparing the amount of water which would have been wasted under the two methods of regulation, it is at once apparent that under Method A very much more water would have been wasted than under Method B. On the other hand, 100 billion cubic feet of available storage capacity, under Method A, would have provided a uniform flow of 5,835 c.f.s. during the extreme low-water period from July, 1910, to July, 1913; whereas, under Method B, by which the flow of 10,000 c.f.s. is maintained into the dry period as far as practicable, the same 100 billion cubic feet of storage would have resulted in a low-water flow, from July, 1910, to April, 1911, of only about 3,500 c.f.s. Generally speaking, it may be said that, under Method B, the natural low-water flow extending over a period of exceptionally dry years is not substantially increased; whereas, under Method A, the increase is very considerable,—a fact, indeed, to be expected, inasmuch as this increase in the extreme low-water flow is made the governing consideration of regulation in this Method A. In other words, under Method A, a uniform rate of outflow is maintained over the entire dry period of some years, and this rate is considerably higher than the minimum to which the regulated flow would fall under Method B.

Upon observing the various regulated rates of discharge under Method B, it may be perceived that an in-

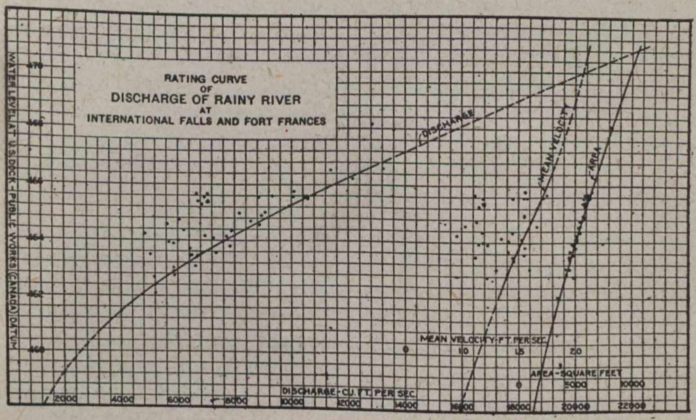


Fig. 2.

crease in available storage capacity of from 100 to 150 billion cubic feet, results in an average increase in utilized discharge, extending over a period of more than twenty-two years, of only 247 c.f.s. This is less than 3 per cent. of the average discharge which could have been utilized during this period with 100 billion storage. The *extreme* low-water flow for the fall and winter of 1910-1911 would, also, only be increased from 3,500 to 4,330 c.f.s.

From the standpoint of equalization of flow alone, it may be stated, that if the Upper Rainy reservoirs are to be operated according to Method B, additional storage capacity over and above the 100 billion cubic feet at present available cannot, economically, be provided. If storage on the Upper Lakes is feasible at all, it must find its justification, primarily, in the increased advantages resulting from a reduction in the extent of the fluctuation of the level of Rainy Lake itself, coupled with such benefits

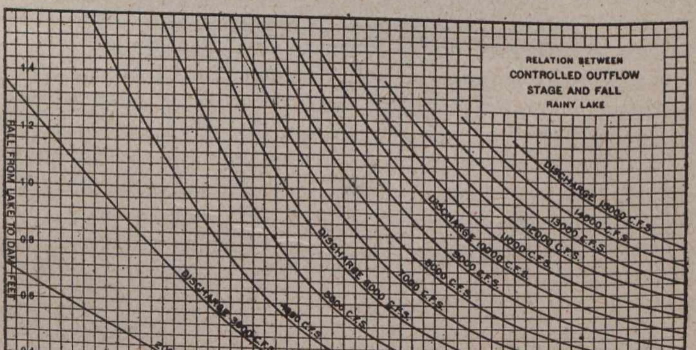
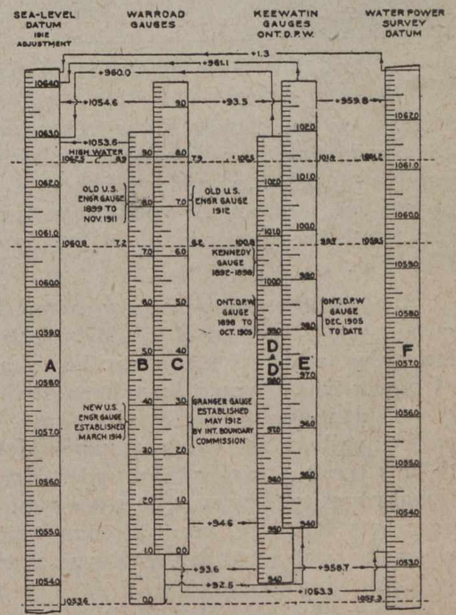


Fig. 3.

as may be derived, locally, from the storage of water in the secondary reservoirs. On the other hand, if the Upper Rainy reservoirs are to be regulated according to Method A, an aggregate storage capacity of 150 billion cubic feet would increase the low-water flow from 5,835 c.f.s. to 6,710 c.f.s., or about 15 per cent. above what could have been obtained from 100 billion cubic feet of storage.

Now, in order to show the effect of these two methods of regulation on power development at International Falls and Fort Frances, frequency curves (see Fig. 1) have been prepared. The computations from which these curves have been made show that on an average, with 100 billion cubic feet of total available storage capacity, 1,968 more horse-power would have been available if Method of Regulation B had been applied instead of Method A.

In making this power study it has been assumed that the secondary storage above Rainy Lake would first be drawn upon, and that during the spring run-off the draft on Rainy Lake, made during the previous winter, would first be replenished. In this power study the available head was determined by subtracting the mean tailwater level for each month from the mean headwater level. The tailwater level was determined from the rating curve of discharge of Rainy River (see Fig. 2) by adding .2 foot for average fall between the gauge to which this rating curve is referred and tailwater at the power plant. The headwater level was determined by subtracting the fall corresponding to the given stage and discharge, as determined from the curves of Fig. 3, from the Rainy Lake level. In addition, a correction was made for backwater. During the winter months and during the period of heavy spring run-off, from December 1st to June 1st inclusive, a correction of one and one-half feet was made for backwater due to ice on the Rainy River, and to floods occurring on the lower tributaries of Rainy River. Between June 1st and December 1st, an average allowance of one-half foot was made for backwater. With these essential data at hand, the available horse-power was then compared on the basis of an over-all efficiency of 75 per cent.

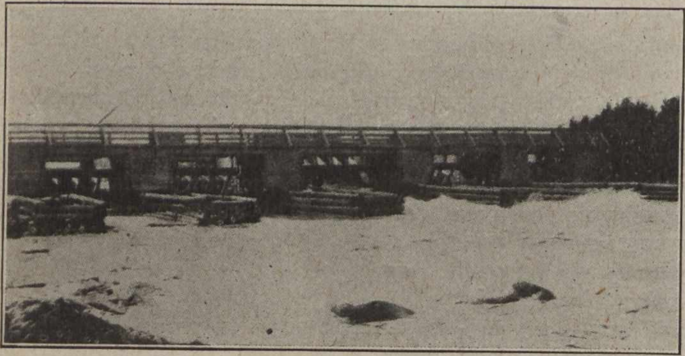


Correlation of Gauges and Datums—Lake of the Woods.

The effect of the various methods of regulating outflow from Rainy Lake on regulation of the levels and outflow from Lake of the Woods will be discussed later. Yet, it may here be commented that, from the viewpoint solely of the advantageous utilization of the waters of Rainy Lake, the more desirable method of regulation is clearly that termed Method B, and which, as before stated, aims to utilize, for power development, the greatest amount of water without making any attempt to substantially increase the *extreme* low-water flow which may or may not recur once in twenty-five or more years. The *extreme* low-water flow, during such low-water period, when it does occur, may only extend over a few months or perhaps a year, and may be expected as an accompaniment of a protracted period of deficient precipitation extending over, say, three, four or five years.

According to the records of the Minnesota & Ontario Power Co., the maximum monthly average power utilized for paper making and other purposes during 1914 was 27,442 h.p. At a discharge of 10,000 c.f.s. and a lake level of 497, the headwater level at the dam would be

496.5, and the tailwater level would be practically 465.5 if there were no backwater. Assuming an average of 0.5 foot of backwater during the summer, as before stated, the average tailwater level would be 466.0 and the net head 30.5 feet. Ten thousand c.f.s., under 30.5 feet head at 75 per cent. over-all efficiency, would develop 26,000 h.p. This is the ordinary maximum amount of power which, for the present study, may be assumed available. Method of Regulation B, with 100 billion cubic feet of



Logging Dam at Outlet of Garden Lake.

storage on the Upper Rainy reservoirs, would, between 1892 and 1914, have furnished an average of 20,968 h.p. against 19,000 h.p. under Method A. For 25 per cent. of the time Method A would have furnished over 22,200 h.p., and, for 50 per cent. of the time, over 17,000 h.p. (See Fig. 1.)

During the protracted dry period, however, the power output under Method B would have fallen to an extreme low value, which is 4,000 h.p. less than the least amount of power which would have been available under Method A. If continuous power is required, therefore, the size of the auxiliary power plant, under Method B, would need to be 4,000 h.p. larger than under Method A. This additional capacity, however, might not be required for fifteen or twenty, or more, years to come, and could be installed when the protracted dry spell was at hand. Even if pro-

Dam on Kawishiwi River (with automatic gauge shelter showing not very clearly at left.)

vided immediately, however, this additional installation would represent an annual fixed charge of only about $35,000.

Between 1892 and 1914, Method B would have furnished an average of 1,968, or practically 2,000, horsepower more than Method A. The difference between the cost of operation of water power and steam power plants for such blocks of power as are under consideration, is not less than $25 per horse-power per year for 24-hour power, in favor of the water power, and hence it may be said that Method B is, annually, worth at least a net amount of $50,000, less $35,000, or $15,000, more to the power interests at Fort Frances and International Falls than Method A.

From the viewpoint of damage to riparian owners, on Rainy Lake, Method B would be more desirable than Method A, because for any given total storage capacity and high-water level, the average level of Rainy Lake, under Method B, would have been about two feet lower than under Method A. Moreover, under Method B the high-water level would have been reached, during the summer and fall months, in only a little more than half as many years, and would have prevailed about half as many months as under Method A.

Navigation interests on the Rainy River below the Lake, under the present state of the river—that is, with no dam at Long Sault Rapids—would, under all ordinary conditions, have found deeper water, but during a total of seven months, between May 1st and November 1st, out of the twenty-two years, the stage would have dropped lower under Method B than under Method A. If a dam be built at Long Sault Rapids, then navigation interests also would, unquestionably, be better served by Method B.

Granting, now, that regulation according to Method B* would permit of the most advantageous use of the waters of Rainy Lake—and we believe no successful combination of these two methods can be applied—it follows, as previously pointed out, that 100 billion cubic feet is about the maximum total storage capacity which need be provided in order to equalize the outflow from Rainy Lake to the greatest practicable extent.

The question of where this storage capacity can most advantageously be provided, is one which chiefly affects the interests on the upper watershed. With the completion of the Kettle Falls dams during 1914, a total of 100 billion cubic feet of storage capacity has already become available. Whether a part of this storage capacity shall be provided on other lakes beside Rainy Lake and the lakes controlled by the dam at Kettle Falls, is largely a question of the cost of storing water on the several available storage lakes. Storage on Rainy Lake is intimately related to available power at the outlet. Every foot of increase in lake level represents an increase of from 800 to 900 continuous horse-power. The cost of additional flowage, bank protection, etc., involved in the securing of further storage on this lake, would be at least partly offset by the increased power which would thereby become available. If such power as might be developed under low and greatly fluctuating head, at Kettle Falls is not considered, then storage on the lakes above would become primarily a question of the cost of flowage or other damage, and of impounding works, per unit of additional storage capacity secured.

In order to assist in determining the feasibility of utilizing auxiliary storage on the Upper Rainy watershed, surveys and reconnaissance examinations were made on

(Continued on page 38.)

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*Arthur V. White—in a hearing before the Commission—drew an analogy between the two proposed methods of regulation (Method A and Method B), and the possible financial policy of two banks. Bank A (or Method A) husbands its money so that it can maintain the same rate of loans over periods of acute financial stress, such as 1873-1878; Bank B (or Method B) does not figure on such extreme circumstances, but allows its money to go out uniformly in as large an amount as possible every year, without regard to what may occur during exceptionally lean years, trusting to make enough extra profit thereby to carry it very easily in comfort over the unusual periods.

DETERMINATION OF MAXIMUM STREAM-FLOW.*

By C. E. Grunsky.

THE formula discussed by me in a preceding article for determining the maximum storm water flow was originally devised for use in connection with urban and suburban areas. Such areas are usually of relatively small extent. The formula is equally applicable to the large watershed. However, no attempt has yet been made to determine what value should be assigned to the coefficient a, which appears in the formula, when the formula is used to estimate the maximum discharge from a large watershed, as in the case of a river.

Before entering upon a discussion of its applicability in determining maximum stream-flow the notation used in the previous article and the basic formula may be recalled. The equation numbering will be continued from the earlier article.

Let A = the area of the watershed in acres.
A' = the area of the watershed in square miles.
a = the coefficient of perviousness of the surface of the entire watershed. This coefficient remains the same regardless of whether the square mile or the acre be made the area unit.
a' = a variable depending on the surface characteristics of the watershed.
t = the critical time in minutes during the continuance of a rainstorm of maximum intensity, for the area under consideration, being that time within which the rain will produce maximum discharge.
T = the time in minutes that it will take water under maximum run-off conditions to flow from the most remote part of the watershed to the point at which the maximum stream-flow is to be determined.
v = the watershed run-off velocity—a function of slope, expressed in feet per second.
I = the maximum average rainfall intensity on entire watershed expressed in inches per hour, during any t minutes.
R = the maximum amount of rain, in inches, on entire watershed in any one hour.
C = the rain intensity constant.
q = the maximum stream-flow in cubic feet per second per acre.
q' = the maximum stream-flow in cubic feet per second per square mile.
Q = the maximum stream-flow, or the maximum rate of run-off expressed in cubic feet per second.

Fundamental equations (see the earlier article):

$$I = \frac{C}{\sqrt{t}} \qquad (1)$$

$$R = \frac{C}{7.75} \qquad (2)$$

$$q = \frac{5\,a\,R}{\sqrt{t}} \qquad (8)$$

$$q = 0.645\,a\,I \qquad (9)$$

When the formula is used to estimate maximum stream-flow it will be found convenient to use the square mile instead of the acre, as the unit area. In that event square miles may be written in the equations (6) and (7) of the earlier article, and for maximum stream-flow the formulæ (8) and (9) will become:

$$q' = \frac{3200\,a\,R}{\sqrt{t}} \qquad (10)$$

$$\text{and } q' = 413\,a\,I \qquad (11)$$

$$\text{from which } Q = \frac{3200\,a\,A'\,R}{\sqrt{t}} \qquad (12)$$

$$\text{and } Q = 413\,a\,A'\,I \qquad (13)$$

In applying this formula, or in attempting to establish a value for the variable a, care must be taken to determine properly the rain intensity for the entire watershed. The individual station record is not as dependable for a large watershed as for a small one. Whenever practicable the relation of the rainfall on the entire watershed to that of a single station or better to a combination of stations should be ascertained, by comparing the station records with the average rain on the whole area determined by isohyetose lines.

Even though no dependable information is at hand relating to the value of a which should be used when the formula is applied to large watersheds, there may be some speculation regarding the run-off which the formula indicates under certain assumptions that seem reasonable in the light of available run-off data.

It is evident that due to soil absorption and evaporation losses the value of a must decrease as the time increases during which water is flowing over the surface of the watershed and that for a small impervious area $a = 1$. This suggests the following expression for a:

$$a = \frac{60}{60 + c\sqrt{t}} \qquad (14)$$

where c is a coefficient to be determined from the surface conditions of the watershed.

The following values are suggested tentatively and remain subject to correction.

For impervious areas $c = 0.10$.
For mountainous areas $c = 2$.
For rolling country $c = 3$.
For flat country (ordinary soil) $c = 10$.
For sandy regions $c = 50$.

These values are suggested for application under ordinary conditions in temperate climates. They will no doubt be found too small in localities where the ground may be frozen, where the ground is water-logged or where the maximum run-off occurs when rain falls upon snow.

Table 1.

The value of the coefficient a in the formula for maximum stream-flow: $q' = 413\,a\,I$ or $q' = \frac{320\,a\,R}{\sqrt{t}}$ based on $a = \frac{60}{60 + c\sqrt{t}}$

Critical time, t Min.	Impervious areas $c = 0.10$	Mountains $c = 2$	Rolling country $c = 3$	Flat country $c = 10$	Sandy regions $c = 50$
5	1.00	0.93	0.90	0.73	0.35
10	1.00	0.90	0.87	0.66	0.27
20	0.99	0.87	0.82	0.57	0.21
30	0.99	0.85	0.79	0.52	0.18
60	0.99	0.79	0.72	0.44	0.13
120	0.98	0.73	0.65	0.35	0.099
180	0.98	0.69	0.60	0.31	0.082
600	0.96	0.55	0.45	0.20	0.049
1,440	0.94	0.45	0.35	0.14	0.028
7,200	0.88	0.26	0.18	0.066	0.013
14,400	0.83	0.20	0.14	0.048	0.009
28,800	0.78	0.15	0.11	0.034	0.007

*Supplement to "Determination of the Maximum Storm-Water Flow," published in "Western Engineering," Vol. V., p. 254.

Areas of various extent but of similar outline, or similar surface conditions, and of similar topographic features will have values of t proportional to $\sqrt{A'}$. Consequently $t = K \sqrt{A'}$

Here K is a coefficient which can be determined for any set of watersheds complying with the condition of similarity as here stated.

A number of similar mountainous watersheds may now be considered for which a value of $K = 20$ has been ascertained. This will make $t = 20 \sqrt{A'}$ and the formula for the maximum run-off rate from such similar areas will be

$$q'' = \frac{3{,}200\, a\, R}{\sqrt{20}\ \sqrt[4]{A'}}$$

$$\text{or } q'' = \frac{715\, a\, R}{\sqrt[4]{A'}}$$

$$\text{and } Q'' = 715\, a\, R\, (A')^{3/4}$$

Here, due to the mountainous condition, a will be determined by the expression $\frac{60}{60 + 2\sqrt{t}}$ or $\frac{60}{60 + 8.95\,(A')^{1/4}}$ and therefore:

$$q'' = \frac{42{,}900\, R}{60\,(A')^{1/4} + 8.95\,(A')^{1/2}} \text{ for the special case only.}$$

$$\text{and } Q = \frac{42{,}900\, R\,(A')^{3/4}}{60 + 8.95\,(A')^{1/4}} \text{ for the special case only.}$$

The following values result for the special case under consideration and areas ranging from 10 to 10,000 square miles:

Area.	t	q''	Q''
10 sq. miles	$t =$ 63 min.	$q'' = 317\,R$	$Q'' =$ 3,170 R
20 sq. miles	$t =$ 89 min.	$q'' = 258\,R$	$Q'' =$ 5,160 R
50 sq. miles	$t =$ 142 min.	$q'' = 191\,R$	$Q'' =$ 9,550 R
100 sq. miles	$t =$ 200 min.	$q'' = 153\,R$	$Q'' =$ 15,300 R
200 sq. miles	$t =$ 283 min.	$q'' = 122\,R$	$Q'' =$ 24,400 R
300 sq. miles	$t =$ 346 min.	$q'' = 105\,R$	$Q'' =$ 31,500 R
400 sq. miles	$t =$ 400 min.	$q'' = 96\,R$	$Q'' =$ 38,400 R
500 sq. miles	$t =$ 450 min.	$q'' = 89\,R$	$Q'' =$ 44,500 R
1,000 sq. miles	$t =$ 630 min.	$q'' = 69\,R$	$Q'' =$ 69,000 R
5,000 sq. miles	$t =$ 1,400 min.	$q'' = 38\,R$	$Q'' =$ 190,000 R
10,000 sq. miles	$t =$ 2,000 min.	$q'' = 29\,R$	$Q'' =$ 290,000 R

It is apparent from these values that in mountainous regions, all exposed to the same rainfall conditions, the rate of run-off per square mile as determined by the formula is eleven times greater from an area 10 square miles in extent than from an area of 10,000 square miles. In the light of the run-off information obtainable from the records of the U.S. Geological Survey and other sources, this relation appears quite reasonable.

The application of the formula can best be illustrated by recourse to a few examples:

1. What is the maximum discharge of a river draining a mountain watershed 1,900 square miles in area, in which the maximum amount of rain on the entire watershed in 24 hours is 8 ins. and for which t = 12 hours or 720 minutes?

Here

$$a = \frac{60}{60 + 2\sqrt{720}} = 0.52$$

From (1) generalized

$$C = \frac{8}{24}\sqrt{1440} = 12.7$$

From (1)

$$I = \frac{12.7}{\sqrt{720}} = 0.47$$

And from (13)

$$Q = 413 \times 0.52 \times 0.47 \times 1{,}900 = 192{,}000 \text{ second-feet.}$$

The conditions suggested in this example are comparable with those which prevail in the watershed of the American River, above Folsom, California, which, from a watershed of 1,900 square miles, at that point, has probably discharged as much as 180,000 to 200,000 second-feet (1861-62).

2. What is the maximum discharge of a river draining a watershed ¾ mountainous and ¼ rolling or foothill land, 9,000 square miles in area, in which the maximum amount of rain in two days' over the entire watershed is 6 ins. and for which t = 30 hours or 1,800 minutes?

Here the value of a will lie between the value determined for a mountainous area and that for a rolling country:

$$a = \frac{3}{4}\left(\frac{60}{60 + 2\sqrt{1{,}800}}\right) + \frac{1}{4}\left(\frac{60}{60 + 3\sqrt{1{,}800}}\right)$$

$$a = 0.39$$

From (1) generalized

$$C = \frac{6}{48}\sqrt{2{,}880} = 6.7$$

From (1)

$$I = \frac{6.7}{\sqrt{1{,}800}} = 0.158$$

And from (13)

$$Q = 413 \times 0.39 \times 0.158 \times 9{,}000 = 229{,}000 \text{ second-feet.}$$

The conditions suggested in this example are comparable with those which prevail in a watershed of the area of Sacramento River above Red Bluff, California, where the maximum recorded discharge has been about 250,000 second-feet.

3. What is the maximum discharge of a river draining a watershed of 250,000 square miles, equal parts of which are classed as mountain and as rolling country, in which the greatest amount of rain in one month over the entire watershed is 6 ins. and for which t = 20 days or 28,800 minutes.

Here

$$a = \frac{1}{2}\left(\frac{60}{60 + 2\sqrt{28{,}800}} + \frac{60}{60 + 3\sqrt{28{,}800}}\right)$$

$$a = 0.13$$

From (1) generalized

$$C = \frac{6}{720}\sqrt{43{,}200} = 1.73$$

$$I = \frac{1.73}{\sqrt{28{,}800}} = 0.0102$$

$$Q = 413 \times 0.13 \times 0.0102 \times 250{,}000 = 137{,}000 \text{ second-feet.}$$

This third illustration is typical of conditions which prevail in the watershed of Colorado River, which has a watershed area of about 225,000 square miles from which the run-off has at times reached 0.6 second-feet per square mile.

There is no need of extending illustrations to still larger watersheds because the necessary basic data are not available and because the values for the constants and coefficients here introduced are only suggestive. Enough has been said, however, to show that the type of formula is reasonable and that it will be found particularly useful in estimating from known conditions of the flow from one watershed what the maximum run-off or stream-flow from another will be in which the climatic conditions are similar, and also in estimating the maximum rate of run-off due to a rain of extreme intensity when the maximum rate due to a less intense, but yet heavy rain, has been ascertained by stream gauging. While a formula has been presented for determining the value of t from the value of T, equation (6) of the earlier article, this is intended to serve only for first approximation purposes. Because the value of t varies with the varying outline of the watershed and with its topography, it should be determined by direct estimate, whenever possible, for use in final computations.

When large portions of a watershed are lake or reservoir-surface, special consideration must be given to this fact, because the retention of water in storage basins reduces the maximum rate of outflow therefrom and reduces the discharge of the stream below the storage basin and may therefore have an effect upon the maximum stream-flow that should not be neglected. Recourse may have to be had to the mass curve in order to determine this effect.

PUMPING MACHINERY—TEST DUTY vs. OPERATING RESULTS.*

By J. N. Chester.

THIS paper is the result of an effort prompted by a request from our secretary starting with the statement that "we now install pumping engines of 200,000,000 foot-pounds guaranteed duty, but get much less every-day efficiency," and ending with the admonition that we "dwell on what the coal piles should give us in water pumped."

History relates that over 2,000 years ago, one Alexander the Great wept because there were no more worlds to conquer. This being the case, we are quite sure that Alec, up to that time at least, had never looked into the possibilities of steam plant efficiency.

From physics we learn that heat and energy, or work, are equivalent and mutually convertible one to the other. Our pump station results represent work. The source of the heat to perform this work is coal, and the means for converting the heat of the coal into work is our steam plant equipment. The percent. of the heat contained in the coal that may be realized in useful work or energy, represented by the water delivered into our mains, and why the losses is the problem before us.

The coal we buy contains, we will say, 100 per cent. of heat, not that coals do not differ in heat-producing or thermal values, but let us assume that the coal we have contains heat represented by 100 per cent., then to those familiar with the subject, it is well known that with the contrivances in use to this date, we have been able to make apparent in work done as represented by the water column delivered with the most efficient machines, less than 20 per cent. of the 100 per cent. of heat in the coal fired under our boilers and in many instances, the net result does not exceed 5 per cent. Why this loss?

In the boilers, due to the heat that must necessarily pass out through the stack so that the temperatures of the hot gases in the boilers may be in excess of the temperature of the steam generated and thus effective throughout their entire travel through the boiler, coupled with the radiation of the exterior parts of the boiler and the incomplete combustion of fuel represented by the combustible matter remaining in the ash, test boiler efficiencies are not obtained greatly in excess of 70 per cent., or otherwise stated, we realize only in net heat delivered as dry steam from the boiler, 70 per cent. of the total heat contained in the coal, which we termed 100 per cent.

*Summary of paper read before the American Water Works Association, June 7, 1916.

Next comes the steam lines and be they long or short, properly or poorly proportioned, bare or well covered, some radiation must take place, and some loss due to friction must be further incurred, for friction represents energy and energy is heat and so, under the best of conditions, we can expect to deliver less than 70 per cent. of heat of the coal converted into steam and less than 70 per cent. at the engine throttle.

Then, due to the ineffective utilization of the heat delivered at the throttle by the highest grade steam motors or engines in use to-day, which seldom exceeds 20 per cent., we find the net results left of our original 100 per cent. in the coal approximately 15 per cent.

As a concrete illustration of the above, recent final and preliminary duty and efficiency tests at Erie, Pennsylvania, showed that of the available heat units determined by analysis of the coal fired, the boilers delivered at their outlet nozzles 71 per cent., and of the available heat units received by the steam pipe at the discharge nozzle of the boilers it delivered to the throttle at the engine 99.6 per cent.; and that the engine converted into useful work but 21.5 per cent. of the available heat units delivered at its throttle by the steam pipe, or that the entire combination, not considering auxiliaries of any sort, showed an overall efficiency of 15.25 per cent., and still the engine on preliminary test (final test not yet run) performed a duty of 205,348,000 foot-pounds for each 1,000 pounds of steam consumed.

But probably, to be more readily understood by the waterworks operator, we would better restate this problem in the terms of the relations between test duties based on 1,000 pounds of dry steam and obtained on tests for acceptance as compared with every-day operating results per 100 pounds of coal fired under the boilers.

Before getting too deeply in either discussion, let it be known that we have accorded the field of plants consuming one million and less to the internal combustion engine and electric motor-driven pump.

For pumping units on which the demand is 1,000,000 gallons or more daily, practical test duties vary from 50,000,000 to 200,000,000 foot-pounds of useful work performed for each 1,000 pounds of dry steam delivered to such unit.

For the same machines, the station duty would be expected to vary from 20,000,000 to 125,000,000 foot-pounds of work done for each 100 pounds of coal fired under the boilers.

Why the difference between the maximum steam duty of 200,000,000 and the coal duty of 125,000,000? Also the minimum steam duty of 50,000,000 and the minimum coal duty of 20,000,000?

Pump station results, as most of us know, are affected by the following: (1) Quality of coal. (2) Efficiency of boilers. (3) Efficiency of steam lines. (4) Station capacity. (5) Head against which water is delivered. (6) Load factor. (7) Adaptability of machinery. (8) Compactness of station. (9) Low vacuum in condensing units. (10) Care in operation and many other minor elements.

For easy handling, let us take the case of a station in which the major unit is capable, when in service, of pumping all the water demanded and whose test duty was one hundred million foot-pounds per 1,000 pounds of dry steam delivered to it, and endeavor to make clear the differences between steam and coal duty.

Naturally, if each pound of coal fired under the boiler would evaporate into steam, at the pressure desired, ten pounds of water and none of this steam became condensed in the channels conveying it to the engine, then there would be no difference between coal duty and steam duty. Unfortunately, this is not the case.

In the average waterworks station with the best coal, Pennsylvania or West Virginia product, we are able to obtain but an average of about 8 pounds of water evaporated per pound of coal burned. Consequently, if nothing more than this existed, our steam duty of 100,000,000 would be reduced to 80,000,000 in station duty. Besides, we cannot all locate our stations within a practical shipping distance from the Pennsylvania or West Virginia coal fields, and those who obtain their coal from the Ohio districts must expect about 7 pounds of water per pound of coal, and from the Illinois and Indiana districts, in the neighborhood of 6 pounds of water per pound of coal, so that the latter has reduced the 100,000,000 duty to 60,000,000 duty without further inroads. So much for the quality of coal.

The above evaporative results are based on the ordinary boiler efficiency of about 65 per cent., which calls for intelligent firing, the boiler operating, within at least 80 per cent. of its rated capacity, grates that will prevent there remaining in the ash more than 5 per cent. combustible, a stack draft that will supply the coal being consumed with the necessary amount of air, boiler walls sufficiently tight to prevent the admission of air above and beyond the grates in quantities that will interfere with the best combustion, and the whole exterior of such a construction as to provide an insulation that will keep down the loss of heat by radiation to the minimum.

Unfortunately, the above are seldom found in the ordinary small waterworks plant in a combination that provides a boiler efficiency greatly in excess of 50 per cent., and efficiencies running as low as 40 per cent. are frequently found, so in the latter case, even when firing a coal from which a high evaporation might be expected, a much lower net result is obtained.

Much more may depend upon whether the steam lines are well proportioned, for if too large and the velocity sluggish, the condensation will be increased. If too small, a portion of the heat must be lost in friction. If bare, a large amount of useful energy escapes through radiation, and in 90 per cent. of our steam plants the covering has been done to no specifications other than the will of the company manufacturing and installing such commodities, wherein the covering generally stops short of flanges or fittings about 2 inches on each side, seldom fits the pipe tight, and in consequence, is but 50 per cent. as efficient as though it were continuous over fittings and provided against air circulating between the covering and the steam pipe. A 10 per cent. loss of energy between boilers and engines is not an uncommon one, when by proper care in design and construction it can be kept within 1 per cent.

The next great loss comes from a source which has been frequently referred to in the meetings of this association, and while many of the pump and boiler builders think it is a hobby, yet you may be assured it exists in 90 per cent. of our plants in stern reality. It is the varying demand and head and consequent load factor of the plant, and it is not a matter that either the designing engineer or the operating officials can correct. Metering intensifies it, so even when brought to the minimum, it still remains a large factor, causing inroads on economy; for though the water may be pumped once and then to a reservoir, a well-selected engine should, at the time it is purchased, exceed in capacity the demands to be made upon it, and although when operated the reservoir may permit it to deliver at its rated capacity while in operation and thus the maximum of economy from this standpoint obtained, it must, a portion of its time, be either shut down or operated below rated capacity, and so subject its economical possibilities to the inroads of the fixed thermal charges that continue whether the pump is in operation at full or half speed.

With 75 per cent. of our waterworks plants there exists no high level reservoir and, consequently, the delivery of the pump is governed by the demand on the mains, which varies throughout the twenty-four hours in the ratio of its extremes of about one to three.

Then again, in many plants having a capacity of less than 10,000,000 gallons daily, fire service is rendered by direct pressure, which calls for an increase when the alarm comes in of from 20 to sometimes 100 per cent., for all of which the boiler power must be kept in constant readiness, thus increasing grossly the fixed thermal losses necessarily to be charged against the station's efficiency.

When an engine is tested for acceptance and to prove the builder's guarantee of efficiency, we unfortunately too often set up in the purchaser's mind a standard which he hopes to but never in the future realizes, not only for the reasons above discussed, but others to follow, one of which is that during this duty test, we simply measure the steam used by the engine, neglecting, of course, that used by many of the auxiliaries such as the feed pump, the electric light engine now to be found in a majority of stations, sometimes the blowing engine that produces the draft, and frequently the air compressor that charges the chambers, the heating of the station and frequently adjacent buildings, all of which may represent a draft on the boilers from 10 to 100 per cent. of that of the main engine.

As an example of the above, extracts from an analysis made by a university professor and his class of a pumping plant in Pennsylvania where one pump in operation did all the work and was ordinarily run at full capacity, the following distribution of steam generated was ascertained:

Consumed by main engine	60.45%
Consumed by air pump	16.20%
Consumed by lighting unit	4.20%
Consumed by Holley vent to atmosphere	6.62%
Consumed by drains, heating coils, etc.	5.47%
Consumed by Holley return and engine jackets	7.06%
Total	100.00%

The boilers were fed from the pressure in the discharge mains.

The writer, some years ago, during an engine test at Jamestown, New York, measured the water into the boilers and from the discharge of the air pump of the main engine which, aside from the feed pump in the boiler room, and a steam jet employed to eject the condensate from the dry well was the only steam-consuming element in operation other than that steam was being supplied to the jackets of two idle engines in order to keep the station warm, the temperature outside being sixteen below zero, and the final results showed the water discharge from the air pump and jackets from the main engine was but a trifle over 50 per cent. of the total fed during the period of the test to the boilers.

In recent years, due to our constantly increasing population and consequent congested districts with modern sanitary methods, the majority of our water supplies, to be safe, must be filtered, and the installation of purification plants unfortunately generally necessitates the water being pumped from the source to the filters and again repumped to the mains whence the consumption is drawn. Notwithstanding this, that individual, to whom the man immediately responsible for the economy of this station must report, forgets this fact and is prone to demand, as a station duty, the test duty of but what may be termed the high service unit, ignoring all of the elements enumerated above and which must necessarily make inroads on the station's efficiency.

There is too great a tendency on the part of the station or operating engineer to neglect or disregard his means for removing the back pressure from the exhaust side of the low pressure piston in his engines, otherwise known as his condenser and air pump or vacuum producing facilities, for with even reciprocating engines, nothing less than 26 inches, as indicated by the mercury column, should be tolerated, while unexpected visits generally find in the neighborhood of 20 inches.

The loss due to low vacuum may be roughly reckoned in reciprocating engines at 1 per cent. for each inch of mercury. Steam turbines require, for economical operation, a much greater vacuum than 26, and while the condensing facilities may be nursed at the time of the test to produce the desired results, the greater is the fall when the lower vacuum of everyday results becomes effective.

Taking up the last of the major elements that tend to reduce plant efficiency, we come to what is too frequently found, to wit: the inadaptability of many of the steam-producing, pumping and auxiliary units due many times, to be sure, to changes that have taken place that are beyond the control of those responsible for their presence, but often, to the exercise of bad judgment, or none at all, in their original selection and installation, arising mainly from an unwillingness to incur expense of expert advice in the selection of same and a proneness to permit manufacturers to make the final recommendation.

The reference to manufacturers must not be construed by them as too great a reflection upon their ability to act in such capacity, but from the writer's own experience which reoccurs nearly every time a new station is to be built or an old one to be rebuilt, when, first, the individual with electric current to sell appears, and many times accompanied by another with an electrically driven pump to dispose of, and endeavors to convince him that electrically driven pumping machinery, regardless of conditions of service, is the only kind that will properly meet the situation; then comes the steam turbine-driven centrifugal advocate followed naturally by the builders of the several different types of reciprocating machinery, all of whom thoroughly believe and earnestly insist that what they have to offer is what the conditions at hand demand, and only a simpering idiot would select anything else, and so the individual, whose experience is limited to the operation of but one plant, too frequently falls a victim to the wiles of the over-zealous and inconsistent salesman.

Ten years ago, says the American Highway Association, there were only 48,000 registered motor cars of all sorts in the United States; last year there were 2,445,664. This is an increase of 500 per cent. per annum. There are approximately 2,275,000 miles of public roads in the United States, not counting streets in cities and towns, so that there is about one motor car for every mile of road.

The Hydro-Electric Power Commission have completed a deal with the bondholders of the Pine River Light and Power Company whereby all of the transmission lines, substation equipments and a portion of the apparatus at the development at Hornings Mills will be taken over and utilized as a portion of the Eugenia system. These lines will be used to serve Shelburne, Orangeville and Hornings Mills, and these municipalities are already being supplied with Hydro power by the amalgamation of the two systems and preparations are being made to serve the municipalities of Grand Valley and Arthur by using the portion of the line purchased from the Pine River Light and Power Company, and by constructing approximately 12 miles of new line.

DRAINAGE AND PREPARATION OF SUBGRADE.

At the second National Congress on Concrete Road Building the committee on drainage and preparation of subgrade presented the following report:—

It is practically impossible to formulate specific directions for the preparation and drainage of the subgrade, that will be of general application. Every piece of road construction involves problems that are complicated by local conditions, financial as well as physical, and that must be solved each by itself. The utmost that your committee can do is to consider ways and means of obtaining, as nearly as local circumstances will permit and at minimum expense, a dry foundation that will provide uniform bearing power for the pavement. It is agreed that moisture in the subgrade directly under the pavement is likely to be destructive to the surface in several ways, chiefly because of unequal vertical movements caused by frost action or unequal swelling or shrinkage of the soil due to lack of uniformity in drainage. It is also to be recognized that only quick drainage is effective for roadways. On the other hand, there is not unanimity of opinion as to whether, because perfect drainage is hard to secure, it is cheaper and better to reinforce the pavement sufficiently to withstand stress set up by frost action or similar movements, or to take additional precautions about drainage. Either expedient will involve additional expense; and which is better must be determined by study of the situation. The necessity, or at least the desirability, of spending more time and money upon thorough preliminary investigations cannot be too strongly emphasized. Such expenditure is in the end a real economy, too little appreciated; and parsimony at the inception of a project often results in needlessly inferior location as to soil and drainage, as well as to line and grade, with consequent increased cost of construction or permanently poorer roads and increased cost of maintenance.

Clearing and Grubbing.—All trees and undergrowth should be cleared from the area within the slope-stakes, and from such additional space on both sides of the road as may be necessary to allow a clear view for traffic in both directions; to allow for ditches and other provisions for drainage; and to provide for the proper construction of the road, and for its permanency and its maintenance. And all such material should be destroyed or otherwise removed from the right-of-way. A general clearing of trees is often unnecessary or objectionable; and the value of trees and even bushes, from the standpoint of beauty, comfort to travellers, prevention of soil-wash, and protection to the roadway, should be more generally recognized. Intelligent trimming of the lower branches will in many cases serve every purpose that can be accomplished by cutting down a tree. Trees should not be carelessly or thoughtlessly sacrificed. Under shallow embankments even long grass or a thin layer of decayed vegetable matter is in general objectionable, because preventing adhesion of new material or causing soft spots; and such matter should not be permitted at or near the surface of the finished subgrade.

The roadway should be grubbed to the full width of excavation; and roots and stumps should be removed from drainage ditches and wherever they will interfere with underdrains, and under embankments so shallow that the decay of the roots and stumps will ultimately cause unequal settlement of the fills. In case their removal is not necessary, stumps under fills should be cut off within one foot of the ground, elsewhere within three feet. It is customary to stipulate some limiting height of embankment such as 5 feet, which makes grubbing unnecessary.

The committee submits that such a limit is undesirable as a minimum; much will depend on the number of stumps, their size, the kind of soil and other local conditions; every precaution should be taken to prevent unequal settlement and so simple an expedient as extra grubbing should not be neglected.

Provisions in the contract and specifications to allow payment for clearing and grubbing by the tree or stump, or some small unit, instead of by the acre or square, should in many cases lead to better treatment of these matters.

Excavation and Embankment.—Four types of cross-section for the subgrade are in use: (1) Subgrade crowned parallel to the concrete wearing surface, which is made of uniform thickness. (2) Subgrade crowned somewhat less than the concrete, which is made thicker at the centre than at the sides. (3) Flat subgrade, all the crown being given by varying the thickness of the concrete. (4) Dished subgrade. The opinion seems to be gaining ground that the last-mentioned form, the slab thickened at the centre and built on a flat or dished sub-base, is best suited to resist cracking, as well as to withstand heavy loads. Whatever form is to be used, and whether the surface is finished smooth or rough, the sub-base should be made of uniform texture, so to speak, should be finished true to the specified outline, and should be kept free from holes, tracks, and ruts until the concrete is laid.

The side slopes to be given to cuts and fills should be determined by studying old railroad or highway embankments in the immediate locality; and the grading should be finished to conform accurately and neatly as to line, gradient, and side slopes, with the specifications. In general, the fills should be made from the cuts and the material excavated from drainage ditches; it may, however, be cheaper to borrow dirt than to handle it from the ditch excavation. But the dirt taken from ditches must be so placed that it cannot wash into and refill the ditches or interfere with drainage. Borrow pits must be thoroughly drained. In case the excavated material exceeds the requirements for fills, provision should be made in the contract and specifications that the excess should be used to widen the embankments; the higher ones, being more dangerous for traffic, should first be widened. Side slopes of cuts and fills should be seeded with some grass suited to the climate and soil.

After the removal or destruction of the perishable matter, the surface under new embankments should be roughened wherever the new material is likely to slide; and on the sides of old embankments, or side-hill slopes, well-defined steps or benches should be made. New earth should be deposited and rolled in layers not exceeding 1 foot in thickness, and the thickness should be varied according to the kind of material available and the type of roller to be used. In some cases the layers should be as thin as 1 or 2 inches. In general, a self-propelled roller weighing 10 tons or more will compact the subgrade cheaply and effectively. For some kinds of soil so heavy a roller is not suitable, and better results can be obtained from lighter machines; and in other materials better results can be attained by the use of grooved rollers or tamping rollers, because the wide, smooth rollers, even when heavy, span over the soft spots. During rolling, the embankment should be kept slightly moistened. The quality of the material to be used in embankments should be considered carefully; and specifications should make definite provision to permit the engineer to reject unsuitable material either from cuts or borrow pits. Your committee suggests that while experience in one locality will enable one to judge of the merits of the material available with more or less accuracy, there are no definite tests, so far as we know, by which the merits of different earths may be predetermined. This seems to offer a profitable field for scientific investigation. In general, it is not desirable to build a concrete surface on any but the shallowest fills, even if the work is thoroughly rolled, until the earth has settled for at least one season.

If the subgrade is an old road surface of gravel, broken stone, or earth, it is likely to be uneven and far from uniform as to bearing power. Filling the low places with suitable material, and rolling, will sometimes suffice to make the old surface into a good subgrade. More often, however, this is not sufficient; and the only satisfactory way is to loosen the old material by scarifying, spading, or plowing, then harrow or cultivate, and reroll it. In some cases, such material should be removed, respread in very thin layers and rolled until uniform and hard.

The subgrade in cuts may be shaped by any suitable method; but unless the soil is of a kind that unquestionably will not compact, it should always be rolled until uniform and hard. Any soft or low places developed by rolling should be filled with suitable soil, the undesirable material being removed, and the surface rerolled as may be necessary. In some instances, the desired necessary uniformity of surface can be attained only by loosening and cultivating the soil to a depth of 6 or 8 inches and rolling; and in other cases this loosened material should be removed, respread, and rolled in thin layers of 1 or 2 inches. Certain clay soils when wet can be rolled only if covered with a layer of sand, gravel, or dry loose earth. In some instances straw or similar material may be used to facilitate rolling. Drainage trenches, if placed under the subgrade, should be completed before the rolling is done. In back-filling trenches the material should be thoroughly tamped in thin layers; and if the trenches are refilled with broken stone or screened gravel, this should be covered with a few inches of sand, loam, fine gravel, or material taken from the immediate excavation, in order to provide a uniform bearing surface and prevent mortar from the concrete from leaking down among the loose stones.

Excavations for drainage should be definitely and carefully arranged for in the specifications, and executed with as much care as any other part of the work.

Drainage.—Road drainage in general involves the disposal of (1) surface water flowing to the road from outside areas; and (2) rain falling on the road itself; and often (3) underground water from outside areas. Problems of road drainage should be approached with the fundamental idea that to be effective in protecting the road surface a drainage system must be capable of acting quickly. Underdrainage is of necessity slow drainage as compared with surface drainage; and every reasonable provision should be made to dispose of surface water coming to the roadway by surface drainage, and to keep it out of the foundation. The conditions imposed by the location often make it necessary that surface water shall be handled by underground channels or even underdrains; but in such cases the underdrains should be proportioned for storm-water capacity and not as for underdrainage alone.

The most obvious and effective way to dispose of surface water coming from outside the road is by means of surface ditches dug outside the top of cuts or along the toes of fills, on one or both sides, as may be necessary. Such ditches protect the side slopes and roadway from wash, diminish the amount of water to be carried by the road gutters, and thus increase their stability, and keep the foundation drier than if all the water coming to the road has to be handled in the gutters. The destructive results of depending upon the road gutters to handle all the surface water were forcibly demonstrated in the regions which were visited by the heavy rainfalls of the summer of 1915. In many cases the road gutters must carry all the water from outside as well as water drained to them from the road surface, and such a condition should be recognized in designing. Ditches and gutters should be of liberal capacity. They should have outlets into culverts or streams at such frequent intervals that the volume of flow will not accumulate to destructive proportions in the channels themselves, nor cause damage when discharged on adjacent land. Gutters immediately adjacent to the travelled surface should be broad and shallow, rather than narrow and deep, because flat ditches are easier to maintain, and are not dangerous to traffic. In general, the proportioning of ditches and gutters may be done successfully without resort to a detailed study of rainfall, run-off, or the hydraulics of flow in channels; but in important or unusual cases and in proportioning culverts such a study should be made.

Your committee has assumed that the design of culverts and storm-water systems for urban streets is too detailed a matter to discuss in this kind of a paper.

Rainfall on the roadway should be disposed of by surface drainage as far as possible; and the road surface, shoulders and gutters should be shaped and built with this end in view, and to minimize seepage into the foundation. If the entire roadway between curbs is covered with dense concrete, well-surfaced and sufficiently crowned, and if the road is adequately provided with catch basins and culverts, water falling in the road should give no trouble. But if the concrete is more or less absorbent, and its surface is irregular, or leaky, or if car-tracks are laid in the street, unless the foundation is naturally dry and sufficiently open to carry off such water as may leak through, some definite provision for handling this water should be made. A layer of coarse sand, cinders, gravel, or broken stone, or shallow blind drains, 10 inches to 12 inches deep, at intervals of from 25 to 50 feet, and draining downhill at an angle with the centre line, either into the side drains or gutters, will usually serve to take care of such seepage water. Unless there is ground water in the roadway such a condition should in general be obviated by good construction.

If the roadbed is in finely divided, retentive, water-bearing soil, some system of underdrainage should be devised that will permanently lower the water table beneath the concrete, and keep the subgrade uniformly dry for a sufficient depth to eliminate the destructive movements caused by freezing and thawing or resulting from alternate saturation and drying of the soil. Finely divided material is in itself slow-draining, and other things being equal, the finer the material the more readily will water rise by capillarity. Underdrains in such soil necessarily operate slowly, and the work required of them should be diminished as much as possible by other expedients. The direction of the underflow should be ascertained, and the drainage ditches outside the roadway should be deepened, if feasible, to divert as much ground water as possible into the nearest culvert or crossdrain. A special ditch for ground water between the surface ditch and the roadway is often desirable; and localized underflow of considerable volume, such as springs, should be provided with direct and ample channels.

The kind of soil, the lay of the land, the amount of water, and other local conditions such as the availability of suitable materials, will determine the method of underdrainage to be employed. The choice would in general be from among the following: (*a*) Pipes of vitrified clay or cement concrete laid in trenches, backfilled with the material excavated from the trenches, that is, "farm drains." (*b*) Pipes laid in trenches backfilled with broken stone, gravel, cinders, or similar open material. (*c*) Trenches filled with stone or gravel, without pipes. "blind drains." (*d*) Layers of cinders, gravel, broken stone or similar material varying in thickness from a few inches to 12 or 15 inches, under the entire road surface. (*e*) A layer of similar material laid in a shallow longitudinal trench with the cross-section of a flat letter V, and known as "V" drains. (*f*) Two or more of these methods used in combination.

Of the three types of trench drains, a pipe covered with coarse gravel or similar filling, for the whole depth of the trench, is the most effective. It will act more quickly than the farm drain, or the blind drain; has more capacity, other things equal; and it serves quickly to take care of that part of the surface water that in most cases eventually gets into the ground.

Pipe drains smaller than 4 inches in diameter should not be used, and a larger size is better. Drains should be given as much fall as feasible to facilitate the removal of silt and to increase their capacity. Water will flow on very flat grades, but more slowly, and in general drains laid on a flat grade will not lower the ground water as much, nor drain as large an area as on steeper slopes. The depth of drains is an important factor in determining the area drained, and to a certain extent the uniformity of drainage. In very fine soils the slope of the ground water table toward the drains must be very steep, in order to carry off the necessary amount of water, often as steep as 1 in 10 and occasionally steeper. Drains should be laid at least 2½ to 3 feet below the subgrade, and 4 feet is oftener better. Unless the drain is laid to sufficient depth it will not lower the water sufficiently under the entire roadway; but rather than increase the depth much over 4 feet, it is usually better to lay additional underdrains.

The location of drains in trenches should be such as will most effectively cut off the ground water and keep it from rising into the top of subgrade. In general, that will be one line laid parallel to the centre line of the road on the side from which the underflow comes, or one line on each side; or a succession of transverse lines laid at a sloping angle across the road. There is much well-founded diversity of opinion as to the relative merits of the two types. It may fairly be said that cross trenches are likely to form weak spots under the concrete, owing to the difficulty of getting them properly backfilled, and they should, if possible, be placed outside. It is desirable to lay underdrains as long as possible before building the road surface, especially if the concrete is to be laid late in the autumn. In many instances a layer of sand, gravel, stone, or cinders has been found to work more satisfactorily than drains in trenches. The committee believes that in very wet foundations the most effective method is a combination of two of the above types, such as pipe laid in trenches 3 feet deep, more or less, at each side of the road and draining into catch basins or culverts in city streets at or under the curb, otherwise in the shoulder, just outside the ends of the concrete, the trench to be backfilled with coarse gravel, cinders or broken stone. In addition, the whole subgrade under the concrete should be covered with a layer of similar material or sand, which makes with the trench filling a continuous drain as well as an insulation for the concrete. Where such loose material is not available, lateral drains laid at depths of 12 to 18 inches, at intervals of 20 to 40 feet across the roadbed from the side drains and draining into them, may serve the same end. The problem is to keep the subgrade dry by the use of such resources and material as are available. But if under adverse conditions this is likely to be impossible, the fact should be recognized in advance and provided for in the design of the concrete slab.

DIFFICULTIES IN THE DESIGNING AND OPERATION OF MEDIUM SIZED WATERWORKS PLANTS.*

By E. B. Black.

IN the early history of the development of the business of furnishing cities with an adequate supply of pure and wholesome water, the cost of such service was necessarily large, as is to be expected in the development of any commercial enterprise, so only those cities of sufficient size, or of such peculiar location that a fair return seemed possible on a considerable investment, could interest private capital in the building of waterworks systems.

The demand for the many advantages a modern water system brings a city comes as a result of education. Gradually the small cities and towns began to demand such improvements, and when private capital could not be interested on account of the limited revenue possible from a small community, special legislation allowed the building of systems by these small cities themselves. It seems now that a town of from 1,000 to 5,000 inhabitants can build a waterworks system, in some sections of the country at least, with as little trouble in financing the proposition as was formerly experienced in towns of 50,000 or 100,000.

Perhaps no other state has made possible the building of public utilities in the same way that Kansas has. With an allowable indebtedness of 15 per cent. of the assessed valuation, and with the assessed valuation taken as the full value of property, it has been an easy matter to vote bonds for the purchase of existing systems or the building of new ones. Three years ago an emergency law was enacted by the state which allowed all cities and towns already owning their waterworks systems to issue bonds for necessary improvements until the total indebtedness of the municipality should reach 15 per cent. These bonds may be issued without vote of the people, the only requirement of the law being the approval of detailed plans, specifications and estimates of cost by the State Utilities Board prior to the issuance of the bonds.

All these things have worked together to stimulate the building of new systems in the small cities and towns of Kansas, and the building of needed improvements in those towns already owning their systems, until now there is a total of about two hundred and fifteen water systems in the state, and about 90 per cent. of the total number are in towns ranging from 500 to 4,000 inhabitants. It may be of interest also to note that not more than a dozen of these systems are privately owned. While conditions in Kansas may have been more favorable than elsewhere in the past for the building of small water systems, such improvements have by no means been confined to that state, and the small cities and towns throughout the country, realizing the benefits coming from the establishment of such systems, have been actively shaping legislation so they also may have water systems.

The design and operation of systems in towns of small size present difficulties foreign to the design and operation of large plants. In designing, the services of an engineer are often dispensed with, and practically never is an engineer retained to advise relative to the operation of the system. This may be on account of the size of the proposition and the desire to put every dollar available into the improvement. Then again, the design may be handled by a concern manufacturing or selling equipment. Admitting the fact that such concerns may have the best engineering talent available, it is not reasonable to assume that other concerns will feel free to bid on their competitor's design; or that the proposal of the designer, if accepted without competition, will be either an efficient or economical one. Many small systems have been built by the contracting engineer. The design and the construction both frequently suffer in such cases. It is difficult for a man to design a system, bid on it with other bidders, and then construct under his own plans, specifications and supervision.

In some systems the poor ideas of the owner or city officials are followed; this is sometimes the case even when an engineer is retained. In going over a sewage disposal proposition recently, plans of the city's sewers were furnished. The engineer who had planned the sewers had evidently suffered from suggestions of various officials; for on one sheet of the profiles was this note:

> The number and location of Wye branches and manholes on this sewer were fixed by the city clerk and the plan does not therefore represent my ideas in the matter. Please refer to my letter of .... date in this connection.

A short time ago a city retained engineers to design a storage reservoir to be used in connection with one already in use. The bottoms of both reservoirs were to be on the same elevation only a few feet from wall to wall, and the water was to be carried in both at the same levels. The superintendent asked that a syphon be used to get the water from one reservoir to the other. He had never built a syphon, here was his opportunity. This reason is probably responsible for more mistakes in construction than any other.

Some of you are familiar with a plant in which the discharge line from the pumps is carried up the outside and over the top of the standpipe supplying the town. This is a scheme evolved by one of the plant engineers, which enables the pumps to operate against a column of water only 10 inches in diameter instead of one 30 feet in diameter, the size of the standpipe. You may have heard of the gravity flow line, the size of which was increased by sections from the upper to the lower end in order to overcome friction..

Some years ago the writer made an appraisal of a privately owned power plant for a town contemplating its purchase. The plant originally consisted of a generator, clutch-connected to a line shaft driven by a water wheel. The time came when the water power was insufficient, so the line shaft was extended and two large gasoline engines belted to it from a single clutch pulley between them; so connected that either or both engines could help the water-wheel. But even then the power was insufficient at times, and at the time the plant was examined a traction engine was belted to the line shaft through a hole in the wall of the power house. The owner was even then looking for more power, and wondering why the plant did not operate satisfactorily. Some one had advised him that all well-regulated plants kept records of ammeter and voltmeter readings. These had therefore been faithfully recorded

*American Water Works Journal, June, 1916.

each thirty minutes of the plant's operation; but the current generated was three-phase and the readings had been taken from one ammeter and one voltmeter connected to one phase only.

These instances are not exceptions in the average small plant; indeed, many more serious mistakes are known to every plant operator. Water systems with no water supply, filtration plants that fail to filter, and many other such mistakes are matters of common knowledge. Practically all of these things can be traced back to a desire to save a few dollars of the original investment, at the expense of comparatively greater cost of operation and maintenance. Even in the power plant, that part of the system where efficiency might be expected, and in plants where tests are made on the equipment at the time of its installation, provision is seldom made so the superintendent can run an overall efficiency test at any time, and the guaranteed efficiencies under which simple equipment is often purchased are frequently impossible of determination by the superintendent, after the plant has been turned over to him. Power plants are installed with separate guarantees on engines, boilers and generators; motor-driven pumps with separate guarantees on motors and pumps. In the small plant and in many large ones, an "overall" efficiency guarantee is possible, and it is the only guarantee for which the superintendent has a great deal of use.

Efficient equipment in a plant counts for little unless you know that that equipment is maintaining its efficiency through efficient operation.

The man never lived who thoroughly enjoyed writing reports and weighing coal, or even changing record cards on recording instruments, but no plant can operate successfully and economically without an accurate and detailed record of its operation. In the small plant, recording instruments and labor-saving devices, or even proper valves and meters are sometimes entirely lacking. How can an operator know what his plant is doing when he only knows how much coal he uses and how much money he takes in each month? Nearly all owners object to the size of the coal bill or to the amount paid out for chemicals for the filtration plant, but very few offer their superintendents expert assistance in cutting down the size of these bills. Business management does not entirely consist in keeping the operating cost within the receipts.

Two years ago the writer visited a privately owned waterworks plant, and inquired of the engineer in charge how much alum he was using, and how often he analyzed the water. His answer was that he used from three to six grains and that in the two years he had been connected with the plant no analyses had been made.

Perhaps one of the hardest problems for satisfactory solution from the standpoint of results, is the operation of filtration plants of small size, for practically every plant of 5,000,000 gallons or less per day has to be operated intermittently, thus presenting added difficulties over the operation of a large plant. Obviously more attention should be given to the design and operation of such plants. It is certainly of prime importance that this phase of the question should be given study it has not been given in the past.

The large waterworks system is not the only one entitled to efficient operation, to dividends and to the enthusiastic support of its patrons. Perhaps the system using the extra line to the top of the standpipe had other much more serious defects in its construction, and with these corrected, it might have operated at a profit instead of a loss. In all probability the owner of the system, the superintendent of which increased the size of the lower sections of the flow line to avoid friction, wondered why his investment was greater than necessary to accomplish a given result. The owner of the filtration plant where no analyses had been made for at least two years probably considered himself abused when patrons objected to the quality of water furnished.

There are good and sufficient reasons for about 99 per cent. of the defects of design and operation of the small waterworks system, and the successful owner or operator is the one who removes these defects by careful and intelligent design and operation. There is little real excuse to-day for poor design or unintelligent operation of the small sized waterworks systems. The various state boards of health have done, and are doing, a remarkably efficient work in the education of the owners and operators of plants from these standpoints, and associations such as the American Waterworks Association, through publications and conventions, make it possible for every waterworks superintendent to get ideas adaptable to the proper solution of the problems of his own plant, and surely no engineer, owner, or superintendent is worthy of his position who fails in demanding the best design and best results of operation for the small as well as for the large system.

AMHERSTBERG, ONT., SEWAGE DISPOSAL.*

THE city of Amherstberg, Ont., is the farthermost downstream municipality on the Detroit River. It possesses no manufacturing establishments of consequence and has had an erratic trend as to growth in past years. The population was 1,936 in 1871, 2,672 in 1881, 2,279 in 1891, 2,222 in 1901, and 2,560 in 1911. There is no apparent reason to expect that it will grow rapidly in the future, and it has been assumed that interceptor provision for 4,000 people and treatment-plant provision for 3,000 people will be ample.

No regular records are kept of the water pumped, but from a statement of the pumping-station engineer as to the number of times the water tank is filled during a day, it appears that the daily per capita consumption is about 165 to 170 U.S. gallons. As this information is of meagre nature, and as Amherstberg is a very old city, so that the water mains are probably in a very leaky condition, the interceptor has been designed on an assumed water consumption of 125 U.S. gallons per capita daily, with a sewage contribution therefrom of 100 U.S. gallons per capita per day, to which has been added a ground-water allowance of 1,000 U.S. gallons per acre per day, and a further allowance of 100 U.S. gallons per capita per day to cover drainage which enters the existing sewers from the rural districts back of the city.

The estimated cost of the project is shown in Table 1, on the next page.

The existing sewerage system has two outlets, one in Richmond Street and the other in Park Street. There is a suitable site for a treatment plant just south of Park Street, which permits the use of a short interceptor line. A short outfall from the proposed treatment plant to the Detroit River will be required.

It has been estimated that the pumping station should operate automatically by means of electric power, and on

*Abstract from report of H. C. McRae, district engineer, to Prof. Phelps, consulting sanitary engineer to the International Joint Commission.

this basis the cost of operation, including labor and materials, has been estimated at $1,300 per year.

Table 1.—Cost of Sewage Disposal Plant for Amherstberg, Ont.

Item.	Unit Price.	Cost.	Total Cost.
1,240 feet of 10-inch sewer	* $2.00	$2,480	
580 feet of 12-inch sewer	* 2.20	1,276	
700 feet of outfall sewer	* 2 00	1,400	
Diversion chambers and tide gates	....	600	$5,756
Pumping station:			
Buildings and pump well	....	2,000	
Machinery and piping	....	2,300	4,300
Treatment plant, including Imhoff tank, sludge bed, grit chamber, and disinfection apparatus	† 2.25	....	6,750
Land	....	....	250
Total	....	....	17,056

* Per foot. † Per capita.

Table 2 shows the annual charge which results from the estimate given in Table 1.

Table 2.—First Cost and Annual Charges of Sewage Disposal Plant, for Amherstberg, Ont.

	Interceptors and other structures.	Machinery.	Land.	Total.
First cost	$14,506	$2,300	$250	$17,056.00
Interest at 4½ per cent	653	104	11	768.00
Amortization	* 81	† 230	....	311.00
Fixed charges	734	334	11	1,079.00
Operation charge	....	....	....	1,300.00
Total annual charge	....	....	....	2,379.00
Annual charge per capita	....	....	....	.79

* 50-year life. † 10 per cent. allowance for amortization and repairs.

TWO METHODS OF REGULATING RAINY LAKE.

(Continued from page 28.)

Rainy Lake and on Lakes Namakan, Kabetogama, Sand Point, Crane, and Little Vermilion. Reconnaisance examination was also made of other secondary storage lakes on the watershed.

As has been pointed out, the storage capacity required for the greatest practical equalization of flow from the viewpoint of the regulation of the levels of the Lake of the Woods is already available. With respect, however, to the relative advantages of variously storing water on the other lakes of the Upper Rainy watershed, it may be said that fuller detailed study will embrace this phase of the subject.

The results which would have been obtained during the years 1892 to 1914, from the two methods of regulation of the Upper Rainy reservoirs, as previously discussed, and with either 100 or 150 billion cubic feet of total available storage capacity, may briefly be summarized as follows:—

1. From the standpoint of the power interests at International Falls and Fort Frances, method of regulation B would have resulted in an average of 1,968 more horse-power being available than under Method A.

2. An aggregate storage capacity of 100 billion cubic feet is all that need be provided on the Upper Rainy watershed if these reservoirs are to be regulated according to method of regulation B.

3. Under method of regulation A, with 100 billion cubic feet of available storage capacity on Rainy Lake and the lakes above Kettle Falls, Rainy Lake would have been full more than 85 per cent. of the time, whereas under method of regulation B it would have been full only about 40 per cent. of the time. These facts may be seen by referring to the frequency curves, Fig. 1.

4. From the viewpoint of damage to riparian owners on Rainy Lake, method of regulation B would have been more advantageous because the average level of the lake would have been about two feet lower and the high level would have prevailed only half as long.

5. The following rates of outflow would have prevailed:—

	Minimum Outflow	Maximum Outflow	Average Utilizable Outflow
UNDER METHOD A	c.f.s.	c.f.s	c.f.s.
With 100 billion cu. ft. storage	5835	31241	7614
With 150 billion cu. ft. storage	6710	28237	7883
UNDER METHOD B			
With 100 billion cu. ft. storage	3500	27651	9029
With 150 billion cu. ft. storage	4230	27651	9276

6. Under Method B, throughout all seasons but one in twenty-two years, a greater ordinary flow would have been available between May 1st and October 1st, for navigation on the Rainy River below the International Falls-Fort Frances dam than under Method A. After July 1st, during that one year, the flow would have fallen to about 3,500 c.f.s., which is less than is required for satisfactory navigation of the river.

7. For about nine months during the extreme low-water period, 4,000 less horse-power would have been available under Method B than under Method A; that is to say, if continuous power is needed, the capacity of the auxiliary power plant under Method B would require to be 4,000 horse-power larger than under Method A.

[NOTE:—The third of this series of articles covering the report made by Engineers White and Meyer will appear in the next issue of *The Canadian Engineer*, and will deal with the regulation of levels and outflow of the Lake of the Woods and with the outflow capacity required under various methods of regulation.—EDITOR.]

NOVA SCOTIA STEEL NEW CATALOGUE.

Beautifully bound in leather and cloth, and printed throughout in two colors on coated paper, the new catalogue of the Nova Scotia Steel & Coal Co. splendidly illustrates the company's plant and products. There are 108 pages, 10" x 11¾" in size, each page containing a 7" x 9" photograph, together with some text descriptive of same. Exterior and interior views of the company's plant at New Glasgow are shown, and also of its allied concern, the Eastern Car Co., Limited. "Scotia," as the steel company is more familiarly called, has become one of the largest industries in Canada, and a world factor in the production of iron ore and of iron and steel.

Editorial

GEORGIAN BAY CANAL REPORT.

To help to decide whether or not it would pay Canada to spend $125,000,000 upon the construction of the Georgian Bay Canal, a royal commission was appointed by the government in 1914 to report upon the commercial feasibility of such a canal. It will be recalled that under the authority of parliament, a survey and investigation of the practicability and probable cost of a deep waterway from Georgian Bay to the harbor of Montreal, by way of the French and Ottawa Rivers, was initiated in 1904 under a board of engineers. In 1909, this board submitted a report (Georgian Bay Ship Canal report upon survey, with plans and estimates of a cost, 1908), the plans providing for a waterway 22 feet deep, with a length of 440 miles, in which there would be 28 miles of canal excavation, 66 miles of channel dredging, and 346 miles of river and lake; with 27 locks of a minimum length of 650 feet, with 65 feet clear width and 22 feet clear depth, the lift ranging from 5 feet to 50 feet; and with a minimum water supply in the summit basin, capable of being increased, which would permit of 20 lockages per day throughout a season of about 210 days. The cost, originally placed at $100,000,000, was, in view of increases in the cost of materials, subsequently estimated at $125,000,000.

The engineers having reported that the canal was practicable, it now remains for the Georgian Bay Canal Commission to recommend whether or not it would prove a profitable undertaking to the nation. In the interim report of the commission, just published under the signature of W. Sanford Evans, chairman, certain economic phases are discussed. No hint, however, is given as to what recommendations the commission ultimately will make. It is an interim report and only that. Replete with information, statistics and guarded suggestions, it gives little inkling as to whether the commissioners will recommend the government to proceed with the construction of the canal. Probably it has been issued in that way because of several important considerations. The first is the fact that the financing and conduct of the war at present constitute the chief business of the country. The second is the railroad situation. The government are selecting an expert commission to consider the position and they will report as to the best solution of the railroad problem. That may involve the nationalization of the railroads or important changes relieving the government of some of their financial and other burdens in connection with two of the railroads.

To some extent, upon the ultimate solution of the railway question depends the construction of the canal. If the railways are nationalized, will the nation need the Georgian Bay Canal as a supplementary transportation factor? If the railroads continue to be operated by private capital, will the canal be needed to help regulate freight rates? These questions in turn will depend upon the conclusions of the Georgian Bay Canal Commission as to whether it would pay the country to spend a large sum of money upon the new undertaking.

Without any reflection upon the commissioners, we believe that they would find little difficulty in accumulating sufficient evidence to come to a conclusion either for or against. After the war is over and the railway question is settled, the decision as to whether or not the canal is to be constructed, will depend in no small way upon the prevailing political atmosphere and the general conditions which will arrive after the war. Reading between the lines of the report we are inclined to believe that the commissioners are, to say the least, not strongly biased against the construction of the big waterway.

THE SAFETY FACTOR IN HIGHWAY DESIGN.

In surveying routes for highways, engineers should pay particular attention to public safety. As the American Highway Association pointed out in a recent bulletin, speed fiends and drunken drivers are already attended to by laws, but there are many very real dangers which receive little or no attention. One of these is the junction of a road with another at right angles, which is concealed by an intervening rise or curve, so that the junction point is not seen until just before the driver on the adjoining road must turn into the main road. Such places are extremely dangerous and should be eliminated to the greatest extent possible in the design of the road.

The American Highway Association suggests that sign-posts should be erected to warn the traveller of the proximity of a danger of this sort. Sign-posts are not always noticed, however, particularly at night, and the highway engineer should strike deeper at the root of the trouble, and eradicate it entirely if possible within reasonable expenditure, instead of merely posting a sign stating that there is a pitfall beyond.

There is danger also from the road intersection where thick shrubbery or trees make it impossible for the driver on one street to see an approaching vehicle on another until the two nearly collide. Slow driving is of little avail in such places. The only remedy is to clear away the obstructions to sight, and this should be required by local regulations. The same observation applied to the shrubbery on the inner side of sharp curves. The underbrush should be cleared away in such places so that a driver can see another vehicle before it is nearer than at least 75 feet. This does not necessitate the destruction of the shrubbery or trees, but merely a thinning out of the growth.

MONTREAL CONTROLLERS REFUSE TO PERMIT AQUEDUCT INVESTIGATION.

Despite the request made by the Montreal city council that members of the Canadian Society of Civil Engineers be asked to report gratuitously upon the aqueduct enlargement, and despite the fact that prominent Montreal engineers had so offered to report, the Board of Control has again refused to allow such an investigation to be made. One of the controllers stated that the only difficulty was that the Board were unable to decide unanimously just who would make the investigation. But whatever may be the reason, the fact remains that they have again refused to permit the investigation.

Meanwhile, Controller Villeneuve has issued a lengthy report upon the aqueduct, securing his statements from the replies which he received to the questions noted on page 667 of the June 22nd issue of *The Canadian Engineer*. Controller Villeneuve says that the aqueduct will cost over eleven million dollars and that it is a blindly recommended project, poorly defined and little studied, and based upon defective plans that are impossible of fulfilment.

Mr. Villeneuve says that $6,742,661 has already been spent or is involved in the work now being done, and that in addition to that sum the following amounts will be necessary to complete the work: Land to be expropriated, $225,000; bridges and head gates, $650,000; ditches and ferries, $50,000; basin and breakwater, $475,000; plant and buildings, $1,500,000; engineers, etc., $90,000; electrical lines, lamps, auxiliary steam plant, interest during construction, etc., $2,000,000. If Controller Villeneuve's figures are right, it will mean that the aqueduct will cost $11,242,000. "This," says the controller, "would put the cost per horse-power at $1,124, which is eight times the cost of Lachine Rapids power." Mr. Villeneuve quotes figures to show that the operating cost per horse-power year will be $76, "which is far from the cost of production as stated by the (former) chief engineer of the city to be only $12.62." This cost of production would in itself be more than three times the price at which the city could purchase power from any of the private companies in the district.

EDITORIAL INDEX.

The index to articles in *The Canadian Engineer* for the year ending June 30th, 1916, will be printed within the next few weeks, and will be distributed to all subscribers as an integral part of one of our regular issues.

PERSONAL.

Major T. M. McAVITY, of St. John, N.B., has been awarded the Distinguished Conduct Medal.

WILLIAM TODD has been appointed superintendent of the Edmonton, Alta., pumping and filtration plant.

GEORGE L. SPRAGUE, of New York, is to be the new principal of the Technical School at Hamilton, Ont.

W. H. HORWOOD has been promoted to the position of sales manager with the Canadian Steel Foundries, Welland, Ont.

MARTIN UPPER has been appointed road commissioner of Stamford Township, Ont., to succeed Sylvester Bradley, resigned.

Sir WILLIAM PRICE, chairman of the Quebec Harbor Commission, has resigned and is going overseas. He is succeeded by D. O. Lesperance.

J. McGREGOR, superintending engineer of the Halifax Ocean Terminals, now under construction, has obtained leave of absence to go to the front.

W. K. JEFFREY, formerly manager of the Ottawa Car Manufacturing Co., has joined the Lyman Tube and Supply Co. as sales manager, with headquarters in Montreal.

W. R. GILMORE, manager of the Canadian Steel Foundries, Welland, Ont., is resigning to become vice-president and general manager of a steel company at Benton Harbor, Mich.

ARTHUR D. LITTLE, who is organizing the research department of the C.P.R., gave an address last week before the Vancouver Branch of the Canadian Society of Civil Engineers.

RICHARD DEANS WAUGH, mayor of the city of Winnipeg, has been appointed chairman of the Board of Commissioners of the Greater Winnipeg Water District, taking the place of the late Samuel H. Reynolds.

A. R. MURPHY, Jun.Am.Soc.S.E., of Knoxville, Tenn., has become associated with the engineering staff of Wallace & Tiernan Co., manufacturers of chlorine control apparatus, New York City. Mr. Murphy is a graduate of the University of Tennessee and has had several years' general waterworks experience.

Prof. ARTHUR L. CLARK, professor of Physics at Queen's University, Kingston, Ont., has returned home from Holland, where he conducted a series of experiments with liquid air at the University of Leiden. Prof. Clark intends to install a liquid air machine at Queen's for the purpose of continuing his experiments.

G. A. McCARTHY, engineer of railways and bridges of the city of Toronto, is engaged in a valuation to determine what the Bloor Street viaduct would have cost the city under the original order of the Board of Railway Commissioners. The city council will then determine the amount payable under the new order.

A. M. MOUAT, C.A., city controller of Edmonton, Alta., has just issued the city's financial report in printed form. These figures, which have already appeared in The Monetary Times, are issued in as simple form as possible, in order that they may be readily understood by the taxpayer. Municipal accounts are, unfortunately, usually submitted in such form that only the professional accountant can comprehend them, and they are frequently a source of much mystery to the man on the street. Mr. Mouat is to be congratulated upon the appearance of his city's financial statements and reports for the past year.

OBITUARY.

MICHAEL A. PIGOTT, a retired contractor of Hamilton, Ont., died suddenly last week aged 66 years. Deceased was born in Guelph, Ont., but resided in Hamilton for forty years. He was an architect, but undertook contracting work many years ago. In partnership with Sylvester Neelan, a well-known St. Catharines contractor, Mr. Pigott built the Hamilton city hall and also secured the contract for Toronto's municipal building, but owing to a dispute the work was taken from them when about half completed. The Guelph-to-Goderich C.P.R. line was one of the firm's jobs; also a considerable amount of government harbor and dock work at Midland, Ont. Following a paralytic stroke two years ago, Mr. Pigott had retired from business.

The results of a series of efficiency tests on a 30,000 kilowatt cross-compound steam turbine in a power station of the Interborough Rapid Transit Company, of New York, have been given in a paper before the American Society of Mechnical Engineers. It was stated that "the thermal efficiency of the turbine was now nearly 25 per cent., equal to that of the gas engine, while the latter involved much higher overhead charges and maintenance costs. For the same reason hydro-electric power, which looked like a gold mine 15 years ago, was, to-day, not a good investment. Even at Niagara Falls, where the development charge was at a minimum, and where the supply of water was practically unlimited, hydro-electric power could not compete with that obtained from a modern steam turbine station when the load factor was less than 50 per cent."