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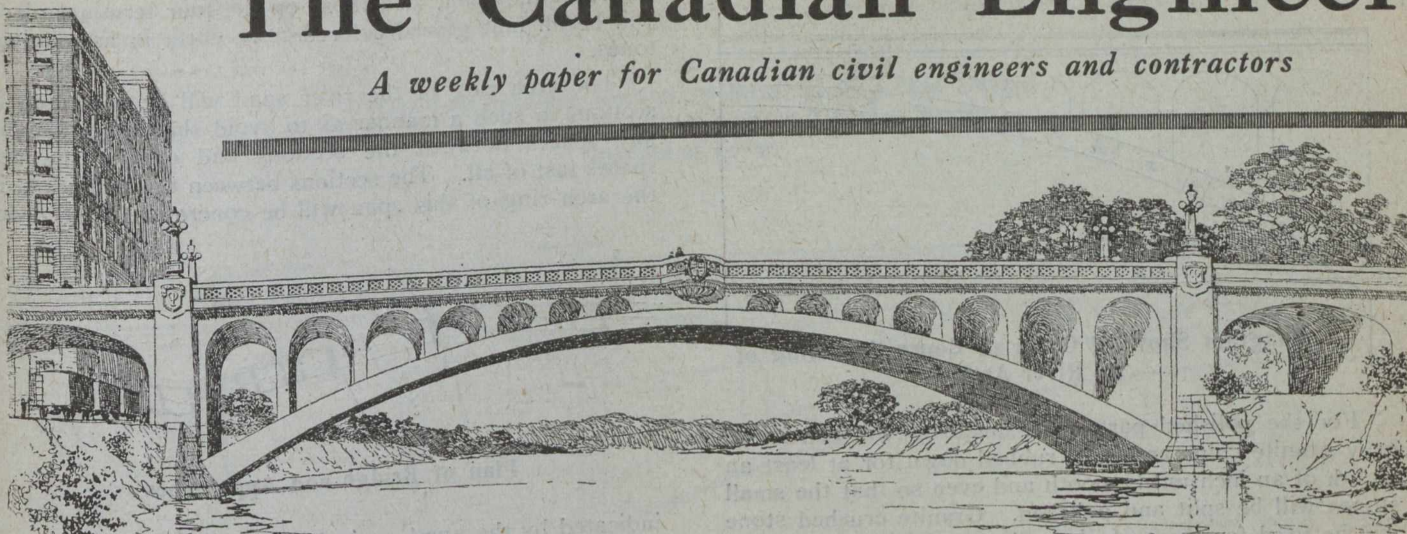
# The Canadian Engineer

*A weekly paper for Canadian civil engineers and contractors*

# Hunter Street Bridge, Peterborough, Ont.

**Will Contain Longest Clear Span Concrete Arch in Canada — Fifteen Arches Totalling 1,065 Feet in Length — Arch Ring of River Span Constructed With Temporary Hinges at Crown and Skewbacks — Heath-Edwards Method of Proportioning Concrete Specified for First Time—Cinder Fill Over Spandrel Arches Carries Pavement**

SEVERAL interesting points of design attract attention to the new high level reinforced concrete bridge that will be built within the next few months at Peterborough, Ont., besides the fact that it will have the longest clear span of any concrete arch in Canada.

The bridge will carry Hunter Street across the Otonabee River and across the Grand Trunk Railway tracks on the west bank of the river. It will replace an old steel bridge which is at the level of the railway tracks and will eliminate a dangerous grade crossing. The project is the result of an agreement between the Quaker Oats Co. and the city of Peterborough. Two advantages accrue to the Quaker Oats Co.: First, the shunting in their yards will not be hampered as previously by the Hunter Street traffic; and, second, an approach from the level of the bridge to the third story of their adjacent plant will enable them, without inconvenience, to have the main offices on the third floor of their building instead of on the ground level, thus somewhat escaping the dirt and confusion of the surrounding railway yards, and also permitting the lower two stories to be used entirely for shipping and receiving.

Frank Barber, consulting engineer, of Toronto, some time ago was requested by the city authorities to prepare plans and specifications for the new bridge, which will probably cost approximately $300,000. Tenders have been called and the contract will probably be let before the end of this year, work will proceed at an early date.

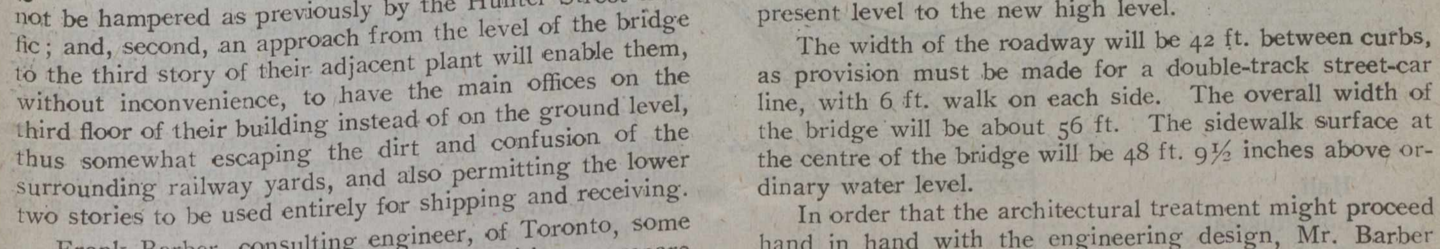
There will be about 17,000 cubic yards of concrete and 250 tons of reinforcing steel in the bridge. Over 7,000 cubic yards of excavation will be required, and about 14,000 cubic yards of fill. The main river arch will have a clear span of 235 feet. Besides the river arch there will be fourteen other arches, seven on each side of the river. The total length of the outline of the fifteen arches will be 1,065 ft., although the length of the entire bridge, including approaches from end to end of retaining walls, will be 1,750 ft.

The bridge will have but one deck, although an opening will be left in each approach pier to provide for foot traffic at the present level. Means are also being provided, by means of ramps, for vehicular traffic from the present level to the new high level.

The width of the roadway will be 42 ft. between curbs, as provision must be made for a double-track street-car line, with 6 ft. walk on each side. The overall width of the bridge will be about 56 ft. The sidewalk surface at the centre of the bridge will be 48 ft. 9½ inches above ordinary water level.

In order that the architectural treatment might proceed hand in hand with the engineering design, Mr. Barber arranged for association with Claude Bragdon, a well-known architect of Rochester, N.Y.

The entire bridge will receive a surface treatment of at least two inches of granite, placed next to the forms



simultaneously with the concrete backing. After hard setting, the entire surface will be either polished or chipped to bring out the desired lighting and color effects.

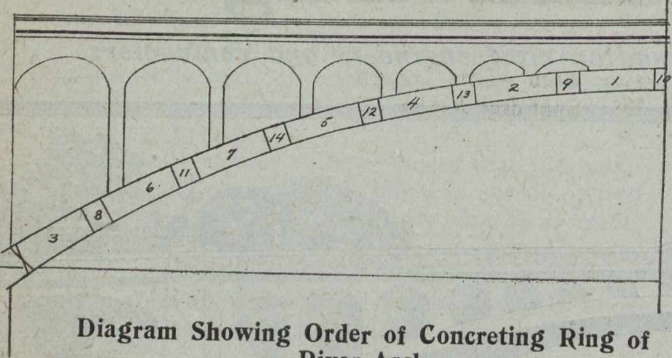


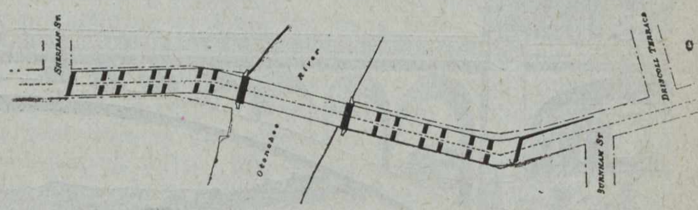
Diagram Showing Order of Concreting Ring of River Arch

For the polished parts, the surfacing will consist of gray granite chips and grit, rubbed down for at least an eighth of an inch until smooth and even so that the small stones will be split and polished. Granite crushed stone will be used for the surfacing where chipped finish is required, and this will be chipped in such a manner as to expose the stone aggregate.

As the total surface area of the bridge will be less than 9,000 sq. yds., this treatment will be an incidental expense compared with the whole cost of the structure.

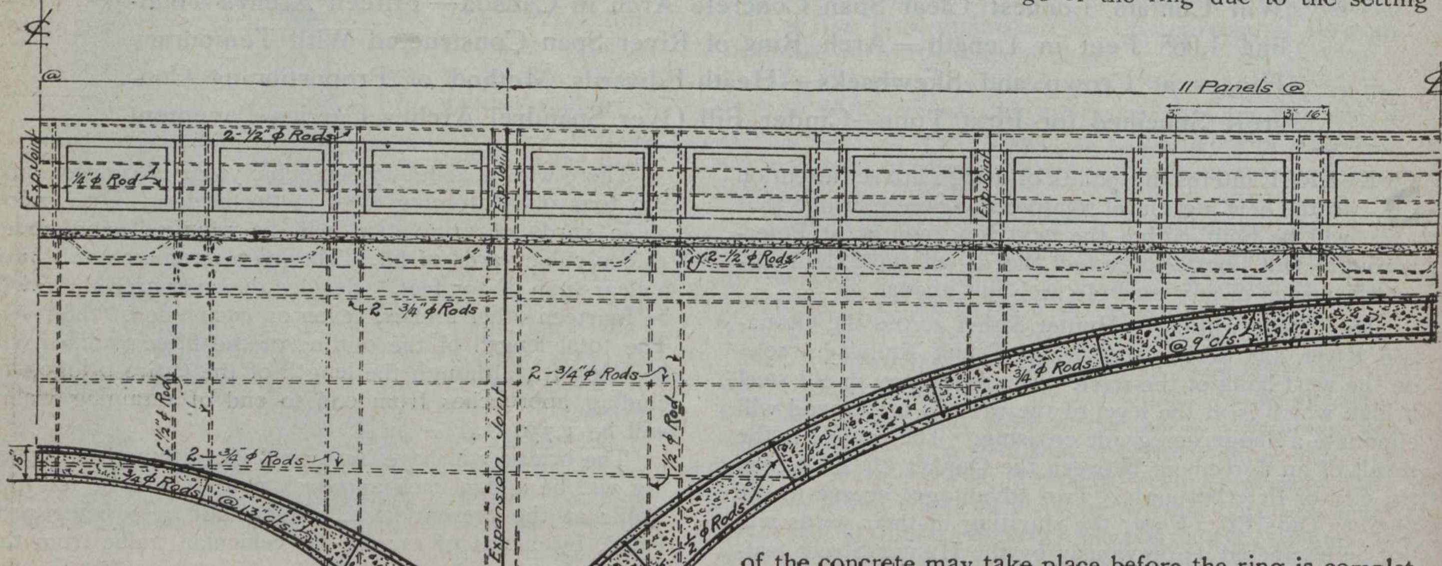
The panels in the balustrade of the river span and the ornamental cartouches on the lookout balconies in the centre of the span, also those on the four terminal piers, will be all mat glazed colored terra cotta in harmonious tones.

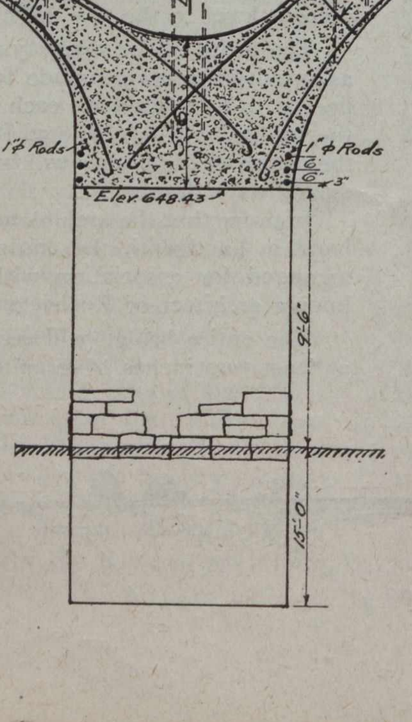
The arch ring of the river span will be concreted in sections in such a manner as to avoid shrinkage, leaving key spaces between the sections and closing the key spaces last of all. The sections between the bulkheads in the arch ring of this span will be concreted in the order



Plan of Bridge and Approaches

indicated by the numbers in Fig. 4, and in all other arch rings concreting will begin at both skewbacks and proceed from both equally to the crown. A key space at the crown approximately 1 ft. wide will be left between the the bulkheads to be concreted at least twenty-four hours after the remainder of the ring has been concreted, in order that the shrinkage of the ring due to the setting



Half Longitudinal Section of Two of the Approach Spans, Showing Reinforcing

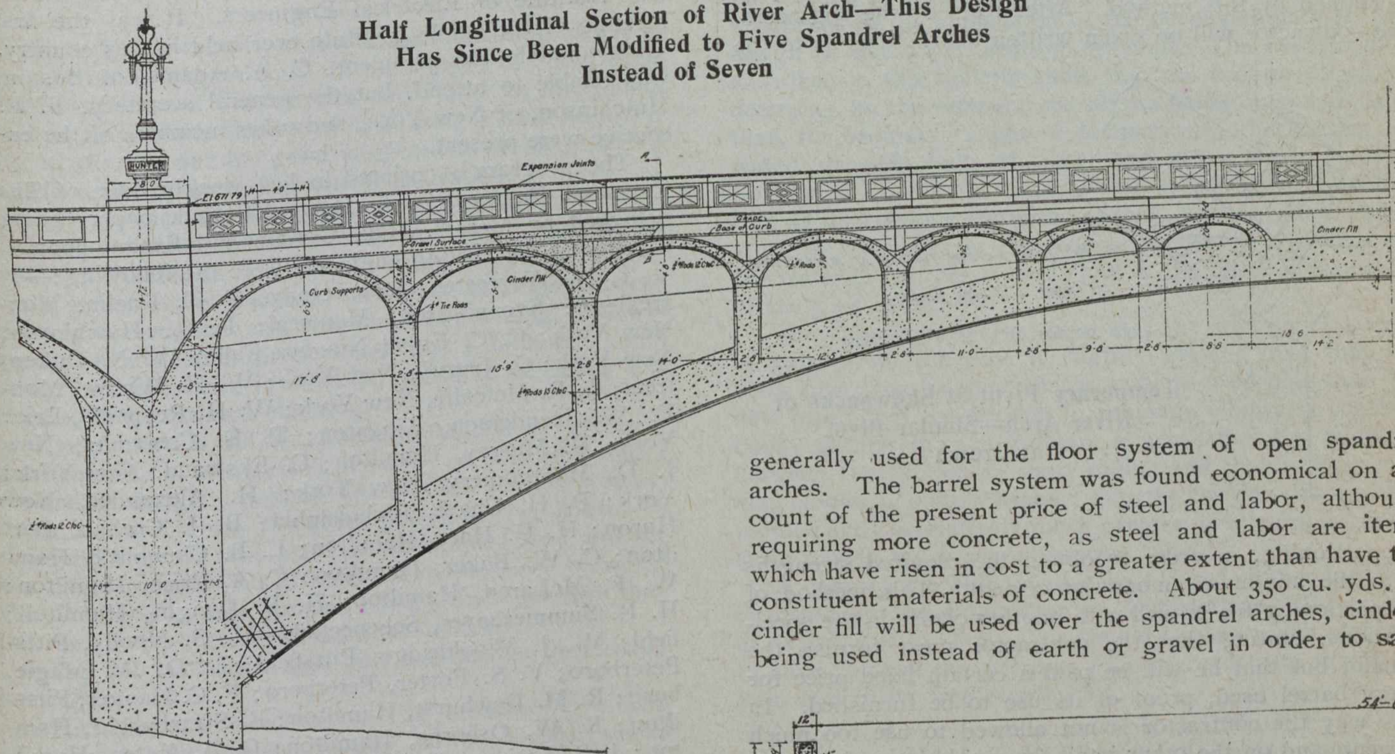
of the concrete may take place before the ring is completed and closed at the crown. Each spandrel arch, however, will be finished without joints; that is, at one operation.

The river arch having been concreted in voussoirs as indicated in Fig. No. 4, closure of the key spaces will be made twenty-four hours or more after the concreting of the larger voussoirs.

Following European practice, temporary hinges or pivots will be used at the skewbacks and the crown of the river span. The voussoir which contains the pivot will be concreted up to the pivot, with steel rods projecting, and then a half-inch layer of melted lead will be poured against the pivot surface, the layer being 15 inches long by the whole width of the arch ring. The remaining part of this voussoir will then be concreted on top of the lead. The steel rods will be placed as shown in Fig. No. 5.

The lead will be mixed with alloy to such a consistency that it will flow at 1,200 lbs. per square inch, sustaining three-quarters of the load, the other quarter being taken up by the steel rods. The steel rods also prevent the arch ring from slipping off the pivot. A thermometer will be embedded in the arch ring and when the temperature of the ring is at 45° Fah. which is considered the average yearly temperature, the joints at the skewbacks and crown

**Half Longitudinal Section of River Arch—This Design Has Since Been Modified to Five Spandrel Arches Instead of Seven**




will be "gunited," that is, filled in with the dense sand-cement mortar shot by the cement-gun. The arch will be completed and the whole dead load will be in place before these joints are thus closed.

This results in minimum temperature stresses and in the practical elimination of rib-shortening stresses. The principal stresses in this arch are the compressive stresses due to dead load, and they are never neutralized or reversed by live load or temperature stresses, consequently the entire rib is always in compression. Therefore there is no reinforcing whatever in the ring of the river arch.

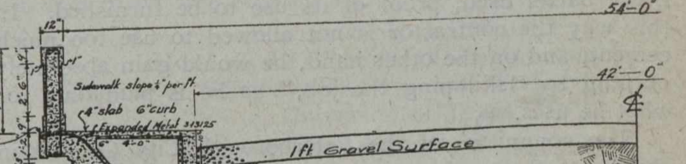
The spans of the five spandrel arches on each side of the centre of the river arch are all different, the length being greatest over the skewbacks and decreasing gradually toward the crown. This is said to be a novel feature but a logical one on account of the decreasing height of the spandrel piers.

It will be noticed from the plan, Fig. No. 3, that the approach arches on the west are not right arches. However, the skew is slight. The additional stresses caused by the skew have been allowed for, as nearly as could be approximated by bunching of the reinforcing along the shorter centre diagonal of the parallelogram formed by the sides of the span and by the piers.

The soffits of the approach arches are semi-ellipses. Flat arches were necessitated by the clearance requirements of the railway yard. The neutral axes of these arches were designed so as to be identical with the line of resultant pressure due to the dead loads plus half the live loads. The arches being necessarily flat, the semi-ellipse fitted the soffit of the arch by the addition merely of a small amount of concrete near the skewbacks to complete the curve. For consistency of design the spandrel arches are also semi-ellipses.

Since the approach spans are alternated, small and large on account of the track layout in the yard, the balancing of the horizontal thrust of adjacent arches was found to be a somewhat tedious problem.

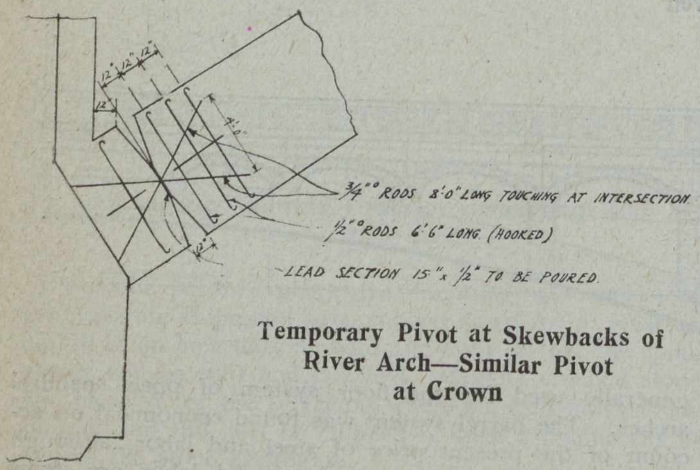
The barrel type of arch was chosen in preference to the beam and slab construction, which has previously been generally used for the floor system of open spandrel arches. The barrel system was found economical on account of the present price of steel and labor, although requiring more concrete, as steel and labor are items which have risen in cost to a greater extent than have the constituent materials of concrete. About 350 cu. yds. of cinder fill will be used over the spandrel arches, cinders being used instead of earth or gravel in order to save




**Half Cross Section Through An Approach Span**

weight. The only beam and slab construction in the bridge will be in connection with stairs leading to a baseball field on the west side of the river and for the sidewalk and curbs, underneath which space for public services is provided.

Another departure in the construction of the bridge will be the adoption of the Heath-Edwards or surface area method of proportioning concrete, as described in the July 4th and July 11th, 1918, issues of *The Canadian Engineer*. The specifications for the bridge require that the quantity of the cement in relation to the aggregates should be proportioned by this method. After tests by the engineers, the contractor will be given written instructions as to the




**Temporary Pivot at Skewbacks of River Arch—Similar Pivot at Crown**

proportioning, in order to obtain the required strengths with the minimum amount of cement. This method of proportioning fits in well with another clause in the specifications, stating that the contractor must furnish the cement but that he will be paid a certain fixed price for every barrel used, proof of its use to be furnished. In this way the contractor is not allowed to use too much cement, and on the other hand, he would gain absolutely nothing by "skimping the job," as he is paid only for what he uses.

The committee of the Peterborough City Council in charge of the construction of the bridge, consists of R. H. Parsons, city Engineer; Archibald Weir, chairman of the Board of Works; G. H. Duncan, mayor; and Malcolm McIntyre. Associated with Mr. Barber, as chief assistants on design, were W. E. Taylor and Fred Christie.

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The Quebec Telegraph urges an early resumption of public work on the St. Charles River, alleging that the work already done is disintegrating because it was not completed.

Last week the first street car passed over the full length of the new Bloor Street Viaduct, Toronto. The ballasting of the track is proceeding rapidly and the line will soon be ready for regular operation.

The Board of Works of the Stratford, Ont., council has petitioned the military authorities for an early release of Lt. A. B. Manson, city engineer, as the city desires to proceed with some necessary engineering work.

Judgment in favor of the J. H. Tremblay Co., Limited, for $22,894.34 has been delivered in a suit instituted by that firm against the Greater Winnipeg Water District. The Tremblay interests were awarded the contract to build two sections of the water line project, and had completed one section of it when engineers discovered cracks in the concrete. Judge Curran, in awarding the contractors the money claimed, averred that the cracking was due to natural causes and that the Tremblay interests could not be held liable for it.

Among the speakers on the programme of the third Annual Conference of the South-West Ontario Town Planning Board, to be held next Monday and Tuesday in the Royal Connaught Assembly Hall, Hamilton, Ont., are Thomas Adams, of Ottawa, on "Proposed New Legislation," and also on "Town Planning in Relation to Housing and Land Taxation"; J. J. Mackay, C.E., of Hamilton, who is general secretary of the board; Prof. C. B. Sissons, Toronto, on "Housing, Urban and Rural"; and Nolan Cauchon, Ottawa, on "New Legislation."

## A.I.E.E. MEETING AT TORONTO

TORONTO was visited last week by a number of leading American electrical engineers who came to Canada to attend the 344th meeting of the American Institute of Electrical Engineers. It was the first meeting of the whole Institute ever held in this country. Unfortunately the president, C. A. Adams, of Boston, was unable to attend, but the general secretary, F. L. Hutchinson, of New York, and other members of the executive were present.

The program as printed in last week's issue of *The Canadian Engineer* was closely followed, papers being delivered by Messrs. Hall, Gordon and Svenningson.

Among the visiting engineers were the following:—

T. A. Worcester, Schenectady; F. T. Kaelin, Montreal; S. Svenningson, Montreal; F. L. Hutchinson, New York; C. C. Egbat Niagara Falls; W. N. Onken, New York; G. Jinguji, New York; W. H. Winter, Montreal; G. R. Metcalfe, New York; W. H. Reynolds, Erie; E. W. Henderson, Kingston; T. E. Crossman, New York; H. Murray, Hamilton; C. Ekstrand, New York; J. D. Montgomery, New York; H. Alexander, New York; P. H. Chase, Philadelphia; B. J. Crahan, Port Huron; H. U. Hart, Hamilton; L. B. Chubbuck, Hamilton; C. W. Baker, Hamilton; C. A. Price, Hamilton; W. F. McLaren, Hamilton; H. B. Dwight, Hamilton; H. R. Summerhayes, Schenectady; W. S. Moody, Pittsfield; W. J. Wooldridge, Pittsfield; L. D. W. Magie, Peterboro; V. S. Foster, Peterboro; F. C. Hanker, Pittsburg; R. M. Dewhurst, Hamilton; E. Strasburger, Hamilton; R. W. Osborne, Hamilton; G. A. Yates, Hamilton; J. S. Lotimer, Hamilton; D. Nakahara, Yawatashi, Japan; S. Sakai, Osakif, Japan; R. H. Balfour, Montreal; Dr. Clayton H. Shifro, New York; W. T. Goddard, Hamilton; T. H. Barnard, Hamilton; C. W. Kent, Bridgeburg; R. F. Howard, Ottawa; N. S. Braden, Hamilton; E. I. Sifton, Hamilton; W. H. Childs, Hamilton; and F. T. Wyman, St. Catharines.

The total registration at the meeting Friday afternoon was 150, while a larger number heard Mr. Svenningson's paper in the evening at the University of Toronto. At the dinner held at the Engineers' Club, 143 members, delegates and guests were at the tables. There was just one after-dinner speech,—by Sir Robert Falconer, president of the University of Toronto.

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## CHANGES AT MONTREAl

ANNOUNCEMENT has been made that Paul E. Mercier, chief engineer of the public works department of the city of Montreal, will hereafter be consulting engineer to that city, and that Capt. Arthur E. Doucet will be acting director of public works.

Capt. Doucet was formerly a district engineer of the Transcontinental Railway. In 1915 he went into private practice, and in July of this year he was appointed as an advisor to the Montreal city commissioners.

The commissioners also appointed Charles Garrett as an engineering and contracting advisor last July. Now Mr. Garrett has tendered his resignation and it has been accepted, the commissioners stating that the work which Mr. Garrett had been asked to do has been completed.

Mr. Mercier has been chief engineer of Montreal since May, 1914, and since last July has also been Director of Public Works. He was formerly in consulting practice in that city.

# Letters to the Editor

## Circular Housing Plan

Sir,—I have recently read a most interesting article in your issue of August 29th, on the subject of "Engineering Possibilities of Circular Housing Plan," by Mr. Lamb, and as an engineer interested in housing I have been trying to winnow out the good suggestions of Mr. Lamb's article with a view to a practical utilization.

I gather that Mr. Lamb has convinced himself that his schemes cannot be adapted to the ordinary street layout which he condemns and without doubt rightfully so, but there are so many pleasant variations en route between the stiff rectangular and stiff circular planning that will appear far more than the extremes, that I think Mr. Lamb would be well repaid in adapting his ideas of public services to more rational planning.

I would like to call Mr. Lamb's attention to several of the items in his comparison of costs, as it is here that I assume he finds his greatest justification for his scheme.

In the first place the development of 333 houses in either plan would take place on unoccupied land and the whole development would be done as one piece of work. Why will houses on a circular scheme cost $300 less than the same houses arranged on a straight line.

Why will excavation cost only one-quarter for circular housing?

Where can he install seven 125 horse-power steam plants with cold storage, pumps, power houses, stacks and three miles of heavily insulated high pressure mains and three miles of heavily insulated return mains, etc., for $28,000?

Where, too, can he show a saving of two-fifths the cost of heating installation in his houses (over say hot water, which is no doubt contemplated in the block system although in the houses he shows hot air heating would be suitable)?

In plumbing can he save one-third by only reducing the length of the service connections?

And then when he gets down to roads, would there really not be considerably more paved area in the circular plan than in any well laid out rectangular system? And then, too, can residents be asked to walk a long distance in the middle of a driveway to approach their houses?

If these items are adjusted, is not the cost of a circular system already higher than the cost of a block system?

But, further, what about the tunnel which will be built? Here is where the engineering comes in and the tunnel will have to be a real one. There is 40 feet of tunnel for each house. Can a waterproof tunnel, with lighting, drainage, and various openings, be built for less than $20 per foot? It appears to me that this item is overlooked, and it involves $800 per house, or $266,400 for the whole development, which adds just one-third to the total shown, without the proper allowances being made for the other items mentioned,

In capital cost I fear Mr. Lamb can find no justification, but it is just possible that he might be able to show a real annual saving by such an arrangement of his public utilities service and provide a great improvement in the actual living conditions of the people

P. H. MITCHELL,
Vice-President and Managing-Director,
Toronto Housing, Company, Ltd

Toronto, Ont., November 21st, 1918.

## Friction of Water in Pipes

Sir,—Under separate cover I am sending you a copy of our bulletin on "The Friction of Water in Pipes and Fittings." This bulletin describes in detail a series of experiments made at the University of Texas to determine the friction of water in pipes and fittings, with particular references to the application of the developed data to the design of hot water heating systems. The experiments described in this bulletin show that the friction of water decreases as the temperature of the water increases and that, for ordinary ranges of temperature, the friction of water in one foot of new and clean black pipe is $(.01533v^{1.77})/(t^{.19}d^{1.275})$ feet of water of the same temperature as that flowing through the pipe; $t$ being the temperature of the water in Fahrenheit degrees; $v$, velocity of the water in feet per second; and $d$, the internal diameter of the pipe in inches.

The experiments also show that the friction of water in galvanized iron pipe is slightly greater than that in black pipe; that is somewhat greater in used pipe than in new pipe; that the friction in drainage elbows is considerably greater than in ordinary elbows; that the relation between the friction in short radius and that in long radius elbows is quite variable; that the friction due to unreamed ends is relatively much greater in small pipes than large pipes; and other interesting facts.

A copy of the bulletin may be secured by writing the chairman of the Committee on Publications, University of Texas, Austin, Tex. We should be very much pleased to have you call the attention of your readers to this bulletin and to its free distribution.

F. E. GIESECKE,
Head of the Division of Engineering,
Bureau of Economic Geology and Technology,
University of Texas.

Austin, Texas, November 7th, 1918.

## RULES FOR CANAL NAVIGATION

The Canadian Lake Protective Association has issued the following bulletin:—

"The particular attention of masters is again called to the rules governing the navigation of the Dominion canals, a copy of which should be in the hands of every canal navigator. The rules relating to bridges may throw an unfair burden upon the ship, particularly in view of the difficult conditions often encountered, due to current, wind or dangerous banks, all requiring the maintenance of steerage way. Nevertheless every effort must be made to observe the rules, and no master is entitled to assume that a bridge is going to be opened, merely because he has signalled to it. The courts have taken the same view in Ontario in cases relating to bridges, and until some change is made in the rules, too much reliance upon the promptness or efficiency of bridge tenders will simply result in damage claims to be met by the ship and her owners. The Canadian Lake Protective Association's committee will make further representations to the Railways and Canals Department, renewing the request for the operation of an effective signal from each bridge in answer to signals from vessels, but until the situation is improved in this way the existing rules must be strictly observed in every way possible.

"A casualty reported with reference to the Morrisburg canal upper entrance calls attention to pending proposals for improvement of conditions there. The committee is aware of the special difficulties at this point and has sought to hasten the work of improvement which has been under consideration by the Railways and Canals Department. The application of the New York and Ontario Power Co. to the International Joint Commission for approval of their plans for power at Waddington brings this question prominently forward, and the difficulties at this point are again called prominently to the department's attention"

# St. Lawrence River Power Situation

## Outline of Problems Now Confronting the Dominion Government—"Guard Our Power Heritage," Says Conservation Commission's Consulting Engineer

AT a luncheon last Friday, the Electrical Club of Toronto was addressed by Arthur V. White, consulting engineer for the Commission of Conservation, on "Canada's Heritage in the St. Lawrence River." The speaker, who for years has studied international engineering problems involving diplomatic relations, referred to the Boundary Waters Treaty as the chief legal agency for safeguarding the interests of the people of both Canada and the United States in the international St. Lawrence River. He cited a number of illustrations to show how even a recent treaty may fail to provide actual and full protection to the public.

Mr. White referred to the fact that the international St. Croix River, between New Brunswick and Maine, may be entirely diverted through a large canal extending for nearly a mile within the State of Maine. This canal was constructed without treaty authority

Another illustration consisted of a reference to the fact that it has been contended that, apart from the Boundary Waters Treaty, the rights of navigation in Lake Michigan are not as free and open to the citizens of Canada as the waters of the Georgian Bay are to the citizens of the United States.

Reference was also made to the bearing of the treaty upon diversion of waters from above Niagara Falls, showing how some interests contend that water authorized for diversion above the falls may, if so desired, be turned directly in to Lake Ontario without coursing the lower Niagara River.

### Permit Valuable

Mr. White also emphasized the valuable permit just granted to the St. Lawrence River Power Co., or the Aluminum Company of America, to construct a submerged dam in one of the navigable channels of the St. Lawrence River in violation of what the Solicitor-General of Canada states to be clearly prior rights under the Webster-Ashburton Treaty.

Mr. White outlined how diverse are the menacing circumstances which may variously arise in connection with attempts to secure permits to exercise special rights in boundary waters, including the St. Lawrence River, and how necessary, therefore, it is for leading public men, especially those in Parliament, to have a good understanding of the paramount importance to Canada of her natural heritage in boundary waters.

### Treat As a Unit

He referred to the preservation of the integrity of the St. Lawrence River from the standpoint of navigation, and said: "It may be summed up that deep-draft navigation from the Great Lakes to the sea involves, absolutely, the treatment and canalization of the St. Lawrence River as a unit."

Mr. White pointed out that the water powers of the St. Lawrence River are as yet largely within the control of the people. Ontario and Quebec respectively own the water power rights within their provincial confines. Consequently, when co-operative effort is made to co-ordinate international public interests incident to the development of these waters, the federal and provincial interests of Canada must respectively be safeguarded, the federal interest being in navigation and the provincial interest being in water power.

### Important Considerations

He briefly emphasized some matters of serious import which, in connection with development require special consideration, such as the ice menace; the question of making the power available for the building up of communities and smaller manufacturing establishments rather than having the power only absorbed by large industrial units such as the electro-chemical industries; and also the question of the exportation of electrical energy really required for use in Canada. Mr. White presented a conservative summary of the low water power potentialities of the St. Lawrence as follows (average estimated 24 hour low water horse power):—

Morrisburg-Rapide Plat, 200,000; Long Sault rapid, 575,000; Coteau rapid, 250,000; Cedars rapid, 500,000; Split Rock and Cascades rapids, 250,000; Lachine rapid, 375,000. Total, 2,150,000.

### Combination Possible

Consideration would be given to the possibility of combining the Coteau, Cedars, Split Rock and Cascades; also of increasing the Lachine power.

The Rapide Plat development is near Morrisburg, and is the one which is under consideration for development by the Ontario Hydro-Electric Power Commission in conjunction with the New York and Ontario Power Co., which has already been operating under certain existing rights; the Long Sault site is near Cornwall, and is the project respecting which the Long Sault Development Co., a subsidiary of the Aluminum Co., had their charter rights cancelled by the State of New York, as confirmed by the United States Supreme Court; the sites from the Coteau to the Cedars are in the vicinity of Valleyfield to Beauharnois, where a number of partial developments have already been made. The Lachine site is adjacent to the city of Montreal.

The speaker concluded with the remark that there is great need for the Canadian public being alert respecting their heritage in the St. Lawrence River. "Wise conservation and administration here will help pay our future taxes," Mr. White asserted.

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Orders have been placed for 200,000 gross tons of steel rails for replacements for Canadian railways. The Dominion Iron and Steel Company obtained 125,000 gross tons and the Algoma Steel Corporation 75,000 tons. Track accessories amount to an additional 65,000 tons.

The Dominion Government was authorized by Parliament last year to place orders amounting to $50,000,000 for equipment for the Canadian railways. It is announced from Ottawa that orders have been placed aggregating practically $48,000,000. These include 100.000 tons of rails at $64 per ton from the Dominion Iron and Steel Company; 37,660 tons of rails purchased from the United States, these rails having been rolled for the Russian Government; 195 locomotives, costing nearly $11,000,000: and 8,696 cars, valued at over $25,000,000. Deliveries have been obtained on nearly all the rails, 65 locomotives and 2,000 cars.

# FLOW OF WATER IN WASH WATER TROUGHS FOR RAPID SAND FILTERS*

By Frank V. Fields

FORMULAE for flow of water in open channels do not apply to the flow in wash water troughs, since at every point along the crests of the trough, water is admitted to it by the crests acting as weirs. The flow within the trough, consequently, is continuously increasing in volume towards the discharge end. The velocity of the water in the trough is also a variable, being zero at the upstream end and a maximum at the outlet. The effect of these variables is to produce a surface profile that will not be a straight line, but one that will adjust itself so as to permit the greatest discharge possible. It might be expected, then, that the point where the maximum depth occurs bears some relation to the maximum capacity. The determination of the surface curves and to find a formula that would be of assistance to designers of wash water troughs, were the principal aims of this investigation.

The galvanized iron trough (see Fig. 1) used in the experiments had a cross section approximating a rectangle surmounting a semi-circle of 7-in. radius. The section was the same as that used in the new filter plant at Cleveland, O. It also was recommended in the

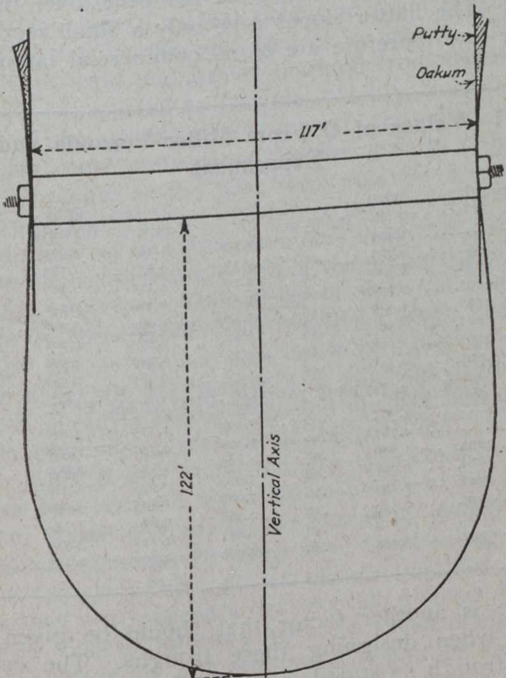


Fig. 1—Section of Trough Used in Experiments

preliminary design of the filtration plant for Lima, O. The length of the trough used (20 ft.) was the same as that recommended for Lima. The troughs in the Cleveland plant, however, are of greater length.

Four series of runs were made: first with the bottom of the trough horizontal, second, with the grade of 0.4 per cent., third with the grade of 0.8 per cent. and fourth with the grade of 2.0 per cent. In each case the crests were horizontal and each side was as nearly the same elevation as it was possible to make them.

An abstract of the data, showing the results of each run, is given in Table 1. In each series the maximum flow was the capacity of the trough for that slope, the water completely filling the trough to the iron braces at some point in the trough.

*Abstract of article in the "Cornell Civil Engineer."

**Table 1—Results of Each Run**

| Run No. | Grade, per cent. | Head on crest, ft. | Discharge, c.f.s. | Condition of flow into trough. |
|---|---|---|---|---|
| 9 | 0.0 | .079 | 3.539 | Nappe free |
| 10 | 0.0 | .067 | 2.850 | Nappe free |
| 11 | 0.0 | .0525 | 2.034 | Nappe free |
| 12 | 0.0 | .036 | 1.313 | Nappe free |
| 13 | 0.0 | .017 | 0.546 | Clinging flow |
| 14 | 0.0 | .049 | 2.034 | Nappe free |
| 15 | 0.4 | .079 | 3.565 | Nappe free |
| 16 | 0.4 | .0555 | 2.218 | Nappe free |
| 17 | 0.4 | .038 | 1.439 | Nappe free |
| 18 | 0.4 | .022 | 0.423 | Clinging flow |
| 19 | 0.4 | .0175 | 0.546 | Clinging flow |
| 20 | 0.8 | .083 | 3.764 | Nappe free |
| 21 | 0.8 | .064 | 2.629 | Nappe free |
| 22 | 0.8 | .038 | 1.413 | Nappe free |
| 23 | 0.8 | .014 | 0.481 | Clinging flow |
| 24 | 2.0 | .091 | 4.323 | Nappe free |
| 25 (a) | 2.0 | .070 | 3.003 | Nappe free |
| 26 | 2.0 | .045 | 1.710 | Nappe free |
| 27 | 2.0 | .012 | 0.387 | Clinging flow |

## Discussion of Results

The principal aim of the investigation was to determine the capacity of this trough, and also to work out, if possible, some formula that would aid in designing troughs of similar section. It was especially desirous to determine the effect on the capacity of putting the bottom of the trough on a slope, instead of horizontal, as is the custom in practice.

The profiles of the water surface in the trough are made up of two parabolas, each of the form—

$$Y = mx^n$$

where $Y$ is the depth at any point and $x$ is the distance of the point from the discharge end of the trough.

From the intercepts of these curves on the line $x=1$, and the values of $Q$ for the runs, the coefficient $m$ is found to be—

$$m = .458Q^{.518}$$

for the level series. The slope of the curves gives the value of $n$ as .138 for the parabola in the lower end of the trough. Similarly for the parabola in the upper end of the trough, the coefficient $m$ is $.587Q^{.545}$ and the value of $n$ is found to be .032. The equations of these curves are then, for the lower parabola—

$$Y = .458Q^{.518}x^{.138}$$

and for the upper parabola,

$$Y = .587Q_1^{.545}x^{.032}$$

These equations were determined for the series with the trough in a horizontal position and lines of the same slope have been drawn through the points obtained for the other series. They fit fairly well except for the low flows, where they evidently do not fit at all. It might be well to point out here that the curves on logarithmic paper do not represent the depth in the trough, except in the level series, but are plotted between distance from the discharge end of the trough and the height of the water surface above the discharge end of the trough.

For the level series the points of intersection between the lower parabola and the one in the upstream end of the trough, also lie on a parabola, and its equation is—

$$z = 10Q^{.3}$$

where $z$ is the distance from the discharge end to the intersection of the two parabolas.

By plotting the surface profiles on cross-section paper, the maximum depths in the trough for all runs were determined and these are plotted in Fig. 2 as abcissas and the corresponding values of $Q$ are plotted as ordinates. These curves show that the discharge varies as some power of the maximum depth in the trough and that this exponent is very nearly the same for the three higher grades. The slope of the trough does not seem to affect the exponent but is a factor in the coefficient of the equation. The equation is of the form—

$$Q = m[1 + f(s)]d^n$$

where $s$ is the per cent. slope and $d$ is the maximum depth occurring in the trough. The equation is found to be—

$$Q = 2.28(1 + .295s^{.69})d^{1.74}$$

This formula does not hold exactly for the level condition and since it is the practice to set these troughs level in actual installations, it might be well to give the formula

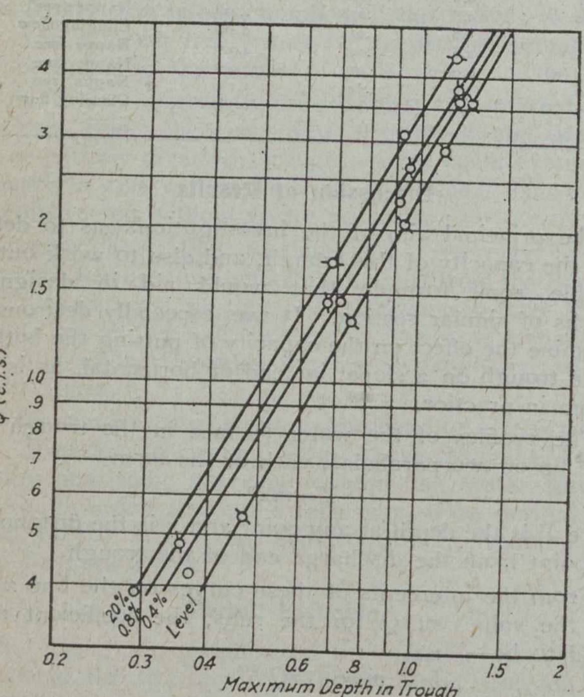


**Fig. 2—Maximum Depth in Troughs for Various Discharges**

that applies strictly to the level condition as indicated by these experiments, which is—

$$Q = 2.28d^{1.91}$$

This condition of flow has been discussed by C. N. Miller, and is included in the book, "Water Purification," by J. W. Ellms. The formula derived by Mr. Miller for a rectangular trough is—

$$Q = 1.91b(y + l\tan a)^{3/2}$$

where $b$ is the width of the trough, $y$ is the depth in the upstream end, $l$ is the length of the trough and $a$ is the angle between the bottom of the trough and the horizontal. Mr. Miller states that for any other section of trough it can be reduced to an equivalent rectangular section and then his formula can be applied. It seems that there would be three "equivalent rectangular sections"; one with the same width, another with the same depth and still a third with the same hydraulic radius, all of which would have the same area. Upon investigation, however, it will be found that no rectangular section can have the same area and hydraulic radius as a section that is made up partly of a semicircle; as the section under consideration is made up of a semicircle surmounted by a rectangular area, only the first two equivalent rectangles will be considered. The values of $Q$ for the maximum quantity and one of the lower quantities have been computed according to the formula mentioned, for each setting of the trough and are given in Table II. The values found by experiment are also given for comparison.

It will be seen from Table II that the discharge for the trough used as computed by Mr. Miller's formula, gives results that are from 6 per cent. to 19 per cent. too small.

The discharging capacity of a trough of the same cross-section but for lengths other than 20 ft., cannot be foretold from these experiments.

## Conclusion

The design of wash water troughs for slopes up to 2 per cent. and of sections similar to that used in these experiments can be aided by the use of the formulas given or from Fig. 3. If the quantity of water to be handled is known, the number of troughs can be assumed and the maximum depth, hence the section, computed. In general it has been the practice to set these troughs level, and in such a case the trough is of uniform section throughout its length. If the bottom is set on a grade, the section is smallest at the upstream end and increases uniformly to the downstream end. In this condition the maximum depth may not occur at the upstream end but at some point farther down in the trough. The amount of metal for either level or sloped setting would be about the same, but the 2 per cent. setting gives an increase in capacity of about 22 per cent. over the level setting. The flatter slopes give only a small increase in capacity and therefore are of no commercial importance.

**Table II—Values of Q From Miller Formula and From Experiment**

| Grade of trough. | Area, sq. ft. | l tan α. | b. | Equiv. y. | Q by formula. | Q by experiment. |
|---|---|---|---|---|---|---|
| Level ........ | 1.395 | .00 | 1.17 | 1.192 | 2.907 | 3.539 |
| Level ........ | 0.767 | .00 | 1.17 | 0.655 | 1.185 | 1.313 |
| 0.4 per cent. .. | 1.300 | .08 | 1.17 | 1.111 | 2.907 | 3.565 |
| 0.4 per cent. .. | 0.698 | .08 | 1.17 | 0.596 | 1.241 | 1.439 |
| 0.8 per cent. .. | 1.245 | .16 | 1.17 | 1.064 | 3.028 | 3.764 |
| 0.8 per cent. .. | 0.590 | .16 | 1.17 | 0.504 | 1,201 | 1,413 |
| 2.0 per cent. .. | 1.110 | .40 | 1.17 | 0.949 | 3.502 | 4.323 |
| 2.0 per cent. .. | 0.445 | .40 | 1.17 | 0.381 | 1.542 | 1.710 |
| FOR Y = DEPTH IN THE UPSTREAM END. | | | | | | |
| Level ........ | 1.395 | .00 | 1.093 | 1.275 | 3.100 | 3.539 |
| Level ........ | 0.767 | .00 | 1.022 | 0.750 | 1.268 | 1.313 |
| 0.4 per cent. .. | 1.300 | .08 | 1.090 | 1.192 | 2.990 | 3.565 |
| 0.4 per cent. .. | 0.698 | .08 | 1.003 | 0.695 | 1.330 | 1.439 |
| 0.8 per cent. .. | 1.245 | .16 | 1.088 | 1.145 | 3.090 | 3.764 |
| 0.8 per cent. .. | 0.590 | .16 | 0.968 | 0.610 | 1.251 | 1.413 |
| 2.0 per cent. .. | 1.110 | .40 | 1.072 | 1.035 | 3.530 | 4.323 |
| 2.0 per cent. .. | 0.445 | .40 | 0.900 | 0.495 | 1.455 | 1.710 |

There is another factor that should be given some thought when designing these troughs. The capacity for this trough as stated in this article, is based on that section of the trough below the lower edge of the row of iron braces that hold the crests apart. These braces are bolted through holes in the trough itself, hence the row is always parallel to the bottom of the trough. In an actual installation there would be fewer of these braces and they would be so placed as to give minimum resistance to the flowing water, hence the whole section of the trough would be used.

Still another factor to be considered is the effect of "flooding" the trough; that is, increasing the flow until the trough is under water. Such a flow was obtained on the 2 per cent. setting, and showed a discharge of 9.26 cu. ft. per second. This flow cannot be used as a criterion, however, since there is a considerable section in this trough above the iron braces and also the effect on

(*Concluded on page 480*)

## COMMENTS ON CONSTRUCTION OF LOCK AND DAM AT TROY, NEW YORK*

**By Maj. Gen. W. M. Black**
Chief of Engineers

It is rare that any extensive work of engineering construction does not afford opportunity for the exercise of originality either in adapting conventional methods to the local conditions or in actually improving on common practice. The Troy lock and dam was no exception to this rule and departures from ordinary practice were made in all stages of its construction. Some were necessitated to meet conditions while others were deemed improvements.

Upon tearing out the old crib dam built in 1823, tests were made of the spruce timbers therein which had been submerged nearly a century. Under test they developed a tensile strength of from 7,000 pounds to 12,000 pounds per square inch. This compares very favorably with a normal tensile strength of 10,000 pounds per square inch as noted in Trautwine.

### Modified "Maine" Cofferdam

One style of cofferdam used consisted of a row of flattened cylindrical chambers, or shells of steel piling, the cylinders being placed side by side with the piles of one locking into those adjacent. The cylinders were driven in water from 12 to 24 feet deep and the largest one was about 40 feet across and 40 feet in height. They were filled with sand and gravel dredged from the river, this material being used to provide stability and water tightness. Although high in first cost, the same piles were used for other cofferdams thereby greatly reducing the cost, especially as the piles were sold for a considerable sum on completion of the construction. The sand and gravel filling of one cofferdam was used to a great extent for the next cofferdam, much of the surplus being used in making concrete. This style of cofferdam was quite similar to the cofferdam of the Black Rock lock at Buffalo, N.Y. In its use care must be taken to place the filling materials progressively in adjacent cylinder pockets. If one cylinder be filled too much in advance of its adjacent cylinder distortion of the partition walls will result. Later it was found that the type could be modified and its cost reduced, by using cylinders of the kind used in the Maine cofferdam at Havanna combined with straight lines of interlocking steel sheet piling. Instead of the cylinders being placed closely together as at Havana, they were spaced a considerable distance apart with a single line of interlocking steel sheet piling between them. This piling was reinforced by both an exterior and interior sand and gravel fill. The cylinders acted as anchors for the lines of piling at corners and along the sides. The piling prevented any cutting out of the earth fills by overflow as well as increasing the tightness of the dam. Sluice ways closed by sliding gates were placed in each cofferdam to admit of flooding the coffer when desirable without interior wash, as happens when the water is allowed to flood over the top of the dam.

For construction of the lock, two small derricks of special pattern were devised. They were made narrow so as to pass each other but had a long reach, so as to cover the wide foundations and the considerable height of walls. This reach was obtained by using an overhanging arm built up of channel irons. The width of the derrick frame is 10 feet and its length 24 feet; the mast is 24 feet high, the boom is 40 feet and the outward overhanging arm has a reach of 16 feet. Owing to the narrow base of the derrick, ampler counter weights of sand and gravel had to be used, and in lifting weights on a low boom a portable guy was strung from the top of the mast.

### Emergency Dam

The emergency dam to shut out the river in case of accidents happening or repairs becoming necessary was of rather unusual size, the depth from the top of the walls to the sill being 26 feet. This dam is of the Boulé gate type consisting of three large steel trestles which, when not in use, are turned down on hinges, the tops extending into recesses in walls. The trestles are spaced over 11 feet apart, and a series of steel gates, each about 5 feet high and with small wheels on the back, are placed in front of them and roll down or up as the dam is closed or opened, being handled by means of chains.

To prevent wall scarring by the boats, vertical strips or fenders of cast iron with rounded fronts projecting two inches from the concrete, were used with success. These strips were placed in short lengths and bolted to the walls, the bolt heads being countersunk. The top edges of the lock faces of the walls were also provided with quarter round cast iron strips or nosings to prevent mooring lines from cutting into the concrete.

Between the vertical fenders there are occasional line hooks to be used by boats for mooring when locking through. These have proved very useful at the Troy lock, as the boatmen can attend to their own lines without the help of lock tenders. These hooks are so designed that they hold the bight of a rope against a downward or horizontal pull but instantly release it when the pull is upward. The hooks are in recess to engage rope easily.

Another convenience in the locks are ladders placed in recesses at intervals along the wall. The tops of the ladders are placed so that a man can pass easily and safely from the top of wall to the ladder, and the reverse, yet the top of wall presents a smooth, unbroken surface for the movement of the lock attendants and for handling lines. All the accessory iron work of the lock is so designed and set that the broken parts can be renewed without disturbing the concrete.

### Wall Construction

Mention might be made of the fact that the tops of the walls were brought to grade and surfaced with wooden floats giving a safe footing. The use of steel floats for surfacing concrete was forbidden since such floats bring an access of cement to the surface which, after a short time, develop a net of unsightly hair cracks.

The upper guide wall is used to protect boats from the effect of cross currents. As a cofferdam would have been very expensive for the 18-foot depth of water and the small amount of masonry involved, the wall was made in a series of isolated piers 10 feet wide with spaces of 10 feet between and with a continuous top above the water line. For laying the piers bottomless reinforced concrete cribs built in the dry were lowered in place after the river bed had been dredged and levelled with bags of concrete. Vertical reinforcing rods were placed through the interiors of these cribs which were then filled with concrete delivered through a tremie. This construction insured a hard and compact surface for the guide wall. The imperfections of the interior concrete fill placed in tht water are thus made of no moment.

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*From Professional Memoirs, Corps of Engineers, United States Army, and Engineer Department At Large. See also *The Canadian Engineer*, December 14th, 1916, "Laying Concrete in Freezing Weather; Troy Lock and Dam."

The large wooden cofferdam, over a thousand feet in length, inclosing the west half of dam was an interesting problem. To avoid expense and risk it was necessary to finish the cofferdam and masonry in it during one working season. A special method known as the Ohio River type, permitted rapid building of the frame work.

### Large Wooden Cofferdam

To build this coffer there were first constructed two frames like trestle bents consisting of vertical posts with sway bracing, and iron bolted and strutted. Two of these frames were stood up on a barge, transverse to the length of the proposed coffer and parallel with about 20 feet between them and properly held in place by slings and guy ropes. To these frames were then bolted the horizontal stringers or waling pieces, at the different elevations on both sides. Vertical plank were also bolted at intermediate points on the stringers so as to give a stiffness to the entire frame but permitting a pivot or pantagraph motion about horizontal axes. The framework at this stage of construction looked like the posts and rails of two parallel board fences tied together. This frame was picked up by slings from the derrick boat which stood alongside the barge and the barge was then pulled ahead lowering one end of the frame into the water. One end of the frame still rested on the barge. Another frame was built, connected, and launched as before, just as a line or universal joint pipe is assembled and lowered to the bottom of the stream, this movement being permitted by the pin or bolt connections between thee sections of the crib. In measure as this framework was placed the sides were made tight with sheet piling driven on the inside and filled by a clam shell bridge.

### Features of the Concrete Dam

The width of the Hudson River at the site of the Troy lock is 900 feet. The width taken up by the lock and by the power head gates is 225 feet. To reduce the flood heights above the dam, it was deemed desirable to give as long a crest to the dam as practicable. In order to reduce the effects of the cross currents formed by the flow across the diagonal arm of the dam, the crest of this arm was placed at an elevation 2 feet higher than the crest of the east arm. The length of the east arm is sufficient to provide for the low water flow of the river within this difference of level, so that the currents leave the dam approximately parallel to the channel of approach to the lock, and cause a minimum of disturbance to traffic in this approach channel.

The dam was of concrete built in 20-foot sections, keyed together by forming each end of each section into a waved surface instead of using the customary isolated tongues or keyways. A maximum interlock is thus obtained and the trouble of placing and moving separate keyway boxes is avoided; expense is saved as no time is lost in placing separate keys; nor is there any resulting breakage loss in moving these key boxes. Alternate sections were first placed and then the filling sections. No effort was made to obtain a bond between the concrete of the various sections.

To provide a safe passage for the transfer of power from the power site on the west end of the dam to the east bank, a tunnel was formed in the body of the dam concrete. This tunnel was drained by means of vents leading to the downstream face of the dam.

In several cases springs were encountered under the site of the dam. These were rendered harmless by piping the water therefrom to the down-stream face of the dam.

To protect the concrete dam as well as the concrete walls of the lock against the effects of frost, the concrete was made dense and every effort was taken to protect the skin formed by the spading along the faces of the forms. Excepting where actual imperfections in the skin were found, no plastering was permitted.

To close the 125-foot gap left in the cofferdam to care for the flow of the Hudson River until everything was ready to make the final closure and turn the water over the portion of the dam finished the year before, a novel plan was used. A steel pile cylinder, 25 feet in diameter was set in the millde of the 125-foot opening, lacing two openings of about 50 feet on each side. A line of piling was started from each side, these piling supported against a wire cable so they could be held against the current, a loop of the wire cable being used near the top and bottom to assist holding them in place.

### Gates Operated by Compressed Air

The lock gates and culvert valves are operated by compressed air, the most suitable source of power for river locks as the machinery can be exposed to high water without damage and is more sturdy and less liable to get out of order than electrical machinery. The operation of the lock gates is made by a spar attached to the rim of a bull wheel which operates like the crank of a steam engine, similar to the lock gate machinery on the Panama Canal. Air is compressed by a water turbine attached to a compressor while a second air compressor operated by a gasoline engine provides for a breakdown of the turbine. The lock gates and valves can also be operated by hand if necessary.

It was on this construction work that one of the concrete foremen solved the puzzle of sand streaks in concrete. These streaks were more prevalent in the wetter concrete and more common where steel forms were used rather than the wooden forms. It had been thought that these streaks were due to the excess of water in the concrete working its way to the face of the form and then percolating downward carrying the cement away from the face. This belief was not substantiated by an examination of the work at Troy as there was no deposit of cement at the lower end of the streaks, nor could any trace of collected cement be found anywhere along the streak.

Again the streaks were wider at the top than at the bottom whereas it seemed that water percolating downward would tend to spread out. It was discovered that the water percolated upward, carrying the cement with it, not only at the face of the forms but at various points in the mass of the concrete. It appeared to be due to the weight of the concrete forcing out the excess water which followed the easiest path of escape, which was upwards, escaping through very small fissures or holes not larger than a pin-head. The flow would continue for a minute or two and when it stopped there would be left upon the surface a small flat cone of cement. This accounts for the fact that no cement appears at the bottom or along the sides of such streaks when the forms are removed as it is all carried upward to the surface.

The design and construction of the lock and dam were carried on under the direction of the writer, then in charge of the First New York District, River and Harbor Improvements. His principal assistants were Colonel (then Captain) R. D. Black; Colonel (then Major) M. J. McDonough; Assistant Engineers D. A. Watt and A. C. Harper; and Junior Engineers F. P. Fifer and J. J. McCabe.

# DETECTING LEAKS IN UNDERGROUND PIPES*

By David A. Heffernan

THE unparalleled severity of the past winter, with all its attending evils, seemed especially applied towards water works, and makes timely a discourse on this subject. There is no question but that, in the memory of any of us, the past winter has never been surpassed in its destructive aggressiveness, at least, since waterworks systems became a fact and not a theory.

The unprecedented depth which the frost reached was the cause of more leaks in two months than the average superintendent would have to combat in a year. The ravages of the cold touched with ill effect most every sort of installation or appliance used in connection with the business, affecting the ball-cock in the tank in the private home on one extreme and the hydrant in the street on the other. It may, therefore, be safely said that, in addition to showing an extraordinary jump in cost of maintenance, waterworks reports for 1918 will also show a large increase in the number of leaks. Even though the cold spell ended five months ago, all its effects have not yet been felt. So that while this little paper is not intended as an authoritative essay it may be that a few helpful hints may be gleaned from it.

## A Matter of Patriotism

It is estimated that the daily per capita consumption of the Metropolitan Water Distribution System, as included in Boston and vicinity, is about 16 per cent. higher than last year. A part of this may be accounted for in the increased use due to work pertinent to the war, but the remainder is an after result of the winter.

In February, the average increase for the district was 29 per cent. in four different cities and towns reaching 135 per cent., 129 per cent., 125 per cent., and 104 per cent. Of course, this was due to running water from inside taps to prevent the possibility of services freezing, and should not be considered as waste water in its narrower sense as applied to leaks. The present high consumption, though a result of the winter, is waste, pure and simple. It is a matter of patriotism for every superintendent to use every means at his disposal to combat the wasteful and needless use of water.

## None Near Perfection

It has been calculated that if the waste water be eliminated from our systems, 150,000 tons of coal would be saved yearly in New England alone. And it must, by this time, have become apparent to us all that if New England does not extricate itself from the position in which she was found last winter the Fuel Administration will not. So it simply devolves upon us here to help in its accomplishment. The efforts of the superintendent should be largely centred on the endeavor to better his system by minor improvements. The war has restricted building, and in some cases completely abolished it. As the necessity for the extension of mains is entirely dependent on new building operations, if building has stopped, the money and labor hitherto used in construction can be better applied in the improvement of the system; and there is no waterworks plant so near perfection that extensive improvements cannot be made. One of the most satisfactory ways of employing this energy is by reducing the waste of water. The responsibility for this waste water lies in two places, with the consumer and distributor. The distributor has or should have the knowledge of how to correct the evil, and also the ability to impart this knowledge to the consumer. To educate the consumer to the point where he realizes that it is for his own benefit to stop the use of the water beyond the amount actually consumed for useful purposes should be the duty of the distributor. I think all are familiar with the recognized methods. Press articles bring good results, and mailed or distributed circulars, with plate showing various-sized openings, with the amount of water each would permit the escape of and the cost of these amounts of water in tables, are satisfactory agents.

## Daily Consumption Comparisons

In conjunction with a campaign of this sort, it is well to have inspectors make a house-to-house canvass, looking for defective plumbing. It should be explained to consumers receiving water by meter that it is much cheaper to pay plumbers' bills than to pay for water from which he has received no benefit. In the case of a flat-rate service, the threat of cutting off his supply, if repairs are not made, is effective. The inspector should be equipped with an aquaphone, and, as he visits each house, should listen on the pipe in the cellar for a leak on the service. In this manner, a quantity of small leaks will be discovered and repaired.

The distribution system presents a different problem. The simplest method of detecting leaks is by means of daily consumption comparisons. This presupposes the fact that there is some means of measuring the amount of water supplied to the distributing mains. This measurement may be made by computing the flow over a weir, by piston displacement of pumping engines, or by the actual measurement by Venturi or other meter. If by the second method, careful allowance should be made for slippage.

## The Test-Pit Scheme

In the consumption book, daily records should be kept of the amount supplied to the mains. With this should be kept the relative conditions that might cause a variance in consumption, such as conditions of weather, fires and their duration, blowing-off operations and leaks discovered. With these facts at hand, it is possible to estimate fairly accurately the cause of any sudden increase in consumption. For instance, Monday usage, as a general thing, will be larger than any other day of the week because it is the customary wash day. A day on which a large fire occurs will show an increase. Opening a number of hydrants on dead ends for the purpose of clearing the mains will show an excess, as will a prolonged spell of dry, hot weather. So that, if a sudden increase in the use of water occurs with none of these factors entering, it is a safe wager that a leak or leaks exist in the system. If the measuring device has a self-registering attachment, to watch the night-flow (between 1 and 4 a.m.), it will give indications of the condition of the system.

After the fact of the presence of a leak has been established, it is next in order to find it and make the necessary repairs. There are several methods for its discovery. The old-fashioned test-pit scheme is as good as any, provided the system is well equipped with valves. The test-pit is nothing more than a manhole of convenient size, built around a gate. On either side of the gate and within the manhole a tap is made and corporation inserted. This furnishes the foundation for a by-pass around the gate. In the by-pass a meter is set, its size dependent on the consumption of the district to be tested. If the

*Paper read before the New England Water Works Association.

section is a large one, necessitating larger than a 2-in. meter, it is well to make a supplementary by-pass with small meter around the larger one. The section to be tested should be carefully laid out, all gates located beforehand to eliminate delay, and tested for tightness. The test for a leaky gate is simple. Close the gates to be tested and open a hydrant on the line between, removing the suction cap. Then open one valve slightly until the water in the hydrant barrel rises to the level of the suction nozzle and just flows out. Then close the valve again. If the water continues to flow from the nozzle, one of the valves must be leaking. Of course, for a few minutes the flow might be from the houses in the district, but this should stop shortly. If the level of the water lowers, it shows a leak somewhere in the cut-off section, below the level of the hydrant nozzle; but it would indicate a good-sized leak, as the open hydrant removes all pressure from the line. Aquaphones may be used on each gate, as further tests of tightness. The best hours for the actual testing are from 1 to 4 a.m., when the flow is less than at any other period of the twenty-four hours.

### Microphone Detector

If all the gates are tight it is evident that no water can enter the section so cut-off unless it passes through the meter on the by-pass. If readings of the meter are taken at set intervals, and gate by gate the area of the section covered by each test-pit lessened, it can be seen with ease that the leak will be discovered to be within a certain pair of valves. After its general location has been determined in this manner, the exact point where the water is escaping can be determined by looking for any luxuriant growth of vegetation at the side of the road, by dampness on the surface of the street, by driving down a bar and noticing any moisture on removing it, by means of an instrument magnifying the hissing sound of escaping water or by using any other convenient method. In this manner, a whole city or town may be covered.

The instrument just mentioned is contained in a small, light box. A small, four-legged brass table is set on the ground over the pipe in the vicinity of where the leak is supposed to be. The box has a raised bottom, so that it may be placed over the table to keep out foreign noises. On the table is a microphone-detector with wire connection to an amplifier battery contained in the box. Very sensitive ear receivers are wired to the battery. When the instrument transmits a rushing noise to the ears, it is evidence that the leak is close at hand. The affair is set at different points along the pipe line until the point where the noise is loudest is reached. The leak will be found directly under the spot. Used in connection with a wireless pipe locator it will be found to be satisfactory.

### Can Use Fire Hose

The beauty of the test-pit scheme is that it is permanent; the pits are always ready in time of need. If it is not desired to construct the manholes, the same results may be obtained by connecting two hydrants with fire hose, one hydrant being inside and the other outside the district to be tested.

Another method, quite as accurate, but probably more complicated and requiring more delicate apparatus, is the use of the Pitometer. There are also several special methods. One was described before a meeting some years ago, by F. J. Hoxie, and is called the "caustic soda method." Another is a special device making use of water hammer. It works on the principle that the sudden closing of a valve produces an impulse which travels through the water in a wave, decreasing in intensity in passing an opening in the pipe. It is claimed that the relative distances from the instrument to the break and to the suddenly closed valve is readily determined, but with what accuracy I cannot say.

From our experience in Milton, we believe the test-pit method to be as accurate as any, and simpler than most. It proved its value this spring when we found our daily consumption, which had been away above normal all winter on account of water being run to prevent services freezing, was not at all reduced when mild weather set in. We were using at the rate of 500,000 gal. daily, which was 56 per cent. above normal. The leak was discovered in short order, and the average daily consumption was reduced by 180,000 gallons.

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## GOVERNMENT RAILWAYS MERGED

Announcement was made last week by the Minister of Railways and Canals that an order-in-council had been passed, transferring the management and operation of the Canadian Government Railways to the Board of Directors appointed to manage the Canadian Northern Railway System. This transfer now places all railways owned by the Government under the direction and operation of this new board.

There have been added to the board Thomas Cantley, of the Nova Scotia Steel and Coal Co., of New Glasgow, N.S.; A. P. Barnhill, of St. John, N.B.; and Sir Hormisdas Laporte, of Montreal.

This change now makes a unified Government system of about 14,000 miles, extending from Sydney, N.S., to Vancouver, B.C.

The Board of Directors a few weeks ago reorganized the Canadian Northern System, and will now proceed with the further changes that are necessary on account of the addition to their system of the Intercolonial and Transcontinental lines.

It is understood that C. A. Hayes, who has been general manager of the Intercolonial Railway, will become general traffic manager of the whole Government railway system, with office in Toronto. F. P. Brady, general manager of the Transcontinental Railway between Winnipeg and Quebec, becomes assistant to the general manager of all Eastern lines. T. S. Brown, general superintendent of the Intercolonial Railway, will be chief operating officer of the Intercolonial Railway under Mr. Brady.

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Arthur Hall, chairman of the Toronto section of the American Institute of Electrical Engineers, stated at a meeting of the Institute held last week that 5,200 graduates and undergraduates of the University of Toronto have been or are on active military service.

According to a statement made by the Canadian Mining Institute, the total number of members resident in Canada or on military service entitled to vote for officers of the society for the coming year is 909, divided as follows: Alberta, 158; British Columbia, 153; Nova Scotia, 112; Ontario, 351; Quebec, 134.

The general annual assembly of the Royal Architectural Institute of Canada will be held at Montreal, January 17th and 18th, 1919, at the same time as the annual convention of the Province of Quebec Association of Architects. The programme of the assembly will be sent to all members of the Insttute next month. Alcde Chausse, hon, secretary.

Industrial Commissioner Sclanders, of Windsor, Ont., upon returning from a trip to Chicago, declares that the prospects are very encouraging for industrial expansion in the Essex border cities, viz., Walkerville, Windsor, Sandwich, etc. "Several of the largest corporations across the border have been looking eagerly toward this district," says Mr. Sclanders, "but have been held back because of war conditions. They are now favorably inclined toward the idea of acquiring factory sites in Canada, and are only waiting until peace has been absolutely assured, and conditions return to normal before actually opening business here."

# Transmission Line Has 4,800-ft. Clear Span

## High Voltage Overhead Construction Across St. Lawrence River Near Three Rivers, P.Q.—Largest Span of Its Kind in the World—Vibration and Other Problems Still To Be Solved—Detailed Description of Foundations

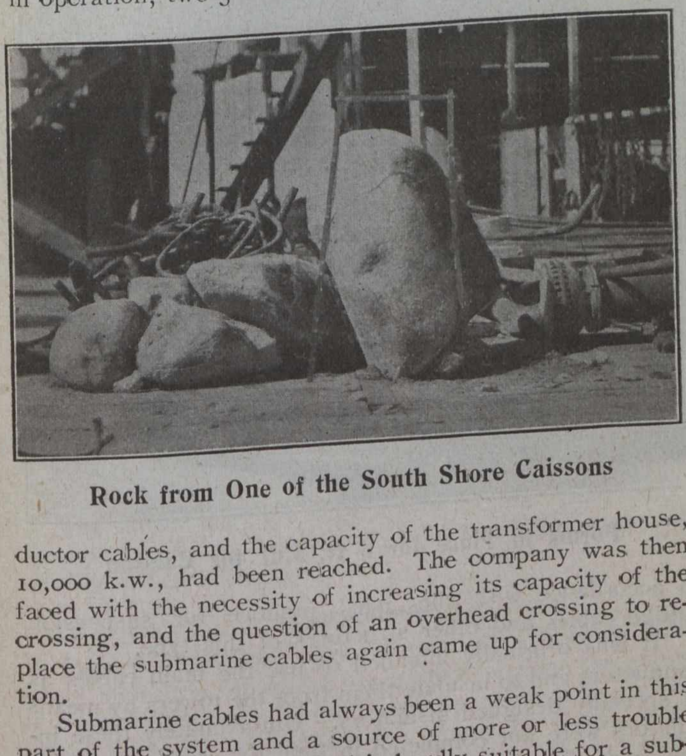
By S. SVENNINGSON

Supervising Engineer, Shawinigan Water & Power Co., Montreal

At Shawinigan Falls, on the St. Maurice River, about 20 miles north of Three Rivers, where the St. Maurice empties into the St. Lawrence River, and about 85 miles north-east of Montreal, is located the principal hydro-electric development of the Shawinigan Water & Power Co.

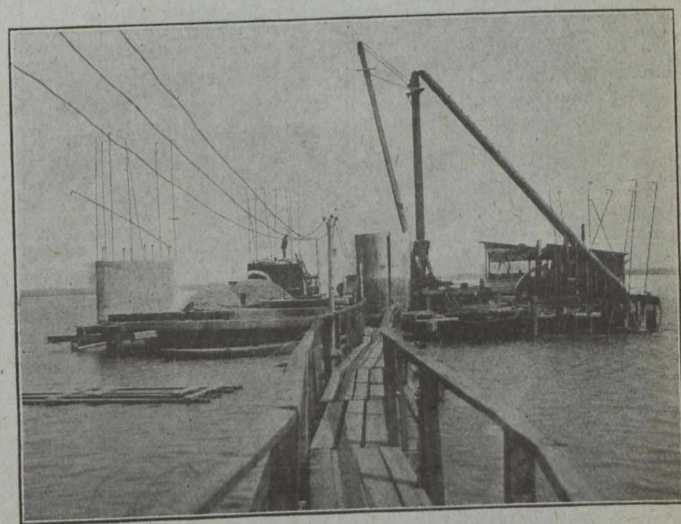
For a number of years the company has been transmitting power to the towns south of the St. Lawrence River. Two lines carry the current to a switching station at Victoriaville, 35 miles south of the St. Lawrence River. At Victoriaville the lines branch, one branch 50 miles long running to Sherbrooke and supplying various towns and industries between, the other branch, 40 miles in length, feeding the asbestos mines and other industries in the Thetford District. The current has been transmitted at 50,000 volts from Shawinigan Falls to the St. Lawrence where the voltage was stepped down to 25,000 for transmission across the river over submarine cables, then stepped up again to 50,000 and transmitted at this voltage to Thetford and Sherbrooke.

The first submarine cable was installed in 1906. At this time the alternative of putting in an overhead crossing was considered, but the amount of power to be transmitted was so small that it was decided that the expense of an overhead crossing was not warranted. However, the demand for power on the south shore steadily increased until by the beginning of 1916 five submarine cables were in operation, two 3-conductor cables and three single-conductor cables, and the capacity of the transformer house, 10,000 k.w., had been reached. The company was then faced with the necessity of increasing its capacity of the crossing, and the question of an overhead crossing to replace the submarine cables again came up for consideration.



Rock from One of the South Shore Caissons

Submarine cables had always been a weak point in this part of the system and a source of more or less trouble and expense. The river bed is hardly suitable for a submarine crossing. It is sandy but with a generous sprinkling of rocks and boulders. The bottom slopes out very gradually from both shores to the steamboat channel, the sides of which are rather steep. Owing to this abrupt change in slope, the cable in the channel is partly suspended and kept constantly swinging by the current in the river so that the armoring quickly wears through and the cable burns out at the points of suspension. The current in the channel carries the cable down stream and has even been strong enough to pull them loose from their moorings and break the connections in the cable house. In the winter the ice usually puts at least one of the cables out of commission, and it has been found necessary at times to erect temporary pole lines across the ice to maintain the service to the south shore. When, therefore, in the fall of 1916 the demand came for more power for the south shore, partly for war work, and it became a question of putting in an additional submarine or an overhead crossing, the company decided in favor of the latter.



View of North Platform, Looking South, While Caissons Were Being Sunk

In order to increase the capacity of the submarine crossing it would have been necessary, in addition to purchasing and installing new cables, to build extensions to our cable houses and install new transformers with their necessary switches, lightning arresters and other equipment. This would have involved an expenditure of at least $150,000 and at the same time the weak point in the line would not have been improved.

The overhead crossing was estimated to cost $200,000, the difference between the two being offset, in the opinion of the company, by the elimination of the weak link, by obtaining greater security from interruptions to the service and by a gain of from 2% to 3% in regulation by cutting out the transformers. A considerable amount of operation and maintenance expense would also be eliminated and the transformers, cables and other equipment tied up in the submarine crossing were needed and could be used to advantage in other parts of the system.

The two shores of the St. Lawrence River upstream as well as downstream of the cable houses, were carefully surveyed in order to find the most advantageous point of crossing. As a result of this preliminary survey it was finally decided to investigate in detail two alternatives:—

(a) A three span crossing at Point-du-Lac, each span approximately 2,200 ft. long.

(b) A single span crossing between the cable houses 4,800 ft. long.

Point-du-Lac is situated at the eastern end of Lake St. Peter about 6 miles up the river from the cable cross-


**Lowering the Caisson—The Four Screws Were Turned Equal Distances Simultaneously**

ing. From a construction point of view the site at this point appeared at first to be the most favorable for an overhead crossing. The St. Lawrence River at this point is about 7,000 ft. wide, but as the water is very shallow except for a distance of 2,000 ft. in the centre, a crossing could have been built using 3 spans of approximately 2,200 ft. each.

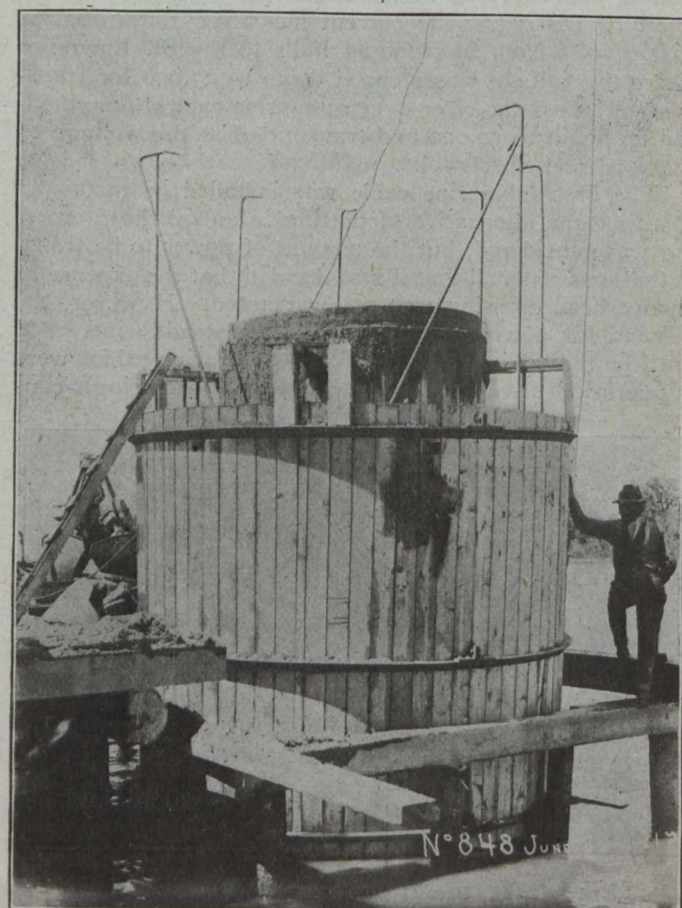
The towers on either side of the main channel would have been about 205 ft. high, while the other two towers would have been about 110 ft. high. The crossing at this point probably would have been somewhat cheaper than the other alternative but it would have necessitated the construction of about 15 miles of double circuit high tension pole lines in order to connect up with the main transmission lines. The cost of these connecting lines would have brought the total cost up to that of the single span scheme.

A fairly strong point against the three span crossing was the inaccessibility of the towers during certain periods in the spring and fall when the river is full of floating ice. The single span scheme was finally decided on as being the most advantageous, although it was fully realized that there were many difficult problems to solve in connection with the design and construction.

The crossing as completed consists of a central span 4,801 ft. long and two anchor spans, the north shore span being 571 ft. long and the south shore span 951 ft. long.

There are two towers 350 ft. high and 60 ft. square at the base, the upstream and downstream faces tapering to a width of 14 feet at the top. A cross arm at the top, 14 ft. wide by 100 ft. long, carries three double groove sheaves 50 ft. apart over which the anchor cables pass. The tower foundation is made up of four circular reinforced concrete piers 11 ft. in diameter placed on the corners of a 60 ft. square. These piers are connected by heavily reinforced concrete beams 4 ft. wide by 8 ft. deep. The tip of the foundation is 14 ft. above the normal water level.

Three lines of $1\frac{3}{8}''$ diameter steel cables 50 ft. apart span the river between the two towers. To each end of each centre span cable is yoked two anchor span cables. These are carried over the tower on the main sheaves and then down to a point about 20 ft. from the anchors. At this point equalizing beams are cut into the lines and the load is transmitted from them to the anchor piers by means of short straps of $1\frac{3}{4}''$ diameter cable. The cables are gripped at the ends by means of heavy steel bridge sockets




**Forms in Place for 6 ft. Lift for Caisson**

in accordance with the usual practice for suspension bridge cables and other structures of this type.

It was originally intended to use the main cables as conductors and to insulate them from the towers by means of specially designed insulators. Unfortunately these insulators were not completed in time for erection and for the present the main cables are used as messengers from which No. 1/0 stranded copper conductors are suspended. These suspended lines are supported every 250 ft. by suspension insulators of 8 units to a string.

The original scheme contemplated the use of part of the submarine crossing as a standby until some future date when an additional overhead crossing identical with the present one would be installed. However, the success that has attended the operation of the steel cables as messengers has led us to the conclusion that it is possible to use the present crossing for two circuits and thus to eliminate most of the expense of an additional overhead crossing. We have at present under consideration two alternatives for the accomplishment of this object. One alternative provides for the use of the steel cables for one circuit and the suspended conductors for the other circuit. The other alternative contemplates the carrying of two conductors on each steel cable. This would be accomplished by suspending the two conductors from the ends of light triangular steel or aluminum frames attached to the steel cables at intervals of 200 to 500 ft.

The anchor piers are large mass-concrete "dead men," each anchor being designed to take the full overturning moment when submerged.

## Foundations

During February, 1917, a number of borings were taken about the site of the towers to determine the nature of the river bottom. These borings penetrated to a depth of 100 ft. and we found that the foundation on which we would have to build our towers consisted for the full depth of these borings of very fine white sand with occasional strata in which a little clay was mixed with the sand. The difficulty of obtaining a secure pile foundation in this kind of soil and the uncertainty as well as the cost of placing a mat foundation in the dry, led us to adopt the form of pier foundation which we used.

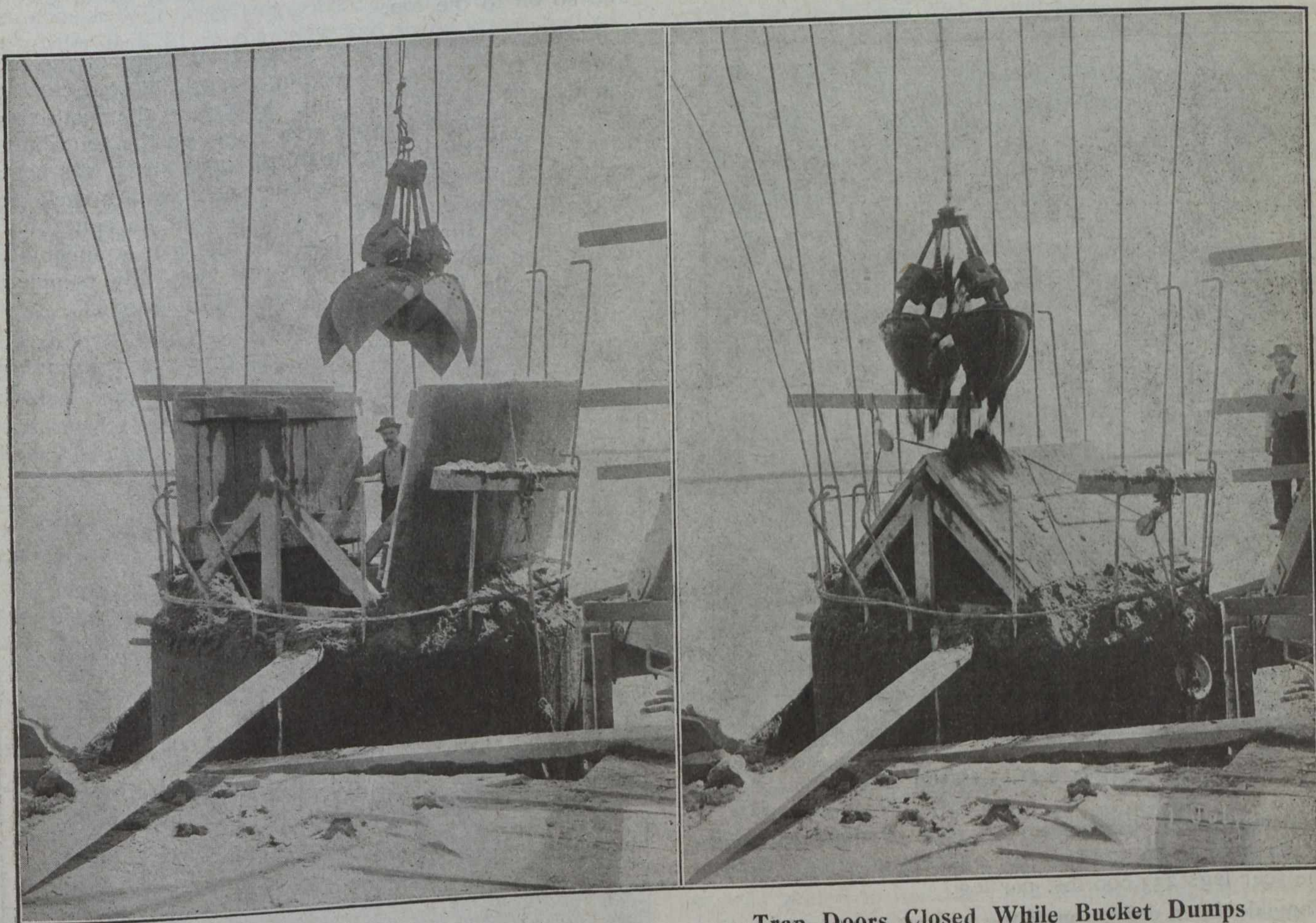
The piers were constructed in the form of hollow cylinders of reinforced concrete, with an outside diameter of 11 ft. and an inside diameter of 7 ft. These cylinders or caissons were poured in 6 ft. lifts, the first tapering on the inside towards the bottom to a diameter of 10 ft. and being shod with a 6″ by 6″ angle cutting edge. This lift was poured on a working platform and lowered into the water by means of four 2″ screws.

The second lift was then poured and after the concrete had set, the bottom was excavated by means of an orange peel bucket rigged up on a derrick. As the caissons gradually settled, successive lifts were poured until they had penetrated to a depth of 40 ft.

Little trouble was experienced on the north side, but on the south side we encountered large numbers of boulders, some of which were so large that they could not be picked up by the bucket, so that we had to drill and shoot them. In order to do this the caissons had to be unwatered, a tedious process which delayed the work considerably.

When a caisson had reached its penetration of 40 ft. a plug of rich concrete was poured in the conical section at the bottom and the inside of the caisson was then filled with mass-concrete. The four piers forming one foundation were finally connected by reinforced concrete beams.

This work was begun early in the year and we expected to have it finished by midsummer, but high water, high



**Trap Doors Open to Admit Bucket** **Trap Doors Closed While Bucket Dumps**

By this arrangement the bucket could be dumped without swinging it from over the caisson, thus avoiding objectionable swaying, especially in windy weather, and saving time.

winds, rain and labor troubles delayed us so much that it was not completed until about the middle of September.

### Tower Design and Erection

The towers were designed for the following loads:—

A vertical load of 530,000 lbs., made up of 350,000 lbs. due to the weight of the tower itself and 180,000 lbs. due to the vertical component of the tension in the cable.

A horizontal load of 42,000 lbs. at the top of the tower, parallel to the line, due to the horizontal component of the tension in the cable.

A horizontal load of 26,000 lbs. at the top of the tower, normal to the line, due to wind load on the line.

A wind load on the towers of 400 lbs. per foot height, or 140,000 lbs. total.


**Concreting a 6 ft. Caisson Lift**

The maximum compression on each of the front legs was estimated to be 575,000 lbs. and the maximum uplift on the rear legs 233,000 lbs. per leg.

The calculated deflection of the towers under maximum load is 4½". The wind and dead load stresses in the tower members do not exceed 20,000 lbs. per square inch.

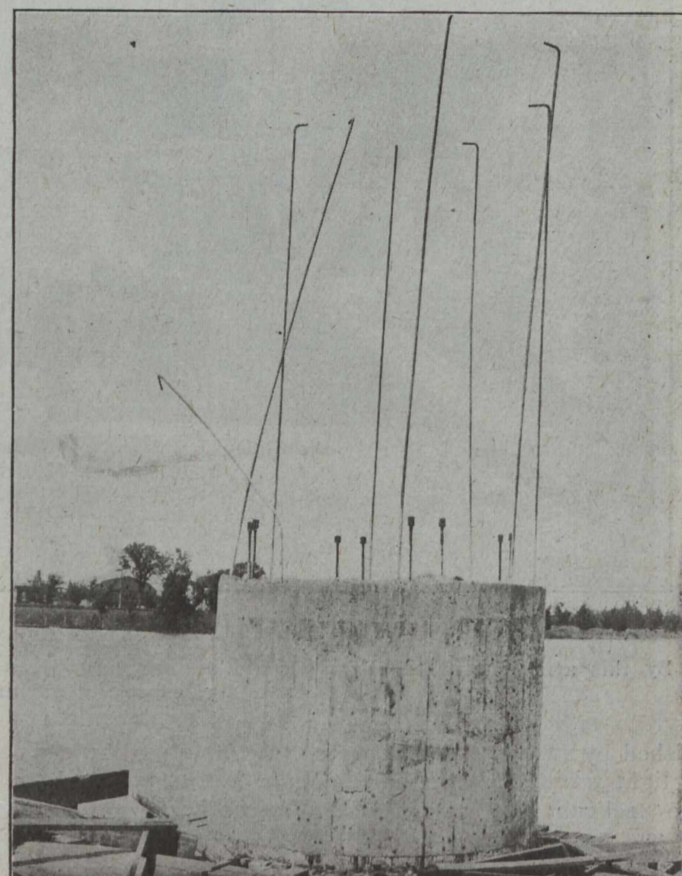
The bottom sections of the tower legs are composed of two 18" I-beams weighing 70 lbs per foot. These sections get lighter towards the top, the top sections being made up of two 12" channels weighing 20½ lbs. to the foot. The two members composing each leg are laced together with 2" by 2" angles.

Access to the top of the towers is obtained by means of Otis-Fensom automatic elevators, operated by electric hoists at the top and controlled by push buttons at top and bottom. The elevated baskets are composed of angle frames with plank floors 3 ft. wide by 4 ft. long. The elevator guides consist of four ⅝" diameter cables, one at each corner of the basket, suspended from the top and weighted at the bottom to provide a constant tension of 1,000 lbs. A steel ladder along one of the legs also provides access to the top and to the various levels at which insulators are strung. The ladder is enclosed from the 50 ft. level to the top with a wire cage and provided with trap doors and seats every 50 ft.

The towers are erected as follows:—

A timber erection tower 10 feet square was built up to a height of about 20 feet on piles which had been driven in the centre of the working platform. A steel derrick with a 60 ft. boom was hoisted to the centre of the erection tower by means of a jin-pole. The material for the towers was unloaded from the cars on the wharf at Three Rivers and transported to the site in barges. The bottom section with its cross-bracing and girts, was erected in place by means of the steel derrick. The erection tower was then raised another 50 feet, the derrick hoisted to the top as before and the next section of tower erected and so on to the top.

The erection tower was guyed to the legs of the steel tower as it went up. Rivetting gangs followed close on the heels of the erection gang. As soon as the north shore tower was completed work was begun on the south shore tower. The erection of the north shore tower was begun early in September. The winter set in rather early and working conditions were so adverse that, in spite of the fact that the Shawinigan Co. paid the men a substantial bonus, the south shore tower was not fully completed until after the first cable was erected in March.



**First Section of a Caisson, Before Lowering**
Showing Long Reinforcing Rods for Tying Into Next Lift, and Shorter Bars for Connection to Screws for Lowering the Lift.

Each of the towers is grounded by means of a copper plate 4 ft. square which is buried in the river bed and connected to one of the tower legs.

The design and erection of the towers was carried out under our direction by the Canadian Bridge Co.

The three sheaves at the top of each tower are 8 ft. in diameter and weigh about 2½ tons each. The rims and hubs are of cast steel and were machined before the sheaves were assembled. The rims are in four sections bolted together. The spokes are built up of steel plates and angles rivetted together and bolted to the rims and hubs. The shafts are 5" in diameter of medium open-hearth steel. After the sheaves were assembled the hubs were shrunk on the shafts. The sheave bearings are of cast steel bushed with bronze. Lubrication is provided by means of wicks.

**Insulators**

The insulators which we proposed using in the steel lines consist of a large ring-girder and two spiders. The ring-girder is 8 ft. in diameter and made up of two 9" channels 12" apart with ⅜" cover plates. The spiders are built up of plates and angles with a heavy steel casting in the centre. The upper spider is connected to the ring-girder by means of three 2½" bolts 10 ft. long, one at the end of each spider arm. The centre spider is supported on the ring-girder by six porcelain insulators of eight skirts each, two insulators at the end of each spider arm. The clear distance between the spiders is about 36".

The porcelain insulators used are special compression insulators having a tested breaking strength of 60 tons each,—this is about 4 times the estimated maximum load. Electrical tests showed a dry flash-over of 302,000 volts and a wet flash-over of 262,000 volts. The completed insulator has a net weight of about six tons.

**Cables**

The cables are 1⅜" in diameter, of galvanized plough steel made up of six strands of 19 wires each and a stranded steel core of 30 wires, with a small hemp centre. Tests made at McGill University showed that the individual wires had an average yield point of 221,000 lbs. per square inch, and an average breaking strength of 258,000 lbs. per square inch.

The completed cable was tested, the yield point being found to be 158,500 lbs. and the ultimate strength 186,400

**Concrete Bucket, With Cover**

lbs. or 193,000 lbs. per square inch and 227,000 lbs. per square inch respectively.

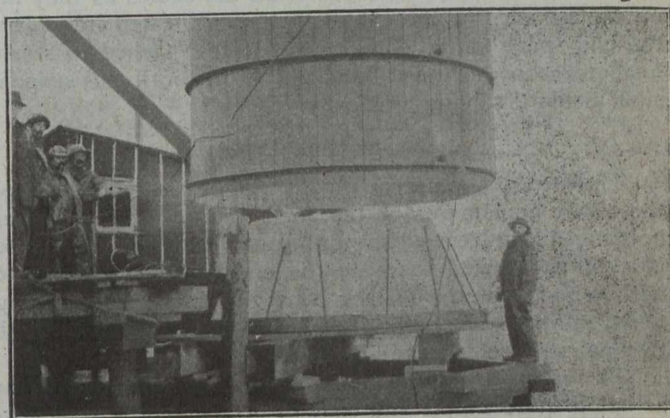
The test of the completed cable indicated a modulus of elasticity of 7,250,000 lbs., or 8,800,000 lbs. per square inch. We are in doubt as to the correctness of our test in this regard on account of the fact that the usually accepted value for the modulus of elasticity for stranded steel cables is about 21,000,000 lbs. per square inch. However, the behavior of the cable during erection bore out the results of the test.

No electrical tests were made on these cables, but their sectional area would indicate a conductivity equivalent to that of a 100,000 c.m. copper cable.

The bridge sockets used for connecting the cables were machined out of solid blocks of steel. Those attached to the centre span cables are 15 inches long, taper in width from 5½" at the front to 8" at the back and are 9" thick, providing a grip of this length on the cable. The sockets attached to the anchor span cables are slightly smaller.

A conical hole, tapering in diameter from 1 9/16" to 5" is bored through the centre of the block from front to back and on each side of this hole a 2⅝" diameter hole is provided for the connecting bolts.

The cable was passed through the tapered hole in the centre of the bridge socket and broomed out on the end



**Setting First Caisson Form**

for a length of 15 to 18 inches. The wires were then cleaned with gasoline and held in place by means of a templet made of ⅛" steel plate which fitted over the back of the bridge socket. The bridge socket was then suspended with the back up and heated by gasoline torches for about half an hour when spelter was poured into the conical hole through a 1 inch diameter hole in the centre of the templet. After the bridge socket had cooled, the ends of the wires projecting from the templet were cut and the templet was removed.

Before adopting this form of connection, tests were run under our direction at McGill University to determine the depth of socket required. We found that if the spelter was heated to just the right temperature, i.e., just hot enough to ignite a sliver of wood thrust into it, that the full breaking strength of the wire was in the majority of cases developed in a length to six inches.

Shortly after the bridge sockets were poured, we found it necessary to shorten two of the cables and the speltered end was cut off. We had one of these cones of spelter cut in the machine shop and found that the spelter adhered so firmly to the wires that the section could be machined without lifting the wires out.

**Erecting Cables**

Owing to constant succession of delays that occurred in the construction of the foundations and in the erection of the towers, we had to abandon our original plan of stringing the cables in the fall of 1917 before the ice formed in the river, and so decided to do this part of the work after the ice had become thick enough to support the weight of the heavy reels of cable.

Throughout the heavy snows of January and February, we managed by constant rolling and scraping to keep a road open between the two towers. Early in March the centre span cables were laid out along this road. The anchor cables were then laid out, measured, and cut, and their bridge sockets attached.

The three lines were erected one at a time, the middle line first and then the downstream and the upstream lines in succession. The ends of the anchor cables were hoisted over the towers, the south shore cables made fast to the centre span cable, drawn over the tower until the bridge sockets touched the main sheave, tied to the top of the tower and attached to the anchor pier. The north shore cables were next attached to the centre cable, the suspension insulators and copper line fastened to this and the cable hoisted into place.

The proper location of the suspension insulators was determined beforehand for both the steel and copper lines, due allowance being made for stretch in the cables, and these distances were chained off on the cables as they lay on the ice, After the cables had been hoisted into place the end spans of the copper lines were attached to strain insulators on the towers and drawn up until the suspension insulators hung vertically.

### A Construction Problem

The hoisting was done by a steam hoist braced against the centre anchor pier on the north shore. Two 5/8″ steel hoisting lines, reeved through two pairs of 3-sheaved blocks, were used to draw the end of the cable up to within 40 ft. of the anchor pier, the final 40 feet being taken up by means of two 3/4″ steel cables reeved through two pairs of six sheave blocks.

The method of pulling up the cables appealed to us as being quite simple and easy of accomplishment. However, we found that it was a nerve-racking process the first time we tried it. The cable seemed to come up easily to within a short distance of the anchor pier, when the hoisting engine began to show signs of distress. When the end of the cable was within 3 ft. of the anchor pier the hoist suddenly coughed and quit.

Those of us who had calculated the length of the cable and measured it as it lay stretched out on the ice, wondered if we had slipped up on our figures or our measurements; for, at the calculated tension in the cable, allowing generously for friction in the blocks, the hoist should have been capable of pulling the cable right home through twelve parts of line without any trouble. The riggers didn't believe that we were able to calculate the tension and were wondering if their pins and lashings would hold.

A wait of ten minutes for the hoist to recuperate and for an inspection of the tackle and connections, a little sand in the clutch, a few more revolutions of the drum, and the cable came in another foot.

Another ten minutes for inspection and recuperation, another puff or two, a few more inches gained and finally, by dint of nursing the hoist and pulling for it on the part of all concerned, the cable was hooked up and made fast.

A slight change in the tackle made it possible to pull up the other two lines without any mental effort whatever.

The copper conductor in each line is supported by 17 suspension insulators spaced about 250 feet apart, the end insulators being about 400 feet from the towers. The copper lines drop from the end insulators to strain insulators on the towers at the 150 ft. level, pass through the towers to the back, where they are connected to another set of strain insulators.

On the north shore the lines pass directly from the main tower to a transmission line tower on the shore a distance of about 600 feet. On the south shore a light structural steel truss, 50 ft. long, hung from two sets of the anchor cables, provides an intermediate point of suspension, forming two spans of 500 ft. each. Access to the insulators attached to the truss is provided by a foot bridge running up from the anchor pier and suspended from the anchor cables.

After the cables were erected, we noticed an almost constant vibration in them, varying in intensity and somewhat similar to that in a violin string, with definite nodes 12 to 15 ft. apart as nearly as could be judged. About a month after the line was put in service, this vibration shook loose the bolts connecting two of the suspension insulators to the cable and they dropped and hung suspended on the copper line.

This condition brought us face to face with a problem which we had considered with misgiving and which had led us to discard the idea of suspended conductors when the crossing first came up for consideration. However, the difficulty was solved for us by two of our riggers who volunteered to go out on the steel cable, for a consideration, and to fish up and re-attach the insulators.

A trolley was devised and constructed of two sheaves on which a small platform was hung and they had little difficulty in getting out to the point from which the insulators had fallen, about 1,000 feet from the tower. By means of a small tackle line, they hauled the insulators back into place and started back towards the tower.

Unfortunately the sheaves on the trolley jammed and they could not pull themselves up. At this point one of the riggers lost his nerve and the episode might have had a tragic ending but for the fact that the other kept his head. He lowered his mate to a boat waiting in the river 250 ft. below, then lowered the trolley, and finally passed the line over the steel cable, slid down to the boat and pulled the line down after him.

A short time later an insulator on one of the other lines broke loose and it was similarly reconnected. This time, however, we profited by our former experience and provided a tail line by means of which the riggers were pulled back to the tower. Since then we have experienced no trouble from this source.

### Clearance

So far as we have been able to judge, the suspension insulators had not been damaged in any way by the vibration in the line, but there is a possibility that it may cause the cement in which the caps and bolts are set to disintegrate. We are at present carrying on investigations with the object of devising some means of absorbing this vibration between the steel cables and the insulators.

The cables as originally strung allowed the following clearances between the copper conductors and the average water level during the season of navigation:—

Downstream, 172.5 ft.; centre, 178.8 ft.; upstream, 180.6 ft.

The temperature at the time of erection was about 20° F. As there is a change in sag of approximately 1 foot for each 10° change in temperature, the above would correspond roughly to clearances at 110° F. of 163.5, 169.8, and 171.6 feet respectively.

At the time these cables were erected, we naturally expected the sag to increase as the cables stretched under the load until the strands were drawn tightly together. There was no data available with regard to the amount of stretch to be expected, so that it was impossible to allow for this in sagging the cables. The hoist, therefore, was left in position so that we could pull up the cables when the sag became too great.

*(Continued on page 479)*

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## Principal Contents of this Issue



## CANADA'S 4,800-FT. CLEAR SPAN

BIGGEST, longest, highest and other superlatives are but fleeting terms in engineering. This truism was again emphasized last week during the discussion of Mr. Svenningson's interesting paper on the long-span, high-voltage transmission line across the St. Lawrence River. This 110,000 volt, overhead transmission line has a clear span of 4,800 feet between towers, and is 400 feet longer than what was previously the longest clear span in the world and which was erected not long ago across the bay at San Francisco. Mr. Svenningson stated that the Aluminum Company of America is now erecting a span over 5,000 feet in length, from one mountain top to another, in one of the southern states. So Canada will possess for only a short time the honor of having the longest clear span of any transmission line in the world.

Nevertheless, there are features in connection with the St. Lawrence span which will likely cause it to remain for some years at least as the most remarkable, if not the longest span of its kind. That the undertaking was quite experimental, and that there are even yet some interesting problems to be solved in connection with it, is shown by Mr. Svenningson's remarks.

The problems faced by the engineers of the Shawinigan Water & Power Co. were complicated to the greatest extent by the fact that the St. Lawrence is a navigable river. It was considered impossible to counterweight the cables and so secure uniform tension instead of uniform sag, because, despite tests on samples, it was impossible to predict exactly what the amount of the sag would be; and heavy ice may be expected occasionally, in the vicinity of Three Rivers, P.Q., before navigation ceases. Just last week there was a heavy sleet along the St. Lawrence and at the same time a 15-ft. tidal wave. A boat passing beneath the span under those conditions would have been in serious danger if an unknown sag had reduced the clearance much below the minimum stipulated by the Dominion Government.

## WAR DEVELOPED ORGANIZING ABILITY

IN addressing the electrical engineers last week at Toronto, Sir Robert Falconer, president of the University of Toronto, drew attention to another phase of the after-the-war problems. "Thousands of our officers and men," said Sir Robert, "will return to Canada in better health and vigor than when they went away; but what is vastly more important, they will return with confidence in the future and the capacity for organization and discipline."

We have found as a result of the war that we are fully the peers of the Huns for all of their boasted powers of scientific organization. The whole war has been a triumph of organization, largely of engineering organization,—whether in connection with railway work or other transportation problems, forestry, artillery work, submarine detection and other naval problems, water supply, etc.

"The two great departments of the war," said Sir Robert, "have been the engineering and medical." Without engineering the war could not have been fought at all. Without the triumphs of surgery and medicine that have developed during the war, the toll of human life would have been many times greater.

Canada has been awakened to a new moral sense. We know now what we can do in an emergency. We know that powers of organization awakened throughout the nation will not again become dormant, but will ensure the quick development of every natural resource in the country. These facts, claimed Sir Robert, should give everyone the utmost confidence in the future of this great country.

## TRANSMISSION LINE HAS 4,800-FT. SPAN

*(Continued from page 478)*

In May of this year we found that the sag in the cables had increased by from 24 to 27½ ft. and that in order to obtain the necessary clearance over the channel we would have to take up 24 feet in the sag of the downstream cable and 13 feet and 14 feet in that of the centre and upstream cables respectively.

The amount by which a cable will stretch in taking up a given amount in the sag varies inversely as the modulus of elasticity of the cable. Owing to the low modulus which we worked out for the cable from the results of the tests made at McGill University, we were in doubt as to the amount of take-up required. We found that in order to take up 24 feet in the sag we would need to pull the cable in between 7.2 and 10.4 feet, depending on the value of this modulus.

This cable was taken in about 8 feet with a consequent reduction in the sag of about 25 feet. This corresponds to the result that would be obtained if the modulus of elasticity of the cable were 17,000,000 lbs. In other words, it would appear that from the time of the original sagging of the cable to the time the cable was re-sagged, the modulus of elasticity had increased from 7,250,000 lbs. to 17,000,000 lbs. This change in modulus is no doubt due to the gradual stretching of the cable, causing the wires and strands to draw more closely together under the constantly applied tension of the span.

### Ice Protection

Ice conditions in the St. Lawrence River at this point are at times very troublesome and we considered it advisable to construct some kind of guard piers outside the towers to obviate the possibility of damage from this source. During the winter we deposited about 3,000 tons of field stone on the river bed on each side about 75 feet from the upstream and river faces of the towers, carrying the rock up to an elevation about 3 feet above the level of the ice. The ice usually goes out about this level, but last year conditions were exceptional, and before the ice moved it had risen above the tops of our ice-breakers and passed clear over them, piling up around the tower foundations to a height of 25 or 30 feet. Fortunately no damage was done. We are at present completing the guard piers by means of reinforced concrete cribs filled with rock and carried to about the level of the maximum recorded high water.

### Sag Calculations

In our calculations for sags, tension, length of cable, etc., under various conditions, we used the parabolic formulae in preference to the hyperbolic formulae for the catenary, on account of the greater simplicity of the former. Comparison was made, however, between the two sets of formulae and we found, as we had expected, that at working tensions the difference was negligible. The formulae for the parabola gave us about 6 inches more sag and about 1 foot less length of cable than the catenary formulae for the same conditions of tension and temperature.

We assumed the maximum load on the cable to be ¾ of an inch of ice all round and 10 lbs. of wind per square foot of projected area, for both the steel and copper lines, at a temperature of 0°F. Under these conditions the calculated tension in the cable is about 106,000 lbs. with a sag of 228 ft. The normal tension at summer temperatures is about 61,000 lbs. with a sag of 185 feet.

### Conclusion

There are still a few details to be worked out in connection with the crossing before we can consider the undertaking completed. The questions of providing two circuits on the crossing and of devising some means of absorbing the vibration are still under consideration. The problem of inspection and renewal of the suspension insulators has been only partially solved.

The crossing has been in uninterrupted service now for about nine months. It has not yet weathered a winter with its low temperatures, gales, and sleet storms, so that we still have something to learn about its action under these conditions. The allowable stresses, however, have been kept within reasonable limits and we hardly expect serious trouble from this source. The success that has attended the operation of the crossing since it was put into service gives us reason to expect a satisfactry solution to the remaining problems.

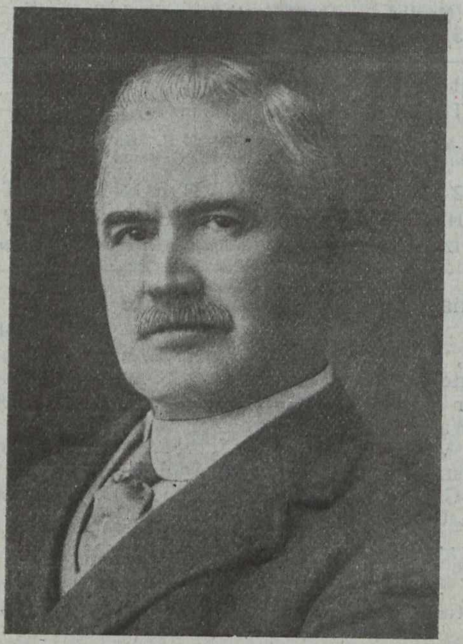
## FLOW OF WATER IN WASH WATER TROUGHS

*(Continued from page 468)*

the area of these iron braces is uncertain. The total area of the trough for this run was 1.88 sq. ft. at the upstream end and 2.26 sq. ft. at the lower end. This seems to indicate that where a trough is designed so that it will carry the expected flows within its section, it will have an overload capacity of possibly 50 per cent. when flooded.

## PERSONALS

RICHARD COTSAM WRIGHT, who was recently appointed chief architect of the Public Works Department of Canada, was educated at London, Ont., in the public and collegiate schools and afterwards by private tutors, thus qualifying as an architect and construction engineer. He also was engaged for a time in surveying with a Toronto firm of provincial land surveyors. Mr. Wright was then successively employed by Richard M. Hunt, Bruce Price and Charles C. Haight, all well-known architects of New York City, and subsequently became associated with Clarence Luce in practice in that city. In 1908 Mr. Wright was employed by the Public Works Department as assistant chief engineer. In 1915 he left for Toronto to supervise, from an architectural standpoint, the construction of the new Union Station, but returned to Ottawa last April as acting chief architect of the department. His appointment as chief architect has now been confirmed.



MAJOR GEORGE A. JOHNSON, formerly a consulting engineer in New York City, and until recently officer-in-charge of the Water and Sewer Section of the Maintenance and Repair Division of the Construction Division of the Army, has been promoted to the rank of lieutenant-colonel, and is now ranking officer under Col. C. D. Hartman, officer-in-charge of the Maintenance and Repair Division.

## OBITUARIES

W. MUIR EDWARDS, of Edmonton, professor of civil and municipal engineering in the University of Alberta, died in the Pembina temporary isolation hospital, Edmonton, on Nov. 14th, from influenza. Professor Edwards had been taking a prominent part in the relief work, when he contracted the disease. He was a graduate of McGill University and was lecturer there in the faculty of applied science before going to Edmonton.

S. B. BENNETT, who was at one time engineer for the municipality of South Vancouver, B.C., died this month in Virginia. Mr. Bennett started an interesting discussion a couple of months ago on the security of tenure in municipal positions in Canada, by writing letters to certain English publications, criticising the conditions under which Canadian municipal engineers hold their positions. Mr. Bennett was born in England and came to Canada about 1906. When he left South Vancouver, he went to Colorado. It is thought that he was visiting in Virginia when he became fatally ill, although full details have not yet been received by his friends in Vancouver.