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The Canadian Engineer

A Weekly Paper for Civil Engineers and Contractors

Studies in Surface Area Proportioning Method

Discussion of R. B. Young's Paper Published Under Above Title in the June 26th, 1919, Issue of The Canadian Engineer—Surface Area vs. Fineness Modulus—Results of Tests on Bulking Effect of Moisture in Sands

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IN a theoretical study, having for its object the development of a well balanced, practical method for designing concretes, we are inevitably bound to recognize two natural divisions of the subject.

These divisions are, however, very closely interrelated and cannot be considered altogether independently. One of these involves a consideration of the physical and chemical properties of the constituent materials; the other a consideration of the methods and operations which enter into the "transition" or "making" stage.

Experience has amply proven that the latter may operate to vitalize and to develop in the finished product all the latent

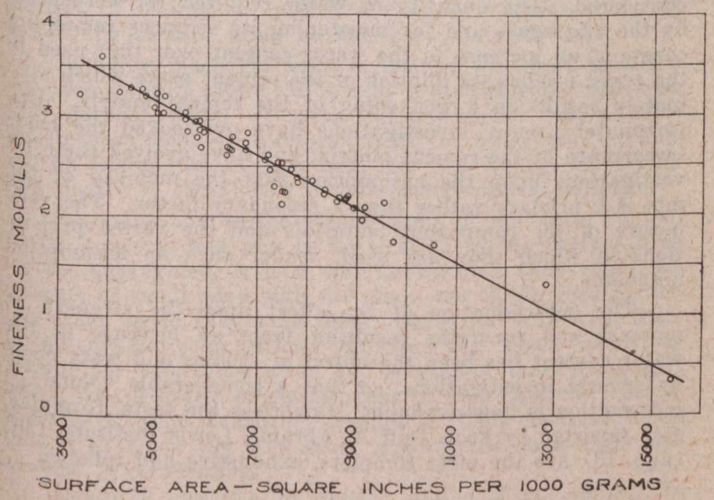


FIG. 1—SURFACE AREA VS. FINENESS MODULUS

functions possessed by the materials or, on the contrary, they may serve to render these functions wholly, or in part, impotent. inert and useless.

It is a situation in which we have two groups of forces tending under favorable conditions to co-ordinate and under unfavorable conditions to disrupt. Naturally enough this dual condition has resulted in a certain amount of confusion in the interpretation of results obtained from experimental tests.

The theoretical studies made by Mr. Young have shown quite conclusively that the grading of the aggregate, or more specifically its surface area, bears a very definite relation to the strengths of mortars and concretes. The relations which he has shown to exist between the surface area, and the cement and the water contents of the mix, are fundamental. Further studies and experimental tests will show a still wider application of this surface area factor than has yet been developed.

When the surface area method of proportioning mortar and concrete mixes was first presented, discussion brought forth the claim that the grading of the aggregate, and consequently its surface area, influenced the strength of a mortar or a concrete only to the extent of fixing the quantity of water necessary to produce a plastic mix. In view of my own investigations and the conclusions drawn from them, I was prepared to accept this notion only with definite limitations, rather than in the "blanket form" in which it was offered.

It is obvious that if we are to establish a uniform method for the testing of the mortar and concrete-making qualities of sand and stone aggregates, we must aim to eliminate factors depending upon the judgment and personal equation of laboratory operators.

The visual or "guess by eye" determination of a "plastic mix" is too indefinite for adoption as part and parcel of such a method.

The writer recognizes fully the importance of the water content of mortar and concrete mixes, both as regards its proper function and its influence when used in excess of that required for the fulfilment of that function. However, he is not prepared to believe that the interpretation of its full effect is based upon the relation of the water to the cement contained in the mix, that is, upon the water-cement ratio.

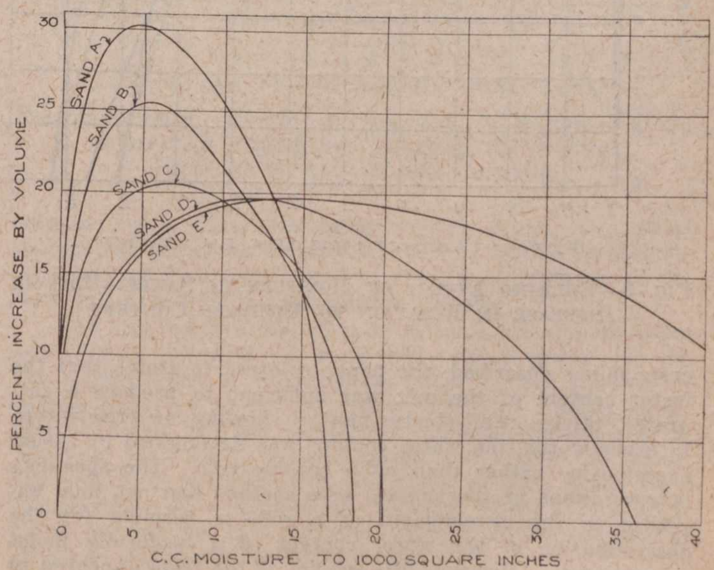


FIG. 2—BULKING EFFECT OF MOISTURE IN SANDS—VOLUME INCREASE IN RELATION TO AREA-WATER CONTENT

The fundamental principles of the surface area method of proportioning preclude the unqualified acceptance of this ratio. In so far as the surface area method is concerned, it is an indirect function rather than a primary one. It varies rather indirectly with the richness of the mortar or the concrete mix.

It is of interest to note that in Table No. II.,* Mr. Young has brought out very clearly the effect of the cement content as tending to vary the water-cement ratio, and has shown that for uniformly moist mixes this ratio is variable. His conclusions are in full accord with the consistency theory of the surface-area method.

The theoretical studies described by Mr. Young do not touch upon the effect of sandy dust (material passing a No. 100 sieve) in its relation to the water content of mortar and concrete mixtures. In the very limited concrete tests described in the writer's paper published in *The Canadian Engineer* in July, 1918, the dust content of the sand aggregate was treated as having the same surface area as an equal portion of material passing a No. 80 and retained upon a No. 100 sieve, and the cement content was proportioned accordingly. As regards the water content of the three con-

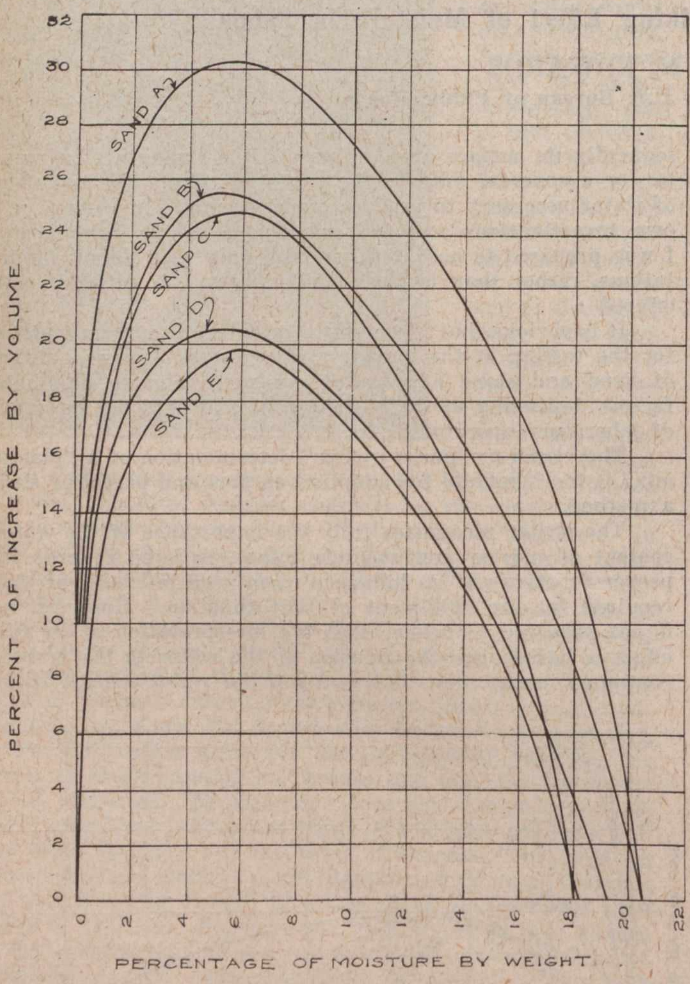


FIG. 3—BULKING EFFECT OF MOISTURE IN SANDS—VOLUME INCREASE IN RELATION TO MOISTURE CONTENT

crete mixes described, the paper referred to states that the water content of the mix was sufficient to produce a saturated, sticky, semi-plastic mortar showing no free water. In other words, the water content was determined by visual examination rather than by a specific rule. The speaker's investigations of the surface area method had not then extended to the development of a rule or formula for the determination of the water content of a uniformly moist concrete mix. However, the results of the tests appeared to be amply consistent to indicate the adaptability of the surface area method to concrete mixes, and for this reason alone they were included.

Tests made within the past six months with the object of obtaining information relating to the effect of sandy dust, have shown quite conclusively that the assumptions just referred to were entirely wrong in principle. Speaking briefly, these tests show that in proportioning the cement content, no allowance should be made for the surface area of dust material and that the water allowance for moistening the dust is nearly equal per unit of weight to that required per unit weight of cement. In other words, we must treat the dust as a "dilutant" or "extender" of the cement, rather than as a component part of the sand aggregate. Incidentally, it may be said that these tests have shown ample evidence of both the reliability and the advisability of the common practice of limiting the dust content, by weight, to approximately 5% of the sand aggregate.

*See *The Canadian Engineer*, issue of June 26th, 1919, p. 565.

Investigation of Various Aggregates

For a proposed concrete structure, the consideration of possible sources of supply of aggregates frequently involves the laboratory examination of the concrete-making qualities of two or more different aggregates varying in their granulometric composition and other physical properties. The field conditions attending the mixing, handling, placing and finishing of the concrete, as well as the ultimate strength, durability, etc., of the concrete producible from them, require that the relative merits of these aggregates be determined under conditions eliminating variations in the relative plasticity and the final strength of the cement matrix which performs the function of holding together the particles of the sand and stone aggregates. When the aggregate contains sandy dust it is important that allowance be made for the quantity of water required to moisten it.

Having determined under uniform conditions the relative concrete-making values of the materials, the question of their adaptability to the production of workable, plastic concrete mixes capable, in the "transitory" stage, of being formed with a reasonable amount of work into the various shapes required in its adaptation to the construction of bridges, buildings, roadways, etc., can be fully and systematically considered. The quantity of water required for absorption by the aggregate and for moistening its surfaces remaining constant, an increase in the water content over that used in the tests, produces a dilution of the cement paste which ultimately results in a weakening of the cement matrix. Unfortunately, many investigations have overlooked the basic importance of the cement matrix, and have evolved their investigations upon the assumption that the mobility of the mix is a primary rather than a secondary factor. The very nature of the component materials and the varied proportions in which they are used, render such an assumption untenable.

The determination of the effect upon the strength of mortars and concretes resulting from an increase in the water content has been the object of studies and tests made by several investigations, so that a considerable volume of information is now available. Doubtless the tests, formulas, etc., reported by Prof. Duff A. Abrams, Lewis Institute, Chicago, Ill., are the most complete, exhaustive and reliable.

Laboratory Testing of Concretes

Having discussed, at least to a limited degree, the fundamental requirements for the investigation of aggregates, it will not be amiss to indulge in a brief consideration of the conditions which are of vital importance to the laboratory testing of concretes and to the establishment of standard methods and practices. While several conditions of lesser importance might be enumerated, the three most important basic conditions entering into comparative tests are the following:—

1.—The water content of the mixes must be sufficient to permit the production, without loss of water, of uniformly sound, well-moulded specimens.

2.—The water content of each mix must be so proportioned as to produce for a given cement a uniformly strong cement matrix.

3.—The cement content of each mix must be so proportioned as to provide a uniform and equal quantity of matrix in relation to the surface area of the aggregates.

In a series of tests adapting the principles just mentioned to laboratory conditions, it has been found that concrete mixes containing a uniform broken stone aggregate, sand aggregates differing in their granulometric analyses (their surface areas varying from 1,830 to 2,490 sq. ft. per 100 lbs.), and with cement contents varying (by ½ lb.) from 1 lb. to 4 lbs. per 100 sq. ft. of surface area of total aggregates, produced generally excellent working conditions when the water content of the mixes was proportioned as follows:—

Water to moisten cement equals weight of cement multiplied by the percentage of water required to produce normal consistency paste.

Water to moisten surface area of aggregates equals 12 cc. per 1,000 sq. ins., equals 0.381 lb. per 100 sq. ft.

Water to moisten dust, if any, equals 0.381 lb. per 100 sq. ft. of surface area, its surface being assumed to be equal to that of sandy material passing a No. 100 sieve and retained upon a No. 200 sieve.

In these tests the concretes produced from lean mixes containing 1 lb. and 1½ lbs. of cement per 100 sq. ft. were harsh, and required an extra amount of labor to effect the placing in the moulds. All other mixes were plastic, the cement paste which they contained being sufficient to lubricate the surfaces of the aggregates. The fat mixes, containing over 3 lbs. of cement per 100 sq. ft., could have been easily placed in the moulds had they contained a slightly less quantity of water. Economy of construction requires that the increased strength thus attainable be taken account of in actual field construction operations.

The origin and development of the water factor for moistening the surfaces of the aggregates and dust will be described by the aid of diagrams.

Surface Area vs. Fineness Modulus

At practically the same time as Mr. Young was seeking to establish a mathematical relation between surface area and fineness modulus, I was engaged in a somewhat similar effort, not with the object of corelating these methods, but, instead, of finding out just how fineness modulus short-circuited the surface area, as had been claimed.

Just how much efficiency is lost by the short circuit is not clear. Fig. 1 shows the results of studies made with over fifty natural sands. It gives the impression that possibly the fineness modulus is intermittently or otherwise "grounded" on surface area. However, like Mr. Young, the writer was unable to establish a definite mathematical relation.

Reverting now to the consideration of the water content of the concrete test mix in its relation to the surface area of the aggregates, it is a well-known fact that while the weights of sands vary with the character of the rock materials from which they take their origin, and also with their granulometric composition, yet, for any given sand, its weight per cubic foot when dry is greater than its weight when wet, provided, of course, that in each case the sand is shoveled into the measuring receptacle. This condition holds true even when the wet sand is subjected to a moderate amount of compacting.

Approved practice in mortar and concrete testing is based upon the use of weighed quantities of dry aggregate, and only upon this basis can the test results be properly interpreted. It is axiomatic that if results consistent with those secured in the laboratory are to be obtained upon field construction work the conditions under which the aggregates are measured, more especially the sand aggregate, must be such as to assure reasonable uniformity in the net volume.

Bulking Effect of Moisture

Notwithstanding the above conditions, there is a marked paucity of published data relating to the effect of varying quantities of water contained in sand aggregate used in mortars and concretes. Its absence is evidence that this factor has not been given sufficient attention by engineers, architects and construction men.

With the primary object of securing information relative to the "bulking" effect of moisture in sands differing in their granulometric analyses, tests were made by the Department of Works, Toronto, under the direct supervision of the writer. Paradoxical as it may seem, these tests have shown that the surface area of the sand particles is a direct function of the volume increases produced by varying the quantity of water.

In describing the tests very briefly, it may be said that the volume-weight method was used for determining the quantity of each sand to be used in the tests; the weight being determined in each case from an average of four tests. Natural sands were used. Two series of tests were made upon each sand, the water content being varied in the first in relation to the weight of the sand and in the second in relation to its surface area. A uniform distribution of the water was secured by working the sand with a trowel or other tool. To secure the desired degree of accuracy, each intermediate test was repeated four times. From these tests, involving eleven different sands, average volume-increase curves were determined. Fig. 3 shows, for five of these sands, the relation of volume-increase to percentage of water content. One per cent. of water was amply sufficient to coat thoroughly the surface of the sand particles. The "bulking" effect resulting mainly from the cohesive action of this small quantity of water, ranged in the tests from 11% to 23%. Two other interesting features of this diagram are the "saturation" stage producing maximum volume, when the water content approximates 5% to 7%, and the "flooding" stage, at which the volume of the sand is the same as that originally

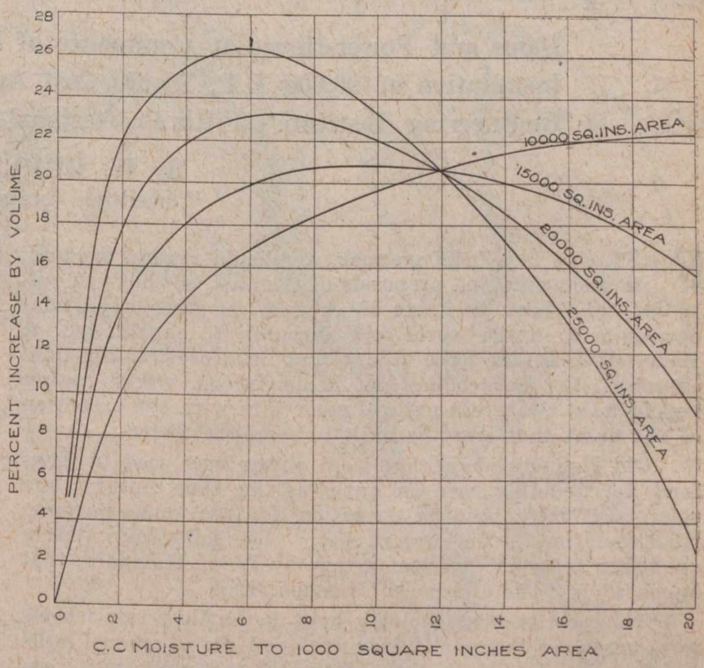


FIG. 4—BULKING EFFECT OF MOISTURE IN SANDS—EXPERIMENTALLY DETERMINED RELATION OF VOLUME INCREASE TO AREA-WATER CONTENT

occupied by it when thoroughly surface dried. The water content for the latter varied from 18% to 28%. Careful observation at this stage failed to discover a tendency of the sand to fall below its original dry volume.

The area-water series of tests led to a rather interesting and entirely unlooked for discovery. Fig. 2 shows, for the five sands shown in the diagram "Fig. 3," the relation of volume increases to the area-water content. Special attention is called to the indications of the existence of a "nucleus" common to all the curves. Although the writer had confidently expected that this series of tests would show the existence of a relation of "bulking" effect to the area-water content, yet this indication of a condition common to all sands tested was quite unforseen.

From a systematic study of the experimentally determined curves, we have deduced those shown upon Fig. 4. Each curve in Fig. 4 was derived mathematically from a consideration of all those determined experimentally. Attention is called to the actual existence of the "nucleus" indicated upon the previous diagram. The water-area ratio of this rather mysterious point is 12 cc. per 1,000 sq. ins., which corresponds to the water factor (0.381 lbs. per 100 sq. ft.) used for moistening the surface of aggregates and dust in concrete test mixes.

The claim is not made that the volume-moisture condition just described bears a direct relation to the many factors involved in concrete making. However, its actual use in this connection has indicated its efficiency as a working tool. We can, I believe, ill afford to cast it aside without a full and careful trial to prove or disprove its value. It is entirely possible that physical factors other than cohesion and adhesion enter into this peculiar volume-moisture condition. In so far as the speaker has a knowledge, its existence has not been previously discovered. However, independent and more recent investigations made by Mr. Young have shown corroborative and unquestionable evidence of its existence.

Since the moisture content in a sand has the rather remarkable effect of increasing its volume by holding the sand particles somewhat rigidly in positions different from those they would readily assume when dry, it seems reasonable to aim to control this "bulking" effect, so far as is possible, in test mixes for concrete.

There is no evidence that the "bulking" effect of the moisture in sands has been given consideration either in written specifications or in field operations incident to mortar and concrete-making. Its consideration will show the fallacy of the commonly used volumetric method in so far as resulting strengths are affected by the moisture contained in the sand aggregate, and will inevitably lead to the opinion that the adoption of methods and appliances whereby the aggregates will be measured by weight rather than by loose volume, will result in a greater uniformity and reliability of the mortars and concretes produced. Important practical considerations render it advisable to continue the present practice of using in concrete mixes a volume of mortar equal to approximately one-half of the gross volume of the stone aggregate.

Proposed Tidal Power Development at Hopewell

Dams and Power-House at Confluence of Petitcodiac and Memramcook Rivers—Initial Installation of 90,000 H.P. Would Cost Approximately $122 per H.P.—Paper Read at Engineering Institute's Fifth Professional Meeting, September 11th, at St. John, N.B.

By W. RUPERT TURNBULL
Consulting Engineer, Rothesay, N.B.

FIRST, and in a brief manner, we should review the tidal developments and proposals of the past, so that you will be led, as I have been, to think that the first large tidal development in the world will probably be carried out at Hopewell, N.B., the little village that lies closest to the tidal estuaries, the Petitcodiac and Memramcook, where nature has founded two great natural reservoirs with the exception of the dams that must be built to complete them.

Old charters show that tidal power was used in England, for grinding corn, as early as the 11th century, and tidal mills have been in operation for the same purpose, from that time to the present day. The following extracts are taken from an excellent article by W. C. Horsnaill, that appeared in "The Engineer," London, Eng.:—

"No records exist showing how the earliest tide wheels were arranged, but particulars are available of several mills which were erected in the eighteenth and nineteenth centuries. In the earlier historic mills no attempts were made to produce a fall, the power being obtained from the flow of the water into and out of the pound. To develop power in this way, a wheel similar to the paddlewheels of steamships was used, but with a reversed action; that is to say, the flow of water drove the wheel. This arrangement entailed the raising and lowering of the wheel to suit the rise and fall of the tide, as only the bottom floats could be immersed if the best results were to be obtained.

Layout of Old Mills

"A corn mill at one time existed at East Greenwich which was driven by tidal power in the way we have described. The pound has an area of about 4 acres and the wheel measured 11 ft. in diameter by 26 ft. long. The power was transmitted by a bevel gear at either end of the water-wheel shaft, the pinions being free to slide up and down two square vertical spindles. The water-wheel and bevel gears were mounted upon a frame which was caused to rise and fall to suit the tides, and the power was transmitted by either bevel wheel according to which way the water-wheel was running, the other bevel pinion being thrown out of gear. By these means the machinery in the mill was always driven in one direction, in spite of the reversal of the water-wheel at each turn of the tide.

"The movable frame, with the water-wheel and gear, weighed some 20 tons and the bottom of it was extended to form a kind of shutter, which filled up the opening underneath the wheel race, all the water flowing into or out of the pound being thus compelled to pass through the wheel.

"Another type of wheel was devised to overcome the drawback of having to move up and down with the tide. This wheel was fitted with hinged floats, which arranged themselves across the stream at the bottom of the periphery, while they travelled through the water edgeways during the remainder of each revolution. With floats of this type the wheel was fixed, and the tide gradually rose over it until in some cases complete immersion took place.

This Design Soon Discarded

"An arrangement of the sluices was also adopted to compel the water to pass through the wheel in the same direction, whether flowing in or out of the pound, thus doing away with the need for reversing gear between the water-wheel and the machinery to be driven.

"These wheels must have been very inefficient, as the loss of power caused by the drag of the upper portion when covered was serious, and the design was soon discarded.

"Following these earlier mills came the more recent examples, many of which are still in existence, while a few of them may be seen in operation. The older mills aimed at using the current of water caused by tidal action, and advantage was taken of the flow in either direction. The more modern tide wheel is arranged to operate with a considerable fall, and only develops power when the water is flowing out of the pound.

"The undershot wheel with straight radial floats is usually adopted, and the mill is started at half ebb or a little later, work being continued for about five hours, or until the water rises under the wheel and chokes the tail race. These arrangements give only five hours of working during each tide."

Listing the tidal mills that actually exist: There is a mill at Woodbridge of 10 to 12 h.p. and one at Walton-on-the-Naze of 85 h.p. These are both small powers, working on a low range of tide, and with only a single, small, natural reservoir that allows of only a partial use of the tidal power for a comparatively short period of time, but Mr. Horsnaill shows that if modern turbines were installed at the plant at Walton-on-the-Naze, and the power used to develop electricity instead of grinding corn, it would show up as a commercial development somewhat better than gas power in spite of a heavy outlay for storage batteries, which would be necessary at a plant situated as is Walton.

The number of proposals for tidal plants is very considerable, and while I think I should not take up your time by discussing all of them, it is worth while to examine a few of the more serious ones.

James Saunders discussed, in the "Engineering Review" of London, Eng., three great plants for developments in England; namely, at Chichester Harbor, at the Menai Straits, and in the Bristol Channel. But in each case either the head of the water was too low, or the cost of forming the artificial reservoirs was too great, to make the proposals commercial at present. His most promising scheme is that for the Bristol Channel, where the tidal head is quite sufficient for successful operation but where the cost of forming the great artificial reservoirs that would be required is prohibitive in relation to the power obtained. The total cost of the plant figured out at $47,000,000 and the horse-power at 240,000 so the cost per h.p. would be $196.

Cost Excessive in England

C. A. Battiscombe, before the London Society of Engineers, also made a tidal proposal for the Bristol Channel, but his cost works out at $237 per h.p., and while neither of these figures would be too high for commercial developments *in some localities*, they are too high to interest English capital, for England is still a country of cheap coal, and in examining *any* hydro-electric development we must constantly keep in mind the cost of power from other sources.

Mr. Boving has proposed a tidal plant for the River Dee, but no estimates of costs are given; and coming nearer home, there have been numerous proposals for obtaining power from the tides at Sackville, at Cape Split, and at the Reversible Falls of St. John. To get *continuous* power at any of these sites, it would be necessary to form large artificial reservoirs, and the formation of such reservoirs is so costly that these proposals are not, at present, commercially feasible.

The Three Main Requirements

The three great desiderata for a tidal plant are:—

1.—That there should be sufficient height of tide to obtain a good head.

2.—That there should be two natural reservoirs of large size so that continuous power can be obtained.

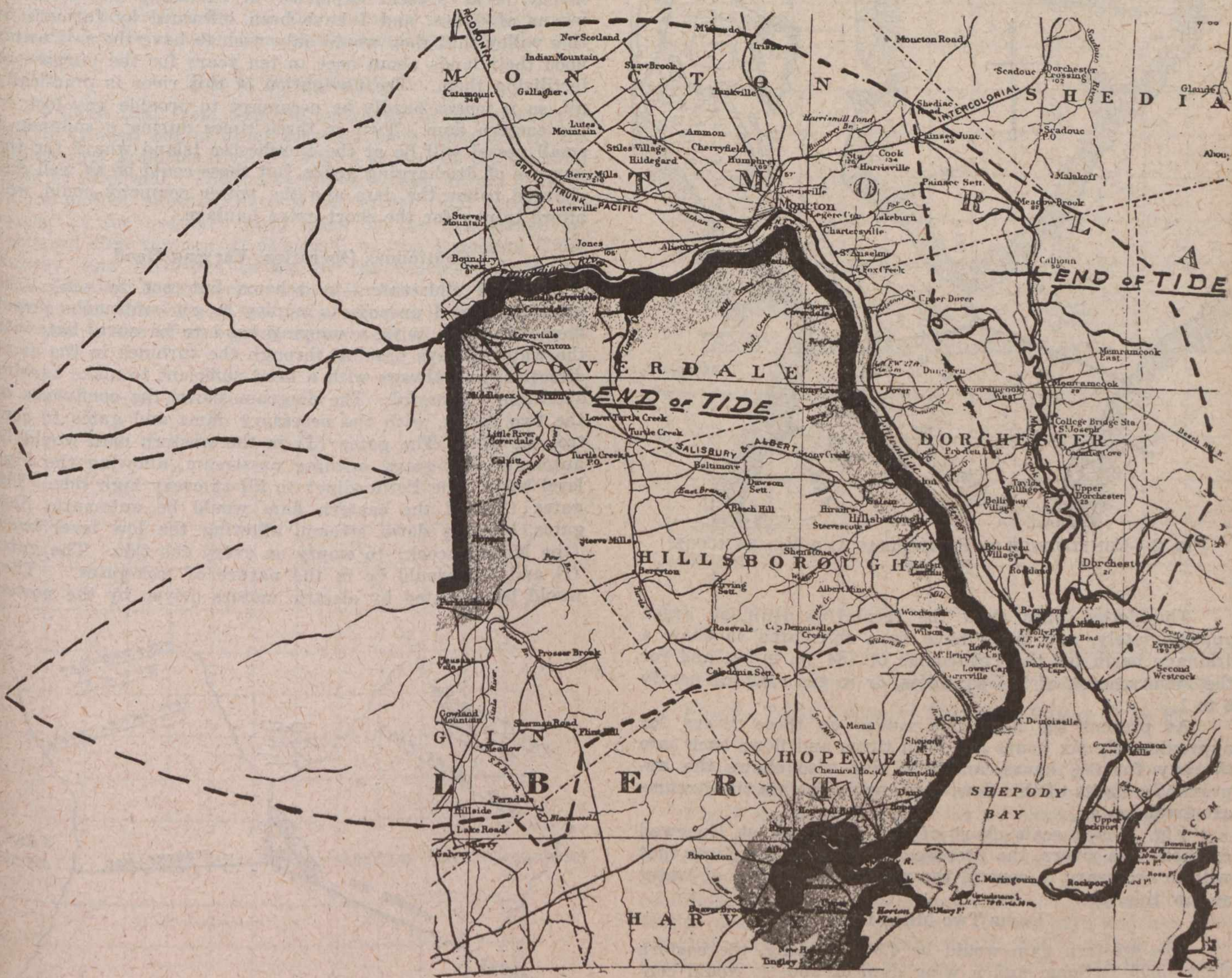


FIG. 1—MAP SHOWING WATERSHED OF THE PETITCODIAC AND MEMRAMCOOK RIVERS

3.—That the power plant should be central to the population that would be served.

And it is these three desiderata that lead us to suppose that the *first* great tidal development in the world will take place at Hopewell. Here we have two large reservoirs almost completely formed by nature, we have a tide which is exceedingly regular and that ranks among the highest tides in the world, with a spring rise of 45 ft., a neap rise of 38 ft., and a normal neap range of 32 ft., and we have this power centrally located to a present population of 250,000 who are *literally starving* for cheap electric power, with no other hydro-electric development in sight, except small ones and those that are too distant from the centres of population to make their development commercial at the present time.

I have examined many other sites for tidal power in those parts of the world where the tides are sufficiently high to make tidal power at all possible, and I am firmly convinced that we have at Hopewell the site that is most promising at the present time, from a commercial standpoint.

I will now discuss the principle which I have proposed for obtaining continuous power from these great tides. Fig. 1 is a map of the two tidal estuaries, the Petitcodiac and the Memramcook rivers. This map shows the general trend of these rivers, and their confluence at Hopewell, and it also shows the fresh-water drainage areas of the two rivers, which, although a minor item as compared to the great volume of salt water that flows up and down these rivers, should still be borne in mind in reviewing this proposal.

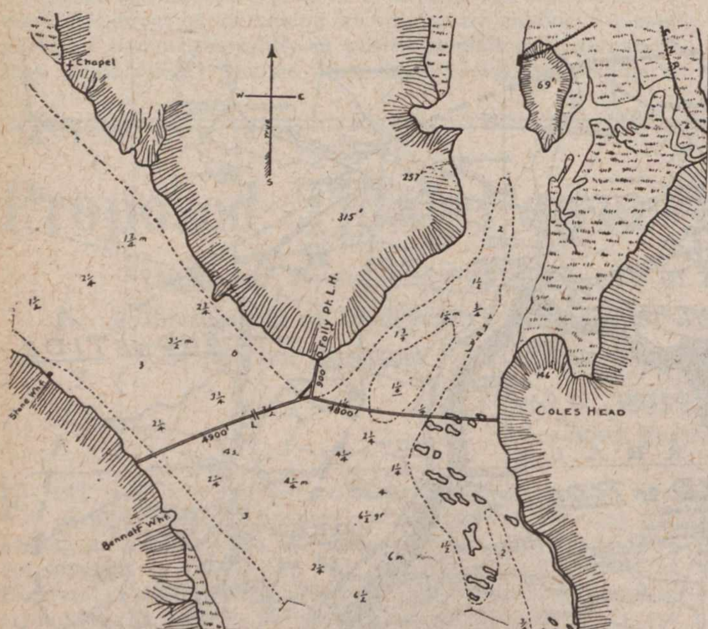


Fig. 2—Confluence of Petitcodiac and Memramcook Rivers

The dotted lines show the respective drainage areas approximately, and these have been calculated to show a drainage basin for the Petitcodiac of 784 sq. miles, and for the Memramcook of 134 sq. miles, or in the ratio of nearly 6 to 1.

At present the flood tide makes up these rivers for approximately six hours and then turns and flows back into the bay for 6½ hours, and this map also shows the approximate limits of this flow, above which points the streams are fresh water.

Fig. 2 is a scale chart of the confluence at Hopewell of these two rivers, the proposed location of the dams that will be necessary to control the waters, the depths of water at low tide, etc.

Dams Two Miles Long

The western dam would be 4,900 ft. long, the eastern dam 4,800 ft. long, and a wing dam of 900 ft. would connect the two, and it would, of course, be part of the plan to have a highway and trolley line (operated by the plant) over the tops of these dams, connecting up the two main shores and the long peninsula between the two rivers. This highway would only be a matter of local benefit, but it would be of immense benefit to the building up of this locality. At present Hopewell can only be reached from Cole's Head by a little ferry that can only operate for a few hours near high tide. The Petitcodiac is only bridged at Moncton, 19 miles above Hopewell. The Memramcook was bridged at Upper Dorchester, 5 miles above Hopewell, but this bridge is now gone, and the government is at present making borings for a new bridge that will cost about $1,000,000.

At present Hopewell, Hillsborough and the bog peninsula are hard places to get into, and still harder places to get out of, and yet they are regions of great mineral resources that only require, but still await, development. In Fig. 2 is indicated a lock in the western dam, through which vessels could be passed at any suitable time of the tide on their way up and down the Petitcodiac. Hillsborough and Moncton would be provided with deep water harbors instead of the mud flats which they at present enjoy at every low tide. The gates of the locks would naturally be swung by electric power furnished by the nearby power house.

The proposal provides for making the Petitcodiac a high level basin in which the water would always be high, and be replenished at every high tide, while the Memramcook river would be a low level basin to be partially filled from the high level basin and to be always emptied during the latter part of the ebb tide. This arrangement would also suit local conditions admirably, for the Memramcook is exclusively a farming district in which much time and money is at present expended in excluding the tides by means of dykes, and I have been informed by farmers of this valley that they would only wish to have the salt water flood their lands about once in ten years for the purpose of fertilizing them. The navigation of this river is practically nil, so it would hardly be necessary to provide any lock in the eastern dam. Two or three times during a summer a small vessel will lie at the Dorchester Island wharf for the purpose of discharging goods, but these could be as well discharged below the dam and the power company could well afford to pay for the short extra haulage.

Continuous Operation, Varying Head

Fig. 3 illustrates—in scheme but not to scale—the principle that I propose to employ to get continuous power from the tides, with a varying head to be sure, but with the water always passing through the turbines in the same direction, and always with a head sufficient to make turbine operation successful. The diagram shows the confluence of the two rivers, with the necessary dams and gates to control the flow. The gates (J) in the western dam would be automatic flap-gates, opening up-stream, allowing the high level basin (the Petitcodiac) to fill at every high tide. The gates (H) of the eastern dam would be automatic flap-gates, opening down stream, allowing the low level basin (the Memramcook) to empty on every ebb tide. The gates (G and G_1) would be in the nature of lock-gates. They would be operated by electric motors driven by the power-

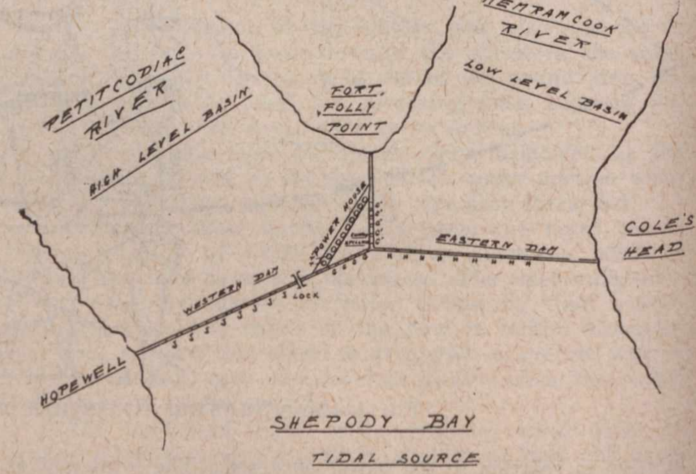


Fig. 3—Layout for Obtaining Continuous Power From the Tides

plant itself, and under the control of the attendants, who would open them and close them, in accordance with the height of the external tide, at stated times that can be fixed for months in advance directly from the tide tables.

The power house is represented as a long building, with turbines (T) extending diagonally from the wing dam to the western dam, these turbines discharging continuously from the high level basin into the common, triangular, spillway.

Let us now follow through a cycle of operations from low tide to the following low tide, remembering that the high level basin was filled automatically at the last high tide, and that the low level basin has just been emptied during the ebb tide, through the gates in the eastern dam.

Beginning with low tide we may at first leave gates (G) open and allow the water from the spillway to discharge directly into the tidal supply (Shepody Bay), but the head will gradually decrease as the tide rises, and at about 2½ hours rise the attendants close gates (G) and open gates (G¹), allowing the discharge from the spillway to enter the low level basin. Into this low level basin, the spillway will continue to discharge for about 6½ hours, or through the last 3½ hours of flood tide and through the first 3 hours of ebb tide, after which time the water in the low level basin will have so risen, and the water of the tidal supply will have so dropped, that it will now be profitable to close gates (G¹), and open gates (G), and once more allow the discharge to occur directly into the source of tidal supply and give the low level basin time to drain out again on the ebbing tide.

Tide is Very Regular

I think that from this the simplicity of the system itself is evident, but in order to study more fully one of the engineering problems involved, Fig. 4 shows the typical tidal cycle at Hopewell. This curve shows a copy, in per cent. of range plotted against time, of an actual tidal record obtained at the ordinary neap tides by the tide gauge established this summer at Hopewell by the Canadian Tidal Survey, and furnished to me by the courtesy of H. W. Jones of that department. You will note how exceedingly regular the tide is, and how little affected by estuary flow, and this is one of the great advantages of Bay of Fundy

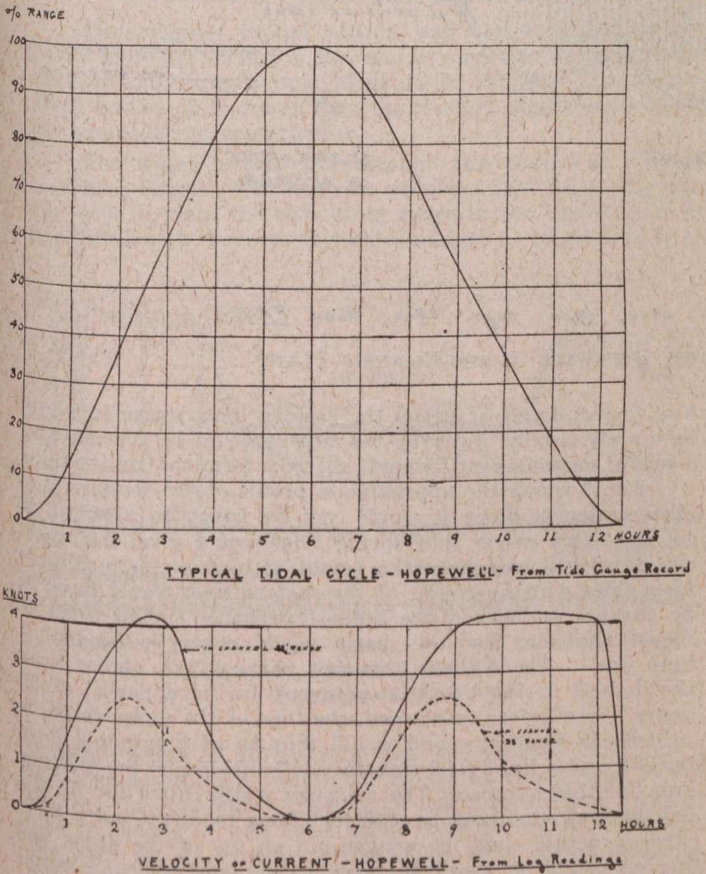


FIG. 4—TYPICAL TIDAL CYCLE AT HOPEWELL

tides, in general. At certain places in the world the diurnal inequality becomes so great that, for several days there is only one tide in 24 hours, and at Southampton there is a second high water occurring about two hours after the first.

In dealing with the question of tidal power at Hopewell, it must be remembered that although the tide is regular in type, nevertheless the *range* of the tide, and not the *rise* is the limiting factor of our power calculations, and it becomes necessary to establish and work on what might be called a "standard range." For this purpose I have analyzed approximately the ranges that will occur in the course of a year. I call spring tides those whose range exceeds 42 ft. They occur about 15% of the time, and I think no attempt should be made to utilize them especially. I call subnormal neap tides those whose range is less than 32 ft. They also occur about 15% of the time, and some means, which are discussed later, would need to be employed to avoid the impairment of our "standard" amount of power.

All other tides I call ordinary neap tides, with a range, at Hopewell of 32 ft. to 42 ft. They occur about 70% of the time, and its is the lower range of 32 ft. that I think

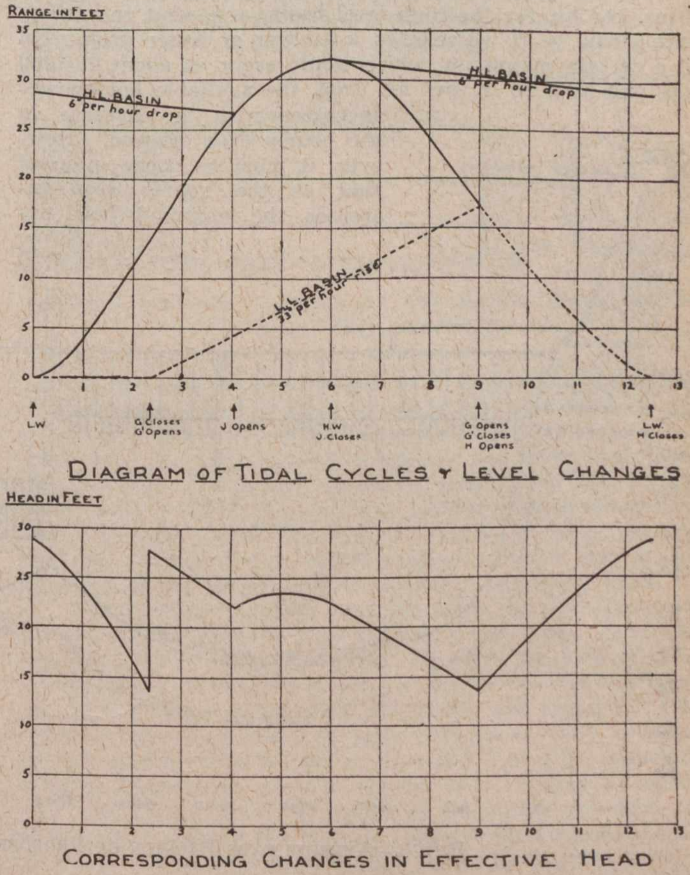


FIG. 5—TIDAL CYCLE AT STANDARD RANGE OF 32 FT.

we should adopt as our "standard range," and the curves and estimates that follow are based on this range of 32 ft.

Fig. 5 shows a tidal cycle at a "standard range" of 32 ft., with an assumed drop in the high level basin of 6 ins. per hr., the level change that will occur in the two basins with the operation of the plant as before described. If the conditions at Hopewell were absolutely ideal, the ratio of effective areas would be as 2 to 1 for the high and low level reservoirs, but unfortunately this is not the case, for the Petitcodiac contains an effective area of about 330,000,000 sq. ft. while the Memramcook has only about 60,000,000 sq. ft., so that the ratio is about 5½ to 1. While the water in the Petitcodiac is dropping 6 ins. per hr., the water in the Memramcook is rising 5½ times this, or 33 ins. per hr., and these level changes are illustrated in Fig. 5, while the changes in effective head on the turbines are plotted in the lower portion of the same figure.

Operation at "Standard Range"

Full lines show the level changes of the high level basin; dotted lines, those of the low level basin. Starting at low water, for 2.35 hours the level of water in the low level basin is unchanged, for the water from the spillway is discharging into the tidal supply direct, but during this time the head is decreasing from 28½ ft. to 13½ft., when it becomes expedient to discharge into the low level basin, when the head will at once rise to 27½ ft.

After this, for 6½ hours the low level basin will rise, but the high level basin will also rise after 4.1 hours (as the flood tide will then be filling in for 1.9 hours) and we have the head decreasing from 27½ ft. to 22 ft., but afterwards increasing to 23½ft. at 5.2 hours.

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

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

It will perhaps be wondered why I select a drop of 6 ins. per hr. for the high level basin, and what the difference would be if we selected a greater or lesser drop. Up to a certain maximum, which would occur at about 250,000 h.p. and at a 26 in. per hr. drop, the available horsepower increases with an increase in the hourly drop allowed. However, it must be borne in mind that as the hourly drop increases, the maximum head de-

level basin and with a probable average turbine efficiency of about 83%.

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

To improve this basin-ratio to the ideal condition in which the Memramcook would have half the effective area of the Petitcodiac, I propose to use electric shovels and an electric tramway (both operated by surplus power from the initial installation), expropriate by government charter the low-lying farms of the Memramcook valley at a fair and equitable rate, and shovel out the basin according to power requirements, removing the material by the electric rail-

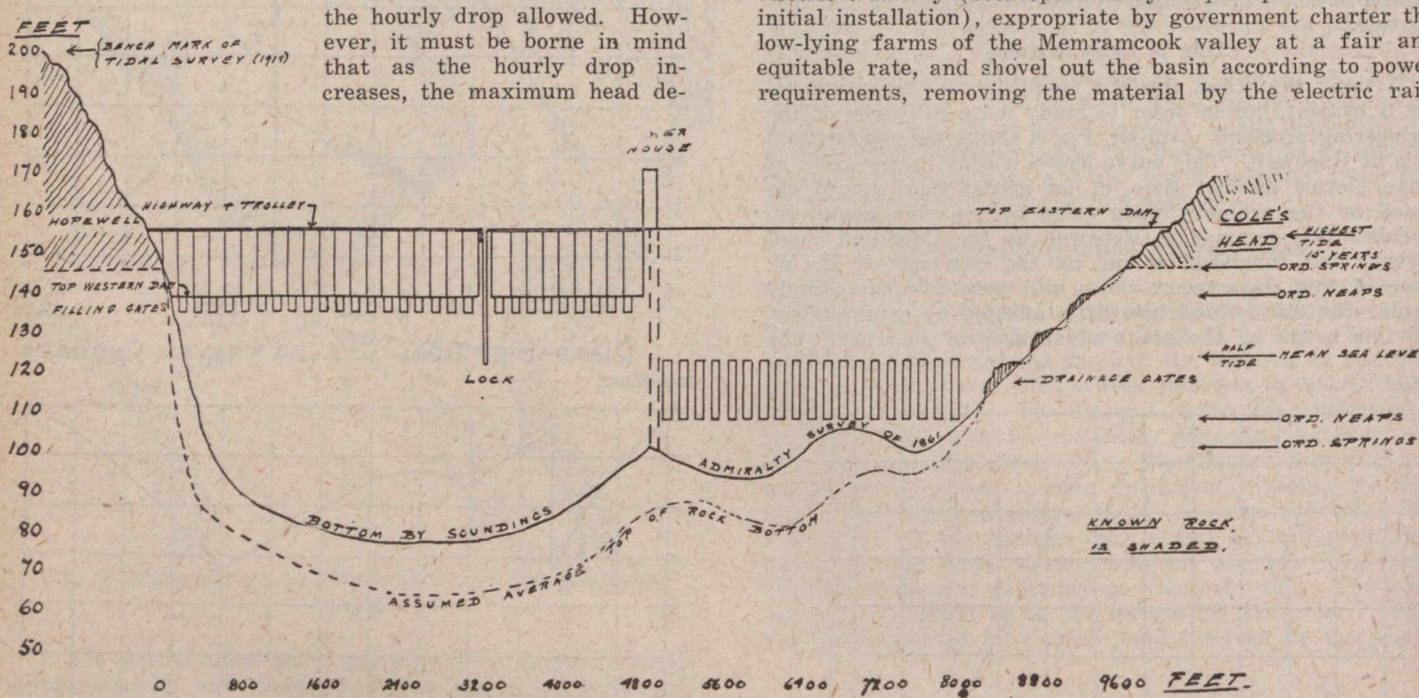


FIG. 6—APPROXIMATE PROFILE OF PROPOSED DAMS, HOPEWELL HYDRO-ELECTRIC PLANT

creases, and the minimum head decreases more rapidly still, and we soon reach a case that cannot well be met by any ordinary turbines.

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

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

way and dumping it below the eastern dam where it would be largely carried away by the tide or could be formed into a useful embankment—wharf, railway terminal, or the like.

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

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

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

(Continued on page 372)

BACKFILLING THE RED RIVER TUNNEL

Concrete Poured Around 60-In. Cast-Iron Pipe in 10x10-Ft. Rock Bore by Dropping it 70 Ft. From Trestle Across River

By J. Armstrong
Division Engineer, Greater Winnipeg Water District, Winnipeg, Man.

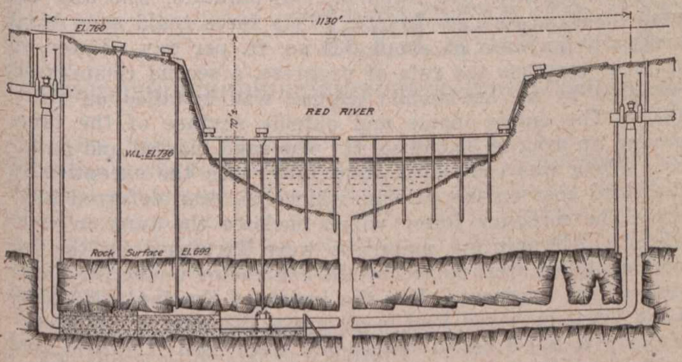
OVER 1,100 ft. of the new Winnipeg aqueduct is in tunnel under the Red river, with the roof of the tunnel about 70 ft. below the high part of the ground and 40 ft. below the river level. The 60-in. cast-iron pipe carrying the water was concreted here into the 10 by 10-ft. tunnel by concrete dropped through holes bored at intervals from the surface.

The tunnel bore is through the solid limestone rock, the rock roof being some 17 ft. in thickness, with overlying soils of various clays, silts, and other river deposits. The rock was found to be fairly hard but badly seamed both horizontally and vertically and containing many pockets of varying size up to two or three cubic yards, evidently the result of water action at some remote period, but now filled with a white, chalky clay. The first 400 ft. of the tunnel from the eastern end discharged considerable water into the boring from these seams, although not in sufficient quantity to require the aid of compressed air, a 4-in. submerged centrifugal pump handling the water at all times with ease. The remaining 600 ft., although seamed and checked, seemed tighter, and comparatively little water was added from this portion.

Practically Perfect Hydrostatic Action

The original tunnel section was designed with a circular roof, but owing to the seamy condition discovered the section was changed to a 10 by 10-ft. square. The eastern shaft is some 2 ft. lower than the western shaft, to facilitate drainage during construction.

The water is conveyed through this tunnel in a 60-in. cast-iron lining. The contract required that after this lining was in place the remaining space in the tunnel bore be backfilled with concrete in such a manner as to guarantee an



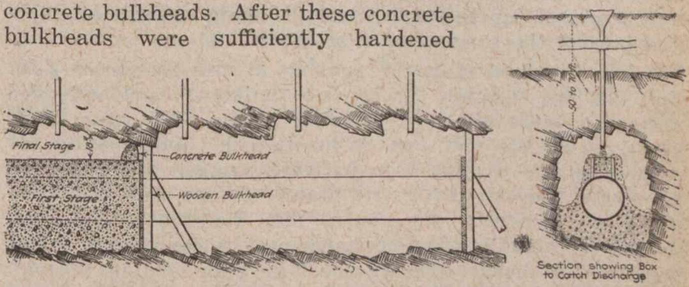
Section Through Red River Tunnel of Winnipeg Aqueduct, Showing Method of Concreting Pipe

absolute filling of all irregularities and pockets. The specifications specifically prohibited the use of compressed air in placing the concrete. After much study the method adopted was by means of well holes bored from the surface through the roof of the tunnel. These holes were drilled with an ordinary well-drilling machine and lined with a 6-in. well casing from the surface to the rock. The spacing of the holes was somewhat irregular but averaged 40 ft., the holes being bored to this spacing at the nearest high spot in the tunnel roof.

For the river crossing a light trestle was built consisting of two-pile bents spaced 8-ft. centres with four longitudinal 8 by 8-in. stringers to act as bracing and to bear the light industrial track carrying the concrete cars. A mixing plant was provided at each end of the tunnel consisting of a ¾-yd. mixer, industrial tracks and concrete cars of various types.

The first operation consisted in the pouring of a concrete floor in the tunnel carefully graded to facilitate the placing of the 60-in. cast-iron lining. This concrete, some 250 yds., was all poured down one hole at the western end and distributed by cars using the same track previously employed in the tunnel excavation and later used to transport the cast-iron pipe.

When the lining was in place the tunnel was bulkheaded off in sections of various lengths and concrete poured until within 18 ins. or 2 ft. of the roof. This was allowed to set and succeeding sections were poured. During the operation a concrete dam or bulkhead was built back on the set concrete, these dams being about 120 ft. apart, thus leaving three well holes between each two concrete bulkheads. After these concrete bulkheads were sufficiently hardened



Sections Showing Details of Concreting Aqueduct by Dropping Through Pipes

to withstand the pressure the final concrete was poured. The two end holes in each compartment were used to discharge the concrete, leaving the middle hole for observation purposes. In every instance when the end holes were filled the concrete rose in the middle or observation hole to the same level, showing a practically perfect hydrostatic action. As these holes provided a head of from 55 to 70 ft. of fluid concrete there seems no doubt that all cavities and seams adjacent to the tunnel section must be completely filled.

For pouring the floor and the main tunnel concrete a mixture of 1 cement to 4 of aggregate was used, all stones passing a ½-in. ring. The stone was kept small to avoid a possibility of choking the pipes. A fairly wet mix was used although nothing approaching sloppiness was required. Observation indicates that aggregate containing stone as large as 1-in. might be safely used in holes of 5½ or 6-in. diameter. Various devices were used to prevent any tendency to segregate the materials of the concrete, the most successful of which was a wooden box slightly larger than a standard nail keg fixed on top of the cast-iron pipe and directly underneath the pipe. The force of the concrete discharging into this box full of concrete gave the constituents a very thorough remixing. As a further precaution an inspector and two workmen were kept in the section being concreted, and properly distributed the concrete. These men were able to work in the concrete until it was within 3 ft. of the roof and it was at this stage the concrete dam or bulkhead was built on previously set concrete preparatory to the final or grouting operation. These men usually sank half way to the knees in the concrete, which will give some idea of the wetness of the mixture. Spouts with a hopper swiveled under the holes were also used successfully but were awkward to handle and seemed to involve more extreme effort than collecting in the box and then working along by shovel and tramping. The inspector had a field telephone installed in the tunnel giving ready communication with the mixer when instructions were necessary.

Filling Top Three Feet

The final stage was the filling of the space left as just described, the mixture used being 1 of cement to 3 of aggregate, the aggregate being in district parlance "rejected sand"; in other words, the excess finer materials screened from the pit run during the process of assembling the standard aggregate. This material was fairly well graded from a coarse sand to fine sand. As before indicated, this concrete was poured down the two outside holes of a three-hole set. A few batches quickly formed a cushion under the hole thus preventing segregation, as was easily proved by samples taken from the middle or observation hole.

The holes with one exception were bored directly on the centre line. This exception was at the eastern end where the rock was exceptionally seamy, and a couple of bad caves resulted. The second one developed an opening completely through the rock, letting in a considerable flood of water and earth from above; not, however, before the caving was visible on the surface 70 ft. above.

Some hesitation was felt about driving a hole through this shaky roof, and fortunately one of the large water worn cavities before mentioned was found here extending some 4 ft. off line of the tunnel. The pouring hole was accordingly driven here without the loosening of any further rock from the roof. Considerable water was coming through the roof in this section and was collected on a sheet-iron roof and run to wooden troughs along the side and carried beyond the concreting in progress.

For the final stage of pouring at this section, a 2-in. pipe was run through the concrete bulkhead and extended upward to near the upper surface of the rock, some 12 or 14 ft. above the roof line of the tunnel. When ready for concreting the wooden troughs were plugged and the compartment allowed to fill with water. When this was completed the concrete was poured into the still water down the pipe. Shortly after commencing the concreting, water began to discharge through the 2-in. pipe leading through the bulkhead. Finally grout began to appear discoloring the water; then the pipe was closed. Concrete was poured into this hole until it finally appeared in the cave-in previously mentioned above the centre of the tunnel.

Twenty-three holes in all were bored, the total cost of well boring, casings, temporary trestle over river and segregation prevention being less than $2.50 per cubic yard of concrete. Experience indicates that at least one-third of these holes might have been omitted. Where a 60-ft. head is obtainable the holes may be safely spaced at 80 ft. and with a certainty that all of the cavities will be thoroughly filled.—From "Enginering News-Record," New York.

PROTECTING IRON AND STEEL STANDPIPES FROM CORROSION

IN an attempt to bring together such information as may be obtainable, relative to actual experience with standpipe paints, Metcalf & Eddy, consulting engineers, of Boston, Mass., sent inquiries to water works in the northern and eastern parts of the United States where iron or steel standpipes are in use, requesting information as to the dates of painting, kind of paint used, preparation of the surface for painting, and other significant data. In general, the water works superintendents were generous with their responses, which were received during the winter of 1916-17, but in a disappointingly large number of cases the responses were, in effect—"The writer has been here but a short time. The former superintendent left no records and I do not know what paint was used or when it was applied."

The significant data received in response to this inquiry, together with some information available from other sources, were given in a paper read last week by C. W. Sherman, of the firm of Metcalf & Eddy, at the convention of the New England Water Works Association. Mr. Sherman's data cannot be summarized or averaged. In a few cases the standpipe had not been painted since erection. There are doubtless cases in which the quality of the water is such that it protects rather than corrodes the metal; such cases are, however, rare and experience seems to indicate that as a rule an iron or steel standpipe should be thoroughly cleaned and painted inside at least as often as once in four or five years, says Mr. Sherman. Perhaps, however, if the cleaning were as thorough as would be accomplished by means of the sand blast, and if a protective coating where then properly applied, the interval between paintings could be materially extended.

A. H. Kneen, of Philadelphia, painted two standpipes with vertical stripes of different kinds of paint, and observed the conditions after two or three years of service. The only paints which gave reasonably good service were a red oxide of iron paint, and red lead. Most of the paints used were undoubtedly proprietary articles, and although one red oxide of iron paint showed "good" or "very good" results, several others characterized the same way in Mr. Kneen's tabulation gave "bad" or "very bad" results. Red lead when used was employed only in the first coat, yet the results were characterized as "good" or "very good." It is not impossible, therefore, says Mr. Sherman, that two or three coats of red lead would have shown better protection than any of the other paints.

Practical Results Confirm Experiment

Mr. Kneen says: "Under the writer's supervision, we have had the inside of fifteen standpipes painted and repainted in the past seven years, the combined area of which is over 112,000 sq. ft., and the results obtained are similar to those obtained in the experimental tanks. Our conclusion to date is that paints having pitch or asphalt for a base, applied in this manner, are not suitable for iron in contact with water, but that two-coat work with a good quality of iron oxide or a paint with a good red lead base for the first coat, and an iron oxide paint for the second coat, if the iron is properly cleaned and the paint is properly applied, will protect the iron in contact with water for at least four years, unless the water is of unusual quality or the conditions are different than usually met with in water works practice."

The engineers of the Metropolitan Water Works (Boston) have adopted the policy of cleaning the steel by sand blast, and applying three coats of red lead to the interior surface of their steel standpipes. The following quotation from the fifteenth annual report of the Metropolitan Water and Sewerage Board describes the cleaning and painting of the Bellevue Hill standpipe:—

"On March 25th, the tank was taken out of service and drained, and the work of sand blasting and painting the steel was begun March 30th, by the W. L. Waples Co., of Boston, subconstractor for this work. The plant used consisted at first of a twenty horsepower gasoline-engine-driven air compressor, a compressed air reservoir, two lines of air hose and two nozzles. Sand blasting was stopped in the latter part of the afternoon of each day, and the cleaned surfaces were painted before rusting commenced. The force employed included one foreman, two painters, who also did sand blasting, and one helper. This force could sand blast and paint an area of about 330 sq. ft. per day. Later, in order to increase the rate of progress, a second compressor, operated by an automobile engine, was installed on April 19th. The entire inside and outside surface of the tank, having an area of 35,650 sq. ft., was sand blasted and painted. Work was completed June 14th, with the exception of the third coat on the outside, which has been deferred until after the masonry tower which encloses the tank in completed. All painting materials were furnished by the department, but were mixed by the contractor under the direction of the engineer. For the inside of the tank, National Lead Company's red lead in oil paste, litharge, and Spencer-Kellogg & Son's boiled linseed oil, were used; the first coat natural color, the second and third coats tinted with lampblack in oil. For the outside of the tank, red lead paste, raw linseed oil, and drier were used for the first coat, and for the second coat white lead, raw linseed oil, turpentine, and drier tinted with lampblack were used. One gallon of red lead paint was sufficient to cover 700 sq. ft. of surface with one coat. The subcontractor's price for sand blasting and painting the tank was $1,600. . . ."

Paint Immediately After Sand-Blasting

Mr. Sherman inclines to the opinion that the method just quoted represents the best practice of the present time. He would put especial emphasis on the thorough cleaning of the metal, and the *immediate* application of the paint to the cleaned metal before the latter has cooled and moisture has condensed upon it. The following quotation is pertinent in this connection:—

"Every specification for painting bristles with clauses prescribing what shall or shall not be done, and still the fact remains that there are more failures than even indifferent successes, especially on work painted at the shops before shipment. The causes for the irregular and indifferent results are not difficult to ascertain. They are the improper application of the paint to dirty, greasy, moist or chilled, rusty or mill-scaled surfaces. No marked improvement in these uncertain results can be had until the same importance it attached to the 'paint question,' not only on paper, but in the actual supervision of the painting in all of its stages, as is given to the minutest construction details."

With regard to the coating to be applied to the metal after cleaning, it appears probable that not only red lead, but several of the graphite and red-oxide-of-iron paints, and perhaps certain enamel-like coatings, will give satisfactory protection.

10,000,000-GAL. RESERVOIR AT DAYTON

IN the year 1917-18, a covered masonry reservoir of 10 million gallons capacity was built for the water department of the city of Dayton, Ohio, to equalize the hourly fluctuations in water demand, to facilitate the making of minor repairs at the pumping station, and to provide fire service storage capacity corresponding to the 1916 standard of the National Board of Fire Underwriters. This reservoir was described by Leonard Metcalf and William T. Barnes, both of the firm of Metcalf and Eddy, of Boston, Mass., in a paper read last week at the convention of the New England Water Works Association. Following is Messrs. Metcalf and Barnes' summary of the points covered by their paper:—

The variation in rate of pumping at the main pumping station, to meet the demands of domestic, commercial and industrial water service, was from about 6 million to 35 million U.S. gallons per day. The storage required to equalize the hourly variations was approximately 3 million gallons, or 25% of the average daily water consumption.

The fire supply storage capacity required by the National Board of Fire Underwriters (1916 standard), corresponding to a ten-hour flow at maximum fire demand, was approximately 7 million gallons.

The sum of these two amounts (3 million and 7 million gallons) determined the capacity of the structure, which was reduced to the minimum on account of the difficulty and high cost of construction during war times.

The reservoir was built of concrete with walls of gravity section; inverted groined arch floor with 8-in. crown thickness; and 26-in. reinforced cylindrical concrete columns varying in length from about 20 ft. to 26 ft., supporting a flat-slab concrete roof 8⅜ ins. thick, reinforced with corrugated bars on the two-way system.

The original design prepared by the engineers for this work contemplated the construction of a groined arch floor and roof type of structure, with lighter walls than those finally used. This design was modified, however, on account of war conditions which made it impossible to secure outside bids and on account of the fact that the local bidders were more familiar with and better equipped to build the flat-slab roof construction and were therefore ready to submit relatively lower figures upon this type of construction than upon the groined arch structure. The unusual saving in cost here found justified the acceptance of a type of structure believed to have a somewhat smaller factor of safety and shorter life.

The total cost of the work to the contractor, excluding profit but including the cost of the materials furnished to him by the city, was $126,805, equivalent to $14.50 per cu. yd. of concrete, $12,680 per million gallons of net storage capacity, $2.08 per sq. ft. of net water surface, and $90,600 per acre of net water surface (the water surface being approximately 60,917 sq. ft., or 1.4 acres, and its depth averaging approximately 23.2 ft.).

The total cost to the city, including the land and engineering was approximately $153,041, equivalent to $17.50 per cu. yd. of concrete, $15,300 per million gallons of net water capacity, $2.51 per sq. ft. of water surface, and $109,300 per acre of water surface.

The character of the finished work was excellent, the concrete being dense and reasonably smooth.

The leakage of the finished structure was found to be 2 5/16 ins. vertically, or 88,000 gals. per day; and three months after putting the reservoir into service, 1¼ ins. vertically, or 48,000 gals. per day.

RECENT PUBLICATIONS

LIFE SAVING STATIONS.—An interesting folder calling attention to the fact that typhoid fever and similar intestinal diseases are caused by pollution of water supplies and that sterilization plants are really life saving stations, located not at the seaside but on reservoirs, streams and pipe lines. Issued by the Wallace & Tiernan Co., Inc., 349 Broadway, New York City.

WEBER CHIMNEYS.—Catalogue, 56 pages and cover, 5 by 9 ins., illustrating and describing reinforced concrete chimneys, including a number in Canada. The frontispiece is a photograph of what is claimed to be the highest chimney in the world, designed and erected by the company at Saganoseki, Japan; height 570 ft.; inside diameter at the top, 26 ft. 3 ins.

YARHOLA OIL PIPE LINES.—Description of the oil pipe lines of the Yarhola Pipe Line Co. extending from near Healdton, Oklahoma, to the Wood River Refinery, near St. Louis, Mo. This line is the property of the Roxana Petroleum Co., one of the producing companes of the Royal Dutch-Shell Co. The pipe line was designed and built by Sanderson & Porter, consulting and contracting engineers, New York, who are the publishers of the pamphlet. Illustrated; 18 pages and cover; 8½ by 11 ins.

JEFFREY STRAITFLO VENTILATORS.—Bulletin No. 270 of the Jeffrey Manufacturing Co., Montreal, P.Q., and Columbus, Ohio; 8 pages, 7½ by 10½ ins.; printed in two colors on coated paper, illustrated. This type of fan is equipped with a central disc about half the diameter of the wheel, on which is mounted blades designed so that the expulsion of the air will be the same at all points. It is used for drying or boosting purposes and also for removal of dust, heat and smoke from factories and other buildings.

ST. LAWRENCE RIVER POWER CO.—Interim order of the International Joint Commission in the matter of the application of the St. Lawrence River Power Co. for the approval of the construction and maintenance of a submerged weir in the south channel of the St. Lawrence river near the mouth of its power canal at Massena, N.Y.; 302 pages, folded maps and cover; 6 by 9 ins. The appendix comprises the larger portion of the book, presenting the detailed stenographic report of the evidence taken at the hearings.

HYDRAULIC TURBINE INSTALLATIONS.—New catalogue issued by the S. Morgan Smith Co., York, Pa., and Montreal, P.Q.; 64 pages and cover, 8½ by 11 ins., printed on coated paper in two colors and profusely illustrated. A number of views are shown of the company's various shops and foundries. A brief reference to the importance of runner and case design, and an explanation of type characteristic, together with efficiency contour charts, add to the reference value of the catalogue. The company's governor and pressure regulator are described, also the Gibbs oil thrust bearing, while the construction and erection of hydraulic turbines is described in considerable detail, illustrated by line drawings and half-tone views of numerous installations in Canada and the United States. The half-tones are beautifully printed and are typical of modern hydro-electric developments.

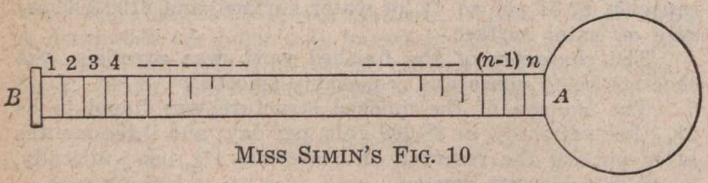
PRESSURES IN PENSTOCKS CAUSED BY THE GRADUAL CLOSING OF TURBINE GATES*

By Otto V. Kruse
Hydraulic Engineer, Larner-Johnson Valve & Engineering Co., Philadelphia, Pa.

THE writer has studied the phenomena of water-hammer, and offers a short analysis of Miss O. Simin's translation of Prof. Joukovsky's notable work, and an elaboration of certain parts of Mr. Gibson's paper.

Any pipe under pressure and containing water in motion may be assumed as a unit of energy under a condition of equilibrium. If we wish to stop the column of water, it means the application of a certain amount of force during a certain time to bring about the conversion of energy and restore the equilibrium. In analysing the phenomena which take place, the writer believes that the most important point to study is the rate of destruction of velocity as affected by the net head, because this determines to a large extent the rate of pressure rise and hence the maximum pressure rise. Apparently, Allievi used this as a basis for the development of his formula, and this, of course, applies to Johnson's formula, which is the same as Allievi's. Any formulas, such as Warren's and Vensano's, which do not take this into account, must be incorrect, except for one set of conditions. The curves given by Mr. Gibson show how far from the truth they may be.

In accordance with the ordinary methods of integration, it may be considered that the gate movement throughout the duration of the stroke is made up of an infinite number of small instantaneous movements. In the case of the formulas



Miss Simin's Fig. 10

of Allievi and Johnson, which involve the dynamic forces, each little movement of the gate produces a pressure:—

$$h = Ldv/gdt$$

The conditions existing, then, at the end of any small movement are:—

1. The original velocity has been decreased by the amount of the velocity destroyed during this first movement;
2. A dynamic pressure has been created due to the destruction of velocity.

A new velocity now exists, which is dependent, not on the original net head, but on a new head made up of the original net head and the dynamic head. As brought out by Mr. Johnson, these curves of dynamic pressure-rise and decrease in velocity may be calculated by a simple method of arithmetic integration, using only the relation, $h = Ldv/gdt$. The Allievi or Johnson formula will give a figure for maximum pressure rise, and Mr. Johnson has also developed a formula for the shape of the pressure curve, these formulas giving the same results as can be obtained approximately by arithmetic integration.

The question to be considered now is: To what extent, and how, do the compressibility of water and extension of the pipe walls affect the pressure and velocity curves? We know that, as the time of closing grows shorter, the Allievi formula approaches infinity. On the other hand, Joukovsky proved that the maximum pressure which obtained for instantaneous closing depended on the velocity destroyed and the speed of propagation of the pressure wave, and could not exceed "maximum water-hammer." It is not difficult to see that, for a long closing time, the compressibility of the water and extension of the pipe walls have very little effect on the maximum pressure, but, as the time of closing grows shorter, the properties of the materials have a greater and greater effect. The writer believes that Mr. Gibson's formulas have successfully supplied this missing gap between Allievi's and Joukovsky's theories, and serve to give correct results for all conditions.

Professor Joukovsky made a great many experiments to determine the magnitude of maximum water-hammer and the speed of propagation of the pressure wave, all based on instantaneous closing of the gates. The details of the experiments will not be given here, but the following is a synopsis of the theory, taken verbatim from Miss Simin's translation:—

"In Fig. 10, let A B be a pipe, in which water flows with velocity, v, from the origin A, past the gate B. If, now, the flow is suddenly stopped by a rapid shutting of the gate, B, the kinetic energy of the water column, A B, will cause an increase of pressure in the pipe.

"Let us consider the column of water, A B, as divided into n very small equal sections, 1, 2, 3,.... $(n-1)$ and n.

"The phenomena of water-hammer take place in a series of cycles, each consisting of four processes, as follows:—

"(1) Section 1, meeting, in the gate, an obstacle to its movement, will be compressed and will stretch the pipe wall surrounding it. All the kinetic energy of this section of water will be used up (a) in its own compression, resulting in the increase of pressure by an increment, P, and (b) in the corresponding stretching of the walls in section 1 of the pipe. As a result of this action, section 1 of the water column has left vacant behind itself a small space, to be occupied by a part of the next arriving section 2. Consequently, it is only after section 1 has been stopped and compressed, and after the small space thus left has been filled, that section 2 can be arrested and compressed.

"Now the kinetic energy of section 2 must be expended in some way. Will it increase the pressure upon the gate, which has already been caused by the arrest of section 1? No, and for the following reason:—

"The pressure upon the gate depends entirely upon the pressure, P, sustained by section 1, which is now in static condition.

"The pressure upon the gate could therefore be increased only if section 1 could be farther compressed, and this could take place only if the pressure upon the surface between it and section 2 (which we may imagine to be a thin piston) could be greater from the side of section 2 than it is from the side of section 1; and this is impossible, because section 2 has only the same kinetic energy as section 1, and this energy will (as in the case of section 1) be used up entirely in compressing the water of the section (section 2) only to the same additional pressure, P, and in stretching that part of the walls surrounding section 2.

"The same is true of each following section, 3, 4,.... $(n-1)$ and n; each of these sections, as it is arrested, being compressed to pressure, P.

"During process (1) a small quantity of water flows from the reservoir into the pipe, to occupy the space formed by the compression of the water and the extension of the pipe walls.

"Finally, when all the sections have been arrested, the entire column will be under the pressure, P. The entire energy of the water column is now stored (as potential energy) in elastic deformation—viz., in the compression of the water column and in the extension of the pipe walls.

"But this condition cannot be maintained; for

"(2) As soon as the additional pressure, P, has been produced in the last section, n, the water in that section will again expand, and the walls of that section of the pipe will again contract, restoring the original conditions in that section, and pushing the water of that section back into the reservoir from which the pipe issues, and restoring the original normal pressure in section n.

"The operation will now be repeated by each section $(n-1)$,....4, 3, etc., in turn, until all the potential energy, stored in the water column when it was under the pressure, P (neglecting the portion lost in friction), has been reconverted into kinetic energy.

"During process (2) the water which entered the pipe during process (1) is forced back into the reservoir.

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

"The condition of the water column is now what it was just before the gate was closed, except that its velocity, v, has now the opposite direction—i.e., toward the origin.

"(3) The kinetic energy of the water column, moving toward the origin or away from the gate, is now reconverted into potential energy, which manifests itself in an extension of volume of the water to a subnormal pressure beginning with section 1, and concluding only when the entire water column has been reduced to the subnormal pressure.

"During process (3) water continues flowing from the pipe into the reservoir.

"(4) When the subnormal pressure has been established throughout the length of the pipe, and all the water has come to rest, the water from the reservoir will again direct itself into the pipe, restoring the normal pressure, first in section n, next to the reservoir, and then, in rapid succession, in the other sections $(n-1)$,....4, 3, etc., until, when the normal pressure reaches the gate, we have once more the conditions which existed just before the gate was closed—viz., the normal pressure is restored and the water is moving toward the gate with the original velocity, v.

"We have now followed these pulsations of pressure (with the accompanying transformations of energy and flow of water into and back from the pipe) through a complete cycle of four movements, each extending through the length of the pipe. For convenience, we may consider two successive movements of this kind as a 'round trip' through the pipe.

"The gate remaining closed, the whole process is now repeated in a second cycle, which, in turn, is followed by a third, and so on, the amplitude of the pressure vibrations gradually diminishing (because of friction) until the pipe and the water come to a state of rest.

"But although the intensity of the pressure becomes gradually less, the time required for each cycle remains constant for all repetitions.

"This propagation of pressure, consisting of its transmission through all points of the length of the pipe, each point successively repeating the same periodical movements is, in its nature, simply a case of wave motion, like that of a sound wave.

"The velocity of wave propagation is independent of the intensity of the pressure, and depends only upon the properties of the medium through which the propagation takes place—in the case of water-hammer, upon the elasticity of the water and of the pipe."

Mr. Gibson has applied this theory to slow-closing gates by considering that each infinitesimal movement of the gate is instantaneous and produces a small rise of pressure which travels through the penstock in the same manner that a wave produced by total instantaneous closing would travel. These small gate movements occur in succession and produce in turn their pressure waves which travel back and forth through the penstock. An algebraic sum of these waves gives the rise of pressure existing at any time during the closing stroke.

The writer is of the opinion that the pressure existing throughout the length of the pipe for a slow-closing gate varies almost directly from a maximum at the gate to zero at the point of relief, except where the time of closing is less than $2L/a$.

In Fig. 11, the pressure existing at four points on the penstock has been plotted. The magnitude of the individual pressure waves is not to scale, but their values are shown for the particular example taken. The pressure existing at a point $L/2$ from the gate is approximately one-half the pressure existing at the gate. The pressure existing at a point $3L/4$ from the gate is approximately one-quarter of the pressure existing at the gate.

The diagram showing the pressure existing at the point of relief consists of a series of instantaneous rises above normal. On account of the assumption of twelve instantaneous movements of the gate, the pressure at the point of relief must of necessity show twelve instantaneous rises of pressure, as each wave travels undiminished to the point of relief. If twenty-four instantaneous movements were chosen, there would appear twenty-four instantaneous rises of pressure in the same time, but of only about half the magnitude. Hence, in the limit of an infinite number of instantaneous movements of the gate, or uniform slow-closing, the pressure at the point of relief becomes zero. This must be true, because the duration of time of any rise of pressure at the point of relief is zero. Wherever rises of pressure have actually been recorded at the point of relief on a penstock, they are probably due to an instantaneous movement of the gate during the stroke, or to some other cause, such as a sudden elastic deformation of the waterway, which would have the same effect.

In the translation of Professor Joukovsky's work there are shown tables of pressure rises at different points on the pipes for different velocities destroyed. In every case the pressure waves travel practically undiminished almost to the point of relief. The readings taken at the station nearest the point of relief, however, show a reduction in pressure of from 60% to 80%. This is very significant to the writer, as an indication that the pressure travelled undiminished to a certain point and then gradually was reduced to zero at the point of relief. Although Professor Joukovsky experimented with very rapidly closing gates, as nearly instantaneous as possible, obviously a certain fraction of a second must elapse while the gate is closing. When the time of closing is less than one interval, $2L/a$, the resultant pressure will travel undiminished to a point which is $Ta/2$, in feet, from the

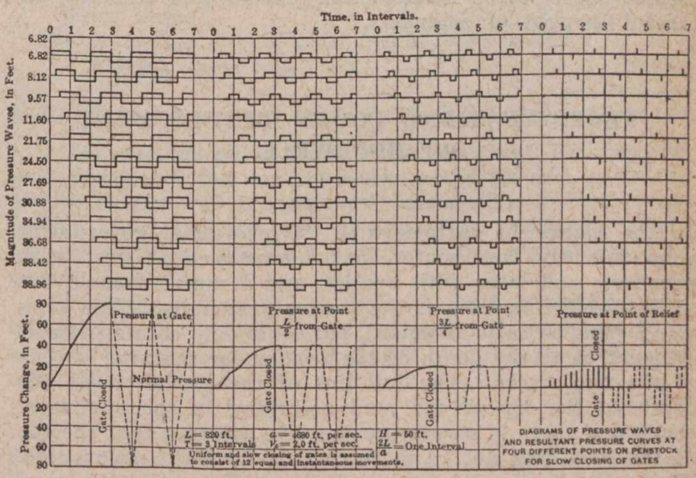


FIG. 11—PRESSURE AT FOUR POINTS ON PENSTOCK

point of relief (where T is equal to the closing time, in seconds). From this point it will gradually be reduced to zero at the point of relief. Granting that some short period of time must elapse during the closing of the gates in Professor Joukovsky's experiments, his actual pressure readings seem to verify the writer's claim.

In the example taken, the duration of closure is assumed to be three intervals. The diagrams and curves have been plotted beyond the three intervals to show the fluctuations of pressure which take place after the gates are closed. Experiments should show that the subsequent fluctuations after the gates are closed are quickly reduced in intensity by internal friction, but this friction should have no effect on the speed of propagation of the pressure waves. Internal friction has of necessity been omitted in these computations, as it is an unknown quantity.

Any other example than that shown by Fig. 11 may be chosen, and the pressure waves may be computed by Mr. Gibson's formulas. A series of diagrams may then be plotted similar to those shown by the writer, and the pressures may be computed for any point on the penstock.

In making the computations for the magnitude of the pressure waves, it is necessary to determine first the number of intervals of time (equal to $2L/a$) contained in the total closing time. The gate movement may then be divided into a number of instantaneous movements equal to the number of intervals. Computations carried out on this basis will give correct points on the pressure-rise curve at the end of each interval. In the event of the closing time being only a few intervals, it is desirable to subdivide the gate movement further in order to develop the curve during the intervals. It is an interesting fact, however, that such further subdivision of gate movement serves only to develop the shape of the curve during the intervals, and has no effect on the values at the end of each interval. In making studies of this character, it is often easier instead of using Mr. Gibson's formulas, to utilize the system of arithmetic integration, which will give correct results, although involving trial-and-error methods.

In order to compute the pressure-rise curve at points on the penstock other than at the gate, the gate movement must be divided into a number of divisions sufficient to cause the pressure waves to overlap. Thus, at least two divisions per interval must be used to compute the curve at a point half way up the penstock, at least four divisions per interval for a point three-quarters of the way up the penstock, and at least eight divisions per interval for a point seven-eighths of the way up the penstock. Referring to Fig. 11, where four divisions per interval were chosen, it is obvious that the pressure beyond the point, $3L/4$ from the gate, cannot be computed without a greater subdivision of the gate movement.

In this discussion the writer has endeavored, principally, to point out the trend of argument and application of existing data used in proving the theoretic correctness of Mr. Gibson's method of handling water-hammer problems. The method seems to be fundamentally sound, and deserves the careful thought of engineers.

PROPOSED TIDAL POWER DEVELOPMENT AT HOPEWELL

(Continued from page 366)

up the matter of borings with the provincial government very urgently last winter, but I was met at first with promises, and later with apathy, and I regret I could not undertake the expense of the matter personally. This summer I went twice to Hopewell, but could only obtain indirect evidence as to the depth of the rock under the muddy beds of the two rivers. The full line of the profile shows the depth of the top of the mud according to the admiralty chart, and there is strong presumptive evidence that the top of the rock does not lie more than 15 to 20 ft. below this. Where the "new wharf" was built at Hopewell, the mud soon washed out for a depth of 12 ft., when hard bottom was reached, and the outcropping of ledge rock at the mouth of the Memramcook indicates that rock bottom for a quarter of a mile is very near the surface. The shores at Hopewell, Fort Folly Point and Cole's Head are all rocky, and the nature of all rock in this locality is the same, viz., shaley sandstone to pure sandstone, sufficiently sound for making grindstones.

As to the best type of dam to build at Hopewell, I feel that I should offer no opinion, for the question of dam design is an engineering specialty and only an expert in this particular branch of engineering could decide the best type to adopt, and he only after a systematic line of borings were obtained along the proposed site. It has been suggested to me that a dam composed of hollow sections of concrete is a satisfactory and cheap type to build, the sections being built in a dry dock, floated into position and then sunk by filling the interior with rock and gravel. However, I doubt if this is a well proven system and, as I say, the question of best type should be decided by an expert of long experience.

Such estimates of cost that I have been able to make have been based on the cyclopean concrete type of dam. In considering the best type to build at Hopewell, the question of the tidal currents would have to be considered, and while these currents are not very swift, when the great height of the tide is considered, nevertheless they should receive attention. Fig. 4 shows some current measurements that I made this summer by using a ship's log attached to an anchored boat. When the tidal range was 38 ft., the channel current reached a maximum of 4.2 knots and you will note that although the flood current soon dies out, the ebb persists at near its maximum until almost the time for the next flood tide to begin. Out of the channel the current runs swiftly for much shorter periods, and the tidal current makes shoreward as the shores are approached, until we often have currents at right angles to the main stream. The dotted curve shows some measurements taken rather off the channel, with a tidal range of 28 ft., and the maximum under these conditions was 2.6 knots.

Before leaving the engineering problems that are presented by this novel plant, I will refer to three other items that should be considered.

The question of subnormal neap tides requires especial attention, for while my calculations are based on the least range, viz., 32 ft., of ordinary neap tides, nevertheless there are certain tides which occur sometimes three days a month, sometimes five days a month, and sometimes not at all in a month, which have a lower range than 32 ft., and may sometimes have a range as low as 24 to 25 ft.

Subnormal Neap Tides

Now, if our plant were built and running at full capacity, with a range of 32 ft., there would be an impairment of the regular capacity whenever these subnormal neaps occurred (which is about 15% of the time), and I think some provisions should be made to deal with them adequately.

One method might be to keep the turbines and generators well ahead of the normal demand and use these extra ones only during the subnormal neaps. Another method would be to keep the Memramcook shovelled out (as already described) well ahead of the regular requirements, and thus improve the average head. Yet another method would be to build a freshwater dam (say just below Turtle creek, about 5 miles west of Moncton) in which freshwater would be impounded and released only to make up the deficiency in head at subnormal neaps. Still another method would be to employ auxiliary steam power to assist the water-power during the water deficiency. One of the last two devices is nearly always resorted to in the case of ordinary hydro-electric plants situated on fresh water rivers. In nearly all districts the amount of rainfall varies enormously during the different months of the year, and the amount of run-off and discharge varies in a direct relation to the rainfall. As an example of this, the discharge at Grand Falls on the St. John river reached a maximum in May, 1909, 50 times greater than the maximum of October, 1900, and the mean discharge for the whole month of May was 20 times greater than the mean for October. In fresh water rivers a certain power may run into thousands of hosepower in the spring, but be reduced to hundreds in the fall of the year unless adequate means are resorted to in order to increase the deficiency of head.

In this matter, a tidal power scores heavily over a fresh-water power. In the case of the freshwater power, neither the time nor quantity of head deficiency can be predicted, but with a tidal power, both the amount and the deficiency are predicted by the tide tables several years in advance, and it will thus be much easier to provide for our head deficiency, which only, after all, amounts to 25% in quantity and occurs only 15% of the total time.

The best means of making up the deficit in the case of the Hopewell Tidal plant would be best figured out in the final estimates as that one which would maintain the normal output at a minimum of cost.

Sediment and Ice

The two other remaining engineering items are sediment and ice. At present the never-ceasing current flows up and down the two rivers, keeps the river mud stirred up, and the water of both rivers shows a considerable amount of sediment, and one would at first jump to the conclusion that this muddy grit (fine though it is) would produce much unusual wear on the turbines. However, what will happen as soon as the mouth of the Petitcodiac is closed by a dam? The mud in the water above the dam will undoubtedly settle, for it will have time to do so, and the water of the Petitcodiac will become clear. At present the water is never still, but with a dam, the rise and fall would be reduced to a few feet and the current would be sluggish.

In a similar way, the building of the dam would entirely alter the ice conditions. Once the dam was built, the whole river would freeze over in severe weather, and the sheet of ice would rise and fall with the water just as it does in the St. John and similar rivers. Nevertheless, while the building of the dam was in progress, the question of large masses of ice moving with the current should be given every attention, and quite likely some special method of cushioning the blows from ice might have to be devised.

Considering the method of distributing the power available at Hopewell, Fig. 7, shows how centrally Hopewell is placed with reference to the centres of population of both New Brunswick and Nova Scotia.

The method of transmission carries with it no special problems, as we would undoubtedly use step-up transformers at the power house, transmit at about 30,000 volts with 3-phase current, and use step-down transformers at the delivery points. All this system has been so thoroughly thrashed out, and is in daily use all over Canada and under Canadian winter conditions, that it has become practically standard and needs no special consideration here.

Population and Power Requirements

The principal feature that should be considered in laying out the transmission lines would be one of expediency and proper return on the capital outlay of transmission lines and line losses. Undoubtedly, a main line should go west to St. John, with power for all intervening towns and villages of any size, and undoubtedly a main line should go east and south to Halifax with power for Sackville, Springhill, Amherst, Truro, etc., and also branch lines should go to Moncton, New Glasgow and Stellarton, as the present population would warrant this. The extension of branch lines to St. Stephen, Fredericton, Newcastle and Chatham is somewhat doubtful at present, but there are railways that well might be economically electrified toward all these points, and I have therefore included them in the preliminary estimate. I have prepared a table of the population that would be served by these lines, and the total works out at 250,000 inhabitants. With this as a basis we should now estimate the probable per capita use and thus obtain an estimate of the proper size of plant for the initial development at Hopewell.

When I prepared my first report on this subject, it was submitted to the well-known firm of Sanderson & Porter, of New York, a firm who specialize in the control and management of some 60 hydro-electric developments, and their condensed criticism was as follows:—

"We think your proposal from an engineering standpoint is sound and it is the only scheme for utilizing tidal power that seems practical, but we are rather in doubt as to whether the population served is sufficiently large to warrant the capital expenditure."

Now this was in 1914, and I presume Sanderson & Porter based their opinion on the per capita use of hydro-electric power in the United States, which works out at 0.10 h.p. per inhabitant, according to their text books. On this early report, and on their opinion, I based the letters to the newspapers that were published in St. John and Halifax, and in these letters I appealed for governmental help, since it seemed at that time that while my proposal was almost a commercial one, it was not conclusively so.

0.36 H.P. per Capita

Since last January, however, newer data has come to hand, and this data is accurate, as I presume it is, the complexion of affairs has changed and the Hopewell plant is really a good commercial proposition at the present time, and needs only the governmental help accorded by a good charter.

The data that I speak of shows the per capita use of hydro-electric power for every province of Canada and for the Dominion as a whole. It shows that British Columbia uses 0.36 h.p. per capita, Ontario 0.288, Quebec 0.267, and the whole of Canada 0.206 h.p., and these figures are for the total population and they should be increased at least 30% for the per capita use of population served. It will thus be seen that Ontario, per capita served, is using about 0.37 h.p. and Quebec about 0.35 h.p., and I think we may properly assume that 0.36 h.p. would be used by the inhabitant of the Maritime Provinces as soon as he is given really cheap electricity.

On this basis, then, the population of 250,000 would require 90,000 gross horse-power, or, say, 45,000 h.p. at the delivery points, and I think the initial development at Hopewell should be for 90,000 h.p., with every provision made for increasing the output, as already outlined in this paper, up to 200,000 gross horse-power as the population and demand increased, as they undoubtedly would when cheap power was available.

Cost of Initial Development

Turning to the question of costs, I have made an estimate of this as follows, and in accordance with the text books on the subject:—

Dam Cost in Cyclopean Concrete

	Western Dam.	
730	lineal feet at equivalent height of 38 ft. and $137 per lineal foot	$ 97,000
4,100	lineal feet at equivalent height of 65 ft. and $300 per lineal foot	1,230,000
	Eastern Dam.	
2,800	lineal feet at equivalent height of 65 ft. and $300 per lineal foot	840,000
2,000	lineal feet at equivalent height of 35 ft. and $120 per lineal foot	240,000
	Wing Dam.	
900	lineal feet at equivalent height of 30 ft. and $94 per lineal foot	84,600
10,530 ft.	Total cost,	$2,491,600

One well-known engineer who examined my first report, considered this too low a figure and thought $4,000,000 should be allowed for dams, and since we will remain uncertain about the dam cost, until borings are made, it will be best to allow this higher figure in the preliminary estimates.

The other items of cost can be more accurately estimated from the known cost of equipment in existing hydro-electric developments, and we have the following table of total estimated cost:—

Dams	$ 4,000,000
Lock in western dam	100,000
Sluices, gates, etc.	300,000
Power house of steel and concrete	200,000
Turbines, generators, etc., at $40 per gross h.p.	3,600,000
Transmission lines, etc.,	1,200,000
Preliminary dredging, dam, trench, etc.	100,000
Promotion, engineering fees, etc.	500,000
Auxiliary plant to supply head deficiency at sub-normal neap tides, say	1,000,000
Total cost of initial development to produce 90,000 gross h.p.	$11,000,000
Or cost per h.p. developed	$ 122.50

Cost of Subsequent Enlargement

The cost for subsequent development is much less, relatively, for in the initial development full allowance has had to be made for the final development in all items except turbines, generators and transmission lines, and the cost of a full development of 200,000 gross h.p. would work out at about $16,000,000, or $80 per gross h.p. In this estimate for final development, the cost of shovelling and dredging the Memramcook farming lands (which would be necessary as previously shown) is not figured, for it would be undoubtedly good policy to start this work as soon as the power house was in operation, and take the necessary cost out of the annual income.

In considering the charge that could probably be made to the consumer initially, we must consider the cost of steam power to the provinces, and I have it on good authority that the actual cost to the largest producers of electricity, by steam, was $60 per h.p.-year before the war, and would be about $90 per h.p.-year at the present time.

Suppose, now, we were to charge half this rate on the average, and place our charge at $45 per h.p.-year, this would be the average charge and the charge in special cases would work out about as follows:—

	Charge per k.w.h.
(1) To power users in excess of 1,000 k.w.h. per month and 24-hr. service	0.75c.
(2) To power users, in excess of 1,000 k.w.h. per month, and 10-hr. service	1.5 c.
(3) To small power users and 10-hr. a day service	2.25c.
(4) To power users, irregular service	3.0 c.
(5) To small lighting service	4.0 c.

These rates are from one-quarter to one-tenth the rates now charged in the Maritime Provinces. They would attract population and develop industries, and they could be gradually reduced, as the demand for greater output increased the size of the plant, to about 60% of these rates.

On this basis, then, the annual income should be $2,025,000 initially, and this would probably be apportioned about as follows:—

Depreciation and operating expenses at 60% on cost	$ 660,000
Royalty on patent rights at 50 cents per gross h.p. produced	45,000
Enlarging Memramcook valley by shovelling and dredging, the power being supplied by the plant itself, say	200,000
Incidentals	20,000
Interest on capital expenditure at 10% of cost	1,100,000
Total	$2,025,000

While $45 per h.p.-year would probably have to be the initial average charge, as time goes on and the output in-

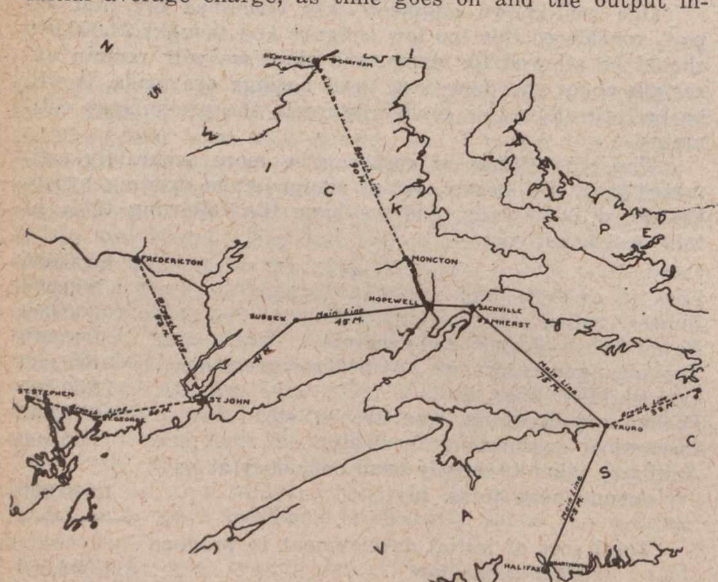


Fig. 7—Proposed Transmission Lines From Hopewell

creases to the full available horse-power, it would be quite possible to reduce the average charge per horse-power-year to $27, without decreasing the rate of 10% per annum on invested capital. This is figured as before, as follows:—

For gross output of 200,000 h.p., delivered h.p. 100,000, cost of development	$16,000,000
Annual income from 100,000 h.p. at $27 per h.p.-year	2,700,000

This income is apportioned as follows:—

Depreciation and operating expenses at 6% on cost	$ 960,000
Royalty on patent rights at 50% per gross h.p. produced	100,000
Enlarging Memramcook valley, nil, since work would by this time be completed	....
Incidentals	40,000
Interest on capital expenditure at 10% on cost	1,600,000
Total	$2,700,000

The Commission of Conservation's book on the "Water-Powers of Canada" (1911) says, in speaking of New Brunswick, that "the larger rivers, for the most part, are long and their fall is gradual"; and again, that "there are comparatively few lakes in the upper portions of the watersheds of the majority of the rivers and, hence, little facility is afforded for the natural storage of waters for the purpose of equalizing the flow during the low-water periods."

This is true of New Brunswick and it is also true of Nova Scotia. The upper provinces are steadily forging ahead of us, both industrially and in population, and this advance can be ascribed largely to their abundant water-powers, and it behooves us of the lower provinces not to mourn our loss of power, but to make full use of that great power that nature has really placed at our door—viz., the Bay of Fundy tides.

Were we to harness the power that is daily wasted in the two great eastern arms of this bay, we could have the use of 3,500,000 h.p., but this is a dream for the future and is not a practical proposition for to-day. The theory is the same as for the Hopewell plant, but the execution and cost would be impossible at present.

The Hopewell plant is to-day a good commercial proposition. Special engineering problems are attached to it, but they are only those that attach to any special plant. The difficulties are small in comparison to many recent engineering works that have been successful.

NEW ENGLAND WATER WORKS CONVENTION

Members of the New England Water Works Association met in Albany, N.Y., last week for their 38th annual convention, which occupied the four days, September 30th to October 3rd, inclusive. They were welcomed to Albany by that city's chief executive, Hon. James R. Watt, who is an engineer and contractor.

The presidential address of Samuel E. Killam laid stress upon the opportunities that exist for the association to exert a helpful influence in public affairs. Particular vigilance is needed, he said, in the matter of legislation affecting public health, and in each state a committee should be appointed to examine proposed legislation of this nature.

Over 300 members, associate members and guests were at the convention. The sessions were all well attended, and much interesting discussion followed the papers. The list of papers presented was published in *The Canadian Engineer*, September 25th issue.

The only paper on Canadian work was by Norman J. Howard, who described the construction and operation of Toronto's drifting sand filtration plant. This paper was discussed in a lively manner for more than an hour. James M. Caird, consulting chemist, took a leading part in the discussion. Mr. Caird particularly inquired regarding costs of operation and the ability of the plant to prevent the passage of alum into the filtered water.

Mr. Howard stated that cost figures are not yet available. As to the alum, he reported the presence of aluminum hydrate in the filtered water, but claimed that its presence is not significant, and that so far as he has been able to ascertain, it can be found in the effluent of all mechanical filtration plants, and that it is entirely harmless and inert.

Mr. Caird differed with Mr. Howard in this regard, but several of the consulting engineers and water works superintendents who were present supported Mr. Howard's statements, and considerable difference of opinion was expressed in regard to the point.

Morris Knowles characterized the Toronto plant as a radical departure in water works purification practice, and said that engineers are awaiting with great interest further detailed data on filtration costs.

All of the members and guests assembled on a Hudson River Line steamer on Wednesday, October 1st, and spent the entire day on the steamer, holding sessions during a trip down the river.

A number of the engineers enjoyed an automobile trip Thursday morning to the Gurley factory at Troy, where the Gurley long-distance water recorder was seen in actual operation.

The Canadian Engineer

Established 1893

A Weekly Paper for Civil Engineers and Contractors

Terms of Subscription, postpaid to any address:

One Year	Six Months	Three Months	Single Copies
$3.00	$1.75	$1.00	10c.

Published every Thursday by
The Monetary Times Printing Co. of Canada, Limited

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PRINCIPAL CONTENTS

PROPORTIONING OF CONCRETE MATERIALS

ENGINEERS and scientific investigators are delving deep in an effort to discover the fundamentals underlying the assembling and combining of the component materials of concrete; the relation of each of these materials to the strength and other physical properties of the final mass; and the effect of various conditions of placing, curing, etc., tending to influence the permanence of structures in which concrete is used.

The old adage that there is an adequate cause for every effect, and, conversely, an effect for every possible combination or condition, holds especially true of concrete. From the investigations to determine the why and how of all things pertaining to the subject, we have learned that concrete is not a complicated material but that far more care must be exercised in its use than we believed to be the case only a few years ago.

It has been proved that some of the practices now in common use are productive of very widely varying results. Of these, the arbitrary proportioning of the cement and aggregates by gross volumes is now claimed to be especially unscientific.

In this issue is published a discussion, by L. N. Edwards, formerly supervising engineer of bridges of the city of Toronto, of the paper, "Studies in Surface Area Proportioning Method," by R. B. Young, assistant laboratory engineer of the Hydro-Electric Power Commission of Ontario. Mr. Young's paper was first published in our issue of June 26th, 1919.

In the laboratory testing of concrete and concrete materials, no detail is of greater importance than that of securing a uniformly strong cement matrix binding together the particles of the sand and stone aggregates. In specifications for concrete construction work, no detail has been described so indefinitely. No ingredient of the mix performs a more important role than does the water content. It is, therefore, of special interest to note that Mr. Edwards strongly advocates the use, in both laboratory and field, of a method whereby the quantity of water to be used in the mix is mathematically determined. In this connection, it is his contention that the property commonly termed workability, flowability or plasticity, is a secondary factor rather than a primary one, and that it is largely dependent upon the character of the aggregates and the richness of the concrete mix.

In the issue of May 15th, 1919, we published an article, "Bulking Effect of Moisture in Sands," written by Mr. Edwards and containing a part of the results obtained from his tests which were at that time in progress. The more complete information secured from these tests, and more especially the relation existing between the area-water content and the bulking effect, described fully in his discussion of Mr. Young's paper, is well worth serious study, inasmuch as this bulking effect of moisture in sands may have some bearing upon the consistency formula proposed for use in testing concrete and concrete materials.

OUR GREATEST NATIONAL MENACE

IN past issues we have frequently referred to the seriousness of Canada's fuel problem. In this connection, in last week's issue, there was published the clearly expressed and statesmanlike address delivered by Arthur V. White, consulting engineer of the Commission of Conservation, at the annual meeting of the Canadian Gas Association. In view of the industrial and economic situation in Canada, we unreservedly agree with Mr. White in his statement that "whether conditions of curtailed fuel supply for Canada be delayed from materializing this coming winter or next winter, or until some time in the future, nevertheless I am firmly convinced that there is no menace to Canada's economic and general welfare at all comparable to the fact that she is at present so largely dependent upon a foreign country for her fuel needs. Without this foreign supply, Canada most assuredly would be put to desperate straits."

In other articles by Mr. White and many other writers, statistics have been given relating to the coal, oil and water power resources of the Dominion. The extent of the development of these resources has been shown; and the lines along which further development and conservation must be expected, have been indicated. If these various phases of the subject are not co-ordinated by some agency sufficiently powerful to take the fuel problem seriously in hand, such as a federal ministry of fuel, we may some day have "Fuelless Months" instead of "Fuelless Mondays."

It is idle to talk about the great natural resources of Canada unless those resources are to be actively developed. As Mr. White says:—"I shall not dwell on these enormous reserves. It seems out of place to emphasize how much we have latent when alongside of it we are not able to show how beneficially these assets are being used both for our own support and for the assistance of other needy nations. When the population of the prairie provinces are insured against yearly fuel shortage, we shall be more interested in hearing emphasis laid upon the enormous fuel reserves of these provinces; and when Canada produces more than her present amount of 2 per cent. of her total annual oil consumption, we shall be more interested in hearing emphasis placed upon the statement that we have the biggest oil fields in the world."

Canada's only sane policy is to develop as rapidly as possible both her fuel and power resources, and by co-ordination of transportation and other cognate agencies, to provide for the distribution and storage of fuel in all communities of the Dominion. As Mr. White says, in some respects it is more important to move coal and have it adequately stored and distributed throughout Canada, than it is to remove grain out of the country.

PERSONALS

CARLETON EMERSON DAVIS, who will preside at the convention of the American Water Works Association to be held next summer in Montreal, Que., is chief of the Bureau of Water, Philadelphia, Pa. He graduated in 1893 at the Massachusetts Institute of Technology. After graduation he was assistant engineer at Manchester, N. H., for one year, and then spent four years as assistant engineer on the construction of extensions to the water works at New Bedford, Mass. After several years as resident engineer on the construction of additional water facilities for Newark, N. J., Mr. Davis was chosen as engineer in charge of water works, sewers and municipal engineering for the Isthmian Canal Commission, Panama. In 1905, he was engaged by J. Waldo Smith, chief engineer of the Board of Water Supply, New York City, as head of the reservoir department in that city, which position he occupied until 1912, when he resigned to accept his present post at Philadelphia, which includes operation and maintenance as well as the assessment of revenues aggregating $5,500,000 per annum. Mr. Davis has also charge of all extensions and improvements to the Philadelphia water works. He was elected president of the American Water Works Association last June at the meeting held in Buffalo, N.Y. For the past twenty years he has been a member of the American Society of Civil Engineers.

A. FRANK WICKSON, of Toronto, has been elected president of the Royal Architectural Institute of Canada for the next twelve months. ALCIDE CHAUSSE, of Montreal, was re-elected as honorary secretary.

C. D. HARRINGTON has been appointed vice-president and manager of Anglin-Norcross, Ltd., a recent amalgamation of two large construction companies in Montreal. Mr. Harrington is an engineering graduate of McGill University.

H. T. ROUTLY, construction engineer of the Ontario Department of Highways, has resigned. It is stated that Mr. Routly tendered his resignation last July, but it was not accepted until October 1st. The duties of his office are being apportioned among the other members of the department's staff.

Letter to the Editor

QUANTITY SURVEYS vs. "PAYMENT"

Sir,—I notice in your issue of September 18th, an article outlining the "Kelley System" of payment to contractors for estimating. I have studied the so-called "Nelson" and "Kelley" plans, and although I may be somewhat dull, I cannot for the life of me understand how either plan could be proposed to take the place of the quantity surveyor.

At the present time, engineers and contractors are trying to get away from conditions that they recognize are unfavorable both to the profession and to the owners who have to pay for the work. The cost of living has in some way to be brought down from its present high level, and, owing to the high wages existing, it seems to me that the only way in which we can reduce the cost of building is to eliminate so far as possible these bad conditions,—eliminate the unnecessary duplication of our work and create efficiency in every department.

Following this argument it seems that the first bad feature to be eliminated is the taking off of quantities by each contractor and having this done by a competent man who specializes in this work, issuing plans, specifications and bills of quantities complete to each contractor tendering.

By introducing the quantity surveyor, every contractor figures on the same bill of quantities. We thereby eliminate the excessive cost of taking off the quantities by having *one man do the work once*, and not a dozen times or more according to the number of contractors figuring. It is very easy to be seen that every contractor figures (in his overhead charges) the cost of taking off quantities and estimating work which he never obtains. Therefore, those who pay for the work executed, also pay for the work which the contractor has done on other tenders of which he did not receive the contract. So, by introducing the quantity surveyor, we eliminate the duplication of work, and also lower the cost of such work to the owners.

Why should Mr. Kelley receive 7½% commission from the contractors? Why should the local agent receive 7½% commission? It seems to me that this system is what might be called, in somewhat slangy language, a "skin game," and, as far as I can see, would by no means tend to raise the status of the contractors in the estimation of the engineers and owners.

In the past many people have looked upon a contractor as almost a crook, and it is undoubtedly a fact that many men who are to-day in the contracting business are better equipped for other spheres of operation rather than the contracting business, and it seems to me that this so-called "Kelley" system would bring into operation many more men of that description.

The contracting business to-day is every bit as much of a profession as is engineering, and it is up to the contractors and engineers to wake up and put themselves in a position where they can claim the respect and admiration of the community. This cannot be attained by any easy-money scheme, such as the "Kelley" plan, to my mind, is liable to become.

The article in question makes no distinction between "taking off the quantities" and the "estimate of cost." We cannot eliminate the cost to the contractor of *estimating*, for he is the only man who can estimate the cost of his work, and this is absolutely a part of his business.

Quantity surveying, however, is a very intricate and necessary branch of the civil engineering profession, and we all know that work done by a man who specializes in any particular line is far better than that done by a "Jack-of-all-trades."

The whole of the questions in the "questionnaire" seem to point to the fact that the whole system is designed as a method of getting easy money and should not be recognized by any contractor, engineer or architect, it being detrimental to the profession in every way.

We can go just as far as we like along the lines of the "Nelson" and "Kelley" plans and never get anywhere but in bad repute with the public. Each plan proposes payment of money for work done which *is absolutely unnecessary work*, and the sooner we come to recognize the fact that the quantity surveyor is absolutely the one and only answer to the question, the the better it will be for the profession.

I think that all those in the engineering and architectural professions will soon see that it is to their advantage to insist on the quantity survey being put into general operation, and the societies of the professions to insist that a bill of quantities be issued together with the plans and specifications.

EDW. H. BECK,
Chief Engineer,
The Lake Huron Steel Corporation, Ltd.

Goderich, Ont., October 2nd, 1919.